"Seismic Behavior of Retrofitting for R.C. structure Using R.C.C Jacketing and Steel Jacketing

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ABSTRACT

A seismic design is based upon mixture of strength and ductility. Regular seismic disturbances, the structure is expected to remain in the elastic range. By seeing the actual dynamic nature of environmental disturbances, more improvements are needed in the design procedures, and some advance methods are used to strengthen the existing structures i.e., different retrofitting methods. The main objective of the present study is to analyze the behavior of Retrofitted building i.e., provision of steel jacketing in growing the performance of building. The present study aims at checking the capability of multi-storey frame structures using retrofitting methods for the seismic excitations. The Retrofitted building i.e., provision of steel jacketing is analyzed and associated with bare frame structure by using time history and pushover analysis method by using Commercial software SAP2000 v16 is used for analysis. The answers of the structure are associated 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 creation during earthquake at beam-column junction can improved performance with use retrofitting method i.e. steel jacketing.

Keyword: **-** *FEMA-356, Retrofitted, Adequacy, Steel wrapping.*

1. INTRODUCTION

2.1 General

A seismic design is based upon mixture of strength and ductility. For small, common seismic disturbances, the structure is expected to remain in the elastic range with all stress well below the yield level. However, it is not practical to expect that the traditional structure will respond elastically when exposed 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 viewpoint has led to the development of a seismic design codes featuring lateral force methods and more recently, inelastic methods. Ultimately, with these methods, the structure is designed to resist an equivalent static load and results have been reasonably successful. Even an estimated 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.

2.2 Techniques of Retrofitting

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

2.3 Fiber Reinforced Polymer Technique

The most shared structural retrofitting methods are concrete and steel jacketing. In recent years fiber- reinforced polymer (FRP) materials are used to substitute 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 central of the column sides, especially for large columns. [18]

2.4 Composite Jacketing System

Advanced composite materials have been recently documented and applied to bridge retrofit. The general outlooks from composite retrofit systems include light weight, high stiffness or strength to weight ratios, etc. Many composite jacketing systems have been developed and validated in laboratory or field conditions. A system containing 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 fused 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 on 40% scale bridge piers wrapped with the glass fiber composite jacketing demonstrated important 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]

2.5 Steel Jacketing Technique

Shear failure of short concrete columns has been one of the main problems that may cause the failure of structures under earthquake attacks. In a construction 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 grow the moment capacity of column. The design of flexural strength based on elastic methods, along with less traditional 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 frequent 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) examined 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 purposely 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. Outstanding seismic behavior was obtained for circular columns. Due to insufficient confinement of concrete in the potential 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 improve 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 outline of the elliptical jacket increases the section of the columns substantially; thus, it may not be desirable from the architectural and practical 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 dissimilar 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, mainly for the potential plastic hinge regions near the ends of a column. Spacing of the hoops and ties lengthways 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.

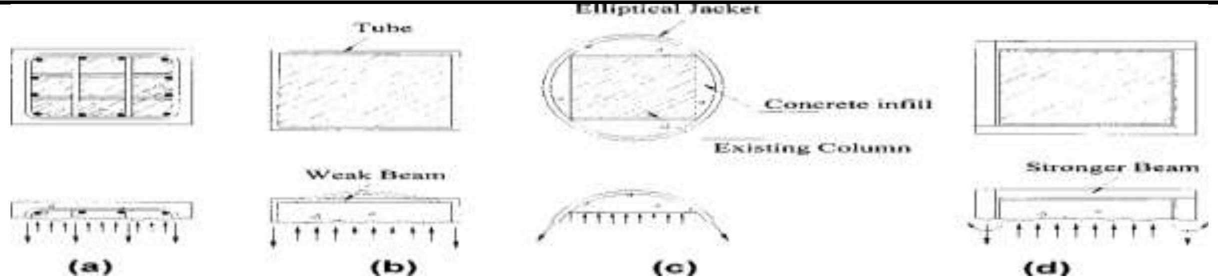


Figure 1. Comparison of different transverse confinements for concrete columns: (a) hoops and ties per current seismic design provisions; (b) steel tube; (c) elliptical steel jacketing; and (d) partially stiffened rectilinear jacketing

3. MODELLING AND ANALYSIS OF BUILDING

2.6 General

The present chapter contains info about geometry of building structure, properties of material used to erect the building model and some statement that are necessary for modelling and analysis. At the commencement 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.

2.7 Building Geometry

In the present work a 3-D structural model is used which includes of G+9 storey reinforced concrete moment resisting frame. The foundation of the structure is expected 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

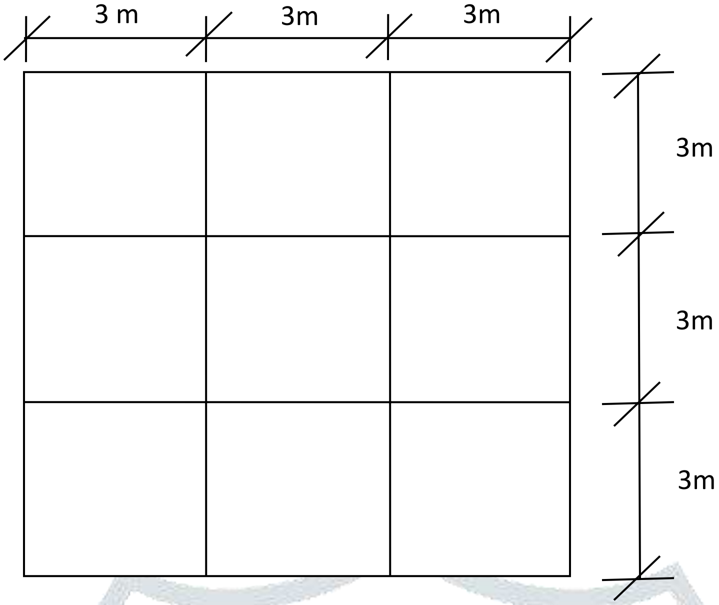


Figure 2.1 - Plan of Modeled Building

2.8 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 fck = 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).

2.9 Steel Jacket Modelling

The steel grade used for jacketing of RC column is Fe250. The steel jacket used for retrofitting purpose is not 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 adding to additional strength of column. Xiao and Wu have suggested a retrofit design process was developed in order to provide additional confinement and shear strength to change 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

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

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

whereAsh = total transverse steel cross-sectional area within spacing s; hc= cross-sectional dimension of column core measured center-to-center of the outermost peripheral hoops; f_c'= specified compressive strength of concrete; fyh = specified yield strength of transverse reinforcement; Ag = gross area of section; and Ach =cross- sectional area of a column measured out-to-out of transverse reinforcement. From Eqs. 3.2 and 3.3 an equivalent transverse pressure feqcan 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 expected 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 mechanism is 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 calculations 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 mn and pn= nominal flexural and tensile strengths per unit width, whereas Ø and Øb = 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 Ach can be considered the same of Ag. Thus, Eq. 2.3 or 2.5 governs the design.

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

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

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

wheretis the thickness of the jacket plate and f y j is its yield strength. Substituting these strength expressions intothe 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:

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

2.10 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 center 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 clarified 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 previous decades are as follows

"Uniform" Lateral Load Pattern

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

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

Where ,

Fi= lateral force at ithstory mi = 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 at that story,

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

Where,

Øi= amplitude of the elastic first mode at ithstory.

"Code" Lateral Load Pattern

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

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

Where

Vb= base shear

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

ΔFN = additional earthquake load added to the Nth story whenhN>25m

(For hN>25m, ΔFN = 0 otherwise; ΔFN = 0.07T1Vb≤ 0.2Vb ,where T1 is the fundamental period of the structure)

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

Where

Qi = Design lateral force at floor i, Wi = Seismic weight of floor i,

hi = 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 properties 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) combinations of the load distributions obtained from the modal analysis of the structures as follows:

Calculate the lateral force at ith storey for nth mode from equations

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

Where,

Γn= modal participation factor for the nth mode Øin= Amplitude of nth mode at ith story

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

Calculate the storey shears, $V_{in} = \sum_{j=1}^{N} F_{jn}$,where N is the total number of storeys

Combine the modal storey shears using SRSS rule, $V_i = \sqrt{\sum_n (V_{in})}$

Back calculate the lateral storey forces Fi , at storey levels from the combined storey shears, Vistarting from the top storey.

Normalize the lateral storey forces by base shear for convenience such that

$$F_i^{'} = F_i / \sum F_i \qquad (2.18)$$

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

3. RESULTS AND DISCUSSION

3.1 Introduction

In this, 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 simple frame model through pushover curves in pushover analysis and storey displacements, storey drift, shear force and moment 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 also analyzed. The results obtained this analysis are associated using tables and graphs.

3.2 Modal Time Period and Frequency

The time period of both bare frame and retrofitted building are considered using modal analysis. The time period and 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 uppermost for first mode 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 designates 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 torsional direction for first, second third and fourth mode.

Table 3.2 - Frequency of Bare Frame and Retrofitted building.

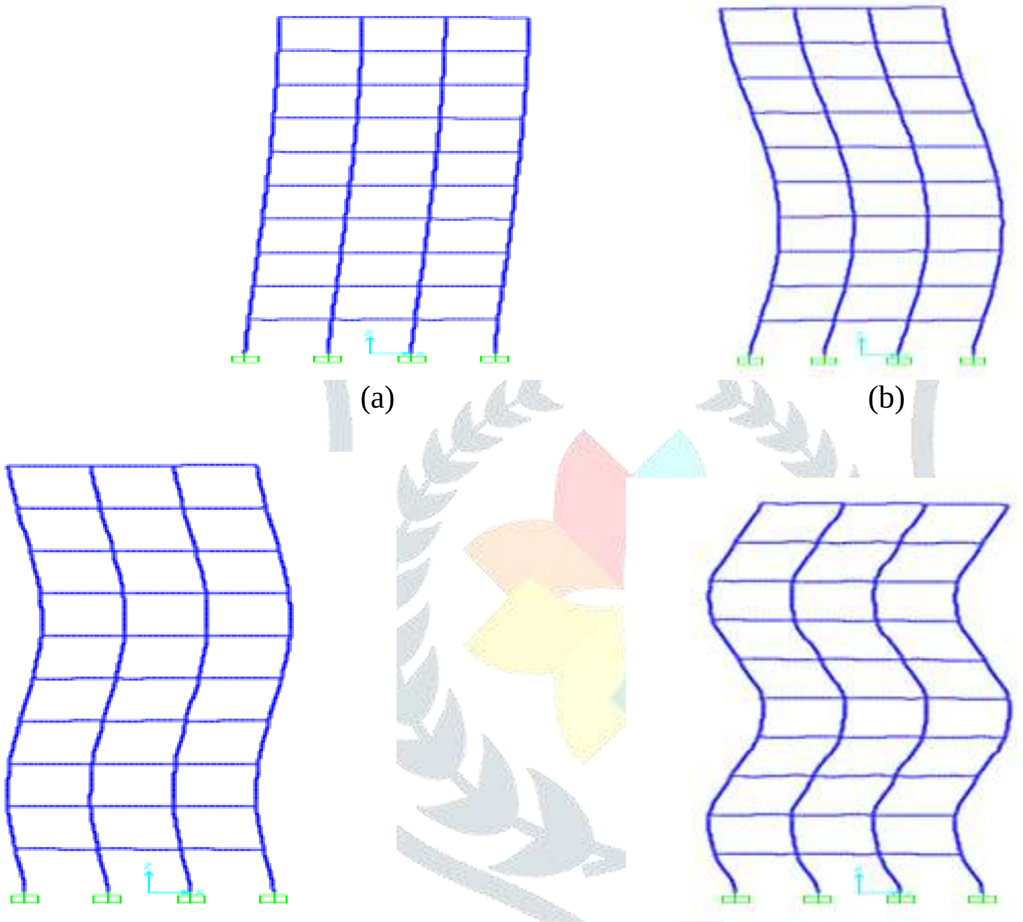
Direction	**Mode No.**	**Frequency (htz)**	
		Bare Frame	**Retrofitted**
X	1	0.743	0.845
	2	2.26	2.579
	3	3.914	4.526
	4	5.57	6.506
Y	1	0.743	0.846
	2	2.26	2.580
	3	3.914	4.526
	4	5.57	6.508
Torsion	1	0.82	0.921
	2	2.49	2.979

	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, 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.

3.3 Mode Shapes

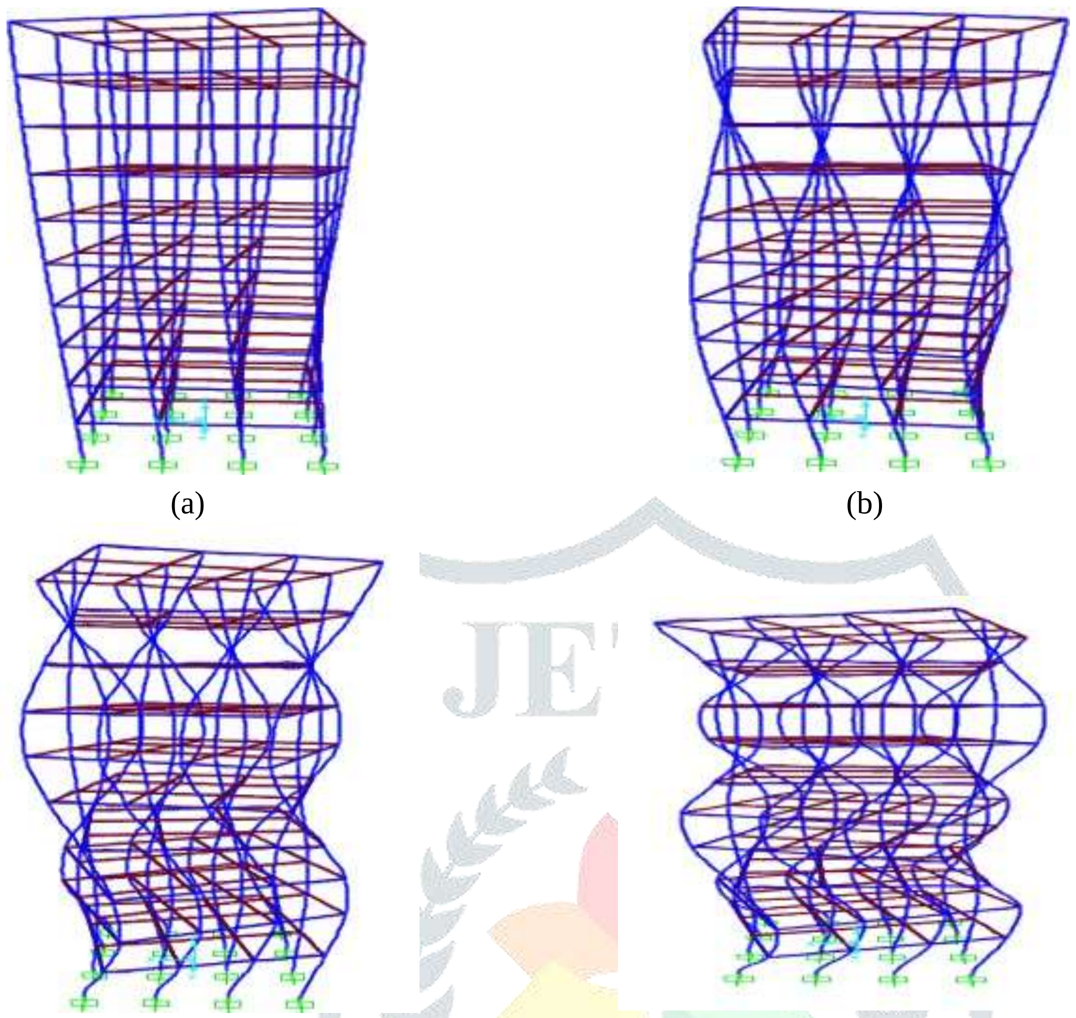
The mode shapes found for bare frame model are shown in Figure 4.1. Same type of mode shapes was got for retrofitted building model. Since the mode shape obtained in X and Y direction are similar therefore mode shape of X and torsional mode are only shown.



(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)

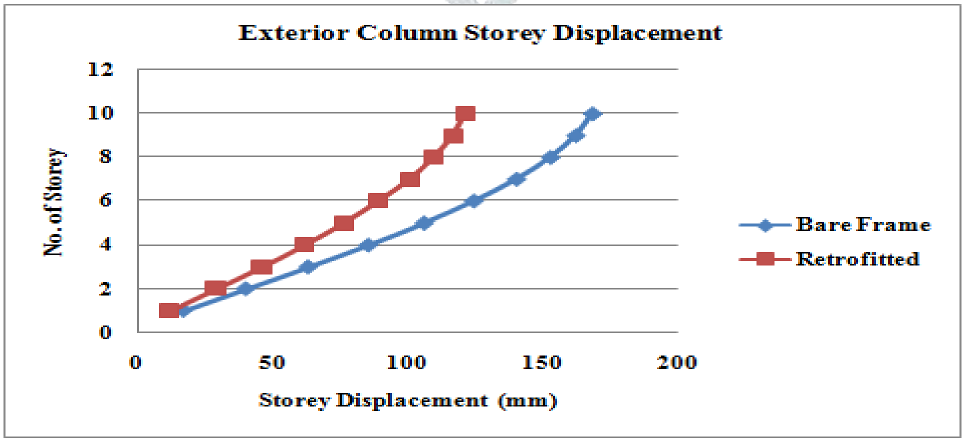
(c) (d)

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

3.3 Linear Time History Analysis

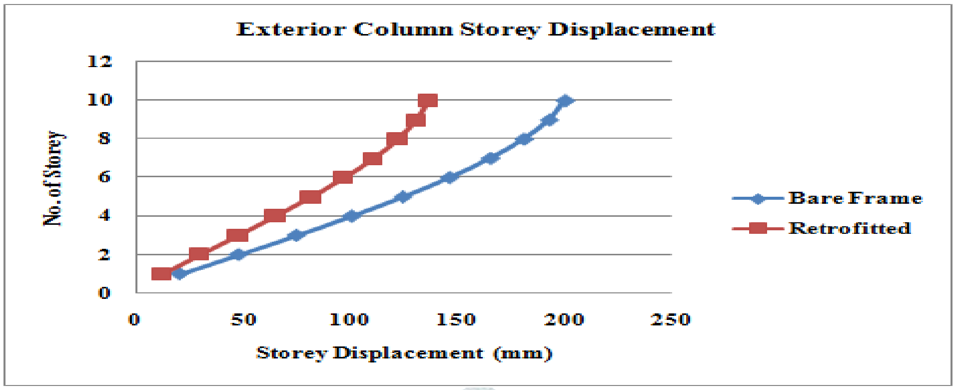
To study the response of building under real earthquake ground motions linear dynamic time history analysis is carried out. This analysis exhibits real earthquake effects and the responses obtained are very practical.

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)

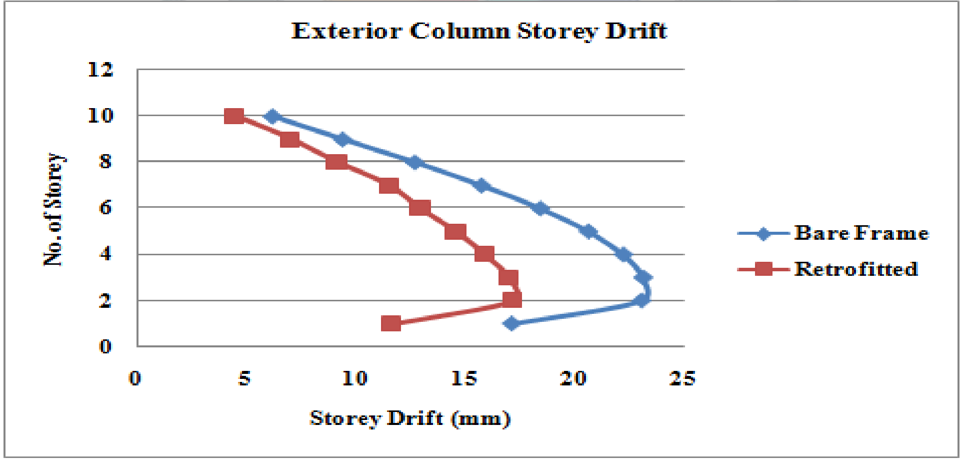


(c)

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

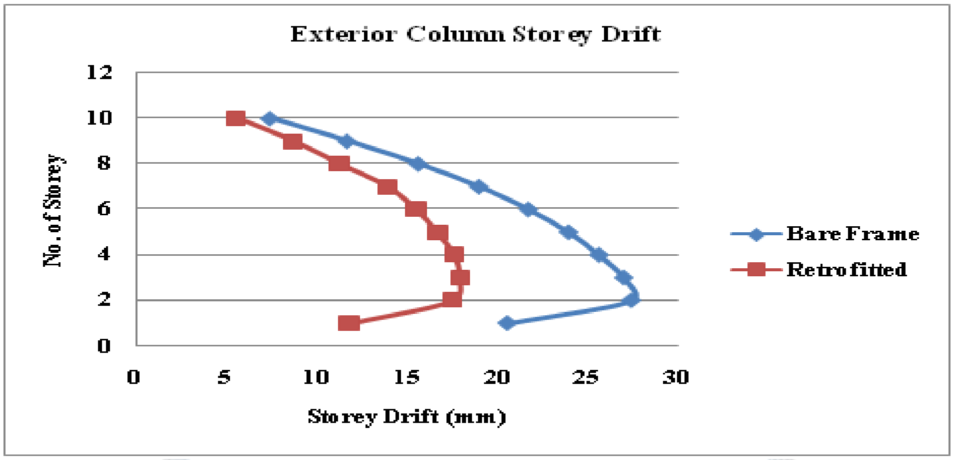
Storey displacement amplified with increasing number of storey in both building structure. But the comparative study of storey displacement for bare frame and retrofitted structure exposed that storey displacement decreased for retrofitted structure. This is the consequence of adding additional stiffness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

Storey drift have destructive effect lateral load resisting element. Therefore, a comparative results of storey drift are 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

4. CONCLUSION

Based on this analytical study following conclusion are drawn:

1. Fundamental time period is more in Bare Frame than Retrofitted building.
2. Displacement of Retrofitted building is (20 % - 40 %) less than the bare frame.
3. Exterior column shear forces of Retrofitted building are (5 % - 20 %) less than the 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 the bare frame due to provision of steel jacketing.

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