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SEISMIC BEHAVIOUR OF RCC FRAMED BUILDINGS WITH DIFFERENT STRUCTURAL FRAMING TYPES

AMBRISH G
Research Scholar, Department of Civil Engineering, University Visvesvaraya College of Engineering (UVCE) Bangalore University, Bangalore, India. Email: ambrishuvce@gmail.com

Dr. CHETHAN K
Associate Professor, Department of Civil Engineering, University Visvesvaraya College of Engineering (UVCE) Bangalore University, Bangalore, India. Email: k.chethan@gmail.com

Abstract

Beam-slab, flat plate, and flat slab systems are commonly used structural systems. Reinforced concrete multistory buildings with large spans can reduce the number of columns needed to support the building, which can reduce cost and increase the useable space. Structural failures due to seismic tremors in densely inhabited areas have led to improvements in seismic codes with the objective of improving seismic performance. The seismic resistance offered by a structure is primarily attributed to the combination of its elastic strength, damping capabilities, and inelastic deformability. Buildings with smaller translational natural periods attract higher design seismic force coefficients The inertial forces develop at each floor owing to floor accelerations, which can lead to damage to the floor and other floor components. Notably, increasing the stiffness of buildings has a counterproductive effect on reducing drift, as higher stiffness leads to greater floor acceleration This paper compares the seismic performance of four different structural systems in a medium-rise reinforced concrete building with a fixed base. The four selected structural forms for a fixed-base reinforced concrete building are subjected to a set of selected earthquakes and analyzed using nonlinear time history analysis to study the seismic response parameters, such as the time period, base shear, peak roof displacement, and peak roof acceleration, for soil type hard in seismic zone v as per BIS code 1893-2016(Part 1).

Keywords: Base shear, modal time period, nonlinear time history analysis, floor acceleration, structural framing.

1. INTRODUCTION

Structural failures due to seismic tremors in densely inhabited areas have led to improvements in seismic codes with the objective of improving seismic performance. A recent trend in building construction is to support slabs directly on columns or walls without employing any beams. This construction, commonly called flat slab construction, has become popular, particularly in commercial buildings.

This paper evaluates the seismic response of a medium-rise fixed-base building. Medium-rise reinforced concrete buildings with four structural forms (ref Fig 1), namely, the column-beam-slab system, column-beam-slab system (with lift core walls), flat slab system without column drops (with lift core walls) and flat slab system with column drops (with lift core walls), are considered and analyzed using a commercial finite element package, ETABS, which adopts nonlinear time history analysis under a set of selected earthquake data ignoring the infill effect. The buildings are assumed to be located in seismic zone v, where the soil type is hard, for an importance factor of 1.5 as per IS 1893-

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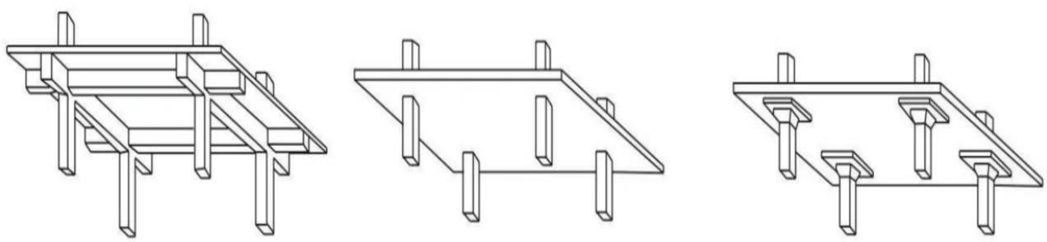
2016 (BIS, 2016). The selected building plans resemble a typical realistic construction for commercial buildings. The variations in the modal time period, base shear, peak roof displacement, and peak floor acceleration obtained upon analysis via nonlinear time history analysis for a set of selected earthquakes are compared.

2. METHODOLOGY

Time history analysis is carried out by adopting fast nonlinear analysis (FNA) using the commercial structural analysis package ETABS. The columns, as well as the beams, are modeled as line elements, while the lift core walls are modeled as shell elements. The floor slab is considered to be a rigid diaphragm and is modeled as a membrane element for the column beam slab system, while it is modeled as a thin shell element for the flat slab models. The typical plan, sectional view, and 3D views of medium-rise 14-floor buildings are shown in Table 2 for the four types of structural framing



a. Column-beam-slab

b. Flat slab without column drops

c. Flat slab with column drops

Fig 1: Framing systems considered for 14-floor building

systems. The material properties adopted for modeling the building, along with other details of the building, are presented in Table 3.

2.1 Loads:

The structural models are subjected to different loads. The loads considered for the analysis are as per the provisions of IS 875-1987 (Part I & II) (BIS, 1987a, 1987b) and IS 1893-2016 (Part 1) (BIS, 2016). The loads considered are i) load, ii) load, and iii) earthquake loads.

The gravity loads, such as dead and live loads, coming on the frames were calculated based on provisions given in IS 875 (Part I & Part II):1987 (BIS, 1987a, 1987b). The dead load consists of the self-weight of structural and nonstructural elements such as the wall load, floor finishes, and partition wall parapet load. The live load is considered for a commercial building.

Earthquake loads are considered for the analysis considering the structure to be located in seismic zone v, as per IS: 1893-2016(Part I) (BIS, 2016). For seismic loads, the mass source is defined according to IS: 1893-Part I, Table 10 (BIS, 2016).

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2.2 Time History Analysis:

The ground motion records recorded during earthquakes by seismographs are termed time histories. A time history record is typically an analog or digital record of ground accelerations.

The time histories of the accelerations were recorded in three orthogonal directions at the instrument location. The maximum amplitude is referred to as the peak ground acceleration or zero period acceleration.

The peak ground velocity and displacement are derived from the acceleration record. Time history analysis is an analysis in which the structure is subjected to an actual time history of past earthquakes.

Since it is a dynamic analysis method and incorporates material nonlinearity, it is considered one of the most realistic analysis methods for understanding structural behavior under seismic loading.

The commercial structural analysis package ETABS was used in this study to perform fast nonlinear analysis (FNA) for the time history analysis.

FNA is a modal analysis method that is preferred for time history analysis owing to its computational efficiency compared to methods such as direct integration (CSI, 2016).

Table 1: Time history data of the selected earthquakes for carrying out the time history analysis

Time history	Max. Acceleration (g)	Max. Velocity (cm/sec)	Max. Displacement (cm)
Bhuj (2001)	0.10	11.19	18.15
Chamoli (1999)	0.19	11.20	5.18
Chichi (1999)	0.36	21.54	21.88
Elcentro (1979)	0.315	62.992	56.505
Kobe (1995)	0.33	27.67	9.54
Loma Prieta (1989)	0.35	44.28	19.04
Northridge (1994)	0.57	51.82	9.00
IS- 1893 (matched)	0.20	15.42	34.18

2.3 Ground Motion Records:

Ground motion records are used to obtain the dynamic characteristics of ground motion, which significantly influence structural stability during an earthquake.

These characteristics include the duration of the earthquake or ground motion along with the peak displacement, velocity, and acceleration, as well as the frequency.

The stability of a structure is influenced by these external ground motion characteristics as well as the slenderness of the structure (Murty et al.). The time history data of the 7 earthquakes selected for the present study are shown in Table 1.

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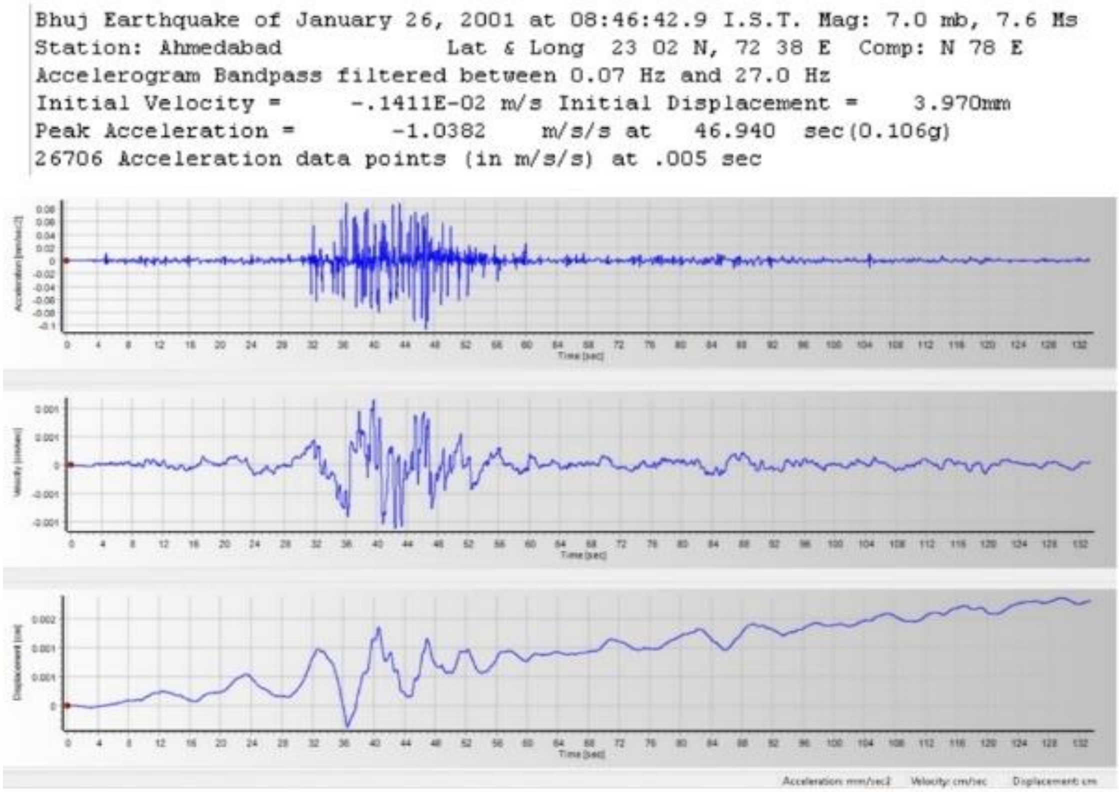

Fig 2: Time history plot of the Bhuj Earthquake

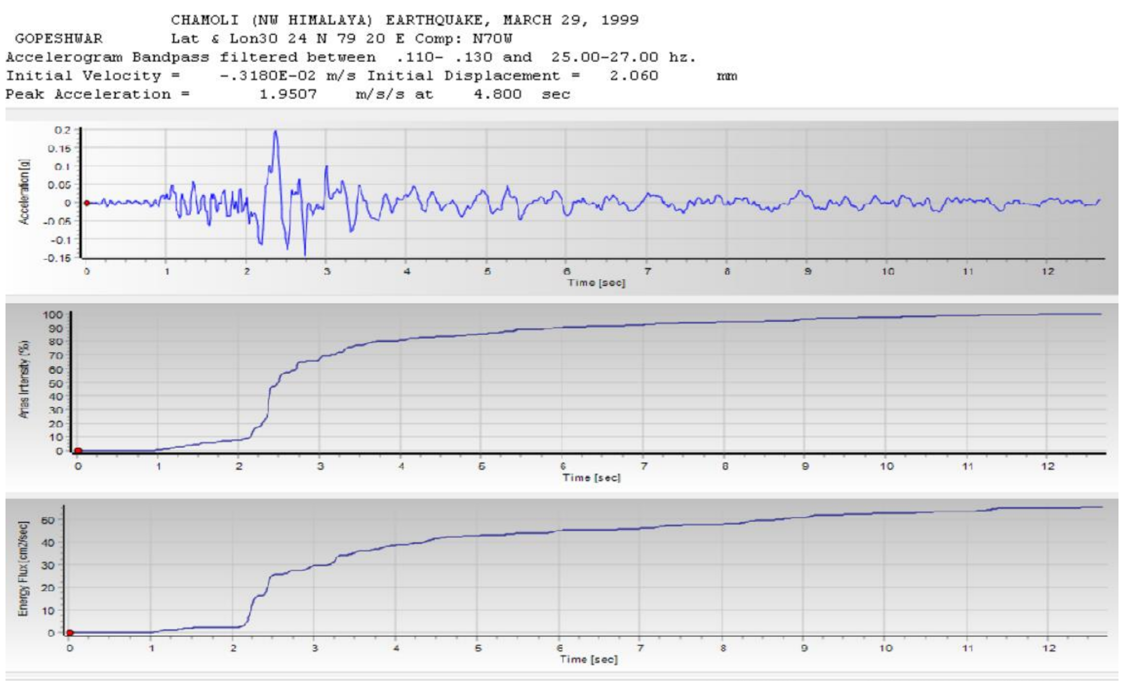

Fig 3: Time history plot of the Chamoli Earthquake

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The Chi-Chi (Taiwan) earthquake of September 20, 1999.
Source: PEER Strong Motion database
Recording station: TCU045
Frequency range: 0.02-50.0 Hz

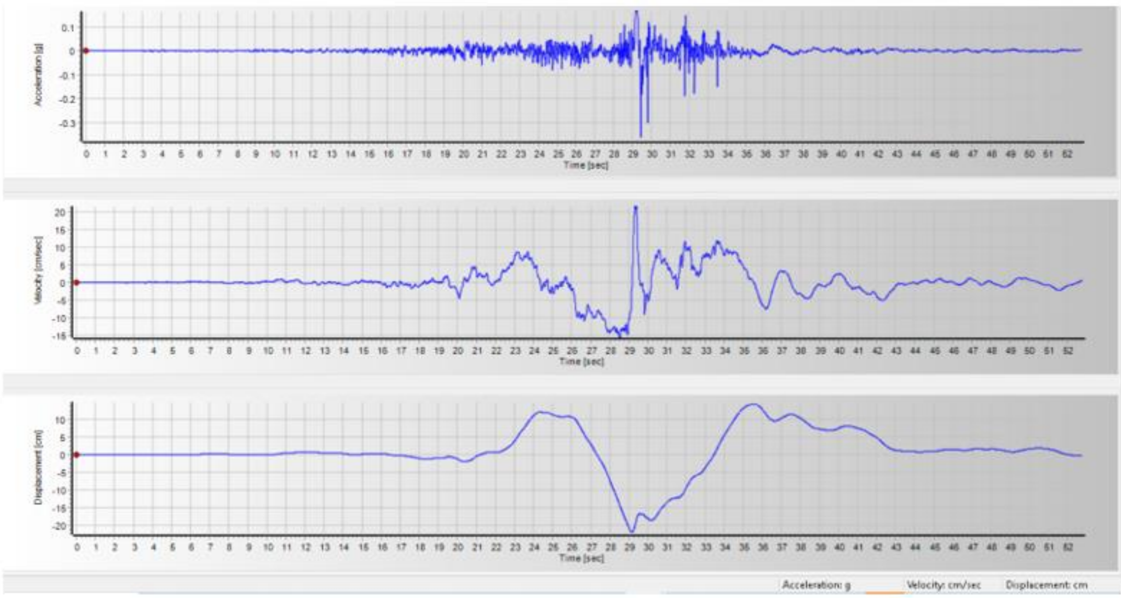

Fig 4: Time history plot of the ChiChi Earthquake

The Imperial Valley (USA) earthquake of October 15, 1979.
Source: PEER Strong Motion Database
Recording station: USGS STATION 5115
Frequency range: 0.1-40.0 Hz

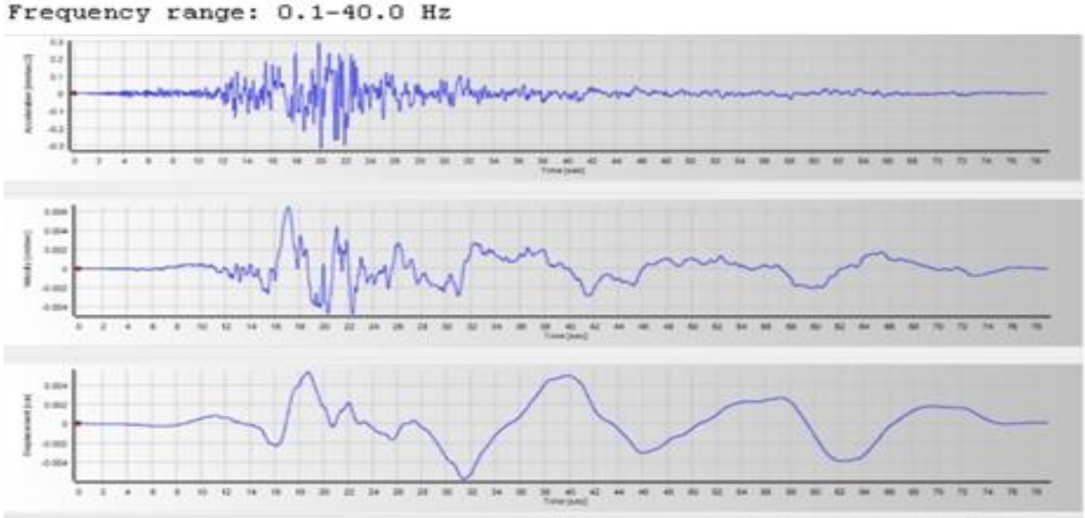

Fig 5: Time history plot of the El Centro Earthquake

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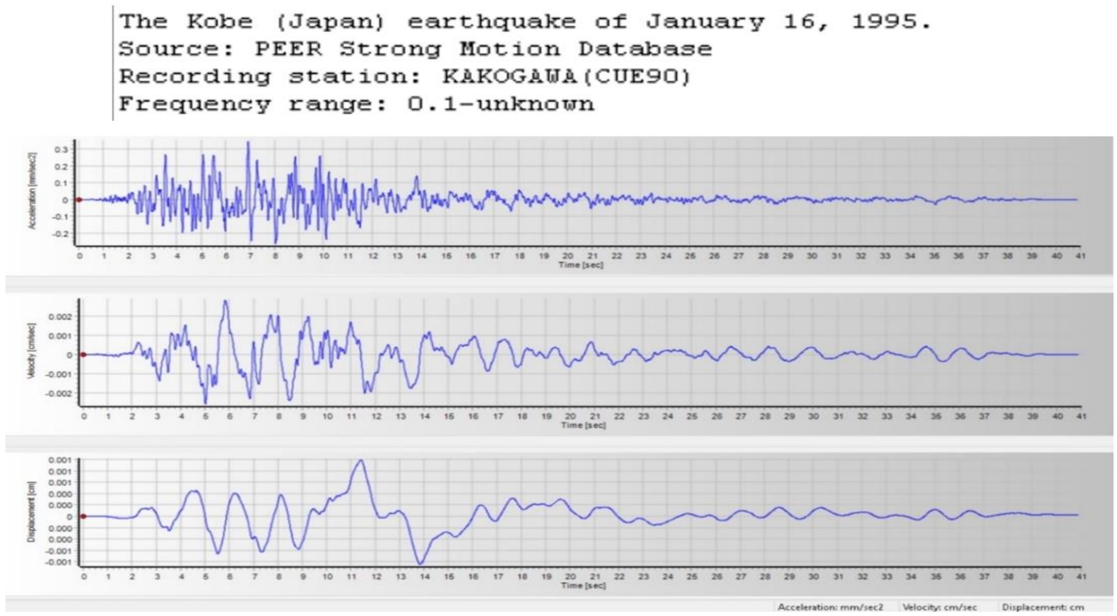

Fig 6: Time history plot of the Kobe Earthquake

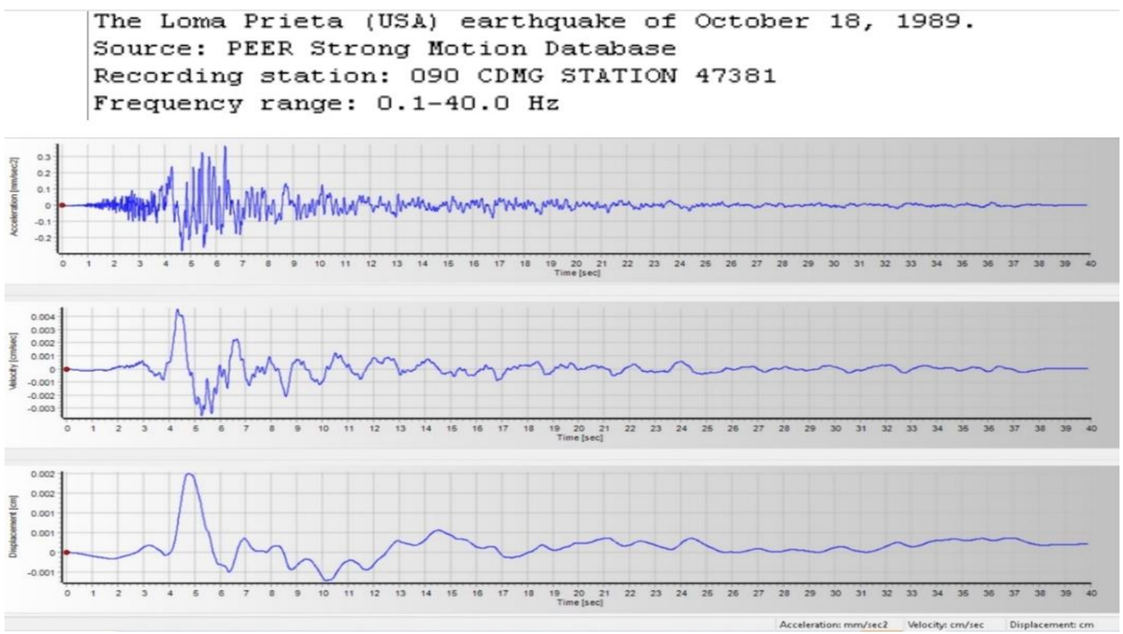

Fig 7: Time history plot of the Loma Prieta Earthquake

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The Northridge (USA) earthquake of January 17, 1994.
Source: PEER Strong Motion Database
Recording station: 090 CDMG STATION 24278
Frequency range: 0.12-23.0 Hz

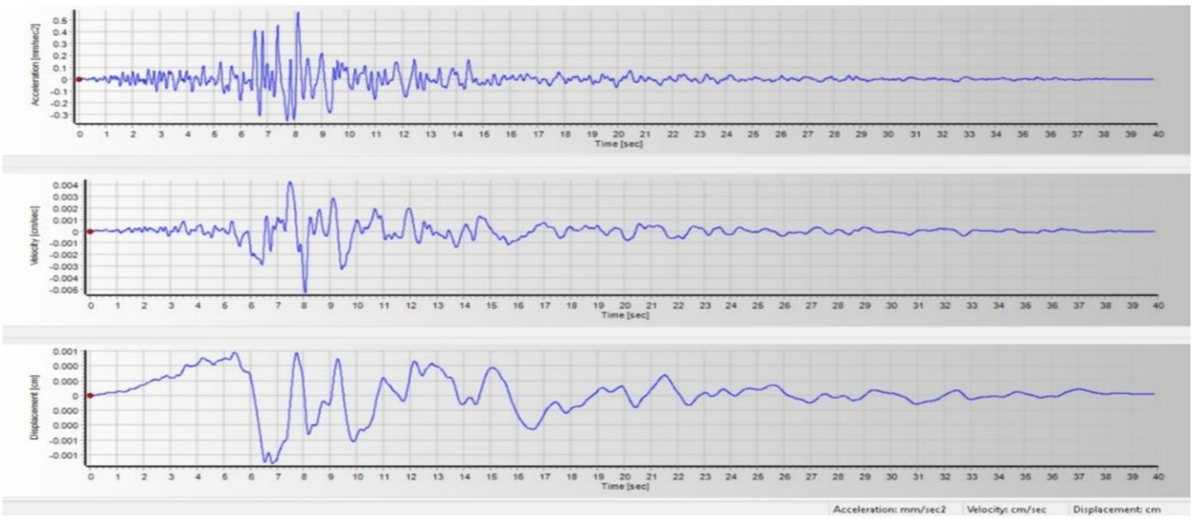

Fig 8: Time history plot of the Northridge Earthquake

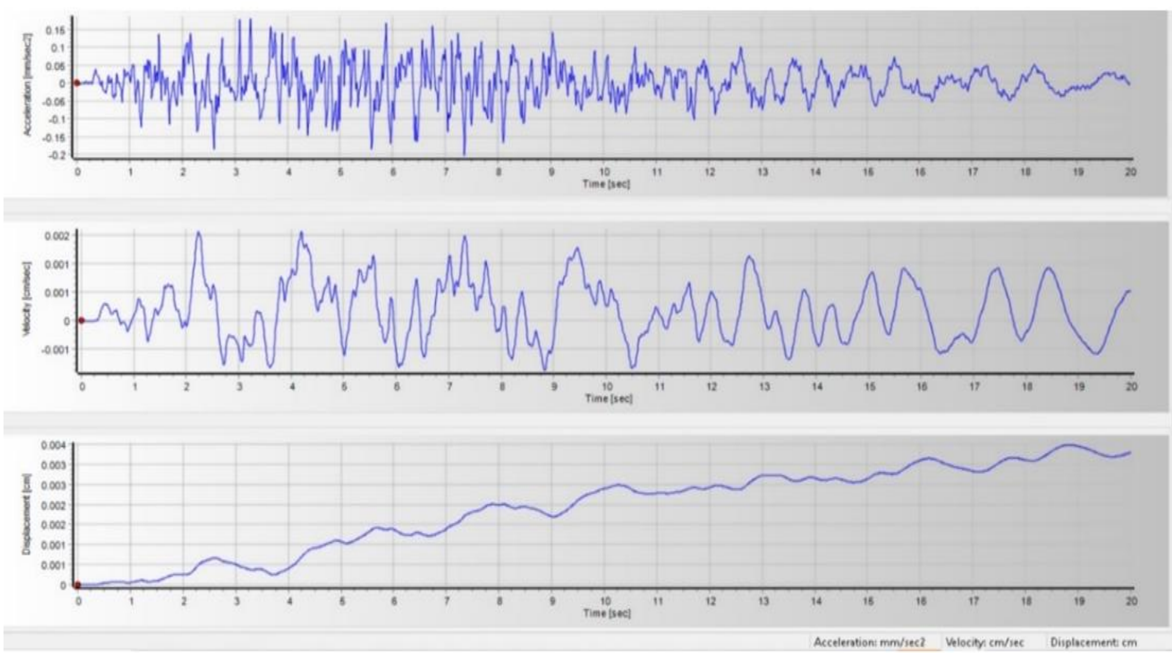

Fig 9: Time history plot of 1893-2016 matching the response spectrum of 1893-2016

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Table 2: 3D view, Plan, and typical section

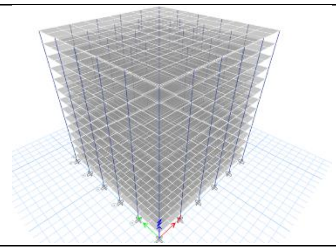
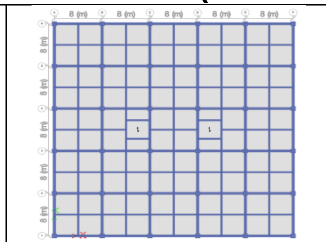
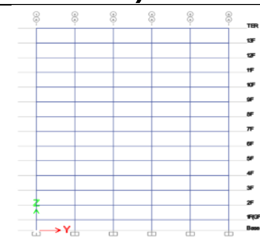
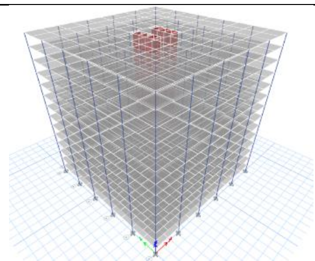
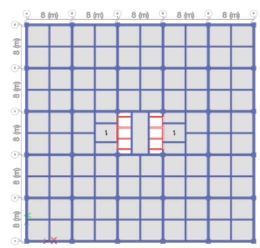
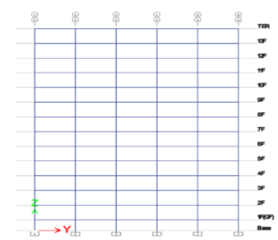
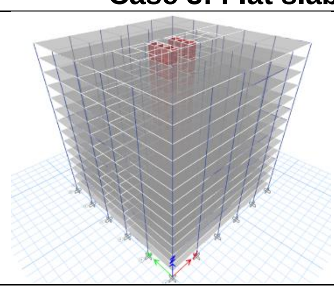
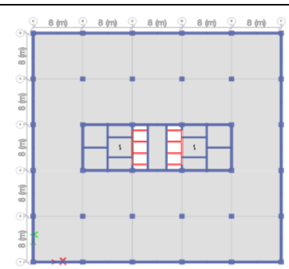
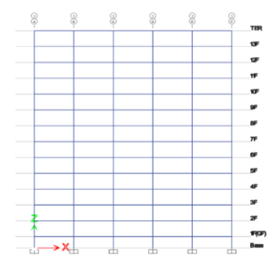
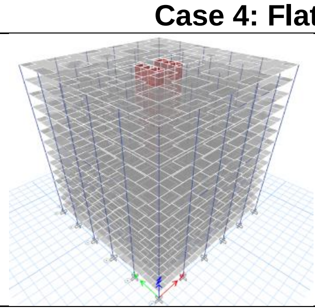
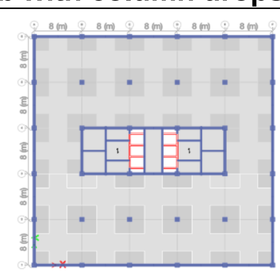
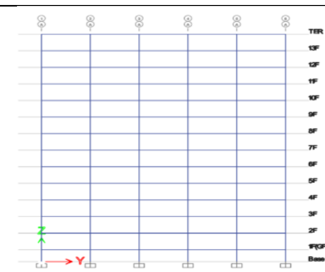
Case 1 : Column-Beam-Slab (without Lift Core walls)		
		
3D-view	**Plan-Typical Floor**	**Typical Section**
Case 2 : Column-Beam-Slab (with Lift Core walls)		
		
3D-view	**Plan-Typical Floor**	**Typical Section**
Case 3: Flat slab with lift core walls & without column drops		
		
3D-view	**Plan-Typical Floor**	**Typical Section**
Case 4: Flat slab with column drops & lift core walls		
		
3D-view	**Plan-Typical Floor**	**Typical Section**

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Table 2: Building details and material properties

Framing	Case 1	Case 2	Case 3	Case 4
Number of floors	14-floors			
Number of Slabs	13			
Height of each floor	3.5 m			
Bay width	8.0 m			
Number of Bays	5			
Total height of building from foundation level	48 m			
Depth of Foundation	2.5 below GL			
Thickness of the slab, mm	150	150	300	Slab-200, drop -600
Main Beam Size, mm	450 x 600	450x600	450 x600	450X600
Secondary Beam Size, mm	300x600	300x600	300x600 (only staircase portion)	
Column Size	800x800			
Material Properties				
Grade of concrete	M40			
Grade of reinforcing steel	Fe 500			

3. RESULTS AND DISCUSSION

3.1 Base Shear:

Figs. 10 and 11 represent the variations in base shear observed for all four frames in both the x and y directions.

During an earthquake, the inertia force causes the building to vibrate. Seismic design codes use a response spectrum averaging several past earthquake ground motions and provide the inertia force in the form of an equivalent lateral force.

This force, known as the seismic design base shear (V_B), is the primary quantity involved in the force-based earthquake-resistant design of buildings. This force depends on the sum of the seismic mass at different floor levels and the seismic hazard at the site (Murty et al.).

When the four structural framing cases are considered, for Case 1, the maximum base shear is observed for the El Centro earthquake in both the x and y directions. In the case of framing types 2 and 3, the maximum base shear is observed for the Loma Prieta earthquake in both the x and y directions.

The maximum base shear is observed for the Kobe earthquake in the x direction and for the El Centro earthquake in the y direction for the case 4 framing type. The maximum base is 25334 kN in the x direction, and 33384 kN in the y direction is observed for framing case 4.

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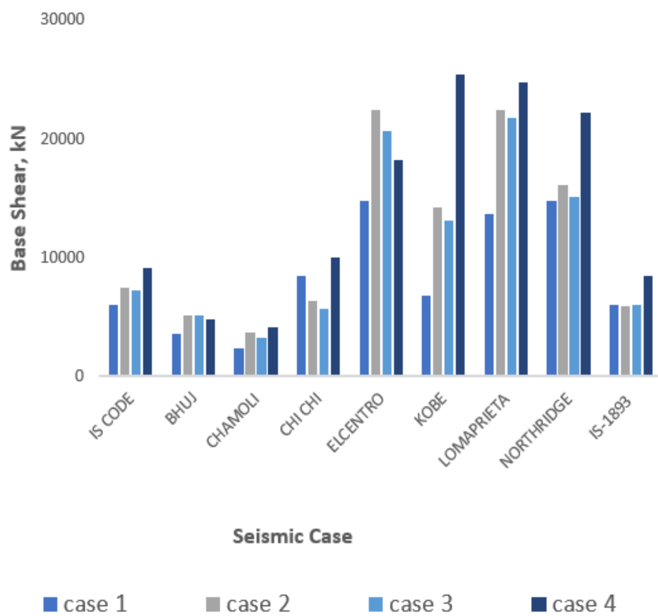
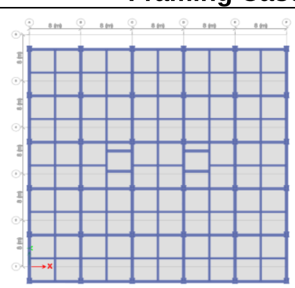
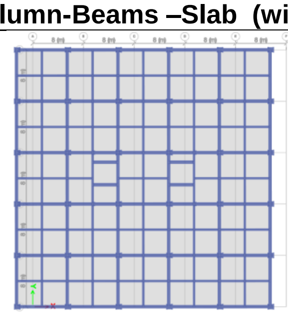
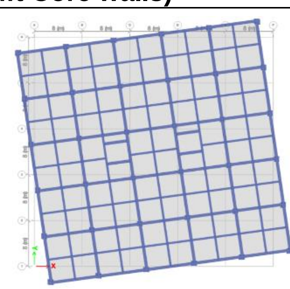
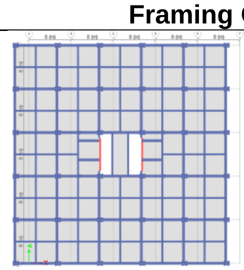
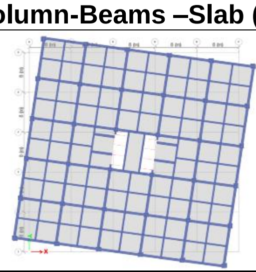
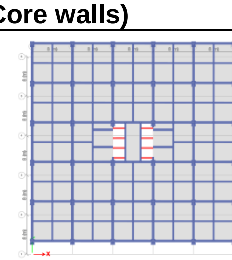


Fig 10: Variation in the base shear in the x-direction for the 14-floor building

Table 4: Comparison of the modal time period and mode shapes

Framing Case-1: Column-Beams –Slab (without Lift Core walls)		
		
Mode -1: 2.848s Translation in y	Mode 2: 2.834s Translation in x	Mode 3: 2.626s Torsional
Framing Case-2: Column-Beams –Slab (with Lift Core walls)		
		
Mode-1: 2.203s Translation in x	Mode-2: 2.20s Torsional	Mode-3: 1.77s Translation in y
Framing Case-3: Flat slab with lift core walls (without column drops)		

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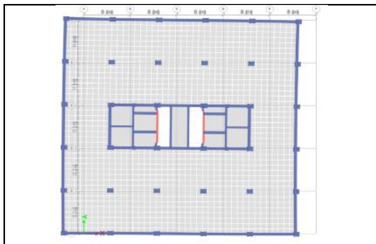
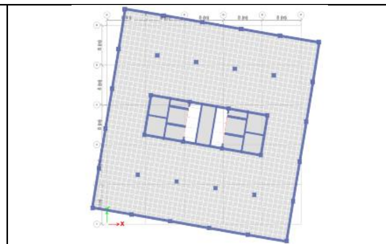
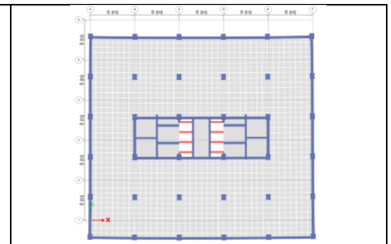
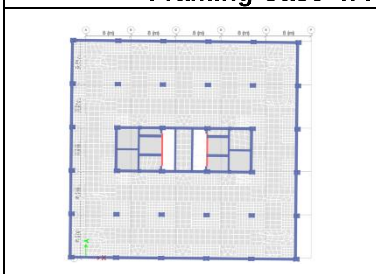
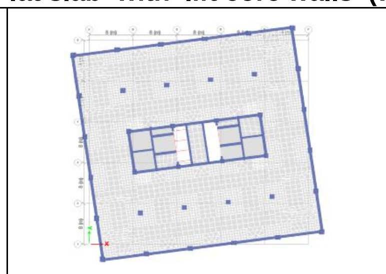
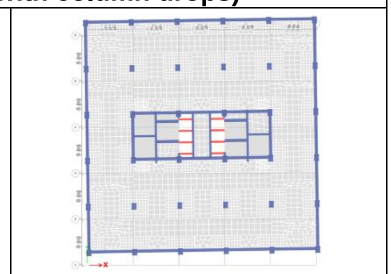
		
Mode-1: 2.256s Translation in x	**Mode- 2: 2.091 s□ Translation in y**	**Mode-3: 1.853s Torsional**
Framing Case-4: Flat slab with lift core walls (with column drops)		
		
Mode-1: 1.833s Translation in x	**Mode- 2: 1.744s Torsional**	**Mode-3: 1.568s Translation in y**

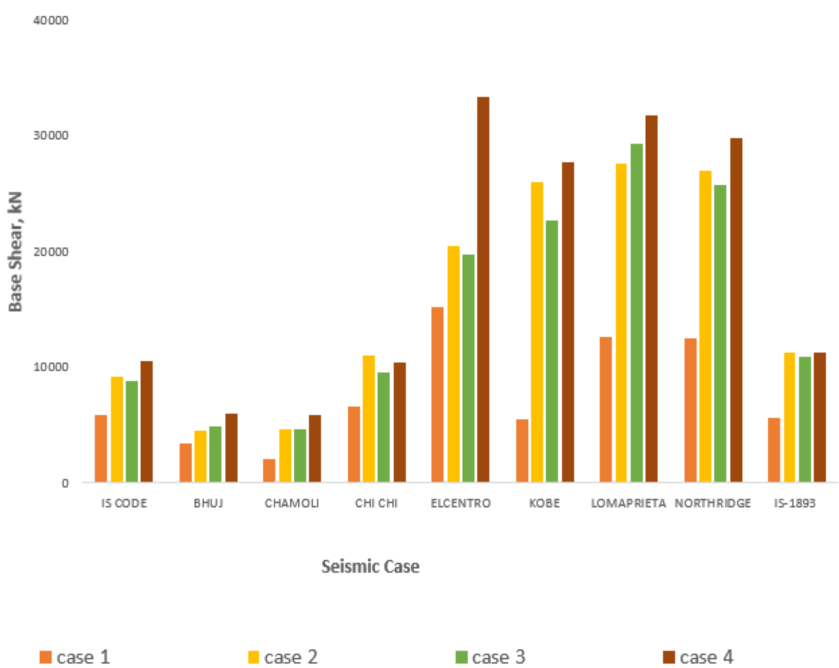


Fig 11: Variation in the base shear in the y-direction for 14-floor buildings

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3.2 Modal time period:

The time period of the first mode of vibration is referred to as the fundamental period or natural time period. This is influenced by the mass and stiffness of the building as well as the presence of infill and is independent of the external ground motion. The design seismic coefficient influences the effect of the natural period on the design horizontal seismic force coefficient (buildings with smaller translational natural periods attract higher design seismic force coefficients) (Murty et al.). Table 4 shows the modal time periods obtained for the four types of framing systems for the 14-floor buildings. Compared to those in Case 1, the time periods in Cases 2, 3, and 4 are reduced by 77%, 79%, and 64%, respectively. This is because of the introduction of concrete walls for the lift core, which increases the stiffness of the structure. In case 1, translation in the x and y directions occurs in modes 1 and 2, and rotation occurs in mode 3. However, for cases 2, 3, and 4, torsional modes are observed in mode 2, which is not desirable.

3.3 Peak Roof Displacement:

The lateral deformation caused in a structure by the application of a lateral force is referred to as lateral displacement. For the comparative study, the absolute values of the maximum roof story displacements in the lateral direction are chosen. The variations in the peak roof displacements are compared against those of four structural framing cases for 14-floor buildings in both the x-direction and y-direction and are shown in Figs. 12 and 13.

The maximum roof displacement is found for the El Centro earthquake for framing cases 1, 2, and 3 and for the Kobe earthquake for framing case 4 in the x direction. However, the maximum roof displacement is observed for framing cases 1 and 4 for the El Centro earthquake and framing cases 2 and 3 for the Kobe earthquake in the y direction. The maximum roof displacement is 154 mm in the x direction for framing type 3 and 148 mm in the y direction for framing type 1 for the El Centro earthquake.

3.4 Peak Floor Acceleration:

The peak floor acceleration (PFA) is a measure of the intensity of ground motion during an earthquake that affects a building's structural response and is defined as the maximum acceleration that a floor experiences during an earthquake. The structure undergoes vibration at an amplified rate compared to ground acceleration. The inertial forces develop at each floor owing to floor accelerations, which can lead to damage to the floor and other floor components. Notably, increasing the stiffness of buildings has a counterproductive effect on reducing drift, as higher stiffness leads to greater floor acceleration (Murty et al.). Figs. 14 and 15 show the variations in peak floor acceleration for the four framing cases against the input acceleration in the x and y directions. Compared with the input acceleration, for Framing case 1, the maximum peak roof acceleration is approximately 62% for the El Centro earthquake and 71% and 72% for the Loma Prieta earthquake for Framing cases 2 and 3, respectively; 86% for the Framing case 4 in the x direction. Similarly, for framing case 1, the peak roof acceleration is approximately 54% under the Bhuj earthquake, 89% under the El Centro earthquake for framing type 2, 72% under the

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Loma Prieta earthquake for framing case 3, and 93% under the El Centro earthquake for framing type 4 in the y direction.

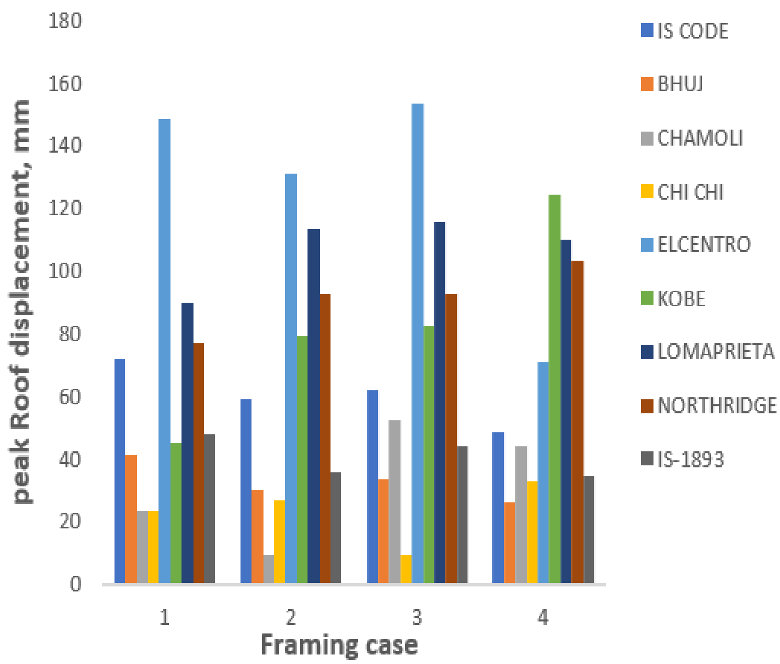

Fig 12: Variation in peak roof displacement for 14-floor buildings in the x direction for four framing types

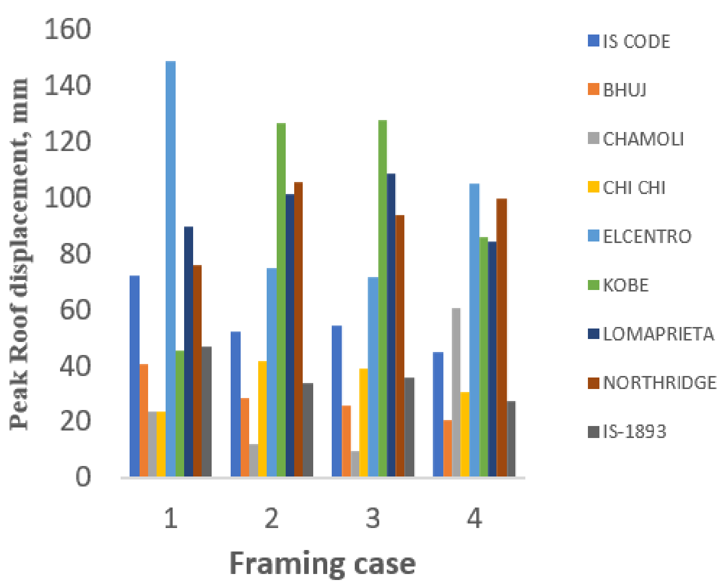

Fig 13: Variation in peak roof displacement for 14-floor buildings in the y direction for four framing types

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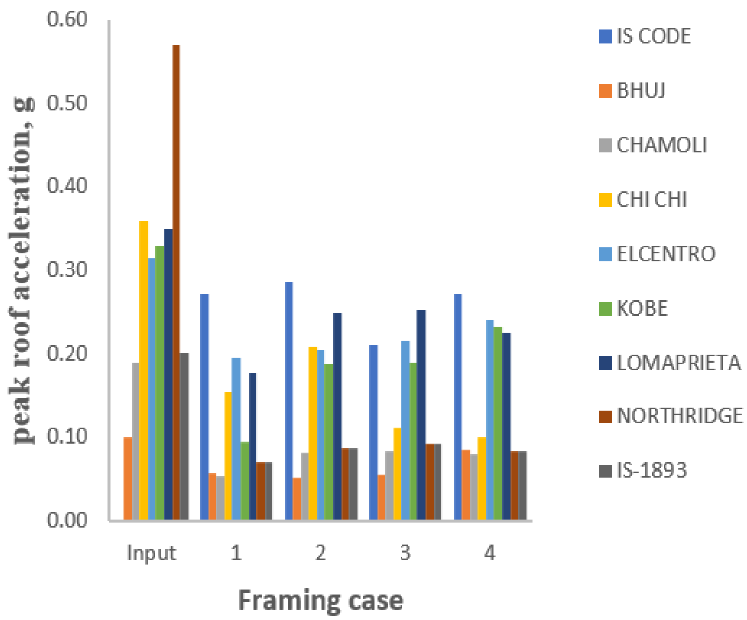

Fig 14: Variation in the peak roof acceleration of the 14-floor building for the four framing cases against the input acceleration in the x direction

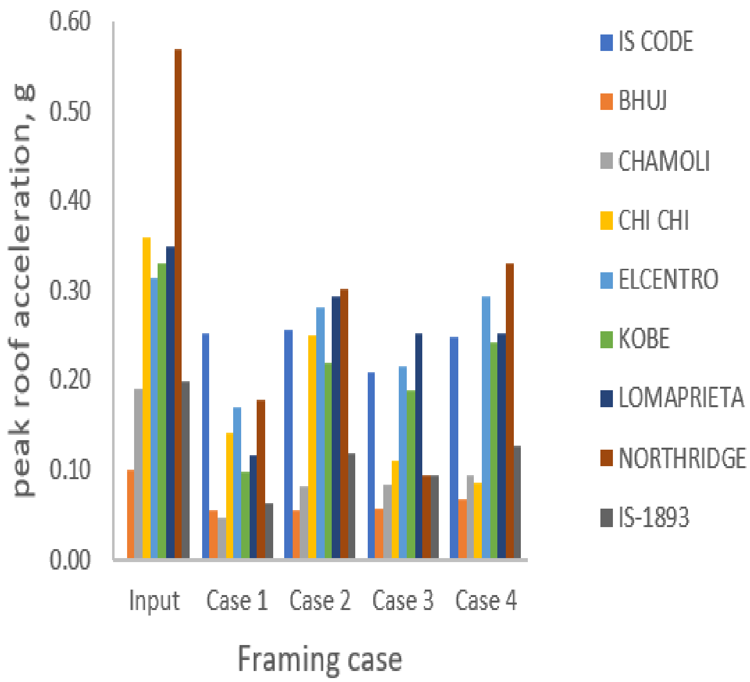

Fig 15: Variation in the peak roof acceleration of the 14-floor building for the four framing cases against the input acceleration in the y direction

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4. CONCLUSION

Four types of structural framing are considered for a 14-floor building in Seismic Zone V with a hard soil type and an importance factor of 1.5 as per IS 1893-2016 (part 1). For the selected set of earthquakes, time history analysis is carried out by adopting fast nonlinear analysis using ETABS software. From the analysis results, the following conclusions are drawn:

1) The stiffness of the structure in both the x- and y-directions increases for framing types 2, 3, and 4 with the introduction of the R C core.
2) The current positioning of the RC lift core wall contributes to the torsional effect in the second mode of oscillation, necessitating structural framing modifications to ensure effective translation in the first and second modes.
3) The resulting base shear is lower for codal consideration than for time history analysis considering earthquakes such as El Centro, Kobe, Loma Prieta, and Northridge for all four framing types.
4) Compared with the traditional column-beam-slab system (cases 1 and 2), the use of a flat slab system for framing types 3 and 4 results in greater base shear
5) The peak roof displacement and acceleration are highest in the structural framing that employs a flat slab system.

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