COMPARATIVE STUDY OF STRUCTURAL BEHAVIOUR OF HIGH-RISE BUILDINGS DUE TO UPGRADATION OF IS 1893 (PART-1) AND RELEASING IS 16700: 2017

THESIS SUBMITTED FOR PARTIAL FULFILMENT OF

REQUIREMENTS FOR THE DEGREE OF

MASTER OF TECHNOLOGY

IN

CONSTRUCTION ENGINEERING WITH SPECIALIZATION IN

STRUCTURAL REPAIR AND RETROFIT ENGINEERING

SUBMITTED BY

SUBHAJIT BARIK

EXAMINATION ROLL NO: **M6CNE19009**

REGISTRATION NO: **137343 OF 2016-2017**

UNDER THE ESTEEMED GUIDANCE OF

PROF. DR. DEBASISH BANDYOPADHYAY

DEPARTMENT OF CONSTRUCTION ENGINEERING

FACULTY OF ENGINEERING AND TECHNOLOGY

JADAVPUR UNIVERSITY

KOLKATA, INDIA

MAY, 2018

FACULTY OF ENGINEERING AND TECHNOLOGY
DEPARTMENT OF CONSTRUCTION ENGINEERING
JADAVPUR UNIVERSITY
KOLKATA, INDIA

CERTIFICATE OF RECOMMENDATION

This is to certify that the thesis entitled **“Comparative Study of Structural Behaviour of High-Rise Buildings Due to Upgradation of IS 1893 (Part-1) and Releasing IS 16700: 2017”** has been prepared by **SUBHAJIT BARIK (**Examination Roll No. - **M6CNE19009**, Registration No. **- 137343 of 2016 - 2017**) for partial fulfilment of the requirement for the award of Master Degree in Construction Engineering (CE) specialization in Structural Repair and Retrofit Engineering, is a record of research work carried out under my Guidance and Supervision. I hereby approve this thesis for submission and presentation.

DR. DEBASISH BANDYOPADHYAY
Thesis Supervisor and HOD
Construction Engineering Department
Jadavpur University
Kolkata , India

DEAN
Faculty of Engineering and Technology
Jadavpur University
Kolkata, India

FACULTY OF ENGINEERING AND TECHNOLOGY
DEPARTMENT OF CONSTRUCTION ENGINEERING
JADAVPUR UNIVERSITY
KOLKATA, INDIA

CERTIFICATE OF APPROVAL*

This foregoing thesis is hereby approved as a credible study of Master of Technology in **CONSTRUCTION ENGINEERING** (Specialization in Structural Repair and Retrofit Engineering) and presented in a manner satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is understood that by this approval the undersigned do not endorse or approval the undersigned do not endorse or approve any statement made, opinion expressed or conclusion drawn therein but approve this thesis only for the purpose for which it is submitted.

Final Examination for
Evaluation of the Thesis___________________________________

____

Signature of Internal Examiner

____

Signature of External Examiner

* Only in case the Thesis is approved.

FACULTY OF ENGINEERING AND TECHNOLOGY
DEPARTMENT OF CONSTRUCTION ENGINEERING
JADAVPUR UNIVERSITY
KOLKATA, INDIA

DECLARATION OF ORIGINALITY AND COMPLIANCE OF ACADEMIC ETHICS

I hereby declare that this thesis contains a literature survey and original research work by an undersigned candidate, as part of his **Master of Technology** in **CONSTRUCTION ENGINEERING** (Specialization in Structural Repair and Retrofit Engineering).

All information in this document has been obtained and presented in accordance with academic rules and ethical conduct.

I also declare that, as required by thesis rules and conduct, I have fully cited and referenced all materials and results that are not original to this work.

NAME: SUBHAJIT BARIK

EXAMINATION ROLL NO: M6CNE19009

REGISTRATION NO: 137343 OF 2016 - 2017

THESIS TITLE:

COMPARATIVE STUDY OF STRUCTURAL BEHAVIOUR OF HIGH-RISE BUILDINGS DUE TO UPGRADATION OF IS 1893 (PART-1) AND RELEASING IS 16700: 2017

Signature with Date

ACKNOWLEDGEMENT

The happiness and ecstasy on the successful completion of any task would be incomplete without mentioning the people, whose constant support and guidance crowned my efforts with success.

I am very grateful to Department of Construction Engineering, Jadavpur University, for giving me an opportunity to work on this project, which is an integral part of the curriculum in ME programme at Jadavpur University, Kolkata.

I would also take this opportunity to express my heartfelt gratitude to my project guide Prof. Dr. Debasish Bandyopadhyay, who provided me all the guidance that I needed and valuable inputs at each and every stage of the project. I would also like to extend my gratitude to my parents for providing me constants supports and the facilities to carry out that work in a successful manner.

I am thankful to all faculty members of the Construction Engineering Department for their invaluable advice and support throughout my project work. Last but not least I would like to thank my college fellows, office colleagues who constantly provided me encouragement and support during the whole term of my stay at Jadavpur University.

ABSTRACT

Now days the large-scale urbanization in India and scarcity of land in metropolitan cities, high-rise building structures e.g. skyscrapers are extensively adopted. Many major cities of India are under the high and moderately high seismic zones, those areas are also prone to high wind effect which poses big trouble for these high rise multi-storeyed buildings. The analysis of such a thorny structure against the transient loads is significantly essential to evaluate the structural behaviour. The sixth revision of IS 1893 (Part-1) in 2016 includes considerable development in earthquake resistant design of buildings compared to the previous revision at 2002. This present work aims at studying revisions in various clauses of new IS 1893 (Part-1): 2016 with respect to old IS 1893 (Part-1): 2002 and their effect on structures, particularly the effect of Separate response spectra for Equivalent static method and Response spectrum method. A simple response spectrum for both Equivalent Static Method and Response Spectrum method for 4.0s periods was provided in old IS 1893 (Part-1): 2002. Expressions are given for calculating design acceleration coefficient (S_a/g), for Rocky/hard soils, medium soils and soft soils. The new IS 1893 (Part-1): 2016 has given response spectra for Equivalent Static Method and Response Spectrum method separately for 6.0s periods. Expressions are given for calculating design acceleration coefficient (S_a/g), for Equivalent Static Method and Response Spectrum method separately for Rocky/ hard soils, medium soils and soft soils. It seems that change will occur in the S_a/g values. Definition of soft storey and weak storey, change in definition of mass, torsion and vertical irregularities has been modified. In the new code importance factor of 1.2 has been specified in new code for residential buildings more than 200 persons instead of 1.0. Thus increase in the design horizontal seismic coefficient A_h will happen for these buildings. New expression for T_a for building with RC structural walls, requirements for rigid and flexible diaphragm has been modified. Modelling of unreinforced masonry infill walls as equivalent diagonal struts, etc. with critical comments have been addressed.

Urbanization to accommodating the huge population migrating thereto is being addressed through vertical enlargements in the form of high-rise buildings. IS 16700: 2017 was introduced by the Bureau of Indian Standards, after the draft finalized by the Special Structures Sectional Committee, which had been approved by the Civil Engineering Division Council. To address the vertical growth of cities, this standard provides dictatorial requirements for structural design of reinforced concrete high-rise

buildings. In the design of tall building other important parameters that need due attention are; wind load analysis by using wind tunnel test, P-Δ effect, secondary effect like creep, shrinkage and temperature. In analysis for seismic loads some changes in comparison to IS 1893 (Part-1): 2016 is also reported. Modelling of tall building and changes in design considerations are studied in detail. Criteria for selection of foundation are specified in the code. The backstay effect, connection with key elements and importance of non-structural elements also specified and design guidelines based on the sensitivity of the elements are provided. In this proposed study the modifications of IS 1893: 2016 with respect to previous revision in 2002 and major clauses of IS 16700: 2017, which effects the design of reinforced concrete tall buildings is discussed with various numerical examples. Firstly, 3D analytical model of G+12 storied buildings have been generated and analyzed as per previous as well as latest revision of IS 1893 (Part-1) and IS 875 (Part-3) adopting numerical building models in FEM based ETABS Platform and compared their structural responses. Secondly, three different heights (G+24, G+30 and G+36 storied) with same area, loading and structural configuration multi-storeyed building models are generated and analyzed using ETABS with due consideration of design stipulation of IS 875 (Part-3) and IS1893 (Part-1) with their previous and latest revision. The behaviours of G+24, G+30 and G+36 storied buildings have been studied considering earthquake and wind effects. The dynamic effects of those models due to seismic forces are studied adopting response spectrum method. Subsequently the wind induced dynamic forces are duly considered adopting gust factor method both along and across wind direction. The structural parameters like Time period, Base shear, Overturning moment, Story displacement, Story drift and Storey stiffness are compared between the different building models. It may be concluded that there are significant impact due to revision and introduction of the IS codes for high-rise buildings. The governance either of earthquake or wind have been observed based on the present study. Both the effects on limit state of collapse as well as serviceability have greatly induced due to introduction of new codes.

LIST OF FIGURES

Figure No.	Figure Description	Page

LIST OF TABLES

Table No.	Table Description	Page

SYMBOLS AND NOTATIONS

A= Area of the building

A_{wi}= Cross-sectional area at 1st Floor level

B= Breadth of the building in considered direction

G= Acceleration due to gravity

G= Gust Factor

h= Floor to Floor Height

H= Total Building Height

I= Importance Factor

I_{gross}= Gross Moment of Inertia

K_1= Risk Coefficient

K_2= Terrain Factor

K_3= Topography Factor

K_4= Cyclonic Factor/Importance Factor

L= Length of the building in considered direction

L_{wi}= Total Length of Structural Wall at 1st Floor level in the direction of Lateral Force

MRF= Moment resisting Frame

NSEs= Non-structural Elements

N_w= Number of Structural Wall in the direction of Earthquake Force

R= Response Reduction Factor

RSM= Response Spectrum Method

S_a/g = Design/ Response acceleration coefficient

SODF= Single Degree of Freedom System

T_a = Fundamental Natural Time period

URM= Unreinforced Masonry

V_b= Basic Wind Speed

W= Seismic Weight

Z= Seismic Zone Factor

TABLE OF CONTENTS

4. CHAPTER-IV: DESIGN CONSIDERATIONS AND MODELLING TECHNIQUES

5. CHAPTER-V: RESULTS AND DISCUSSIONS

6. CHAPTER-VI: CONCLUSION

CHAPTER-I

1. INTRODUCTION

1.1. GENERAL

India is susceptible to strong earthquake shaking, and hence earthquake resistant design is necessary. The Engineers do not try to make earthquake proof structures that will not get blemished even during the infrequent but robust earthquake. Such structures will be too inappropriate for commercial, functional and residential uses, and also too extortionate. Many types of research and experimentations have been done in order to mitigate excitements and embellish the performance of high-rise building against transient loads. Tall buildings are big projects demanding incredible logistics and management and require an enormous financial investment. Diligent coordination of the structural framing system, structural arrangement and the shape of a building and which minimize the lateral displacement, may offer considerable savings. Now days, the challenge of designing a cost-effective tall building has substantial changed. The conventional approach to tall building design in the past was to limit the forms of the building to a rectangular and regular shape mostly, but today, much more complicated building geometries could be utilized by using a special type of structural system.
The sixth revision of IS 1893 (Part 1): 2016 was revised by the Bureau of Indian Standards, after the draft nailed down by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

Structures designed as per this standard are expected to prolong damage during massive earthquake ground shaking. The provisions of this standard are intended for earthquake resistant design of only standard normal building structures (without energy dissipation devices or in-built systems).

To preoccupy loss of life and property, base isolation or other advanced techniques may be considered. Presently, the Indian Standard is under research for the design of such special type buildings; until the standard becomes available to us, specialist literature should be enquired for design, detail, installation and periodic maintenance of such buildings.

IS 1893: 1962 ‘Recommendations for earthquake resistant design of structures’ was first promulgated in 1962, and revised in 1966, 1970, 1975 and 1984. Further, in 2002, the

Committee distinct to present the provisions for different types of structures in separate parts, to keep conversant with rapid developments and extensive research carried out in the earthquake-resistant design of various structures.

In this latest revision, the following changes have been modified:

- Design response spectra are defined for the natural time period up to 6.0 s;
- Same design response spectra are nominative for all buildings, irrespective of the material of construction;
- Bases of various load combinations to be considered have been made consistent for seismic effects, with those specified in the other codes;
- All temporary structures are brought under the scope of this standard;
- Importance factor provisions have been chained to introduce intermediate importance category of buildings, to acknowledge the density of occupancy of buildings;
- A provision is familiarized to ensure that all buildings are designed for at least a minimum lateral force;
- Buildings with flat slabs are brought under the scope of this standard;
- Additional clarity is brought in on how to manage different types of the irregularity of the structural system;
- Effect of masonry infill walls has been included in the analysis and design of moment frame buildings;
- The method is introduced for reaching the approximate natural period of buildings with basements, step back buildings and buildings on hill slopes;
- Provisions on torsion have been easy; and
- For liquefaction potential analysis simplified method is introduced.

On the other hands, the large-scale civilization to accommodating the huge population migrating thereto is being addressed through vertical enlargements in the form of high-rise buildings. For that IS 16700: 2017 was adopted by the Bureau of Indian Standards after the draft finalized by the Special Structures Sectional Committee had been approved by the Civil Engineering Division Council. This standard provides dictatorial requirements for the structural design of reinforced concrete high-rise buildings, whose design is governed not just by structural safety aspects, but also by serviceability aspects, especially under the conditions of wind effects. This standard has therefore been formulated to cover these aspects relating to reinforced concrete buildings of height greater than 50 m, but less than or equal to 250 m.

This standard provides functional requirements for the design of reinforced concrete tall buildings. The following prominent aspects, which are based on the practical approach, are considered in this standard:

- Structural systems that can be adopted in a tall building with respect to height and plan aspect ratio;
- General requirement of Structures including; (i) limitations of height of different structural systems, (ii) elevation and plan aspect ratios, (iii) lateral drift of structure, (iv) stiffness and strength of storey, (v) buildings density, (vi) vibration modes, (vii) structural floor systems, (viii) material properties, and (ix) progressive collapse mechanism;
- Seismic and Wind effects: (i) load combinations, and (ii) acceptable serviceability criteria for lateral accelerations;
- Methods of structural analysis to be adopted, and section properties (in cracked and un-cracked states) of reinforced concrete member to be considered in analysis for both serviceability and collapse aspects;
- Structural design features for various applicable structural systems;
- Design considerations for the design of foundation systems;
- Systems needed for health monitoring of a structure.

1.2. OBJECTIVE:

The objective of this presentation is to study and compare the effect of revised stipulations of IS 1893 (Part-1): 2016 and releasing new code IS 16700: 2017 on the structural behaviour of tall building due to transient forces.

1.3. SCOPE OF WORK:

To fulfil the above objective the scope of work limited with the followings

1. To study the clauses provided in IS 1893 (Part-1): 2016, and compared them with IS 1893 (Part-1): 2002.
2. To highlights some salient features of IS 16700: 2017 for Structural safety of tall reinforced concrete building.

3. Development of numerical models of mid-rise and high-rise buildings on ETABS platform and subsequent analysis has been performed for study of structural behaviour
   (a) based on Response Spectrum Method as per IS 1893 (Part-1): 2002 for mid-rise and high-rise buildings
   (b) based on Response Spectrum Method as per IS 1893 (Part-1): 2016 for mid-rise building
   (c) based on Response spectrum method as per IS 1893 (Part-1): 2016 and IS 16700: 2017 for high-rise buildings
4. Comparative study on structural behaviour of G+12 storied residential building (mid-rise), situated at Kolkata (seismic zone-III and soil type-II), analyzed using stipulations of IS 1893 (Part-1): 2002 and IS 1893 (Part-1): 2016 (latest revision) considering the followings
   A. Regular Building with Structural Wall + Moment Frame System :
      I. Modelling as per IS 1893 (Part-I): 2002 **[G12-2002]**
      II. Modelling as per IS 1893 (Part-I): 2016 **[G12-2016]**
5. Comparative study on structural behaviour of high-rise building, situated at Kolkata (seismic zone-III and soil type-II), analyzed using stipulations of IS 1893 (Part-1): 2002 and IS 1893 (Part-1): 2016 (latest revision) conjugate with IS 16700: 2017 considering the followings
   A. G+24 storied residential building:
      I. Modelling as per IS 1893 (Part-1): 2002 **[G24-2002]**
      II. Modelling as per IS 1893 (Part-1): 2016 and IS 16700: 2017 (Serviceability aspects) **[G24-2016700]**
   B. G+30 storied residential building:
      I. Modelling as per IS 1893 (Part-1): 2002 **[G30-2002]**
      II. Modelling as per IS 1893 (Part-1): 2016 and IS 16700: 2017 (Serviceability aspects) **[G30-2016700]**
   C. G+36 storied residential building:
      I. Modelling as per IS 1893 (Part-1): 2002 **[G36-2002]**
      II. Modelling as per IS 1893 (Part-1): 2016 and IS 16700: 2017 (Serviceability aspects) **[G36-2016700]**

1.4. ORIENTATION OF THESIS:

In Chapter-II, extensive review of various literatures of the Specified topic has been carried out to understand the present state of art prior to the actual work of the present study.

In Chapter-III, a comprehensive study of various clauses of new IS 1893 (Part-1): 2016 and old IS 1893 (Part-1): 2002 has been made. Many clauses of old IS 1893 (Part-1): 2002 has been revised in new IS 1893 (Part-1): 2016. The revisions in major clauses have been presented in this chapter with critical remarks. This Chapter also deals introduction with new IS 16700: 2017, “criteria for structural safety of tall building”. In this chapter highlight some major clauses of IS 16700: 2017 which affects the structural behaviour of tall buildings.

Chapter-IV contents the design consideration such as material properties, members properties, Basic loads (dead load, superimposed dead load, live load, seismic load wind load and temperature loads), and generation of load combinations, modelling techniques and structural analysis of the selected different types of structure for the intended research work.

Chapter-V contents a comprehensive study on structural performance and behaviour of considered buildings models and compared them with each other.

In Chapter-VI based on detail numerical studies on the structural behaviour of various model the results is concluded.

CHAPTER-II

2. LITERATURE REVIEW

2.1. GENERAL

Extensive review of various literatures of the Specified topic has been carried out to understand the present state of art prior to the actual work of the present study.

S.K. Ahirwar, S.K. Jain and M. M. Pande (2008), This paper is prepared for the seismic load easement for multi-storied buildings as per IS 1893 (Part-1): 1984 and IS 1893 (Part-1): 2002 design considerations. Here, four multi-storied RC framed buildings are considered for analysis, which are ranging from three storeys to nine storeys. Five individual analysis sequences for each building are prepared and the seismic response viz. storey shear and base shear are computed from the analysis results as per the two versions of seismic code. The seismic forces, computed by IS 1893 (Part-1): 2002 are found to be remarkably more. The difference between results of two codes varies with structure properties. It is concluded that such comparison should be done for individual structure to predict seismic vulnerability of RC framed buildings that were designed using earlier code and due to revisions in the codal provisions may have rendered unsafe.

Prashanth. P, Anshuman. S and Pandey. R. K (2012), Now days, leading design softwares are STAAD Pro and ETABS in the market. These softwares are used for project design purposes in many companies. So, this research mainly deals with the comparative analysis of the results obtained when the design of a regular and a plan irregular [as per IS 1893 (Part-1): 2002] multi-storied building structure is designed using STAAD Pro and ETABS platform separately. These results will also be compared with manual calculations of a sample beam and column of the same structure designed as per IS 456: 2000. From the design results of beams, it may conclude that ETABS gave lesser area of required steel as compared to STAAD PRO. It is found out from previous studies on comparison of STAAD results with manual calculations that STAAD Pro gives conservative design results which is again proved in this study by comparing the results of STAAD Pro, ETABS and Manual calculations. From the design results of column, the required steel for the column forces in this particular problem is less than the minimum steel limit of column (i.e., 0.8%). Hence, the amount of steel calculated by both the software is equal. So comparison of results for this case is not required.

Bagheri et al., (2012), A 20 storied irregular building model is created and analysis is done by using software's ETABS and SAP 2000 for seismic zone-V in India. The effect of the variation of the building height on the structural response of the shear wall building is also considered in this paper. Dynamic analysis is carried out under the earthquakes EL-CENTRO 1949 and CHI-CHI Taiwan 1999. The accuracy of the non-linear dynamic method (Time History analysis) is compared with linear static and dynamic methods (Equivalent Static and Response Spectrum method respectively) in this paper and the followings were the conclusions: (i) Static method gave higher displacement values than dynamic method, (ii) The most ideal method for the seismic analysis of buildings is Time history method, (iii) To obtain accurate results, dynamic analysis should be performed for high rise structures, (iv) There are small difference in displacement values between both the methods for the lower stories whereas the higher stories shows higher displacement values, the displacement values increases along with the height, (v) Equivalent static analysis is not considered as an economical method because the displacement values obtained been higher.

Wakchaure et al., (2012), T-shaped and Oval-shaped building models are created and analysed with 35 and 39 storied respectively. Considering the plan irregularity analytical analysis during seismic events is carried out and studied. The seismic performance of high rise buildings and the effects of structural irregularities in stiffness, strength, mass and combination of these factors are considered. The analysis was done using the software ETABS. Shear walls are located in the core and analyzed.

Rama Raju et al., (2013), A 3B+G+40 storied reinforced concrete frame structure is modelled. Design inputs are given according to the limit state design under wind and earthquake loads as per described in the IS 1893 (part-1): 2002 and IS 875 (Part-3): 1987 respectively. Allowable limits prescribed for base shear, roof displacement; inter storey drifts, accelerations are checked for safety of structure.

S. Mahesh and B.Panduranga Rao (2014), A residential multi-storied building of G+11 storied is studied for earthquake and wind load using ETABS and STAAD Pro V8i. Assuming that all material properties are linear, static and dynamic analysis is performed. Different seismic zones are considered for this analysis and the behaviour of the structure at each zone is checked and for three different types of soils namely Hard, Medium and Soft, behaviour of building is checked. Different response like story drift, displacements base shear are plotted for different seismic zones and different types of soils.

Dr. D. K. Paul (2016), IS 1893 (Part-1): 2016 is revised in 2016, with basic design philosophy same the structures designed as per this Standard is expected to sustain damage under strong earthquake.

The in plane stiffness of the floor and roof slabs can be assumed as rigid. Each rigid floor shall be modelled as SDOFS in the direction earthquake excitation. The beam and column members are modelled as beam and column elements with appropriate sectional properties. The structural walls are modelled as in plane stress elements/ shell elements. The URM infill masonry walls shall be modelled by using equivalent diagonal struts taken to be pin jointed on either end.

Analysis of building for design earthquake loads, following methods are adopted for

1) Equivalent Static Method, and
2) Dynamic Analysis Method

Dynamic analysis can be performed in three ways,

- [1] Response Spectrum Method,
- [2] Modal Time History Method, and
- [3] Time History Method

Gauri G. Kakpureand Ashok R. Mundhada (2016), This paper presents a review of the previous work done on earthquake analysis of multi-storied buildings. It focuses on static and dynamic analysis of buildings. This paper presents an assessment of the comparison of static and dynamic analysis of multi-storied building. Design parameters such as Displacement, Bending moment, Base shear, Storey drift, Torsion, Axial Force were the focus on this study.

K Venu Manikanta and Dr. Dumpa Venkateswarlu, (2016), The main purpose of this study is to carry out a detailed analysis on simulation tools ETABS and STAAD pro, which have been used for analysis and design of rectangular plan with vertical regular and rectangular plan with vertical geometrically irregular multi-storied building. This study is focused on bringing out advantages of using ETABS over current practices of STAAD pro versions to light. As per observation, ETABS is more user friendly, accurate, compatible for analyzing design results and many more advantages are there over STAAD pro. In this study Pros and cons of using these softwares are also mentioned.

Here, the effect of both vertical aspect ratio (H/B ratio i.e. Slenderness Ratio) and horizontal or plan aspect ratio (L/B ratio) are discussed, where H is the total height of the building frame, B is the base width and L is the length of the building frame with different plan configurations on the seismic analysis of multi-storied regular reinforced concrete Buildings.

The test structures are kept regular in elevation and in plan. Here, height and the base dimension of the buildings vary according to the aspect ratios. For different configurations of low, medium and high-rise building models, different aspect ratios are assigned.

Total 16 types of building models are analyzed for different load combinations by linear elastic static analysis (Equivalent static force analysis) with the help of ETABS software. The results on seismic response of buildings obtained from these models have been summarized.

N. Veerababu and B Anil Kumar, (2016), In the present study an endeavour has been made to produce reaction spectra utilizing site particular soil parameters for a few destinations in seismic zone V, i.e. Arunachal Pradesh and Meghalaya and the produced reaction spectra is utilized to break down a few structures utilizing business programming STAAD Pro. The impact of soil properties in the reaction range is discussed here. At long last examinations have been made in the middle of the structure outlined by taking IS 1893 (Part-1): 2002 reaction spectra under thought with the structure planned by considering the created reaction spectra for different sorts of soil for the seismic zone as far as twisting minute, shear powers and fortification.

Pardeshi Sameer (2016), In this study a 3D analytical model of G+15 storied buildings is created for symmetric and asymmetric building models and analyzed using structural analysis tool ETABS software. Building mass and storey stiffness are two basic parameters to assess the dynamic response of a structural system. The effects of various vertical irregularities on the seismic response of a structure are observed. The objective of the project is to carry out Response Spectrum Analysis (RSA) of regular and irregular reinforced concrete building frames and Time History Analysis (THA) of regular reinforced concrete building frames and carry out the ductility based design using IS 13920 corresponding to response spectrum analysis. The results of analysis of irregular structures and regular structure are compared in this paper.

A. A. Kale and S. A. Rasal (2017), In this proposed study four different shapes of same area multi-storied building models are created and compared using ETABS under the guideline of IS 875 (Part-3) and IS 1893 (Part-1): 2002. The behaviours of 15, 30 and 45 storied buildings have been studied. The Dynamic effects are found out by Response Spectrum Method (RSM). All the parameters like Story Displacement, Story Drift, Base Shear, Overturning Moments, Acceleration and Time Period are calculated. All building shapes results are compared. From the results, the convenient section is found out and either seismic or wind effect is critical.

Narayan Malviya, Sumit Pahwa (2017), They were presented concerned with the study of seismic analysis and design of high-rise building. The structural analysis of high rise multi-storied reinforced concrete symmetrical and asymmetrical frame building is done with the help of SAP software. The Response Spectrum Analysis (RSA) of regular reinforced concrete building frames is compared with Response Spectrum Analysis of regular building and is carried out the ductility based design. As per IS 1893 (Part-1): 2002 and IS 1893 (Part-1): 2016. In the Maximum deflection is get low value to compare old code. Shear force value and bending moment get low value to compare old code 1893-2002.

Prakash Channappagoudar, Vineetha Palankar, R. Shanthi Vengadeshwari, Rakesh Hiremath (2018), A building in Pune is taken into consideration for analysis with respect to wind loads for different number of floors. Analysis is done for both codes of IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015 for different parameters affecting the stability of building. Important points of IS 16700: 2017 which takes both the previous codes of Wind and Earthquake and specifies a new code of conduct for design of tall buildings ranging from 50m to 250m are considered. Lateral forces for dynamic analysis for wind code of 1987 and 2015 revisions for 27 storied and 39 storied buildings are compared. The comparison shows that the lateral forces in the along direction has reduced in code IS 875 (Part-3): 2015 when compared to earlier code, the columns under consideration, steel requirement in IS 875 (Part-3): 2015 is higher compared to IS 875 (Part-3): 1987. Time period increases as there is increase in height of the structure for 27 storied and 39 storied buildings. Acceleration has to be limited to certain value such as the human is perceptible to that certain limit at that height of the building. Earlier codes had no clear definition and limit regarding this peak acceleration whereas IS 16700: 2017 code "Safety Criteria for Tall Buildings" limits the value of this peak acceleration to 0.15 m/s^2 for residential buildings. Hence here on the buildings that are to be constructed, should have peak acceleration limited to 0.15 m/s^2. Base Reaction study in the code IS 875 (Part-3): 1987 should be less than that of code IS 875 (Part-3): 2015.

Khuzaim J. Sheikh, Krutarth S. Patel, Bijal Chaudhari (2018), In this paper, the response of the various structural system used in the buildings and its comparison are studied. Structural Wall + Moment Resisting Frame, Structural Wall System, Core Structural Wall system and Outrigger Structural System (Belt Truss System)- these four different structural systems were investigated. 39 storied building having typical height 3.650m was considered. Moreover, Response Spectrum analysis and Static wind analysis were also performed and different structural parameters such as Base Shear, Storey Drift, and Storey Displacement are compared. Seismic analysis with response spectrum method and wind load analysis are used for analysis of G+39 storied reinforced concrete building as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015, and IS 16700: 2017 codes respectively. The building with slenderness ratio of 8.55 for G+39 storied was studied, which is within limits of slenderness give in IS 16700: 2017. The building with aspect ratio 2.46, which is less than 5 limits specified by, IS 16700: 2017. Different structural systems like moment resisting frame + structural wall system, Structural wall system, Core structural wall system and outrigger structural system are studied.

2.2. CRITICAL REMARKS

Comparison between the fourth revision of IS 1893 (Part-1): 1984 and the fifth revision of IS 1893 (Part-1): 2002 should be done for individual structure to predict seismic vulnerability of RC framed buildings that were designed using earlier code and due to revisions in the codal provisions may have rendered unsafe.

ETABS is more user friendly, accurate, compatible for analyzing design results and many more advantages are there over STAAD pro.

Lateral forces in the along direction has reduced in code IS 875 (Part-3): 2015 when compared to earlier code, the columns under consideration, steel requirement in IS 875 (Part-3): 2015 is higher compared to IS 875 (Part-3): 1987. Base Reaction study considering the effects of cross-wind effect in the code IS 875 (Part-3): 2015 should be more than that of code IS 875 (Part-3): 1987.

Slenderness ratio, plan aspect ratio at different seismic zone of different structural system like Moment Resisting Frame + Structural Wall System, Structural Wall System, Core Structural Wall System and Outrigger Structural System (Belt Trussed System) are studied.

CHAPTER-III

3.1. MODIFICATION OF CODAL STIPULATIONS OF IS 1893 (PART-I): 2016 FOR TALL BUILDING WITH RESPECT TO PREVIOUS REVISION IN 2002

A comprehensive study of various clauses of New IS: 1893 (Part 1): 2016 and Old IS 1893 (Part 1): 2002 has been made. Many clauses of old IS 1893: 2002 has been revised in new IS 1893-2016. The revisions in major clauses have been presented below with critical comments on that.

3.1.1. IMPORTANCE FACTOR:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl. 6.4.2 Importance factor 1.5 was for important structures and 1.0 for all other structures, as per Table-6.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl. 7.2.3 For Residential or commercial buildings, with occupancy more than 200 persons importance factor 1.2 has been assigned, in new code, as per Table-8.

REMARKS

As value of importance factor increase, A_h will increase and therefore Base shear V_B will increase. This may lead to increase an amount of lateral loads on the structure and eventually increases the sizes of the lateral load resisting members and reinforcement. Ultimately structure cost may increase, but at the same time the structural strength is also increased towards earthquake forces.

3.1.2. DESIGN ACCELERATION SPECTRUM:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl.6.4 old IS: 1893-2002 has given one response spectra for Equivalent Static Method and Response Spectrum method. The response spectrum is given for 4.0s periods. Expressions are given for calculating design acceleration coefficient (S_a/g), for Rocky/ hard soils, medium soils and soft soils.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.6.4.2.1 New IS: 1893-2016 has given response spectra for Equivalent Static Method and method separately in Fig.3.1A and Fig: 3.1B.The response spectra are given for 6.0s periods. Expressions are given for calculating design acceleration coefficient (S_a/g), for Equivalent Static Method and Response Spectrum method separately for hard, medium and soft soils.

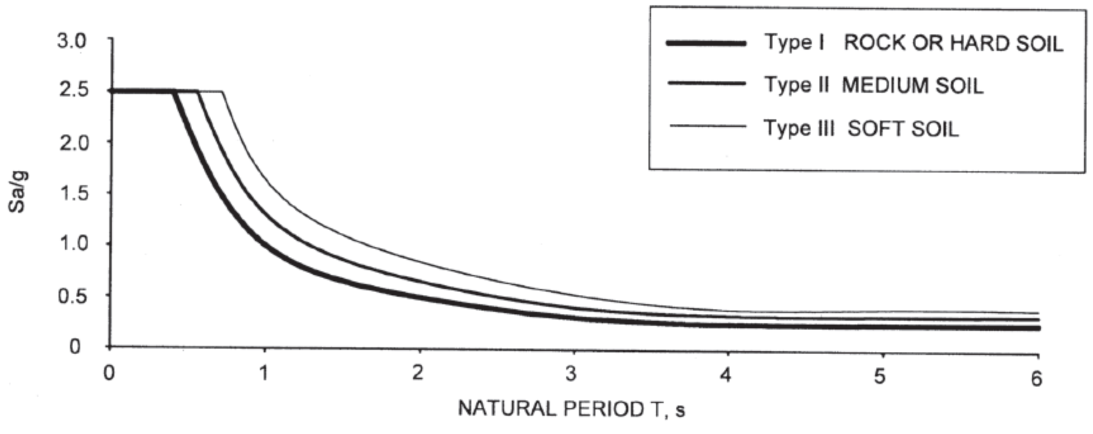


Fig: 3.1A Spectra for Equivalent Static Method *[Courtesy: IS 1893 (Part-1): 2016]*

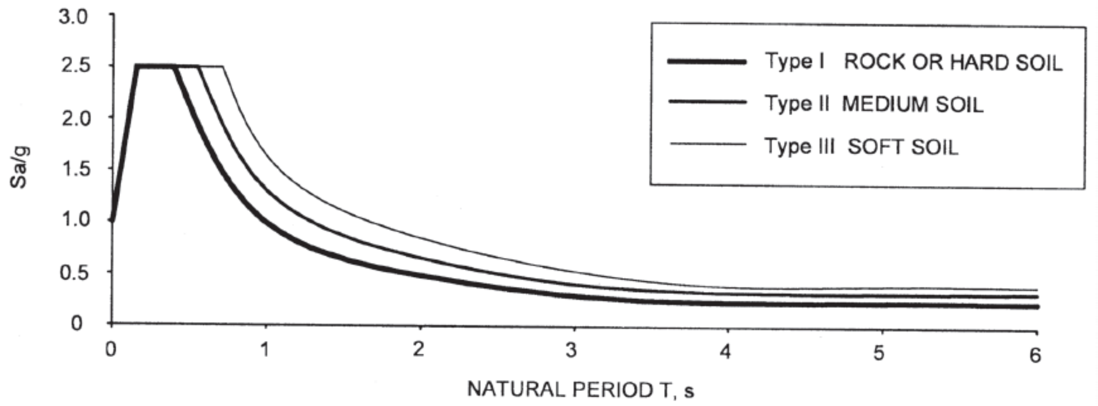


Fig: 3.1A Spectra for Response Spectrum Method *[Courtesy: IS 1893 (Part-1): 2016]*

REMARKS

As response spectra for Equivalent Static Method and Response Spectrum method are given separately, in both cases S_a/g values will change. It will change the values of A_h and V_B. Alternatively, as the Expressions for calculating design acceleration coefficient

(S_a/g), for Rocky/hard soils, medium soils and soft soils are given separately for static and dynamic analysis, it will Change the values of A_h and V_B. Structure cost may increase, but at the same time the structural strength is also increased towards earthquake forces.

3.1.3. URM INFILL WALLS MODELLING:

AS PER OLD CODE IS: 1893 (PART-1): 2002

Code is silent about modelling of masonry infill walls. Only equation for $T_a = 0.09h/\sqrt{d}$ for Buildings with masonry infill walls is given. Cl.7.6.1 Hence, in analysis T_a is taken considering masonry infill, but stiffness of infill wall is not considered in analysis.

AS PER REVISED CODE IS 1893 (PART-1): 2016

RC Framed Building with Unreinforced Masonry Infill walls: Cl.7.9 this clause has been newly added and discusses the calculation of earthquake loads when infills are considered. A detail procedure for URM infill by Equivalent diagonal strut method has been given in Cl.7.9.2.2

REMARKS

As per old code the modelling of brick infill was not incorporated in the code and because of this the designers used the empirical formula which is conservatively written for all the reinforced concrete structures. Hence the modelling of brick infill by equivalent Strut represent the actual stiffness distribution of structure as a whole thus the time period calculation will be more close to realistic condition of Building Structure. As per IS: 1893 -2016 New code, Modelling with URM infill considers the stiffness of the infill in analysis thus, sizes of columns /shear walls may Increase or decrease as per the stiffness distribution of brick infill in the Structure.

3.1.4. SOFT STOREY:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl. 4.20 a soft storey is defined as the storey in which the lateral stiffness is less than 70% of that in the storey above, or less than 80% of the average lateral stiffness of the three storey above. Infill, but stiffness of infill is not considered in analysis.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.4.20.1 a soft storey is defined as the storey in which the lateral stiffness is less than that in the storey above.

REMARKS

In new code IS 1893-2016, the criteria for soft story are made stricter. The stiffness of lower story should not be less than that of the upper story. Soft story is a source of weakness in the structure and should be avoided.

3.1.5. DYNAMIC ANALYSIS REQUIREMENT:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl.7.8.1 for Regular Buildings:

Zone- IV, V-------- height > 40m

Zone- II, III-------- height > 90m

For Irregular Buildings:

Zone- IV, V-------- height > 12m

Zone- II, III-------- height > 40m

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.7.1 Equivalent static analysis shall be applicable for regular buildings with height < 15m in seismic Zone II. [Cl.7.6. and cl.7.7.1]

Equivalent Static method should be used for regular building structure with approximate natural periods is less than 0.4 sec. [Cl.6.4.3]

REMARKS

Dynamic analysis considers different mode shapes, modal mass participation in each mode and modal combinations. Hence, in seismic zones III, IV and V and height of building more than 15 m, it is safer to perform dynamic analysis. I.e. Dynamic analysis is compulsory for almost all buildings in all zones.

3.1.6. MOMENT OF INERTIA:

AS PER OLD CODE IS 1893 (PART-1): 2002

Clause regarding Moment of Inertia is not mentioned in old code.

Thus, analysis is made considering full Moment of Inertia, i.e. Un-cracked section is considered.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.6.4.3.1 the moment of inertia for structural analysis shall be taken as given below.
For RC and Masonry Structures:
I_{eq}= 0.70 I_{gross} for columns
I_{eq}= 0.35 I_{gross} for beams
For Steel structures:
I_{eq}= I_{gross} for beams and columns

This clause of code takes into account, the cracked section properties.

REMARKS

This clause is added for safety and post-earthquake effect. In old IS 1893 (Part-1): 2002 full section, i.e. full moment of inertia of columns and beams is considered. In new code IS 1893 (part-1): 2016, cracked section with 70% MI of columns and 35% MI of beams is considered. As concrete is seems to be cracked section all time, one cannot consider the full MI of RC section for analysis. Full MI of RC members make structure stiff hence the deflection at top storey, drift of storey, lateral displacement of storey etc. are estimated incorrectly as smaller values. On the other hand by considering the cracked moment of inertia lateral deflection, drifts etc. will increase and to control one should have to increase the sizes of lateral load resisting members, which ultimately cause safety of structure. Hence for safety it is more reasonable to consider cracked section properties in analysis.

3.1.7. TORSION IRREGULARITY:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl.7.1, Table-4 Torsion irregularity as per old code is $\Delta_2 > 1.2\ (\Delta_1+\Delta_2)/\ 2$

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.1 Table-5 Torsion irregularities as per new code is $\Delta_{max} > 1.5\ \Delta_{min}$.

When, $\Delta_{max} > (1.5\text{-}2.0)\ \Delta_{min}$ configuration shall be revised.

REMARKS

As per old code IS 1893 (Part-1): 2002, torsion irregularity is based on 1.2 times average drift of structure, while as per new code it is based on 1.5 times minimum displacement.

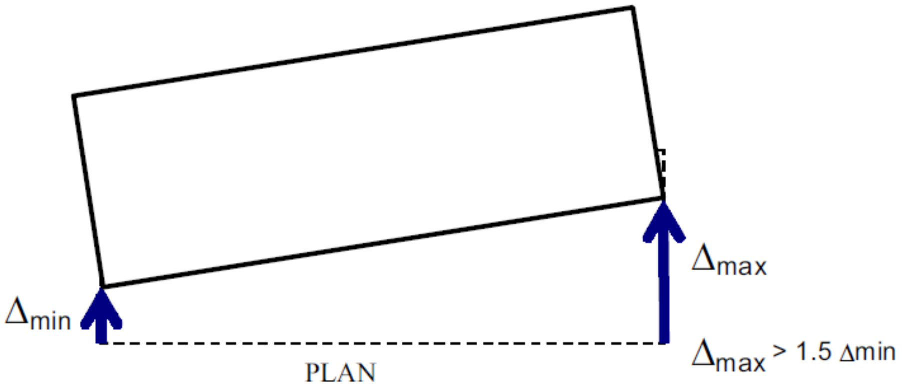


Fig: 3.2 Torsional Irregularity Criteria *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.8. RE-ENTRANT CORNERS:

AS PER OLD CODE IS 1893 (PART-1): 2002

Cl.7.1, Table-4 as per old code, for re-entrant corner,

A/L > 0.15-0.20

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.1 Table-5 as per new code, for re-entrant corner,

A/L > 0.15

REMARKS

As per new code for re-entrant corners, A/L values have been restricted to 0.15 which was permitted by old code as 0.15-0.20. For buildings with re-entrant corners three-dimensional dynamic analysis shall be performed.

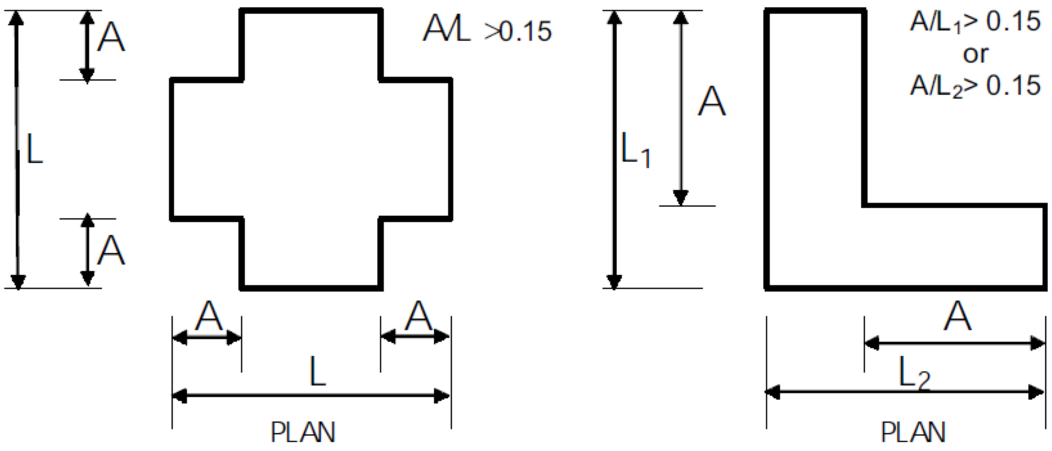


Fig: 3.3 Re-entrant Corner *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.9. DIAPHRAGM DISCONTINUITY:

AS PER OLD CODE IS 1893 (PART-1): 2002

(Major cut-outs) Cl.7.1 Table: 4 in old code Flexible or rigid diaphragm consideration are not mentioned.

If $A_o > 0.5\ A_{total}$ it is mentioned discontinuous diaphragm.

Where, A_o= cut-out or open area.

AS PER REVISED CODE IS 1893 (PART-1): 2016

(Major cut-outs) As per new code, Cl.7.1 Table-5

If $A_o > 0.5A_{total}$ - Flexible Diaphragm

If $A_o < 0.5A_{total}$ - Rigid Diaphragm

REMARKS

As per new code when cut out or opening is located near the edge of the slab and If $A_o > 0.1A_{total}$ it is considered flexible diaphragm. For continuity of in plane stiffness, the diaphragm shall be rigid.

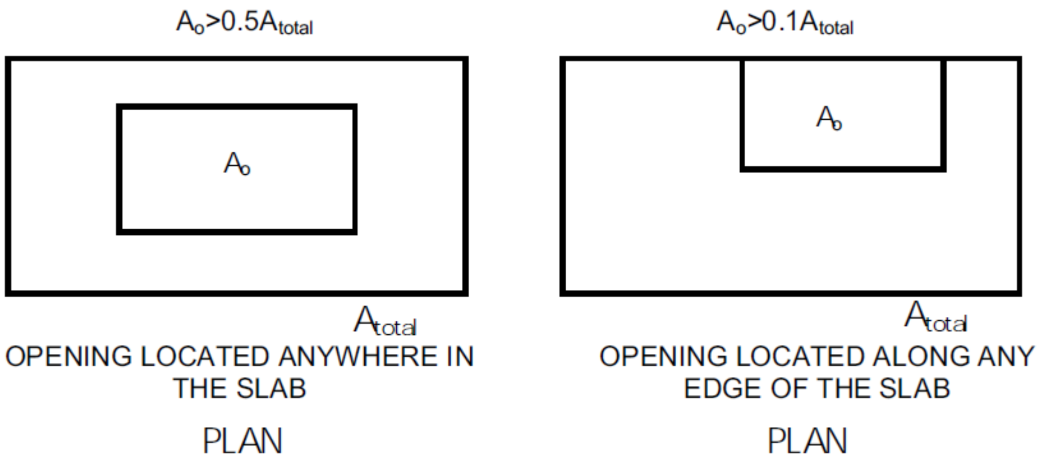


Fig: 3.4 Opening in Slab *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.10. MASS IRREGULARITY:

AS PER EXISTING CODE IS 1893 (PART-1): 2002

Cl.7.1, Table-5, as per old code, mass irregularity is considered to exist when the seismic weight of any floor is more than 200 % of that of the floor below or above.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.1, Table-6, as per new code, mass irregularity is considered to exist when the seismic weight of any floor is more than 150 % of that of the floor below.

$W_i > 1.5\ W_{i-1}$.

$W_i > 1.5\ W_{i+1}$.

In buildings with mass irregularity and located in seismic zones III, IV and V dynamic analysis shall be performed.

REMARKS

The criteria for a building to become mass irregular have been made stricter in new code. In old code mass variation of any floor with respect to near floor was allowed 200%, which has been reduced to 150 %.

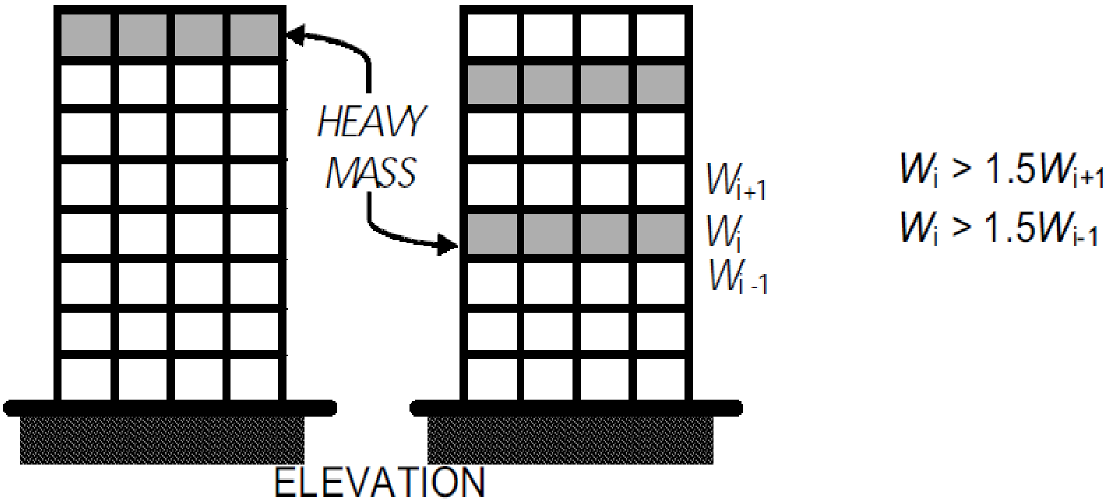


Fig: 3.5 Mass Irregularities *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.11. VERTICAL GEOMETRIC IRREGULARITY:

AS PER EXISTING CODE IS 1893 (PART-1): 2002

Cl.7.1, Table-5, as per old code, the vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 150 % of the storey below or above. A/L > 0.15L, L_2/L_1 >1.5.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.1, Table-6,as per new code, the vertical geometric irregularity considered to exist, when the horizontal dimension of the lateral force resisting system in any story is more than 125 % of the storey below. A/L > 0.125L, L_2/L_1 >1.25

REMARKS

The criterion for vertical geometric irregularity has been made stricter in new code. In old code variation of horizontal dimension of lateral load resisting system was allowed up to 150%, which has been restricted in new code up to 125%.

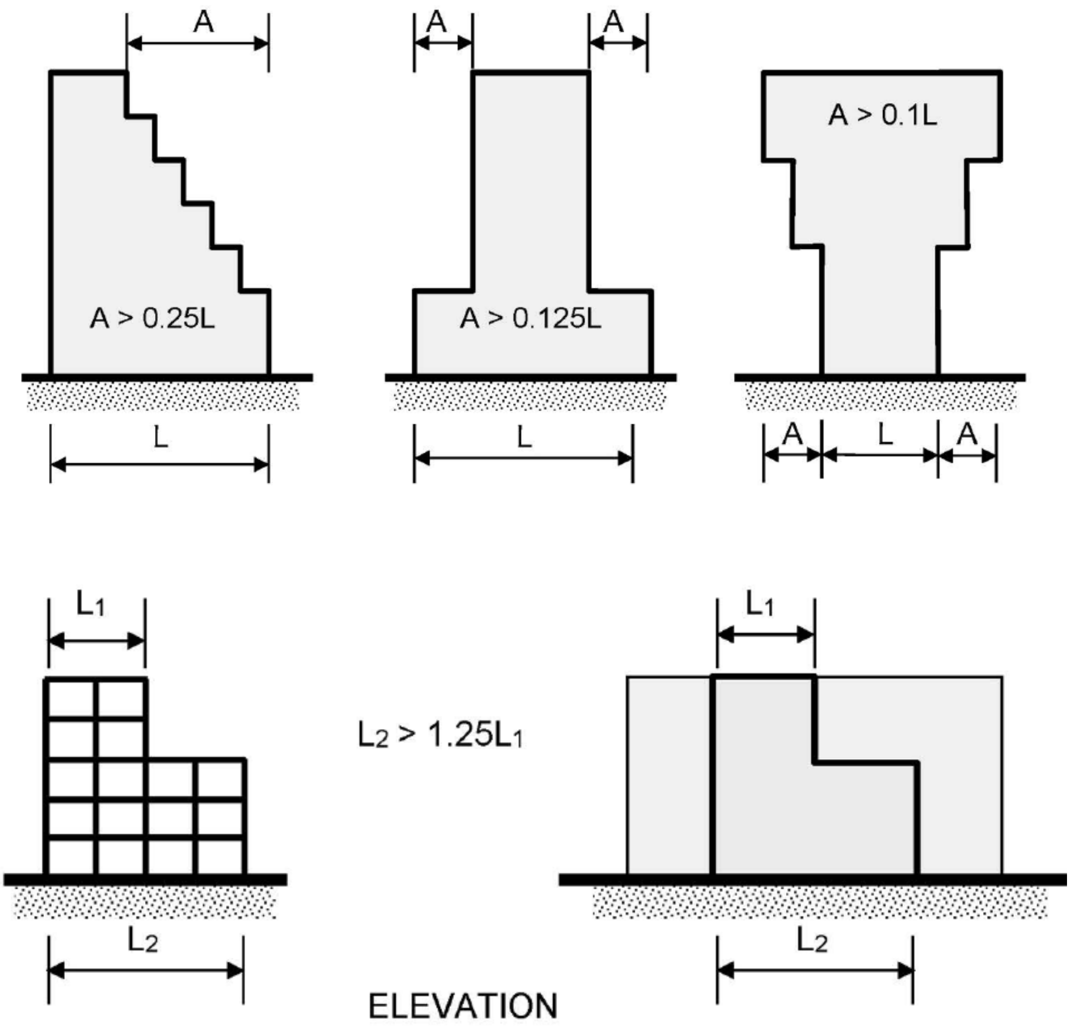


Fig: 3.6 Vertical Geometric Irregularities *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.12. DIAPHRAGM:

AS PER OLD CODE IS 1893 (PART-1): 2002

Clause regarding flexible or rigid diaphragm does not appear in old code.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl.7.6.4, the requirements for the floor diaphragm to be rigid or flexible is revised

When, $\Delta_{middle} > 1.2\Delta_{ave}$

It is considered flexible diaphragm, otherwise it is rigid diaphragm. Usually floor slab with plan aspect ratio (L/B) < 3 is considered rigid diaphragm.

REMARKS

Usually reinforced concrete slab with monolithic slab-beam floors are considered to be rigid diaphragm. In case of flexible diaphragm design storey shear shall be distributed to the various vertical elements of lateral load resisting system considering the in-plane flexibility of the diaphragms. (Cl.7.6.4)

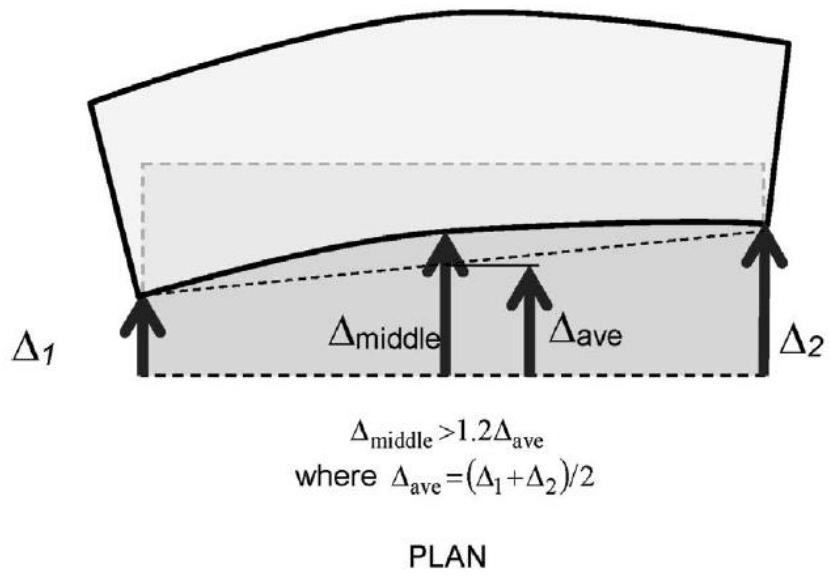


Fig: 3.7 Definition of Flexible Floor Diaphragm *[Courtesy: IS 1893 (Part-1): 2016]*

3.1.13. DAMPING RATIO:

AS PER EXISTING CODE IS 1893 (PART-1): 2002

Cl. 7.8.2.1, damping of 2% was allowed for steel structures in old code, which is now 5%. In Table-3 of old code, multiplying factors for obtaining values for other damping.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Cl. 7.2.4, the value of damping shall be 5% of critical damping for calculating A_h, irrespective of the material of construction (steel, reinforced concrete, masonry, etc.) of its lateral load resisting system. The value of damping is same (5%) irrespective of the method of analysis used, namely, Equivalent Static Method, or Dynamic analysis Method. Table-3 of old code, multiplying factors for obtaining values for other damping has been removed.

REMARKS

For steel structures in new code damping allowed is 5% which was 2% in old code. As per Table 3 of old code multiplying factor for 5% damping is 1.0 while for 2% damping it is 1.40. As damping increases S_a/g value decreases.

3.1.14. INCREASE IN ALLOWABLE SOIL PRESSURE:

AS PER EXISTING CODE IS 1893 (PART-1): 2002

Cl.6.3.5.2, when earthquake forces are considered, increase in allowable pressure in soils for different types of soils (Type-I, II, III) and different types of foundations, namely, piles, raft, well foundations etc, was given in Table-1 from 25% to 50%.

AS PER REVISED CODE IS 1893 (PART-1): 2016

Increase in net pressure on soils in design of foundations Cl.6.3.5.2 New code IS 1893-2016, gives percentage increase in net bearing pressure and skin pressures for soil types A, B, and C as 50%, 25%, and 0% respectively in Table-1. For soft soil no increase in bearing pressure shall be applied because, settlements cannot be restricted by increasing bearing pressure.

REMARKS

For determining percentage increase in net bearing pressure, soils have been classified in to four types, Type- A, B, C, and D in Table-2, which is not available in old code. Soil Type-D is included and designated as unstable collapsible, liquefiable soils. When N values are less than desirable N values in Table 1, it is stipulated that using suitable ground improvement technique; the N values should be increased. In old code compacting was suggested for increase of N. The new code is silent for the method. It is necessary to know, for how much depth, the compaction is required. Dynamic compaction is a costlier method and can be used in VIP structures.

3.1.15. RESPONSE REDUCTION FACTORS:

AS PER EXISTING CODE IS 1893 (PART-1): 2002

Response reduction factor as per Table -9 of IS: 1893 (part-1): 2002

AS PER REVISED CODE IS 1893 (PART-1): 2016

Response reduction factor as per Table -9 of IS: 1893 (part-1): 2016

REMARKS

1. RC and Steel structures in Seismic Zones III, IV and V shall be designed to be ductile. Hence, this system is not allowed in these Seismic zones.
2. Eccentric Braces shall be used only with SBFs.
3. Buildings with Structural Walls also include building s having Structural Walls and Moment Frames, but where: Frames are not designed to carry design lateral loads, or
4. Frames are designed to carry design lateral loads, but do not fulfil the requirements of 'Dual Systems'.

In these buildings,

i. Punching shear failure shall be avoided, and
ii. Lateral drift at the roof under design lateral forces shall not exceed 0.1 percent.

3.2. SOME SILENT STIPULATIONS OF NEW IS 16700: 2017 "CRITERIA FOR STRUCTURAL SAFETY OF TALL REINFORCED CONCRETE BUILDINGS"

3.2.1. TALL BUILDING:

It is a building of height greater than 50m, but less than 250m, normally intended to be used as residential, office and other commercial buildings.

3.2.2. SUPER TALL BUILDING:

It is a building of height greater than 250m.

3.2.3. HEIGHT LIMIT FOR STRUCTURAL SYSTEMS:

The maximum building height (in m) shall not exceed values given in table 1 for buildings with different structural systems.

Table 3.1 Maximum value of Height, H above Top of Base Level of Buildings with Different Structural Systems, in meter *[Courtesy: IS 16700: 2017]*

<table>
<tr><th rowspan="3">Sl. No.</th><th rowspan="3">Seismic Zone</th><th colspan="6">Structural System</th></tr>
<tr><th rowspan="2">Moment Frame</th><th colspan="2">Structural Wall</th><th rowspan="2">Structural Wall + Moment Frame</th><th rowspan="2">Structural Wall + Perimeter Frame</th><th rowspan="2">Structural Wall + Framed Tube</th></tr>
<tr><th>Located at Core</th><th>Well-Distributed [1)]</th></tr>
<tr><td>(1)</td><td>(2)</td><td>(3)</td><td>(4)</td><td>(5)</td><td>(6)</td><td>(7)</td><td>(8)</td></tr>
<tr><td>(i)</td><td>V</td><td>NA</td><td>100</td><td>120</td><td>100</td><td>120</td><td>150</td></tr>
<tr><td>(ii)</td><td>IV</td><td>NA</td><td>100</td><td>120</td><td>100</td><td>120</td><td>150</td></tr>
<tr><td>(iii)</td><td>III</td><td>60</td><td>160</td><td>200</td><td>160</td><td>200</td><td>220</td></tr>
<tr><td>(iv)</td><td>II</td><td>80</td><td>180</td><td>220</td><td>180</td><td>220</td><td>250</td></tr>
</table>

[1)] Well- distributed shear walls are those walls outside of the core that are capable of carrying at least 25% of the lateral loads.

3.2.4. SLENDERNESS RATIO:

The maximum values of the ratio of height h to minimum base width shall not exceed values given in table 2.

Table 3.2 Maximum Slenderness Ratio (H_t /B_t) *[Courtesy: IS 16700: 2017]*

Sl. No.	Seismic Zone	Structural System					
		Moment Frame	Structural Wall		Structural Wall + Moment Frame	Structural Wall + Perimeter Frame	Structural Wall + Framed Tube
			Located at Core	Well-Distributed 1)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
(i)	V	NA	8	9	8	9	9
(ii)	IV	NA	8	9	8	9	9
(iii)	III	4	8	9	8	9	10
(iv)	II	5	9	10	9	10	10

3.2.5. PLAN ASPECT RATIO:

The maximum plan aspect ratio (L/B) of the overall building shall not exceed 5.0

3.2.6. LATERAL ACCELERATION:

From serviceability considerations, (human comfort) under standard wind loads with return period of 10 years, the maximum structural peak combined lateral acceleration a_{max} in the building for along and across wind actions at any floor level shall not exceed values given in table- 4, without or with the use of wind dampers in the building.

Table 3.3 Permissible Peak Combined Acceleration (Clause 6.2.3)

[Courtesy: IS 16700: 2017]

Sl. No.	Building Use	Maximum Peak Combined Acceleration, a_{max} (m/s^2)
(1)	(2)	(3)
(i)	Residential	0.15
(ii)	Mercantile	0.25

3.2.7. MINIMUM DESIGN BASE SHEAR COEFFICIENT:

Vertical shaking shall be considered simultaneously with horizontal shaking for tall buildings in seismic Zone V. For buildings in seismic Zone IV and V, deterministic site-specific design spectra shall be estimated and used in design. When site specific investigations result in higher hazard estimation, site-specific investigation results shall be used.

Design base shear coefficient of a building under design lateral forces, shall not be taken less than that given in Table 4

Table 3.4 Minimum Design Base Shear Coefficient (Clause 6.3.3)

[Courtesy: IS 16700: 2017]

Sl. No.	Building Height, H	Seismic Zone			
		II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
(i)	$H \leq 120m$	0.7	1.1	1.6	2.4
(ii)	$H \geq 200m$	0.5	0.5	0.75	1.75

NOTE: For buildings of intermediate height in the range 120m-200m, linear interpolation shall be used.

REMARKS ON 3.2.3, 3.2.4, 3.2.5, 3.2.6 AND 3.2.7

Initial sections of the Code i.e. section 1 to 4 presents scope of the code, references, terminologies and symbols. The code defines its scope that the clauses are applicable for buildings with height ranging from 50 m to 250 m and the corresponding occupancy less than 20,000 people. The code also restricts its applicability for area within 10 km of seismic fault lines i.e. near field. In section 5 the code deals with height, slenderness and aspect ratio of the various structural systems. This section also discusses about materials, floor system and progressive collapse analysis of the building. Section 5.1 of the Code discusses about elevational aspects of tall buildings. In this section limitation for three important elevational features viz. height, slenderness ratio and aerodynamic effects are provided. Limitation on height of tall buildings is defined on the basis of seismic zones and structural system. The code suggests six different structural systems for tall buildings viz. ‘moment frame’, ‘structural wall located at core’, ‘structural wall well distributed', 'structural wall + moment frame', 'structural wall + perimeter frame' and 'structural wall + framed tube'. The code restricts use of 'moment frame' for tall buildings in seismic zone IV and V, whereas, in seismic zone III and II it limits the height to 60 m and 80 m, respectively. 'Structural wall' system in code is divided into two sub-groups i.e. 'located at core' and 'well distributed'. The 'well distributed' structural wall system has higher height limits as compared to the core system. For 'structural wall + moment frame' system the height limits are similar to structural wall located at core and for 'structural wall + perimeter frame' the height limits are similar to 'structural wall well distributed' system. For higher stories other systems such as 'structural wall + frame tubes', 'tube-in-tube', 'outrigger' and bundled tube system are efficient. However code provides the height limits only for 'structural wall + frame tubes' among the above mentioned systems. For 'structural wall + frame tubes' system the maximum height allowed by the code in seismic zone V and VI is limited to 150 m, whereas for seismic zone III to 220 m and seismic zone II up to 250 m. Maximum slenderness ratio (height to the minimum base width) of 9 is allowed in seismic zone IV and V for 'structural wall + framed tube system', whereas, in seismic zone II, III the slenderness ratio up to 10 is allowed for this system. . The less slender building performs better than buildings with high slenderness. It is observed that the slenderness ratio increases drift, acceleration and torsional moment on buildings. To consider aerodynamic effects code suggests selecting elevation profile, façade features and plan shape of the building to attract minimum wind drag effects. Features like sharp corners, projected balconies etc. are to be considered in design. For reducing the aerodynamic effect, aerodynamic treatment of a tall building is done. These treatments reduce the wind load on the building by making the building like a bluff body. The aerodynamic treatment can be done in two ways i.e. by design treatment and by modification treatment of elevation. In design treatment, complete elevation changes as per wind profile while in the modification treatment small changes such as corner treatments are done. Section 5.2 of the Code deals with plan geometry and plan aspect ratio (larger plan dimension / smaller plan dimension; L/B, where L>B) of tall buildings. Code deals with basic geometries like rectangular, square and elliptical plan shapes. The plan aspect ratio is restricted to 5 by the code. It is observed that seismic performance of buildings with lower aspect ratio is better than buildings with higher aspect ratio. They indicated that a square configuration shows best performance, moreover, aspect ratio greater than 4 is not desirable. Section 5.6 of the Code deals with various aspects of floor systems in tall building. The code suggests using cast-in-situ floor system. However, the code also allows precast floor system with a minimum screed thickness of 75mm for seismic zone III, IV and V; however, for seismic zone II the clause relaxes to use 50 mm screed thickness. A rigid floor system imparts diaphragm action in the building. Cast-in-situ floor system is generally rigid, whereas, the rigidity of precast floor system depends on screed thickness. Therefore, the code suggests to model the in plane stiffness of floor slab. Moreover, opening in the floor system reduces stiffness and thus the diaphragm action, therefore, code provides stringent criteria about size and location of opening and maximum opening is restricted to 30% of the floor plan area. The code allows maximum vertical vibration frequency of floor system up to 3 Hz and for higher value it demands demonstration for acceptability. The code also provides limiting value for vertical acceleration of the floor system under gravity loading. In section 5.7, the Code limits minimum and maximum grade of concrete to be used in tall buildings to M30 and M70, respectively. For higher concrete grades experimental study is recommended to ensure a minimum of 0.002 strains in compression. As per IS 456: 2000 the minimum grade of concrete is M20, whereas, IS 13920: 2016 recommends M25 for buildings taller than 15 m in seismic zone III, IV and V. Similarly, other national codes also recommend using higher grades for seismic regions than for non-seismic regions. As reported by **Pauley and Priestley (1992**) that with increase in grade of concrete strain at peak stress as well as at first crushing decreases, therefore, the brittleness of high grade concrete is a matter of concern and designers shall be careful while using high strength concrete. In section 5.8, guidelines to avoid progressive collapse of tall buildings are included. Progressive collapse can be defined as the failure initiates at local element level and then propagates from element by element, finally causes collapse of entire building. As per code, progressive collapse can be precluded by; selecting suitable structural system, selection of critical member and providing adequate redundancy in the building. Code also suggests using key elements to safeguard the building from progressive collapse. In addition to the methods indicated in the code to reduce the chances of progressive collapse, other methods are also suggested in various literature; these are by improving the ductility of the member, identifying and strengthening of critical locations of building and by providing alternate load paths.

3.2.8. STRUCTURE MODELLING:

i. Rigid end offsets of linear members in the joint region, when centreline modelling is adopted.
ii. Floor diaphragm flexibility, as applicable;
iii. Cracked cross sectional area properties as per Table-5; and
iv. P-Δ effects.

Table 3.5 Cracked RC Section Properties (Clause 7.2)

[Courtesy: IS 16700: 2017]

Sl. No.	Structural Elements	Un-factored Loads		Factored Loads	
		Area	Moment of Inertia	Area	Moment of Inertia
(i)	Slabs	$1.0A_g$	$0.35I_g$	$1.0A_g$	$0.25I_g$
(ii)	Beams	$1.0A_g$	$0.70I_g$	$1.0A_g$	$0.35I_g$
(iii)	Columns	$1.0A_g$	$0.90I_g$	$1.0A_g$	$0.70I_g$
(iv)	Walls	$1.0A_g$	$0.90I_g$	$1.0A_g$	$0.70I_g$

REMARKS

Section 7.2 of the Code deals with the various additional considerations in modelling of tall buildings such as rigid offset, floor diaphragm flexibility, crack section and P-Δ effect. Crack section properties of the IS code is similar to the ACI code, as shown in Table 3.6. In general crack formation under gravity loads and secondary effects such as shrinkage, temperature and hydration of concrete are negligible. In seismic effect, reversal of stress results in flexural crack in the structural elements. The crack width varies along the length of the section, to consider the reduced the stiffness and strength parameter of the element; moment of inertia of the sections are reduced. Cracked reinforced concrete sections by **Paulay and Priestly (1992),** as shown in Table 3.7. Kumar and Singh (2010) also proposed simplified formula for calculation of stiffness of frame elements based on axial load ratio as shown in Table 3.8.

Table 3.6 Cracked RC section properties of various national codes

[Courtesy: ACI 318-14, IS 16700]

Member	ACI 318-14	IS 16700 Un-factored load	IS 16700 Factored load
Beam	$0.35I_g$	$0.70I_g$	$0.35I_g$
Column	$0.70I_g$	$0.90I_g$	$0.70I_g$
Wall Uncrack	$0.70I_g$	--	--
Wall crack	$0.35I_g$	$0.90I_g$	$0.70I_g$
Slab / Flat plate	$0.25I_g$	$0.35I_g$	$0.25I_g$

Table 3.7 Cracked RC section properties

[Courtesy: Literature, Paulay and Priestly, 1992]

Member	Range	Recommended value
Rectangular Beam	$0.30I_g$-$0.50I_g$	$0.40I_g$
T and L beams	$0.25I_g$ -$0.45I_g$	$0.35I_g$
Column ($P>0.5F_cA_g$)	$0.70I_g$-$0.90I_g$	$0.80I_g$
Column ($P>0.2F_cA_g$)	$0.50I_g$-$0.70I_g$	$0.60I_g$
Column ($P>0.05F_cA_g$))	$0.30I_g$-$0.50I_g$	$0.40I_g$

Table 3.8 Effective stiffness ratio for NSC and HSC

[Courtesy: Literature, Kumar and Singh, 2010]

Effective stiffness ratio for normal strength concrete (NSC)	Effective stiffness ratio for normal strength concrete (HSC)
$\frac{EI_{eff}}{E_cI_g}=\begin{cases}0.35 & for\frac{P}{A_gf'_c}\le 0.2\\ 0.175+0.875\frac{P}{A_gf'_c} & for\ 0.2\le\frac{P}{A_gf'_c}\le 0.6\\ 0.7 & for 0.6\le\frac{P}{A_gf'_c}\end{cases}$	$\frac{EI_{eff}}{E_cI_g}=\begin{cases}0.35 & for\frac{P}{A_gf'_c}\le 0.1\\ 0.24+1.1\frac{P}{A_gf'_c} & for\ 0.1\le\frac{P}{A_gf'_c}\le 0.6\\ 0.9 & for\ 0.6\le\frac{P}{A_gf'_c}\end{cases}$

3.2.9. CONCEPT OF BACKSTAY LOAD PATH:

In estimating the backstay effect, two lateral load paths shall be considered, namely:

1) Direct load path, where overturning resistance is provided by the tower core elements and foundation directly beneath the tower; and

2) Backstay load path, where overturning resistance provided by in-plane forces in backstay elements (lower floor diaphragms and perimeter walls). Backstay floor diaphragms shall be modelled considering there in-plane out-of-plane floor flexibility. Any large discontinuity present in the slab shall be modelled.

Backstay diaphragm floor shall be at least 150mm thick, and shall have two curtains of vertical & horizontal reinforcements of amount not less than 0.25% of cross section area in each direction.

Adequate measurement shall be taken to prevent shear sliding failure at connections of diaphragm to structural walls; here, the inclination of the strut shall be taken as 45°. Additional reinforcement shall be provided to resist the shear force at the interface between diaphragm and structural walls.

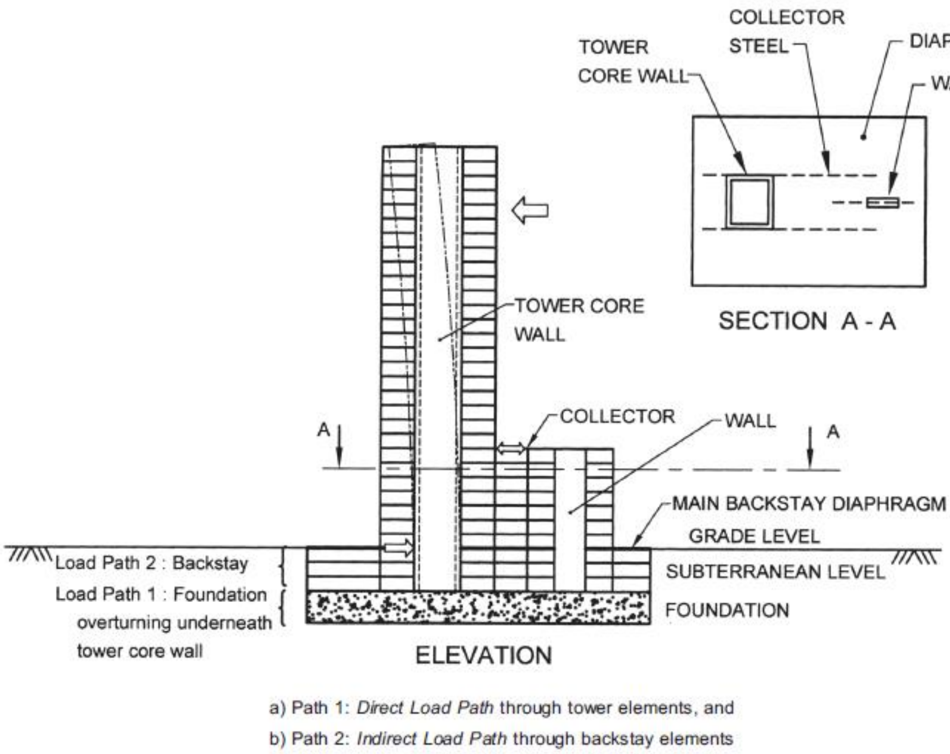


Fig 3.8 Load Paths in Lateral Overturning Resistance of Tall Buildings with Podiums

[Courtesy: IS 16700: 2017]

3.2.10. CONCEPT OF FLAT SLAB – STR. WALL SYSTEM:

The structural wall shall carry all lateral loads on the building, and the column strips of the flat slab system shall not be included in the lateral load resisting system.

3.2.11. CONCEPT OF FRAMED TUBE SYSTEM, TUBE-IN-TUBE SYSTEM AND MULTIPLE TUBE SYSTEM:

The plan shape of a tube in tube system shall be regular with a length to width ratio not more than 2 and, the inner tube shall be centred with the outer tube. Re-entrant corners and sharp edges to tubular form should be avoided. Column spacing of framed tube shall preferably be not more than 5m.

In framed-tube system the area of corner column shall be 1 to 2 times that of internal column and the height to width ratio of the opening shall be similar to ratio of storey height to column spacing.

The beams carrying predominantly gravity load shall be directly supported on columns or on walls and not on frame beams.

The minimum requirements for reinforcement bar diameters in beams of moment frames of framed-tube structures are given in Table 9.

Table 3.9 Reinforcement requirements in beams (Clause 8.7.8)

[Courtesy: IS 16700: 2017]

Sl. No.	Reinforcement Type	Seismic Zone	
		II	III, IV and V
(1)	(2)	(3)	(4)
(i)	Stirrup diameter	≥ 8	≥ 10
(ii)	Stirrup spacing	≤ 150	≤ 100
(iii)	Main (Longitudinal) reinforcements	≥ 16	≥ 16

REMARKS ON 3.2.9, 3.2.10 AND 3.2.11

Section 8 of the Code deals with design criteria of the various structural system viz. ‘frame buildings’, ‘structural wall system’, ‘moment frame + structural wall system’, ‘flat slab + structural wall system’ and ‘structural wall +framed tube system’. Section 8.1.2 emphasizes that those staircase which are not confined by structural wall shall be included in modelling. The staircase changes the dynamic characteristic of buildings due to associated truss action of the waist slab, moreover, the location and orientation also has significant effect in structural performance. Section 8.1.3.2 of the code deals with sensitivity analyses of the building. In addition to cracked section analysis, the code suggests to conduct lower and upper bound sensitivity analyses. In sensitivity analyses code suggests to reduce the section properties of floor diaphragm and stiffness parameters of perimeter wall of podium and floor system at backstay level. In upper bound sensitivity analyses the area and moment of inertia properties of the section reduced to 50% of the gross parameters whereas, in lower bound sensitivity analyses aforementioned parameters properties of the section reduced to 15% of the gross parameters. Section 8.2 emphasizes to ensure the various aspects of ductility in a tall building. Section 8.3 of the Code deals with provisions related to 'frame buildings'. Code suggests that tall buildings shall have at least 3 bays and 2 frames to resist seismic effect. It is observed that minimum number of bays may be used for the frame mechanism of failure during earthquake. Code mandates that for moment frames system to be detailed as per 'special moment frame' system in seismic zone III which is in line with IS 13920: 2016. Section 8.4 deals with dual system (moment frame - structural Wall). Code necessitates supporting structural wall on stiff and ductile elements and restricts discontinuing it in lower stories. The discontinuity of the structural wall even in higher stories of building creates larger lateral drift in the top stories and uneven distribution of the shear demand in the building. Section 8.5 of the code deals with 'structural wall' system. Code suggests maintaining a minimum thickness of the structural wall as greater of 160 mm or hi/20 where h_i is inter-storey height of i^{th} floor. As per IS 13920: 2016, the minimum thickness of the structural wall shall be 150 mm (300 mm for buildings with coupling beams). In general, opening in the structural wall are provided for various purpose such as windows, etc. Code suggests if opening size is less than 800 mm or $1/3^{rd}$ length of the wall (whichever is smaller) then the influence due to openings can be neglected in overall stiffness of the building. Code mandates to provide additional reinforcement at four sides of the opening as per IS 13920 with minimum bar diameter of 12mm. It was observed in the literature that large opening in the 'structural wall' system increases the flexibility of the building. In structural wall with coupling beam, coupling beams undergoes higher rotational due to large openings. For resisting rotations in the coupling beam code recommends using diagonal reinforcements as per IS 13920:2016. In code maximum reinforcement in coupling beams are restricted based on span – depth ratio. The failure analysis of the staggered openings in the structural wall; it was observed that the structural wall with staggered openings is more rigid and have a higher load carrying capacity compared with ordered openings. The code withholds supporting beams or columns with high vertical load on coupling beams. Section 8.6 of the Code deals with ‘flab slab + Structural wall’ system. Code recommends that structural wall shall carry entire lateral load of the building. Moreover, column strip of the flat slab system shall not be included in lateral load resisting system. Section 8.7 of the Code deals with ‘framed tube system’, ‘tube-in-tube system’ and ‘multiple tube system’. The tubular systems act in building like a hollow cylinder perpendicular to the ground. In this system, structural walls/columns shall be placed very close to each other to create a 3D cylindrical member to resist lateral load of the building. Code restricts length to width ratio of a ‘tube-in-tube’ system not exceeding to 2. For creating the hollow cylinder in the system code restricts spacing between columns/structural walls to be not more than 5m. The re-entrant corner and sharp changes in tubular system reduce the cylindrical action of the structural wall. Code recommends avoiding re-entrant corners and sharp changes in this system. In tubular system, outer columns resist lateral loads while the internal columns are used for resisting gravity load on the building. Code recommends corner columns area shall be at least 2 times of the internal columns. Code recommends minimum 16 mm diameter beam longitudinal diameter in all seismic zones. For stirrups code recommends using 10 mm diameter bars in seismic zone III, IV and V, whereas, 8 mm is allowed in seismic zone II. Code also recommends keeping stirrups spacing not more than 100 mm is seismic zone III, IV and V, however, in zone II stirrups spacing shall not be more than 150 mm.

3.2.12. CONCEPT OF DEPTH OF FOUNDATION:

The embedded depth of the building shall be at least 1/15 of height of the building for raft foundation and 1/20 of the height of building for pile and pile raft foundation (excluding pile length). But this requirement may be relaxed, when the foundation reset on heard rock or when there is no uplift under any portion of the raft in the service load combination and provided the minimum competent founding strata requirement is fulfilled.

Podium or basement roof slab should be capable of transferring in plane shear from the tower to the foundation.

Expansion joints should preferably be avoided in basements of tall buildings.

REMARKS

Section 9 of the Code deals with factor of safety of building, geotechnical investigation, minimum depth of foundation, modelling of the soil and permissible settlement of foundation. Code recommends a uniform factor of safety of 1.5 for both overturning and sliding of building under different loads. IS 1904:1984 provides 'design and construction of foundation in soil' in which factor of safety ranges from 1.5 - 2 and 1.5 - 1.75 for overturning and sliding, respectively. Code recommends that all the geotechnical investigation shall be conducted for the tall building site including liquefaction potential analysis, soil spring constant and modulus of sub grade reaction to establish the safety of the building. Code recommends minimum 3 boreholes testing per tower and spacing between the boreholes in the plan area as 30m. Generally, IS 1892:1979 code is used for 'sub surface investigation for foundation', IS 1892 recommends 5 pits for 0.4Hectares. Section 9.3 of the Code recommends a minimum depth of foundation for raft and pile foundations as 1/15 and 1/20 of height of the building, respectively. As per IS 1904 there are no minimum depth criteria for any type of foundation. Section 9.7 of the code deals with modelling of the soil in the software. Code recommends to model soil through spring constants, zoned spring constant or modulus of sub grade reaction of the soil. Code recommends the implementation of soil structure interactions for buildings that are more than 150 m in height using actual column loads and column positions on the foundation. Code suggests that permissible settlement of foundation shall be as per IS 1904 and IS 12070 requirements.

3.2.13. CONCEPT OF NON-STRUCTURAL ELEMENT OF TALL BUILDINGS:

Non structural element of tall buildings shall comply with all relevant existing national standards and guidelines as laid down by the various statutory and non-statutory bodies as well as the client/owner of the building. The specification laid down in 10.1, 10.2 and 10.3 of is: 16700-2017 shall be applicable for,

i) Planning, design and construction of NSEs of new tall buildings.

ii) Re-planning, assessment and retrofitting of NSEs of existing tall buildings

NSEs shall be classified into 3 types depending on their earthquake behaviour, namely
1) Acceleration-sensitive NSEs
2) Deformation-sensitive NSEs
3) Acceleration-and-Deformation -sensitive NSEs

NSEs in tall buildings shall be protected against the effect mentioned above. Major NSEs shall be protected based on engineered calculation as per clauses given in IS 16700-2017

REMARKS

Section 10 of the code deals with design of non-structural elements (NSEs) in tall buildings. The design of NSEs is required because the cost of non-structural elements in the total cost of the project may be significant. During past earthquakes it was observed that performances of the NSEs are poor. Failure of the non-structural elements causes problem to the building occupants. In order to achieve operational or immediate occupancy seismic performance level it is important to appropriately design non-structural elements otherwise even minor disruption such as lack of water or power supply can compromise the functionality of the building. In the literature it is recommended that when NSEs significantly affects structural response of the building, they shall be considered in design and modelling of the building.

3.2.14. RECOMMENDATIONS FOR HEALTH MONITORING DEVICES FOR DEFORMATIONS OF BUILDINGS:

Earthquake Shaking: - All tall buildings in Seismic Zone V and tall buildings exceeding 150 m in Seismic Zones III and IV shall be instrumented with tri-axial accelerometers to capture translational and twisting behaviour of buildings during strong earthquake shaking.

Wind Oscillations: - Buildings over 150m in height may be instrumented with anemometer and accelerometers to measure wind speed, acceleration and direction on top of the buildings.

REMARKS

Section 11 of the code provides various recommended monitoring the building during earthquake. Code also deals with wind oscillation and foundation settlement throughout the life of the building. Code suggested for buildings in seismic zone V and buildings taller than 150 m in seismic zone III, IV to instrument with tri-axial accelerometer for monitoring the behaviour (translational and twisting behaviour) during strong earthquake. Code also suggested for buildings taller than 150 m to instrument anemometer and accelerometer to measure the wind speed, acceleration and direction at top of the building. It is also required by the code to record the settlements of the foundation by using permanent settlement markers during various levels of construction and building life. The data which is obtained from the devices are useful for validation of the design loads applied on the building. The settlement record is used to cross check the permissible limits with the code. The settlement data is used to observe soil behaviour during construction period.

Table 3.10 Design relative displacement for deformation sensitive NSEs

[Courtesy: IS 16700: 2017 and ASCE 7-10]

Code	Design relative displacement for NSEs
IS 16700 Displacement within the building in X and Y direction	$\Delta_x = 1.2(\delta_{Z1}^{Ax} - \delta_{Z2}^{Ax})$ $\Delta_y = 1.2(\delta_{Z1}^{Ay} - \delta_{Z2}^{Ay})$
ASCE 7-10 Displacement with in building	$D_p = \delta_{xA} - \delta_{yA}$ The limiting drift D_p is $D_p = (h_x - h_y)\frac{\Delta_{aA}}{h_{sx}}$
IS 16700 Displacement between building	$\Delta_x = \lvert\delta_{z1}^{Ax}\rvert + \lvert\delta_{z2}^{Bx}\rvert$ $\Delta_y = \lvert\delta_{z1}^{Ay}\rvert + \lvert\delta_{z2}^{By}\rvert$
ASCE 7-10 Displacement between building	$D_p = \lvert\delta_{xA}\rvert + \lvert\delta_{yB}\rvert$ The limiting drift D_p is $D_p = \frac{h_x \Delta_{aA}}{h_{sx}} + \frac{h_y \Delta_{aB}}{h_{sx}}$

CHAPTER-IV

1.1. DESIGN CONSIDERATIONS AND MODELLING TECHNIQUES

4.1. GENERAL:

This section deals with assigning material properties, member properties, assigning Basic loads [dead load, super-imposed dead load, earthquake load (static & dynamic), wind load (static and dynamic) and temperature load], and generation of load combinations, modelling techniques and structural analysis of the selected different type of structures for the intended research work.

4.2. APPROACH:

The buildings are planned as a combination of beam-slab system with columns and structural walls according to their respective type. After preliminary sizing of various structural members, computerized numerical models of the structural framing plans of the respective buildings are generated for carrying out the structural analysis considering all design stipulations of respective Indian Standards for the effects of vertical loads and lateral loads that are likely to be imposed on the structure.

The numerical models of building structure are developed and analyzed on ETABS Platform [ETABS 2017 version 17.0.1.], the integrated software package of CSI for the structural analysis and design of multi-storey buildings. The software is thoroughly tested and globally accepted for its outstanding performance.

The seismic analysis and wind analysis are carried out as per IS 1893 (Part-I): 2002/ IS 1893 (Part-1): 2016 and IS 1875 (Part-3): 1987/ IS 875 (Part-3): 2015. The permissible values of the load factors and stresses are utilized within the purview of the Indian Standards. The computer analysis of individual structure is performed to evaluate all comparative data of all respective structures such as time periods, storey displacement, storey drift, base shear, overturning moment, base Reactions at foundation level and deflection pattern of the whole structure and in the individual structural members.

To achieve the desire optimum results of building performances several structural analyses must be required to satisfy the strength, stiffness and stability in all respects.

4.3. FEM SOFTWARE PLATFORM [ETABS (CSI)]:

The innovative and revolutionary new ETABS is the ultimate integrated software package for the structural analysis and design of buildings. Incorporating 40 years of continuous research and development, this latest ETABS offers unmatched 3D object based modelling and visualization tools, blazingly fast linear and nonlinear analytical power, sophisticated and comprehensive design capabilities for a wide-range of materials, and insightful graphic displays, reports, and schematic drawings that allow users to quickly and easily decipher and understand analysis and design results.

From the start of design conception through the production of schematic drawings, ETABS integrates every aspect of the engineering design process. Creation of models has never been easier - intuitive drawing commands allow for the rapid generation of floor and elevation framing. CAD drawings can be converted directly into ETABS models or used as templates onto which ETABS objects may be overlaid. The state-of-the-art SAP Fire 64-bit solver allows extremely large and complex models to be rapidly analyzed, and supports nonlinear modelling techniques such as construction sequencing and time effects (e.g., creep and shrinkage).

Design of steel and concrete frames (with automated optimization), composite beams, composite columns, steel joists, and concrete and masonry shear walls is included, as is the capacity check for steel connections and base plates. Models may be realistically rendered, and all results can be shown directly on the structure. Comprehensive and customizable reports are available for all analysis and design output, and schematic construction drawings of framing plans, schedules, details, and cross-sections may be generated for concrete and steel structures.

ETABS provides an unequalled suite of tools for structural engineers designing buildings, whether they are working on one-story industrial structures or the tallest commercial high-rises. Immensely capable, yet easy-to-use, has been the hallmark of ETABS since its introduction decades ago, and this latest release continues that tradition by providing engineers with the technologically-advanced, yet intuitive, software they require to be their most productive.

MODELLING OF STRUCTURAL SYSTEM:

Fundamental to ETABS modelling is the generalization that multi-story buildings typically consist of identical or similar floor plans that repeat in the vertical direction. Modelling features that streamline analytical-model generation, and simulate advanced seismic systems, are listed as follows:

a) Templates for global-system and local-element modelling
b) Customized section geometry and constitutive behaviour
c) Grouping of frame and shell objects
d) Link assignment for modelling isolators, dampers, and other advanced seismic systems
e) Nonlinear hinge specification
f) Automatic meshing with manual options
g) Editing and assignment features for plan, elevation, and 3D views

LOADING, ANALYSIS AND DESIGN:

Once modelling is complete, ETABS automatically generates and assigns code-based loading conditions for gravity, seismic, wind, and thermal forces. Users may specify an unlimited number of load cases and combinations.

Analysis capabilities then offer advanced nonlinear methods for characterization of static-pushover and dynamic response. Dynamic considerations may include modal, response-spectrum, or time-history analysis. P-delta effects account for geometric nonlinearity.

Given enveloping specification, design features will automatically size elements and systems, design reinforcing schemes, and otherwise optimize the structure according to desired performance measures.

OUTPUT, INTEROPERABILITY, AND VERSATILITY:

Output and display formats are also practical and intuitive. Moment, shear, and axial force diagrams, presented in 2D and 3D views with corresponding data sets, may be organized into customizable reports. Also available are detailed section cuts depicting various local response measures. Global perspectives depicting static displaced configurations or video animations of time-history response are available as well.

ETABS also features interoperability with related software products, providing for the import of architectural models from various technical drawing software, or export to various platforms and file formats. SAFE, the floor and foundation slab design software with post-tensioning (PT) capability, is one such option for export. CSI coordinated SAFE to be used in conjunction with ETABS such that engineers could more thoroughly detail, analyze, and design the individual levels of an ETABS model.

While ETABS features a variety of sophisticated capabilities, the software is equally useful for designing basic systems. ETABS is the practical choice for all grid-like applications ranging from simple 2D frames to the most complex high rises.

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analyses that ETABS can handle easily:

a) Multi-story commercial, government and health care facilities
b) Parking garages with circular and linear ramps.
c) Buildings with curved beams, walls and floor edges.
d) Buildings with steel, concrete, composite or joist floor framing.
e) Complex shear walls and cores with arbitrary openings.
f) Performance based design utilizing nonlinear dynamic analysis.
g) Buildings based on multiple rectangular and/or cylindrical grid systems
h) Buildings subjected to any number of vertical and lateral load cases and combinations, including automated wind and seismic loads.
i) Multiple response spectrum load cases, with built-in input curves.
j) Automated transfer of vertical loads on floors to beams and walls.
k) P-Delta analysis with static or dynamic analysis.
l) Large displacement analysis.
m) Design optimization for steel and concrete frames.
n) Floor modelling with rigid or semi-rigid diaphragms.
o) The column, beam and brace elements may be non-prismatic, and they may have partial fixity at their end connections

Etc…

4.4. PERFORMANCE OF BUILDINGS UNDER DESIGN LOADS

Although wind and seismic forces both are basically dynamic, there is a fundamental difference in the manner in which they are induced in a structure. Wind loads, applied as external loads, are characteristically proportional to the exposed surface of a structure, this is force type loading. While the earthquake forces, principally internal forces, resulting due to distortion, produced by the inertial resistance of the structure to earthquake motions which is random motion of ground. So this is displacement type force. After the comparison, one can be able to come to a conclusion that which structure should be constructed under different conditions.

4.4.1. NATURAL TIME PERIOD:

Natural Period T_n of a building is the time taken by it to undergo one complete cycle of oscillation. It is an inherent property of a building controlled by its total mass m and stiffness k. These three quantities are related by

$$T_n = 2\pi\sqrt{\frac{m}{k}}$$

Its units are seconds (s). Thus, buildings that are heavy (with larger mass m) and flexible (with smaller stiffness k) have larger natural period than light and stiff buildings. Buildings oscillate by translating along X, Y or Z directions, or by rotating about X, Y or Z axes, or by a combination of the above. The reciprocal ($1/T_n$) of natural period of a building is called the Natural Frequency f_n ; its unit is Hertz (Hz). The building offers least resistance when shaken at its natural frequency (or natural period). Hence, it undergoes larger oscillation when shaken at its natural frequency than at other frequencies

Factors influencing Natural Period are

1. Effect of Stiffness
2. Effect of Mass
3. Effect of Building Height
4. Effect of Column Orientation
5. Effect of Unreinforced Masonry Infill Walls in RC Frames
6. Effect of Cracked Sections on Analysis of RC Frames

4.4.2. WIND PERFORMANCE:

Wind pressure on a building surface depends primarily on its velocity, the shape and surface structure of the building. Wind load has been an aspect in the design of lateral force resisting system, with added significance as the height of the building increased. As wind hits the structure and flows around it as shown in figure 5.1, several effects are possible. Pressure on the windward face and suction on the leeward face creates drag forces. Unsymmetrical flow around the structure can create lift forces. Air turbulence around the leeward corners and edges can create vortices, which are high-velocity air currents that create circular updrafts and suction streams adjacent to the building. Periodic shedding of vortices causes the building to oscillate in a direction transverse to the direction of the wind and may result in unacceptable accelerations at the upper floors of tall buildings. All buildings sway during windstorms, but the motion in old tall buildings with full-height heavy partitions has usually been undetectable and, therefore, has not been a cause for concern. Structural innovations coupled with lightweight construction. This have reduced the stiffness, mass, and damping characteristics of modern buildings. In these buildings, objects may vibrate. If the building has a twisting action, its occupants may get an illusory sense that the world outside is moving, creating symptoms of vertigo and disorientation. It is generally agreed that acceleration response that includes the effects of torsion at the top floors of a tall building, is the best standard for evaluation of motion perception. Experience a constant flow of wind is not possible, but intermittently sudden gusts of rushing wind may also be experienced.

GUEST EFFECT:

The wind velocity at any locality varies considerably with time. In addition to a steady wind there are effects of gust which last only for few seconds. The modern trend is to adopt peak gust loading as the basis for design, in place of mean load average over one minute.

$$\text{Gust factor (G)} = \text{Peak Load} / \text{Mean Load}$$

This sudden variation in wind speed, called gustiness or turbulence, is an important factor in determining dynamic response of tall buildings. Aside from the effects of uplift forces on large roof areas, flow of wind is considered two-dimensional, consisting of along wind and transverse wind. The term along wind or simply wind is used to refer to drag forces while transverse wind is the term used to describe crosswind as depicted in figure 5.2 and 5.3.Generally, in tall building design, the crosswind motion perpendicular to the direction of wind is often more critical than along-wind motion. Gusty wind velocities change rapidly and even abruptly, creating effects much larger than if the same loads were static. Wind loads, therefore, need to be studied as if they were dynamic, to some extent similar as seismic loads. The intensity of dynamic load depends on how fast the velocity varies as well as on the response of the structure itself. Therefore, whether pressures on a building due to wind gust, is dynamic or static entirely depends on the gustiness of wind and the dynamic properties of the building to which it is applied. Wind has been implemented in model and its consequences have been judged based on relative stipulations IS 875 (Part-3): 2015

EFFECT OF WIND ON TALL STRUCTURE:

Most of structures stand bluff (perpendicular) and wind blows horizontally as the structure are not generally made streamline or smooth wind flow over this obstructions tends to be turbulent. In addition the velocity of wind itself varies with time. The stiffness of structure also varies from structure to structure. When a bluff body is exposed to wind, vortices tend to be shed from the side of the body crating a pattern in its way called “Vortex shedding” as shown in Fig 4.2 and Fig 4.3.

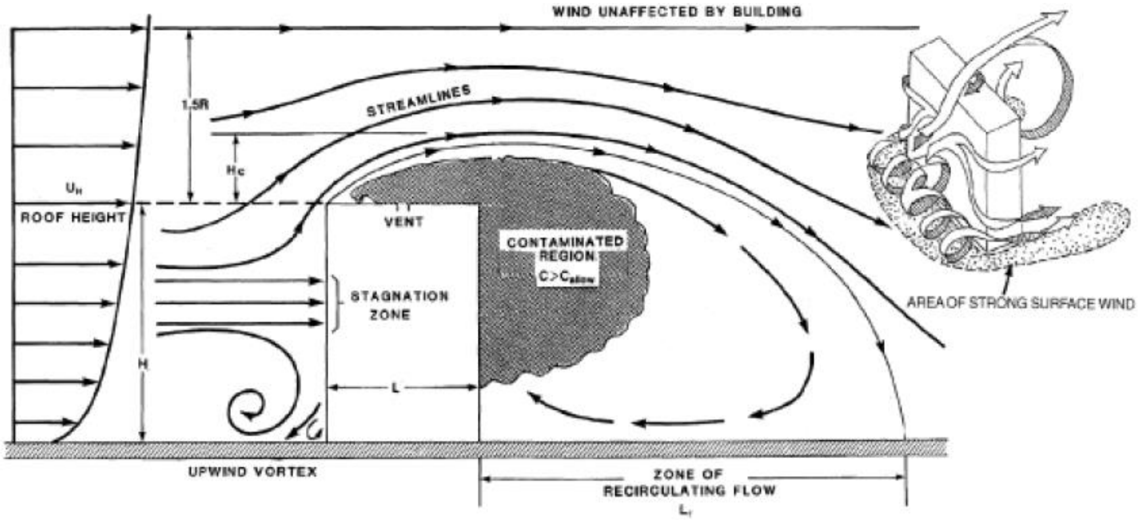


Fig 4.1 Flow Pattern around Rectangular Building

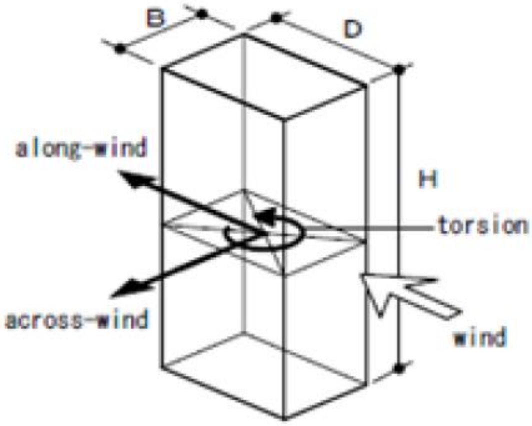


Fig 4.2 Type of Movement of Building due to Application of Wind

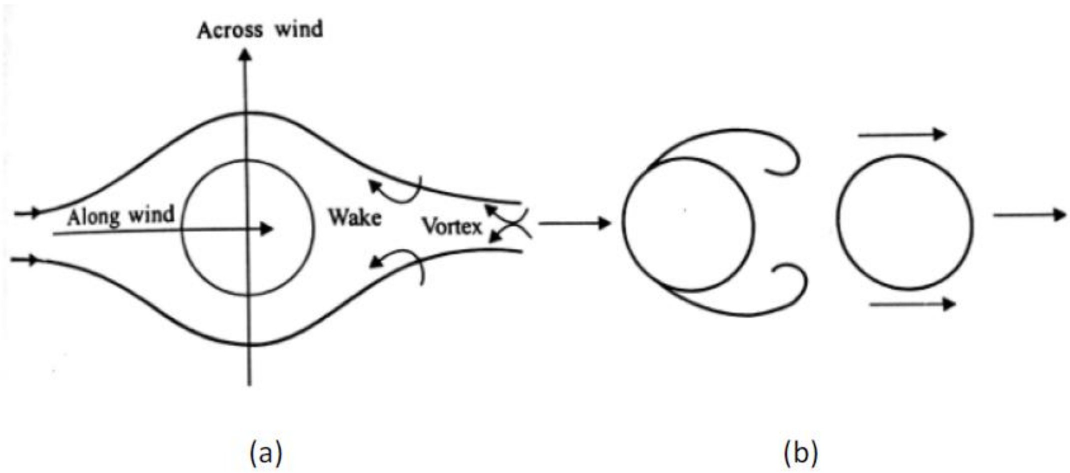


Fig 4.3 Wind Effect on Tall Structure (a) Vortex Shedding (b) Interference

The structure must have a system to resist shear as well as bending as shown in Fig 4.4. In resisting shear forces, they must not break by shearing off and in general must not strain beyond the limit of elastic recovery.

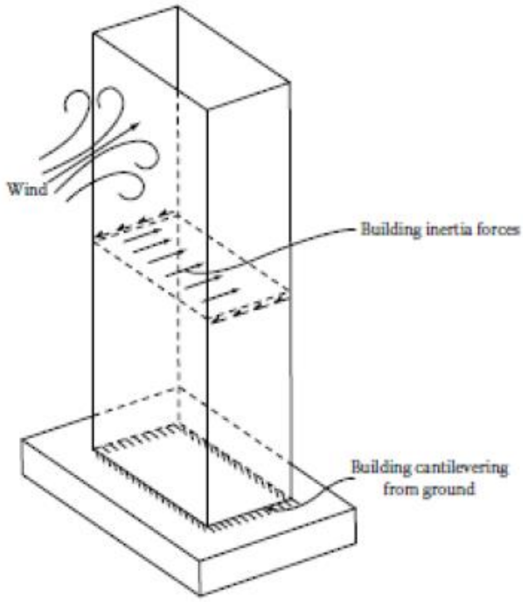


Fig 4.4: Wind Effect on Tall Structure

4.4.3. SEISMIC PERFORMANCE:

The behaviour of a building during an earthquake is a vibration concern. The seismic motions of the ground do not damage a building by impact, or by externally applied pressure such as wind. Building is damaged by internally generated inertial forces due to the vibration of the building mass. So an increase in mass has adverse effects on the earthquake design. In general, tall buildings respond to seismic motion differently than low-rise buildings. The magnitude of inertia forces induced in an earthquake depends on the building mass, ground acceleration, the nature of the foundation, and the dynamic characteristics of the structure .If a building and its foundation were infinitely rigid, it would have the same acceleration as the ground, resulting in an inertia force F = ma, for a given ground acceleration, a. However the force tends to be less than the product of buildings mass and acceleration because buildings have certain flexibility. Tall buildings are invariably more flexible than low-rise buildings and generally they experience much lower accelerations than low-rise buildings. But a flexible building subjected to ground motions for an extended period, may experience much larger forces if its natural period is near that of the ground waves. Thus, the magnitude of lateral force is not only function of the acceleration of the ground alone. It is influenced to a great extent by the type of response of the structure itself as well as type of its foundation.

Dynamic analysis is very essential for tall building to understand significant response characteristics, such as

(1) The effects of structure's dynamic characteristics on the vertical distribution of lateral forces.

(2) The increase in dynamic loads due to torsional motions; and

(3) The influence of higher modes, resulting in an increase in story shears and deformations.

With uniform mass and stiffness distribution, symmetrical buildings behave in a foreseeable manner whereas buildings that are asymmetrical or with areas of discontinuity or irregularity do not. Static methods specified in building codes are based on single-mode response which is though acceptable for small simple building, not for complex structure. The simplified procedures do not take into account the full range of seismic behaviour of complex structures. Therefore, dynamic analysis is the favoured technique for the design of buildings with unfamiliar or irregular geometry.

A large amount of the study work has been performed in the direction on Seismic behaviour for dynamic performance. Critical issues related with seismic behaviour are storey drift, Base shear, deformation and Mass irregularity. Structural engineer have to deal with all these critical issues to deliver safe structure under the seismic effect. Seismic has been implemented in model and its effect has been judged based on code IS 1893-2016. Fundamental translational natural period of oscillation (in second) for different structures are evaluated by using formulas as follows.

$T_a = 0.075h^{0.75}$ (RC bare MRF building)

$T_a = 0.08h^{0.75}$ (RC-steel composite bare MRF building)

$T_a = 0.08h^{0.75}$ (Steel bare MRF building)

$T_a = 0.075h^{0.75} / (A_w)^{0.5} \geq 0.09h / d^{0.5}$ (Shear wall and dual system building)

A_w is total effective area (m^2) of wall in 1st floor of building.

$A_w = \sum [A_{wi}\{0.2+(L_{wi}/h)^2\}]$ for $(1 \leq i \leq N)$

A_{wi}= Cross Sectional area in SQM at 1st floor level

L_{wi}= Total length of structural wall in m at 1st floor level in the direction of lateral force.

L_{wi}= Base dimension of building in m at plinth level along the direction of earthquake force.

N_w = Number of structural wall in the direction of earthquake force.

Design approach for a building under natural action of (a) Seismic, ground movement at base and (b) Wind pressure applicable on building is explained in Fig 4.5

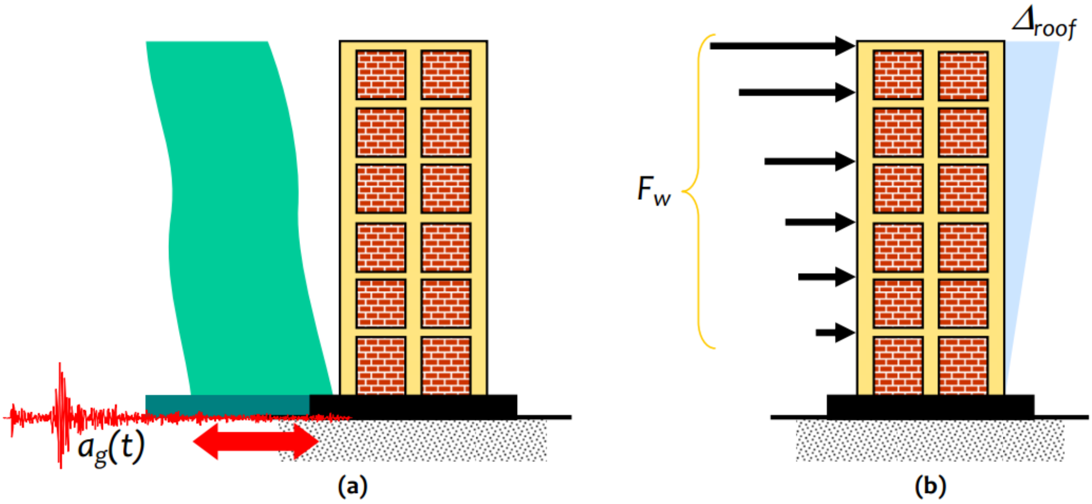


Fig 4.5 Ground Movement of Building Due to (a) Seismic Effect (b) Wind Effect

4.4.4. CONCERNED FEATURES:

For both of lateral loads, seismic and wind, following concerns must be taken into account with its significance for tall building or structures.

I. STOREY DRIFT

Sway or drift is the magnitude of the relative lateral displacement between a given floor and the one immediately below it. Storey drift of any structure depend on stiffness of structure. Higher stiffness of structure or building causes less storey drift. As the height increases, the forces of nature particularly due to the wind begin to dominate. Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Stress or strength limitations in ductile materials do not always provide adequate drift control, especially for tall buildings with relatively flexible moment-resisting frames or narrow shear walls. Total building drift is the absolute displacement of any point relative to the base. Adjoining buildings or adjoining sections of the same building may not have identical modes of response in earthquake, and therefore may have a tendency to pound against one another. Building separations or joints must be provided to permit adjoining buildings to respond independently to earthquake ground motion.

II. DEFORMATION:

Earthquake-induced motions, even when they are more forceful than those induced by wind, induce a totally different human response

1. As because earthquakes occur much less frequently than windstorms,
2. As because the duration of motion generated by an earthquake is generally short.

Lateral deflections that occur during earthquakes should be limited to prevent distress in structural members and architectural components. Non load-bearing in-fills, external wall panels, and window glazing should be designed with sufficient clearance or with flexible supports to accommodate the predicted movements.

Table 4.1: Sway Criteria of Building Structures

Loading	Deflection Limits	
	Overall	Story drift
Wind loads	H/500	_
Seismic loads	H/250	h/250

H = Total Building Height
h = Floor to Floor Height

III. BASE SHEAR AND RESULTANT OVERTURNING MOMENT:

Earthquake shaking is random and time variant. But, most design codes represent the earthquake-induced inertia forces as the net effect of such random shaking in the form of design equivalent static lateral force. This force is called as the Seismic Design Base Shear V_B and remains the primary quantity used for force-based earthquake-resistant design of buildings. This force depends on the seismic hazard at the site of the building represented by the Seismic Zone Factor Z. Also, in keeping in mind the philosophy of increasing design forces to increase the elastic range of the building and thereby reduce the damage in it.

A building is considered tall, when, it is governed by the lateral forces acting on it. The fact is that lateral forces in the building are greatest at the base of the building. The lateral force at the base of the building is called the base shear. If **ΣF** denotes the overall force acting on the base of a building due to the earthquake or wind and the forces acting on the different floors on the building i.e. F_1, F_2 etc. are the storey shear that simply means the lateral forces because of the earthquake or wind at different floors. The base shear is equal to the sum of all the storey shear forces at different floors. Lateral shear and external shear are local terms synonyms to storey shear.

Over turning moment is at the base of structure caused by the force developed or may be applied at all storey level. This will has the tendency to overturn the structure .The effect will be sever for tall slender building if it was not judge properly and foundation system is not capable to take pull or tension induced by the moment.

4.5. INTENT OF DOCUMENTS:

The intent of this document is to identify and record of all the relevant input requirements, analysis for different structural system as mentioned above of the Residential complex located in Kolkata, West Bengal. The proposed scheme is compatible with the architectural aspects and the basic structural arrangement provided satisfying the functional needs, at the same time conforming to the Indian Standards and other applicable building norms to achieve safe, stable, strong, durable and yet optimally economical structures.

The parameters adopted in this report are going to be the basis of the structural analysis as per IS codes and other relevant codes.

The analysis will aim to achieve

1. Structural and functional integrity.
2. Desirable structural performance under characteristic service design loads such as dead loads, super-imposed dead loads and live loads.
3. Resistance to loads due to natural phenomena i.e. wind, earthquakes and temperature.
4. Structural strength, stiffness and stability.
5. Structural durability and performance ability.

4.5.1. BUILDING DATA:

Table 4.2 Building Data

MKD	Building Height above ground level to roof level (m)	Building Height above ground level to roof level (m)	Floor to floor Height (m)		Built-up Area (m^2)
Building G12 (G+12)	Structural wall + Moment Frame	40.45	Typical Floor	3.05	8686
			Ground Floor	3.85	
Building G24 (G+24)	Structural Wall System	88.10	Typical Floor	3.50	10070
			Ground Floor	4.10	
Building G30 (G+30)	Structural Wall System	109.10	Typical Floor	3.50	12588
			Ground Floor	4.10	
Building G36 (G+36)	Structural Wall System	130.10	Typical Floor	3.50	15106
			Ground Floor	4.10	

4.5.2. LOADING PARAMETERS:

Self –weight of the structural members will be considered as follows.

Table 4.3 Loading Parameters

Material Property	Density	Source
Soil	1.80 T/ m^3	IS: 875 Part -I
Water	1.00 T/ m^3	IS: 875 Part -I
Plain cement concrete	2.40 T/ m^3	IS: 875 Part -I
Reinforced cement concrete	2.50 T/ m^3	IS: 875 Part -I
Floor Finish	2.40 T/ m^3	IS: 875 Part -I
Plaster	2.10 T/ m^3	IS: 875 Part -I
AAC- Block Masonry	1.0-0.8 T/ m^3	Manufacturer Specifications

4.5.3. BUILDING PLANS:

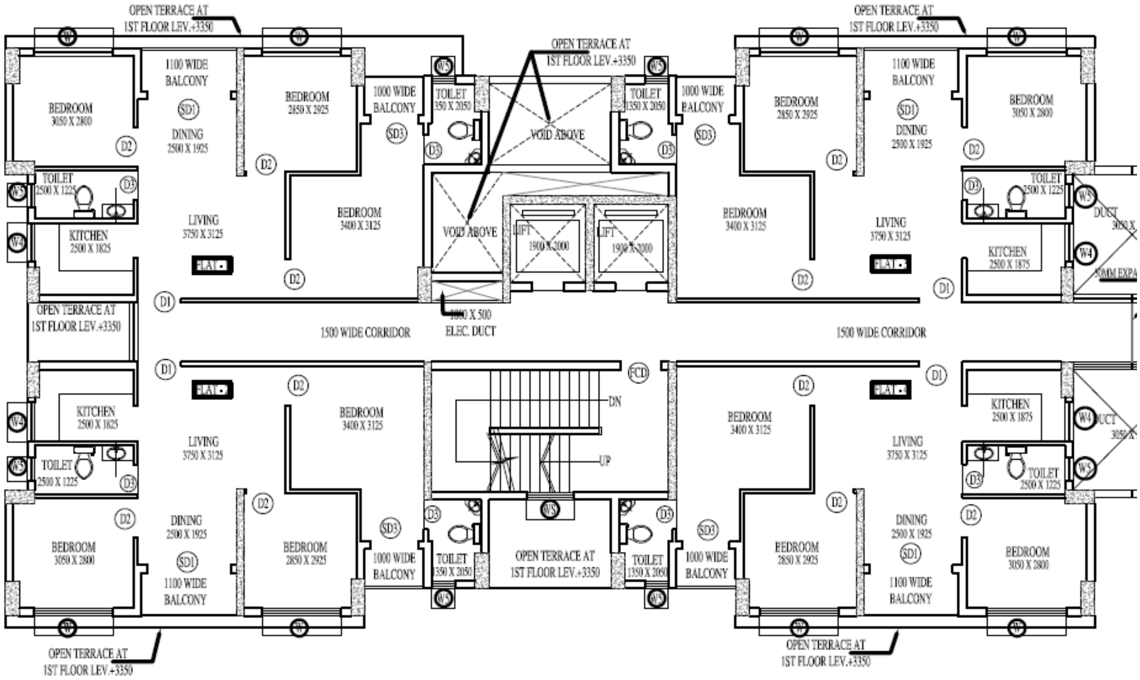


Fig 4.6 Architectural Plan of Typical Floor of G+12 Storied Buildings

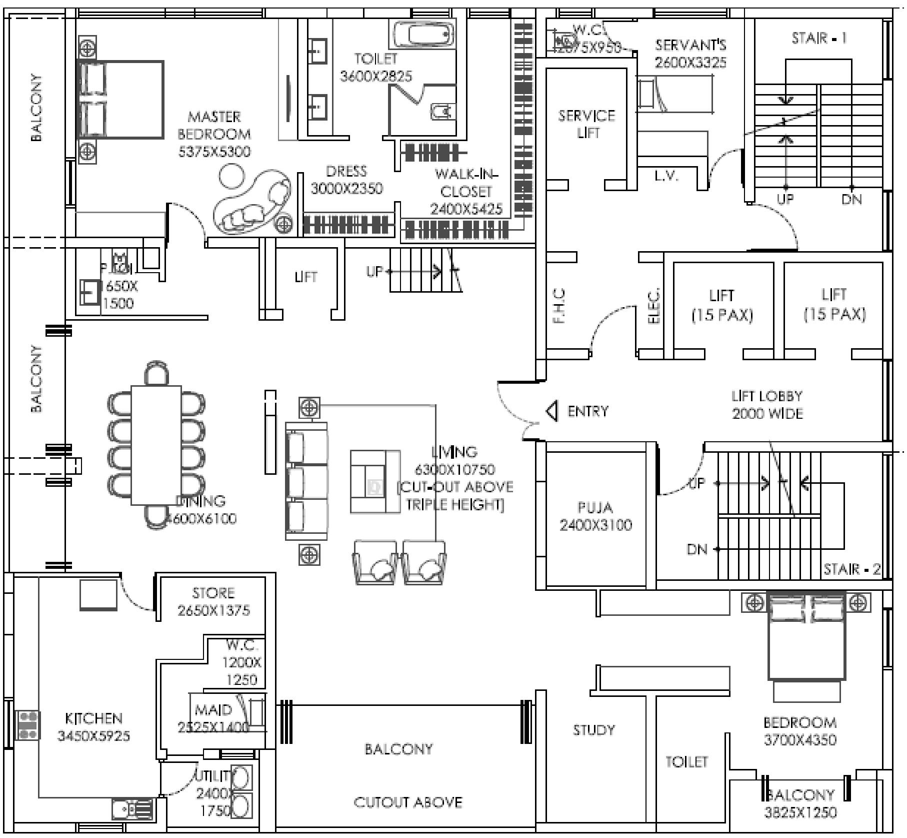


Fig 4.7 Architectural Plan of Typical Floor of G+24, G+30 and G+36 Storied Buildings

4.6. ANALYSIS APPROACH

4.6.1. GENERAL

Analysis and study of the structural behaviour seems to be essential prior to achieving more conclusive correlation. In this proposed study two types of mid-rise buildings and six type of high-rise buildings, situated at Kolkata, are modelled with incremental height difference using both the codes IS 1893 (Part-I): 2002 and IS 1893 (Part-I): 2016 conjugate with IS 16700: 2017 and compared them with their output results.. The nomenclature of the structural System considered for the numerical study are shown in figure 4.8 below

1. A comparative study of G+12 storied mid-rise buildings

- **G12-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
- **G12-2016** [Analysis as per IS 1893 (Part-1): 2016 and IS 875 (Part-3): 2015]

2. A comparative study of G+24, G+30 and G+36 Storied High-rise buildings
   1. Comparative study of G+24 storied buildings
   - **G24-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
   - **G24-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]
   2. Comparative study of G+30 storied buildings
   - **G30-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
   - **G30-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]
   3. Comparative study of G+36 storied buildings
   - **G36-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
   - **G36-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]

4.6.2. BUILDING DATA

1. G+12 Storey Mid-rise buildings

- Type of Building: G+12 Storied Residential Building
- Geometry: Length= 57.100m,Width= 14.400m, Height=40.45m
- Structural Arrangement: Ductile RC Wall with SMRFs

- Material Properties: Slabs= M25, Beams= M25, Column and Shear Wall= M40 (G-2nd Floor), M30 (2nd F- 7th F) and M25 (7th F-Roof)
- Member Property: Slab thickness= 100mm, 115mm, 175mm (Stair Slab), Beam Sizes= (250X600) and (200X600), Column Sizes (300X900), Wall Thickness= 200mm

2. A. G+24 Storied High-rise buildings:

- Type of Building: G+24 Storied Residential Building
- Geometry: Length= 20.3m,Width= 19.70m, Height=88.10m
- Structural Arrangement: Ductile RC Wall System
- Material: Slabs= M30, Beams= M30, Column and Shear Wall= M50 (G-8th Floor), M40 (8th F- 16th F) and M30 (16th F-Roof)
- Member Property: Slab thickness= 125mm, 175mm, 200mm, 250mm, Beam Size= (300X600). Wall Thickness= 300mm, 450mm, 500mm and 600mm

2. B. G+30 Storied High-rise buildings:

- Type of Building: G+30 Storied Residential Building
- Geometry: Length= 20.3m,Width= 19.70m, Height=109.10m
- Structural Arrangement: Ductile RC Wall System
- Material: Slabs= M30, Beams= M30, Column and Shear Wall= M50 (G-10th Floor), M40 (10th F- 20th F) and M30 (21st F-Roof)
- Member Property: Slab thickness= 125mm, 175mm, 200mm, 250mm, Beam Size= (300X600). Wall Thickness= 300mm, 450mm, 500mm and 600mm

2. C. G+36 Storied High-rise buildings:

- Type of Building: G+36 Storied Residential Building
- Geometry: Length= 20.3m,Width= 19.70m, Height=130.10m
- Structural Arrangement: Ductile RC Wall System
- Material: Slabs= M30, Beams= M30, Column and Shear Wall= M50 (G-12th Floor), M40 (12th F- 24th F) and M30 (24th F-Roof)
- Member Property: Slab thickness= 125mm, 175mm, 200mm, 250mm, Beam Size= (300X600). Wall Thickness= 300mm, 450mm, 500mm and 600mm

4.6.3. LINE LOAD SCHEDULE OF G+12 STORIED BUILDINGS:

SCHEDULE OF LINE LOADS

Please see the coloured scheme on the drawing sheet for better clarity

(A)	200 mm Thk. AAC Wall on Beam [ETABS input]		(600 mm Depth)					
LEDGEND	200	thk solid wall	3.05	1.0	t/m^3	=	0.49	T/m.
	30	thk plaster	3.05	2.1	t/m^3	=	0.19	T/m.
		For solid wall					0.68	T/m.
		80% of solid wall					0.55	T/m.

(B)	100 mm Thk. AAC Wall on Beam [ETABS input]		(600 mm Depth)					
LEDGEND	100	thk solid wall	3.05	1.0	t/m^3	=	0.25	T/m.
	30	thk plaster	3.05	2.1	t/m^3	=	0.19	T/m.
		For solid wall					0.44	T/m.
		90% of solid wall					0.39	T/m.

(C)	200 mm Thk. AAC Wall on Slab [ETABS input]		(110 mm Depth)					
LEDGEND	200	thk solid wall	3.05	1.0	t/m^3	=	0.59	T/m.
	30	thk plaster	3.05	2.1	t/m^3	=	0.19	T/m.
		For solid wall					0.78	T/m.
		60% of solid wall					0.47	T/m.

(D)	100 mm Thk. AAC Wall on Slab [ETABS input]		(115 mm Depth)					
LEDGEND	100	thk solid wall	3.05	1.0	t/m^3	=	0.29	T/m.
	30	thk plaster	3.05	2.1	t/m^3	=	0.19	T/m.
		For solid wall					0.49	T/m.
		90% of solid wall					0.44	T/m.

(E)	200mm Thk. AAC Parapet Wall [ETABS input]		(000 mm Depth)					
LEDGEND	200	thk solid wall	1.50	1.0	t/m^3	=	0.30	T/m.
	30	thk plaster	1.50	2.1	t/m^3	=	0.09	T/m.
		For solid wall					0.39	T/m.

(F)	200 mm Thk. AAC Wall on Beam [ETABS input]		(600 mm Depth)					
LEDGEND	200	thk solid wall	2.60	1.0	t/m^3	=	0.40	T/m.
	30	thk plaster	2.60	2.1	t/m^3	=	0.16	T/m.
		For solid wall					0.56	T/m.

(G)	MS Railing on Beam [ETABS input]							
LEDGEND		MS Railing				=	0.15	T/m.
		Load on Beam					0.15	T/m.

Beam/Slab depth has been deducted from floor height

4.6.4. FLOOR LOAD SCHEDULE OF G+12 STORIED BUILDINGS:

Imposed Gravity loads on floors

Imposed loads will be considered on the basis of IS: 875 Part-2

SCHEDULE OF SUPER DEAD LOADS

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

A .	ROOF OR TERRACE LOAD :-								REMARKS
	a)	100	thk roof treatment @	2200	kg/m^3	=	0.220	T/m^2	
	b)	6	thk ceiling plaster @	2100	kg/m^3	=	0.013	T/m^2	
			SUPERDEAD :-			=	0.233	T/m^2	[ETABS input]

B .	TYPICAL FLOOR LOAD :-								REMARKS
LEDGEND	a)	50	thk Finish @	2400	kg/m^3	=	0.120	T/m^2	
	b)	100	thk filling (brick bat coba) @	1000	kg/m^3	=	0.100	T/m^2	
	c)	6	thk Ceiling Plaster @	2100	kg/m^3	=	0.013	T/m^2	
			SUPERDEAD (Utility/Toilet) :-			=	0.233	T/m^2	[ETABS input]
			SUPERDEAD (ELSE) :-				0.133	T/m^2	[ETABS input]

C.	STAIRCASE LOAD :								REMARKS
			Rise :	145.2					
			Tread :	250					
			Approx slope of waist slab :	30.00 °					
LEDGEND	A. SUPER DEAD LOAD:-								
	a)	6	thk Ceiling Plaster @	2100	kg/m^3	=	0.020	T/m^2	
	b)	50	thk Floor Finish @	2400	kg/m^3	=	0.190	T/m^2	
	c)		Concrete Steps @	2400	kg/m^3	=	0.174	T/m^2	
			TOTAL :-			=	0.4	T/m^2	[ETABS input]

SCHEDULE OF LIVE LOADS

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

LEDGEND	A. 1ST FLOOR LOAD :-				[ETABS input]		SOURCE
	Live Load For All Rooms, Kitchens,Toilet, Bathrooms etc @	200	kg/m^2	=	0.200	T/m^2	IS:875 (PART-II)
	Live Load For Coridoor, Stair Cases, Balconies, Fire Refuge Balconies etc @	300	kg/m^2	=	0.300	T/m^2	IS:875 (PART-II)

SCHEDULE OF LIVE LOADS ON ROOF

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

LEDGEND	A. ROOF LOAD :-				[ETABS input]		SOURCE
	Live Load For Accessible Roof etc @	150	kg/m^2	=	0.150	T/m^2	IS:875 (PART-II)
	Live Load For Non Accessible Roof etc @	75	kg/m^2	=	0.075	T/m^2	IS:875 (PART-II)
	Live Load For Lift Machine Room etc @	500	kg/m^2	=	0.500	T/m^2	IS:875 (PART-II)

4.6.5. LINE LOAD SCHEDULE OF G+24, G+30 AND G+36 STORIED BUILDINGS:

SCHEDULE OF LINE LOADS

Please see the coloured scheme on the drawing sheet for better clarity

(A)	250 mm Thk. AAC Wall on Beam [ETABS input]		(600 mm Depth)					
LEDGEND	250	thk solid wall	3.50	0.8	t/m^3	=	0.58	T/m.
	30	thk plaster	3.50	2.1	t/m^3	=	0.22	T/m.
		For solid wall					0.80	T/m.
		80% of solid wall					0.64	T/m.

(B)	125 mm Thk. AAC Wall on Beam [ETABS input]		(600 mm Depth)					
LEDGEND	125	thk solid wall	3.50	0.8	t/m^3	=	0.29	T/m.
	30	thk plaster	3.50	2.1	t/m^3	=	0.22	T/m.
		For solid wall					0.51	T/m.
		90% of solid wall					0.46	T/m.

(C)	250 mm Thk.AAC Wall on Slab [ETABS input]		(175 mm Depth)					
LEDGEND	250	thk solid wall	3.50	0.8	t/m^3	=	0.67	T/m.
	30	thk plaster	3.50	2.1	t/m^3	=	0.22	T/m.
		For solid wall					0.89	T/m.
		80% of solid wall					0.71	T/m.

(D)	125 mm Thk.AAC Wall on Slab [ETABS input]		(175 mm Depth)					
LEDGEND	125	thk solid wall	3.50	0.8	t/m^3	=	0.33	T/m.
	30	thk plaster	3.50	2.1	t/m^3	=	0.22	T/m.
		For solid wall					0.55	T/m.

(E)	MS Railing on Beam [ETABS input]							
LEDGEND		MS Railing				=	0.15	T/m.
		Load on Beam					0.15	T/m.

Beam/Slab depth has been deducted from floor height

4.6.6. FLOOR LOAD SCHEDULE OF G+24, G+30 AND G+36 STORIED BUILDINGS:

Imposed Gravity loads on floors

Imposed loads will be considered on the basis of IS: 875 Part-2

Dead & Live Load pressure for slab is as follows:

SCHEDULE OF SUPER DEAD LOADS

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

A .	ROOF OR TERRACE LOAD :-								REMARKS
	a)	100	thk roof treatment @	2200	kg/m^3	=	0.220	T/m^2	
	b)	6	thk ceiling plaster @	2100	kg/m^3	=	0.013	T/m^2	
	c)	_	Services @	25	kg/m^3	=	0.025	T/m^2	
			SUPERDEAD :-			=	0.258	T/m^2	[ETABS input]

B .	TYPICAL FLOOR LOAD :-								REMARKS
LEDGEND	a)	50	thk Finish @	2400	kg/m^3	=	0.120	T/m^2	
	b)	50	thk Waterprofing @	2400	kg/m^3	=	0.120	T/m^2	
	c)	6	thk Ceiling Plaster @	2100	kg/m^3	=	0.013	T/m^2	
	d)	_	Services @	25	kg/m^3	=	0.025	T/m^2	
			SUPERDEAD (Utility/Toilet) :-			=	0.278	T/m^2	[ETABS input]
			SUPERDEAD (ELSE) :-				0.158	T/m^2	[ETABS input]

C.	STAIRCASE LOAD :								REMARKS
			Rise :	159					
			Tread :	250					
			Approx slope of waist slab :	33.00 °					
LEDGEND	A. SUPER DEAD LOAD:-								
	a)	6	thk Ceiling Plaster @	2100	kg/m^3	=	0.021	T/m^2	
	b)	50	thk Floor Finish @	2400	kg/m^3	=	0.196	T/m^2	
	c)		Concrete Steps @	2500	kg/m^3	=	0.199	T/m^2	
			TOTAL :-			=	0.42	T/m^2	[ETABS input]

SCHEDULE OF LIVE LOADS

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

LEDGEND	A. 1ST FLOOR LOAD :-		[ETABS input]				SOURCE
	Live Load For All Rooms, Kitchens,Toilet, Bathrooms etc @	200	kg/m^2	=	0.200	T/m^2	IS:875 (PART-II)
	Live Load For Corridoor, Stair Cases, Balconies etc @	300	kg/m^2	=	0.300	T/m^2	IS:875 (PART-II)

SCHEDULE OF LIVE LOADS ON ROOF

PLEASE SEE THE COLOURED SCHEME ATTACHED FOR BETTER CLARITY

LEDGEND	A. ROOF LOAD :-		[ETABS input]				SOURCE
	Live Load For Accessible Roof etc @	150	kg/m^2	=	0.150	T/m^2	IS:875 (PART-II)
	Live Load For Non Accessible Roof etc @	75	kg/m^2	=	0.075	T/m^2	IS:875 (PART-II)

4.6.7. SEISMIC LOAD OF G+12 STORIED BUILDINGS:

Zone= III; Zone Factor, Z= 0.16

R= Response reduction Factor= 4 [As per Table-9 of IS 1893 (Part-I): 2016]

I= Importance Factor= 1.2 [As per Table-8 of IS 1893 (Part-I): 2016]

Soil Type= II (Medium Soil) [As per Geo-technical Investigation Report]

Damping= (5%)= 0.05

Time Period= As Calculated

The Approximate Fundamental Translational Natural Period T_a of Oscillation (Buildings with RC Structural Wall):
(As per IS:1893-2016 Cl. No.-7.6.2.a)

$$T_a = 0.075h^{0.75} / (A_w)^{0.5} \geq 0.09h / d^{0.5}$$

Where, A_w is the total effective area (m^2) of walls in the first storey of the building given by

$$A_w = \sum [A_{wi}\{0.2+(L_{wi}/h)^2\}] \text{ for } (1 \leq i \leq N)$$

Where, h = height of Building as defined in 7.6.2 (a) of IS: 1893-2016, in m

A_{wi} = effective cross-sectional area of wall i in first storey of building, in m^2

L_{wi} = length of structural wall i in first storey in the considered direction of lateral forces, in m

d = base dimention of the building at the plinth level along the considered direction of earthquake shaking, in m

N_w = number of walls in the considered direction of earthquake shaking.

Note: The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

Building A-1B:

h = Height of the building	h =	40.450 m
dx = Base Dimension of the building in X- Direction	dx =	57.100 m
dy = Base Dimension of the building in Y- Direction	dy =	14.400 m

A. Wall Dimensions (X-Direction):

	No.	Length (m)	Thk. (m)
A_{xw1} =	8	0.750 m	0.300 m
A_{xw2} =	4	1.700 m	0.200 m
A_{xw3} =	1	4.400 m	0.200 m
A_{xw4} =	1	3.000 m	0.200 m

B. Wall Dimensions (Y-Direction):

	No.	Length (m)	Thk. (m)
A_{yw1} =	8	3.200 m	0.200 m
A_{yw2} =	6	3.500 m	0.200 m
A_{yw3} =	1	2.700 m	0.200 m
A_{yw4} =	2	4.500 m	0.200 m

Fundamental Translational Natural Period "X" Direction (T_{ax}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wx} = **0.945 m²**

Calculated approximate fundamental translational natural period T_{ax}:

T_{ax} = 1.238 sec

Fundamental Translational Natural Period "Y" Direction (T_{ay}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wy} = **2.420 m²**

Calculated approximate fundamental translational natural period T_{ay}:

T_{ay} = 0.959 sec

4.6.8. SEISMIC LOADING OF G+24, G+30 AND G+36 STORIED BUILDINGS:

1. G+24 Storied Building:

Zone= III; Zone Factor, Z= 0.16

R= Response reduction Factor= 4 [As per Table-9 of IS 1893 (Part-I) : 2016]

I= Importance Factor= 1.2 [As per Table-8 of IS 1893 (Part-I) : 2016]

Soil Type= II (Medium Soil) [As per Geo-technical Investigation Report]

Damping= (5%)= 0.05

Time Period= As Calculated

2. G+30 Storied Building:

Zone= III; Zone Factor, Z= 0.16

R= Response reduction Factor= 4 [As per Table-9 of IS 1893 (Part-I) : 2016]

I= Importance Factor= 1.2 [As per Table-8 of IS 1893 (Part-I) : 2016]

Soil Type= II (Medium Soil) [As per Geo-technical Investigation Report]

Damping= (5%)= 0.05

Time Period= As Calculated

3. G+36 Storied Building:

Zone= III; Zone Factor, Z= 0.16

R= Response reduction Factor= 4 [As per Table-9 of IS 1893 (Part-I) : 2016]

I= Importance Factor= 1.2 [As per Table-8 of IS 1893 (Part-I) : 2016]

Soil Type= II (Medium Soil) [As per Geo-technical Investigation Report]

Damping= (5%)= 0.05

Time Period= As Calculated

A. Calculation of Fundamental Time Period for G+24 Storied Building:

The Approximate Fundamental Translational Natural Period T_a of Oscillation (Buildings with RC Structural Wall):
(As per IS:1893-2016 Cl. No.-7.6.2.a)

$$T_a = 0.075h^{0.75} / (A_w)^{0.5} \geq 0.09h / d^{0.5}$$

Where, A_w is the total effective area (m^2) of walls in the first storey of the building given by

$$A_w = \sum [A_{wi}\{0.2+(L_{wi}/h)^2\}] \text{ for } (1 \leq i \leq N)$$

Where, h = height of Building as defined in 7.6.2 (a) of IS: 1893-2016, in m

A_{wi} = effective cross-sectional area of wall *i* in first storey of building, in m^2

L_{wi} = length of structural wall *i* in first storey in the considered direction of lateral forces, in m

d = base dimention of the building at the plinth level along the considered direction of earthquake shaking, in m

N_w = number of walls in the considered direction of earthquake shaking.

Note: The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

Building B-1B:

h = Height of the building	h =	*88.100 m*
dx = Base Dimension of the building in X- Direction	dx =	*21.300 m*
dy = Base Dimension of the building in Y- Direction	dy =	*19.700 m*

A. *Wall Dimensions (X-Direction):*

	No.	*Length (m)*	*Thk. (m)*
A_{xw1} =	1	2.700 m	0.300 m
A_{xw2} =	1	3.850 m	0.300 m
A_{xw3} =	1	2.200 m	0.300 m
A_{xw4} =	1	6.050 m	0.300 m
A_{xw5} =	1	0.750 m	0.300 m
A_{xw6} =	1	4.550 m	0.300 m
A_{xw7} =	1	5.750 m	0.300 m
A_{xw8} =	5	1.550 m	0.400 m
A_{xw9} =	1	1.975 m	0.400 m
A_{xw10} =	1	3.350 m	0.450 m

B. *Wall Dimensions (Y-Direction):*

	No.	*Length (m)*	*Thk. (m)*
A_{yw1} =	1	5.550 m	0.300 m
A_{yw2} =	1	3.075 m	0.300 m
A_{yw3} =	2	2.650 m	0.300 m
A_{yw4} =	1	3.350 m	0.300 m
A_{yw5} =	1	1.750 m	0.400 m
A_{yw6} =	1	0.700 m	0.400 m
A_{yw7} =	1	1.200 m	0.400 m
A_{yw8} =	2	4.550 m	0.400 m
A_{yw9} =	2	2.475 m	0.400 m
A_{yw10} =	1	2.825 m	0.450 m
A_{yw11} =	1	3.350 m	0.450 m
A_{yw12} =	1	2.000 m	0.450 m
A_{yw13} =	1	2.450 m	0.450 m
A_{yw14} =	1	1.875 m	0.600 m

Fundamental Translational Natural Period "X" Direction (T_{ax}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wx} = **2.657 m²**

Calculated approximate fundamental translational natural period T_{ax}:

T_{ax} = ***1.718 sec***

Fundamental Translational Natural Period "Y" Direction (T_{ay}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wy} = **3.661 m²**

Calculated approximate fundamental translational natural period T_{ay}:

T_{ay} = ***1.786 sec***

B. Calculation of Fundamental Time Period for G+30 Storied Building:

The Approximate Fundamental Translational Natural Period T_a of Oscillation (Buildings with RC Structural Wall):
(As per IS:1893-2016 Cl. No.-7.6.2.a)

$$T_a = 0.075h^{0.75} / (A_w)^{0.5} \geq 0.09h / d^{0.5}$$

Where, A_w is the total effective area (m^2) of walls in the first storey of the building given by

$$A_w = \sum [A_{wi}\{0.2+(L_{wi}/h)^2\}] \text{ for } (1 \leq i \leq N)$$

Where, h = height of Building as defined in 7.6.2 (a) of IS: 1893-2016, in m

A_{wi} = effective cross-sectional area of wall i in first storey of building, in m^2

L_{wi} = length of structural wall i in first storey in the considered direction of lateral forces, in m

d = base dimention of the building at the plinth level along the considered direction of earthquake shaking, in m

N_w = number of walls in the considered direction of earthquake shaking.

Note: The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

Building B-2B:

h = Height of the building	h =	109.100 m
dx = Base Dimension of the building in X- Direction	dx =	21.300 m
dy = Base Dimension of the building in Y- Direction	dy =	19.700 m

A. Wall Dimensions (X-Direction):

	No.	Length (m)	Thk. (m)
A_{xw1} =	1	2.700 m	0.300 m
A_{xw2} =	1	3.850 m	0.300 m
A_{xw3} =	1	2.200 m	0.300 m
A_{xw4} =	1	6.050 m	0.300 m
A_{xw5} =	1	0.750 m	0.300 m
A_{xw6} =	1	4.550 m	0.300 m
A_{xw7} =	1	5.750 m	0.300 m
A_{xw8} =	5	1.550 m	0.400 m
A_{xw9} =	1	1.975 m	0.400 m
A_{xw10} =	1	3.350 m	0.450 m

B. Wall Dimensions (Y-Direction):

	No.	Length (m)	Thk. (m)
A_{yw1} =	1	5.550 m	0.300 m
A_{yw2} =	1	3.075 m	0.300 m
A_{yw3} =	2	2.650 m	0.300 m
A_{yw4} =	1	3.350 m	0.300 m
A_{yw5} =	1	1.750 m	0.400 m
A_{yw6} =	1	0.700 m	0.400 m
A_{yw7} =	1	1.200 m	0.400 m
A_{yw8} =	2	4.550 m	0.400 m
A_{yw9} =	2	2.475 m	0.400 m
A_{yw10} =	1	2.825 m	0.450 m
A_{yw11} =	1	3.350 m	0.450 m
A_{yw12} =	1	2.000 m	0.450 m
A_{yw13} =	1	2.450 m	0.450 m
A_{yw14} =	1	1.875 m	0.600 m

Fundamental Translational Natural Period "X" Direction (T_{ax}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wx} = **2.648 m²**

Calculated approximate fundamental translational natural period T_{ax}:

T_{ax} = 2.128 sec

Fundamental Translational Natural Period "Y" Direction (T_{ay}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wy} = **3.652 m²**

Calculated approximate fundamental translational natural period T_{ay}:

T_{ay} = 2.212 sec

C. Calculation of Fundamental Time Period for G+36 Storied Building:

The Approximate Fundamental Translational Natural Period T_a of Oscillation (Buildings with RC Structural Wall):
(As per IS:1893-2016 Cl. No.-7.6.2.a)

$$T_a = 0.075h^{0.75} / (A_w)^{0.5} \geq 0.09h / d^{0.5}$$

Where, A_w is the total effective area (m^2) of walls in the first storey of the building given by

$$A_w = \sum [A_{wi}\{0.2+(L_{wi}/h)^2\}] \text{ for } (1 \leq i \leq N)$$

Where, h = height of Building as defined in 7.6.2 (a) of IS: 1893-2016, in m

A_{wi} = effective cross-sectional area of wall i in first storey of building, in m^2

L_{wi} = length of structural wall i in first storey in the considered direction of lateral forces, in m

d = base dimention of the building at the plinth level along the considered direction of earthquake shaking, in m

N_w = number of walls in the considered direction of earthquake shaking.

Note: The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

<u>Building B-3B:</u>

h = Height of the building	h =	130.100 m
dx = Base Dimension of the building in X- Direction	dx =	21.300 m
dy = Base Dimension of the building in Y- Direction	dy =	19.700 m

A. Wall Dimensions (X-Direction):

	No.	Length (m)	Thk. (m)
A_{xw1} =	1	2.700 m	0.300 m
A_{xw2} =	1	3.850 m	0.300 m
A_{xw3} =	1	2.200 m	0.300 m
A_{xw4} =	1	6.050 m	0.300 m
A_{xw5} =	1	0.750 m	0.300 m
A_{xw6} =	1	4.550 m	0.300 m
A_{xw7} =	1	5.750 m	0.300 m
A_{xw8} =	5	1.550 m	0.400 m
A_{xw9} =	1	1.975 m	0.400 m
A_{xw10} =	1	3.350 m	0.450 m

B. Wall Dimensions (Y-Direction):

	No.	Length (m)	Thk. (m)
A_{yw1} =	1	5.550 m	0.300 m
A_{yw2} =	1	3.075 m	0.300 m
A_{yw3} =	2	2.650 m	0.300 m
A_{yw4} =	1	3.350 m	0.300 m
A_{yw5} =	1	1.750 m	0.400 m
A_{yw6} =	1	0.700 m	0.400 m
A_{yw7} =	1	1.200 m	0.400 m
A_{yw8} =	2	4.550 m	0.400 m
A_{yw9} =	2	2.475 m	0.400 m
A_{yw10} =	1	2.825 m	0.450 m
A_{yw11} =	1	3.350 m	0.450 m
A_{yw12} =	1	2.000 m	0.450 m
A_{yw13} =	1	2.450 m	0.450 m
A_{yw14} =	1	1.875 m	0.600 m

Fundamental Translational Natural Period "X" Direction (T_{ax}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wx} = **2.643 m²**

Calculated approximate fundamental translational natural period T_{ax}:

T_{ax} = 2.537 sec

Fundamental Translational Natural Period "Y" Direction (T_{ay}):

Calculated total effective cross-sectional area of walls in first storey of building, in m^2

A_{wy} = **3.646 m²**

Calculated approximate fundamental translational natural period T_{ay}:

T_{ay} = 2.638 sec

4.6.9. WIND LOAD OF G+16 STORIED BUILDINGS:

Terrain Category= Terrain Category 2 (Considering)

Basic wind speed (V_b)= 50 m/sec

Probability Factor, k_1= 1.0 [As per clause 6.3.1 & Table 1 of IS 875 (Part-3): 2015]

Terrain, Height and Structure Size Factor, k_2 depending on height of structure.

Topography Factor, k_3= 1.0 [As per clause 6.3.3 of IS 875 (Part-3): 2015]

Topography Factor, k_4= 1.0 [As per clause 6.3.2.1 of IS 875 (Part-3): 2015]

4.6.10. WIND LOAD OF G+24, G+30 AND G+36 STORIED BUILDINGS:

Terrain Category= Terrain Category 2 (Considering)

Basic wind speed (V_b)= 50 m/sec

Probability Factor, k_1= 1.0 [As per clause 6.3.1 & Table 1 of IS 875 (Part-3): 2015]

Terrain, Height and Structure Size Factor, k_2 depending on height of structure.

Topography Factor, k_3= 1.0 [As per clause 6.3.3 of IS 875 (Part-3): 2015]

Topography Factor, k_4= 1.0 [As per clause 6.3.2.1 of IS 875 (Part-3): 2015]

4.7. TYPICAL MODELLING TECHNIQUES IN ETABS

4.7.1. STRUCTURAL FRAMING PLAN IN ETABS

Structural Framing Plan of G+24, G+30 and G+36 Storied Building in ETABS Platform are shown below

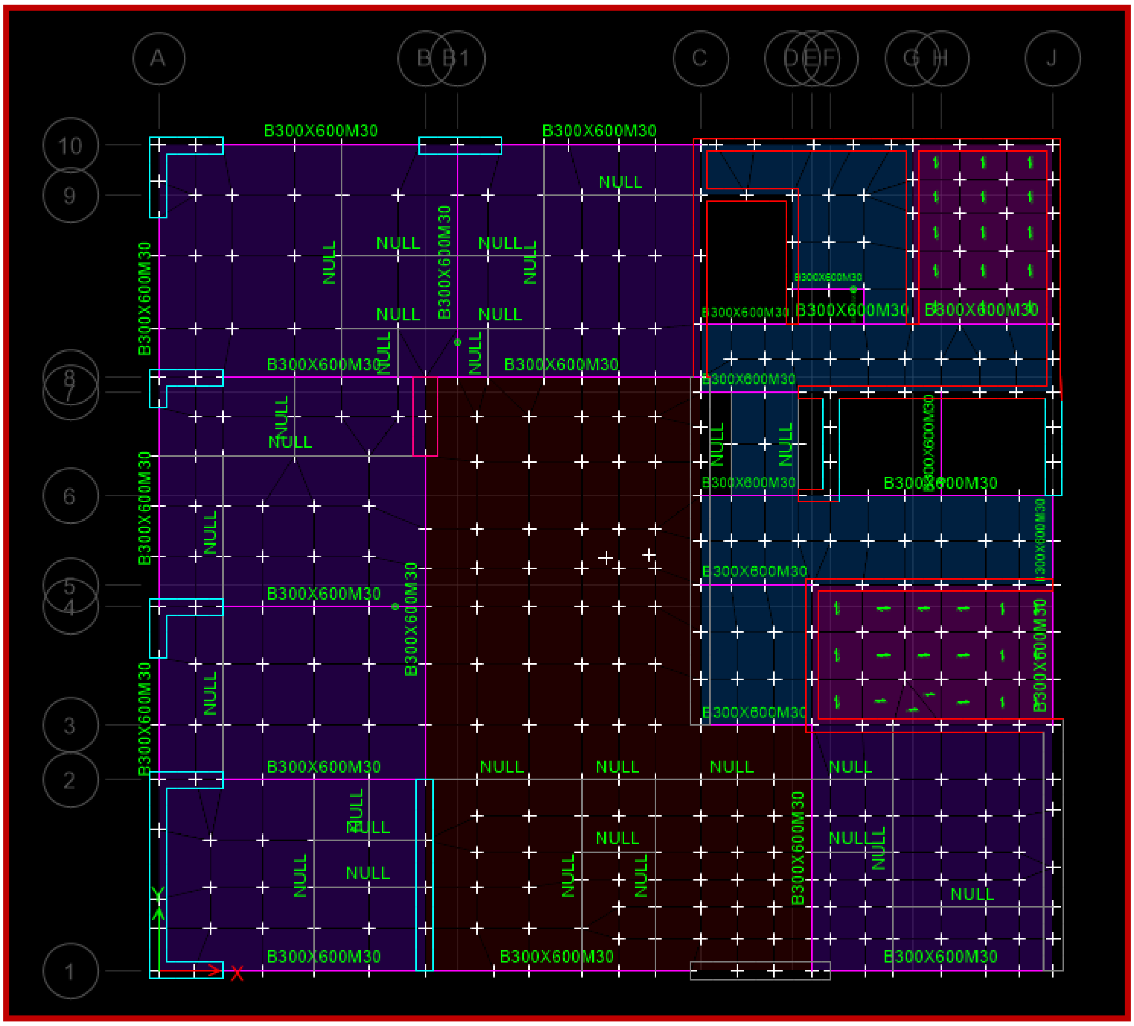
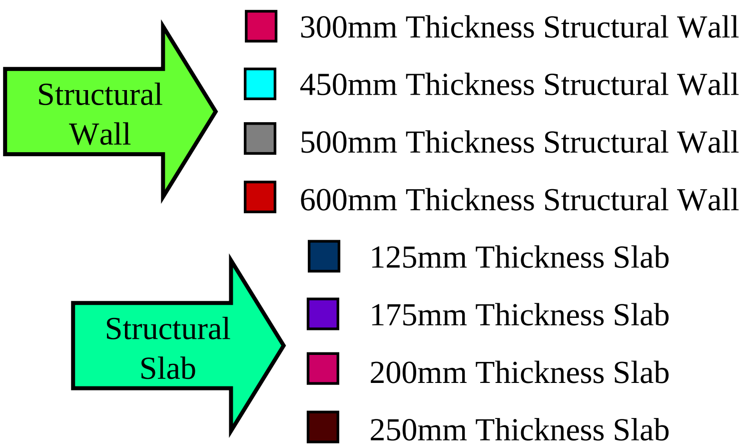


Fig 4.8 Structural Framing Plan of G+24, G+30 and G+36 Storied Buildings



4.7.2. ASSIGNING MATERIAL PROPERTY

Assigning material property of M30 grade concrete and HTSD500 grade rebar in ETABS Platform are shown below

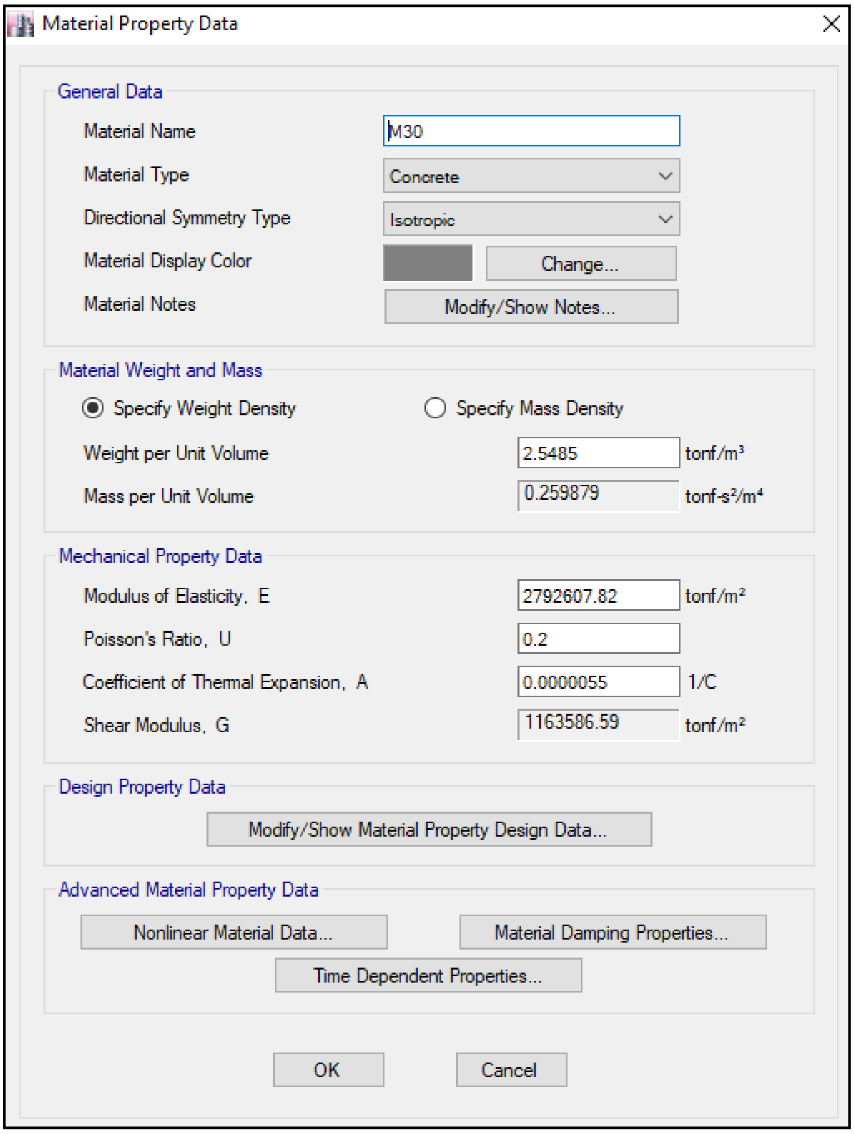
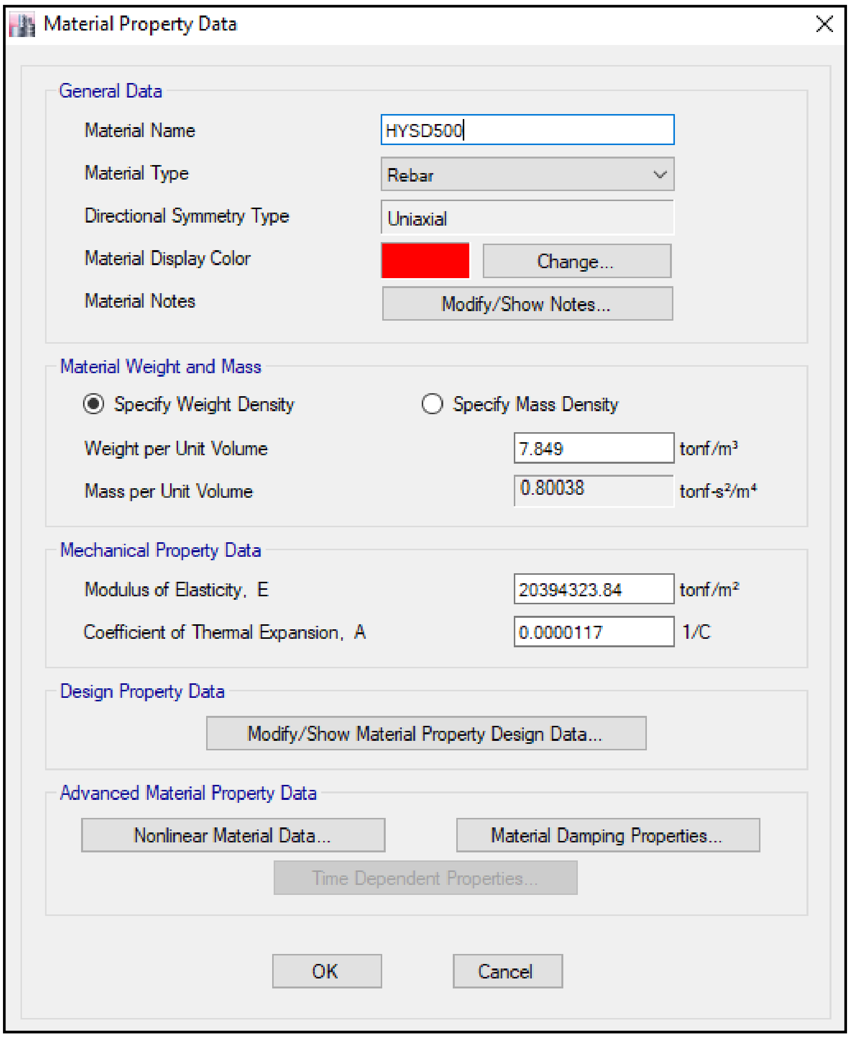


Fig 4.9 ETABS Input- Material Property of M30 Grade Concrete



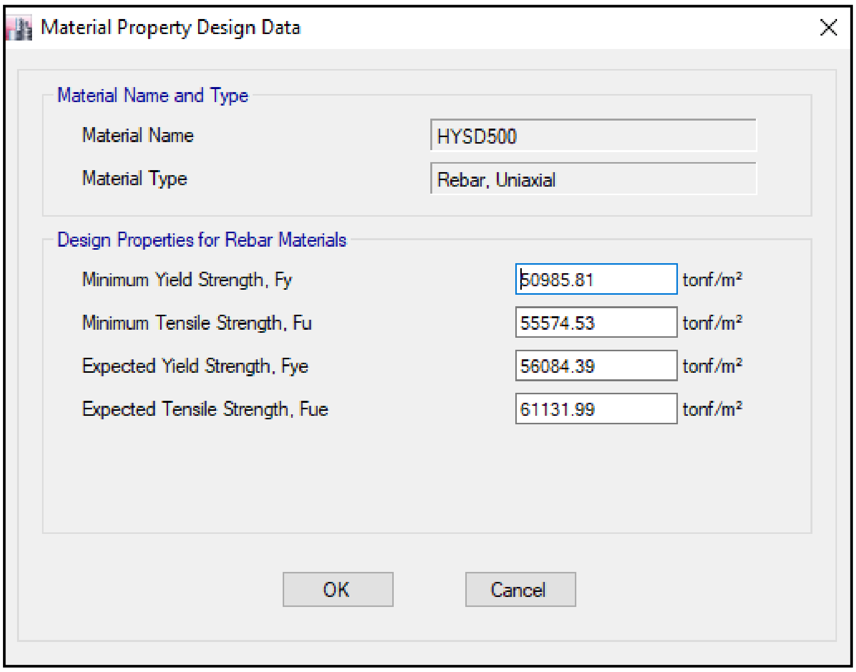
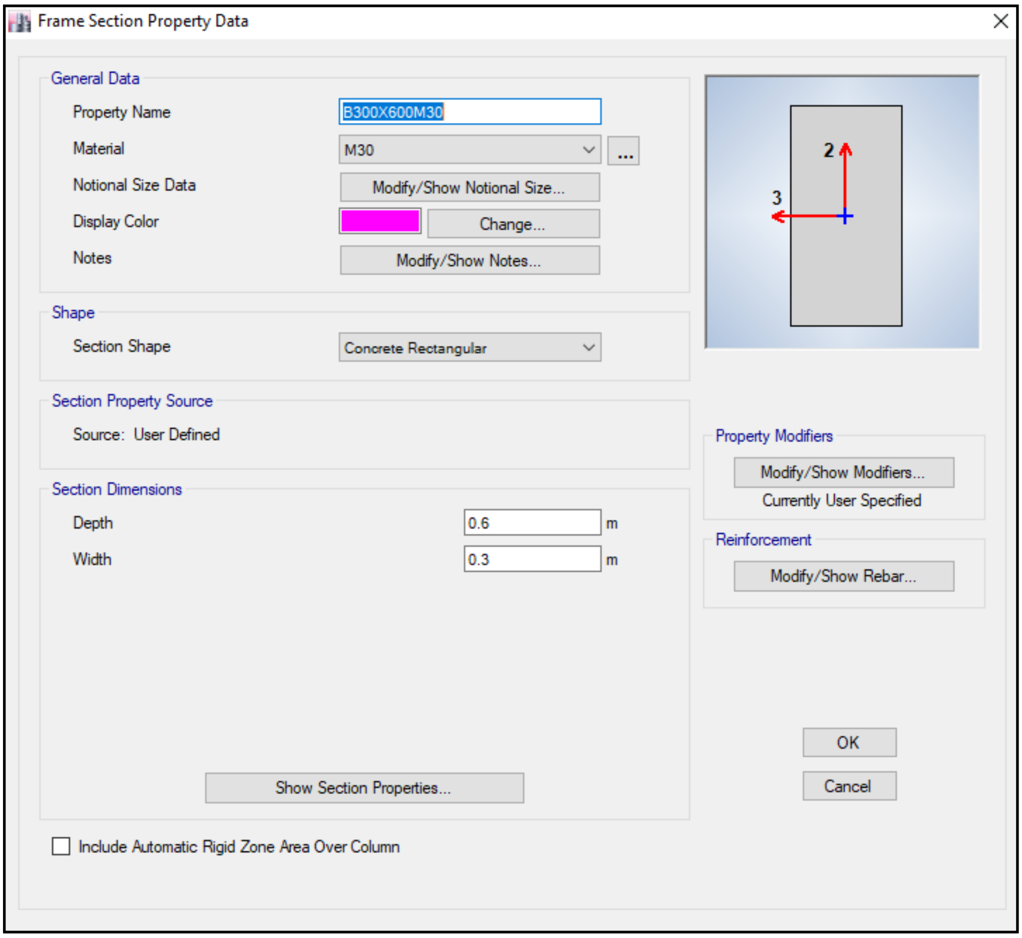


Fig 4.10 ETABS Input- Material Property of HYSD500 Grade Rebar

4.7.3. ASSIGNING MEMBERS PROPERTY

Assigning members property such as beams, slabs and structural walls in ETABS Platform are shown below



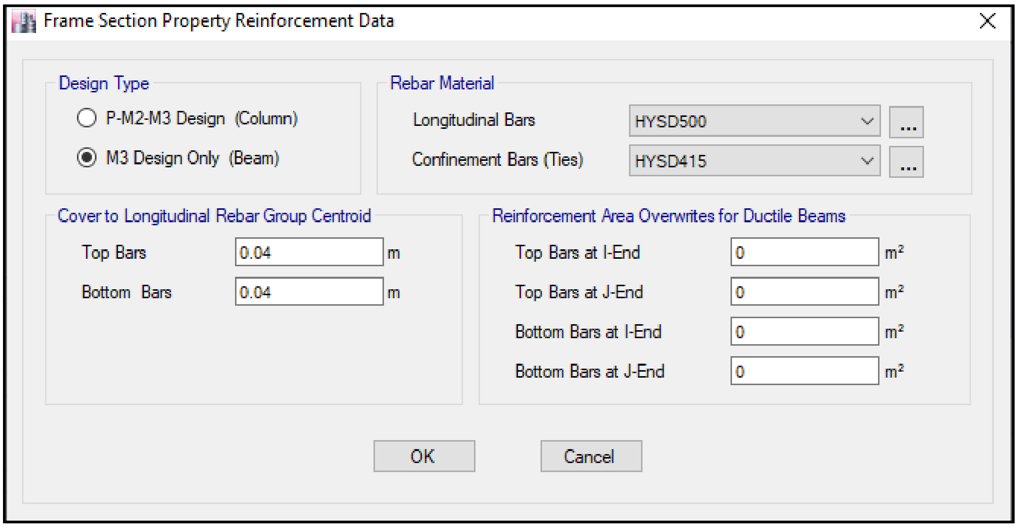


Fig 4.11 ETABS Input- Member Property of Beam Element (300mmX600mm)

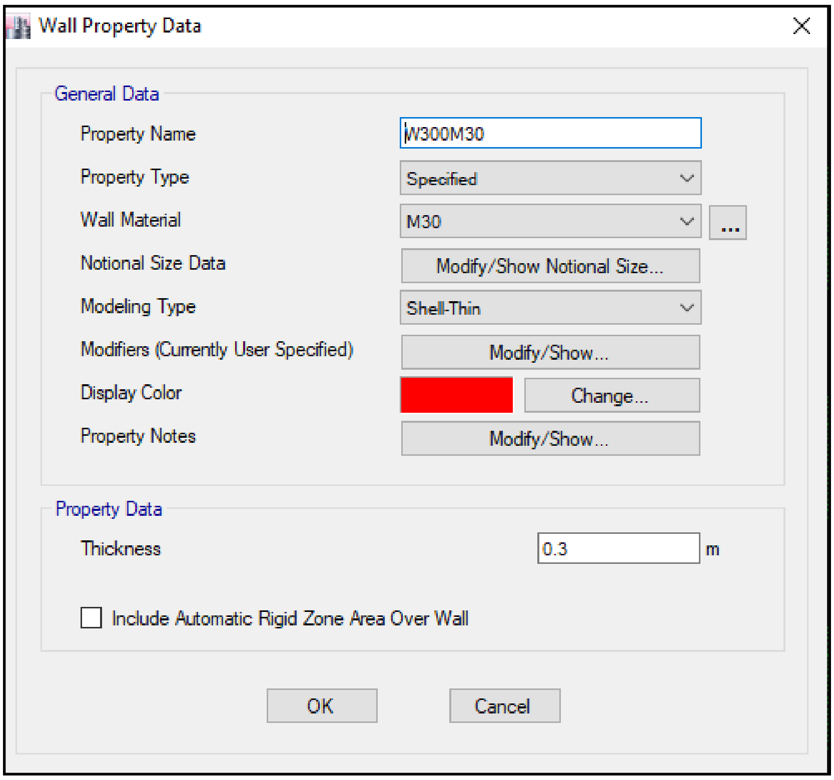


Fig 4.12 ETABS Input- Member Property of Wall Element (300mm Thickness)

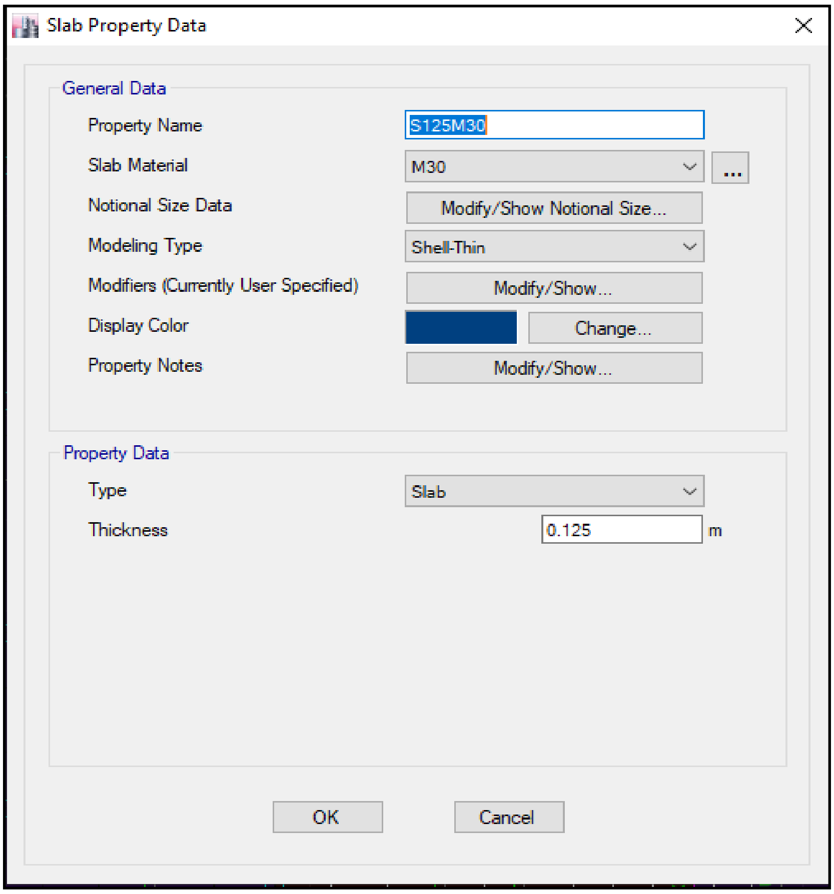


Fig 4.13 ETABS Input- Member Property of Slab Element (125mm Thickness)

4.7.4. DEFINING LOAD PATTERNS

Defining load patterns of different load cases in ETABS Platform are shown below

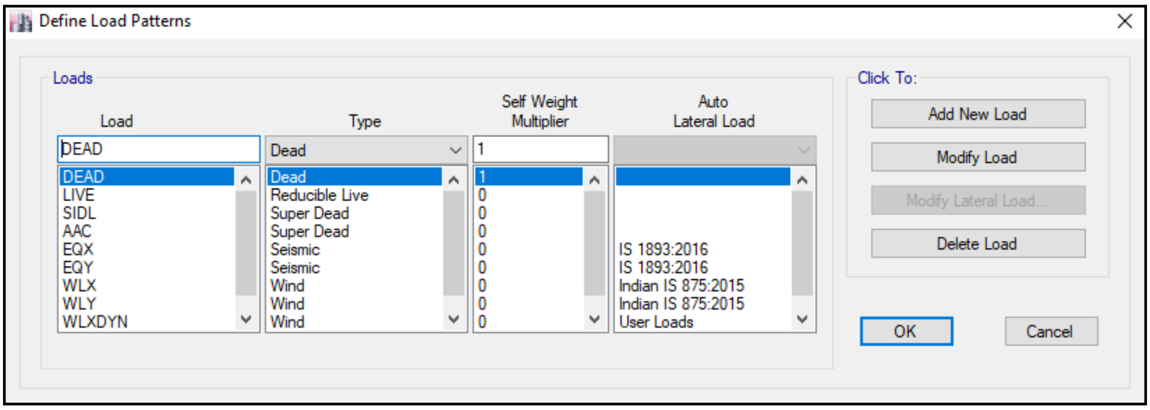
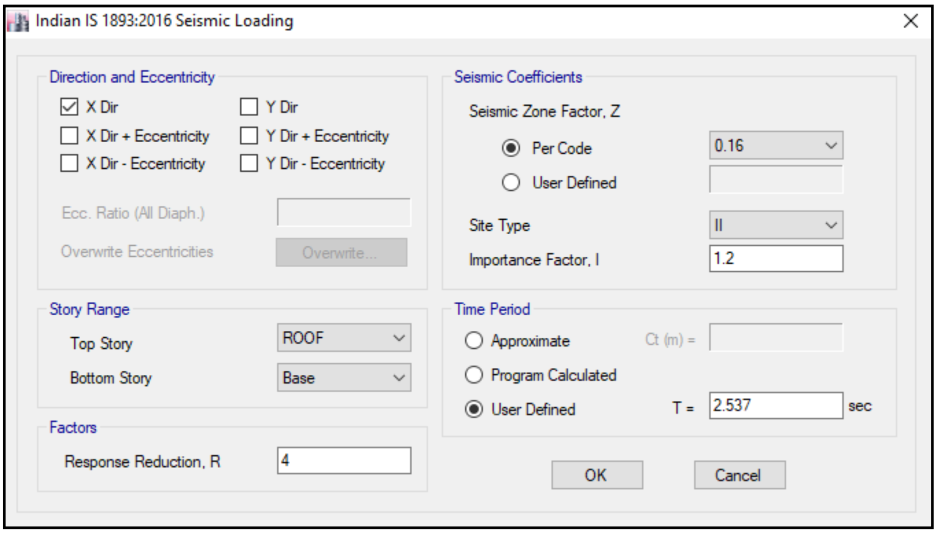


Fig 4.14 ETABS Input- Load Patterns of different load cases



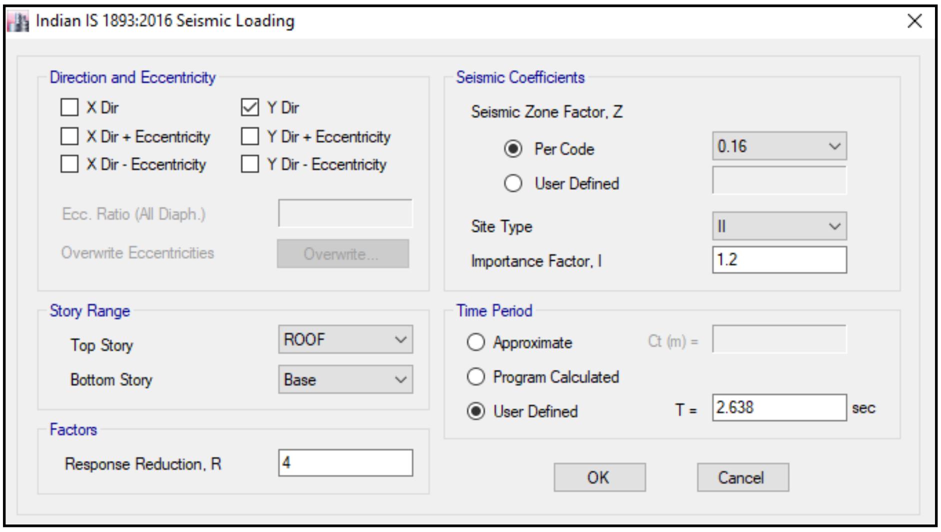


Fig 4.15 ETABS Input- Seismic load definition in X-direction and Y-direction

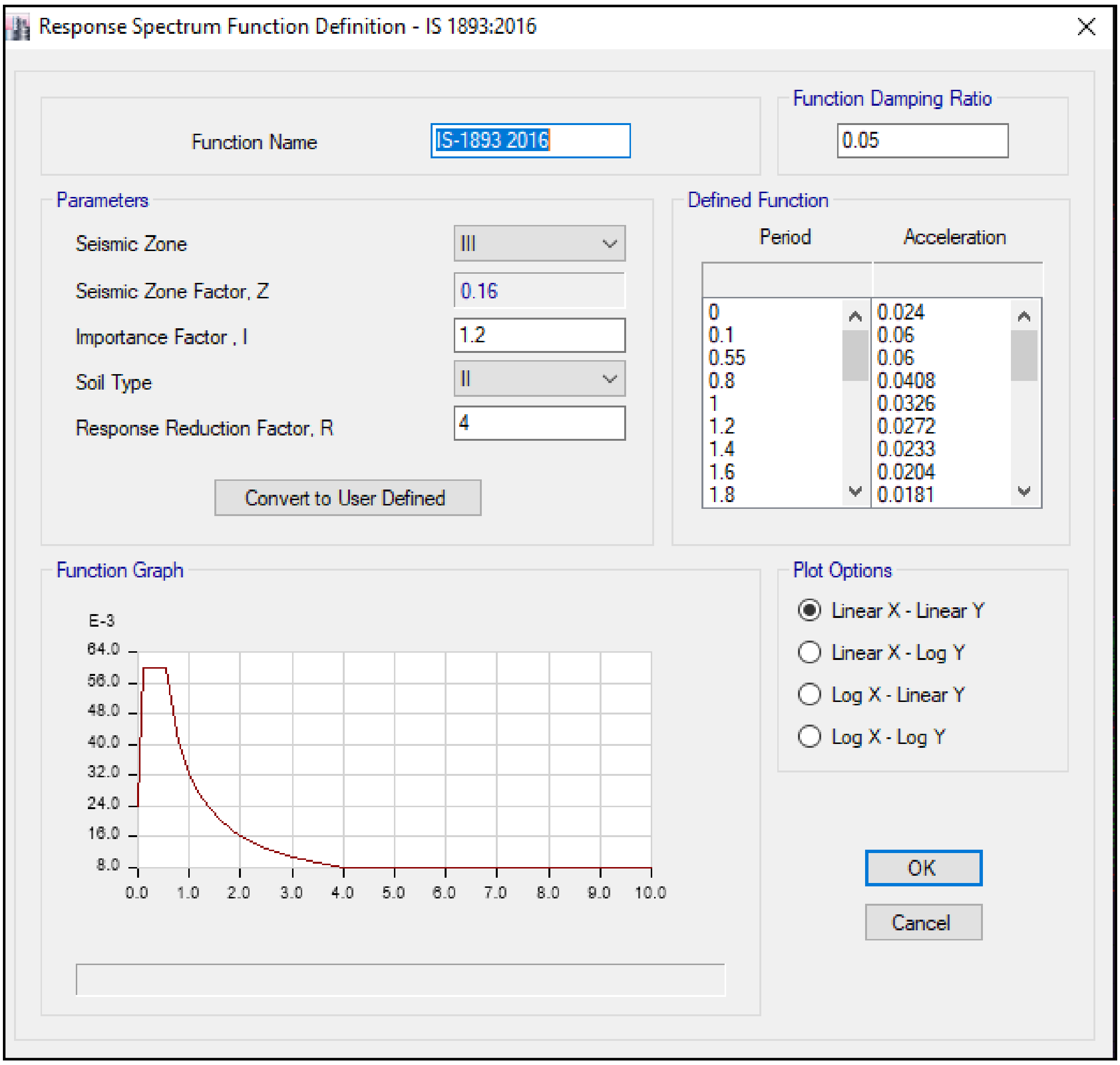
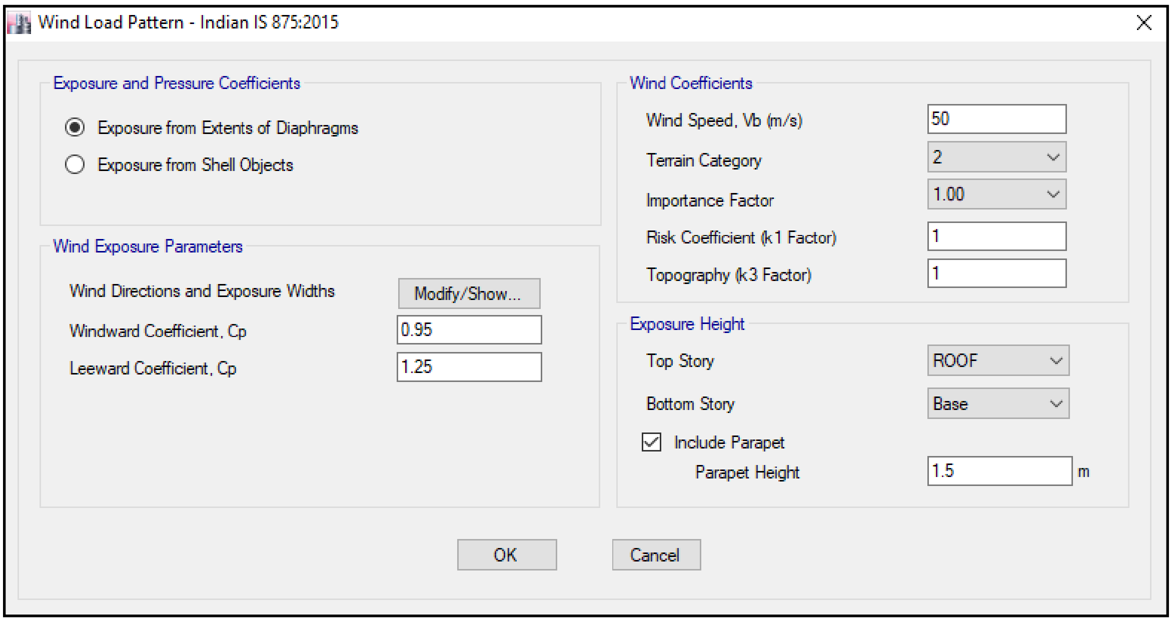


Fig 4.16 ETABS Input- Definition of Response Spectrum function



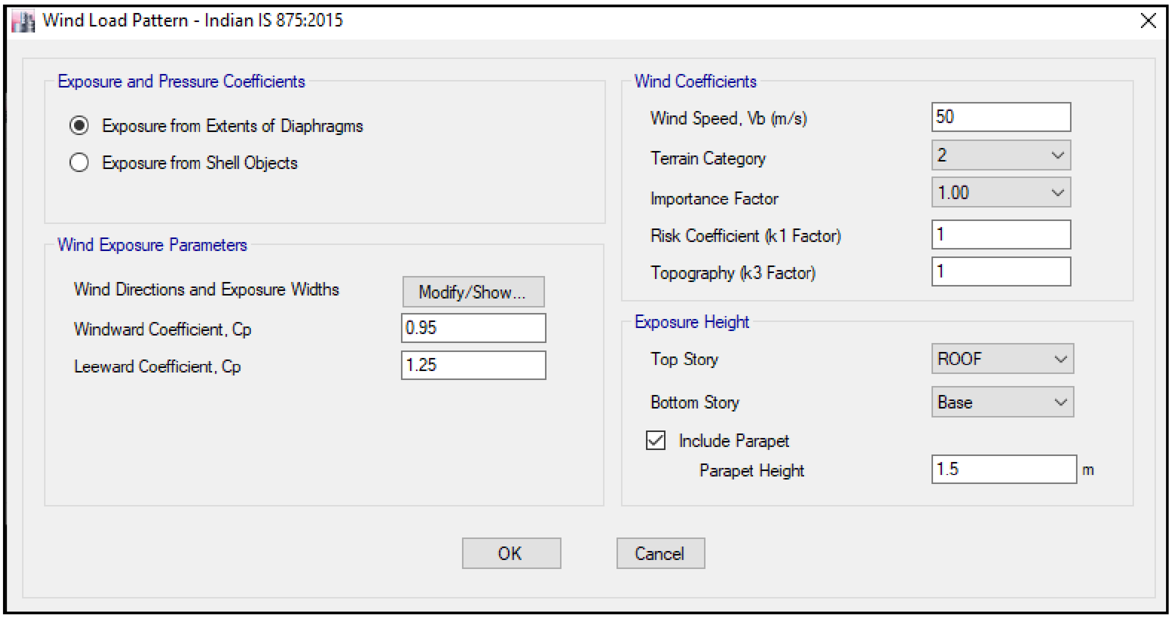


Fig 4.17 ETABS Input- Wind load definition in X-direction and Y-direction

CHAPTER-V

1.2. RESULTS AND DISCUSSIONS

5.1. GENERAL:

Analysis and study of the structural behaviour seems to be essential prior to achieving more conclusive correlation. In my 1st problem a G+12 storied mid-rise residential building, situated at Kolkata, is modelled with dual system and the seismic analysis of the building is carried out in ETABS platform considering the design stipulations of both the codes old IS 1983 (Part-1): 2002 as well as new IS 1983 (Part-1): 2016 separately. After analyses these two buildings compared the structural response such as time period, modal participating mass, base shear, overturning moment, maximum storey displacement, maximum storey drift, storey stiffness etc. for transient loads (Seismic load and wind load) between two buildings respectively. In my 2nd problem a G+24 storied, a G+30 storied and a G+36 storied buildings, situated at Kolkata, are modelled in ETABS platform with same structural framing plan and structural system. The analyses of all three buildings are carried out considering the design stipulations of IS 16700: 2017, criteria for structural safety of tall building, and compared the structural response with previous models with incremental height difference.

5.2. PROBLEM NO.-1:

Comparative study of G+12 storied Mid-rise buildings which are analyzed as per old IS 1893 (Part-1): 2002 and new IS 1893 (Part-1): 2016 with respect to the structural behaviour and its response against the transient loads.

4. Comparative study of G+12 storied buildings

- **G12-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
- **G12-2016** [Analysis as per IS 1893 (Part-1): 2016 and IS 875 (Part-3): 2015]

Table 5.1 Total Input Data of Structure at a Glance for Problem No.-1

Item Description	**Type of Building Structure**	
	Building G12-2002 (G+12)	**Building G12-2016(G+12)**
Length (L)	57.10	57.10
Breadth (B)	14.40m	14.40m
Height (H)	40.45m	40.45m

Table 5.1 Total Input Data of Structure at a Glance for Problem No.-1

Item Description		Type of Building Structure	
		Building G12-2002 (G+12)	**Building G12-2016 (G+12)**
Typical F-F Height		3.05m	3.05m
Finish Load		50mm thick floor finish, refer Cl. No. 4.6.4	
Live Load		0.20 T/m^2 for all rooms, toilet except balcony, staircase and lobby where 0.30 T/m^2, for accessible roof 0.15 T/m^2 and inaccessible roof 0.075 T/m^2 refer Cl. No. 4.6.4	
Masonry Work		200mm and 100mm thickness AAC Block Work considering unit weight 1.0 T/m^3 with 15mm Thickness Plaster at inside and outside of wall, refer Cl. No. 4.6.3	
Mass Source		DL=1.0, SIDL=1.0, ACC Masonry Work= 1.0, Live Load= 0.25	
Material	Wall	Describe Blow along with Fig: 5.1B	
	Slab	Describe Blow along with Fig: 5.1B	
	Beam	Describe Blow along with Fig: 5.1B	
		Seismic Input Data	
Z		0.16	0.16
I		1.0	1.2
R		4	4
T	T_X	Program Calculated	1.238s
	T_Y		0.959s
Damping		0.05	0.05
Soil Type		II	II
		Wind Input Data	
V_b (m/sec)		50	
Terrain		2	
k_1, k_3, k_4		1.0 (k_4 not introduced here)	1.0
k_2		As per IS 875 (Part-3): 1987	As per IS 875 (Part-3): 2015
Damping		0.016	0.02
H/B		2.81	2.81
L/B		3.97	3.97
Windward		0°= 0.80, 90°= 0.70	0°= 0.80, 90°= 0.70
Leeward		0°= 0.10, 90°= 0.40	0°= 0.10, 90°= 0.40

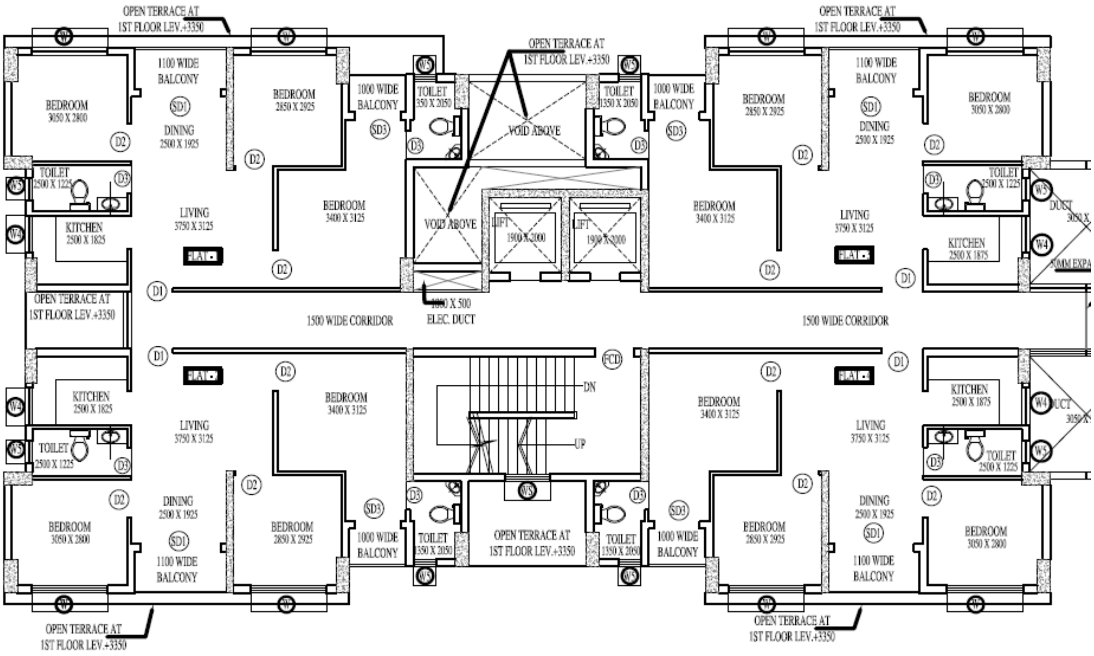


Fig 5.1A Architectural Typical Floor Plan of Left Part of G+12 Storied Building

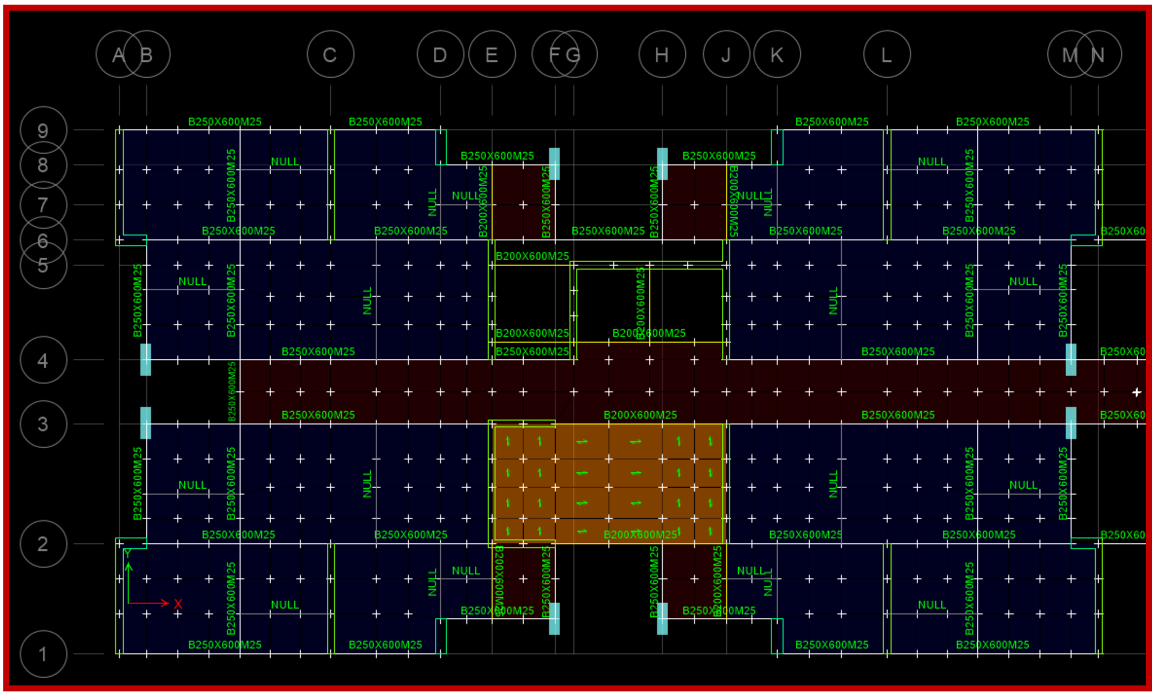


Fig 5.1B Structural Typical Floor Plan of Left Part of G+12 Storied Building

DESCRIPTION OF STRUCTURAL WALLS:

- 200mm Thickness Structural Wall*
- 300mm Thickness Structural Wall*

*G+12 Story: (Fdn-2nd F)- M40 Grade, (2nd -7th F)- M30 Grade, (7th -R)- M25 Grade.

DESCRIPTION OF STRUCTURAL COLUMNS:

- Column size (300X900)*

* G+12 Story: (Fdn-2nd F)- M40 Grade, (2nd -7th F)- M30 Grade, (7th -R)- M25 Grade.

DESCRIPTION OF STRUCTURAL SLABS:

- 110mm Thickness Slab- M25 Grade (Fdn-R)
- 115mm Thickness Slab- M25 Grade (Fdn-R)
- 175mm Thickness Slab- M25 Grade (Fdn-R)

DESCRIPTION OF STRUCTURAL BEAMS:

- Beam size (250X600)- M25 Grade (Fdn-R)
- Beam size (200X600)- M25 Grade (Fdn-R)

5.2.1. STRUCTURAL RESPONSE:

I. COMPARATIVE STUDY OF MODAL RESULT

For analysis 12 nos. of modes are considered to obtain a combined modal mass participation of at least 90% of the actual mass in each of the orthogonal horizontal directions of response considered by the model. The dynamic response of different building models has been enlisted in following table 5.2A and 5.2B shows below.

Table 5.2A Modal Participating Mass Ratio of Building G12-2002, G+12 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	1.423	0.6873	0.0033	0.0401	0.6873	0.0033	0.0401
2	1.208	0.0024	0.5185	0.1586	0.6897	0.5218	0.1986
3	1.188	0.0426	0.1603	0.4848	0.7322	0.6822	0.6834
12	0.075	0.0089	0.0009	0.0002	0.9853	0.9177	0.9171

Table 5.2B Modal Participating Mass Ratio of Building G12-2016, G+12 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	1.848	0.6957	0.0036	0.0261	0.6957	0.0036	0.8509
2	1.447	0.0161	0.1031	0.5649	0.7119	0.1067	0.8531
3	1.419	0.0142	0.5809	0.0955	0.7261	0.6876	0.8878
12	0.091	0.0092	0.0001	0.00015	0.985	0.9167	0.9165

II. COMPARATIVE STUDY OF TIME PERIOD

Natural frequency and natural time period of the structure depends on mass and stiffness of the structure and the mode of vibration depends on the structural framing plan, base dimension of the structure, material property, types of infill masonry works, loadings, height of the building and foundation soil profile etc. The fundamental natural time period of 1st mode of vibration is obtained numerically for different structures, which are shown in Fig 5.2

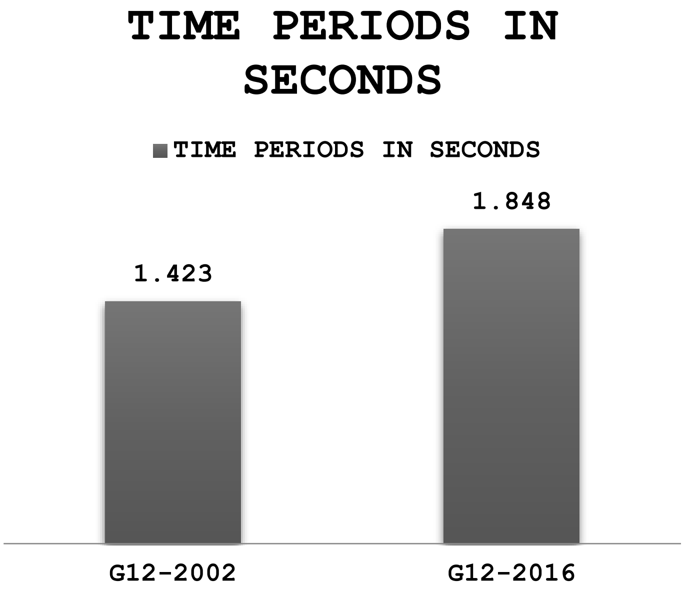


Fig 5.2 Fundamental Time Period for Different Types of Buildings

As per clause no, 6.4.3.1 of IS 1893 (part-1): 2016 for structural analysis, the moment of inertia shall be taken as 70% of I_{gross} of columns or shear walls, and 35% of I_{gross} of beams for reinforced concrete structures. But such kinds of recommendations are not given in old IS 1893 (Part-1): 2002. So, analyse the research model G12-2016 by using stiffness modifier as per new IS 1893 (part-1): 2016, is reduced the overall stiffness of the structure and the structure becomes more flexible than the respective research model G12-2002 which is analysed as per design stipulations of old IS 1893 (Part-1): 2002. For that reason time period of research model G12-2016 is increased even though the structural framing, material properties, soil properties, loading and all other respective design stipulations are not being changed.

III. STRUCTURAL RESPONSE FOR SEISMIC FORCES

The seismic analysis has been performed for all buildings based on the input data given in table 5.1. The summary of building responses due to seismic loading are compared between different numerical models, which are shown below

1. STOREY LEVEL VS STOREY SHEAR

The storey shears are developed along X and Y directions due to Seismic forces at different storey levels for different building models are reflected in Fig 5.3A and Fig 5.3B

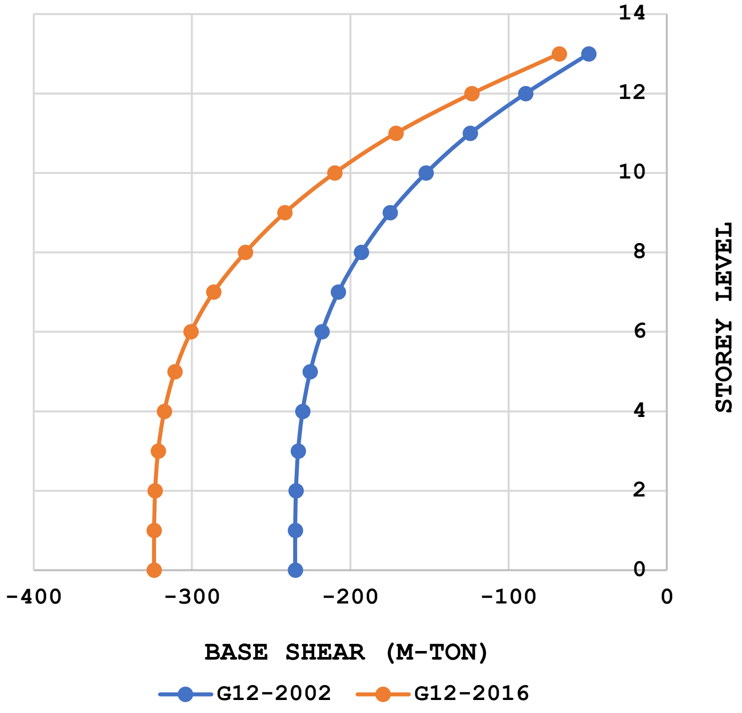


Fig 5.3A Storey Level Vs Storey Shear (Seismic X-Direction)

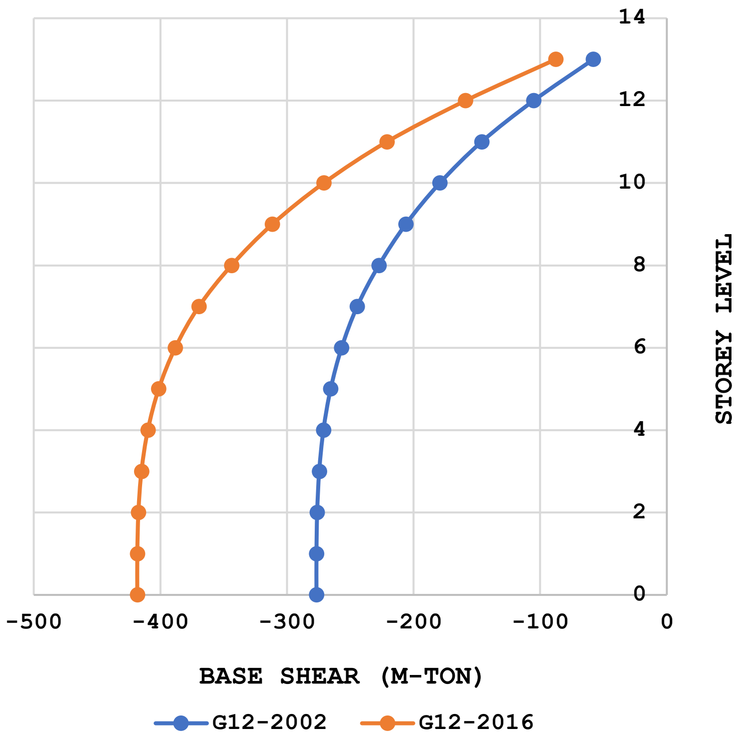


Fig 5.3B Storey Level Vs Storey Shear (Seismic Y-Direction)

It is observed from the graphical representation of storey level vs storey shear, fig 5.3A and fig 5.3B, that the base shears of research model G12-2016 is more than the base shear of research model G12-2002. As per clause number 7.2.3 and table 8 of new IS 1893 (Part-1): 2016, for estimating design lateral force V_B of building the importance factor I of research model G12-2016 is taken 1.2 because, the number of occupancy is more than 200 persons. For that reason the value of design horizontal earthquake acceleration coefficient A_h is increased even though the seismic mass W is not changed. On other hand the formula for calculating approximate fundamental translational natural period T_a of oscillation for building with RC structural wall is implemented in new IS 1893 (Part-1): 2016 and is considered in research model G12-2016. But, there are no such recommendation is given in old IS 1893 (Part-1): 2002. So, natural period of oscillation is considered program calculated for research model G12-2002. If time period of the structure is changed the value of S_a/g will also be changed. The value of A_h is depended on value of S_a/g, which is reflected in calculation of base shear V_B. Because, $V_B = A_h \, x \frac{ZI}{2R} \frac{S_a}{g}$.

2. STOREY LEVEL VS OVERTURNING MOMENT

The overturning moments developed due to seismic forces along X and Y directions at different storey level for different building models have been reflected in Fig 5.4A and Fig 5.4B

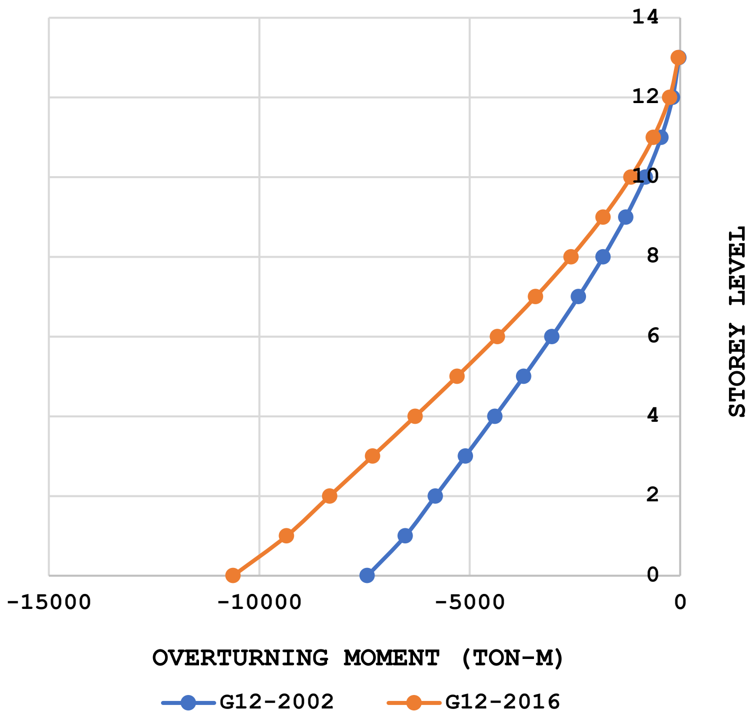


Fig 5.4A Storey Level Vs Overturning Moment (Seismic X-Direction)

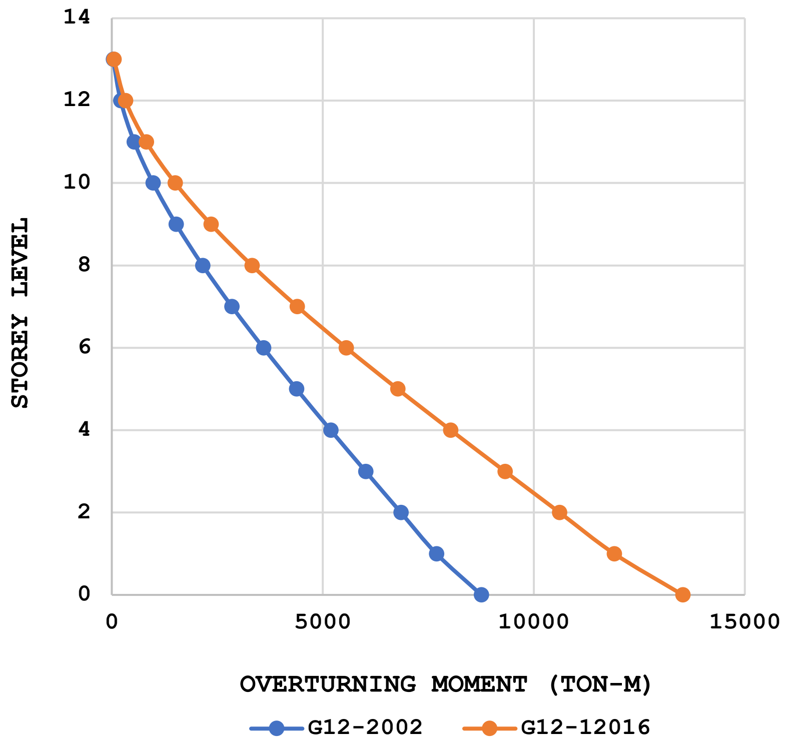


Fig 5.4B Storey Level Vs Overturning Moment (Seismic Y-Direction)

Overturning moment is caused by storey force developed at different floor level for ground movement due to transient forces. The value of overturning moment is depends on storey shear and height of the structure. It is observed from Fig: 5.4A and Fig: 5.4B that the overturning moment of research model G12-2016 is more than the research model G12-2002 because the storey shears of building G12-2016 at different levels are greater than the storey shears of building G12-2002, refer Fig 5.3A and Fig 5.3B.

3. STOREY LEVEL VS STOREY DISPLACEMENT

The storey displacement caused by seismic forces along X and Y directions at different storey levels for all building types have been shown in Fig 5.5A and Fig 5.5B

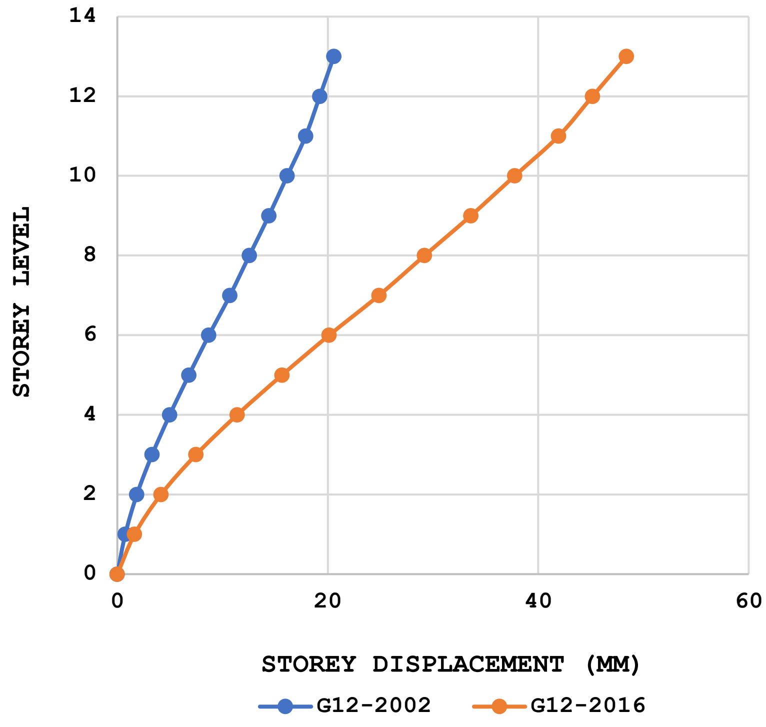


Fig 5.5A Storey Level Vs Storey Displacement (Seismic Y-Direction)

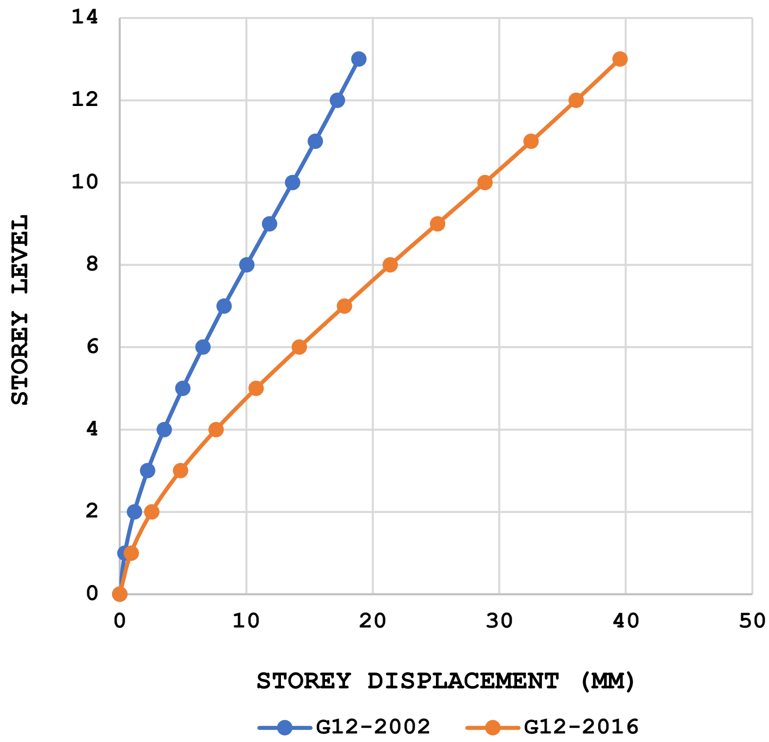


Fig 5.5B Storey Level Vs Storey Displacement (Seismic Y-Direction)

It is observed from above graphical representation of Fig: 5.5A and Fig: 5.5B that the storey displacement of research model G12-2016 is more than the research model G12-2002 because the storey shears of building G12-2016 at different levels are greater than the storey shears of building G12-2002. Also using of stiffness modifier in structural elements are reduced the stiffness which is increased the flexibility of the structure even though the material properties, structural configurations, loading, seismic parameters etc are not being changed.

4. STOREY LEVEL VS STOREY DRIFT

The storey drift caused by seismic forces along X and Y directions at different storey levels for all building types have been shown in Fig 5.6A and Fig 5.6B

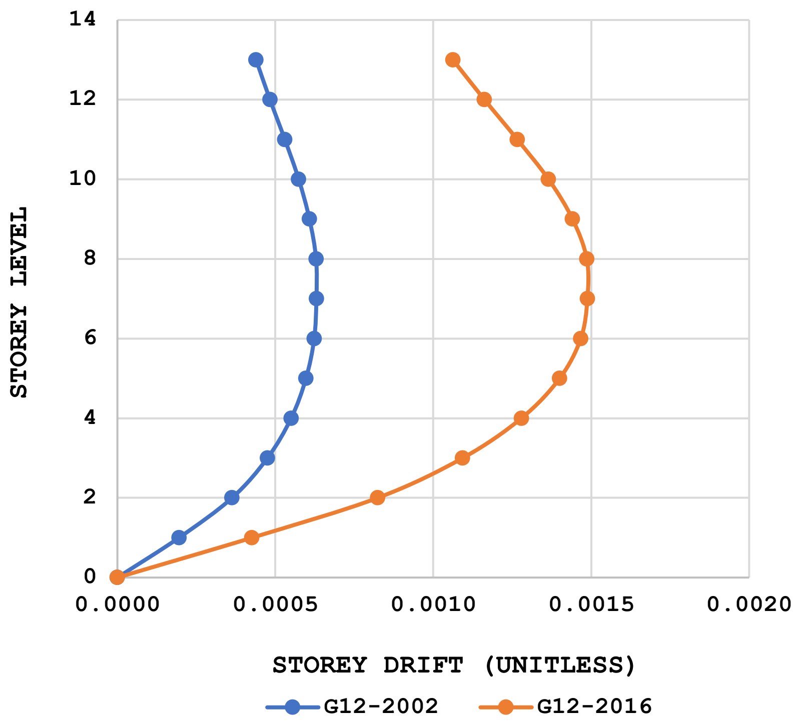


Fig 5.6A Storey Level Vs Storey Drift (Seismic X-Direction)

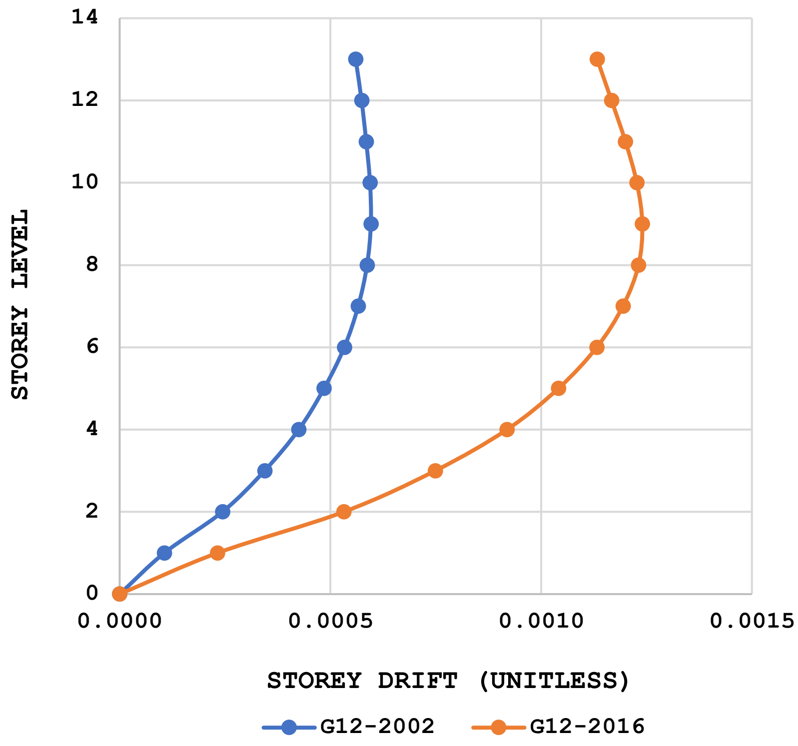


Fig 5.6B Storey Level Vs Storey Drift (Seismic Y-Direction)

Storey drift is unit less because it is ratio of storey displacement of respective floor to storey height. From the graphical representation of Fig 5.6A and Fig 5.6B it is noted that all buildings experiences max drift at top one third of the total building height for seismic loading and with increase of storey level the value of drift declines. In all the cases the storey drifts are within the permissible limit h/250, where h is defined respective storey height of that building. From figures it is observed that the storey drift of research model G12-2016 is more than G12-2002 due to their varying stiffness (use of stiffness modifier as per IS 1893 (Part-1): 2016) and base shears respectively.

5. STOREY LEVEL VS STOREY STIFFNESS

The storey stiffness along X and Y directions due to Earthquake loading at different storey levels for different building models are depicted in Fig 5.7A and Fig 5.7B

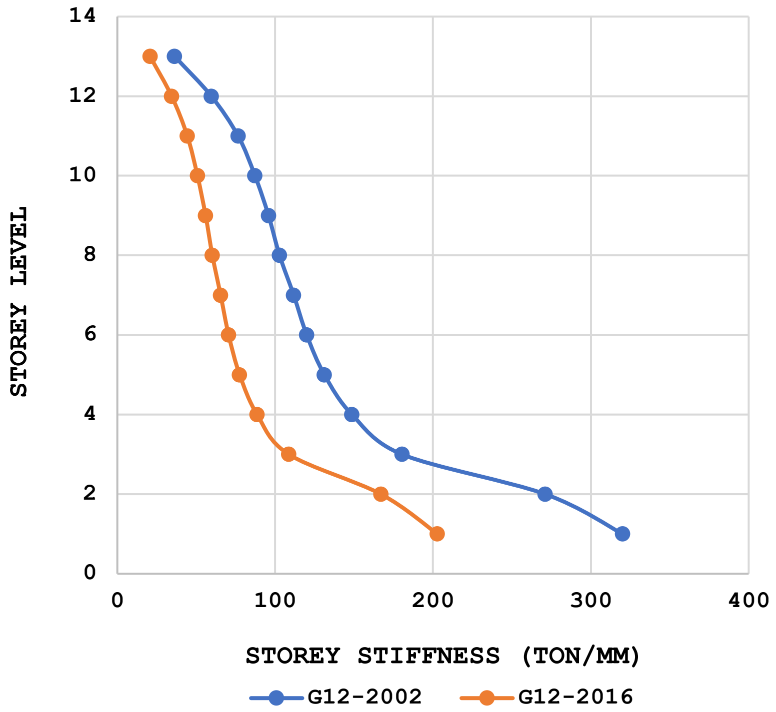


Fig 5.7A Storey Level Vs Storey Stiffness (Seismic X-Direction)

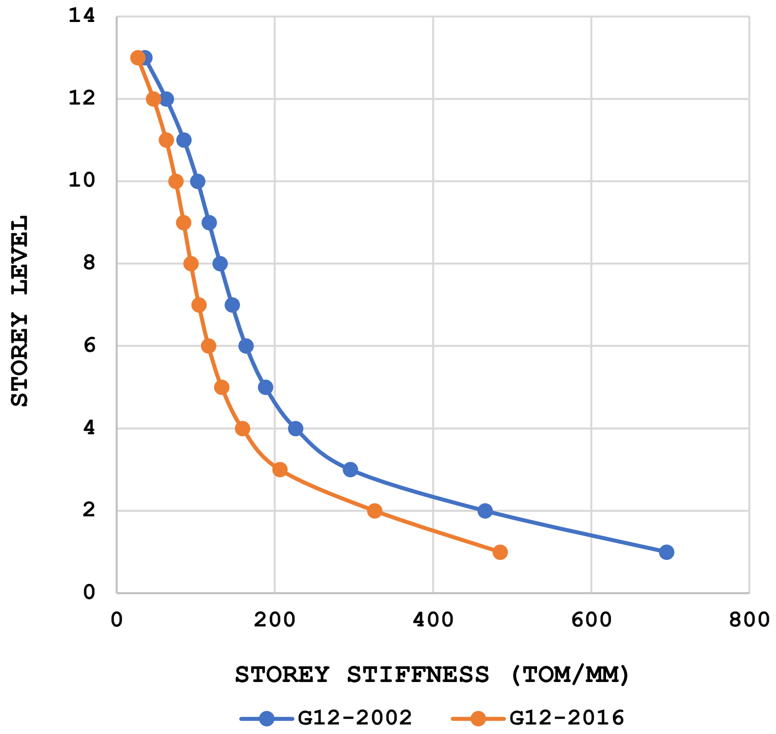


Fig 5.7B Storey Level Vs Storey Stiffness (Seismic Y-Direction)

From Fig 5.7A and Fig 5.7B it is studied that the storey stiffness is gradually reduced from 1st floor level to roof level for all types of building models. Though structural wall configurations are remaining same and the storey height at 1st floor level is higher than the above floors, due to higher elastic modulus the stiffness at 1st floor level is more than the above floor levels. On other hand the stiffness of research model G12-2016 is less than the stiffness of model G12-2002. For this kind of result, it is concluded that even though the structural arrangement, element sizes, material properties and height of the buildings are remaining same, reduction of storey stiffness is due to implementation of stiffness modifier in structural walls, columns and beams as per clause number 6.4.3.1 of new IS 1893 (Part-1): 2016, which is reduced the gross moment of inertia of the structural elements resulting reduction in storey stiffness.

IV. STRUCTURAL RESPONSE FOR WIND FORCES

The wind analysis has been performed for all buildings considered based on the input data given in table 5.1. The summary of building responses due to static wind loading are compared between different numerical models, which are shown below

1. STOREY LEVEL VS STOREY SHEAR

The storey shear developed along X and Y directions due to wind load applicable at different storey levels for building models have been reflected in Fig 5.8A and Fig 5.8B

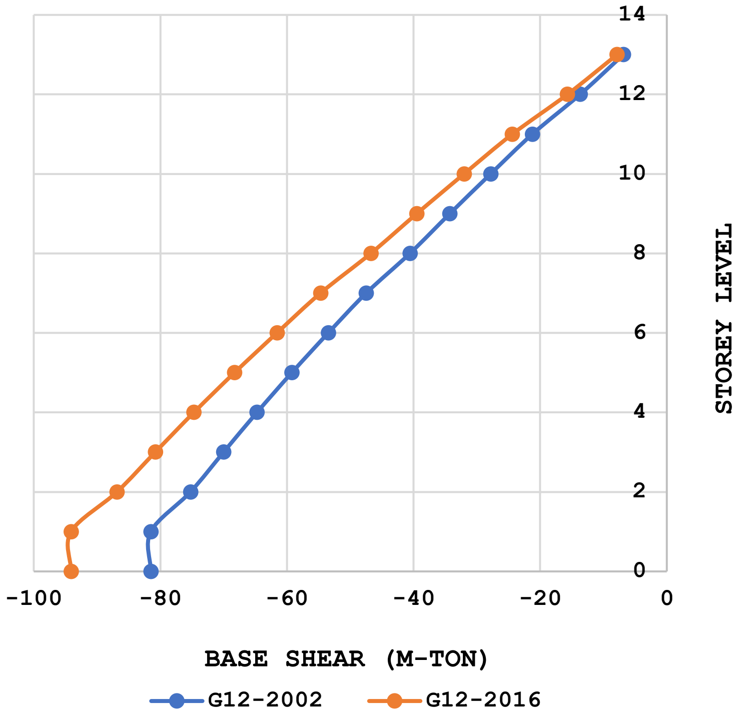


Fig 5.8A Storey Level Vs Storey Shear (Wind X-Direction)

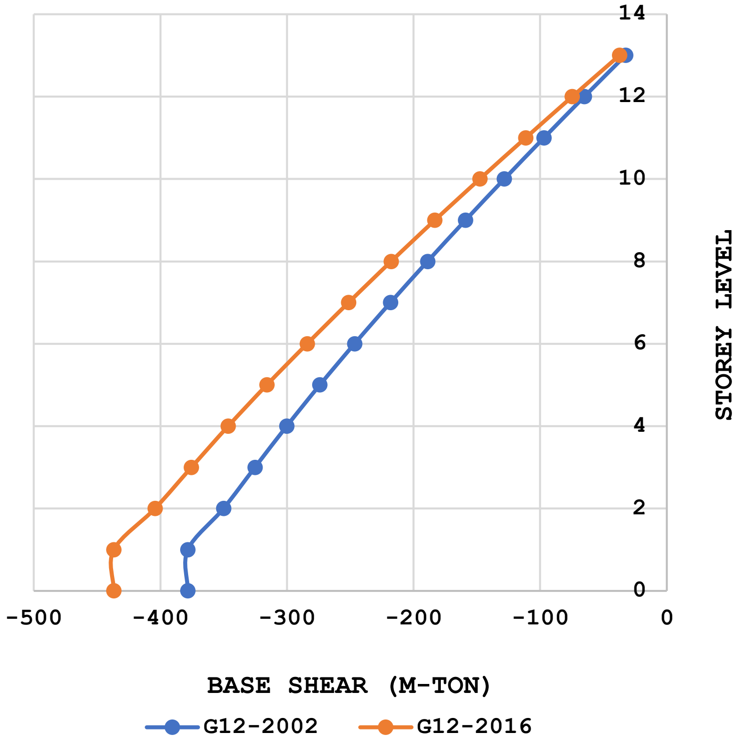


Fig 5.8B Storey Level Vs Storey Shear (Wind Y-Direction)

It is observed from Fig 5.8A and Fig 5.8B that the total applied wind force on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. In this new code a factor k_4 (importance factor) is introduced and also the value of k_2 with respect to the height of the structure seems to be higher side in this latest revision. Although the wind force is calculated with same amount of exposed area and obviously considered wind speed is constant. Amount of static wind force applied on structure does not depend on inherent properties of the structure.

2. STOREY LEVEL VS OVERTURNING MOMENT

The overturning moment developed along X and Y directions due to wind force applicable at different storey levels for different building models have been reflected in Fig 5.9A and Fig 5.9B

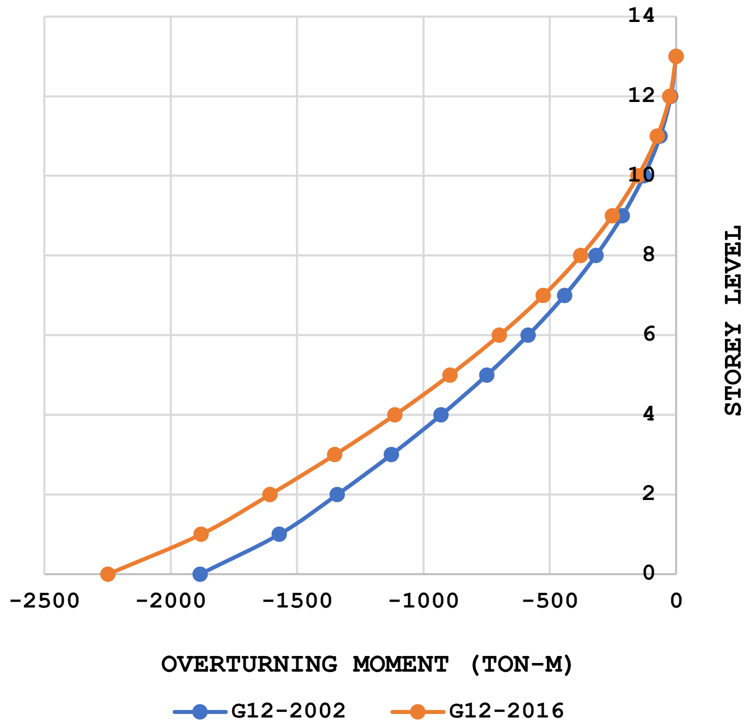


Fig 5.9A Storey Level Vs Overturning Moment (Wind X-Direction)

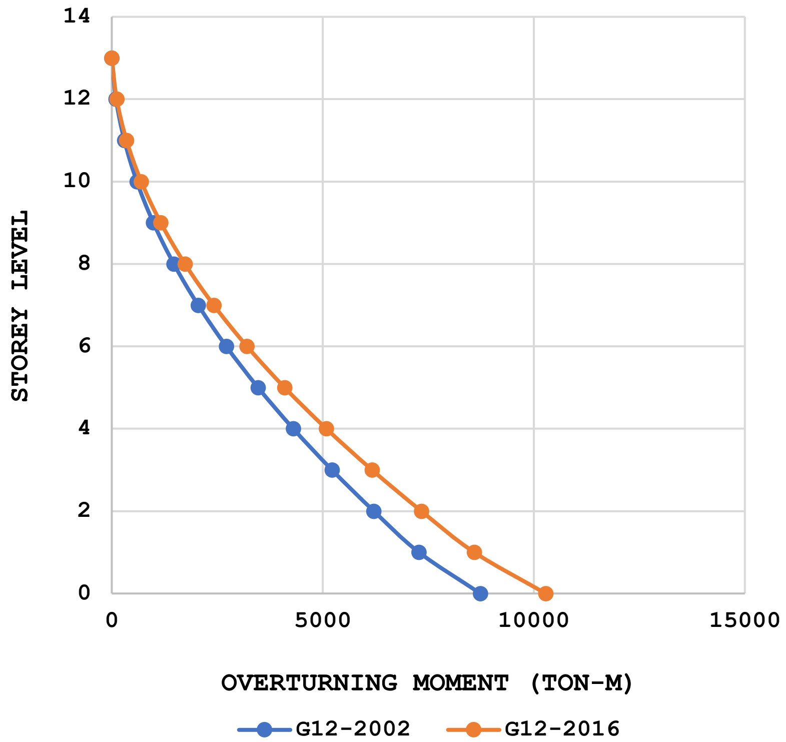


Fig 5.9B Storey Level Vs Overturning Moment (Wind Y-Direction)

It is observed from Fig 5.8A and Fig 5.8B that the calculated storey shears on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. The value of overturning moment is depends on storey shear and height of the structure. It is observed from Fig 5.9A and Fig 5.9B that, due to higher storey shear the overturning moment of research model G12-2016 is greater than model G12-2002, because height and exposed area of both the models are same.

3. STOREY LEVEL VS STOREY DISPLACEMENT

The storey displacement caused by wind force along X and Y directions at different storey levels for different building models have been shown in Fig 5.10A and Fig 5.10B

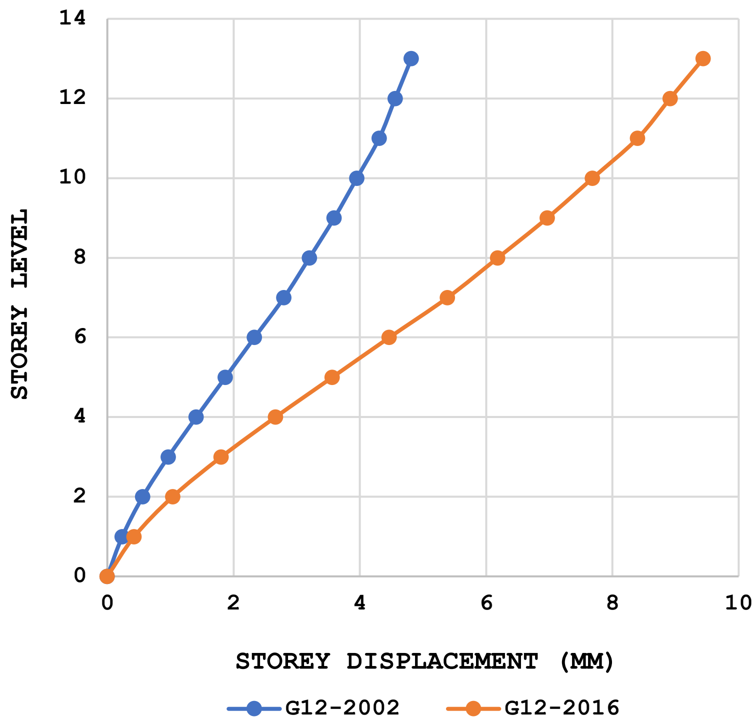


Fig 5.10A Storey Level Vs Storey Displacement (Wind X-Direction)

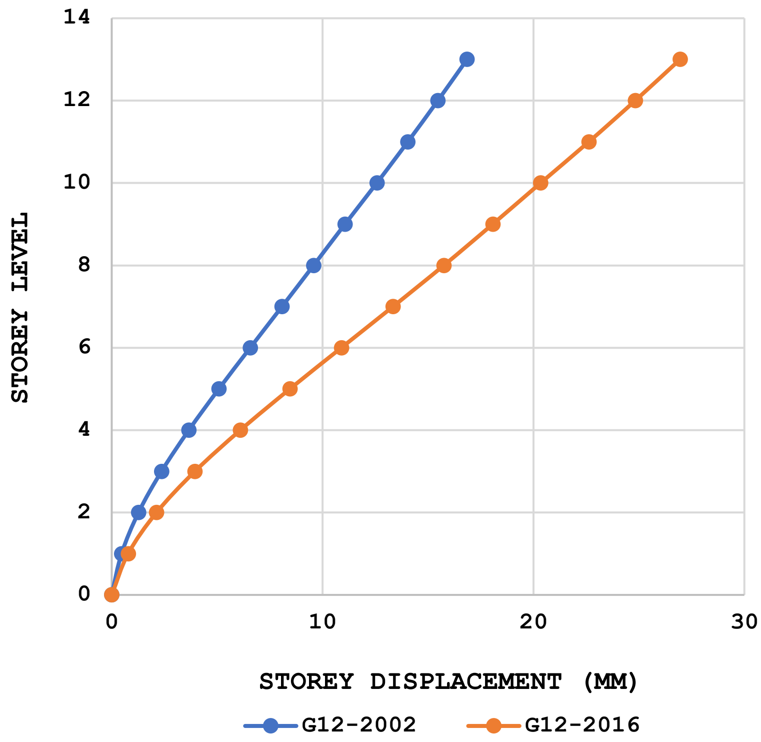


Fig 5.10B Storey Level Vs Storey Displacement (Wind Y-Direction)

It is observed from above graphical representations, Fig 5.10A and Fig 5.10B, that storey displacement of research model G12-2016 is greater than the storey displacement of model G12-2002 because the storey shears of model G12-2016 at different levels are more than the storey shears of model G12-2002 also the research model G12-2016 is less stiffer than research model G12-2002 due to using of stiffness modifier in structural wall, columns and beams. But maximum storey displacements at roof level of these two models are within the permissible limit (H/500, where H is the height of building including the parapet height) of respective Indian standards.

4. STOREY LEVEL VS STOREY DRIFT

The storey drift caused by wind forces along X and Y directions at different storey levels for all building types have been depicted in Fig 5.11A and Fig 5.11B

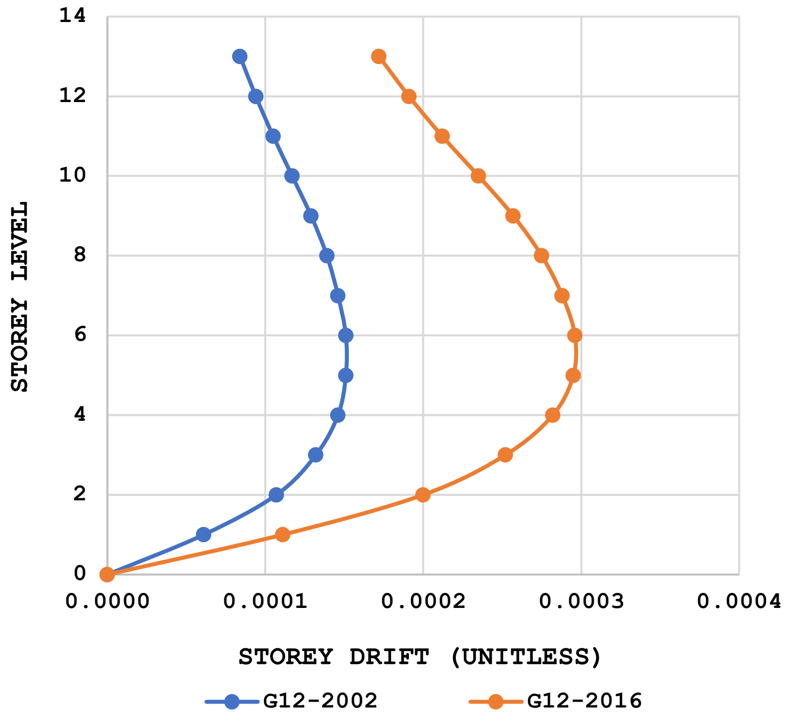


Fig 5.11A Storey Level Vs Storey Drift (Wind X-Direction)

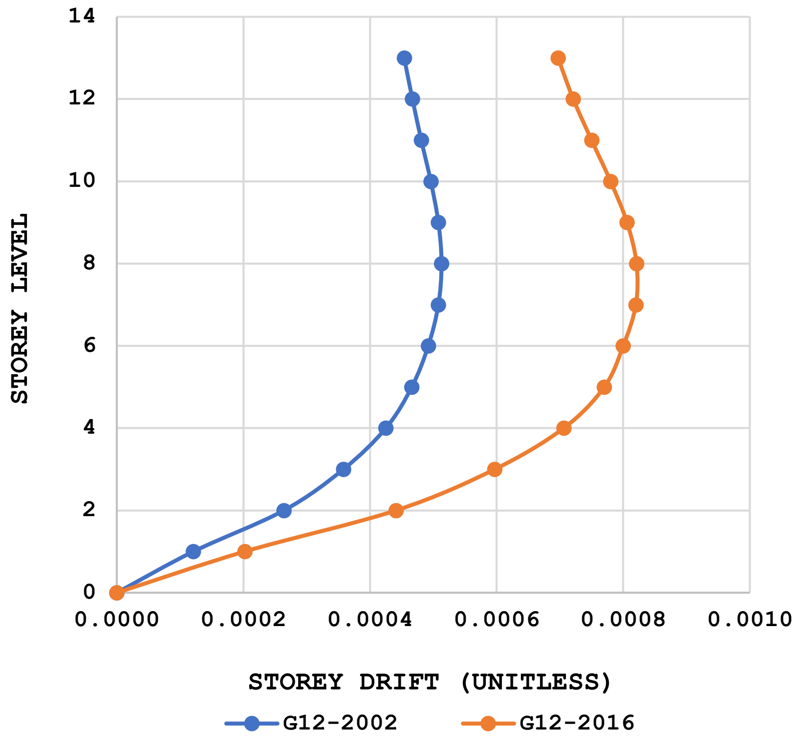


Fig 5.11B Storey Level Vs Storey Drift (Wind Y-Direction)

Storey drift is unit less because it is ratio of storey displacement of respective floor to storey height. From the graphical representation of Fig 5.11A and Fig 5.11B it is noted that all buildings experiences max drift at bottom half of the total building height for wind loading and with increase of storey level the value of drift declines. In all the cases the storey drifts are within the permissible limit. From figures it is observed that the storey drift of research model G12-2016 is more than G12-2002 due to their varying stiffness (using of stiffness modifier in model G12-2016) and wind base shears respectively.

Table 5.3 Comparison of Structural Response due to Codal Revision for Problem No.-1

Item Description	Type of Loading	Direction of Loading	Type of Building Structure		Change in Response (%)
			G12-2002	G12-2016	
Time Period (Sec) at 1st Mode			**1.423**	**1.848**	**(+) 29.87**
Base Shear (Ton)	Seismic Loading	**EQX**	**235**	**324**	**(+) 37.87**
		EQY	**277**	**418**	**(+) 50.90**
	Wind Loading	**WLX**	**82**	**94**	**(+) 14.63**
		WLY	**378**	**437**	**(+) 15.61**
Overturning Moment (Ton-m)	Seismic Loading	**EQX**	**7431**	**10621**	**(+) 42.93**
		EQY	**8758**	**13531**	**(+) 54.50**
	Wind Loading	**WLX**	**1884**	**2248**	**(+) 19.32**
		WLY	**8733**	**10281**	**(+) 17.73**
Storey Displacement (mm)	Seismic Loading	**EQX**	**21.0**	**48.0**	**(+) 128.57**
		EQY	**19.0**	**40.0**	**(+) 110.53**
	Wind Loading	**WLX**	**5.0**	**9.0**	**(+) 80.00**
		WLY	**17.0**	**27.0**	**(+) 58.82**
Maximum Storey Drift	Seismic Loading	**EQX**	**0.0006**	**0.0015**	**(+) 150.00**
		EQY	**0.0005**	**0.0012**	**(+) 140.00**
	Wind Loading	**WLX**	**0.0002**	**0.0003**	**(+) 50.00**
		WLY	**0.0005**	**0.0008**	**(+) 60.00**
Storey Stiffness (Ton/mm)	Seismic Loading	**EQX**	**320**	**203**	**(-) 36.56**
		EQY	**695**	**485**	**(-) 30.22**

5.3. PROBLEM NO.-2:

Comparative study of G+24, G+30 and G+36 storied High-rise buildings which are analyzed as per old IS 1893 (Part-1): 2002 and new IS 1893 (Part-1): 2016 conjugate with IS 875 (Part-3) and IS 16700: 2017 with respect to the structural behaviour and its response against the transient loads.

1. Comparative study of G+24 storied buildings

- **G24-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
- **G24-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]

2. Comparative study of G+30 storied buildings

- **G30-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
- **G30-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]

3. Comparative study of G+36 storied buildings

- **G36-2002** [Analysis as per IS 1893 (Part-1): 2002 and IS 875 (Part-3): 1987]
- **G36-2016700** [Analysis as per IS 1893 (Part-1): 2016, IS 875 (Part-3): 2015 and IS 16700: 2017]

Table 5.4 Total Input Data of Structure at a Glance for Problem No.-2

Item Description		Type of Building Structure					
		Building G24-2002	**Building G24-2016700**	**Building G30-2002**	**Building G30-2016700**	**Building G36-2002**	**Building G36-2016700**
Length (L)		20.30m	20.30m	20.30m	20.30m	20.30m	20.30m
Breadth (B)		19.70m	19.70m	19.70m	19.70m	19.70m	19.70m
Height (H)		88.10m	88.10m	109.10m	109.10m	130.10m	130.10m
Typ. F-F Ht.		3.50m	3.50m	3.50m	3.50m	3.50m	3.50m
Finish Load		50mm thick floor finish with service load refer Cl. No. 4.6.4					
Live Load		0.20 T/m^2 for all rooms, toilet except balcony, staircase and lobby where 0.30 T/m^2, for accessible roof 0.15 T/m^2 refer Cl. No. 4.6.4					
Masonry Work		200mm AAC Block Work considering unit weight 0.80 T/m^3 with 15mm Thickness Plaster inside and outside of wall, refer Cl. No. 4.6.3					
Mass Source		DL=1.0, SIDL=1.0, ACC Masonry Work= 1.0, Live Load= 0.25					
Material	Wall	Describe Blow along with Fig: 5.12B					
	Slab	Describe Blow along with Fig: 5.12B					
	Beam	Describe Blow along with Fig: 5.12B					
		Seismic Input Data					
Z		0.16	0.16	0.16	0.16	0.16	0.16
I		1.0	1.2	1.0	1.2	1.0	1.2
R		4	4	4	4	4	4
T	T_X	Program Calculated	1.718s	Program Calculated	2.128s	Program Calculated	2.537s
	T_Y		1.786s		2.212s		2.638s
Damping		0.05	0.05	0.05	0.05	0.05	0.05
Soil Type		II	II	II	II	II	II
		Wind Input Data					
V_b(m/sec)		50					
Terrain		2					
k_1, k_3, k_4		1.0 (k_4 not introduced IS 875 (Part-3): 1987, introduced revision at 2015)					
K_2		As per IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015 respectively					
Damping		0.016	0.02	0.016	0.02	0.016	0.02
H/B		4.47		5.54		6.61	
L/B		1.03		1.03		1.03	
Windward		0°= 0.80, 90°= 0.80		0°= 0.80, 90°= 0.80		0°= 0.95, 90°= 0.95	
Leeward		0°= 0.25, 90°= 0.25		0°= 0.25, 90°= 0.25		0°= 1.25, 90°= 1.25	

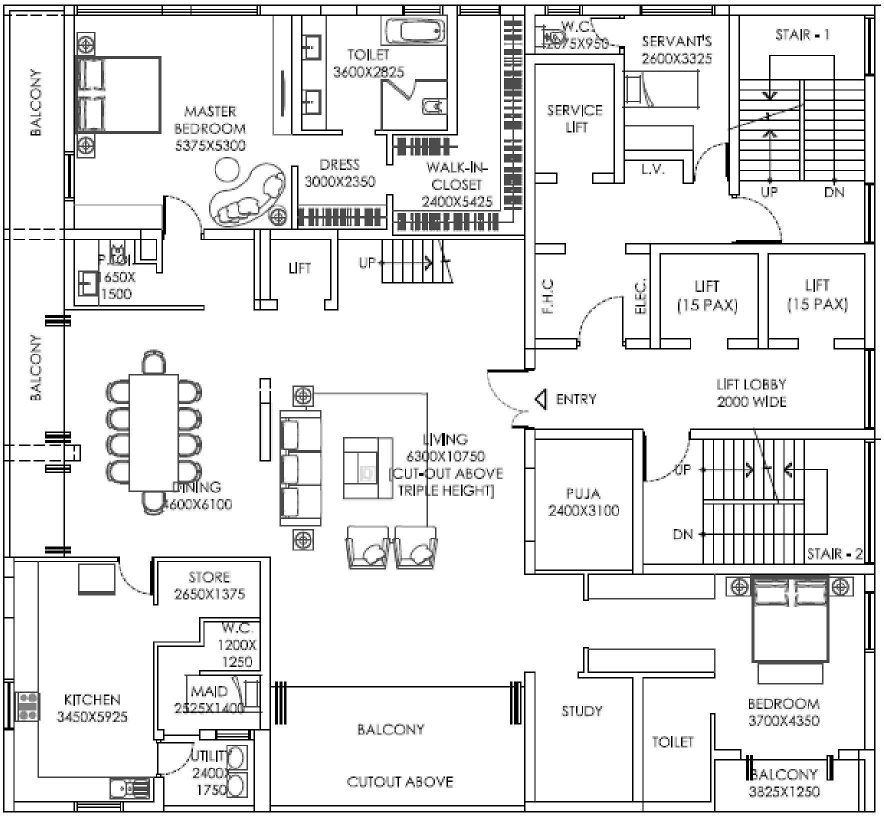


Fig 5.12A Architectural Typical Floor Plan of G+24, G+30 and G+36 Storied Buildings

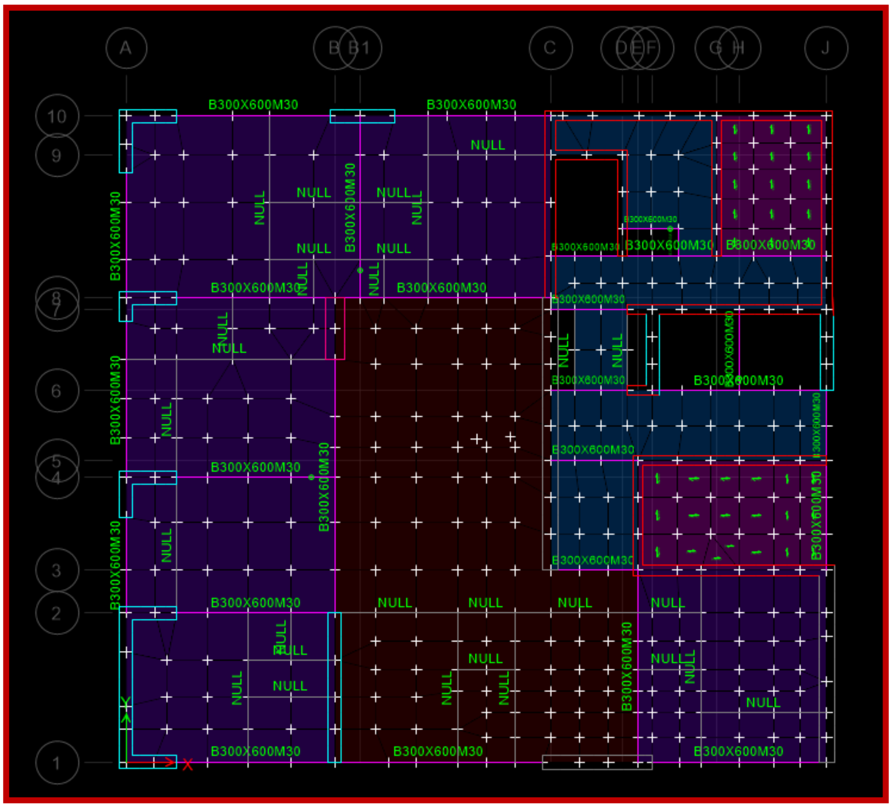


Fig 5.12B Structural Framing Plan of G+24, G+30 and G+36 Storied Buildings

DESCRIPTION OF STRUCTURAL WALLS:

- 300mm Thickness Structural Wall*
- 450mm Thickness Structural Wall*
- 500mm Thickness Structural Wall*
- 600mm Thickness Structural Wall*

*G+24 Story- (Fdn-8th F)- M50 Grade, (8th -16th F)- M40 Grade, (16th -R)- M30 Grade.

*G+30 Story- (Fdn-10th F)- M50 Grade, (10th -20th F)- M40 Grade, (20th -R)- M30 Grade.

*G+36 Story- (Fdn-12th F)- M50 Grade, (12th -24th F)- M40 Grade, (24th -R)- M30 Grade

DESCRIPTION OF STRUCTURAL SLABS:

- 125mm Thickness Slab- M30 Grade (All Buildings Fdn-R)
- 175mm Thickness Slab- M30 Grade (All Buildings Fdn-R)
- 200mm Thickness Slab- M30 Grade (All Buildings Fdn-R)
- 250mm Thickness Slab- M30 Grade (All Buildings Fdn-R)

DESCRIPTION OF STRUCTURAL BEAMS:

- Beam size (300X600)- M30 Grade (All Buildings Fdn-R)

5.3.1. STRUCTURAL RESPONSE:

I. COMPARATIVE STUDY OF MODAL RESULT

For analysis 12 nos. of modes are considered to obtain a combined modal mass participation of at least 90% of the actual mass in each of the orthogonal horizontal directions of response considered by the model. The dynamic response of different building models has been enlisted in following table 5.13A to table 5.13F shows below.

Table 5.5A Modal Participating Mass Ratio of Building G24-2002, G+24 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	2.272	0.6416	0.0002	0.043	0.6416	0.0002	0.043
2	1.601	0.0005	0.6544	0.0345	0.6421	0.6545	0.0775
3	1.135	0.0288	0.0262	0.6624	0.6709	0.6806	0.7399
12	0.111	0.0121	0.0003	0.007	0.9503	0.9379	0.9363

Table 5.5B Modal Participating Mass Ratio of Building G24-2016700, G+24 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	2.581	0.6338	0.0001	0.0443	0.6338	0.0001	0.0443
2	1.775	0.0004	0.6456	0.0393	0.6342	0.6457	0.0836
3	1.238	0.0298	0.0303	0.6512	0.664	0.676	0.7348
12	0.119	0.0124	0.0003	0.0071	0.9498	0.9374	0.9357

Table 5.5C Modal Participating Mass Ratio of Building G30-2002, G+30 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	3.129	0.6513	0.0005	0.0312	0.6513	0.0005	0.0312
2	2.242	0.0001	0.6604	0.0202	0.6513	0.6609	0.0514
3	1.502	0.0184	0.0127	0.691	0.6698	0.6736	0.7424
12	0.153	0.0134	0.0001	0.0061	0.9435	0.9318	0.9301

Table 5.5D Modal Participating Mass Ratio of Building G30-2016700, G+30 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	3.594	0.643	0.0011	0.032	0.643	0.0011	0.032
2	2.493	0.0003	0.652	0.0259	0.6434	0.6531	0.0579
3	1.651	0.0189	0.0168	0.6792	0.6623	0.6699	0.737
12	0.164	0.0139	0.0007	0.006	0.9429	0.9312	0.9293

Table 5.5E Modal Participating Mass Ratio of Building G36-2002, G+36 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	4.085	0.6575	0.0009	0.0243	0.6575	0.0009	0.0243
2	2.987	0.0005	0.6609	0.0121	0.658	0.6618	0.0364
3	1.888	0.0119	0.0062	0.7088	0.6699	0.668	0.7452
12	0.199	0.0145	0.0002	0.0048	0.9388	0.9276	0.9255

Table 5.5F Modal Participating Mass Ratio of Building G36-2016700, G+36 Storied

Mode	Period (sec)	UX	UY	RZ	Sum UX	Sum UY	Sum RZ
1	4.735	0.6485	0.0023	0.025	0.6485	0.0023	0.025
2	3.322	0.0016	0.6536	0.0173	0.6501	0.6558	0.0423
3	2.09	0.0122	0.0094	0.6973	0.6623	0.6653	0.7396
12	0.214	0.0154	0.0005	0.0043	0.9381	0.9269	0.9247

II. COMPARATIVE STUDY OF TIME PERIOD

Natural frequency and natural time period of the structure depends on mass and stiffness of the structure and the mode of vibration depends on the structural framing plan, base dimension of the structure, height of the building and foundation soil profile. The fundamental natural time period of 1st mode of vibration is obtained numerically for different structures, which are shown in Fig 5.13

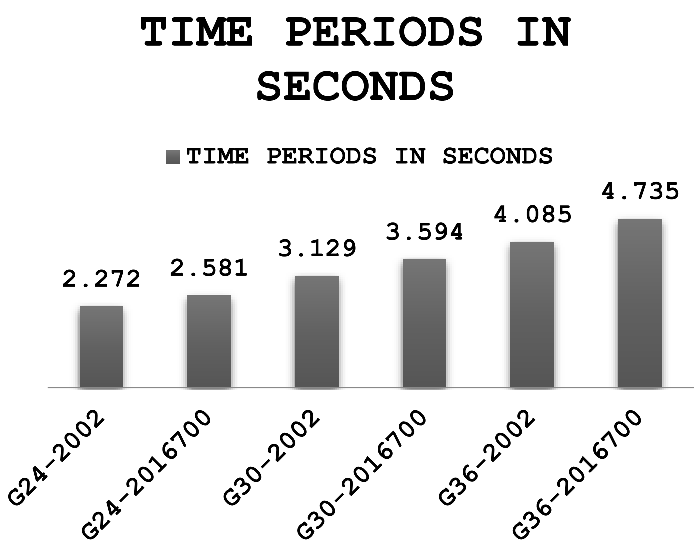


Fig 5.13 Fundamental Time Period for Different Type of Buildings

If the height of the buildings increases gradually the structures become more flexible, so the time period of the structures increases accordingly as shown in Fig 5.13. Although uses of modifier in structural walls, slabs and beams as per IS 16700: 2017 reduces the stiffness of the structural elements, so time period of the research models G24-2016700, G30-2016700 and G36-2016700 are increased with respect to G24-2002, G30-2002 and G36-2002 respectively.

III. STRUCTURAL RESPONSE FOR SEISMIC FORCES

The seismic analysis has been performed for all buildings based on the input data given in table 5.1. The summary of building responses due to seismic loading are compared between different numerical models, which are shown below

1. STOREY LEVEL VS STOREY SHEAR

The storey shears are developed along X and Y directions due to Seismic forces at different storey levels for different building models are reflected in Fig 5.14A and Fig 14.3B

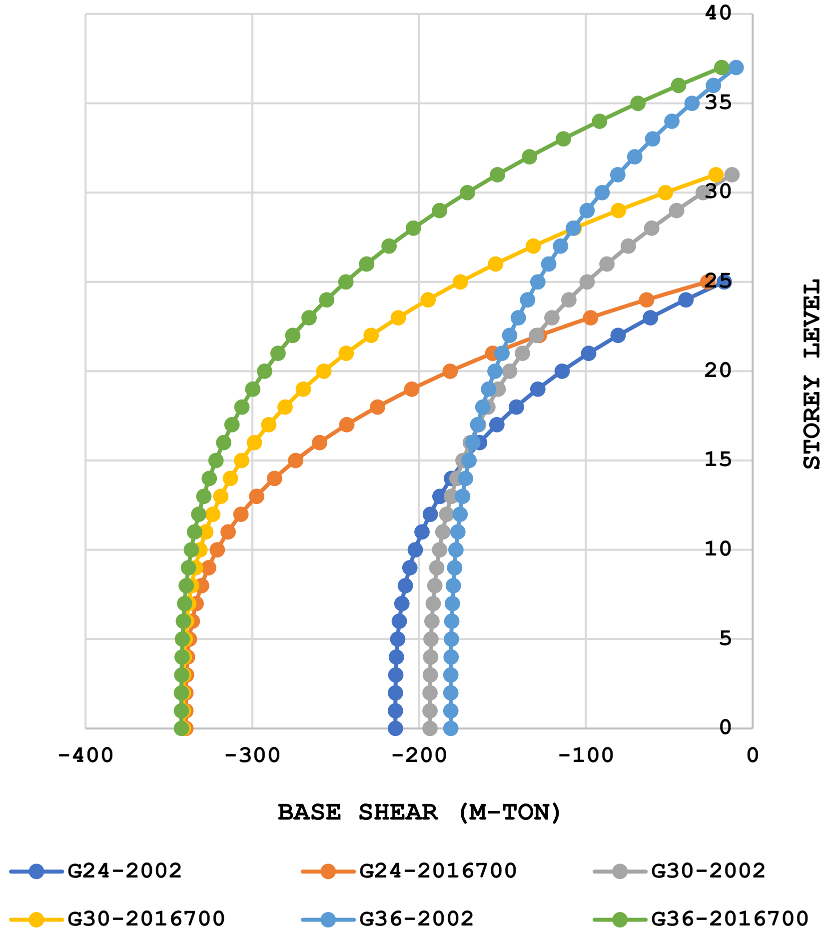


Fig 5.14A Storey Level Vs Storey Shear (Seismic X-Direction)

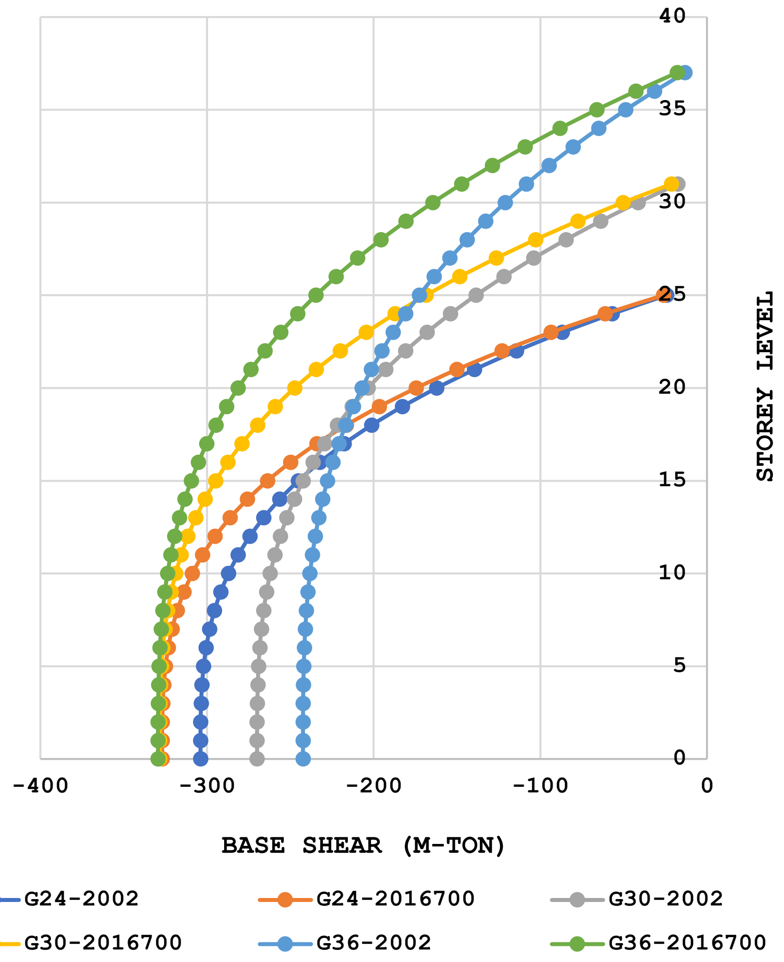


Fig 5.14B Storey Level Vs Storey Shear (Seismic Y-Direction)

It is observed from the graphical representation of storey level vs storey shear, Fig 5.14A and Fig 5.14B that the research models which are analyzed as per design stipulations of old code, the base shear of those models are decreased with height increment even though the seismic mass of the models are increased with increment of building height. Because having same framing configuration, the height increment increased the natural time period of the structures. So that the structures are became more flexible. The value of S_a/g is less at certain stage for flexible structure. So that the base share of tallest structure is reduced than the taller structure, even though the seismic mass of the structure is more. On other hand analyses as per new code, by using stiffness modifiers the stiffness of the structure is reduced from previous one. So that the time period of the structure is increased and the structure becomes flexible.

2. STOREY LEVEL VS OVERTURNING MOMENT

The overturning moments developed due to seismic forces along X and Y directions at different storey level for different building models have been reflected in Fig 5.15A and Fig 5.15B

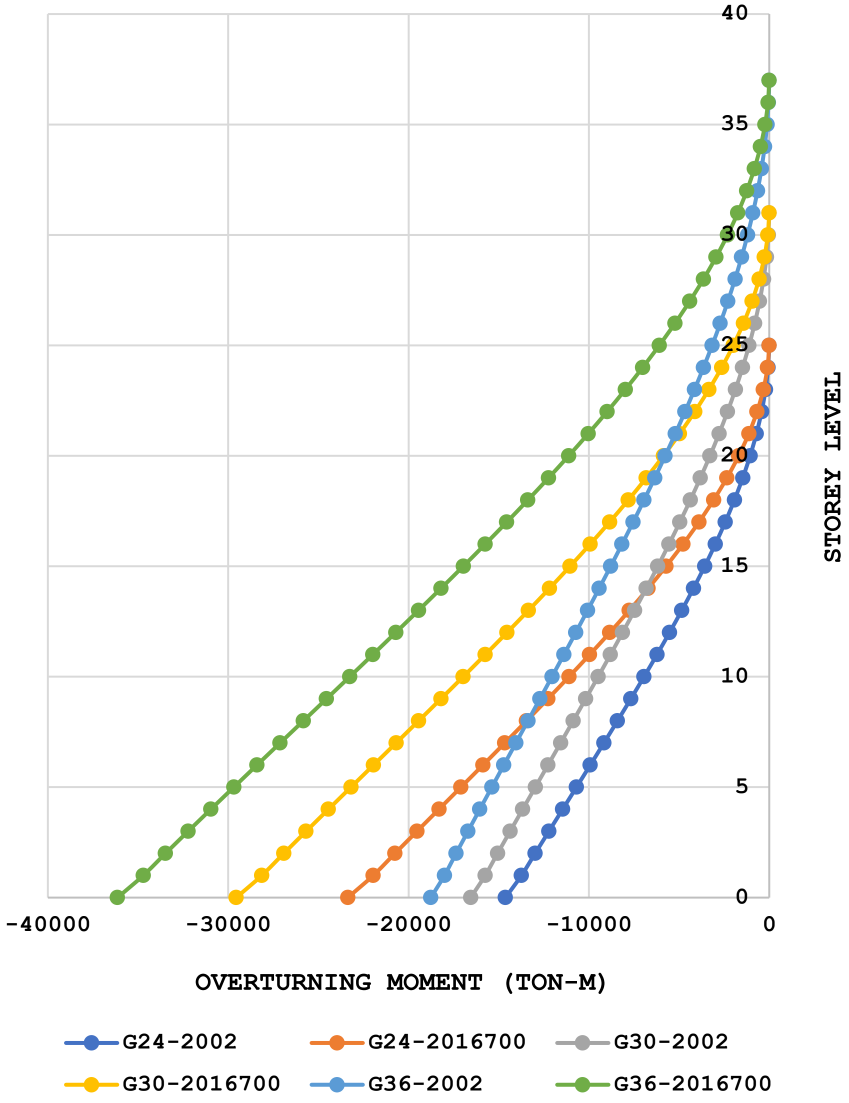


Fig 5.15A Storey Level Vs Overturning Moment (Seismic X-Direction)

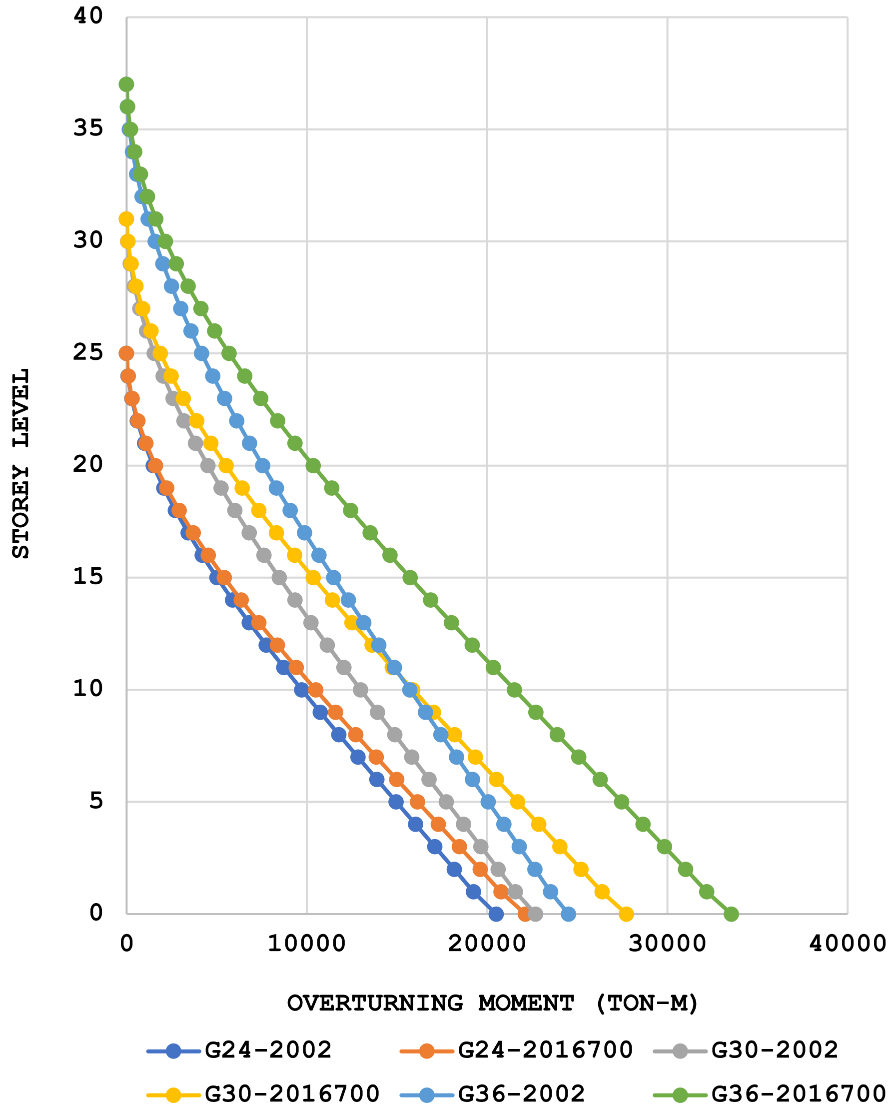


Fig 5.15B Storey Level Vs Overturning Moment (Seismic Y-Direction)

Overturning moment is caused by storey force developed at different floor level for ground movement due to transient forces. The value of overturning moment is depends on storey shears and height of the structure. It is observed from Fig 5.15A and Fig 5.15B that the overturning moment of G+36 Storey buildings are more than G+30 storey and G+24 storey buildings even though the summation of storey shear is quite less (refer Fig 5.14A and Fig 5.14B). Analyses as per new design stipulations of IS 1893 (Part-I): 2016 the base shear of the buildings increase due to incremental value of importance factor from 1.0 to 1.2 for same height buildings. So, those buildings which are analyzed as per new codal stipulations the overrunning moments at both the directions are increased.

3. STOREY LEVEL VS STOREY DISPLACEMENT

The storey displacement caused by seismic forces along X and Y directions at different storey levels for all building types have been shown in Fig 5.16A and Fig 5.16B

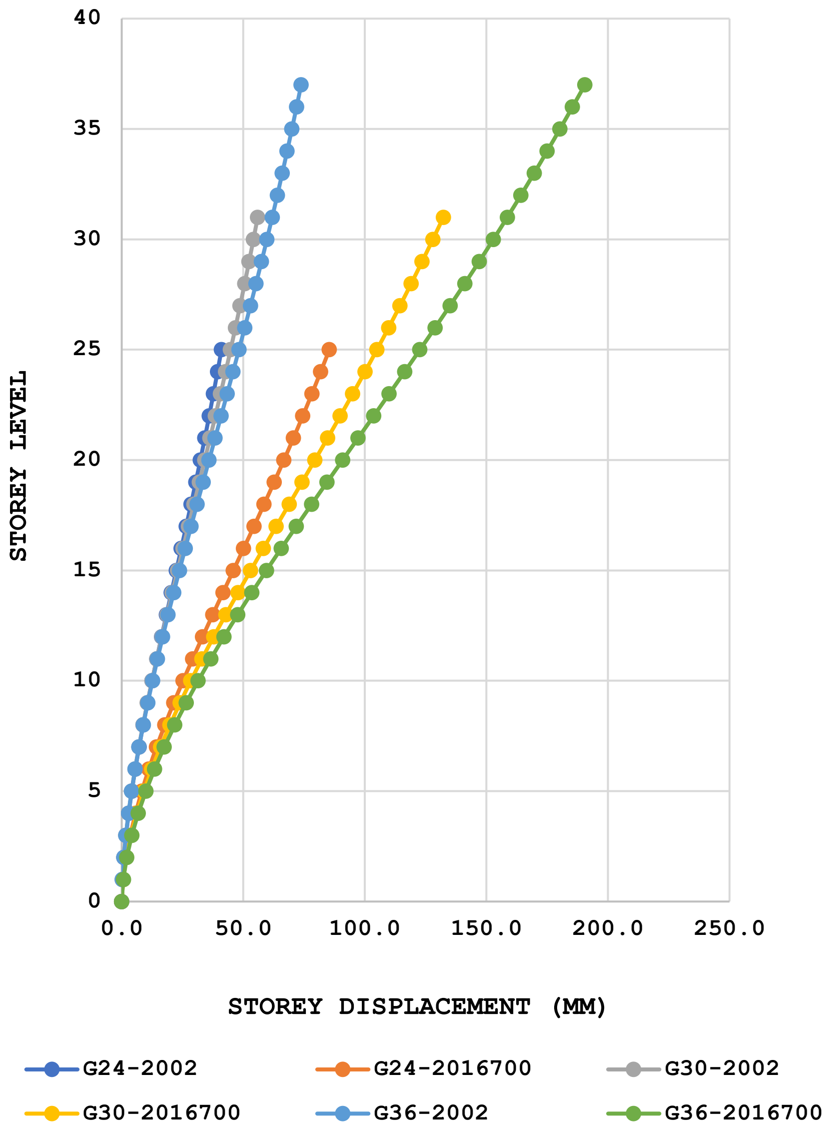


Fig 5.16A Storey Level Vs Storey Displacement (Seismic X-Direction)

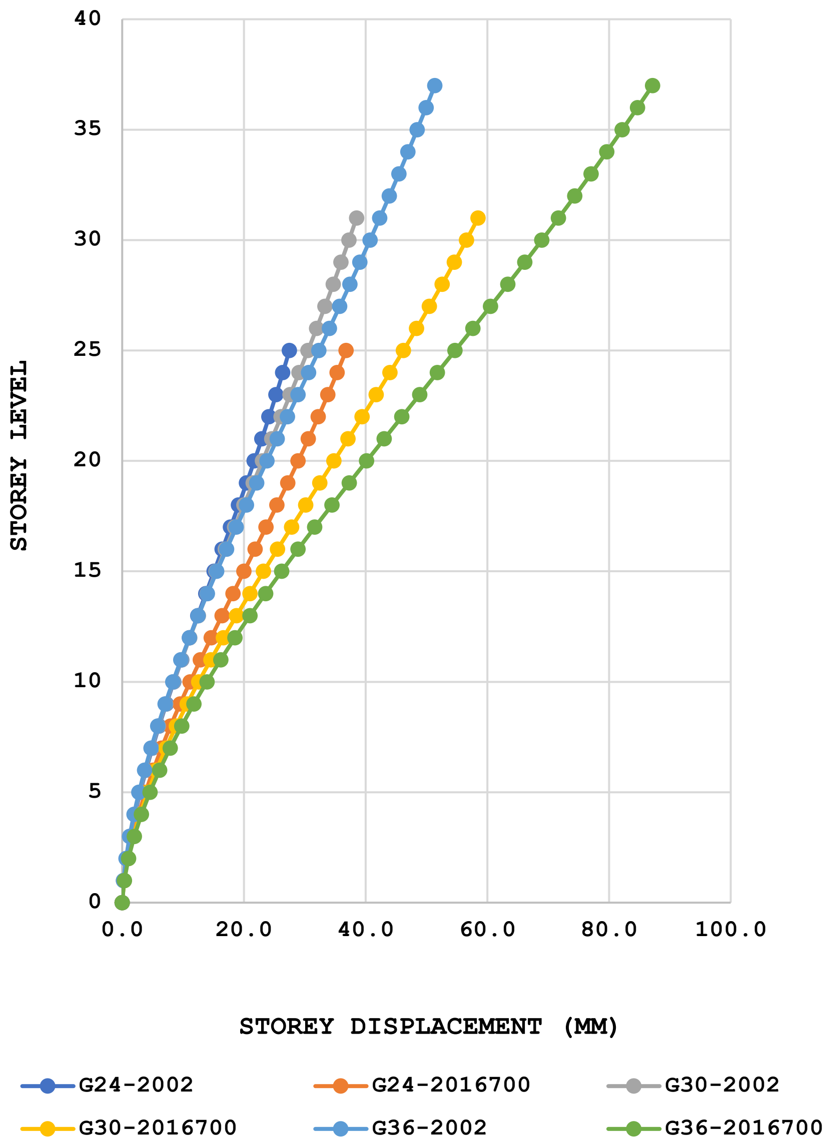


Fig 5.16B Storey Level Vs Storey Displacement (Seismic Y-Direction)

It is observed from above graphical representation of Fig 5.16A and Fig 5.16B that the storey displacement of buildings marked G24-2016700, G30-2016700 and G36-2016700 are more than the buildings marked G24-2002, G30-2002 and G36-2002 respectively also the height increment increases the differential displacements of respective building. Stiffness modifier in structural elements are reduced the stiffness and increased flexibility of the structure even though the structural configurations are same. Also height increment is possessed more flexible structure, so that the storey displacement of G+36 storied buildings (G36-2002 and G36-2016700) are more than the G+30 storied buildings (G30-2002 and G30-2016700) and G+24 storied buildings (G24-2002 and G24-2016700) respectively.

4. STOREY LEVEL VS STOREY DRIFT

The storey drift caused by seismic forces along X and Y directions at different storey levels for all building types have been shown in Fig 5.17A and Fig 5.17B

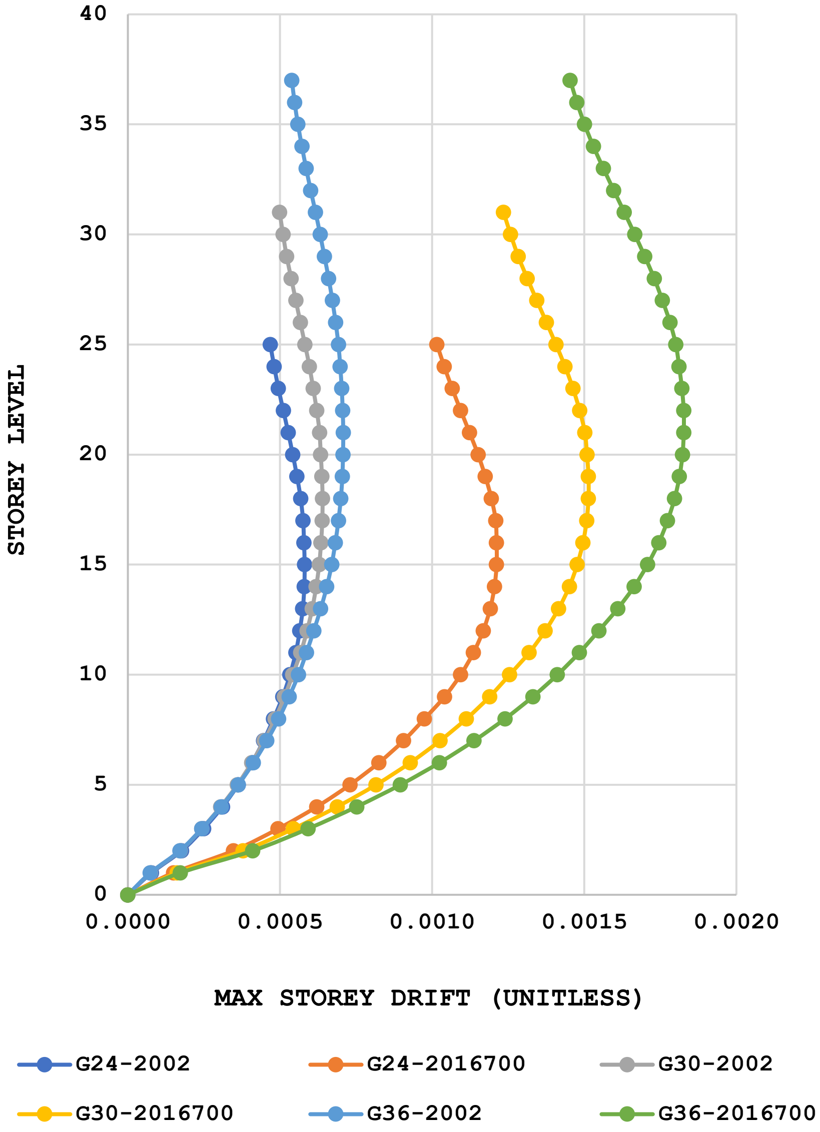


Fig 5.17A Storey Level Vs Storey Drift (Seismic X-Direction)

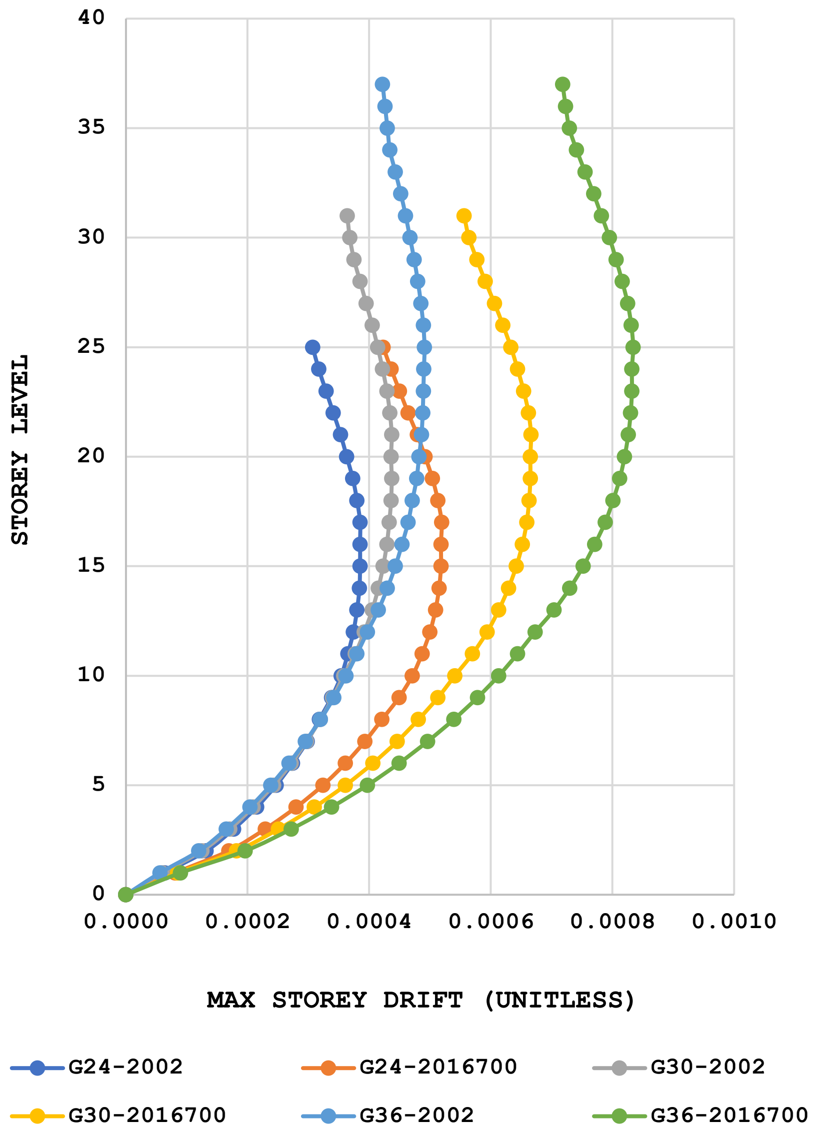


Fig 5.17B Storey Level Vs Storey Drift (Seismic Y-Direction)

Storey drift is unit less because it is ratio of storey displacement of respective floor to storey height. From the graphical representation of Fig 5.17A and Fig 5.17B it is noted that all buildings experiences max drift at top one third of the total building height for seismic loading and with increase of storey level the value of drift declines. In all the cases the storey drifts are within the permissible limit h/250, where h is defined respective storey height of that building. From figures it is observed that the storey drift of buildings marked G24-2016700, G30-2016700 and G36-2016700 are more than the buildings marked G24-2002, G30-2002 and G36-2002. Also storey drift at all floor levels of G+24 storey building is less than G+30 and G+36 storey building due to their varying stiffness and height, because using of stiffness modifier as per IS 16700: 2007 and height increment of the buildings respectively.

5. STOREY LEVEL VS STOREY STIFFNESS

The storey stiffness along X and Y directions due to Earthquake loading at different storey levels for different building models are depicted in Fig 5.18A and Fig 5.18B

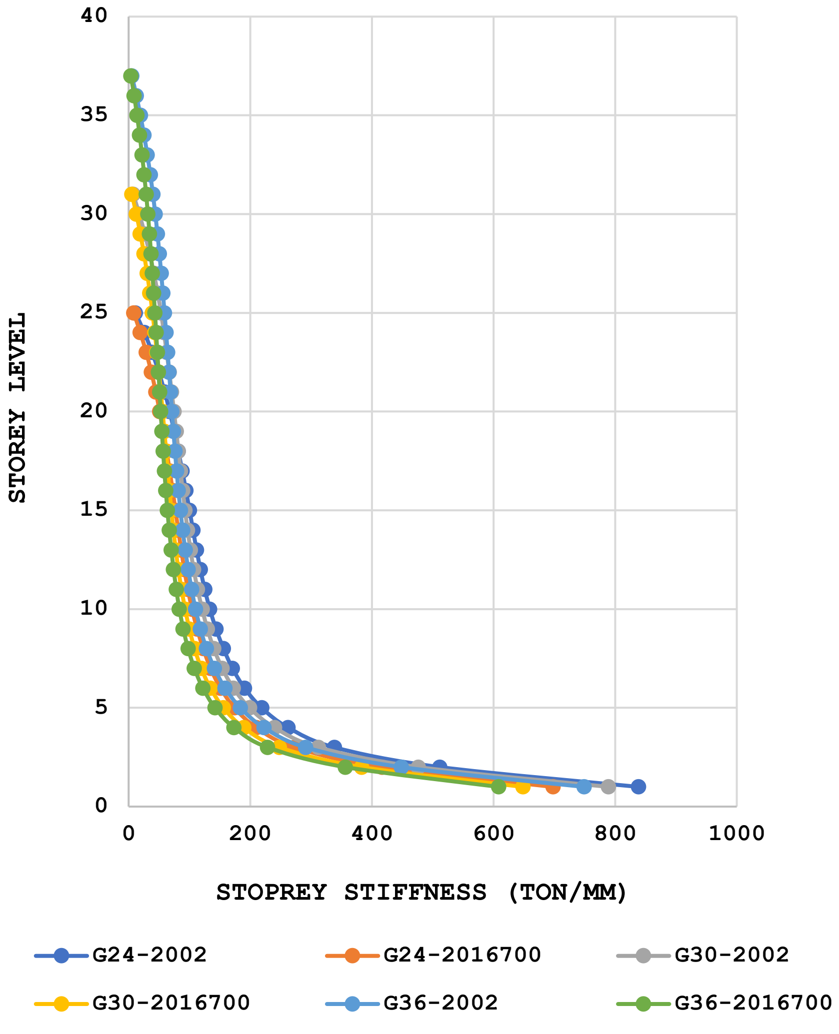


Fig 5.18A Storey Level Vs Storey Stiffness (Seismic X-Direction)

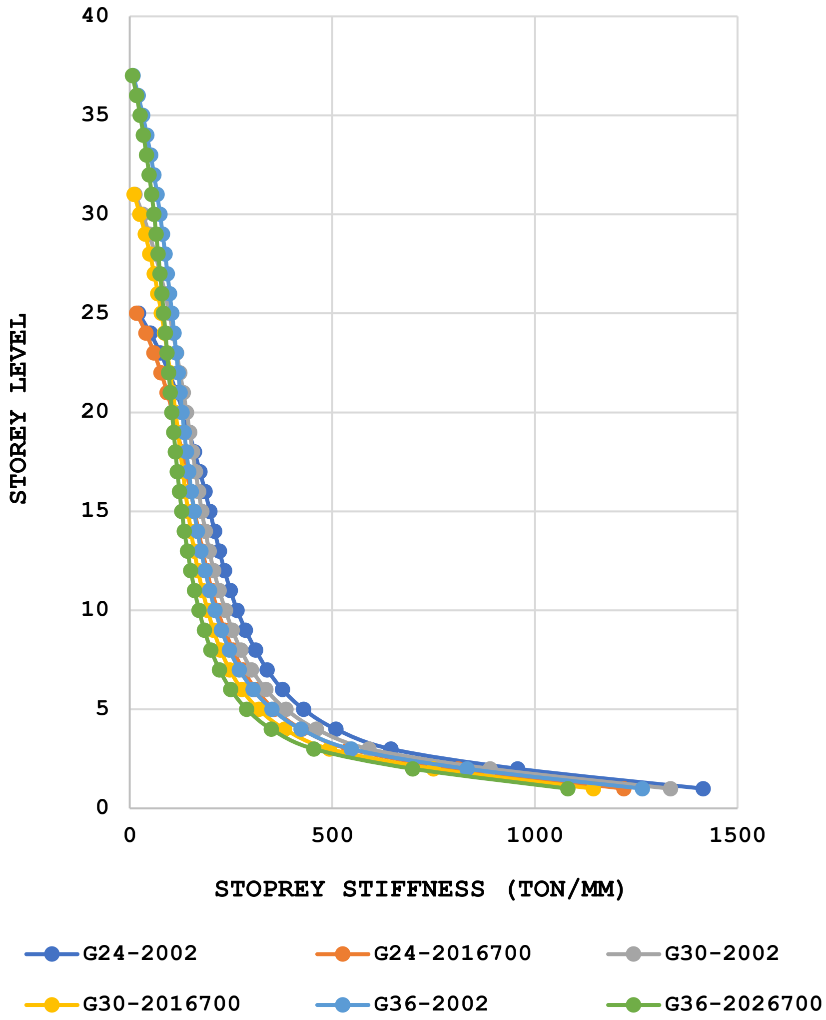


Fig 5.18B Storey Level Vs Storey Stiffness (Seismic Y-Direction)

From Fig 5.18A and Fig 5.18B it is studied that the storey stiffness is gradually reduced from 1st floor level to roof level for all types of building models. Though structural wall configurations are remaining same and the storey height at 1st floor level is higher than the above floors, due to higher elastic modulus the stiffness at 1st floor level is more than the above floor levels. On other hand the stiffness of research models G24-2016700, G30-2016700 and G36-2016700 are less than the stiffness of models G24-2002, G30-2002 and G36-2002. Also the height increment of structure causes lesser stiffness compared with low height structures. For this kind of result, it is concluded that even though the structural arrangement, element sizes, material properties and loading of the structures are remaining same, the height increment and reduction of storey stiffness is reduced due to implementation of stiffness modifier in structural walls, columns and beams as per table no. 6 of IS 16700: 2017, is reduced the gross moment of inertia of the structural elements resulting reduction in storey stiffness.

IV. STRUCTURAL RESPONSE FOR STATIC WIND FORCES

The wind analysis has been performed for all buildings considered based on the input data given in table 5.4. The summary of building responses due to static wind loading are compared between different numerical research models, which are shown below

1. STOREY LEVEL VS STOREY SHEAR

The storey shear developed along X and Y directions due to wind load applicable at different storey levels for building models have been reflected in Fig 5.19A and Fig 5.19B

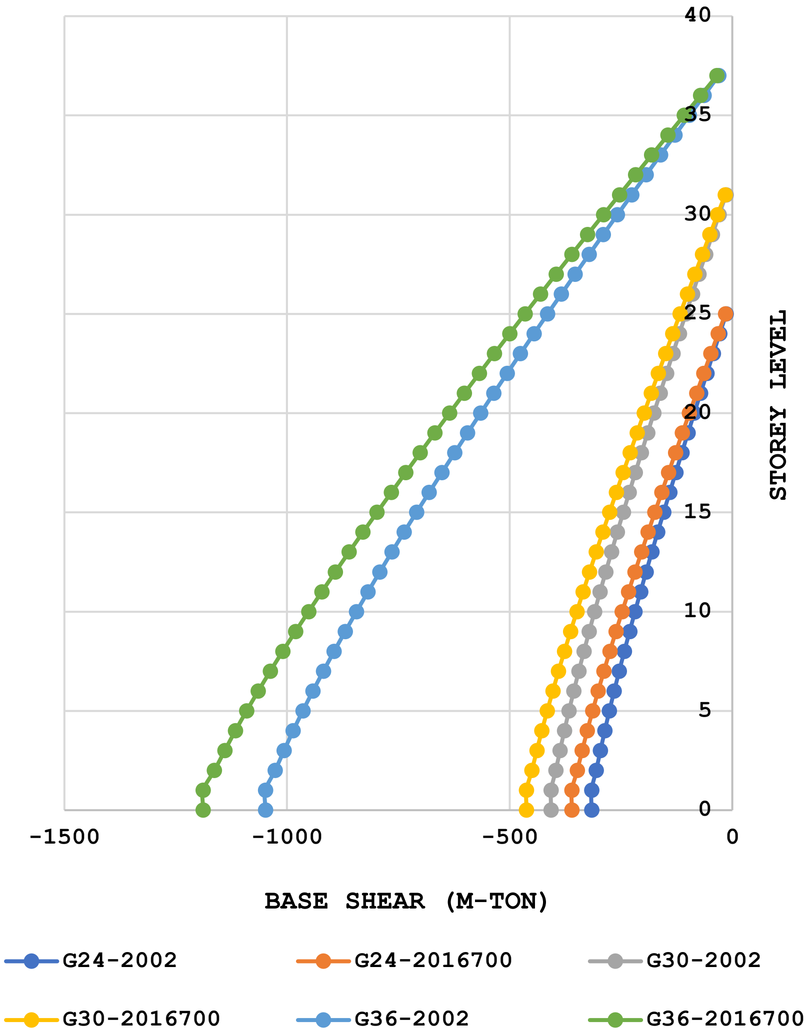


Fig 5.19A Storey Level Vs Storey Shear (Wind X-Direction)

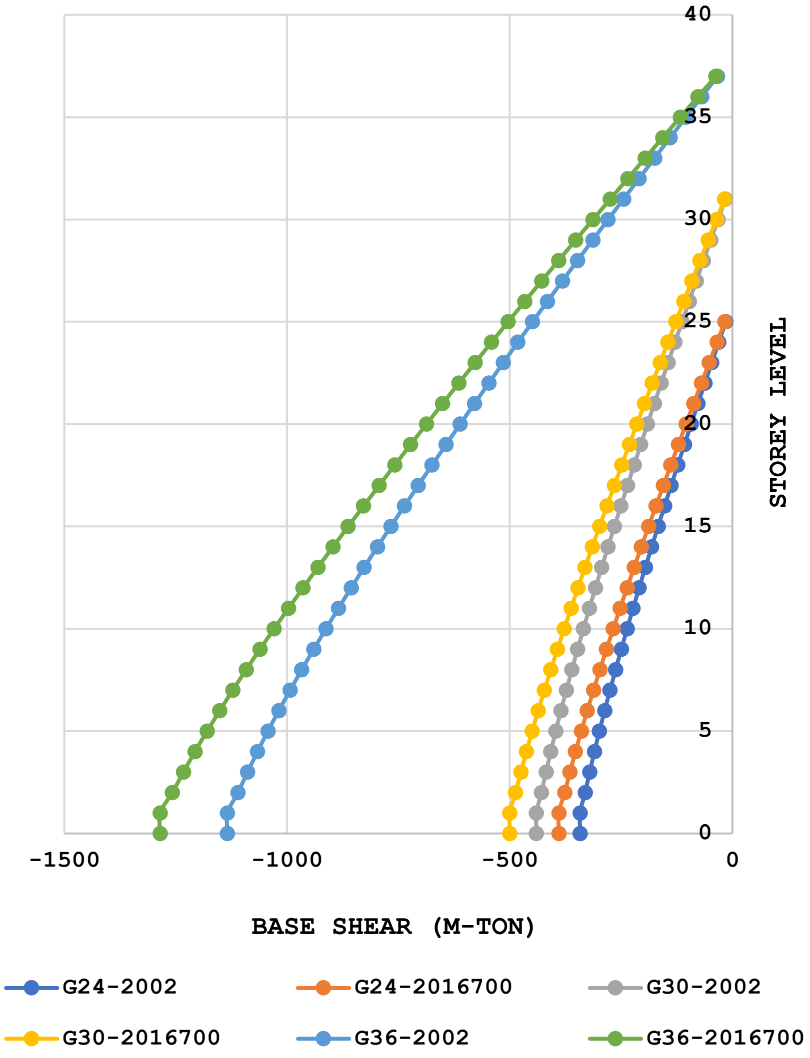


Fig 5.19B Storey Level Vs Storey Shear (Wind Y-Direction)

It is observed from Fig 5.19A and Fig 5.19B that the total applied wind force on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. In this new code a factor k_4 (importance factor) is introduced and also the value of k_2 with respect to the height of the structure seems to be higher side in this latest revision. Although the wind force is calculated with same amount of exposed area and obviously considered wind speed is constant. Amount of static wind force applied on structure does not depend on inherent properties of the structure. On other hand due to height increment of structures the wind pressures as well as the exposed area of the structures are increased, for that reason the storey shears of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-2002 and G24-2016700) respectively.

2. STOREY LEVEL VS OVERTURNING MOMENT

The overturning moment developed along X and Y directions due to wind force applicable at different storey levels for different building models have been reflected in Fig 5.20A and Fig 5.20B

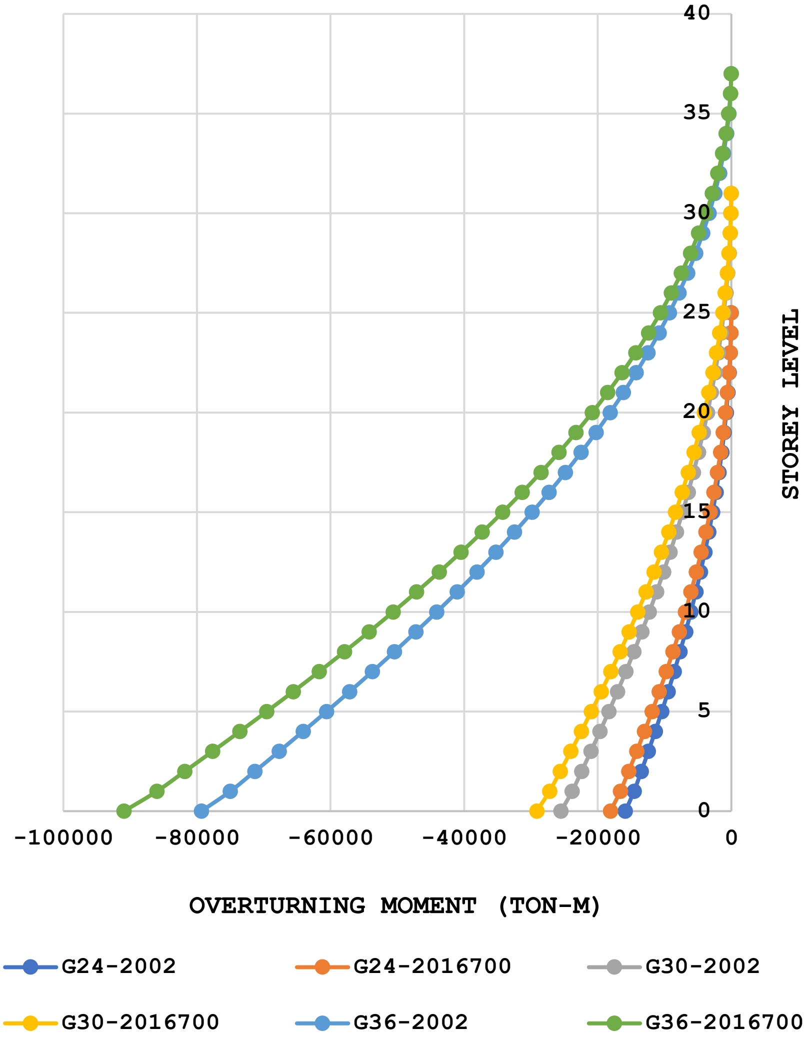


Fig 5.20A Storey Level Vs Overturning Moment (Wind X-Direction)

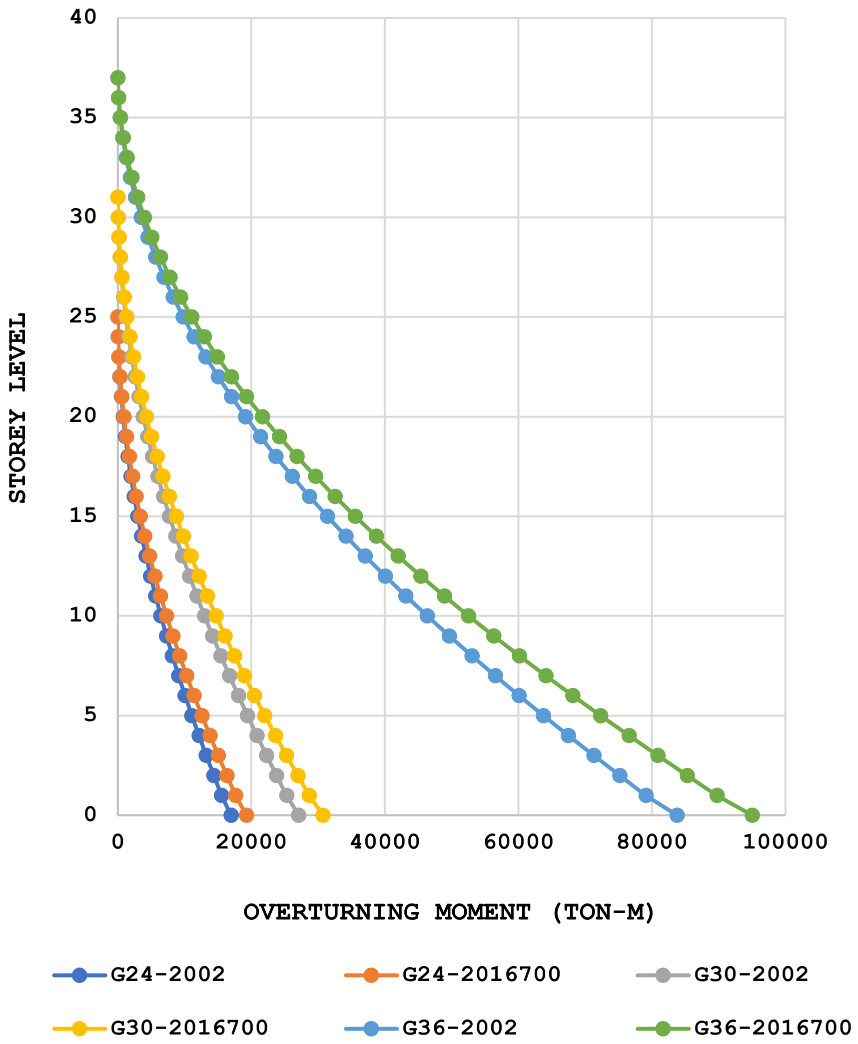


Fig 5.20B Storey Level Vs Overturning Moment (Wind Y-Direction)

It is observed from Fig 5.19A and Fig 5.19B that the calculated storey shears on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. The value of overturning moment is depends on storey shear and height of the structure. It is observed from Fig 5.20A and Fig 5.20B that, due to storey shear the overturning moment of research models G24-2016700, G30-2016700 and G36-2016700 are greater than models G24-2002, G30-2002 and G36-2002. Also the overturning moment of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-2002 and G24-2016700) due to height increment.

3. STOREY LEVEL VS STOREY DISPLACEMENT

The storey displacement caused by wind force along X and Y directions at different storey levels for different building models have been shown in Fig 5.21A and Fig 5.21B

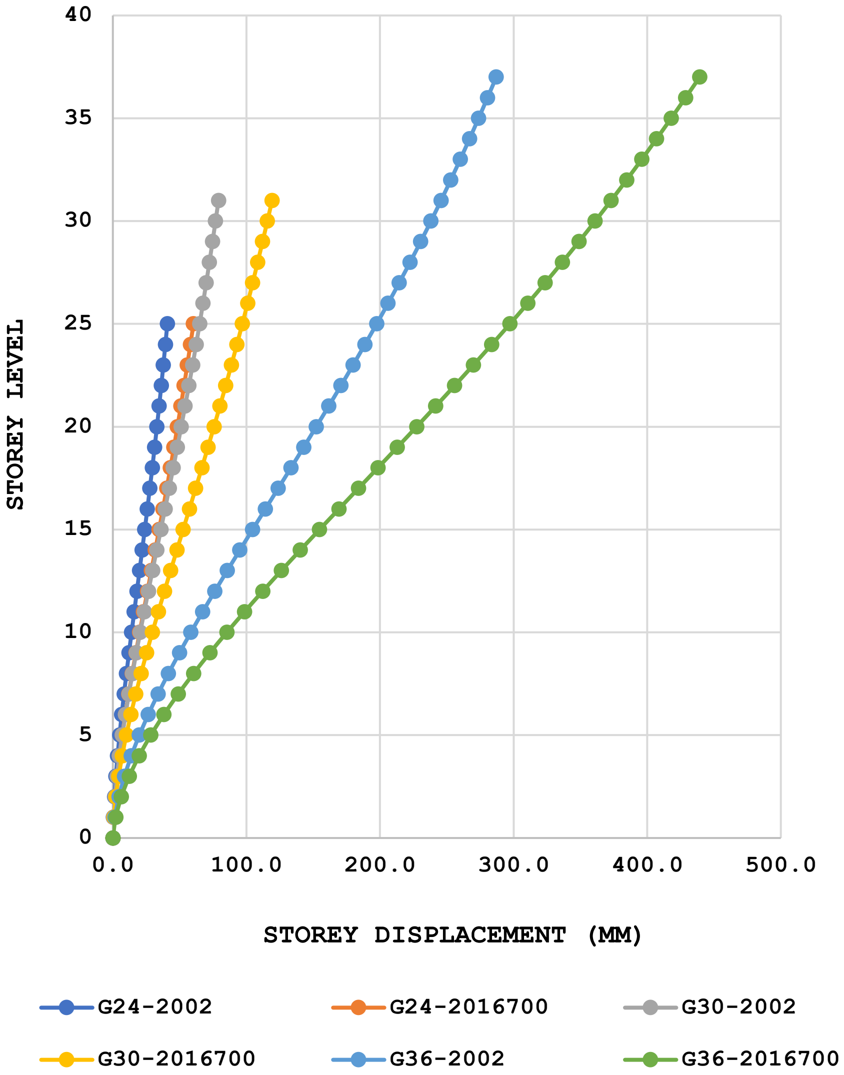


Fig 5.21A Storey Level Vs Storey Displacement (Wind X-Direction)

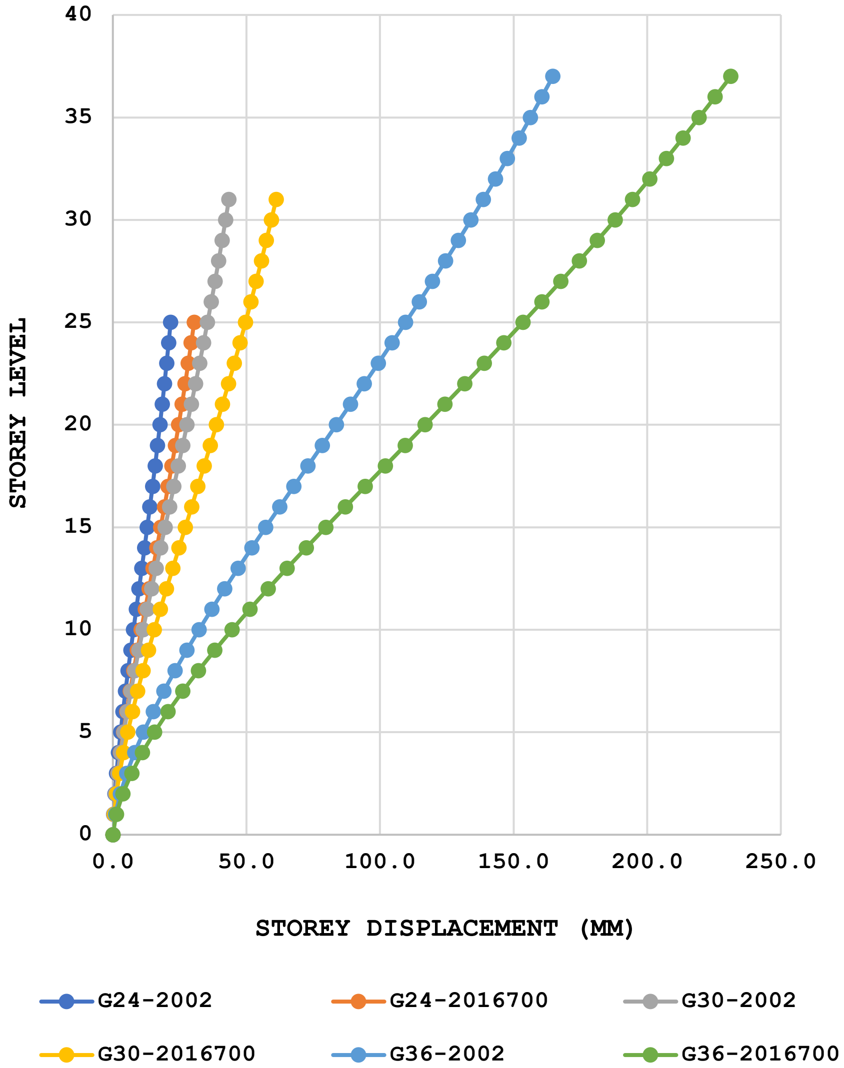


Fig 5.21B Storey Level Vs Storey Displacement (Wind Y-Direction)

It is observed from above graphical representations, Fig 5.21A and Fig 5.21B, that storey displacement of research models G24-2016700, G30-2016700 and G36-2016700 are greater than the storey displacement of models G24-2002, G30-2002 and G36-2002 because the storey shears of models G24-2016700, G30-2016700 and G36-2016700 at different levels are more than the storey shears of models G24-2002, G30-2002 and G36-2002 respectively. Also the research models G24-2016700, G30-2016700 and G36-2016700 are less stiff than research models G24-2002, G30-2002 and G36-2002 respectively due to using of stiffness modifier in structural wall, columns and beams as per table 6 of IS 16700: 2017. Also the storey displacement of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-

2002 and G24-2016700) due to differential height respectively. But maximum storey displacements at roof level of these all models are within the permissible limit (H/500, where H is the height of building including the parapet height) of respective Indian standards.

4. STOREY LEVEL VS STOREY DRIFT

The storey drift caused by wind forces along X and Y directions at different storey levels for all building types have been depicted in Fig 5.22A and Fig 5.22B

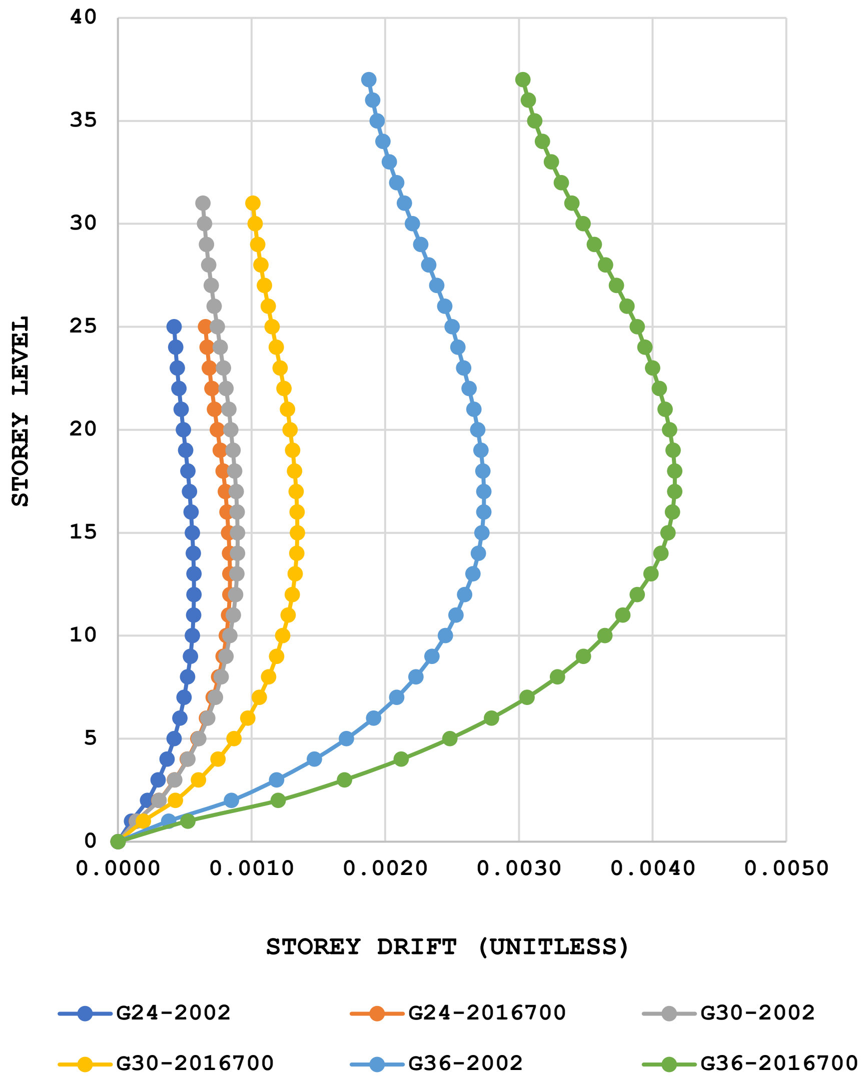


Fig 5.22A Storey Level Vs Storey Drift (Wind X-Direction)

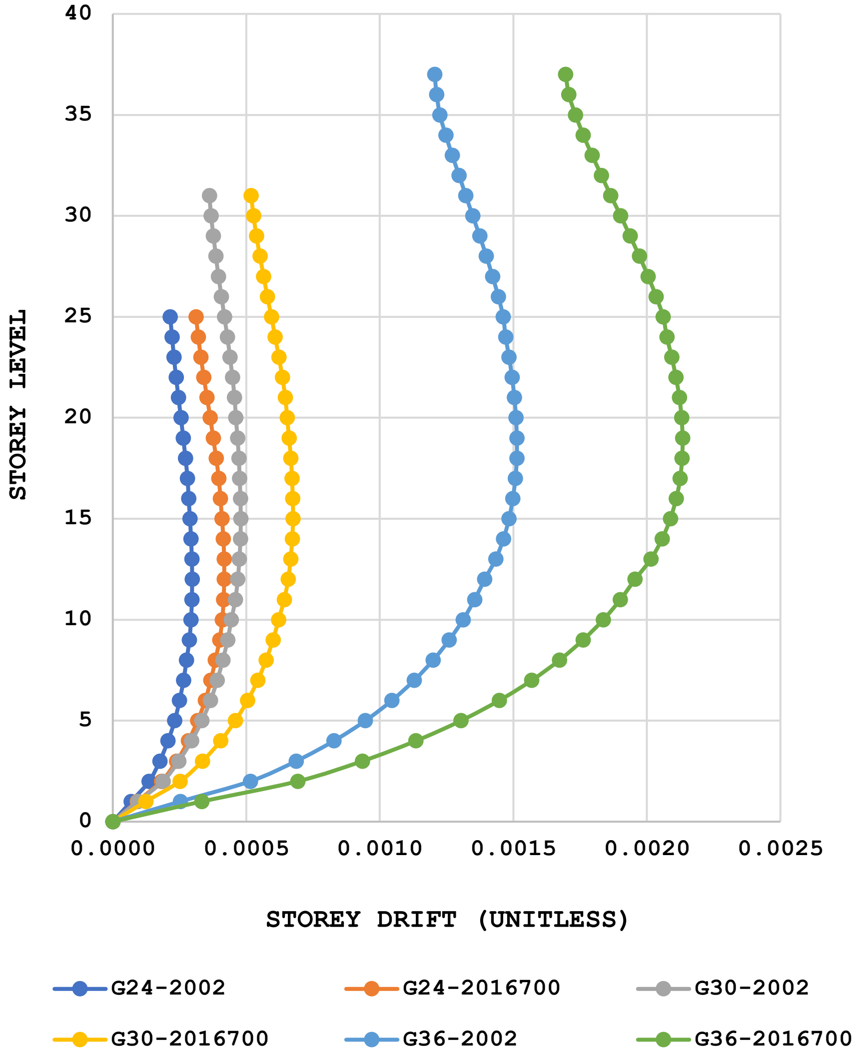


Fig 5.22B Storey Level Vs Storey Drift (Wind Y-Direction)

Storey drift is unit less because it is ratio of storey displacement of respective floor to storey height. From the graphical representation of Fig 5.22A and Fig 5.22B it is noted that all buildings experiences max drift at half of the total building height for wind loading and with increase of storey level the value of drift declines. In all the cases the storey drifts are within the permissible limit. From figures it is observed that the storey drift of research models G24-2016700, G30-2016700 and G36-2016700 are more than G24-2002, G30-2002 and G36-2002 due to their varying stiffness (using of stiffness modifier in models G24-2016700, G30-2016700 and G36-2016700) and wind storey shears respectively. Also the storey drift of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-2002 and G24-2016700) due to height difference respectively.

Table 5.6 Comparison of Structural Response due to Codal Revision for Problem No.-2

Item Description	Type of Loading	Direction of Loading	Type of Building Structure								
			Building G24-2002	Building G24-2016700	Change in Response (%)	Building G30-2002	Building G30-2016700	Change in Response (%)	Building G36-2002	Building G36-2016700	Change in Response (%)
Time Period (Sec) at 1st Mode			2.272	2.581	(+) 14	3.129	3.594	(+) 15	4.085	4.735	(+) 16
Base Shear (Ton)	Seismic Loading	EQX	214	340	(+) 59	193	341	(+) 77	181	343	(+) 90
		EQY	304	327	(+) 8	270	328	(+) 21	242	329	(+) 36
	Wind Loading	WLX	316	360	(+) 14	407	462	(+) 14	1049	1188	(+) 13
		WLY	342	389	(+) 14	440	500	(+) 14	1134	1285	(+) 13
Overturning Moment (Ton-m)	Seismic Loading	EQX	14625	23367	(+) 60	16554	29555	(+) 79	18762	36136	(+) 93
		EQY	20504	22119	(+) 8	22692	27736	(+) 22	24520	33557	(+) 37
	Wind Loading	WLX	15866	18096	(+) 14	25502	29122	(+) 14	79336	90911	(+) 15
		WLY	16961	19272	(+) 14	27106	30754	(+) 13	83788	95059	(+) 13
Storey Displacement (mm)	Seismic Loading	EQX	41.0	85	(+) 107	56	132	(+) 136	74	191	(+) 158
		EQY	27.5	37	(+) 35	39	59	(+) 51	51	87	(+) 71
	Wind Loading	WLX	40.8	60	(+) 47	79	119	(+) 51	287	439	(+) 53
		WLY	21.6	30	(+) 39	43	61	(+) 42	165	231	(+) 40
Maximum Storey Drift	Seismic Loading	EQX	0.0006	0.0012	(+) 100	0.0006	0.0015	(+) 150	0.0007	0.0018	(+) 157
		EQY	0.0004	0.0005	(+) 25	0.0004	0.0007	(+) 75	0.0005	0.0008	(+) 60
	Wind Loading	WLX	0.0006	0.0008	(+) 33	0.0009	0.0013	(+) 44	0.0027	0.0042	(+) 56
		WLY	0.0003	0.0004	(+) 33	0.0005	0.0007	(+) 40	0.0015	0.0021	(+) 40
Stiffness (Ton/mm)	Seismic Loading	EQX	838	698	(-) 17	789	649	(-) 18	749	601	(-) 20
		EQY	1415	1220	(-) 14	1335	1145	(-) 14	1266	1081	(-) 15

V. STRUCTURAL RESPONSE FOR DYNAMIC WIND FORCES

The wind analysis has been performed for all buildings considered based on the input data given in table 5.4. The summary of building responses due to dynamic wind loading are compared between different numerical research models, which are shown below

1. STOREY LEVEL VS STOREY SHEAR

The storey shear developed along X and Y directions due to wind load (Along Wind + Across Wind) applicable at different storey levels for building models have been reflected in Fig 5.23A and Fig 5.23B

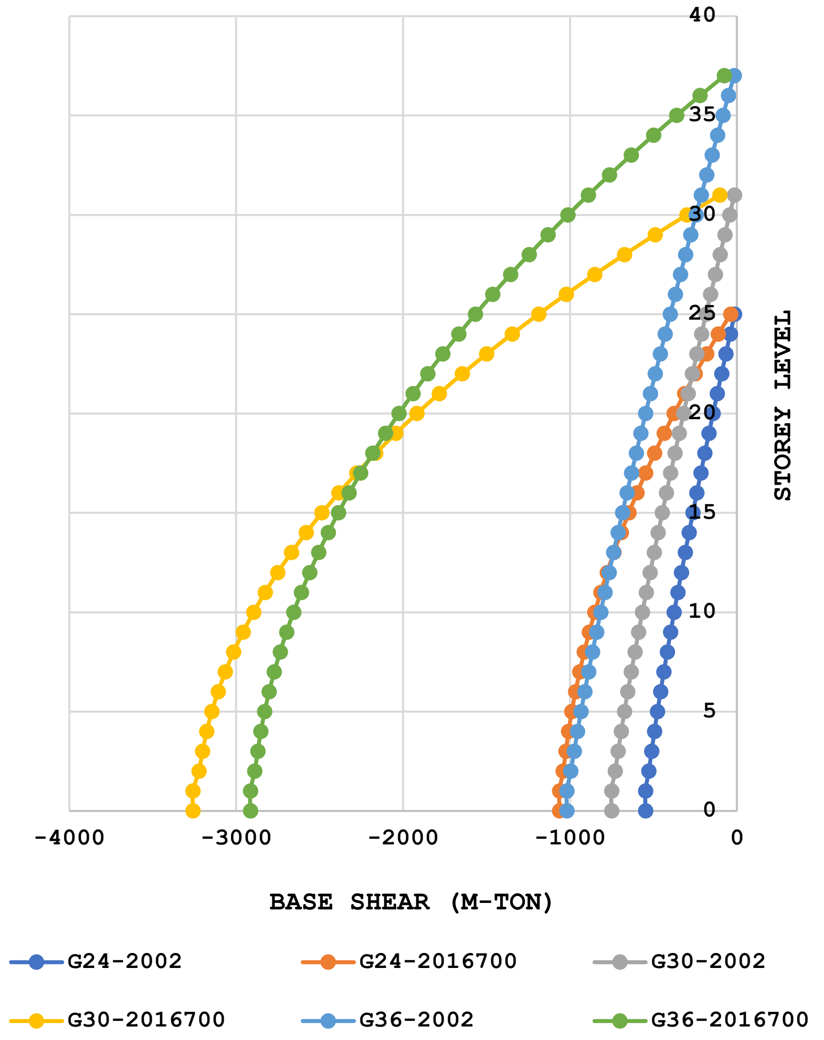


Fig 5.23A Storey Level Vs Storey Shear (Dynamic Wind X-Direction)

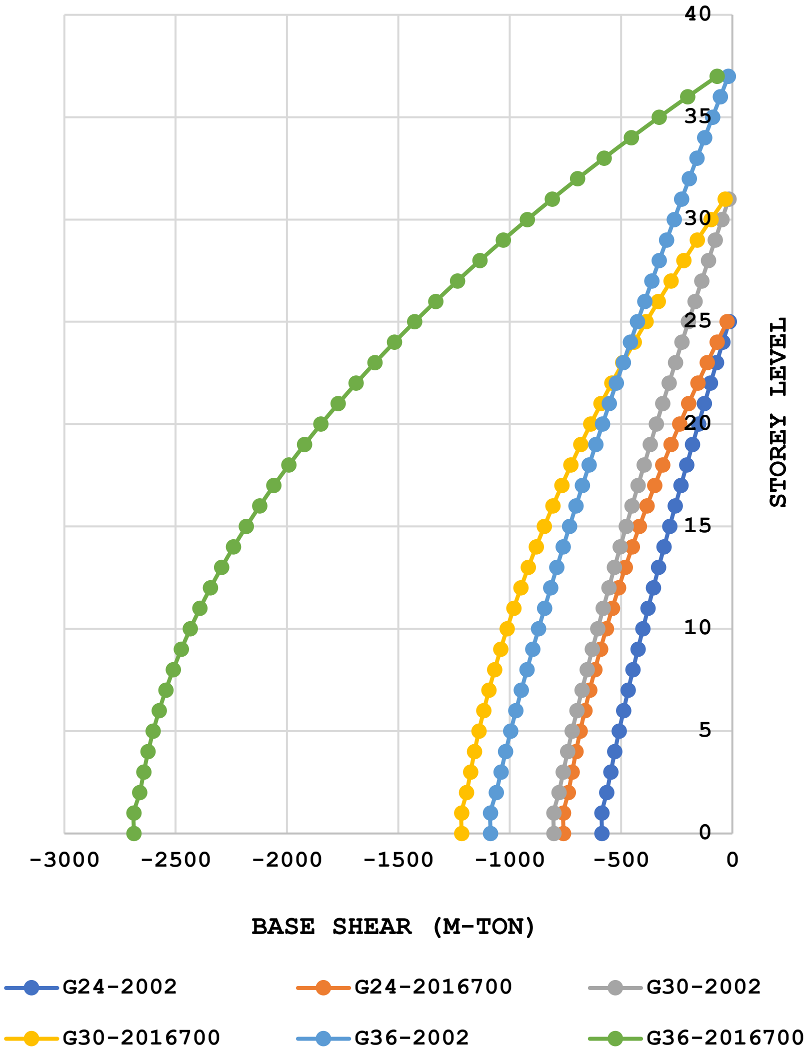


Fig 5.23B Storey Level Vs Storey Shear (Dynamic Wind Y-Direction)

It is observed from Fig 5.23A and Fig 5.23B that the total applied wind force on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. In this new revision of code a factor k_4 (importance factor) is introduced and also the value of k_2 with respect to the height of the structure seems to be higher side in this latest revision. Also across wind effect is considered with along wind for calculating design base shear as per latest revision of IS 875 (Part-3). So that the design values of base shears of latest models are much greater than the previous models, which are analyzed as per older version of IS 875 (Part-3). The across wind effect depends on inherent properties of the structure and the vertical as well as plan aspect ratio of the structures.

2. STOREY LEVEL VS OVERTURNING MOMENT

The overturning moment developed along X and Y directions due to wind force (Along Wind + Across Wind) applicable at different storey levels for different building models have been reflected in Fig 5.24A and Fig 5.24B

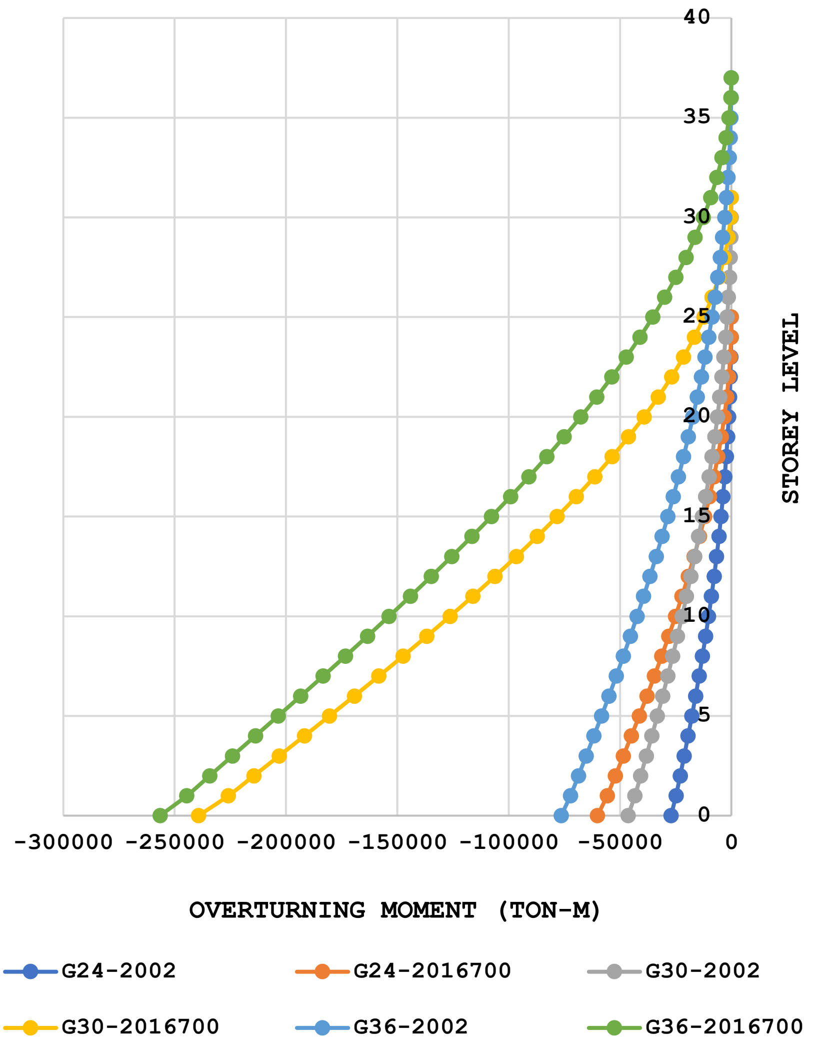


Fig 5.24A Storey Level Vs Overturning Moment (Dynamic Wind X-Direction)

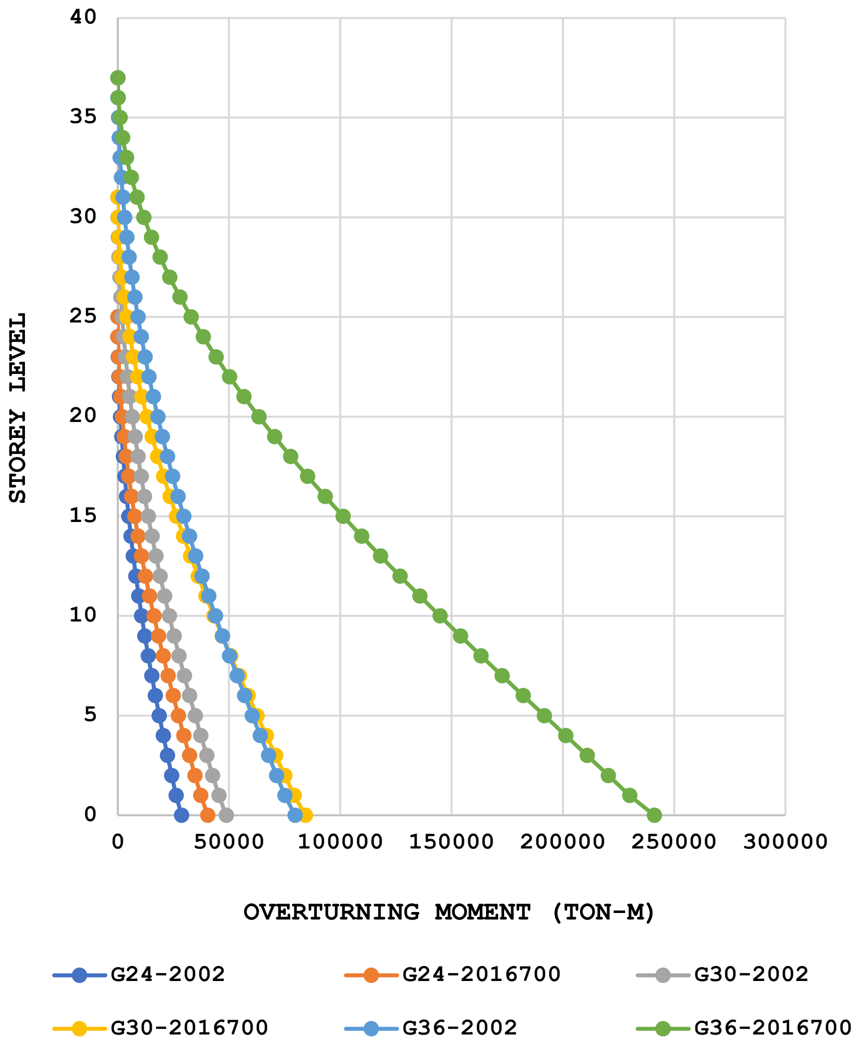


Fig 5.24B Storey Level Vs Overturning Moment (Dynamic Wind Y-Direction)

It is observed from Fig 5.24A and Fig 5.24B that the calculated storey shears on different floor levels are not same for all types of structures because changes in calculation of design wind pressure with respect to IS 875 (Part-3): 1987 and IS 875 (Part-3): 2015. The value of overturning moment is depends on storey shear and height of the structure. It is observed from Fig 5.23A and Fig 5.23B that, due to storey shear the overturning moment of research models G24-2016700, G30-2016700 and G36-2016700 are greater than models G24-2002, G30-2002 and G36-2002. Also the overturning moment of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-2002 and G24-2016700) due to height increment.

3. STOREY LEVEL VS STOREY DISPLACEMENT

The storey displacement caused by wind force (Along Wind + Across Wind) along X and Y directions at different storey levels for different building models have been shown in Fig 5.25A and Fig 5.25B

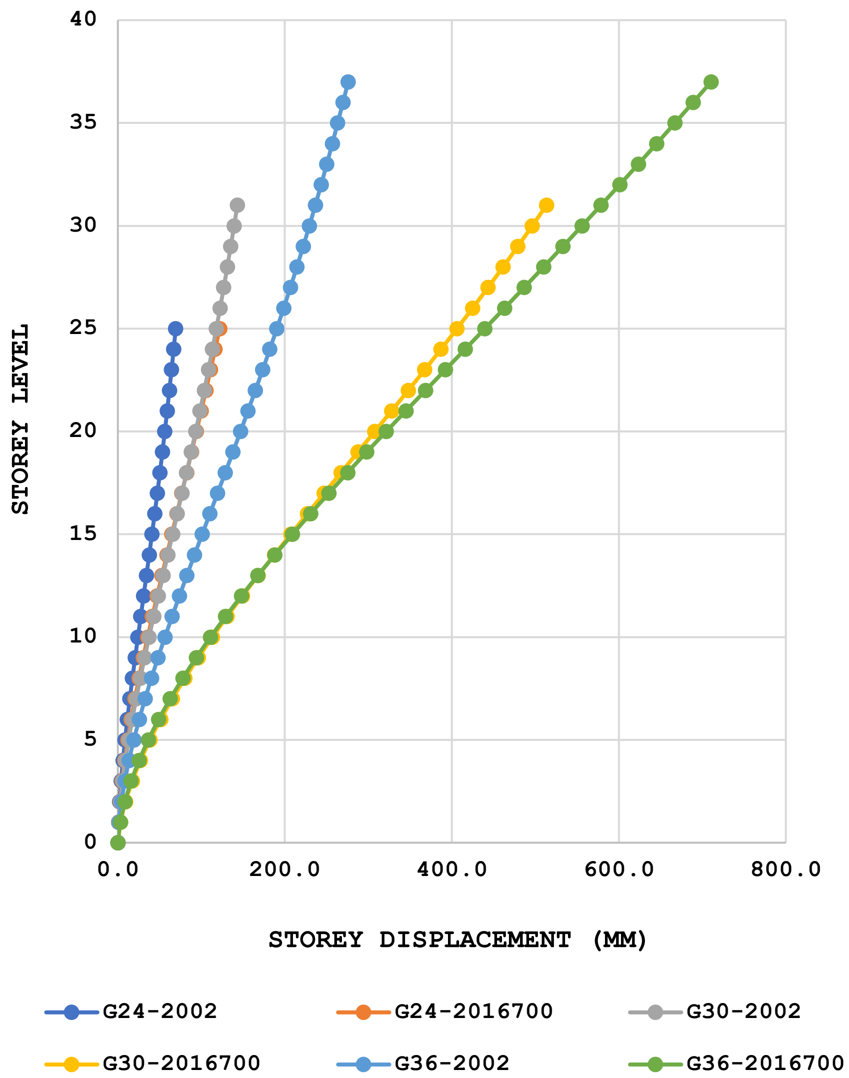


Fig 5.25A Storey Level Vs Storey Displacement (Dynamic Wind X-Direction)

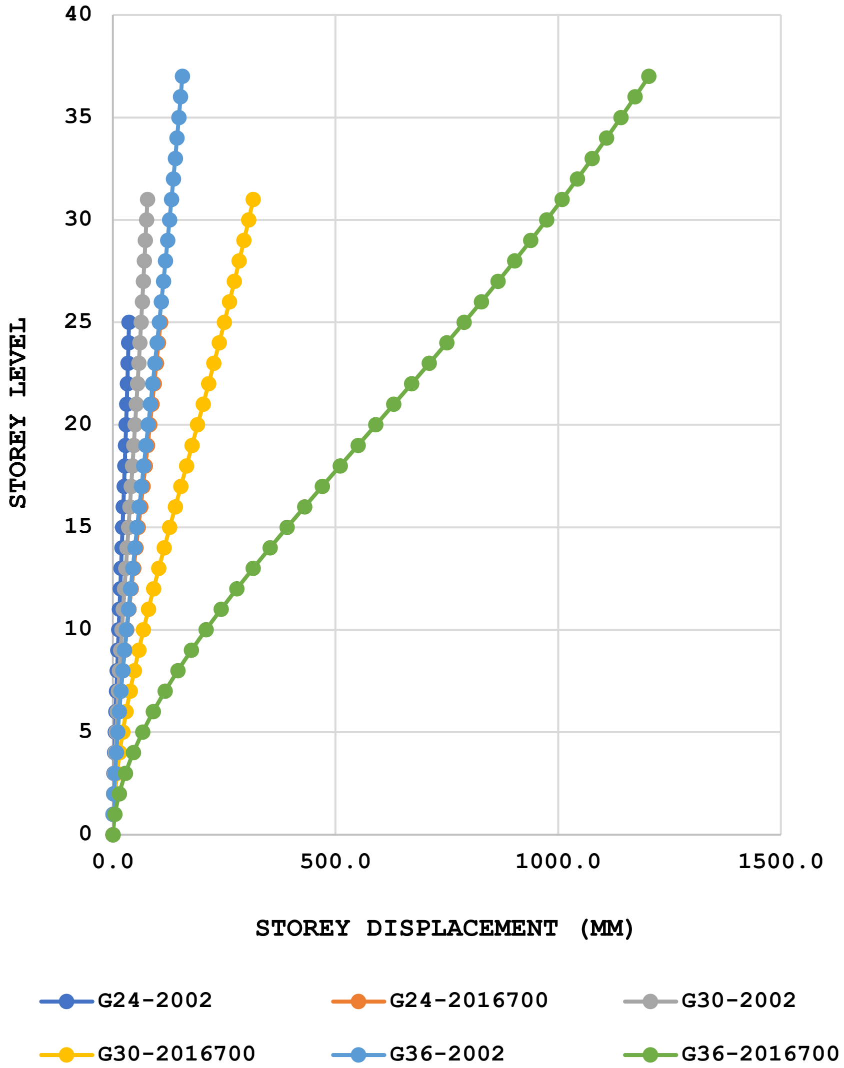


Fig 5.25B Storey Level Vs Storey Displacement (Dynamic Wind Y-Direction)

It is observed from above graphical representations, Fig 5.25A and Fig 5.25B, that storey displacement of research models G24-2016700, G30-2016700 and G36-2016700 are greater than the storey displacement of models G24-2002, G30-2002 and G36-2002 because the storey shears of models G24-2016700, G30-2016700 and G36-2016700 at different levels are more than the storey shears of models G24-2002, G30-2002 and G36-2002 respectively. Also the research models G24-2016700, G30-2016700 and G36-2016700 are less stiff than research models G24-2002, G30-2002 and G36-2002 respectively due to using of stiffness modifier in structural wall, columns and beams as per table 6 of IS 16700: 2017. Also the storey displacement of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-

2002 and G24-2016700) due to differential height respectively. But maximum storey displacements at roof level models G36-2002 and G36-2016700 are exceed the value of permissible limit (H/500, where H is the height of building including the parapet height) of respective Indian standards.

4. STOREY LEVEL VS STOREY DRIFT

The storey drift caused by wind forces (Along Wind + Across Wind) along X and Y directions at different storey levels for all building types have been depicted in Fig 5.26A and Fig 5.26B

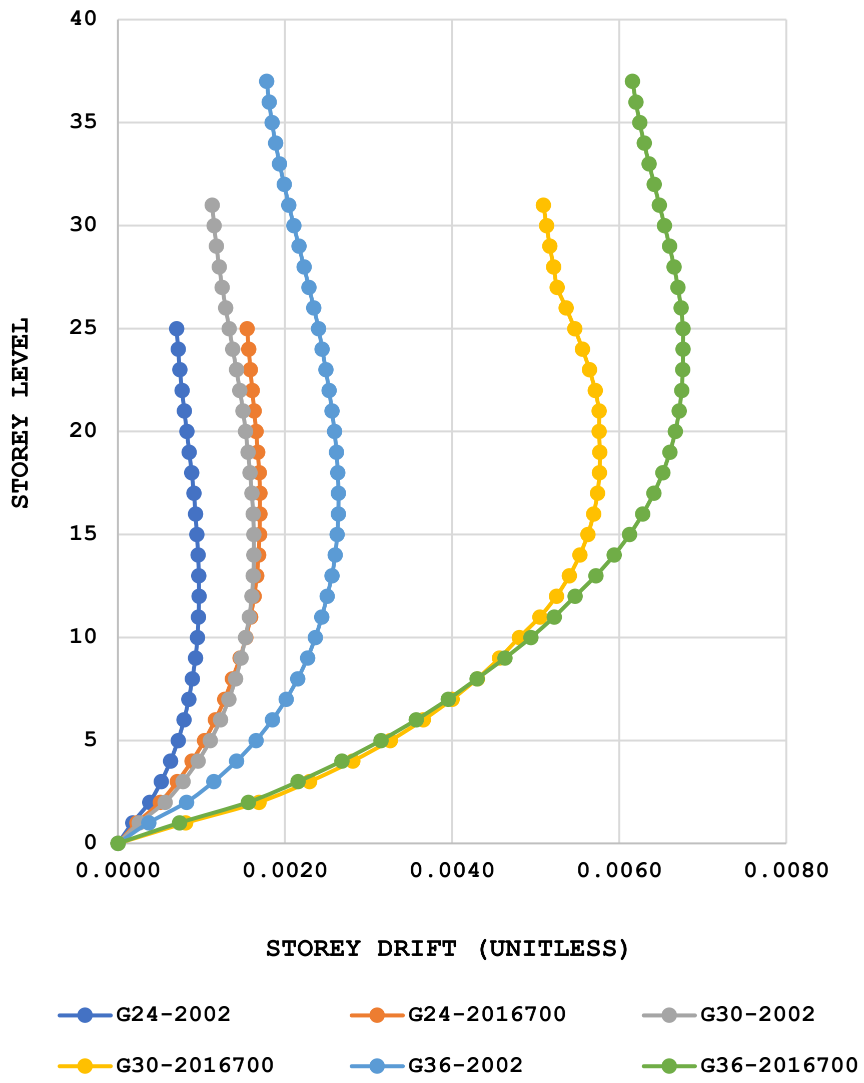


Fig 5.26A Storey Level Vs Storey Drift (Dynamic Wind X-Direction)

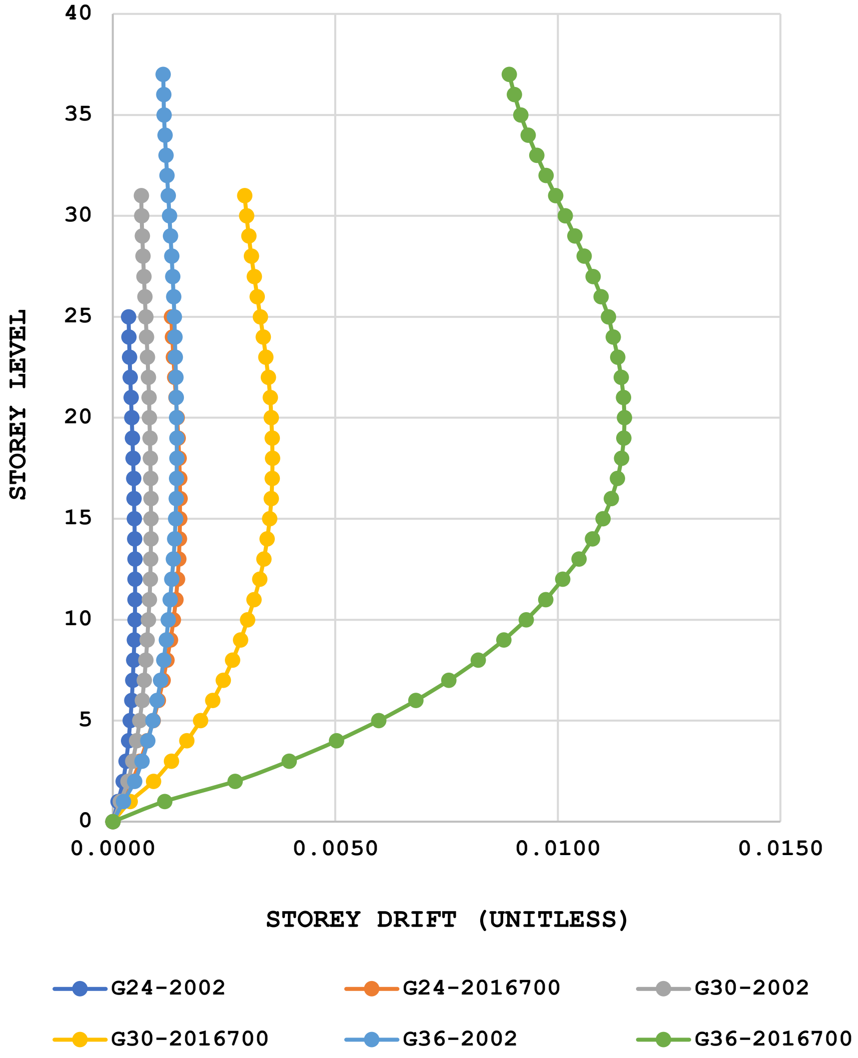


Fig 5.26B Storey Level Vs Storey Drift (Dynamic Wind Y-Direction)

Storey drift is unit less because it is ratio of storey displacement of respective floor to storey height. From the graphical representation of Fig 5.26A and Fig 5.26B it is noted that all buildings experiences max drift at half of the total building height for dynamic wind loading. From figures it is observed that the storey drift of research models G24-2016700, G30-2016700 and G36-2016700 are more than G24-2002, G30-2002 and G36-2002 due to varying stiffness of structural components (using of stiffness modifier in models G24-2016700, G30-2016700 and G36-2016700) and along + across wind effects at different storey levels respectively. Also the storey drift of models G36 (G36-2002 and G36-2016700) are more than G30 (G30-2002 and G30-2016700) and G24 (G24-2002 and G24-2016700) due to incremental height difference and increment of wind forces respectively. For model G30-2016700 and G36-2016700 the storey drifts exceed the permissible limit.

Table 5.7 Comparison of Structural Response for Dynamic Wind due to Codal Revision for Problem No.-2

Item Description	**Type of Loading**	**Direction of Loading**	**Type of Building Structure**								
			Building G24-2002	**Building G24-2016700**	**Change in Response (%)**	**Building G30-2002**	**Building G30-2016700**	**Change in Response (%)**	**Building G36-2002**	**Building G36-2016700**	**Change in Response (%)**
Time Period (Sec) at 1st Mode			**2.272**	**2.581**	**(+) 14**	**3.129**	**3.594**	**(+) 15**	**4.085**	**4.735**	**(+) 16**
Base Shear (Ton)	Wind Loading	**WLX**	**548**	**1063**	**(+) 94**	**751**	**3259**	**(+) 334**	**1019**	**2916**	**(+) 186**
		WLY	**585**	**758**	**(+) 30**	**802**	**1216**	**(+) 52**	**1086**	**2688**	**(+) 148**
Overturning Moment (Ton-m)	Wind Loading	**WLX**	**27117**	**60141**	**(+) 122**	**46415**	**239294**	**(+) 416**	**76466**	**256571**	**(+) 236**
		WLY	**28649**	**40434**	**(+) 41**	**48712**	**84308**	**(+) 73**	**79624**	**241142**	**(+) 203**
Storey Displacement (mm)	Wind Loading	**WLX**	**69.3**	**122**	**(+) 76**	**143**	**513**	**(+) 259**	**276**	**710**	**(+) 157**
		WLY	**36.3**	**107**	**(+) 195**	**78**	**315**	**(+) 304**	**156**	**1204**	**(+) 672**
Maximum Storey Drift	Wind Loading	**WLX**	**0.001**	**0.0017**	**(+) 70**	**0.0016**	**0.0058**	**(+) 263**	**0.0026**	**0.0068**	**(+) 162**
		WLY	**0.0005**	**0.0015**	**(+) 200**	**0.0009**	**0.0036**	**(+) 300**	**0.0015**	**0.0115**	**(+) 667**

The structural responses of different buildings are shown in the following comprehensive charts.

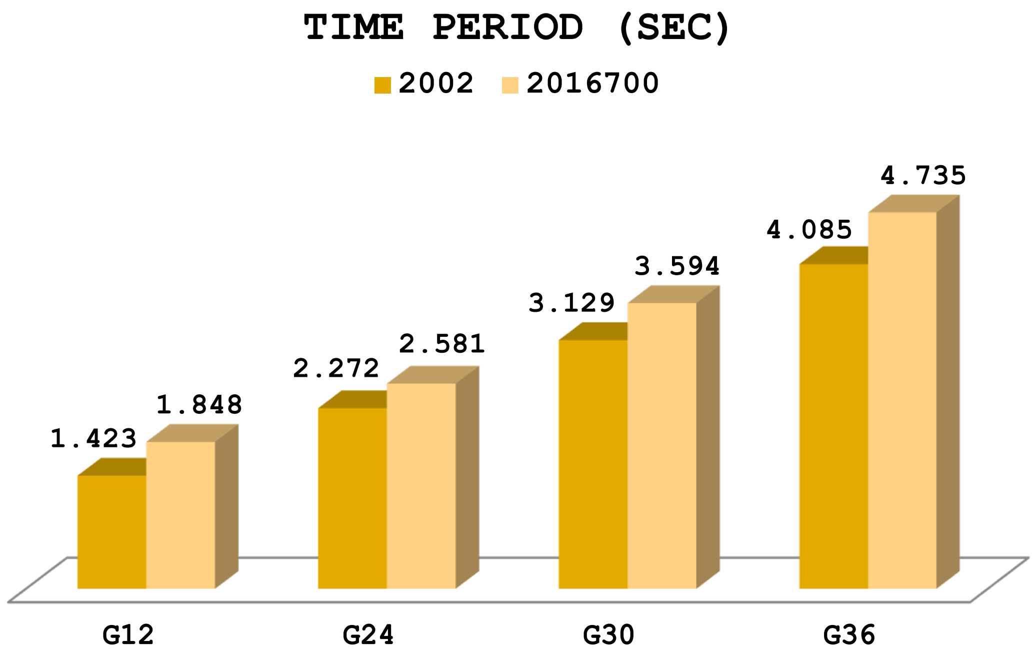


Fig 5.27 Variation of Time Period of Different Buildings

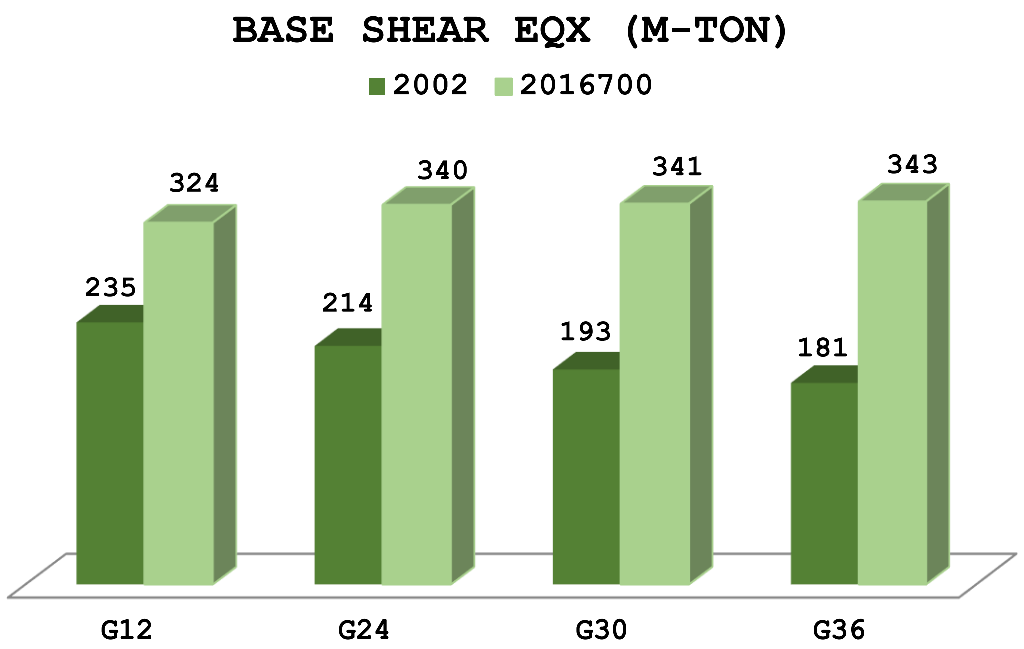


Fig 5.28A Variation of Base Shears of Different Buildings for Earthquake Force (EQX)

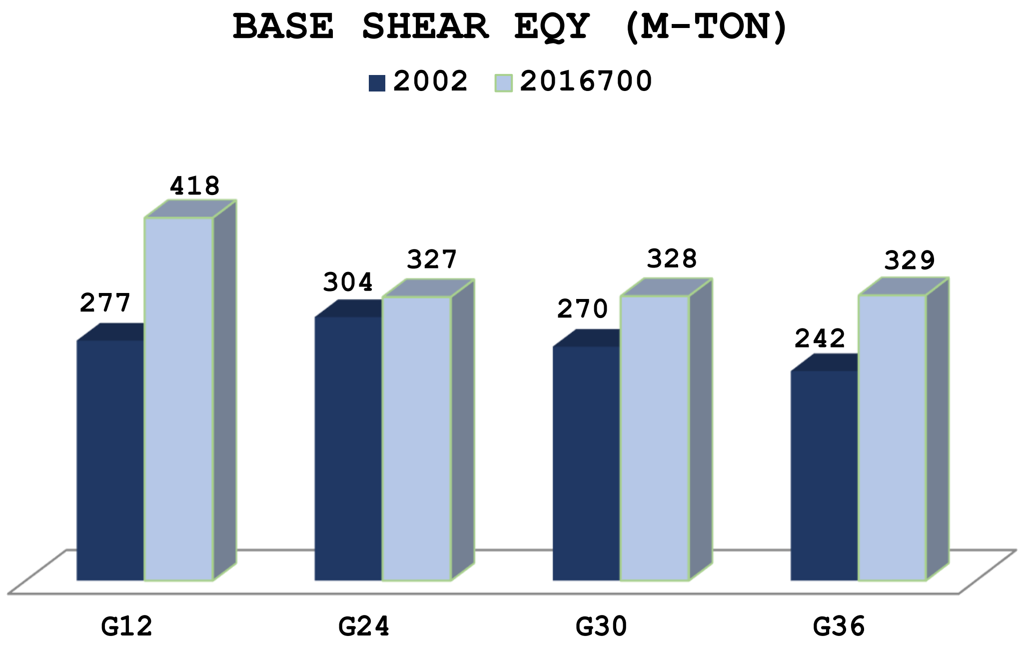


Fig 5.28B Variation of Base Shears of Different Buildings for Earthquake Force (EQY)

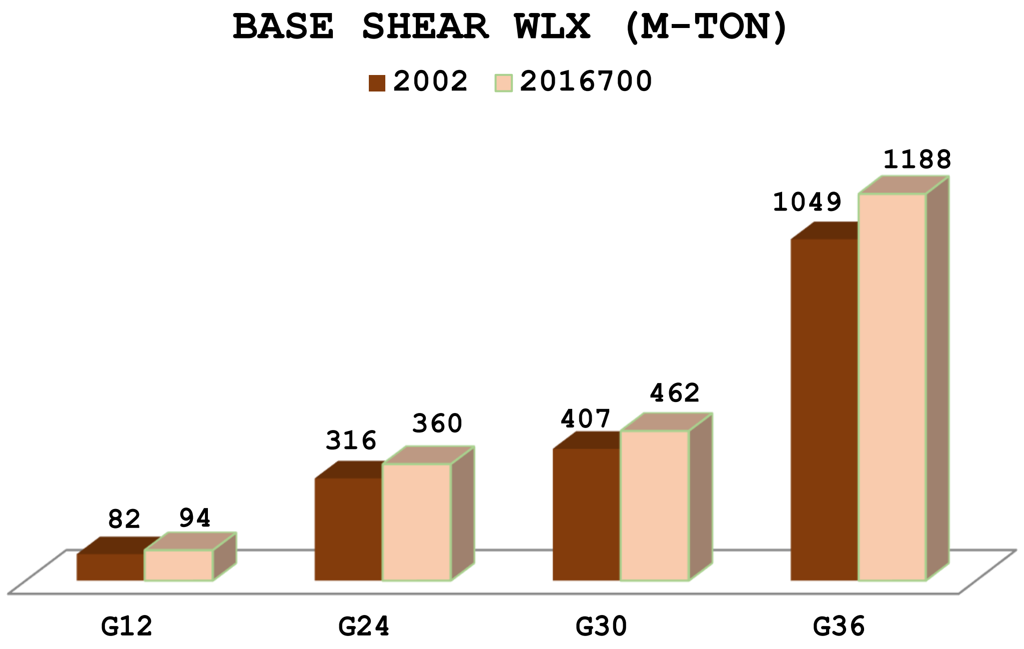


Fig 5.28C Variation of Base Shears of Different Buildings for Static Wind Force (WLX)

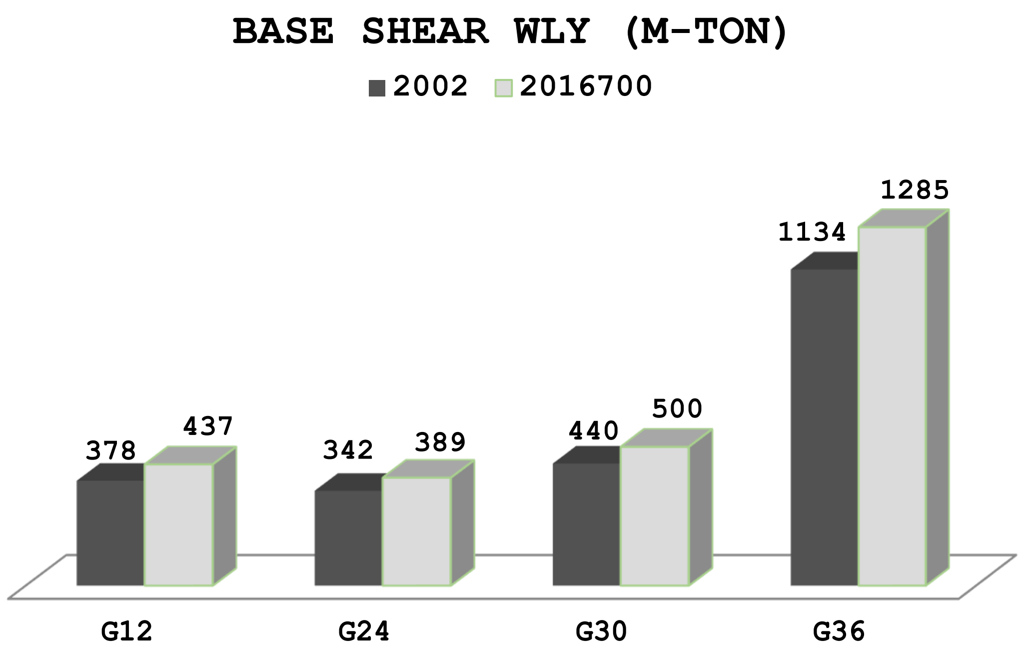


Fig 5.28D Variation of Base Shears of Different Buildings for Static Wind Force (WLY)

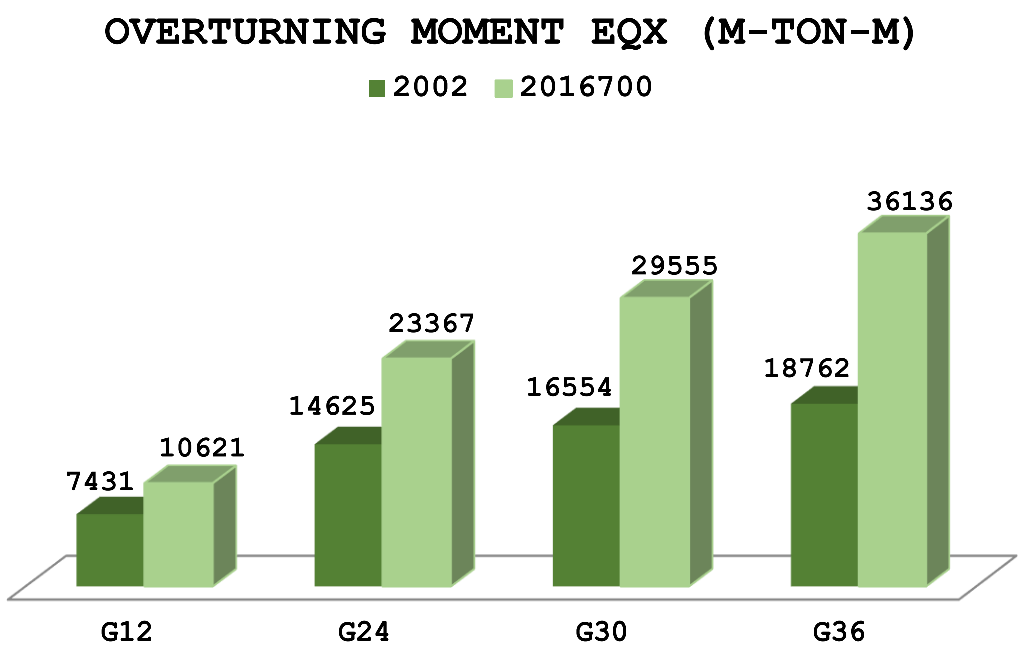


Fig 5.29A Variation of Overturning Moments of Different Buildings for Earthquake Force (EQX)

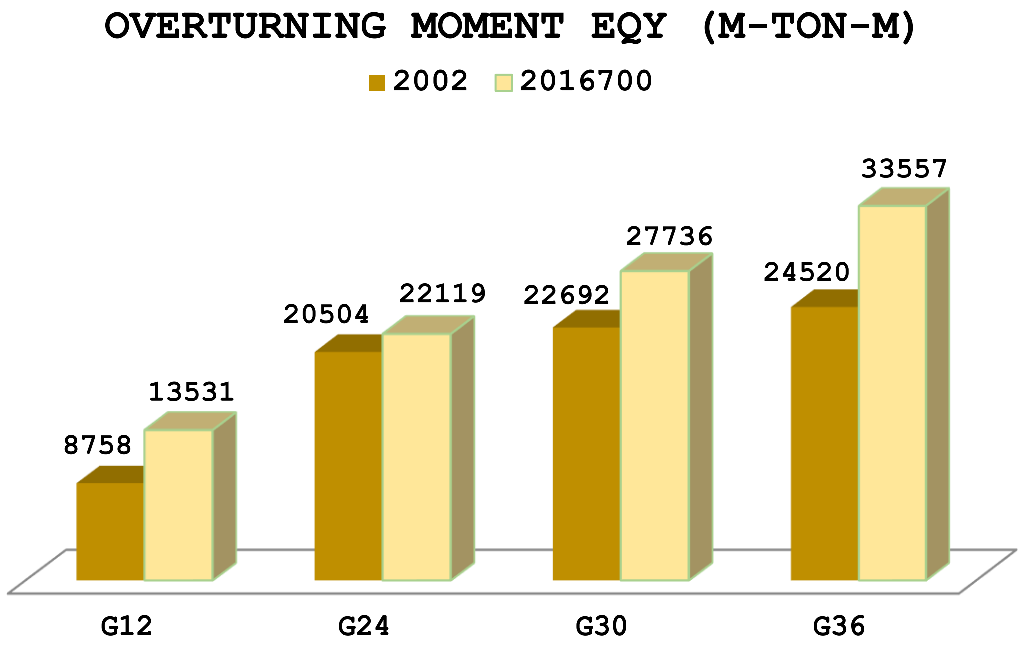


Fig 5.29B Variation of Overturning Moments of Different Buildings for Earthquake Force (EQY)

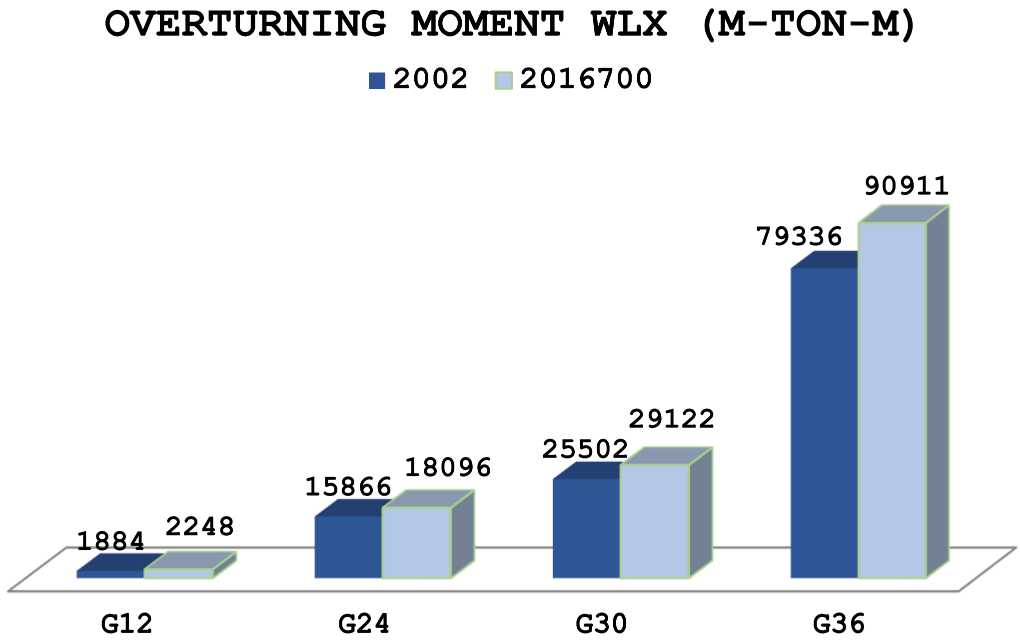


Fig 5.29C Variation of Overturning Moments of Different Buildings for Static Wind Force (WLX)

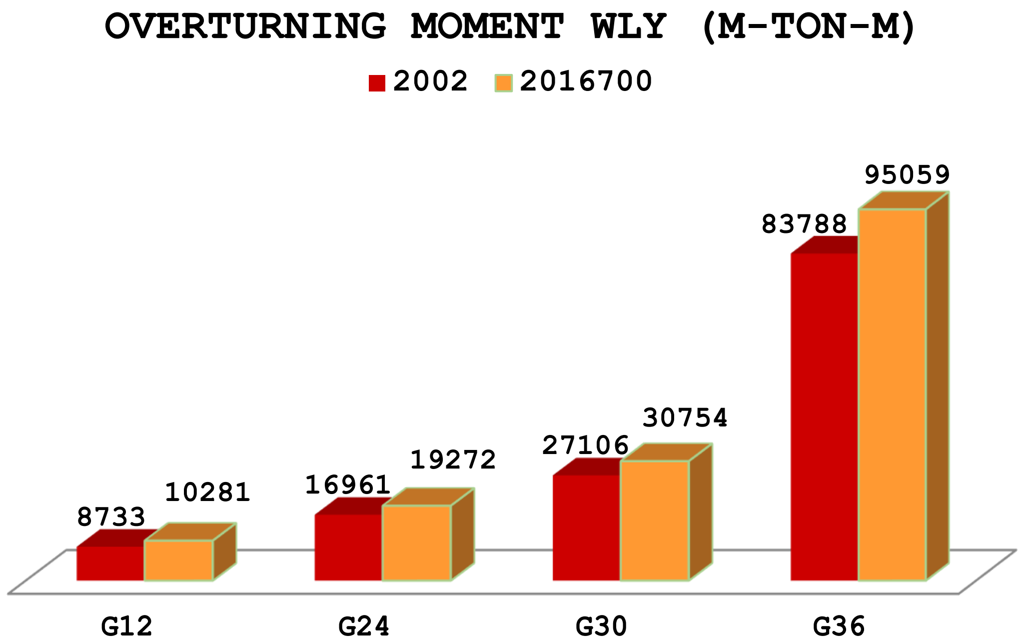


Fig 5.29D Variation of Overturning Moments of Different Buildings for Static Wind Force (WLY)

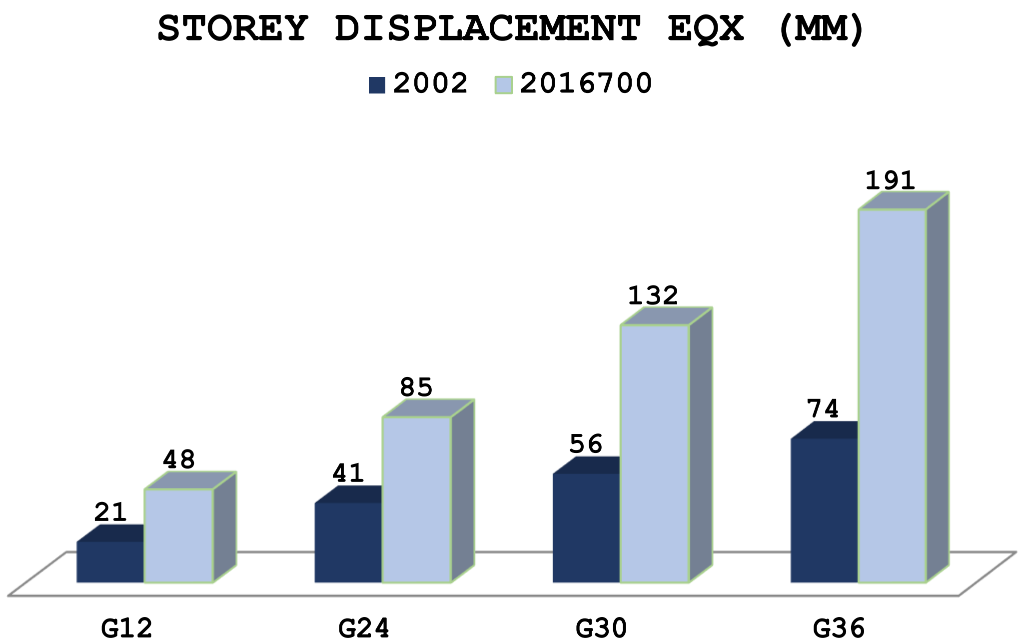


Fig 5.30A Variation of Maximum Storey Displacement of Different Buildings for Earthquake Force (EQX)

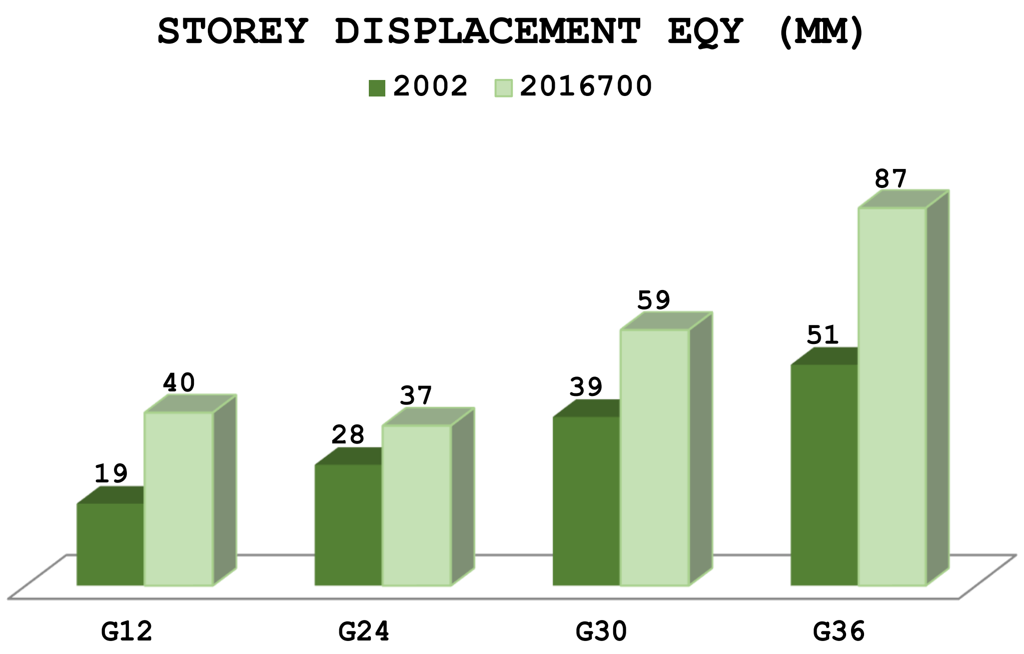


Fig 5.30B Variation of Maximum Storey Displacement of Different Buildings for Earthquake Force (EQY)

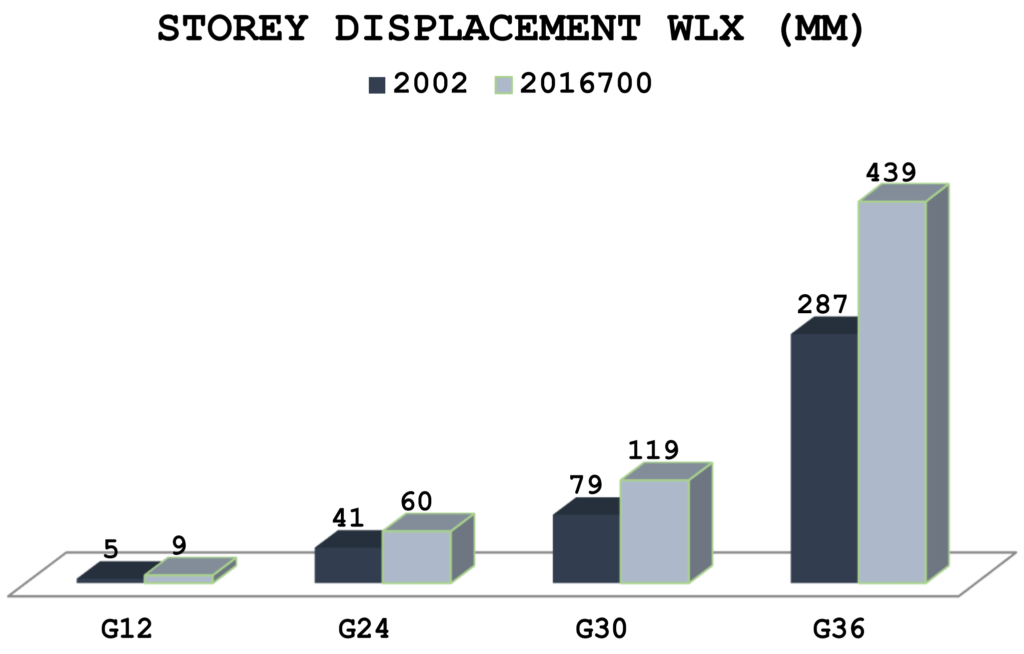


Fig 5.30C Variation of Maximum Storey Displacement of Different Buildings for Static Wind Force (WLX)

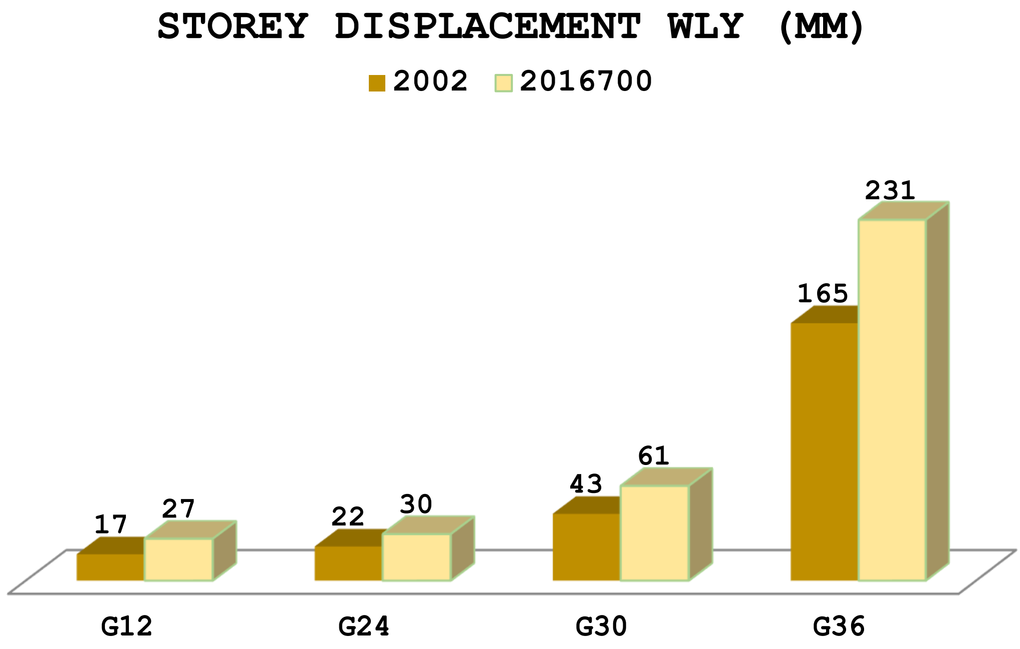


Fig 5.30D Variation of Maximum Storey Displacement of Different Buildings for Static Wind Force (WLY)

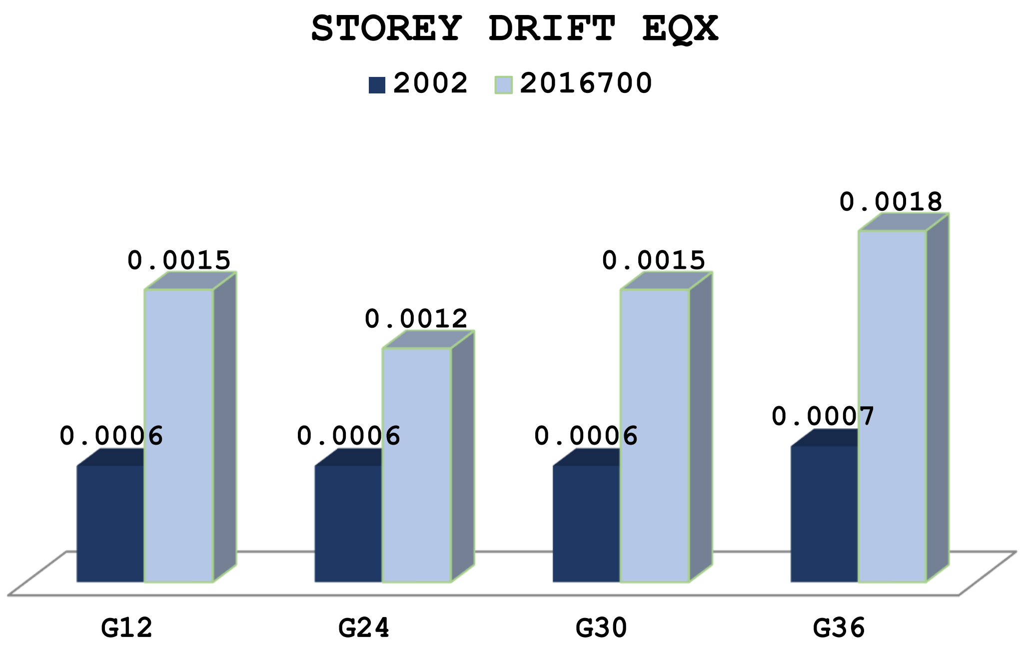


Fig 5.31A Variation of Maximum Storey Drifts of Different Buildings for Earthquake Force (EQX)

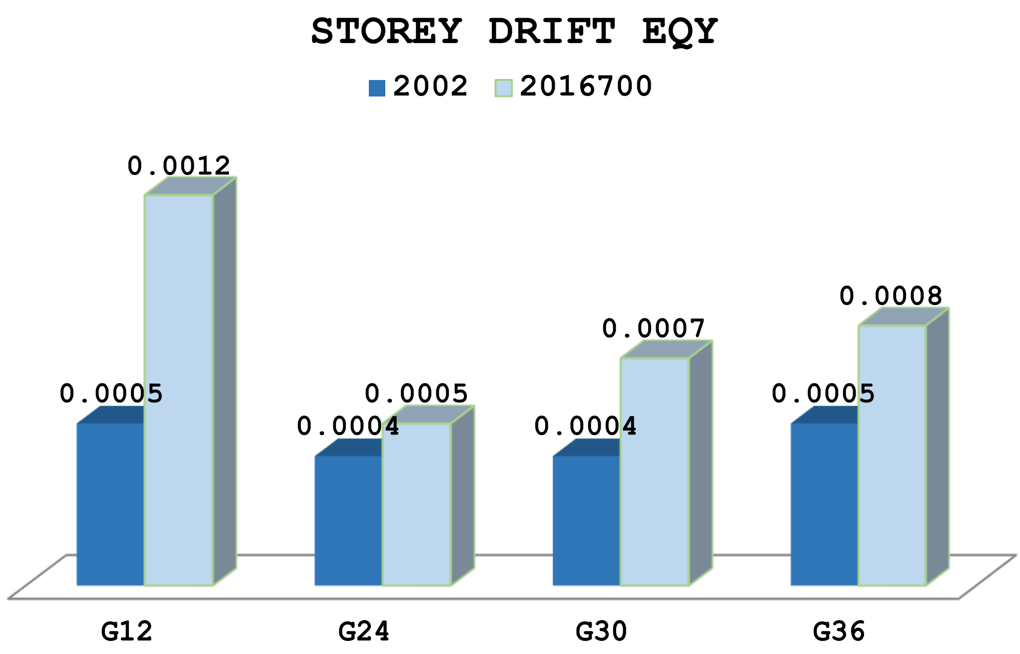


Fig 5.31B Variation of Maximum Storey Drifts of Different Buildings for Earthquake Force (EQY)

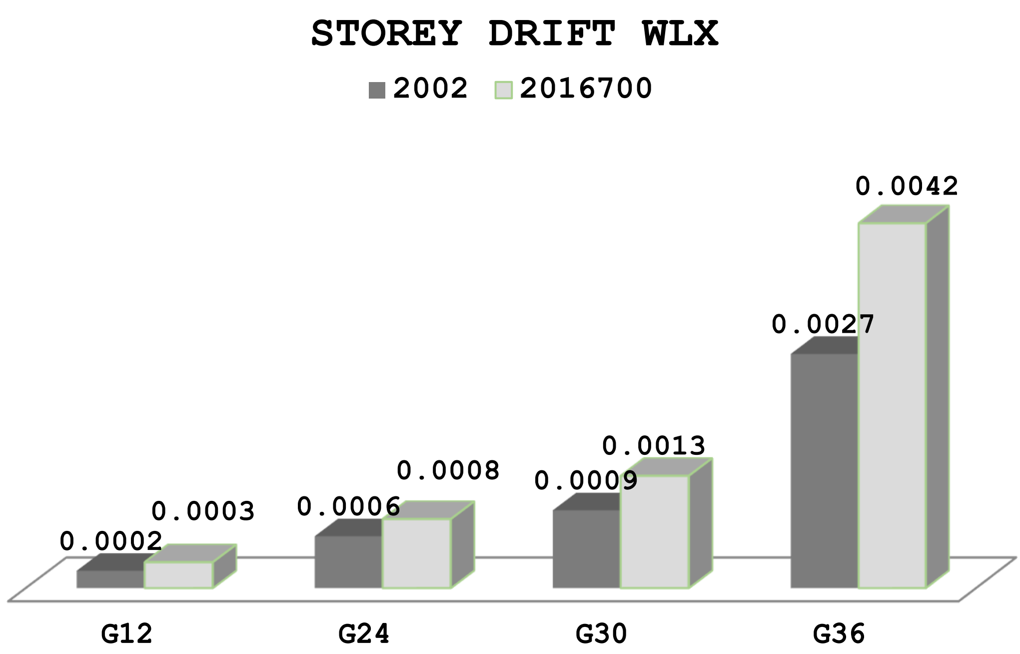


Fig 5.31C Variation of Maximum Storey Drifts of Different Buildings for Static Wind Force (WLX)

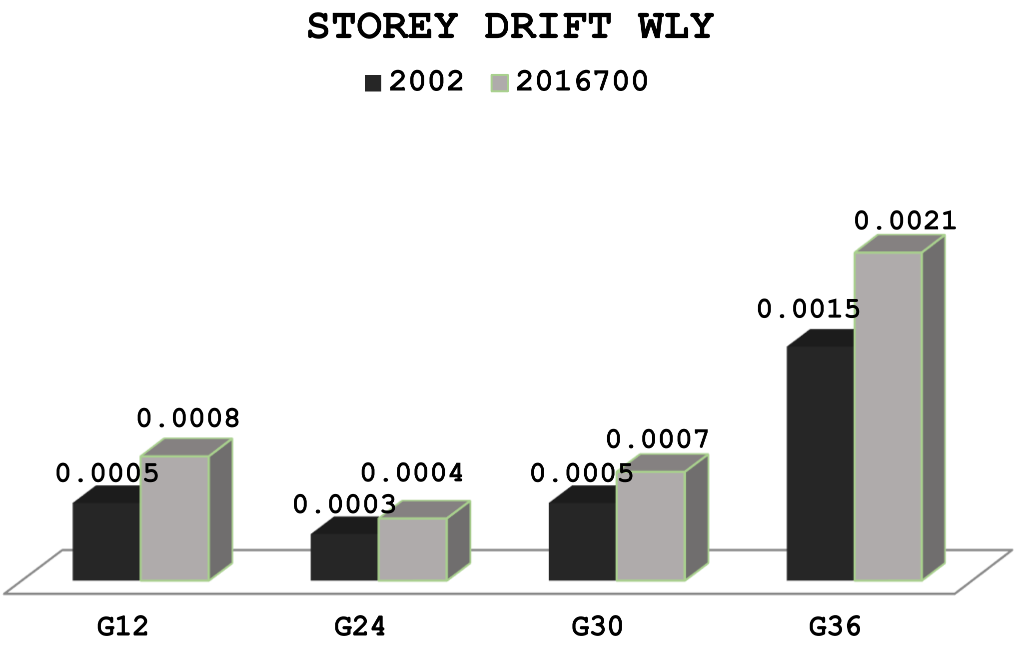


Fig 5.31D Variation of Maximum Storey Drifts of Different Buildings for Static Wind Force (WLY)

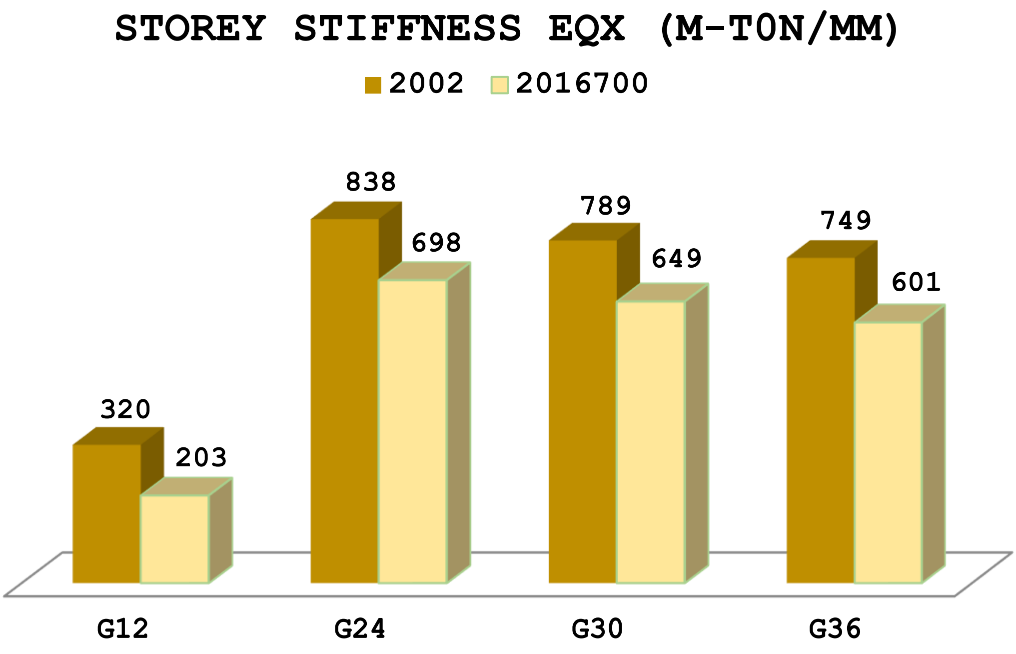


Fig 5.32A Variation of Storey Stiffness of Different Buildings for Earthquake Force (EQX)

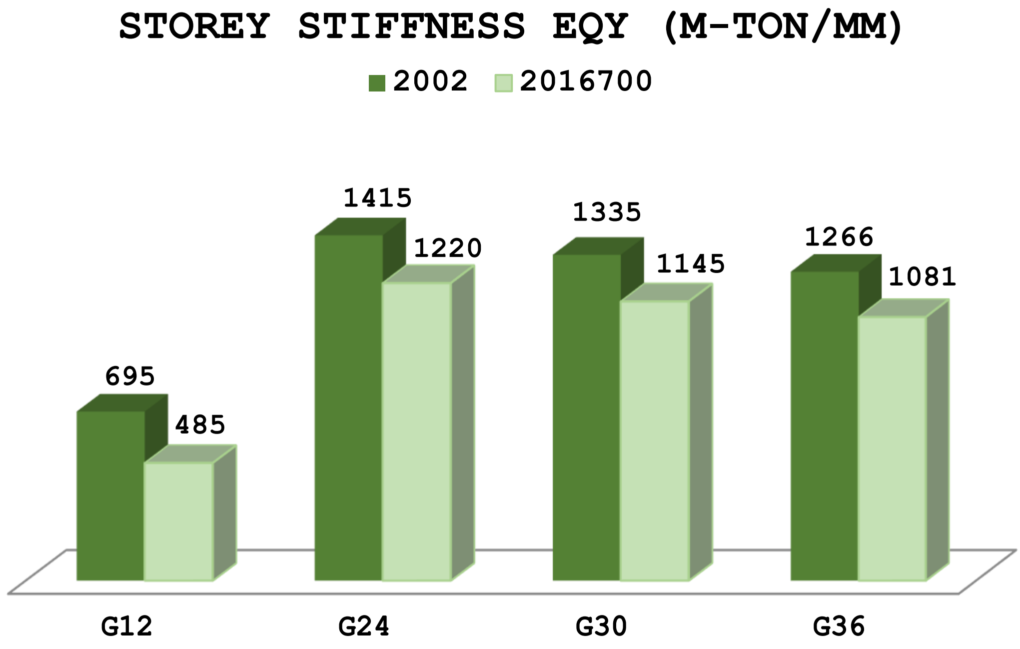


Fig 5.32B Variation of Storey Stiffness of Different Buildings for Earthquake Force (EQY)

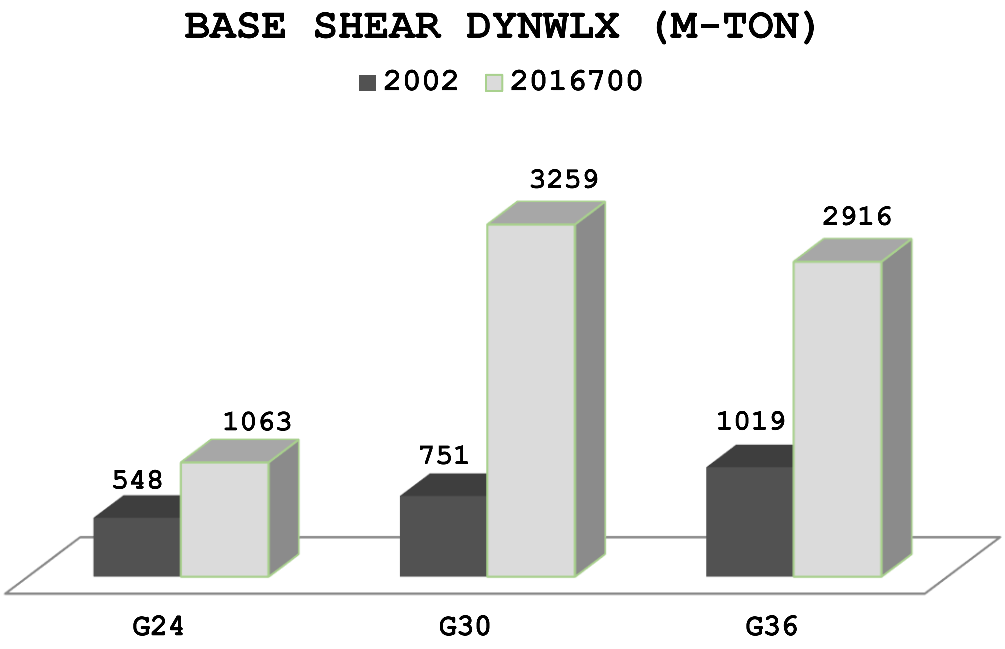


Fig 5.33A Variation of Base Shears of Different Buildings for Dynamic Wind (DYNWLX)

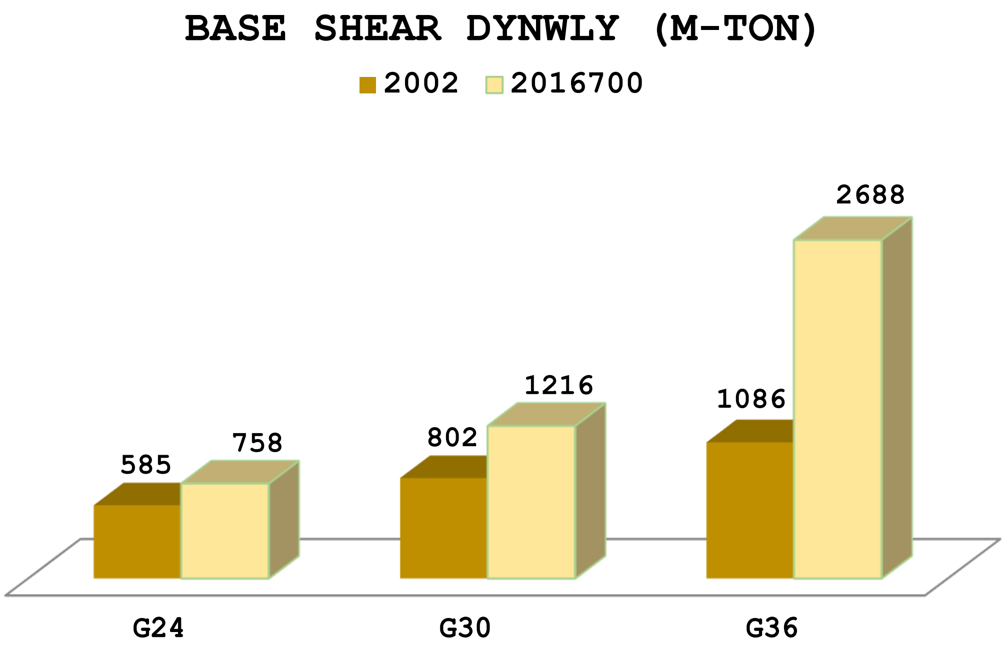


Fig 5.33B Variation of Base Shears of Different Buildings for Dynamic Wind (DYNWLY)

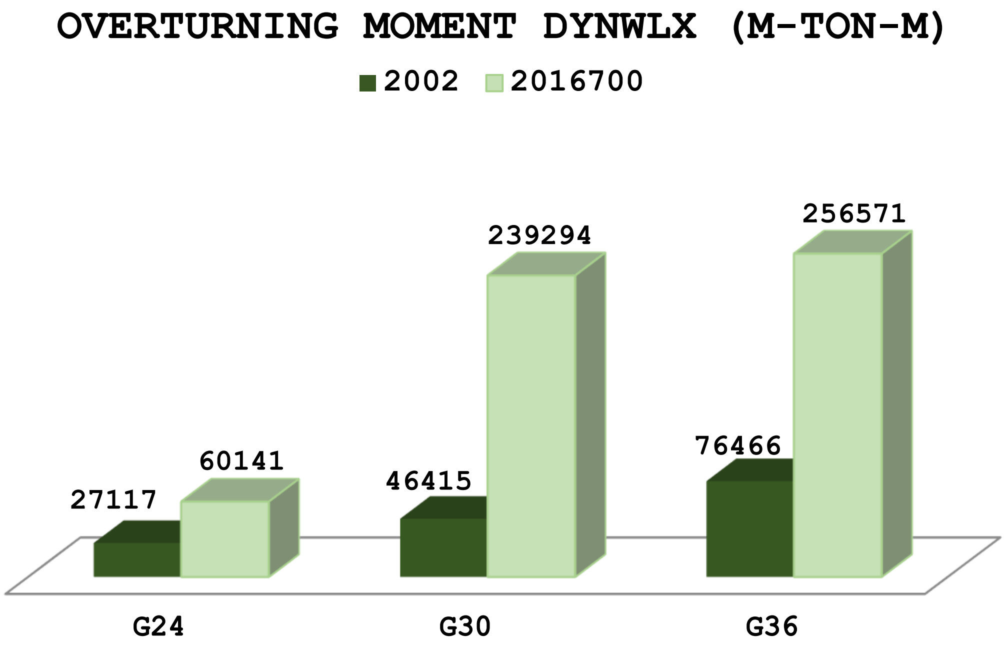


Fig 5.34A Variation of Overturning Moments of Different Buildings for Dynamic Wind (DYNWLX)

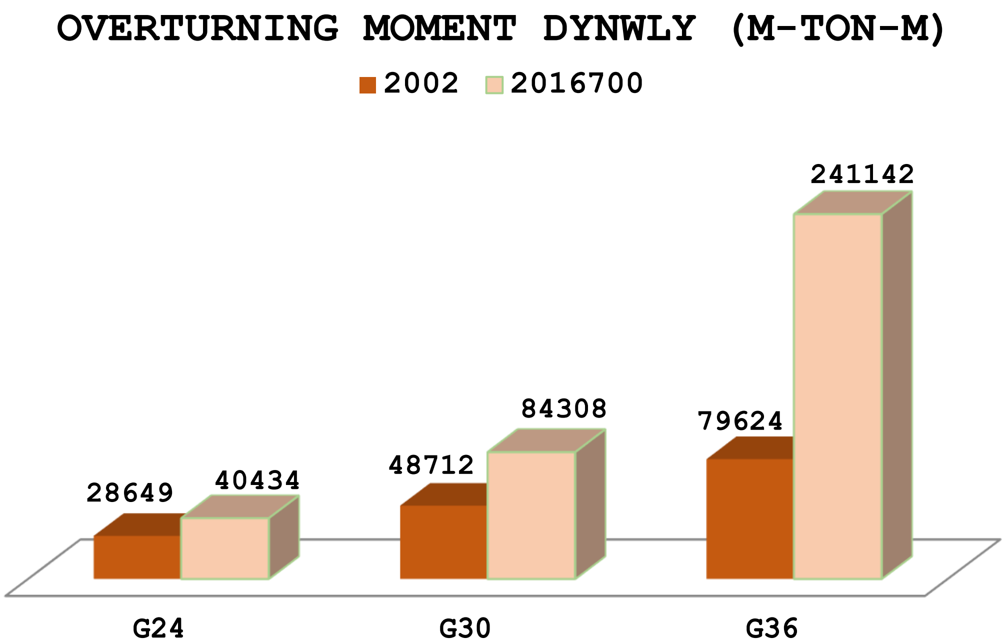


Fig 5.34B Variation of Overturning Moments of Different Buildings for Dynamic Wind (DYNWLY)

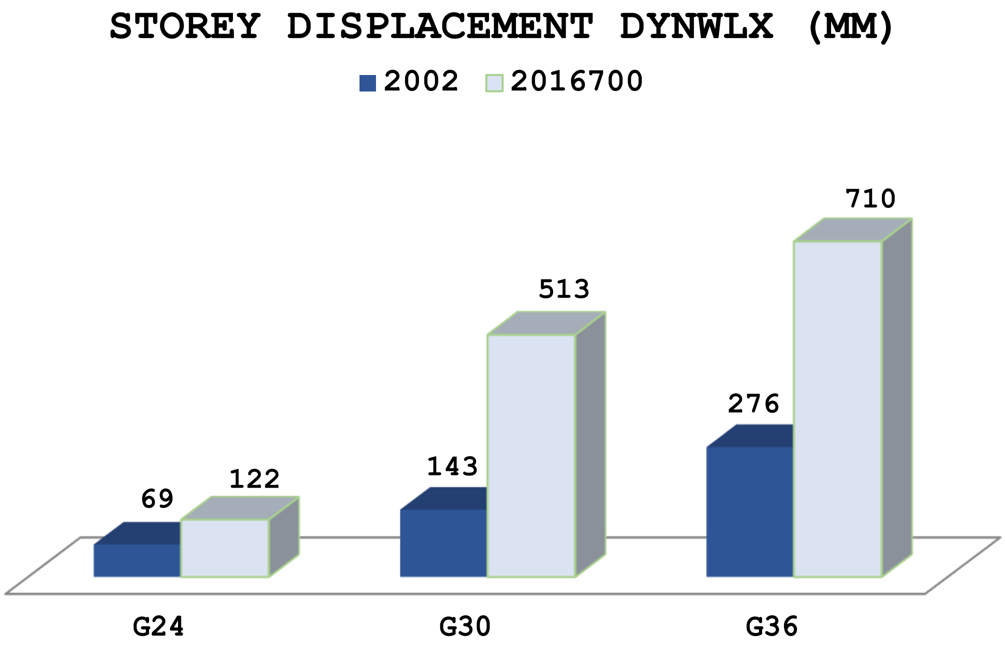


Fig 5.35A Variation of Maximum Storey Displacements of Different Buildings for Dynamic Wind (DYNWLX)

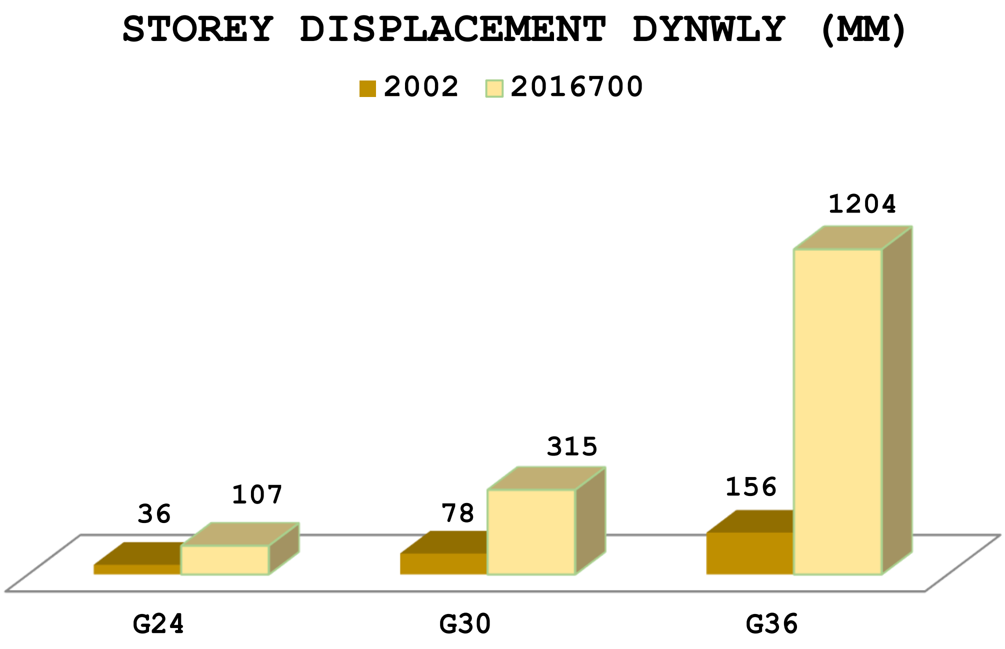


Fig 5.35B Variation of Maximum Storey Displacements of Different Buildings for Dynamic Wind (DYNWLY)

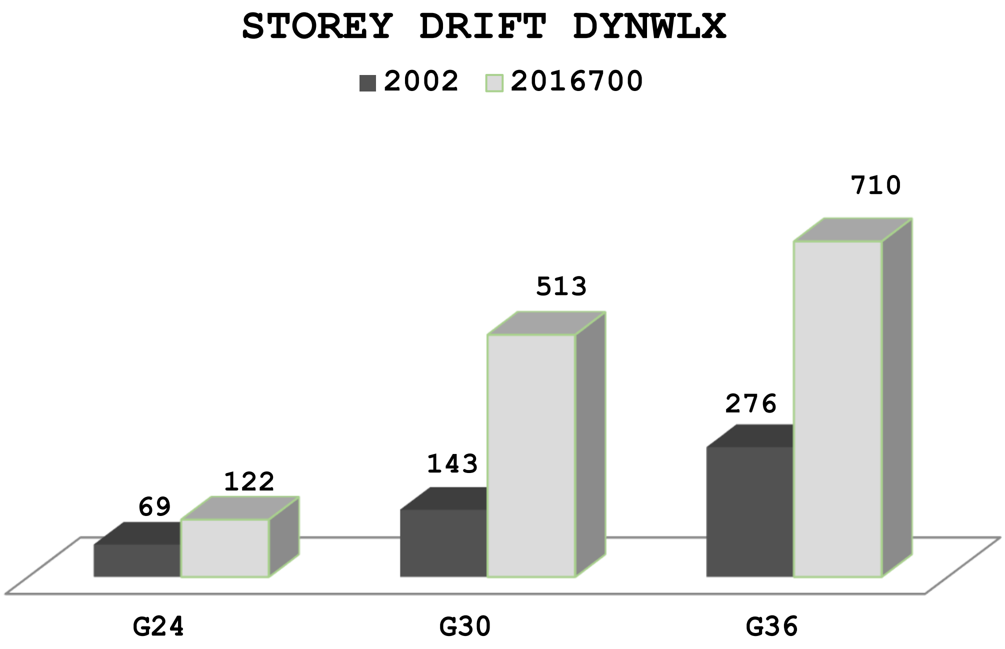


Fig 5.36A Variation of Maximum Storey Drifts of Different Buildings for Dynamic Wind (DYNWLX)

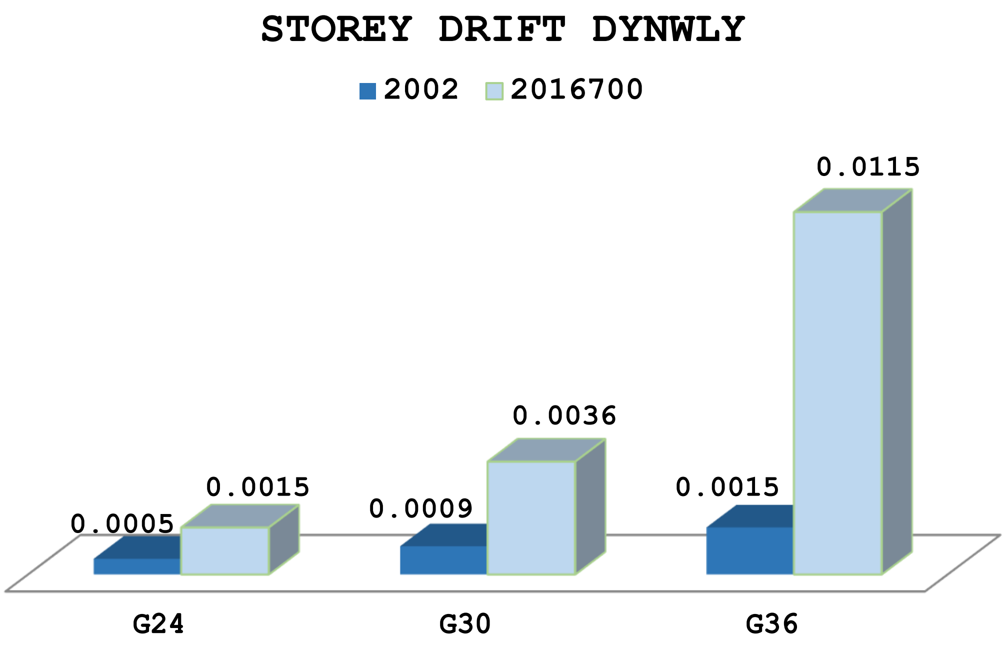


Fig 5.36B Variation of Maximum Storey Drifts of Different Buildings for Dynamic Wind (DYNWLY)

CHAPTER-VI

6.1. CONCLUSION:

Based on the detail numerical study of the structural behaviour of various high-rise building models the following may be concluded

A. For mid height G+12 storied building due to revision of IS 1893 (Part-1)

1. The introduction of stiffness modifier in the structural elements as per latest revision of IS 1893 (Part-1): 2016 makes the building more flexible than previous one. The Natural Time Period (T_n) of the building model is increased to the tune of 30% for the considered building.
2. However, with the introduction of empirical formula for shear wall structure in the revised code, the approximate time period (T_a) is less than the natural time period (T_n) and thus the advantage of less S_a/g is not available. From the present study S_a/g as per new codal provision increased to the tune of 13%.
3. Though the building model became flexible as per IS 1893 (Part-1): 2016 the base shear is increased to a greater extent (51%) than the previous version model. This may be attributed to the increase of the importance factor to 1.2 instead of 1.0 for this type of residential buildings, where number of occupancies is more than 200 persons.
4. In addition, the new expression for approximate fundamental translational time period (T_a) for building with RC structural walls is introduced in latest revision of IS 1893 (Part-1) 2016 for that the value of S_a/g also increased. The value of S_a/g mainly depends on the time period of structure and type of founding soil. Increment of importance factor and response acceleration coefficient (S_a/g) has increased the Base shear even though the stiffness of the latest model is quite less.
5. The overturning moment is caused by the storey forces, developed at different storey levels for ground movement due to transient forces. The overturning moment is increased significantly (50%) following the latest code due to increment of storey shear, as base shear is increased.

6. The lateral displacement of a structure depends on the storey shear, vertical height and lateral stiffness of the structure. In the model based on latest code, the storey shear is increased and the lateral stiffness is decreased resulting significant increment (more than twice) of storey displacement with respect to those following previous code. The storey drift of the model based on latest code is also increased accordingly.
7. The wind effect seems to be negligible [slight increment of the value of k_2 in the latest revision of IS 875 (Part-3), 2015] in most of the locations. It indicates that no significant effect due to the dynamic condition of wind is observed for this mid-rise building structure. However, in case of cyclonic zone a new factor k_4 is introduced to accommodate the greater effect of wind at those zones.
8. The story displacement and storey drift of the latest model are also increased due to the wind effect and reduction of lateral stiffness of building model.

B. For Tall buildings (G+24, G+30 and G+36 storied) due to the upgradation of IS 1893 (Part-1), 2016 and introduction of IS 16700: 2017.

1. The effect of modifier is also observed in case of tall (G+24, G+30 and G+36) buildings. However, this effect of modifier seems to be reduced in case of serviceability criterion as compared with the mid-rise (G+12) building due to incorporation of separate clause in IS 16700: 2017. Thus Natural Time Period (T_n) of the building models are increased to the tune of 14-16% with respect to those following previous code.
2. However, with the introduction of empirical formula for shear wall structure the approximate time period (T_a) is less than the natural time period (T_n) and thus the advantage of less S_a/g is not available. From the present study S_a/g as per new codal provision increase to the tune of 25 % to 55%
3. Though the building models are became flexible as per IS 1893 (Part-1): 2016 the base shears are increased to a greater extent (59-90%) than the previous version. This may be attributed to the increase of the importance

factor to 1.2 instead of 1.0 for this type of residential buildings, where number of occupancies is more than 200 persons.

4. In addition, the new expression for approximate fundamental translational time period (T_a) for building with RC structural walls is introduced in latest revision of IS 1893 (Part-1) for that the value of S_a/g marginally increased. The value of S_a/g mainly depends on the time period of structure and type of founding soil. Increments of importance factor and response acceleration coefficient (Sa/g) have increased the Base shear even though the stiffness of the latest model is quite less.
5. The overturning moment is caused by the storey forces, developed at different storey levels for ground movement due to transient forces. The overturning moment is increased significantly (60-90%) following the latest code due to increment of storey shear, as base shear is increased.
6. The lateral displacement of a structure depends on the storey shear, vertical height and lateral stiffness of the structure. In the model based on latest code, the storey shear is increased and the lateral stiffness is decreased resulting significant increment (more than twice) of storey displacement with respect to those following previous code. The storey drift of the model based on latest code is also increased accordingly.
7. The dynamic effect of wind in the along direction is reduced for the tall buildings. However, the effect due to across wind (newly introduced) has significantly increased as compare to the earlier codal stipulations.
8. The displacement has increased in many folds for the tall building models incorporating the latest IS 875 (Part-3): 2015 and IS 16700: 2017. These may be attributed to the greater flexibility offered by the modifier and dynamic across wind effect.
9. The revision of IS 1893 (Part-1): 2016 and introduction of IS 16700: 2017 result in increase of both the strength and stiffness demand to comply with the limit state of collapse and serviceability. Based on the revision of codal stipulations significant effects on the design consideration of high-rise building structure is observed. However these upgradations provide a more rational consideration for mid-height and high-rise buildings.

6.2. FUTURE SCOPE:

The work presented in this thesis provides different possibilities for further work

1. Study of different shape of building, plan and vertical aspect ratio of building structure, for minimizing the across-wind effect on structure can be done.
2. Effect of adopting different structural systems such as tube structure, tubes in tube structure, outriggers with belt-truss system and diagrid system etc. described in IS: 16700: 2017 to increase the lateral stiffness of the structures can be done.
3. Study of installation of different passive strengthening devices, to increase the lateral stiffness of the structure during the effects of transient loads can be done.
4. Effect of using steel-concrete Composite structural system with respect to other conventional systems to enhance the structural behaviour can be done.

APPENDIX

REFERENCES

- A. Kale and S. A. Rasal (2017), "Seismic and Wind Analysis of Multi-storey Building: A Review", International Journal of Science and Research (IJSR), Volume 6 Issue 3, March 2017.
- ACI 318M: (2014) "Building Code Requirements for Structural Concrete".
- ASCE 7: (2010), "Minimum Design Loads for Buildings and Other Structures".
- Burak, B. and Hakki, G.C. (2013), "Effect of shear wall area to floor area ratio on the seismic behaviour of reinforced concrete buildings", Journal of Structural Engineering, 139 (11), 1928-1937.
- Deger, Z.T., Yang, T.Y., Wallace, J.W. and Jack, M. (2014), "Seismic performance of reinforced concrete core wall buildings with and without moment resisting frame", The Structural Design of Tall and Special Buildings, 24 (7), 477-490.
- Dr. D. K. Paul "IS 1893 (Part-1): 2016, Criterion for Earthquake Resistant Design of Structures", General Provisions and Buildings (Sixth Revision), Department Engineering of Earthquake Engineering, IT Roorkee, 2016.
- FEMA 369: (2001), "NEHRP recommended provisions or seismic regulations for new buildings and other structures".
- FEMA 74: (2011), "Reducing the Risks of Non-structural Earthquake Damage".
- Gunel, M.H., and Ilgin, H.E. (2007), "A proposal for the classification of structural systems of tall buildings", Journal of Building and Environment, Elsevier, Science Direct 42, 2667- 2675.
- IS 16700: (2017), "Criteria for structural safety of tall concrete building".
- IS 1893 (Part-1): (2002) Code of Practice for Earthquake Resistant Design of Structures Fifth Revision (Bureau of Indian Standards, New Delhi)
- IS 1893 (PART 1): (2016) Code of Practice for Earthquake Resistant Design of Structures Sixth Revision (Bureau of Indian Standards, New Delhi)
- IS 456: (2000), "Plan and Reinforced Concrete – Code of Practice".
- IS 875 (Part-3): (2015), "Code of practice for Design Loads (Other than Earthquake) for buildings and Structure- Code of Practice".

- K.Venu Manikanta and Dr. DumpaVenkateswarlu, (2016) “Comparative Study On Design Results of a Multi-Storied Building Using STAAD Pro And ETABS For Regular And Irregular Plan Configuration”, Volume 2, Issue 15, PP: 204 - 215.
- Kumar, R., and Singh, Y. (2010), “stiffness of Reinforced concrete frame members for seismic analysis”, ACI Structural Journal, Sep-Oct, 607-615.
- Kumbhar, O.G., Kumar, R., and Adhikary, S. (2015), “Effect of staircase on seismic performance of RC frame building”, Earthquakes and structures, 9 (2), 375-390.
- Mosoarca, M. (2014), “Failure analysis of RC shear wall with staggered opening under seismic loads”, Engineering failure analysis, Elsevier, 46-54.
- Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R. and Mehta, V.V. (2012a) “Some concept in earthquake behaviour of buildings”.
- Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R., Kumar, R.P. and Mehta, V.V. (2012b) “Introduction to earthquake protection of non-structural elements in buildings”.
- Nilupa, H., Priyan, M., Tuan, N. and Nicholas, H. (2010), “Seismic performance of super tall buildings”, International Conference on sustainable built environment, 160-168.
- Park, R., Paulay, T. (1975), “Reinforced Concrete Structures”.
- Paulay, T., Priestley M.J.N. (1992), “Seismic Design of Reinforced Concrete and Masonry Buildings”.
- Prashanth.P, Anshuman.S, Pandey. R.K and Arpan Herbert, “Comparison of design results of a Structure designed using STAAD and ETABS Software”, International Journal of Civil and Structural Engineering Volume 2, No 3, 2012.
- S.K. Ahirwar, S.K. Jain and M. M. Pande (2008), “Earthquake Loads on Multi-storey Buildings as Per IS 1893 (Part-1): 1984 and IS 1893 (Part-1): 2002: A comparative Study”, the 14th World Conference on Earthquake Engineering. October 12-17, 2008, Beijing, China.
- Sanjay, K.S. and Umesh, P. (2016), “Effect of aspect ratio and plan configuration on seismic performance of multi-storeyed regular R.C.C. building: An evaluation by static analysis”, International Journal of Emerging Technology and Advanced Engineering, 6(1), 192-197.

- Tanaka, H., Tamura, Y., Ohtake, K., Nakai, M., Kim. Y.C. and Bandi, E.K. (2013) "Aerodynamic and flow characteristics of tall buildings with various unconventional configurations", International journal of high-rise buildings, 2 (3), 213-228.
- Tarek, A.A. (2011), "Slenderness ration influence on the structural behavior of residential concrete tall buildings", Journal of civil engineering and architecture, 5(6), 527-534.