iJRASET
International Journal For Research in
Applied Science and Engineering Technology

INTERNATIONAL JOURNAL
FOR RESEARCH
IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 11 **Issue:** VI **Month of publication:** June 2023

DOI: https://doi.org/10.22214/ijraset.2023.54484

www.ijraset.com

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International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 11 Issue VI Jun 2023- Available at www.ijraset.com

Seismic Evaluation of Building having Steel Concrete Composite Columns andRC Beams

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Abstract:** Steel concrete composite structures are gaining popularity due to the advantages they offer over the conventional reinforced concrete and steel structures like the ease and speed of construction. In the light of this, it becomes essential to understand the behaviour of this type of structures when used in buildings. This study mainly focuses on the use of building frame consisting of steel-concrete composite column section with the reinforced concrete beams. To achieve this objective, the seismic analysis of the buildings was chosen for evaluating the performances of the buildings designed and analyses using Indian standards. The building selected for the study was rectangular in plan andhad an elevation of 30 meters with no plan or vertical irregularity present. The gravity loads considered for the building are in compliance with the IS 875 Part 1 and IS 875 Part 2. However, the gravity loads used in case of both buildings were kept to be same as the prime focusof the study was seismic analysis. The materials used for the design of the building with respect to Indian standards are E345 grade of steel and M30 grade of concrete. Various codes of practices including IS 456, IS 11384, IS 1893 and IS 13920 have been used throughout the study in case of the building designed and analysed with respect to Indian standards. The assumptions regarding the seismic characteristics of the buildings are also selected of the same nature for both the buildings. A finite element modellingbased software ETABs is used to carry out the design and analysis for the buildings. The buildings were designed with respect to the selected gravity loads and the seismic loads to obtain the section details to be used for seismic analysis. The building with finally obtainedsection sizes is used for carrying out the nonlinear analysis. This includes both the nonlinear static and nonlinear dynamic analysisas they both have their respective advantages in predicting the behaviour of the structure. For carrying out nonlinear analysis both material as well as geometric nonlinearity is considered. In case of the nonlinear dynamic analysis, the ground motions with respect to the guidelines of FEMA P695 and ASCE 7-16 were selected and scaled to perform the analysis. A total of 11 time histories were considered for NLTHA to have a thorough analysis. Various factors such as thecapacity curves, story displacements, base shears and story drifts were considered for evaluation of the seismic performance of both the buildings. The results obtained from the analysis of both thebuildings are represented in the form of tables and graphs, and are compared with each other to study the differences observed. It can be concluded that the buildings designed composite columns offers a better ductility and thus is more earthquake resistant.

I. INTRODUCTION

Today, the need for fast paced and high strength construction has become a very important aspect of the construction industry. Due to the increasein migration of high number of people from all corners of the country to the cities, the need of office spaces and residences in the city areas is growing at a very rapid pace. To cater to thisneed of increasing population in cities it becomes important for the construction industry to adopt high rise building systems to accommodate large number of occupants in the limited space available in cities. As stated earlier, the most important perspective in constructing a high rise building is the pace that has to be maintained in the construction, while also maintaining the high strength requirements of the structures. Traditionally, RCC i.e. reinforced cement concrete is used to cater to this need. RCC has its unique set of advantages that it offers and thushas been one of the most sought after method of construction since a long period of time. But withthe development of modern machineries and techniques of construction various new concepts for constructing the structures with higher strength and lesser time are being developed.

II. CONCEPT OF STEEL-CONCRETE COMPOSITESTRUCTURES

Steel-concrete composite structures are gaining high importance in the construction of bridgesand highways, high rise buildings, etc. The sections in steel-concrete composite structures tend to use the compressive strength offered by concrete and the property of high resistance to tension and compression offered by the structuralsteel. Thus when these properties are combined in a section, the resultant section is a highly efficient and comparatively light weight section which most commonly find its way in the construction of high rise multi-storey buildings and highway bridges.

Along with the goodness ofstrengths from concrete and structural steel, steel concrete structures offer certain more benefits as well. They offer high resistance to corrosion and thus are highly durable in nature, they are considerably low maintenance structures when compared with RCC or steel structures, whichgives it a edge in becoming a preferred economic solution in life cycle of the structure. To have a better idea, it is found that the use weight ofcomposite structures to be lighter than as much as 25% when compared to rccstructures (Rathoreand Gupta 2020). This results in lesser efforts in erection and installation of the structure, thus saving labour and construction cost. These economic savings in construction of steel concrete composite structures can be as high as 10% when compared with the traditional RCC framed structures and around 7% when comparedwith steel structures (Rathore and Gupta 2020).

III. LITERATURE REVIEW

1) Chung and Lawson (2000)

In this research, an important aspect of steel concrete composite design i.e. the design of steel concrete composite beams with large web opening is discussed in detail. It is elaborated that how the proper positioning and sizing of openings in a steel concrete composite beam can affect on its favour ability towards bending as well as shear resistance. Here, various tests have also been performed to justify the analytical results obtained through the research. Also in thispaper the effect of openings on the deflections is explained with the help of juvenile factor whichis basically dependent on the location as well as the size of the openings. The results of the study are presented with the help of typical design tables. This paper is thus basically a design guide to Eurocode which is a detailed code of practice for analysis and design of steel and concrete composite structures for construction in European countries. Various tables are provided in order to facilitate the engineer to design those beam sections comfortably and suitably as per the requirement of the openings in the beam sections.

2) Spacone and Sherif (2004)

In this research paper, the present state of the art non-linear analysis in steel concrete compositestructures. It focuses on how frame elements can be computationally more faster as compared to continuum finite element models. Some of the systems that were used for the analysis purpose have used a great number of elements and degrees of freedoms which was not even thinkable to analyse a few years ago. In this research mostly the analysis has been carried out on the structural walls used in building construction. Here the models that are lumpedand have a distributed inelasticity as well as the models that a perfect and partial connections are also covered In this research paper, the present state of the art non-linear analysis in steel concrete composite structures. It focuses on how frame elements can be computationally more faster as compared to continuum finite element models. Some of the systems that were used for the analysis purpose have used a great number of elements and degrees of freedoms which was not even thinkable to analyse a few years ago. In this research mostly the analysis has been carried out on the structural walls used in building construction. Here the models that are lumped and have a distributed inelasticity as well as the models that a perfect and partial connections are also covered.

3) Wang (2005)

In this research paper, the focus of study is directed towards the performance of composite structures in fire and the design of fire resistant composite structures. The main components of a composite structure i.e. floor systems, joints and the slab are considered in this study. Various experiments including the experiments on the joints provided in fire, simulation using finite element software, the component method, the performance of different types of joints under the action of elevated temperatures and also the effect of behaviour of joint on overall steel concrete composite structure. Also, the effects of the structural behaviours as a whole are discussed. With the help of this paper we have understood the recent advancements in structural behaviour under the conditions of fire. Thus ithas shed light on most recent studies in the design and execution of composite floors, composite column and beam column joints in steel-concrete composite structures.

4) Bouazaoui and Perrenot (2006)

In this paper the mechanical behaviour of a steel and concrete composite structures using experimental analysis is discussed. Here the steel and concrete i.e. the steel girder and the concrete slab are assembled with the help of adhesives. Hence the effect of the natures of the adhesive used and the irregularity generated using the adhesives are studied and its effects on parameters such as ultimate load and mechanical performance is studied. Two different adhesives are used in this experiment and they are epoxy adhesives and polyurethane. Also their thicknesses in longitudinal and transverse directions are varied for studying the results.

And it can be concluded based on this research that a steel concrete composite structure can be enhanced using adhesives. It is observed that the failure of such sections is due to the crack in concrete slab and yield of the steel girder used. In such a situation there is no problem with the adhesive whatsoever used. Another failure pattern observed particularly in case of polyurethane adhesive is that crack of the concrete slab and vertical displacement of steel girder. But in conclusion, the composite structures constructed using the adhesives give better performance in all types of composite structures application including that of a bridge structure.

5) Maiorana et al. (2008)

In this paper, in order to increase the flexural strength of a steel concrete composite structure technique using the Fibre Reinforced polymer is used and discussed. Extensive research has been done using FRP on concrete and masonry structures but very few literatures are available on strengthening of steel concrete composite structures. Guidelines regarding the application of externally bonded FRP strengthening is available but to analyse its behaviour pattern one has to rely on the hypothesis posed by the elastic behaviour. In this research, focus has been devoted towards the analytical procedure to predict the behaviour of the FRP reinforced steel-concrete composite structure. Also the non-linear behaviour is taken into account. In order to provide a basis of research the analytical results obtained are to be compared to that of some previously available experimental results. In general sense, in this research an approach towers analytical prediction of behaviour based on the strain compatibility and cross-sectional compatibility in steel-concrete composite structures is proposed.

6) Marian and Khalil (2010)

In this research, the aspect that when a steel concrete composite beam is subjected to extreme bending or hogging moment , the bottom flange of the used steel section preferably a steel I- section of the steel concrete composite beam is found to be under compression and thus is very sensitive toward the phenomenon of restrained distortional buckling. As it is known that the design criterion suggested in the Eurocode 4 is based on the inverted U-frame model, thus it does not consider the inelastic moment redistribution mostly in case of the steel concrete composite beams which are statistically indeterminate. In this research, the emphasis is given on the present methods of the calculation of the buckling strength in distortion in steel-concrete composite beams and suggests some improvements over the presently prevailing methods.

7) Patil and Kumbhar (2013)

In this paper, a ten storied tall Reinforced concrete framed building structure is considered for the purpose of analysis by the technique of non-linear dynamic analysis. The ideology behind this research is to run the analysis for the different seismic intensities and then to study the seismic responses of those buildings. For the purpose of analysis, a software platform called SAP2000 is used. For the purpose of experimentation 5 different time histories are considered by the researchers, which consider 5 different intensities of earthquake i.e. V, VI,VII, IX AND X as per the Modified Mercalli's Intensity Scale i.e. MMI. Thus an attempt to study the relation between the seismic intensity and seismic response is made. Thus from this study it concludes that it is essential for a designer to carry out detailed time history analysis of a structure to ensure its full safety against the earthquake forces. In conclusion, it is said that the seismic responses in the form of base shear and story drifts and story displacements vary in similar manner for all intensities and all time histories considered. This also concludes that Time History Method is very much more realistic method used for the purpose of seismic analysis of a building and gives better check results for the safety of the considered structure.

8) Wagh and Waghe (2014)

In this research paper, it is said that since recent times widespread acceptance has been given to use of composite structures i.e. the structures with steel and concrete it need to compared on practical grounds with conventional reinforced concrete structures. Here, four different tall buildings are considered to be analysed by using Staad. Pro and also a detailed estimations of cost is carried out using MS-Excel. It was observed that, in case of composite structures due to smaller section sizes the self-weight of the structures reduces significantly and ultimately affects the cost the structure. Also due to less bending moments incurred in composite structures the size of foundations also reduce. Due to enhanced ductility from steel composite structures also tend to perform better under the action of earthquakes. And to summarize, composite structures also take less time as compared to reinforced concrete structures for constructions and are economical in overall scene.

9) Sebastian and McConnel (2015)

In this research paper, a nonlinear finite element program of advanced level was developed for the purpose of analysis of in general structures made of composite materials i.e. steel and concrete.

Here the concrete used is described as nonlinear isotropic and elastic before the cracks appear and orthotropic and nonlinear afterwards. Whereas the steel is taken as initially elastic and strain hardening capabilities after yielding. It is observed that the results obtained from the use of this program on steel-concrete composite sectionsyielded results very similar to the ones observed on actual experiments performed on the sample till the failure of the sections. Thus it can beconcluded that the program developed isextremely effective in the predictions ofbehaviour during failure in case of general steel concrete composite structures.

10) Yu-Tao-Guo et al. (2018)

In this research paper, it is stated that the steel- concrete-steel composite structures having orthogonal longitudinal and transverse steel websgenerally showcase higher values in ductility, strength, blast resistance ,construction efficiency, impact value, etc when compared with the conventional systems like Reinforced concrete structures or steel structures and thus offering a competitive option of multiple types of projects.It mainly finds its applications in protective structures, marine structures like bridges etc. The steel-concrete-steel composite structures havemultiple mechanisms to resist and transfer theshear forces as compared to steel beams or RC beams. In order to study this through a total of 16 tests for shear transfer mechanisms were carried out and a thus a theoretical analysis was conducted. It was observed that the major reason for the failure of steel-concrete-steel composites under shear was due to shear compression. The angle formed in concrete cracks was somewhere around 45 degrees initially and later on developed to be around 30 degrees when the loads were increased.

11) Rathore and Gupta (2020)

In this paper, a time history analysis report performed on a 10 storey reinforced concrete building is presented. As per the researcher, in case a building is improperly or inadequately designed and analysed for earthquake response, itleads to a devastating collapse of partial or entire building structure causing great losses of life and property. Time History analysis being a reliable and practical oriented approach for the analysisof a building by non-linear dynamic method is preferred by the researcher to carry out his research in this report. The software platform used by the researcher used in this research is ETABs by Computes and Structures Inc. i.e. CSI and earthquake time history used by the researcher to analyse the 10 storey RCC structureis El-Centro 1940 Earthquake. As per the report the main parameters in seismic analysis of a structure are the mass, ductility, damping, load carrying capacity and the stiffness of the structure. Also different response parameters like story displacements, base shears and story drifts are also determined and compared against thedesirable limits from the IS 1893:2002.

IV. RESEARCH GAP AND METHODOLOGY ADOPTED

In India, IS 11384 is the code designated for the guidelines regarding the construction of Steel-Concrete Composite Buildings. However, it does not provide a detailed design and analysis procedure for the same. Furthermore, the concept of Steel-Concrete Composite Column has been totally neglected in the code. Due to this reason, in India most of the design of Steel-Concrete Composite Column is carried out using the American code AISC 360-16. This gives rise to differences in design approaches as for analysing and designing a steel-concrete composite building we rely on both Indian and American standards. This can be explained as, for designing of the steel-concrete composite column AISC 360-16 is used, for beam and other structural member design the prominent Indian Standards like IS 456:200 is used and for earthquake analysis IS 1893:2016 is used. Hence, the project focuses on analysis and design of a building using the practices followed in India for construction of steel-concrete composite structures To carry out this analysis and design, the finite element modelling based software from The Computers and structures Inc., ETABs is used. The time histories for performing nonlinear time history analysis are obtained from Pacific earthquake Engineering Research (PEER) groundmotion database. A software developed by a Seismosoft software company named as “Seismomatch” is used for the purpose of spectral matching of the earthquake time historieswith respect to the target response spectrum.

V. OBJECTIVE OF THE STUDY

The present study is aimed at determining the seismic behaviour of a building under nonlinear time history analysis. The objectives of the present study includes :

1) To analyse and design a steel-concrete composite regular building using Indian standards.

ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 11 Issue VI Jun 2023- Available at www.ijraset.com

2) To compare the difference in the section sizes obtained after the design.
3) To perform nonlinear time history analysis using scaled time histories of 11 earthquakes on the above buildings in both the directions using their respective codes of practices.
4) To compare the results from nonlinear analysis in form of factors like capacity curve, story displacements, story drifts and base shears of the buildings with each other and tabulate the observations.
5) To draw conclusions based on the observations from comparisons.

VI. MODELLING

A. Selection of Steel-Concrete CompositeColumn Section

After detailed evaluation of various types of steel-concrete structures ,different sections of steel-concrete composite columns were selected for final selection. Firstly, a concrete in filledsteel section was considered and was supposed tobe connected with the help of welding with beams of steel I-section. However, as we know, during earthquake loading, high amounts of loadshaving cyclic nature are imparted on the building.It is evident that, the welding technique of connection is apparently not very effective when it comes to fatigue resistance that will be arising due to earthquake loads. Hence, this arrangement was not considered for the study. To tackle this issue of the beam-column connection that would be efficient in the event of an earthquake, a concrete encased steel-concrete composite section was selected with reinforcing bars presentat the edges for better and easy connectivity with the beams. The beams were designated to be made of reinforced cement concrete and thus the frames could be considered as reinforced concrete frame with steel-concrete composite columns and RC beams. The cross section of the column section selected is as shown in the Figure3.2

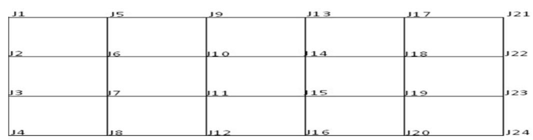


Figure 3.2 Joint Designation for the Building

In this column section, the steel section used and its properties are as given in Table .

Table: Steel Section Specifications for Indian Building

Steel Section Data	Indin
Steel Section Used	ISM200
Depth of The Section	200
Width of Flange	100
Thickness of Flange	10.8
Thickness of Web	5.7

The grade of steel used is E345 in case of the building designed using Indian Standards. The grades of concrete selected are M30 and C25/30 for the building to be designed using Indian standards.

Table :General Properties of the Selected Frame

Sr.No	Assumed Property	Specification
1	Height of the Frame	5 meters
2	No. of Stories	1
3	Size of Column	(300 X 300) mm
4	Size of Beam	(300 X 300) mm
5	Earthquake Zone	5
6	Zone Factor	0.36
7	Importance Factor	1
8	Response Reduction Factor	5.0
9	Soil Type	Medium Stiff Soil

The characteristics of the sections selected for the framewith steel-concrete Composite columns and reinforced concrete column are as given in Table

Table 4.3 Characteristics of the section in the frame with Steel-concrete Composite column and Reinforced concrete Column

Property	Frame with Steel-Concrete Composite Column	Frame with Reinforced Concrete Column
Grade of Concrete	M30	M30
Grade of Steel	E345	-
Grade of Rebars	Fe500	Fe500
No. of Rebars	4 rebars of 12 mm dia at the Corners	4 rebars of 12 mm dia at the Corners
Steel section used	ISMB 200	-

B. Building Plan

The building plan selected for this research is primarilya regular rectangular building plan with (6 X 4) bays. The rectangular building plan is selected so as to enable the analysis in the two orthogonal directions. The dimensions of the building plan have been selected in such a way that the time period of the building along both the directions vary by more than 10% so as to be considerable as different seismic response along the respective directions. The centre to centre distance considered between the two columns in the plan is 4 meters. Thus, the dimensions of the building plan are (20 X 12) meters.

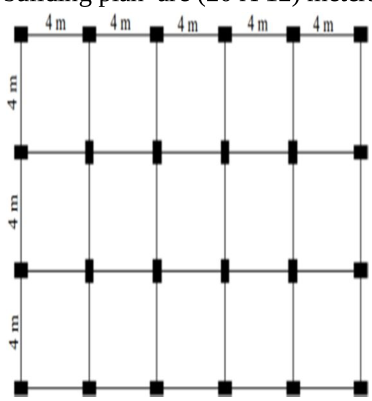
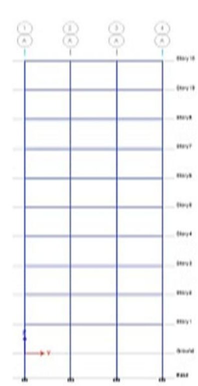
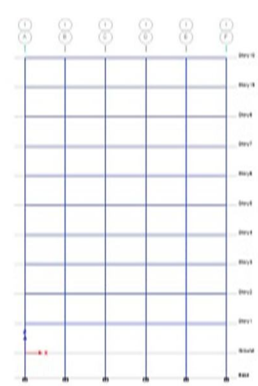


Figure 4.2 Plan of the Building Selected(AutoCAD v18)



Elevation of the Building Selected from face along Y-DirectionEABsv23.1.1



Elevation of the Building Building Selected from face along X- Direction

Table 4.4 Specifications and Consideration in the Building Plan

1	Number of floors in the building	10 (G+9)
2	Span of beam along X – Direction	4 m
3	Span of beam along Y – Direction	4 m
4	Height of the column	3 m
5	Length of the building along X – Direction	20 m
6	Width of the building along Y – Direction	12 m
7	Slab Thickness	150 mm
8	Grade of Concrete	M 30
9	Grade of Longitudinal Reinforcements	Fe 415
10	Grade of Transverse Reinforcements	Fe 415
11	Cover to Beams	20 mm
12	Cover to Columns	40 mm
13	Minimum Diameter of Longitudinal Reinforcements	12 mm
14	Minimum Diameter of Transverse Reinforcements	8 mm
15	Seismic Zone and Zone Factor	V and 0.36
16	Type of the Frame	SpecialMomentResistig Frame (SMRF)
17	Type of Soil	Medium Stiff Soil
18	Importance Factor	1

VII. LATERAL LOAD CALCULATIONS AS PER IS1893:2016

For calculating the laterals loads as per IS 1893, static analysis procedure has been used. This method gives thevalues of base shear and lateral forces a each story based on analytical formulas considering the properties assumed in the building. Based on the calculations, the total seismic mass of the building was calculated to be 15431.67 kN and on the basis of the diensions of the building the approximate fundamental translational natural period of the building was found to be asfollows :

Time period in X-direction, Tax = 0.604 secondsTime period in Y-direction, Tay = 0.780 seconds

Design Horizontal Seismic Coefficient, Ah for abuilding can be determined by :

Ah= Z/2*I/R*Sa/gHere,

Z represents zone factor,

I represents Importance Factor,

R represents Response Reduction Factor,

Sa/g represents the design accelerationcoefficient

The value of Sa/g can be calculated using,Sa/g = 1.36/Ta (4)

(Sa/g)x = 2.251

(Sa/g)y = 1.74

Thus, based on the Sa/g value and theassumptions discussed in the previous section,

Ahx = 0.0810

Ahy = 0.0626

With the help of the value of Design horizontal Seismic Coefficient, the value of base shear can be determined as follows,

Vb = Ah x W.. (5)

Where, W is the seismic mass of the building.Vbx = 1249.96 kN

Vby = 956.750 kN

Based on these values of base shear, the natural forces and the story shear values can be obtained. The values of the story shear forces are as shown in Table

Table 3.4. Lateral Loads on the Building to be Designed by Indian Standards

Floor	Height	Q_{ix}	Q_{iy}	V_{ix}	V_{iy}
TerraceFloor Level	30	324.61	248.51	324.60	248.50
Nineth Floor Level	27	262.93	201.30	587.53	449.80
EighthFloor Level	24	207.75	159.05	795.28	608.85
SeventhFloor Level	21	159.06	121.77	954.34	730.62
Sixth Floor Level	18	116.86	89.47	1071.20	820.08
Fifth Floor Level	15	81.15	62.13	1152.35	882.21
Fourth Floor Level	12	51.94	39.76	1204.29	921.98
Third Floor Level	9	29.21	22.37	1233.51	944.34
Second Floor Level	6	12.98	9.94	1246.49	954.28
First Floor Level	3	3.25	2.49	1249.74	956.77

C. *Load Combinations for the Building Designed using Indian Standards*

As the primary goal of this study, is to carry out in depth earthquake analysis, the sructure is designed in order to bear the complete earthquake load in one particular direction at a instance. For attaining this purpose, a total of 13 load combinations, as prescribed by the IS 1893 : 2016 are considered. The load combinations considered are as shown below in Table4.4

Sr. No.	Load Combinations
1	1.5 (DL + LL)
2	1.5 (DL + EQ-X)
3	1.5 (DL - EQ-X)
4	1.5 (DL + EQ-Y)
5	1.5 (DL - EQ-Y)
6	1.2 (DL + LL + EQ-X)
7	1.2 (DL + LL - EQ-X)
8	1.2 (DL + LL + EQ-Y)
9	1.2 (DL + LL - EQ-Y)
10	0.9 DL + 1.5 EQ-X
11	0.9 DL - 1.5 EQ-X
12	0.9 DL + 1.5 EQ-Y
13	0.9 DL - 1.5 EQ-Y

D. *Need for Non-linear Analysis*

In general, a nonlinear analysis is needed when there exsists a nonlinear relationship in between the force and the dispacements arising due to that force in a building or any structure, which in other words can be said to as involvement of considerable inelastic deformation of a building or a structure under the action of a given load or force. Usually a finite element analysis based computing software is used to run the nonlinear analysis in order to obtain a the capacity of a structure to undergo inelastic deformation before failure. Thus, the practical need for non-linear analysis can be summarised as follows:

1) In the design of new building particularly in case, the building is located in a active earthquake zone.
2) To retrofit the existing building based on its behaviour during nonlinear analysis.

3) To determine the safety and reliability of the newly constructed buildings in event of an eatrhquake.
4) To study the progressive failure pattern of a structure

Table 4.5 Gravity Loads Considered For the Building

Sr. No	Type of Loading	Load to be Applied
1	Live Load on Floor	3 kN/m^2
2	Live Load on Roof	1.5 kN/m^2
3	Floor Finish	1 kN/m^2
4	Roof Finish	1 kN/m^2
5	Water Proofing	1 kN/m^2
6	Dead Load of Wall	20kN/m^3

E. *Load Combinations Considered for the Designof the buildings*

In this study, the buildings to be designed by both Indian standards are first designed using the gravity loads only. The reason for following this path is to have the section sizes more suitable for calculating the lateral forces to obtain more accurate results from the seismic analysis. Thus, the load combinations from the Indian standards and the involving only the gravity loads are considered for this analysis. For the building tobe designed with the help of Indian standards the load combination used is (1.5DL + 1.5LL), whereDL and LL stand for Dead Load and Live Load, respectively.

Table 4.6 Summary of Design of Buildings using Indian and European Standards

Parameter	Building Designed using Indian standards
Load Combinations used	(1.5DL + 1.5LL)
Size of Beam	(360 X 300) mm
Reinforcement Details forBeams	2 Bars of 12 mm at the top2 Bars of 16 mm at the bottom
Stirrups for Beams	8 mm spaced at 250 mm c/c
Size of the Column	(400 X 350) mm
Reinforcement Details for Columns	4 Bars of 20 mm at corners
Stirrups for Columns	8 mm spaced at 150 mm c/c
Steel section used in Columns	ISMB 200
Remarks	The section sizes obtained for this building are comparatively larger owing to the loadcombination used in design.

Table 3.6 Ground Motions Considered for Analysis

	Name of Earthquake	RSN	Station	PGA/PGV	Magnitude
1	Chuetsu-oki_Japan	4845	Joetsu Oshimaku Oka	2.93	6.8
2	Northern Calif	101	Cape Mendocino	1.97	5.2
3	Victoriya Mexico	265	Cerro Prieto	1.91	6.33
4	Coyote Lake	145	Coyote Lake Dam - SouthwestAbutment	1.39	5.74
5	Parkfield	28	Cholame - Shandon Array	1.17	6.19
6	Chi-Chi_Taiwan	1197	CHY028	1.03	7.62
7	Helena, Montana	1	Carroll College	1.02	6.0
8	Kobe Japan	1119	Takarazuka	1.01	6.9
9	Chuetsu-oki_Japan	4874	Oguni Nagaoka	0.79	6.8
10	Hectormine	1795	Joshua Tree N.M. -Keys View	0.68	6.8
11	Kobe Japan	1120	Takatori	0.54	6.9

VIII. RESULTS AND DISCUSSION

A. *Moment-Curvature Diagram*

Moment-curvature diagram represents thechange in moment of resistance of a section with respect of the curvature in graphical form. The axial force levels for the columns with steel- concrete composite columns and the reinforced concrete columns are 27.37 kN and 24.79 kNrespectively. The comparative moment-curvature diagram of the column section with the steel-concrete composite section and moment curvature diagram of the column with the reinforced concrete section as shown in figure 5.1

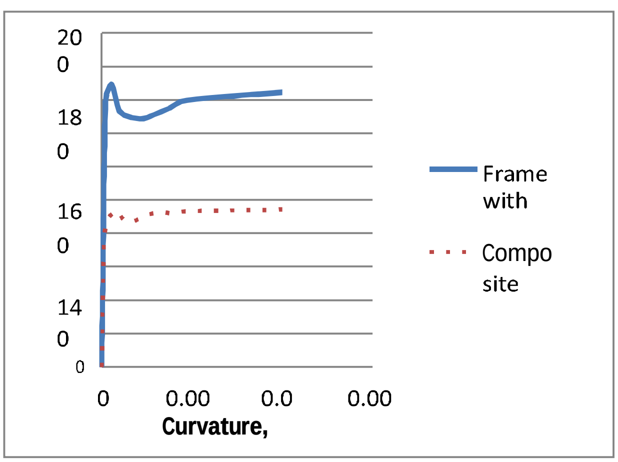


Figure 5.1 Moment-Curvature Diagram For ColumnSection (ETABs v23.1.1)

ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.538
Volume 11 Issue VI Jun 2023- Available at www.ijraset.com

B. P-M Interaction Curve

P-M interaction curve is useful in determining the design strength of the column sections, as it isknown that in case of designed considering the bending moment, shear force and the axial force, unlike the beams in which the axial force does not play a very significant role. Thus, the P-M interaction curve gives an indication of the strength capacity of the member for resisting theP and M which is the axial capacities and the acceptable moments in the section. Figure 5.2

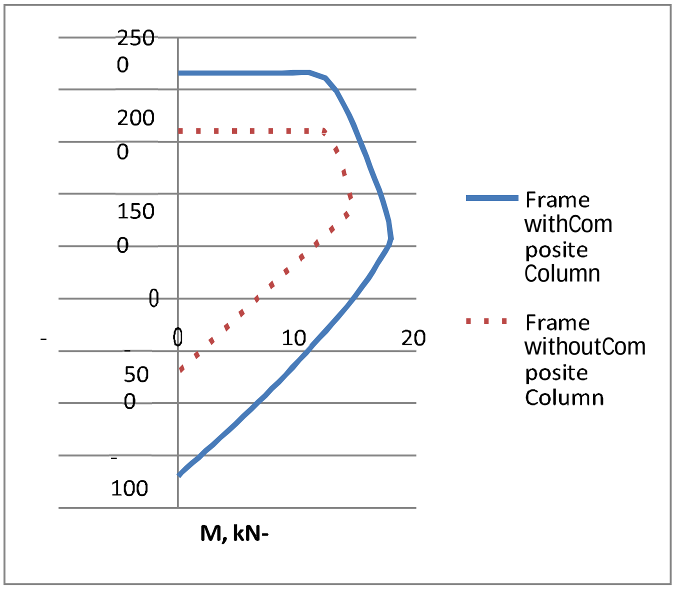


Figure 5.2 P-M Interaction Curve for Column Section (ETABs v23.1.1)

From the P-M interaction or the column interaction curve obtained for the steel-concrete composite column and the reinforced concrete column of same size, it can be clearly seen that, the steel-concrete compositecolumn offers a higher strength capacity for resisting the axial forces and the acceptable moments in the section.

C. Results from NLTHA in X-Direction

Table 5.1 Maximum Displacement from Building Designed Using IndianStandards along X-Direction for Chuetsu-oki Japan

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift %
Terrace Floor Level	30	177.5	0.01
Nineth Floor Level	27	174.6	0.02
Eighth Floor Level	24	169.7	0.03
Seventh Floor Level	21	161.9	0.04
Sixth Floor Level	18	149.8	0.06
Fifth Floor Level	15	132.2	0.08
Fourth Floor Level	12	108.3	0.09
Third Floor Level	9	80.6	0.10
Second Floor Level	6	49.3	0.10
First Floor Level	3	17.9	0.06
Ground Floor	0	0	0

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Table 5.2 Maximum Displacement from Building Designed Using IndianStandards along X-Direction for Northern Calif

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift %
Terrace Floor Level	30	231.2	0.02
Nineth Floor Level	27	226.1	0.03
Eighth Floor Level	24	216.7	0.05
Seventh Floor Level	21	202.2	0.07
Sixth Floor Level	18	181.6	0.09
Fifth Floor Level	15	155.2	0.10
Fourth Floor Level	12	123.8	0.12
Third Floor Level	9	89.2	0.12
Second Floor Level	6	52.4	0.11
First Floor Level	3	18.4	0.06
Ground Floor	0	0	0

Table 5.3 Maximum Displacement from Building DesignedUsing IndianStandards along X-Direction for Victoriya Maxico

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift %
Terrace Floor Level	30	238	0.01
Nineth Floor Level	27	233.8	0.02
Eighth Floor Level	24	226.5	0.03
Seventh Floor Level	21	216.2	0.05
Sixth Floor Level	18	199.9	0.08
Fifth Floor Level	15	176.3	0.10
Fourth Floor Level	12	145	0.13
Third Floor Level	9	106.7	0.14
Second Floor Level	6	63.8	0.14
First Floor Level	3	22.7	0.08
Ground Floor	0	0	0

Table 5.4 Maximum Displacement from Building Designed Using Indian Standards along X-Direction for Coyote Lake

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift %
Terrace Floor Level	30	260.7	0.02
Nineth Floor Level	27	256.2	0.03
Eighth Floor Level	24	248	0.05
Seventh Floor Level	21	234.5	0.06
SixthFloor Level	18	215.5	0.09
Fifth Floor Level	15	188.7	0.12
Fourth Floor Level	12	153.5	0.14
Third Floor Level	9	112.11	0.15
Second Floor Level	6	66.1	0.14
First Floor Level	3	23	0.08
Ground Floor	0	0	0

Table 5.5 Maximum Displacement from Building Designed Using Indian Standards along X-Direction for Parkfield

Story	Elevation(m)	MaximumDisplacement (mm)	Maximum Drift %
Terrace Floor Level	30	237.9	0.02
Nineth Floor Level	27	231.8	0.04
Eighth Floor Level	24	220.2	0.06
Seventh Floor Level	21	201.5	0.09
Sixth Floor Level	18	175.3	0.11
Fifth Floor Level	15	143.7	0.09
Fourth Floor Level	12	116.3	0.10
Third Floor Level	9	87.1	0.12
Second Floor Level	6	52.6	0.11
First Floor Level	3	18.7	0.06
Ground Floor	0	0	0

Table 5.6 Maximum Displacement from Building Designed Using Indian Standards along X-Direction for Chi-Chi Taiwan

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift %
Terrace Floor Level	30	210.8	0.01
Nineth Floor Level	27	208	0.02
Eighth Floor Level	24	202.8	0.03
Seventh Floor Level	21	193.9	0.05
Sixth Floor Level	18	180.3	0.07
Fifth Floor Level	15	160.4	0.09
Fourth Floor Level	12	133.2	0.11
Third Floor Level	9	99.4	0.13
Second Floor Level	6	59.5	0.13
First Floor Level	3	21.4	0.07
Ground Floor	0	0	0

Table 5.7 Maximum Displacement from Building Designed Using Indian Standards along X-Direction for Helena, Montana

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift%
Terrace Floor Level	30	186.5	0.01
Nineth Floor Level	27	181.7	0.02
Eighth Floor Level	24	173	0.04
Seventh Floor Level	21	159.4	0.06
Sixth Floor Level	18	143.3	0.08
Fifth Floor Level	15	123.9	0.10
Fourth Floor Level	12	100.1	0.13
Third Floor Level	9	72	0.15
Second Floor Level	6	43.2	0.14
First Floor Level	3	15.8	0.07
Ground Floor	0	0	0

Table 5.8 Maximum Displacement from Building Designed Using Indian Standards along X-Direction for Kobe, Japan (1119)

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift%
Terrace Floor Level	30	186.5	0.01
Nineth Floor Level	27	181.7	0.02
Eighth Floor Level	24	173	0.04
Seventh Floor Level	21	159.4	0.06

Sixth Floor Level	18	143.3	0.08
Fifth Floor Level	15	123.9	0.10
Fourth Floor Level	12	100.1	0.13
Third Floor Level	9	72	0.15
Second Floor Level	6	43.2	0.14
First Floor Level	3	15.8	0.07
Ground Floor	0	0	0

Table 5.9 Maximum Displacement from Building Designed Using IndianStandards along X-Direction for Chuetsu-oki Japan (4874)

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift%
Terrace FloorLevel	30	202.9	0.01
Nineth Floor Level	27	198.9	0.02
Eighth Floor Level	24	191.7	0.03
Seventh Floor Level	21	180.4	0.04
Sixth Floor Level	18	163.8	0.06
Fifth Floor Level	15	142.8	0.08
Fourth Floor Level	12	117.8	0.09
Third Floor Level	9	87.8	0.10
Second Floor Level	6	53.1	0.10
First Floor Level	3	19.1	0.06
Ground Floor	0	0	0

Table 5.10 Maximum Displacement from Building Designed Using IndianStandards along X-Direction for Hectormine

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift%
Terrace Floor Level	30	240.7	0.02
Nineth Floor Level	27	235.5	0.03
Eighth Floor Level	24	225.7	0.05
Seventh FloorLevel	21	209.6	0.08
Sixth Floor Level	18	186.5	0.10
Fifth Floor Level	15	157.6	0.09
Fourth Floor Level	12	129.4	0.12
Third Floor Level	9	94	0.13
Second Floor Level	6	54.9	0.12
First Floor Level	3	19.5	0.07
Ground Floor	0	0	0

Table 5.11 Maximum Displacement from Building Designed UsingIndianStandards along X-Direction for Kobe (1120)

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift %
Terrace Floor Level	30	186.5	0.02
Nineth Floor Level	27	181.7	0.03
Eighth Floor Level	24	173	0.05
Seventh FloorLevel	21	159.4	0.05
Sixth Floor Level	18	143.3	0.06
Fifth Floor Level	15	123.9	0.08
Fourth Floor Level	12	100.1	0.09
Third Floor Level	9	72	0.10
Second Floor Level	6	43.2	0.09
First Floor Level	3	15.8	0.05

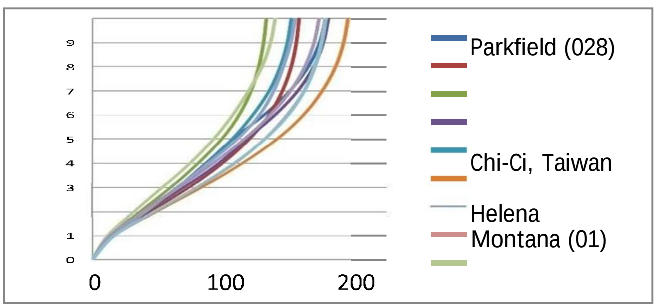

Figure 5.3 Maximum Lateral Displacements from the BuildingDesigned UsingIndian Standards along X-Direction

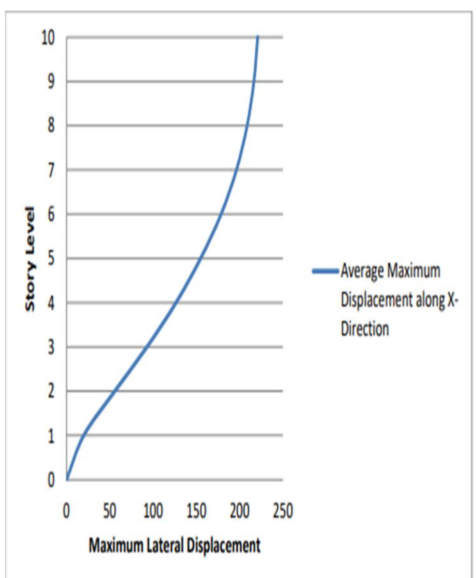

Figure5.4 Average of Maximum Lateral Displacements on Building designed Using Indian Standards along X- direction

The average maximum value of the base shearalong the Y-Direction on the Indian Building is 20274.55 kN.

D. Results from NLTHA along Y-Direction

The displacement and the drift values obtainedfrom the Indian Model along Y-Direction are as given below :

Table 5.13 Maximum Displacement from Building Designed Using IndianStandards along Y-Direction forChuetsu-Oki Japan

Story	Elevation(m)	MaximumDisplacement (mm)	Maximum Drift%
Terrace Floor Level	30	266.8	0.01
Nineth Floor Level	27	262.9	0.02
Eighth Floor Level	24	255.7	0.04
Seventh Floor Level	21	243.3	0.07
Sixth Floor Level	18	223	0.10
Fifth Floor Level	15	192.9	0.12
Fourth Floor Level	12	158	0.13
Third Floor Level	9	118.7	0.16
Second Floor Level	6	71.9	0.15
First Floor Level	3	25.6	0.09
Ground Floor	0	0	0

Table 5.14 Maximum Displacement from Building Designed UsingIndianStandards along Y-Direction for Northern Calif

Story	Elevation(m)	Maximum Displacement(mm)	MaximumDrift%
Terrace Floor Level	30	214.6	0.02
Nineth Floor Level	27	208	0.04
Eighth Floor Level	24	196.5	0.06
Seventh Floor Level	21	179.9	0.05
Sixth Floor Level	18	165.1	0.06
Fifth Floor Level	15	146.6	0.08
Fourth Floor Level	12	123.1	0.09
Third Floor Level	9	96.3	0.12
Second Floor Level	6	60.6	0.13
First Floor Level	3	22.4	0.07
Ground Floor	0	0	0

Table 5.15 Maximum Displacement from Building Designed UsingIndianStandards along Y-Direction for Victoriya Mexico

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift%
Terrace Floor Level	30	211.8	0.01
Nineth Floor Level	27	207.6	0.02
Eighth Floor Level	24	200.3	0.03
Seventh FloorLevel	21	190	0.05
Sixth Floor Level	18	173.7	0.08
Fifth Floor Level	15	150.1	0.10
Fourth Floor Level	12	118.8	0.07
Third Floor Level	9	99.2	0.14
Second Floor Level	6	56.3	0.14
First Floor Level	3	15.2	0.05
Ground Floor	0	0	0

Table 5.16 Maximum Displacement from Building Designed UsingIndianStandards along Y-Direction for Coyote Lake

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift%
Terrace Floor Level	30	234.8	0.01
Nineth Floor Level	27	230.4	0.03
Eighth Floor Level	24	222	0.05
Seventh Floor Level	21	208	0.08
Sixth Floor Level	18	183.7	0.08
Fifth Floor Level	15	160	0.11
Fourth Floor Level	12	127.2	0.09
Third Floor Level	9	-99.4	0.12
Second Floor Level	6	-63.6	0.13
First Floor Level	3	-24.1	0.08
Ground Floor	0	0	

Table 5.17 Maximum Displacement from Building Designed UsingIndianStandards along Y-Direction for Parkfield

Story	Elevation(m)	Maximum Displacement (mm)	Maximum Drift
			%
Terrace Floor Level	30	176.2	0.02
Nineth Floor Level	27	171.5	0.02

Eighth Floor Level	24	165.3	0.03
Seventh Floor Level	21	156	0.05
Sixth Floor Level	18	142.4	0.06
Fifth Floor Level	15	-125.3	0.06
Fourth Floor Level	12	-107.6	0.08
Third Floor Level	9	-83.3	0.10
Second Floor Level	6	-52.2	0.11
First Floor Level	3	-19.3	0.06
Ground Floor	0	0	0

Table 5.18 Maximum Displacement from Building Designed UsingIndianStandards along Y-Direction for Chi-Chi Taiwan

Story	Elevation	Maximum	Maximum Drift
	(m)	Displacement (mm)	%
Terrace Floor Level	30	278.1	0.02
Nineth Floor Level	27	271.6	0.04
Eighth Floor Level	24	261	0.06
Seventh Floor Level	21	244	0.08
Sixth Floor Level	18	220.8	0.10
Fifth Floor Level	15	189.8	0.12
Fourth Floor Level	12	154.3	0.14
Third Floor Level	9	113.1	0.15
Second Floor Level	6	67.8	0.14
First Floor Level	3	24.4	0.08
Ground Floor	0	0	0

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Table 5.19 Maximum Displacement from Building Designed UsingIndian Standards along Y-Direction for Helena Montana

Story	Elevation(m)	Maximum Displacement(mm)	Maximum Drift %
Terrace Floor Level	30	301.1	0.02
Nineth Floor Level	27	294	0.04
Eighth Floor Level	24	280.6	0.06
Seventh Floor Level	21	261.8	0.08
Sixth Floor Level	18	238.4	0.10
Fifth Floor Level	15	207	0.13
Fourth Floor Level	12	168.2	0.15
Third Floor Level	9	122.1	0.17
Second Floor Level	6	71.2	0.15
First Floor Level	3	25.4	0.08
Ground Floor	0	0	0

Table 5.20 Maximum Displacement from Building DesignedUsing IndianStandards along Y-Direction for Kobe 1119

Story	vation(m)	MaximumDisplacement(mm)	Maximum Drift %
Terrace Floor Level	30	309.3	0.02
Nineth Floor Level	27	302.7	0.04
Eighth Floor Level	24	290.2	0.07
Seventh Floor Level	21	269.5	0.10
Sixth Floor Level	18	239.7	0.12
Fifth Floor Level	15	203.7	0.13
Fourth Floor Level	12	165.6	0.15
Third Floor Level	9	120.3	0.17
Second Floor Level	6	70.8	0.15
First Floor Level	3	24.7	0.08
Ground Floor	0	0	0

Table 5.21 Maximum Displacement from Building Designed Using IndianStandards alongYdirection for Chuetsu-Oki Japan 4874

Story	Elevatio(m)	MaximumDisplacement(mm	Maximum Drift%
Terrace Floor Level	30	244.8	0.02
Nineth Floor Level	27	239.9	0.03
Eighth Floor Level	24	230.8	0.05
Seventh Floor Level	21	216.6	0.07
Sixth Floor Level	18	196.2	0.09

Fifth Floor Level	15	168.6	0.11
Fourth Floor Level	12	134.3	0.13
Third Floor Level	9	95.6	0.13
Second Floor Level	6	55.8	0.12
First Floor Level	3	19.9	0.07
Ground Floor	0	0	0

Table 5.22 Maximum Displacement fromBuilding Designed UsingIndianStandards along Y-Direction for Hectormine

Story	Elevation (m)	Maximum Displacemet (mm)	Maximum Drift%
Terrace Floor Level	30	226	0.02
Nineth Floor Level	27	220.2	0.03
Eighth Floor Level	24	210.4	0.05

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Seventh Floor Level	21	195.8	0.07
Sixth Floor Level	18	176.2	0.07
Fifth Floor Level	15	155.9	0.09
Fourth Floor Level	12	129.5	0.11
Third Floor Level	9	96.3	0.13
Second Floor Level	6	58.7	0.12
First Floor Level	3	21.5	0.07
Ground Floor	0	0	0

Table 5.23 Maximum Displacement from Building Designed Using IndianStandards along YDirection forKobe 1120

Story	Elevation(m)	MaximumDisplacement (mm)	Maximum Drift %
Terrace Floor Level	30	222.9	0.01
Nineth Floor Level	27	219.6	0.02
Eighth Floor Level	24	213.9	0.03
Seventh Floor Level	21	204.4	0.05
Sixth Floor Level	18	189.8	0.07
Fifth Floor Level	15	168.9	0.10
Fourth Floor Level	12	139.9	0.12
Third Floor Level	9	102.6	0.13
Second Floor Level	6	62.2	0.13
First Floor Level	3	23.3	0.08
Ground Floor	0	0	0

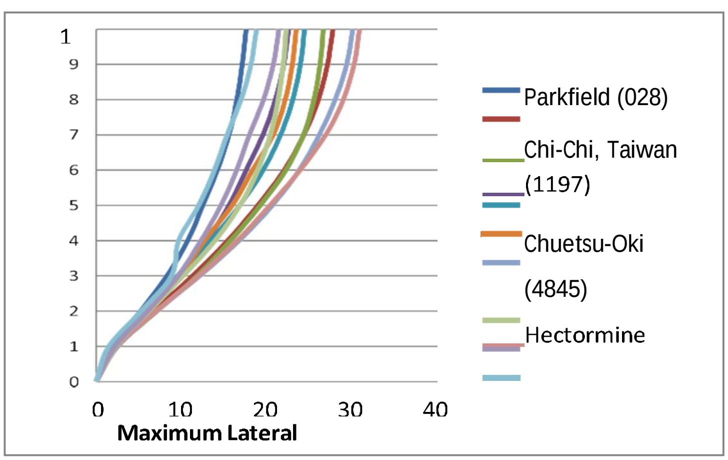

Figure 5.5 Maximum Lateral Displacements from the Building Designed UsingIndian Standards along Direction

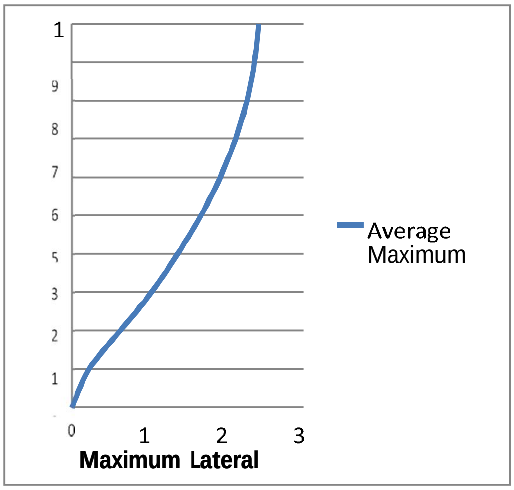

Figure 5.6 Average of Maximum Lateral Displacements from Building DesignedUsingIndian standards along Y-Direction

The values of the base shear for the Indian Model along Y-Direction are as given belowin

Table 5.24 Maximum Base Shear for Building DesignedUsing Indian standards along Y-Direction

Sr. No	Earthquake TimeHistory		Maximum BaseShearkN)	Minimum BaseShear(kN)	Absolute Maximum Base Sear(kN)
1	Chuetsu-oki_Japan	2	6200	-17820	26200
2	Northern Calif	1	9890	-23180	23180
3	Victoriya Mexico	1	8990	-22610	22610
4	Coyote Lake	1	9730	-25860	25860
5	Parkfield	2	1560	-19660	21560
6	Chi-Chi_Taiwan	1	7520	-25650	25650
7	Helena, Montana	1	9500	-26460	26460
8	Kobe Japan	2	4430	-21150	24430
9	Chuetsu-oki_Japan	1	8060	-20120	20120
10	Hectormine	2	1920	-23150	23150
11	Kobe Japan	2	4540	-22010	24540

IX. CONCLUSION

The building designed using Indian Standards differ in terms of section sizes. The study has been carried out on the behaviour of the buildings using different sections of steel-concrete composites. The study involves the effect of use of such sections on the building when compared to the regular reinforced concrete building. Various aspects of building behaviour are considered in different studies to predict the appropriateness of the steel-concrete composite sections under different circumstances. Also, studies have been carried out on the safety and serviceability of these sections in events of adversities like fire outbreaks.

Furthermore, effect of change in intensities of the earthquakes on the response of the structures have been studied. Different softwares offer different specialties and can be used based on the need andnature of the study. Extensive research has been carried out to understand the seismic behaviour of the buildings under varied circumstances. Thus, it can be concluded that, the steel-concrete composite sections used in a building result in imparting higher ductility to the building which constitutes to better seismic performance. The nonlinear dynamic analysis i.e. the time history analysis, among all other methods of seismic analysis including the equivalent static analysis, response spectrum method and pushover analysis, proves to be a very reliable method

X. FUTURE SCOPE

The present study is carried out for a regularbuilding with concrete encased steel column sections and RC beams. So, the area of research that has to be done are,

1) Different shape of column section can be used for analysis.
2) Study can be carried out on behaviour of building with concrete in filled steel column and steel beams.
3) The building selected in this study was a regular building, hence irregularities can be introduced in the building to study the change in behaviour.

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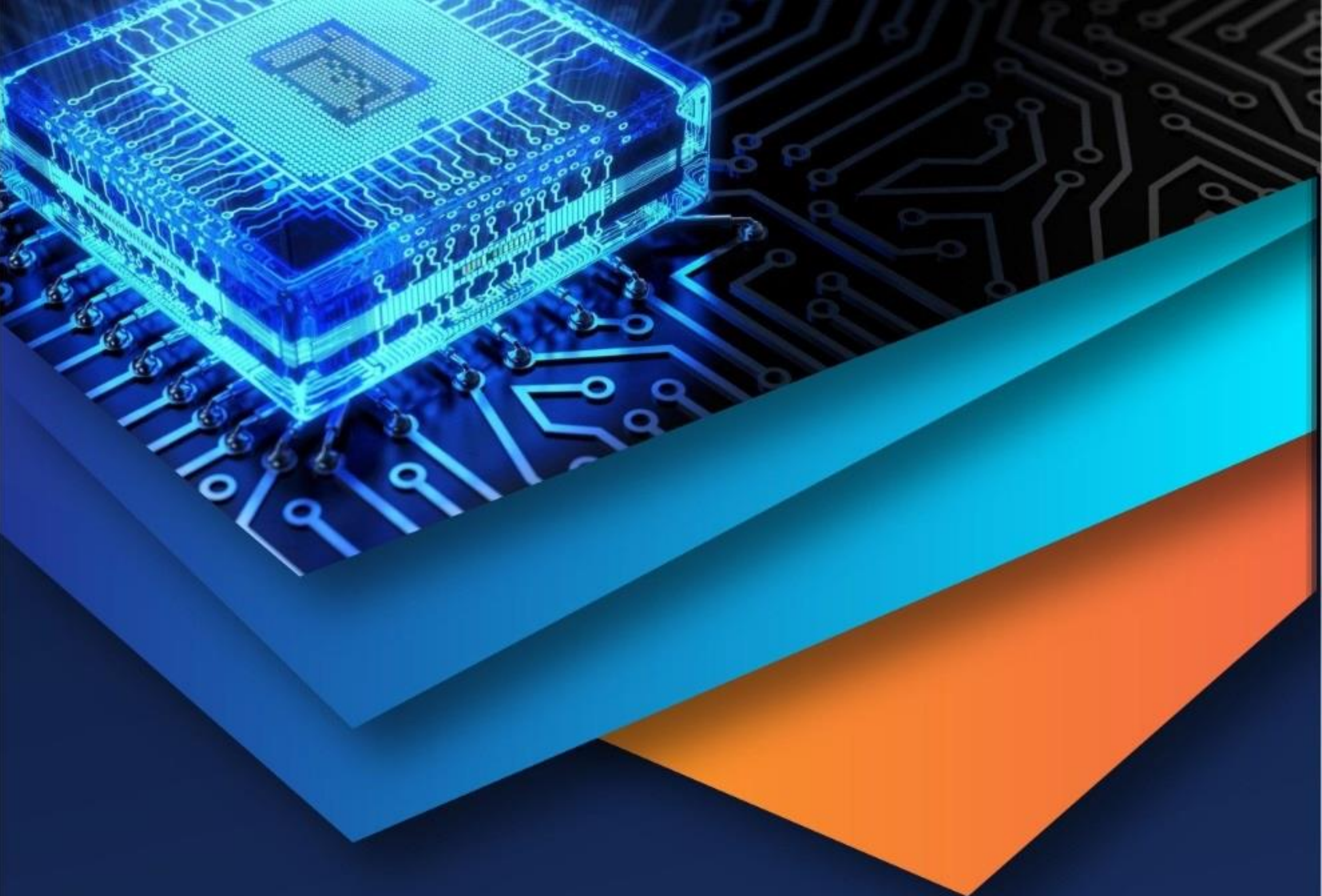

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