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Assessment of Seismic Response and Structural Performance of a Retrofitted G+9 Reinforced Concrete Building Utilizing RCC Jacketing and Steel Wrapping Techniques

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Abstract: A seismic design is based upon combination of strength and ductility. Frequent seismic disturbances, the structure are expected to remain in the elastic range. By considering the actual dynamic nature of environmental disturbances, more improvements are needed in the design procedures. And some advance techniques are used to strengthen the existing structures i.e. different retrofitting methods. All these methods have their own advantages. The main objective of the present study is to analyze the behavior of Retrofitted building i.e. provision of steel jacketing in increasing the performance of building. The present study aims at checking the adequacy of multi-storey frame structures using retrofitting methods for the seismic excitations. The Retrofittedbuilding i.e. provision of steel jacketing is analyzed and compared with bare frame structure by using time history and pushover analysis method by using Commercial software SAP2000 v16 is used for analysis. The responses of the structure are compared by considering different parameters i.e. displacement, base shear, plastic hinges, time period of mode shapes from FEMA – 356. The result shows that plastic hinge formation during earthquake at beam-column junction can improved performance with use retrofitting method i.e. steel jacketing.
Keyword: FEMA-356, Retrofitted, Adequacy, Steel jacketing

I. INTRODUCTION

A. General

A seismic design is based upon combination of strength and ductility. For small, frequent seismic disturbances, the structure is expected to remain in the elastic range with all stress well below the yield level. However it is not reasonable to expect that the traditional structure will respond elastically when subjected to major earthquake. Instead the design engineer relies upon the inherent ductility of the building structure to prevent catastrophic failure while accepting certain level of structural and non-structural damage. This philosophy has led to the development of a seismic design codes featuring lateral force methods and more recently, inelastic methods. Ultimately, with these approaches, the structure is designed to resist an equivalent static load and results have been reasonably successful. Even an approximate accounting for lateral effects will almost certainly improve building survivability. However, by considering the actual dynamic nature of environmental disturbances, more improvements were made in the design procedures. As a result from the dynamical point of view, new and innovative concepts of structural protection system advanced and are at various stages of development.

B. Techniques of Retrofitting

There are various ways of retrofitting the building structure. RCC jacketing, steel jacketing, fiber reinforced polymer jacket, composite jacketing, shortcreting, passive energy dissipation devices, active energy dissipation device and base isolation system. All these techniques have their own advantages and disadvantages. One should be very precise and selective while adopting the method of retrofit. All these methods are briefly described further. [12]

C. Fiber Reinforced Polymer Technique

The most common structural retrofitting methods are concrete and steel jacketing. In recent years fiber- reinforced polymer (FRP) materials are used to replace steel for jacketing due to its advantages in speed and ease of installation, reduced maintenance, high strength, light weight, superior durability, and lower increase in structural stiffness, which leads to a smaller increase in seismic inertial force. The general conclusion is that FRP jacketing is highly effective for circular or elliptical shaped columns. However, flexural retrofitting of square/rectangular RC columns by jacketing is much less effective due to the poor confinement of concrete in the middle of the column sides, especially for large columns. [18]

D. Composite Jacketing System

Advanced composite materials have been recently recognized and applied to bridge retrofit. The general expectations fromcomposite retrofit systems include light weight, high stiffness or strength to weight ratios, etc. Several composite jacketing systems have been developed and validated in laboratory or field conditions. Asystem consisting of carbon fiber sheets wrapped longitudinally and transversely in the potential plastic hinge region or in the region of main bar cutoff is suggested. Carbon fiber sheets were bonded to the concrete surface using epoxy resin. Another composite wrapping system using E-glass fiber, which is much more economical than carbon fiber, has been experimentally studied. The test results on40% scale bridge piers wrapped with the glass fiber composite jacketing demonstrated significant improvement of seismic performance with increased strength and ductility. An experimental validation of carbon fiber retrofit system that uses an automated machine to wrap carbon bundles to form a continuous jacket has been successfully reported. [6]

E. Steel Jacketing Technique

Shear failure of short concrete columns has been one of the major problems that may cause the collapse of structures under earthquake attacks. In a structure where the columns have different lengths, shorter columns tend to attract a greater portion of the seismic input during an earthquake and require the generation of large seismic shear forces to develop the moment capacity of column. The design of flexural strength based on elasticmethods, along with less conservative shear strength provisions in older design codes, typically resulted in expected shear strength of columns in many existing structures being less than the flexural strength. These have been evidenced by the brittle failure of columns that caused numerous structures to collapse in previous earthquakes. The use of a steel jacket or tube to enhance the strength of columns and to improve deformability was studied previously. Sakino and Ishibashi(1985) investigated the seismic performance of concrete-filled steel tubular (CFT) columns and found that plastic buckling of the steel tube in the hinge regions tended to occur when the columns were subjected to large cyclic lateral displacements. Tomii, Sakino, and Xiao (1987) and Xiao(2001)investigated steel-tubed short columns in building structures as a measure to prevent shear failure and to improve ductility. To avoid the buckling of the steel tube observed by Sakino and Ishibashi (1985) for conventional CFT columns, the tube was deliberately terminated to leave gaps from the column ends, thus ensuring the tube to function mainly as hoop reinforcement rather than also contributing in flexural strength. Excellent seismic behavior was obtained for circular columns. Due to inadequate confinement of concrete in thepotential plastic hinge region, it was found that deterioration of response was inevitable for rectangular columns, unless a thick steel tube was used, particularly for columns with axial load exceeding 30% of axial load capacity. The issues become relatively less severe for steel-tube high-strength concrete columns subjected to lower axial load. [5]

Priestley et al. (1994) investigated elliptical jackets to enhance the shear strength of rectangular columns. This method has now been widely used in retrofitting rectangular columns in bridges in California and elsewhere. However, the profile of the elliptical jacket increases the section of the columns substantially; thus, it may notbe desirable from the architectural and functional points of view, particularly for retrofitting columns in buildings where most columns are rectangular or square [4]. Aboutaha et al. (1996) tested a system that combined a through bolt with a relatively thin rectangular jacket, and showed enhanced confinement efficiency

In this study; the writers developed another improved jacketing method to retrofit square columns using welded rectilinear steel jackets and stiffeners. [5]

Fig. 1 summarizes and schematically compares the four different transverse reinforcements. In a well-confined reinforced concrete column design based on modern seismic design provisions, as shown in Fig. 1-a, hoops or spirals and cross ties are provided to contain the core concrete, particularly for the potential plastic hinge regions near the ends of a column. Spacing of the hoops and ties along the column and the intervals of the cross ties within the section are limited in order to achieve better efficiency of confinement.

A similar confinement mechanism is achieved for retrofitted columns using the combined jacketing and through bolting method by Aboutaha et al. (1996). In a tube column with a square or rectangular section, as shown in Fig. 1-b, the weak out-of-plane stiffness results in poor confinement of portions of the concrete section. As exhibited in Fig. 1-c, the use of an elliptical-shaped steel jacket for retrofit can provide a continuous transverse confinement to the existing concrete section. The partially stiffened rectilinear steel jacket developed in this study intends to rely on a beam action of the confinement elements (stiffeners) to develop efficient transverse confinement to the concrete section, as illustrated in Fig. 1-d.

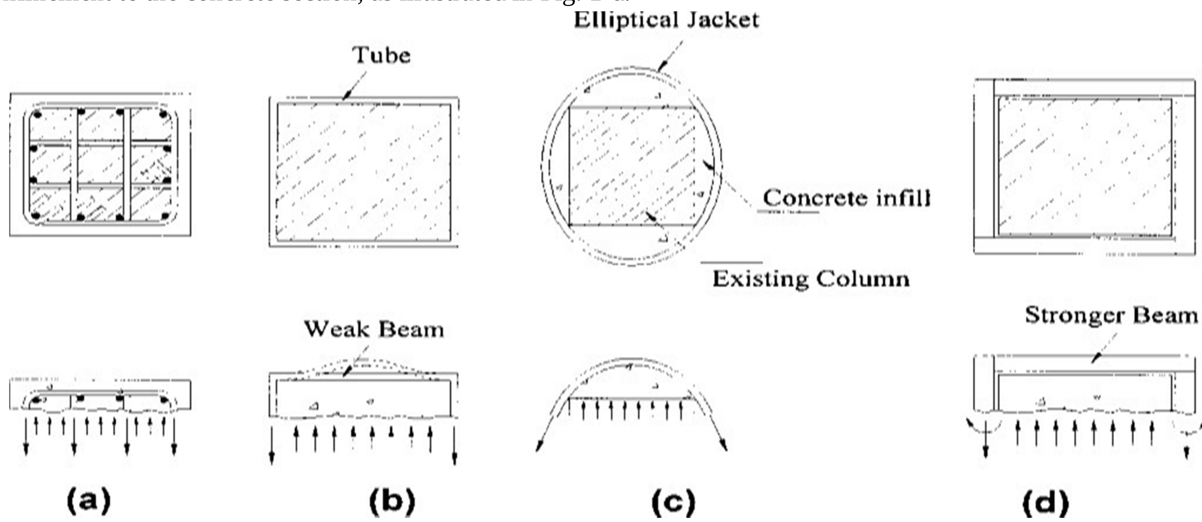

Figure 1.Comparison of different transverse confinements for concrete columns: (a) hoops and ties per current seismic design provisions; (b) steel tube; (c) elliptical steel jacketing; and (d) partially stiffened rectilinear jacketing

II. MODELLING AND ANALYSIS OF BUILDING

A. General

The present chapter contains information about geometry of building structure, properties of material used to erect the building model and some assumption that are necessary for modelling and analysis. At the beginning a bare frame building structure is modeled and a retrofitted building is modeled using steel jacketing technique and pushover analysis and linear time history analysis is carried out.

B. Building Geometry

In the present work a 3-D structural model is used which comprises of G+9 storey reinforced concrete momentresisting frame. The foundation of the structure is assumed to be fixed. The data assumed for the analysis of building is shown in Table 2.1.

Table 2.1: General Description of Building

Sr. No	Entity	Description
1	No of Bays in X Direction	3
2	No of Bays in Y Direction	3
3	Width of Bay in X Direction	3 m
4	Width of Bay in Y Direction	3 m
5	Storey Height	3 m
6	Live Load	3 kN/m^2
7	Floor Finish	1 kN/m^2
8	Concrete Grade	M20
9	Rebar	Fe415
10	Beam Size	250 mm x 250 mm
11	Column Size	300 mm x 300 mm

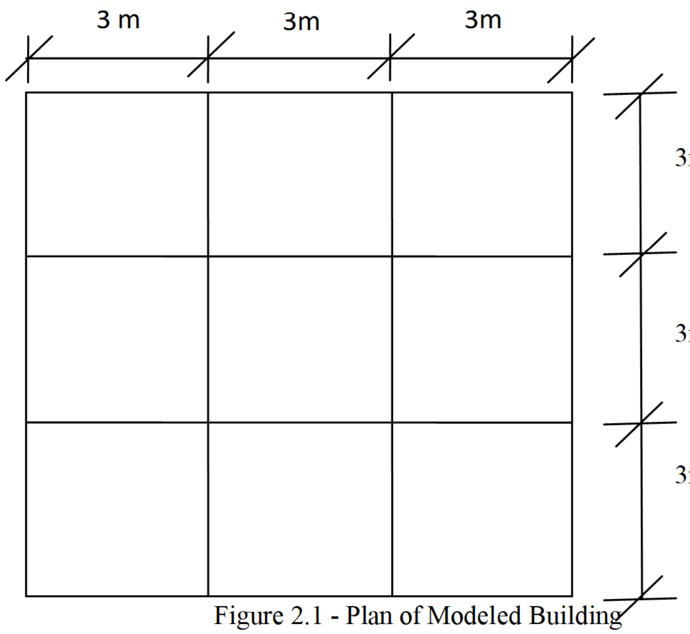

Figure 2.1 - Plan of Modeled Building

C. Material Properties

M-20 grade of concrete and Fe-415 grade of reinforcing steel are used for all the frame models used in this study. Elastic material properties of these materials are taken as per Indian Standard IS 456 (2000). The short- term modulus of elasticity (Ec) of concrete is taken as:

$$E_c = 5000\sqrt{f_{ck}} \qquad (2.1)$$

Where f_{ck} = characteristic compressive strength of concrete cube in MPa at 28-day (20 MPa in this case). For the steel rebar, yield stress (fy) and modulus of elasticity (Es) is taken as per IS 456 (2000).

D. Steel Jacket Modelling

The grade of steel used for jacketing of RC column is Fe250. The steel jacket used for retrofitting purpose isnot provided over the full length of column but it only provided at possible hinge location. The jacket provided around the column should only undergo shearing action and should not participate in bending of column addingto additional strength of column. Xiao and Wu have suggested a retrofit design procedure was developed in order to provide additional confinement and shear strength to convert an existing deficient column to the condition satisfying current seismic design provisions. In the seismic design provisions of the current ACI 318 code (1999) to ensure the rotational deformability of the potential plastic hinges near column ends, the transverse reinforcement is specified as

$$A_{sh} \geq 0.3\frac{sh_c f_c^{'}}{f_{yh}}\left(\frac{A_g}{A_{ch}} - 1\right) \qquad (2.2)$$

$$A_{sh} \geq 0.09 sh_c \frac{f_c^{'}}{f_{yh}} \qquad (2.3)$$

whereA_{sh} = total transverse steel cross-sectional area within spacing *s*; h_c= cross-sectional dimension of column core measured center-to-center of the outermost peripheral hoops; $f_c^{'}$= specified compressive strength of concrete; f_{yh} = specified yield strength of transverse reinforcement; A_g = gross area of section; and A_{ch} =cross- sectional area of a column measured out-to-out of transverse reinforcement. From Eqs. 3.2 and 3.3 an equivalent transverse pressure f_{eq}can be defined as

$$f_{eq} = \frac{A_{sh} f_{yh}}{sh_c} \geq 0.3 f_c^{'}\left(\frac{A_g}{A_{ch}} - 1\right) \qquad (2.4)$$

or

For the retrofit design, it is suggested that the equivalent confinement pressure shall be provided to a column under consideration. It is assumed that the confinement element shall sustain a uniformly distributed equivalent transverse pressure. The design for the confinement element is based on a limit state where a yield mechanismis formed with plastic hinges at middle and corner sections along each side. Thus, the following equilibrium conditions can be established to calculate the moment and axial force demands, m and p, per unit width for the confinement element

$$m = \frac{1}{16} h^2 f_{eq} \qquad (2.6)$$

$$p = \frac{1}{2} h f_{eq} \qquad (2.7)$$

On the other hand, the following equations for beam column design specified in AISC (1999) can be used to design the confinement element.

$$\frac{p}{\emptyset p_n} + \frac{8}{9}\frac{m}{\emptyset_b m_n} \leq 1, \quad for \frac{p}{\emptyset p_n} \geq 0.2 \qquad (2.8)$$

$$\frac{p}{2\emptyset p_n} + \frac{m}{\emptyset_b m_n} \leq 1, \quad \text{for } \frac{p}{\emptyset p_n} < 0.2 \qquad (2.9)$$

Where m_n and p_n= nominal flexural and tensile strengths per unit width, whereas Ø and $Ø_b$ = corresponding resistance factors, taken as 1.0 in this study.

In a retrofit design situation where an additional jacket is provided to confine the full column section, Eqs. 2.2 and 2.3 are automatically satisfied, since A_{ch} can be considered the same of A_g. Thus, Eq. 2.3 or 2.5 governs thedesign.

For the case where steel plates are welded to confine concrete, the strengths per unit width can be easily found as,

$$m_n = \frac{t^2 f_{yj}}{2} \qquad (2.10)$$

$$p_n = t f_{yj} \qquad (2.11)$$

Where *t* is the thickness of the jacket plate and f_{yj} is its yield strength. Substituting these strength expressions into the above equations and noting that Eq. 3.9 governs the design, the following equation can be derived to determine the thickness of the jacket plate:

$$t = \frac{h}{\sqrt{\frac{1}{4} + \frac{4f_{yj}}{f_{eq}}} - \frac{1}{2}} \qquad (2.12)$$

E. Pushover Analysis – An Overview

The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970"s but the potential of thepushover analysis has been recognized for last two decades years. This procedure is mainly used to estimate thestrength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. The effectiveness of pushover analysis and its computational simplicity brought this procedure in to several seismic guidelines (ATC 40 and FEMA 356) and design codes (Euro code 8 and PCM 3274) in last few years.

F. Lateral Load Profile

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general case, the centre of mass location at the roof of the building is considered as control node. In pushover analysis selecting lateral load pattern, a set of guidelines as per FEMA 356 is explained in Section 2.5.2. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behavior. Different types of lateral load used in past decades are as follows

"Uniform" Lateral Load Pattern

The lateral fore at any story is proportional to the mass at that story.

$$F_i = \frac{m_i}{\sum m_i} \quad (2.13)$$

Where ,

F_i= lateral force at i^{th} storym_i = mass of i-th story

"First Elastic Mode" Lateral Load Pattern

The lateral force at any story is proportional to the product of the amplitude of the elastic first mode and mass atthat story,

Where,

$$F_i = \frac{m_i \emptyset_i}{\sum m_i \emptyset_i} \quad (2.14)$$

$\emptyset_i$= amplitude of the elastic first mode at i^{th}story.

"Code" Lateral Load Pattern

The lateral load pattern is defined in Turkish Earthquake Code (1998) and the lateral force at any storey iscalculated from the following formula:

$$F_i = (V_b - \Delta F_N)\frac{m_i h_i}{\sum_{j=1}^{N}(m_j h_j)} \quad (2.15)$$

Where

V_b= base shear

h = height of i-th story above the baseN = total number of stories

ΔF_N = additional earthquake load added to the N^{th} story whenh_N>25m

(For h_N>25m, ΔF_N = 0 otherwise; $\Delta F_N = 0.07T_1V_b \leq 0.2V_b$,where T_1 is the fundamental period of the structure)

$$Q_i = V_b \frac{W_i h_i}{\sum_{j=1}^{n}(W_j h_j)} \quad (2.16)$$

Where

Q_i = Design lateral force at floor i,W_i = Seismic weight of floor i,

h_i = Height of floor i measured from base, and

n = Number of stories in the building is the number of levels at which the masses are located.

"Multi-Modal (or SRSS)" Lateral Load Pattern

The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregular structures and the lateral force at any story is calculated Square Root of Sum of Squares (SRSS) combinationsof the load distributions obtained from the modal analysis of the structures as follows:

1. Calculate the lateral force at i^{th} storey for n^{th} mode from equations

$$F_{in} = \Gamma_n m_i \emptyset_{in} A_n \quad (2.17)$$

Where,

Γ_n= modal participation factor for the n^{th} mode$\emptyset_{in}$= Amplitude of n^{th} mode at i^{th} story

A_n= Pseudo-acceleration of the n-th mode SDOF elastic system

2. Calculate the storey shears, $V_{in} = \sum_{j=1}^{N} F_{jn}$,where N is the total number of storeys
3. Combine the modal storey shears using SRSS rule, $V_i = \sqrt{\sum_n (V_{in})}$
4. Back calculate the lateral storey forces F_i , at storey levels from the combined storey shears, $V_i$startingfrom the top storey.
5. Normalize the lateral storey forces by base shear for convenience such that

$$F_i' = F_i / \sum F_i \quad (2.18)$$

The first three elastic modes of vibration of contribution was considered to calculate the "Multi-Modal (orSRSS)" lateral load pattern in this study.

III. RESULTS AND DISCUSSION

A. Introduction

In this chapter the bare frame model and retrofitted building model are analyzed using linear time history analysis and pushover analysis. The behavior of the retrofitted building model is compared with bare framemodel through pushover curves in pushover analysis and storey displacements, storey drift, shear force andmoment in exterior frame column in linear time history analysis. Some parameter of both buildings are evaluated at performance point. The time period and frequency of building along with mode shapes are alsoanalyzed. The results obtained these analysis are compared using tables and graphs.

B. Modal Time Period and Frequency

The time period of both bare frame and retrofitted building are calculated using modal analysis. The time periodand frequency are analyzed in X, Y and torsional direction. Table 4.1 shows time period for bare frame and retrofitted building in X, Y and torsional direction for first, second, third and fourth mode of vibration

Table 3.1 - Modal Time Period of Bare Frame and Retrofitted building.

Direction	Mode No.	Time Period (sec)	
		Bare Frame	Retrofitted
X	1	1.345	1.182
	2	0.442	0.387
	3	0.255	0.220
	4	0.179	0.153
Y	1	1.345	1.181
	2	0.442	0.387
	3	0.255	0.220
	4	0.179	0.153
Torsion	1	1.212	1.084
	2	0.4	0.357
	3	0.235	0.208
	4	0.165	0.143

From Table 3.1 it can observed that modal time period for bare frame and retrofitted building is highest for firstmode and reduces with increasing mode number in X, Y and torsional mode of vibration. Moreover it is also observed that modal time period in X and Y direction for first, second, third and fourth mode is same which clearly indicates that the building is symmetric in geometry. When the modal time period of bare frame structure and retrofitted building are compared in their respective mode and direction, the modal time period is found less in case of retrofitted building than bare frame building. This is the result of the increased stiffness which has occurred due to steel jacketing of the RCC column near the plastic hinge region.

The frequencies of bare frame and retrofitted structure are compared in Table 3.2 in X, Y and torsionaldirection for first, second third and fourth mode.

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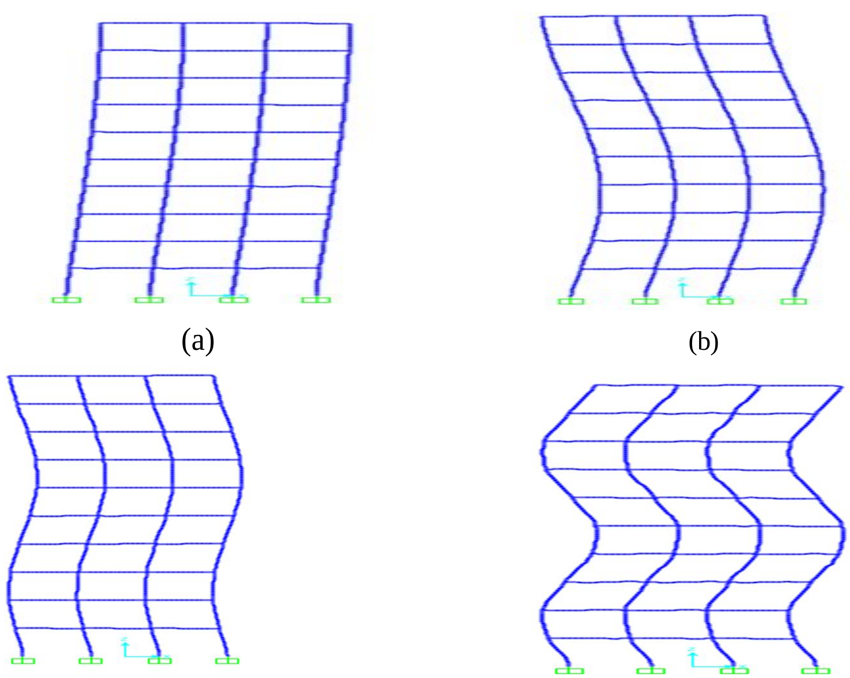
Table 3.2 - Frequency of Bare Frame and Retrofitted building.

Direction	Mode No.	Frequency (htz)	
		Bare Frame	Retrofitted
X	1	0.743	0.845
	2	2.26	2.579
	3	3.914	4.526
	4	5.57	6.506
Y	1	0.743	0.846
	2	2.26	2.580
	3	3.914	4.526
	4	5.57	6.508
Torsion	1	0.82	0.921
	2	2.49	2.979
	3	4.238	4.802
	4	6.05	6.947

The results of Table 3.2 says that frequency is maximum in case of fourth mode and reduces thereby with decreasing mode number in both bare frame and retrofitted structure. When the frequencies of bare frame and retrofitted structure are compared the values of retrofitted structure had increased with small margins in their respective mode and direction. This change was observed due to steel jacketing which increased the stiffness of column. This increased frequency and lowered time period of the retrofitted building signifies that the acceleration of the structure had increased and the displacements that will occur in retrofitted building are less in comparison to bare frame structure.

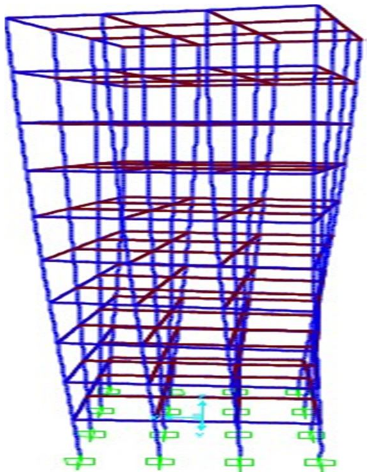
C. Mode Shapes

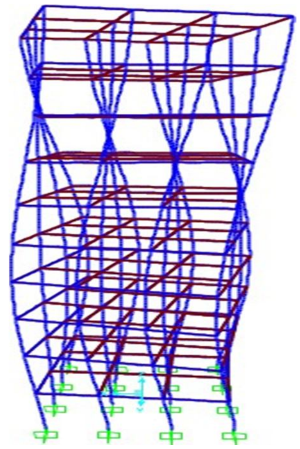
The mode shapes obtained for bare frame model are shown in Figure 4.1. Same type of mode shapes wereobtained for retrofitted building model. Since the mode shape obtained in X and Y direction are similar therefore mode shape of X and torsional mode are only shown.



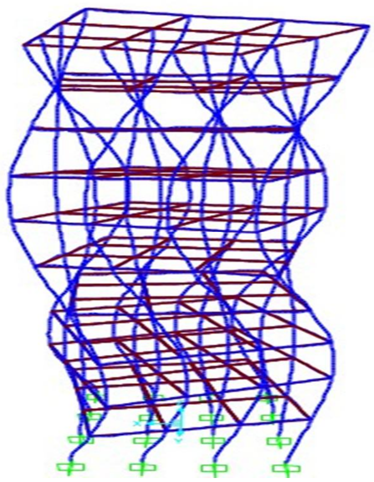
(a) (b) (c) (d)

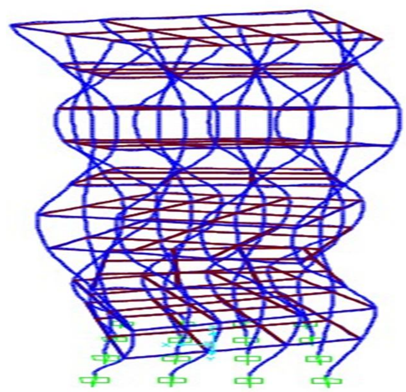
Figure 3.1 - Picture (a), (b), (c) and (d) represent first, second, third and fourth mode shape in X and Y directions.





(a) (b)





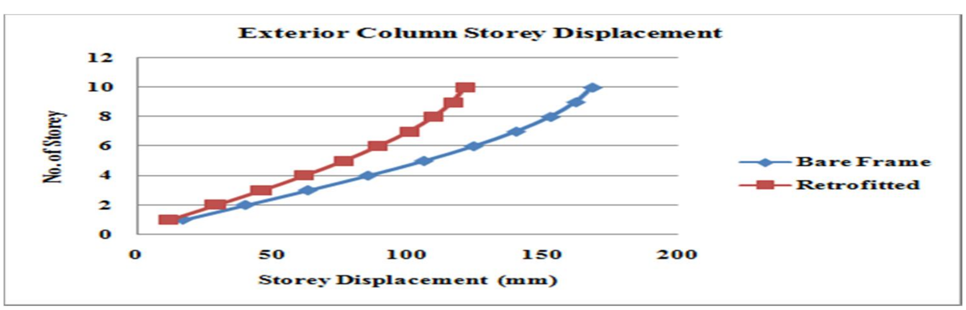
(c) (d)

Figure 3.2 - Picture (a) depicts first mode shape (b) depicts second mode shape (c) depicts third mode shape and (d) fourth mode shape in torsion

D. Linear Time History Analysis

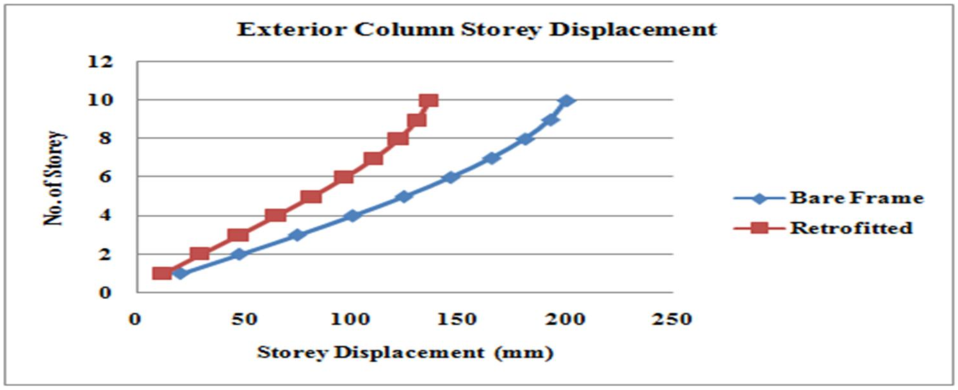
To study the response of building under real earthquake ground motions linear dynamic time history analysis is carried out. This analysis exhibits real earthquake effects and the responses obtained are very practical.

Therefore the behavior of building with steel jacketing technique is studied under three acceleration time histories of different earthquake ground motions. Table 4.3 depicts storey displacement of bare frame and retrofitted building for three different acceleration time histories.



(a)

(b)

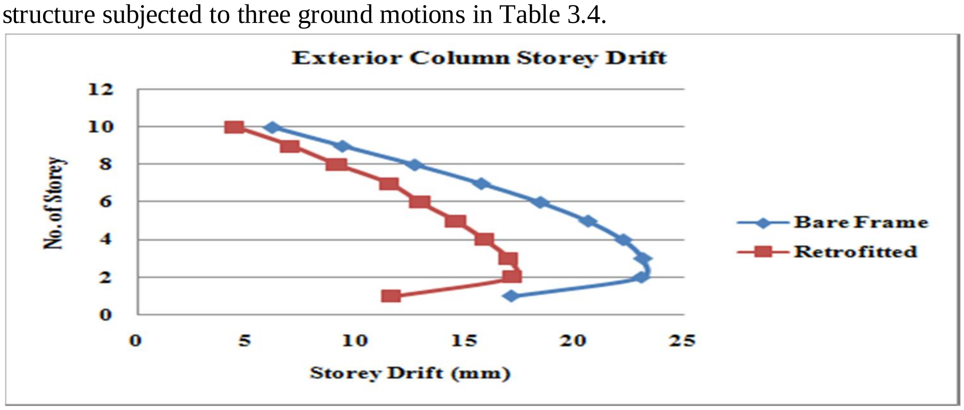


(c)

Figure 3.3 - Storey Displacement for Bare Frame and Retrofitted Building for (a) Imperial Valley (b) North Ridge and (c) Loma Prieta Earthquake

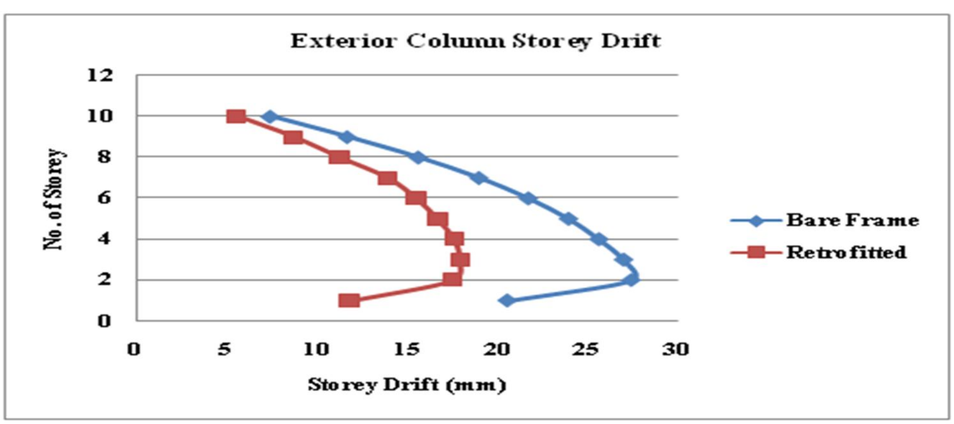
Storey displacement increased with increasing number of storey in both building structure. But the comparative study of storey displacement for bare frame and retrofitted structure revealed that storey displacement decreased for retrofitted structure. This is the consequence of adding additional stiffness to the building column by steel jacketing technique.

Storey drift have damaging effect lateral load resisting element. Therefore a comparative results of storey driftare framed for bare frame and retrofitted structure subjected to three ground motions in Table 3.4.



(a)

(b)



(c)

Figure 3.4 - Storey Drift for Bare Frame and Retrofitted Building for (a) Imperial Valley (b) North Ridge and (c) Loma Prieta Earthquake

IV. CONCLUSION

Based on this analytical study following conclusion are drawn:

1) The fundamental time period is more for Bare Frame than Retrofitted building.
2) The displacement of Retrofitted building is (20 % - 40 %) less than bare frame.
3) Exterior column shear forces of Retrofitted building are (5 % - 20 %) less than bare frame.
4) Base shear of Retrofitted building with steel jacketing is more than the Bare Frame.
5) Inelastic capacity of Retrofitted building with steel jacketing is more than the Bare Frame.
6) The Retrofitted building performs well in earthquake than bare frame due to provision of steel jacketing.

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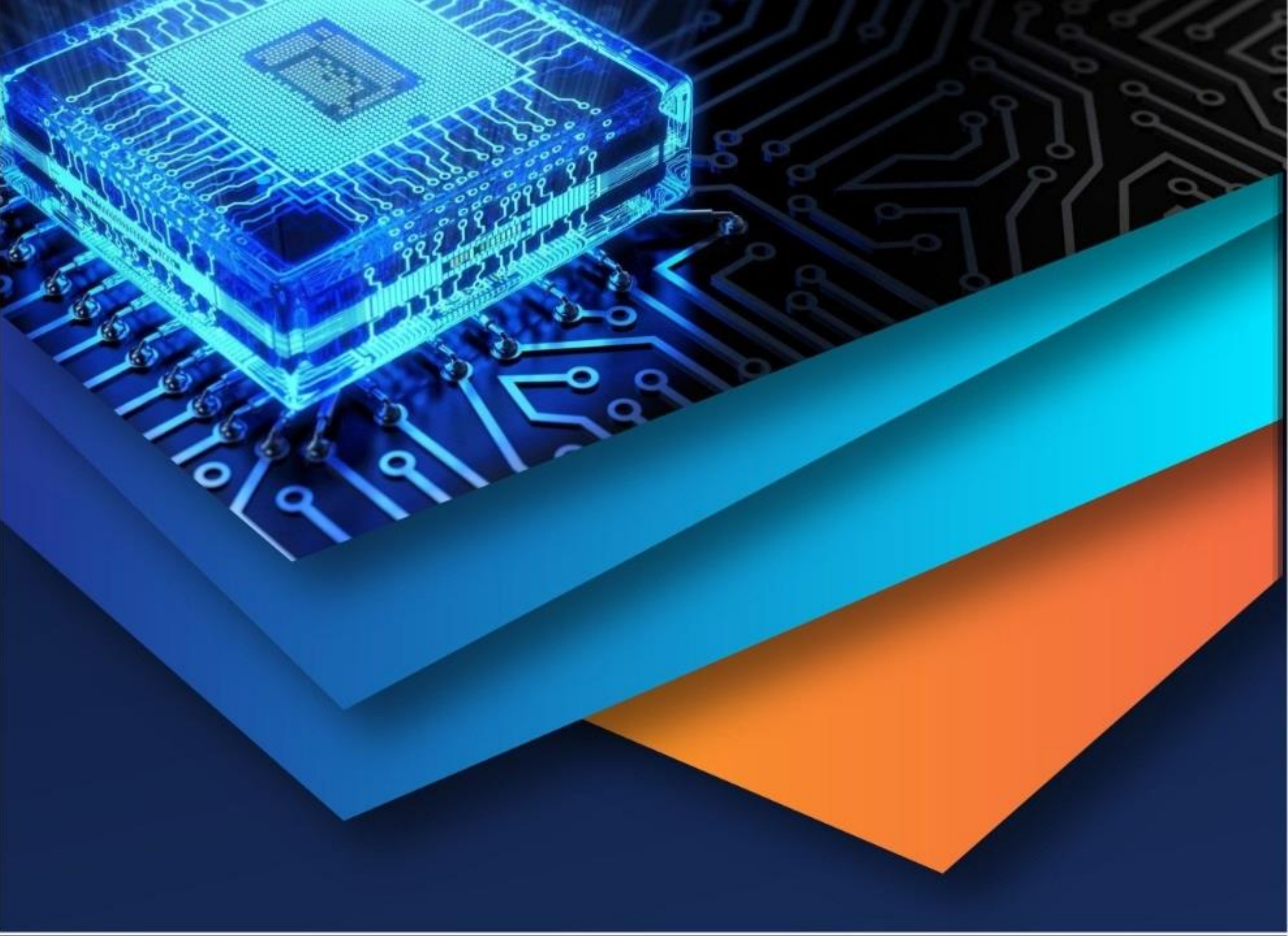

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