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Seismic Response of Retrofitting for High Rise R.C. Structure by Using R.C.C Jacketing and Steel Wrapping

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***Abstract:** A seismic design depends on mix of solidarity and pliability. Incessant seismic aggravations, the structure are relied upon to stay in the flexible reach. By thinking about the real powerful nature of ecological unsettling influences, more upgrades are required in the plan methodology. What's more, some development procedures are utilized to fortify the current structures i.e. distinctive retrofitting methods. All these technique shave their own benefits. The principle objective of the current examination is to dissect the conduct of Retrofitted fabricating i.e. provision of steel jacketing in expanding the presentation of building. The current examination targets checking the adequacy of multi-story outline structures utilizing retrofitting strategies for the seismic excitations. The Retrofitted building for example arrangement of steel jacketing is investigated and contrasted and exposed edge structure by utilizing time history and sucker examination strategy by utilizing Commercial programming SAP2000 v16 is utilized for examination. The responses of the structure are compared by considering various boundaries i.e. displacement, base shear, plastic pivots, time-frame of mode shapes from FEMA – 356. The outcome shows that plastic pivot formation during quake at shaft section intersection can further developed execution with use retrofitting strategy for example steel jacketing.*
***Keyword:** FEMA-356, Retrofitted, Adequacy, Steel jacketing*

I. INTRODUCTION

A. General

A seismic plan depends on blend of solidarity and flexibility. For little, continuous seismic disturbances, the structure is required to stay in the flexible reach with all pressure well underneath the yield level. Nonetheless it's anything but sensible to expect that the customary construction will react elastically when exposed to serious quake. Rather the plan engineer depends upon the inborn flexibility of the structure design to prevent catastrophic disappointment while tolerating certain degree of primary and non-underlying harm. This way of thinking has led to the improvement of a seismic plan codes including parallel power techniques and all the more as of late, in elastic methods. At last, with these methodologies, the construction is intended to oppose a comparable static burden and results have been sensibly successful. Even an inexact representing sidelong impacts will in all likelihood further develop building survivability. Be that as it may, by thinking about the real unique nature of environmental disturbances, more upgrades were made in the plan techniques. Accordingly from the dynamical point of view, new and imaginative ideas of primary security framework progressed and are at different phases of advancement.

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 instructural 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 ofsquare/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 material shave been recently recognized and applied to bridge retrofit . The general expectations from composite 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. A system 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 hand 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 disappointment of short substantial sections has been one of the serious issues that may cause the breakdown of structures under quake assaults. In a construction where the sections have various lengths, more limited columns tend to draw in a more noteworthy bit of the seismic contribution during a tremor and require the age of large seismic shear powers to foster the second limit of segment. The plan of flexural strength dependent on elastic methods, alongside less traditionalist shear strength arrangements in more established plan codes, ordinarily came about in expected shear strength of segments in many existing designs being not exactly the flexural strength. These have been proved by the fragile disappointment of columns that made various constructions breakdown in previous earthquakes The utilization of a steel coat or cylinder to upgrade the strength of sections and to further develop deformability was studied previously. Sakino and Ishibashi(1985) explored the seismic execution of cement filled steel tubular(CFT) segments and tracked down that plastic clasping of the steel tube in the pivot locales would in general happen when the sections were exposed to enormous cyclic horizontal displacements. Tomii, Sakino, and Xiao(1987)and Xiao(2001)investigated steel-tubed short segments in building structures as an action to forestall shear disappointment and to further develop flexibility . To stay away from the clasping of the steel tube saw by Sakino and Ishibashi(1985)for regular CFT sections, the cylinder was intentionally ended to leave holes from the segment closes, thus ensuring the cylinder to work primarily as loop support instead of additionally contributing in flexural strength. Excellent seismic conduct was acquired for round segments. Because of lacking restriction of cement in the potential plastic pivot locale ,it was discovered that decay of reaction was unavoidable for rectangular sections, except if a thick steel tube was utilized, especially for segments with hub load surpassing 30% of axial load limit. The issues become moderately less serious for steel-tube high-strength substantial segments subjected to lower pivotal burden. [5]

Priestley et al. (1994) researched circular coats to improve the shear strength of rectangular sections. This method has now been generally utilized in retrofitting rectangular segments in spans in California and elsewhere. However, the profile of the circular coat expands the segment of the sections substantially; thus ,it may not be alluring from the design and useful perspectives, especially for retrofitting segments in structures where most segments are rectangular or square [4]. Aboutaha et al. (1996) tried a framework that joined a through bolt with a generally slight rectangular jacket, and showed improved imprisonment productivity .In this study;the authors fostered another improved jacketing technique to retrofit square segments utilizing welded rectilinear steel jackets and stiffeners. [5]

Fig. 1 sums up and schematically thinks about the four diverse cross over fortifications. In an all around bound built up substantial segment configuration dependent on current seismic plan arrangements, as displayed in Fig. 1-a, bands or twisting sand crossties are gave to contain the center concrete, particularly for the possible plastic pivot locales close to the closures of a segment. Dividing of the loops and ties along the segment and the time frames crossties inside the part are restricted to accomplish better proficiency of imprisonment.

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A comparative confinement mechanism is accomplished for retrofitted sections utilizing the joined jacketing and through blasting technique by Aboutaha et al. (1996). In a cylinder segment with a square or rectangular area, as displayed in Fig. 1-b, the weakout-of-plane solidness brings about helpless restriction of segments of the substantial segment. As displayed in Fig. 1-c,the utilization of a curved molded steel coat for retrofit can give a consistent cross over repression to the existing substantial segment. The mostly hardened rectilinear steel coat created in this examination intends to relyon a pillar activity of the repression components (stiffeners) to foster proficient cross over imprisonment to the concrete section, as delineated in Fig. 1-d.

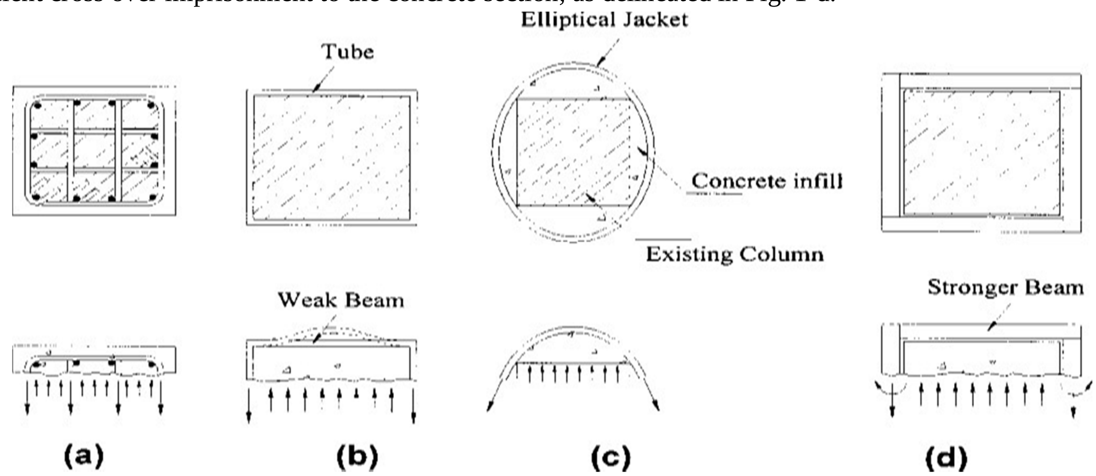

Figure1. Comparisonofdifferenttransverseconfinementsforconcretecolumns:(a)hoopsandtiespercurrentseismicdesignprovisions;(b)steeltube;(c) elliptical steel jacketing; and (d) partially stiffened rectilinear jacketing

II. MODELLINGAND ANALYSISOFBUILDING

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 modeling and analysis. At the beginning 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 current work a 3-D underlying model is utilized which involves G+9 story built up concrete moment resisting outline. The establishment of the construction is thought to be fixed. The information expected for the examination ofbuildingisshown in Table 2.1.

Table2.1:GeneralDescriptionofBuilding

Sr.No	Entity	Description
1	NoofBays in XDirection	3
2	NoofBaysin YDirection	3
3	Widthof BayinXDirection	3 m
4	Widthof BayinYDirection	3 m
5	StoreyHeight	3 m
6	LiveLoad	3 kN/m^2
7	FloorFinish	1 kN/m^2
8	ConcreteGrade	M20
9	Rebar	Fe415
10	BeamSize	250 mm x250mm
11	ColumnSize	300 mm x300mm

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C. Material Properties

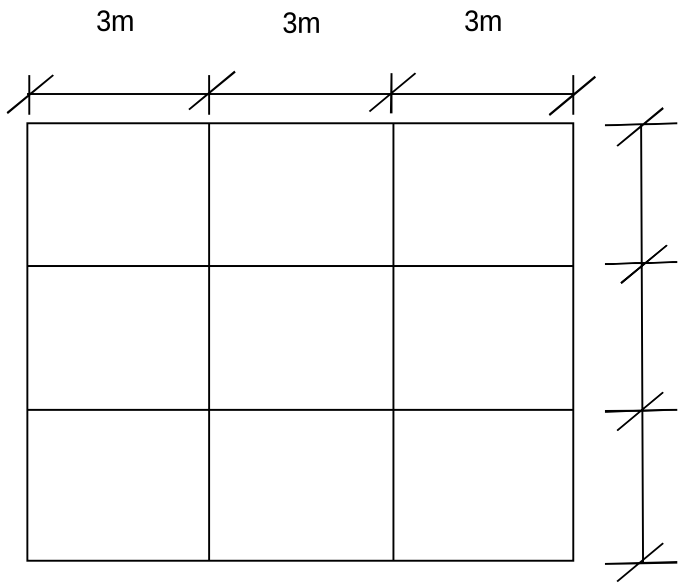

Figure2.1 –Plan of Modeled Building

M-20 grade of concrete and Fe-415 grade of reinforcing steel are used for all the frame models used in thisstudy. Elastic material properties of these materials are taken as per Indian Standard IS 456 (2000). The short-termmodulus 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). Forthesteel rebar,yieldstress (fy) andmodulus of elasticity(Es) is takenas perIS456(2000).

D. Steel Jacket Modelling

The grade of steel used for jacketing of RCcolumnis Fe250.The steeljacket used for retrofitting purpose is not gave over the full length of segment yet it just gave at conceivable pivot area. The coat provided around the section ought to just go through shearing activity and ought avoid bowing of segment adding to extra strength of segment. Xiao and Wu have recommended a retrofit plan method was created in order to give extra control and shear solidarity to change a current lacking segment over to the condition fulfilling current seismic plan arrangements. In the seismic plan arrangements of the current ACI 318code(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 columncore measured center-to-center of the outermost peripheral hoops;f '= specified compressive strength ofconcrete; fyh= specified yield strength of transverse reinforcement; Ag = gross area of section; and Ach =cross-sectionalareaofacolumnmeasuredout-to-outoftransversereinforcement.FromEqs.3.2and3.3anequivalenttransversepressure feqcan bedefined 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

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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 designfortheconfinementelementisbasedonalimitstatewhereayieldmechanismis 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 mn and pn= nominal flexural and tensile strengths per unit width, whereas Ø and Øb= correspondingresistance factors, takenas 1.0 in this study.

In a retrofit design situation where an additional jacket is provided to confine the full column section, Eqs. 2.2and 2.3 are automatically satisfied, since Achcan be considered the same of Ag. 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 foundas,

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

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

Where is the thickness of the jacket plate and fyjis 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 todeterminethe thickness of thejacket plate:

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

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 the pushover analysis has been recognized for last two decades years. This procedure is mainly used to estimate the strength 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 40andFEMA356) and design codes(Euro code8and PCM3274) in last few years.

F. Lateral Load Profile

The examination results are delicate to the determination of the control hub and choice of sidelong burden design. In general case, the focal point of mass area at the top of the structure is considered as control hub. In pushover analysis choosing parallel burden design, a bunch of rules according to FEMA 356 is clarified in Section 2.5.2. The lateral load commonly applied in both positive and negative ways in mix with gravity load (deadload and a part of live burden) to examine the genuine conduct. Various kinds of sidelong burden utilized in past decades are as follows

"Uniform"LateralLoadPattern

Thelateral foreat anystoryis proportional to the mass at that story.

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

Where,

Fi=lateral force at ithstorymi=mass ofi-th story

"FirstElasticMode"LateralLoadPattern

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

Where,

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

Øi=amplitudeof theelasticfirstmodeatithstory.

"Code"LateralLoadPattern

ThelateralloadpatternisdefinedinTurkishEarthquakeCode(1998)andthelateralforceatanystoreyiscalculated from thefollowingformula:

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

Where

Vb=baseshear

h=heightofi-thstoryabovethebaseN=total number ofstories

ΔFN =additional earthquakeload added to theNthstorywhenhN>25m

(ForhN>25m,ΔFN=0 otherwise;ΔFN=0.07T1Vb≤0.2Vb,whereT1is thefundamentalperiod ofthe structure)

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

Where

Qi=Designlateralforceatfloori,Wi=Seismic weightof floor i,

hi=Heightof floorimeasuredfrom base,and

n=Number ofstories in thebuildingis thenumber oflevelsat which themasses arelocated.

"Multi-Modal(orSRSS)"LateralLoadPattern

The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregularstructuresandthelateralforceatanystoryiscalculatedSquareRootofSumofSquares(SRSS)combinationsoftheload distributions obtained fromthe modalanalysis of thestructures as follows:

Calculate thelateral forceat ithstoreyfornthmode from equations

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

Where,

*Γ*n=modal participation factor for the nth modeØin=Amplitude ofnthmodeat ithstory

An=Pseudo-accelerationofthen-thmodeSDOFelasticsystem

Calculatethe storeyshears, $V_{in} = \sum_{j=1}^{N} F_{jn}$, whereNis thetotalnumber ofstoreys

Combinethe modalstoreyshearsusingSRSSrule, $V_i = \sqrt{\sum_n (V_{in})}$

BackcalculatethelateralstoreyforcesFi,atstoreylevelsfromthecombinedstoreyshears,Vistartingfromthetop storey.

Normalizethe lateralstoreyforces bybaseshearforconveniencesuch that

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

Thefirstthreeelasticmodesofvibrationofcontributionwasconsideredtocalculatethe"Multi-Modal(orSRSS)"lateral load pattern in this study.

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III. RESULTSANDDISCUSSION

A. Introduction

In this part the uncovered casing model and retrofitted fabricating model are examined utilizing direct time history analysis and weakling examination. The conduct of the retrofitted assembling model is contrasted and uncovered frame model through sucker bends in weakling examination and story relocations, story float, shear power and moment in outside outline segment in direct time history investigation. Some boundary of the two structures areevaluatedatperformance point. The time period and frequency of building along with mode shapes are also analyzed. The results acquired the se analysis are thought about 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 period and frequency are analyzed in X, Y and torsional direction. Table 4.1 shows time period for bare frame and retro fitted building in X,Y and torsional direction for first, second, third and fourth mode of vibration

Table3.1-Modal Time Period of Bare Frame and Retrofitted building.

Direction	ModeNo.	TimePeriod(sec)	
		BareFrame	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

FromTable3.1 it can observed that modal time period for bareframe and retrofitted building is highest for first mode and decreases with expanding mode number in X, Y and torsional method of vibration. Additionally it is also observed that modular time span in X and Y course for first, second, third and fourth mode is same which clearly shows that the structure is symmetric in math. At the point when the modular time-frame of exposed frame structure and retrofitted fabricating are analyzed in their individual mode and bearing, the modular time-frame is found less if there should arise an occurrence of retrofitted working than uncovered edge building. This is the aftereffect of the expanded stiffness which has happened due to steel jacketing of the RCC column near the plastic hinge region.

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The frequencies of exposed edge and retrofitted structure are thought about in Table 3.2 in X, Y and torsional direction for first, second third and fourth mode.

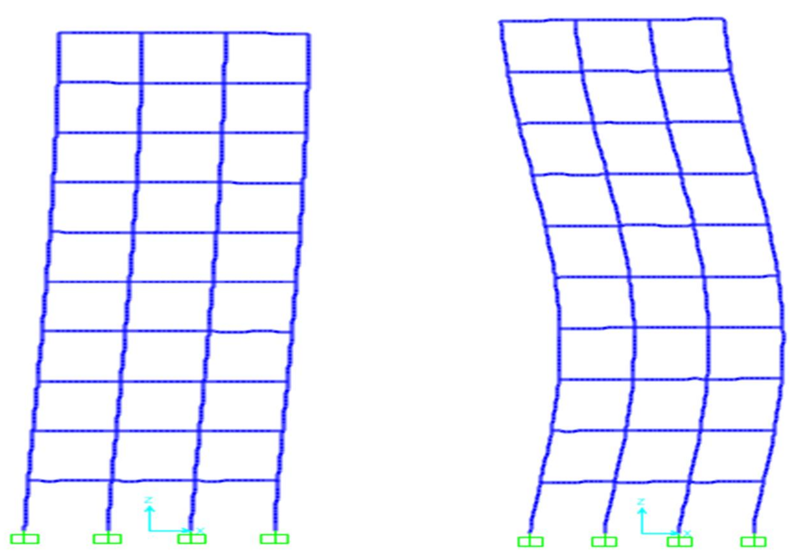
Table3.2-Frequencyof BareFrameandRetrofittedbuilding.

Direction	ModeNo.	Frequency(htz)	
		BareFrame	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 aftereffects of Table 3.2 says that recurrence is greatest in the event of fourth mode and decreases accordingly with decreasing mode number in both uncovered edge and retrofitted structure. At the point when the frequencies of uncovered edge and retrofitted structure are analyzed the upsides of retrofitted structure had expanded with little edges in the irrespective mode and direction. This change was observed due to steel jacketing which increased the stiffness of column. This expanded recurrence and brought down time-frame of the retrofitted fabricating connotes that the acceleration of the construction had expanded and the relocations that will happen in retrofitted assembling are less in comparison to bare frame structure.

C. Mode Shapes

The mode shapes obtained for bare frame model are shown in Figure4.1. Same type of mode shapes were obtained 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)

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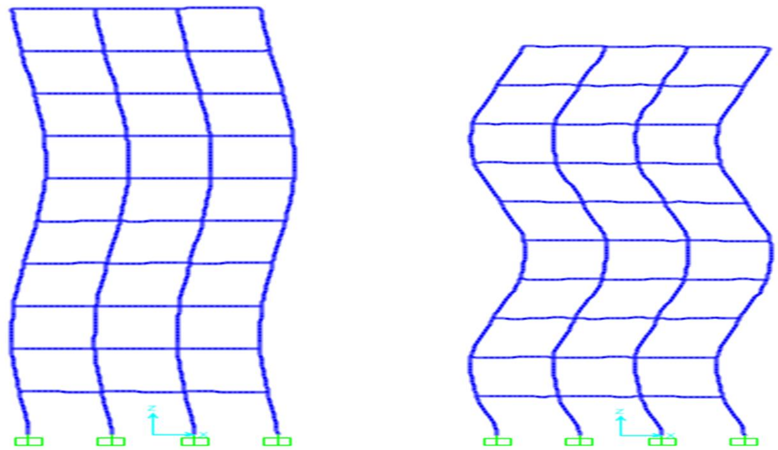
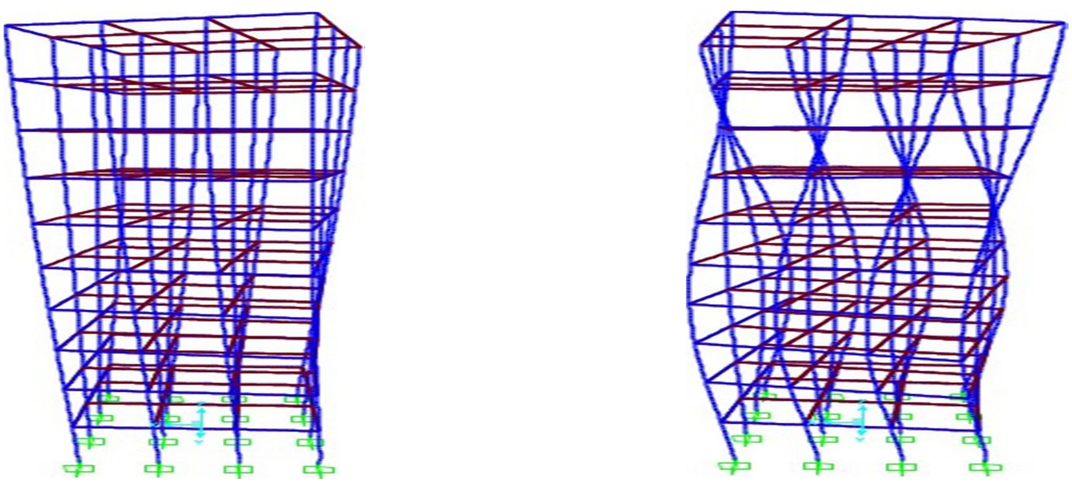
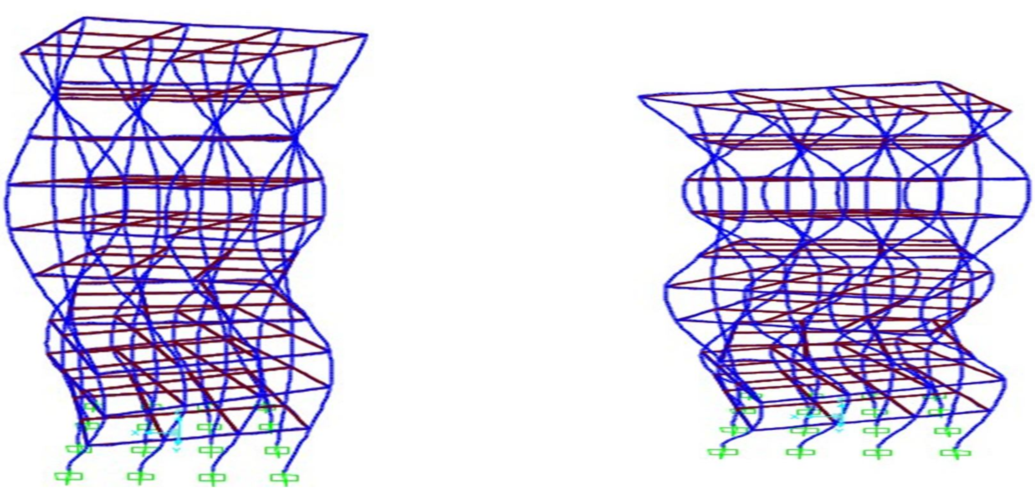


(c) (d)

Figure3.1-Picture(a),(b),(c)and(d)representfirst,second,thirdandfourthmodeshapeinXandYdirections.



(a) (b)

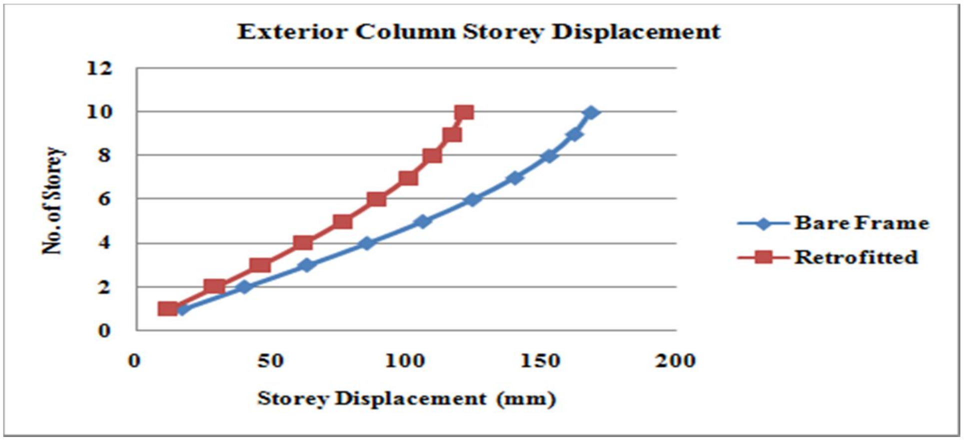


(c) (d)

Figure3.2-Picture(a)depictsfirst modeshape(b)depictssecondmodeshape(c)depictsthird modeshapeand(d)fourthmodeshapeintorsion

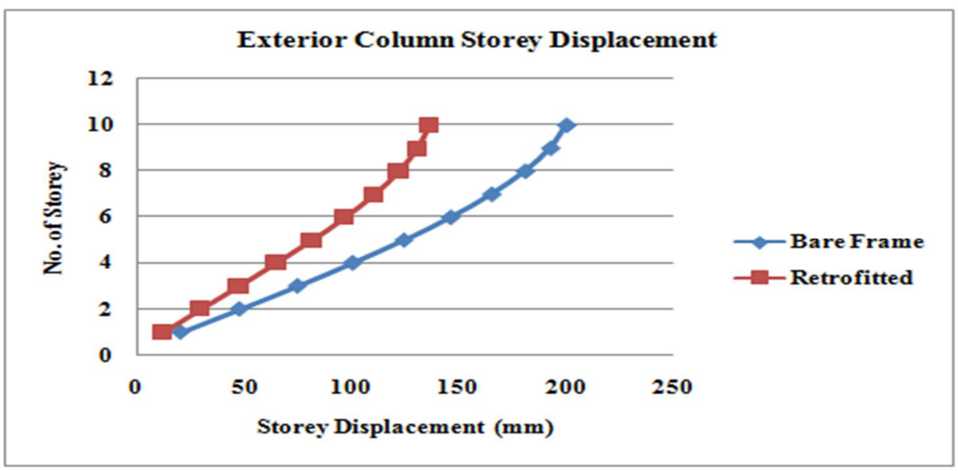
D. Linear Time History Analysis

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. Table4.3 depicts storey displacement of bare frame and retrofitted building for three different acceleration time histories.



(a)

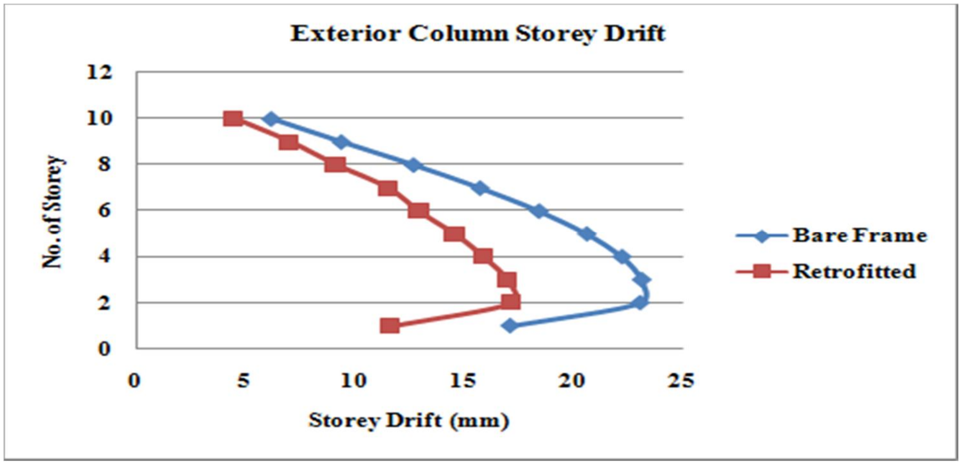
(b)



(c)

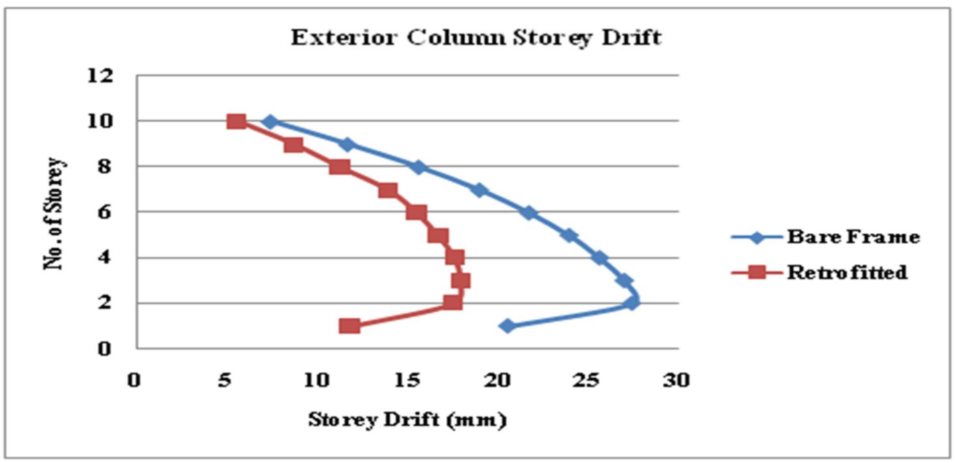
Figure3.3-StoreyDisplacementforBareFrameandRetrofittedBuildingfor(a)ImperialValley(b)NorthRidgeand(c)LomaPrietaEarthquake

Storey displacement increased with increasing number of storey in both building structure. But the comparativestudyofstoreydisplacementforbareframeandretrofittedstructurerevealedthatstoreydisplacementdecreased 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.There fore a comparative result so fstorey drift are framed for bareframe and retrofitted structure subjected to three ground motions in Table 3.4.



(a)

(b)



(c)

Figure3.4-StoreyDriftforBareFrameandRetrofittedBuildingfor(a)ImperialValley(b)NorthRidgeand(c)LomaPrietaEarthquake

IV. CONCLUSION

Basedon this logical studyfollowingconclusion aredrawn:

A. The fundamental time period is more for Bare Frame than Retrofitted building.
B. The displacement of Retrofitted building is(20 %-40 %)less than bareframe.
C. Exterior column shear forces of Retrofitted buildingare(5%-20 %)less than bareframe.
D. Base shear of Retrofitted building with steel jacketing is more than the BareFrame.
E. In elastic capacity of Retrofitted building with steel jacketing is more than the Bare Frame.
F. The Retrofitted building performs well in earthquake than bare frame due to provision of steel jacketing.

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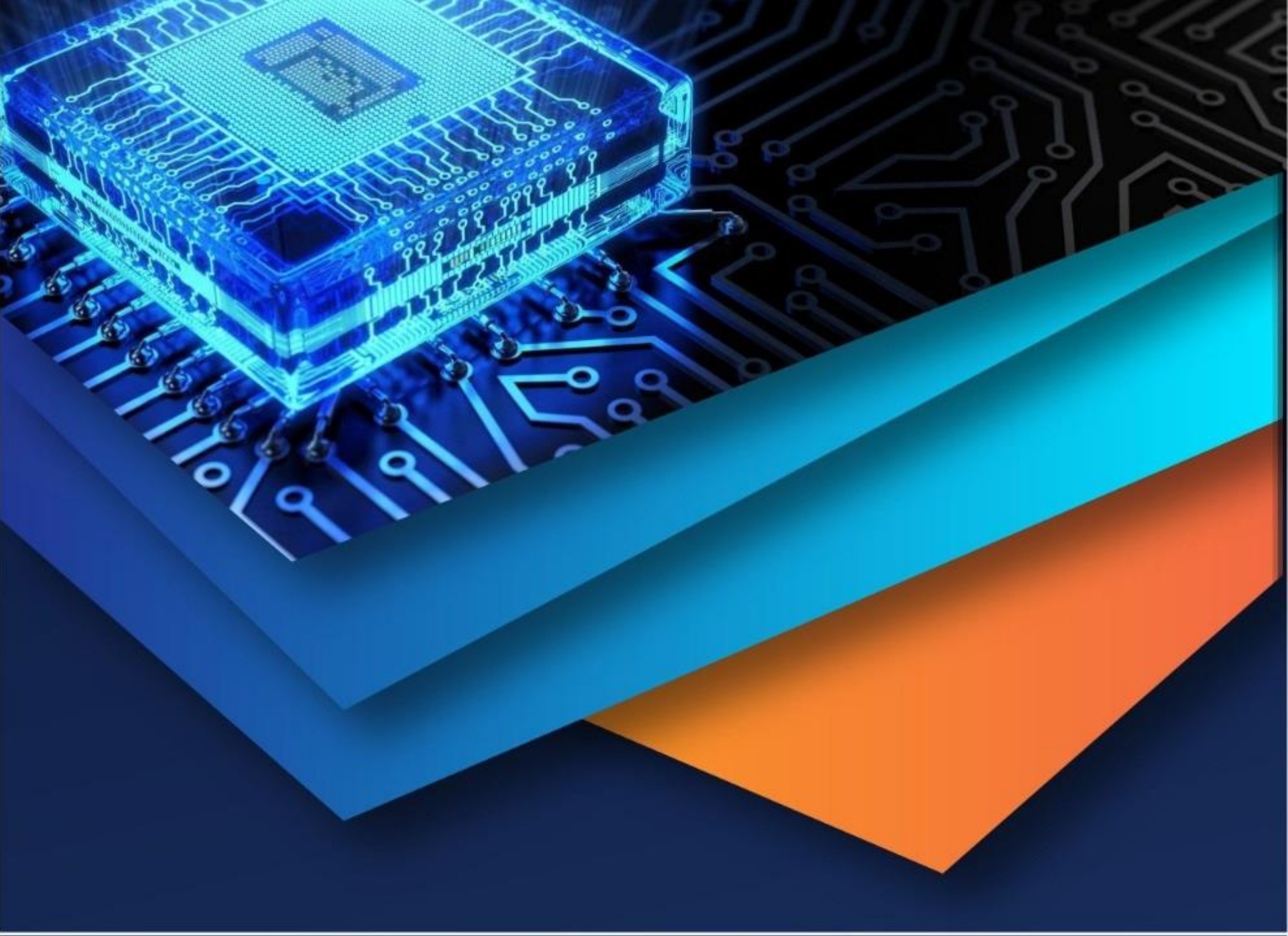

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