

# Seismic Retrofit of RC Building Using Jacketing – A Review and Case Study

Gattu Ashok Kumar<sup>1</sup>, B Harish Nayak<sup>2</sup>

P.G. Student, Dept. of CE, Shri Shirdi Sai Institute of Science and Engineering, Ananthapuram, India

Assistant Professor, HOD, Dept. of CE, Shri Shirdi Sai Institute of Science and Engineering, Ananthapuram, India

**ABSTRACT:** : Many parts of the country have suffered earthquake in last three decades. In coastal part of South India faced Tsunami. In first three earthquakes it was found that many of damaged structures were built in non-engineered masonry techniques. Unreinforced masonry structures are the most vulnerable during an earthquake. Normally they are designed for vertical loads and since masonry has adequate compressive strength, the structures behave well as long as the loads are vertical. When such a masonry structure is subjected to lateral inertial loads during an earthquake, the walls develop shear and flexural stresses. In previous earthquakes many R.C.C buildings have also collapsed and are found unsafe due to faulty workmanship. Many other causes are responsible for major collapse and damage to the R.C.C structures. This thesis presents a case study of PL+SC+S+G+5 storey building, where this PL+SC+S+G+5 storey residential building is being converted to commercial building which results in increase of live load in existing building..

## I. INTRODUCTION

When the residential building is being converted to commercial building which results in increase of live load in existing building, from the physical and experimental investigations it was concluded that the building either should be demolished or at least should be retrofitted with suitable technique to increase its service life. Which can be overcome by new technique called retrofitting by jacketing. It was then decided to implement RCC column jacketing technique due to its feasibility and ease for execution. All the columns, Beams on both the floors are now suitably jacketed, the loose pockets of concrete that were investigated during the test are re-concreted, and the rusted reinforcement is replaced with new reinforcement as per the design requirement.

Retrofitting utilizes the concept of confinement by strengthening an existing beam or column with external lateral pressure. Confinement in reinforced concrete uses lateral steel reinforcement to delay the lateral expansion of the confined concrete, known as the concrete core, while also strengthening the longitudinal steel reinforcement by bracing it and increasing its buckling strength. Certain reinforced concrete columns are designed utilizing the concept of confinement with steel rebar hoops or spirals inside the concrete core, but the need for additional confinement is sometimes necessary. It is important to know how the new retrofit material will enhance and strengthen the existing members.

## II. LITERATURE

Yen-Po Wang [11] introduced the fundamentals of seismic base isolation as an effective technique for seismic design of structures. Spring-like isolation bearings reduce earthquake forces by changing the fundamental time period of the structure to avoid resonance. However, sliding-type isolation bearings filter out the earthquake forces via discontinuous sliding interfaces and forces are prevented from getting transmitted to the superstructure because of the friction. The design of the base isolation system includes finding out the base shear, bearing displacement etc. in accordance with site-specific conditions.

M C Griffith And A V Pinto [4] have investigated the specific details of a 4-storey, 3-bay reinforced concrete frame test structure with unreinforced brick masonry (URM) infill walls are described along with estimates of its likely weaknesses with regard to seismic loading. The concrete frame is shown to be essentially a “weak-column strong-

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beam frame” which is likely to exhibit poor post yield hysteretic behaviour. Based on the results of an extensive literature review, the building is expected to have maximum lateral deformation capacities corresponding to about 2% lateral drift. The unreinforced masonry infill walls are likely to begin cracking at much smaller lateral drifts, of the order of 0.3%, and to completely lose their load carrying ability by drifts of between 1% and 2%.

Hiroshi Fukuyama and Shunsuke Sugano [3] have emphasized on the importance of seismic rehabilitation, taking a leaf out of the destruction caused by the 1995 Kobe earthquake.

Many new techniques like seismic isolation, supplemental damping and continuous fibre wrapping were implemented after the disaster out of which the fibre wrapping became the most popular method. Details about the technique, replete with diagrams, were discussed.

Luigi Di Sarno and Amr S. Elnashai [8] assessed the seismic performance of a 9-storey steel perimeter MRF (moment resisting frame) using the three types of bracings: special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). Local (member rotations) and global (inter-storey and roof drifts) deformations were employed to compare the inelastic response of the retrofitted frames. MBFs were found to be the most cost-effective bracing system with the least storey drifts and hence, the most attractive system to be employed.

## III. RESEARCH METHODOLOGY

### INTRODUCTION

ETABS is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. ETABS Version 9.7.4 features an intuitive and powerful graphical interface coupled with unmatched modelling, analytical, and design procedures, all integrated using a common database. Although quick and easy for simple structures, ETABS can also handle the largest and most complex building models, including a wide range of nonlinear behaviours, making it the tool of choice for structural engineers in the building industry.

### HISTORY AND ADVANTAGES OF ETABS

As computers and computer interfaces evolved, ETABS added computationally complex analytical options such as dynamic nonlinear behaviour, and powerful CAD-like drawing tools in a graphical and object-based interface. Although ETABS Version 9 looks radically different from its predecessors of 30 years ago, its mission remains the same: to provide the profession with the most efficient and comprehensive software for the analysis and design of buildings. To that end, the current release follows the same philosophical approach put forward by the original programs, namely:

Most buildings are of straightforward geometry with horizontal beams and vertical columns. Although any building configuration is possible with ETABS, in most cases, a simple grid system defined by horizontal floors and vertical column lines can establish building geometry with minimal effort. \_ many of the floor levels in buildings are similar. This commonality can be used numerically to reduce computational effort.

The input and output conventions used correspond to common building terminology. With ETABS, the models are defined logically floor-by-floor, column-by-column, bay-by-bay and wall-by-wall and not as a stream of non-descript nodes and elements as in general purpose programs. Thus the structural definition is simple, concise and meaningful.

In most buildings, the dimensions of the members are large in relation to the bay widths and story heights. Those dimensions have a significant effect on the stiffness of the frame.

ETABS corrects for such effects in the formulation of the member stiffness, unlike most general-purpose programs that work on centreline- to-centreline dimensions.

The results produced by the programs should be in a form directly usable by the engineer. General-purpose computer programs produce results in a general form that may need additional processing before they are usable in structural design.

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### IV. MODELLING AND SEISMIC ANALYSIS OF EXISTING BUILDING BEFORE RETROFITTING (MODEL 1)

Here a G+5 storey building that is Existing Building before Retrofitting modelled by using ETABS 9 software. The detailed features of the normal building are given below.

#### FEATURES OF BUILDING

Structure Building)	=	Before retrofitting of the building (Model 1)
Floors	=	PL+SC+C+G + 5
Bays in X-direction	=	1
Bays in Y-direction	=	12
Bay width in X-direction	=	19.88 m
Bay width in Y-direction	=	The bay width of building in y-direction with Varying in the length
Live load on slab	=	4.0 kN/m <sup>2</sup> (all floors except terrace floor)
= 2.0 kN/m <sup>2</sup> (terrace floor)		
Dead load on slab	=	2.0 kN/m <sup>2</sup> (all floors except terrace floor)
= 1.5 kN/m <sup>2</sup> (terrace floor)		
Floor Finish	=	1.0 kN/m <sup>2</sup>
Water proofing	=	1.0 kN/m <sup>2</sup>
Storey height	=	3 m
Wall Thickness	=	0.25 m (exterior wall)
= 0.15 m (interior wall)		
Thickness of Slab	=	0.15 m
Grade of concrete	=	M25 (For beams)
	=	M30 (For columns)
Grade of steel	=	Fe415

Showing the 3-D view of a Model 1 building

Showing the elevation view in XZ view of a Model 1 building

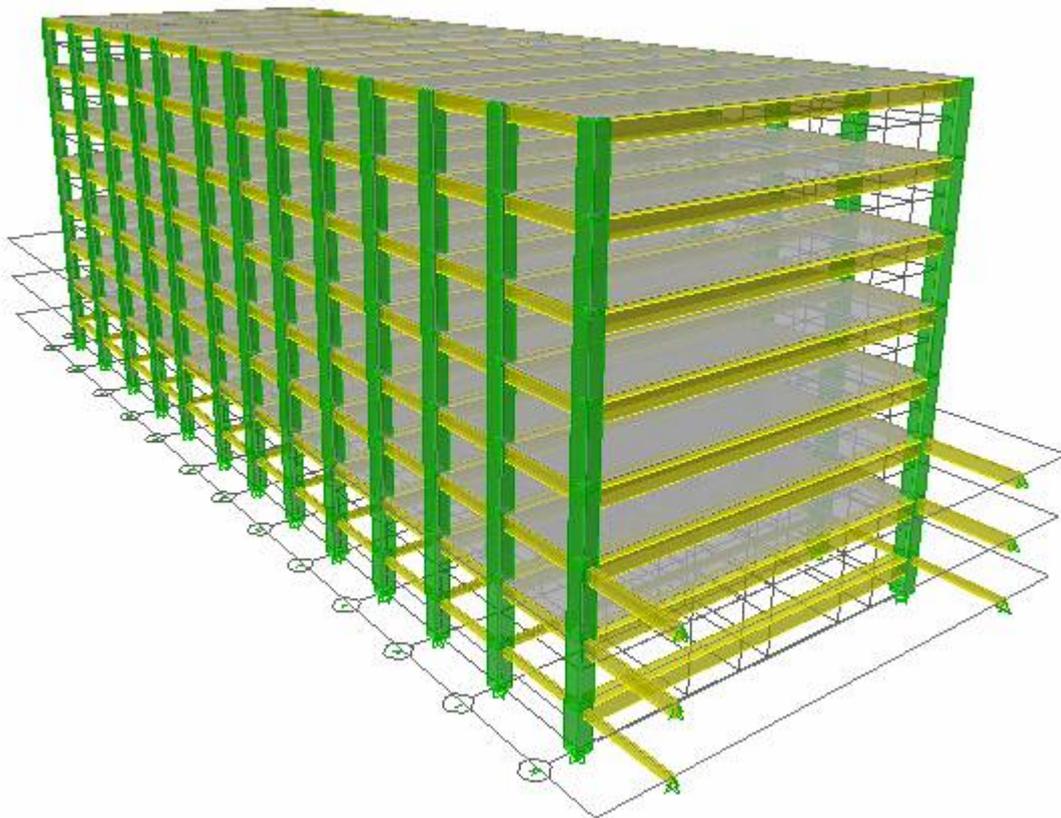
#### DIMENSIONS OF BEAMS AND COLUMNS

For a Model 1 building by applying the dead load and live load the structure will be modelled for minimum sizes of beams and columns. After the application of earthquake forces in X and Y directions the structure will be unsafe with the available sizes of beams and columns. So for that purpose the sizes of beams and columns are increased to withstand the lateral forces applied by the earthquake.

#### SO FOR THE MODEL 1 BUILDING DIMENSIONS OF BEAMS AND COLUMNS ARE AS FOLLOWS

Size of beam-I	=	0.23 m X 0.6 m,
Size of beam-II	=	0.6m X 0.75m,
Size of beams-III	=	0.75m X 0.75m.
Size of column-I	=	0.30m X 0.70 m,
Size of column-II	=	0.6m X 0.6m,
Size of column-III	=	0.75m X 0.75m.

#### SEISMIC WEIGHT OF BUILDING (MODEL 1)



For a building with edge columns the seismic weight of building is calculated on the basis of the total dead load of structure and the live loads applied. As per the Indian Standard code 1893 that is earthquake code of clause 7.4 states that the seismic weight of building is equal to the sum of full dead load and appropriate amount of imposed load as specified in of IS 875(Part 2).

As per Code IS 875 it states that if the imposed load that is live load on a floor is less than 3 kN/m<sup>2</sup> then the imposed load is taken as 25 % of imposed load, if the imposed load on a floor is greater than 3 kN/m<sup>2</sup> then the imposed loads is taken as 50 % of imposed loads. For our building live load on floor is 4 kN/m<sup>2</sup> that is imposed load is greater than 3 kN/m<sup>2</sup> , So we take imposed load as 50% of applied live load

## V. CALCULATION

Dead load calculation

Roof (Terrace)

Mass of slab = ((thickness of slab \* unit weight of concrete) + Floor finishes + Water proofing)\*(length of slab in X direction)\*(length of slab in Y direction)

$$\text{Mass of slab} = ((0.15 * 25) + 1 + 1) * 19.88 * 58.81 = 6722.57 \text{ KN} \quad (4.1)$$

Mass of beams = (Depth \* Width \* Unit weight of concrete \* Length of beam \* No. of beams)

$$\text{Mass of main beams} = (0.75 * 0.75 * 25 * 19.88 * 7) = 1956.93 \text{ KN} \quad (4.2) \text{ (i)}$$

$$\text{Mass of secondary beams} = (0.60 * 0.75 * 25 * (15.23 * 4 + 4.65 * 1))$$

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$$\begin{aligned} &= 737.6625 \text{ KN} && (4.2) \text{ (ii)} \\ \text{Mass of other beams} &= (0.23 \times 0.60 \times 25 \times ((58.81 \times 2) + (19.88 \times 2) + (5.64 \times 3) + \\ &\quad + (4.65 \times 3)) + 26.38 + 1.98 + 1.98 + 2.288 \\ &= 713.9016 \text{ KN} && (4.2) \text{ (iii)} \end{aligned}$$

$$\text{Total} = 3408.5016 \text{ KN} \quad (4.2)$$

Mass of Parapet wall = (Width \* Height \* Unit weight of brick \* length of wall \* No. of sides)

$$\begin{aligned} \text{Mass of parapet wall} &= (0.25 \times 1.2 \times 20 \times (58.81 \times 2 + 19.88 \times 2)) \\ &= 944.28 \text{ KN} && (4.3) \end{aligned}$$

Remaining Floors

Mass of slab = ((thickness of slab \* unit weight of concrete) + Floor finishes + Water proofing)\*(length of slab in X direction)\*(length of slab in Y direction)

$$\begin{aligned} \text{Mass of slab} &= ((0.15 \times 25) + 1 + 1) \times 19.88 \times 58.81 \\ &= 6722.57 \text{ KN} && (4.4) \end{aligned}$$

Mass of beams = (Depth \* Width \* Unit weight of concrete \* Length of beam \* No. of beams)

$$\begin{aligned} \text{Mass of beam-I} &= (0.75 \times 0.75 \times 25 \times 19.88 \times 7) \\ &= 1956.93 \text{ KN} && (4.5) \text{ (i)} \end{aligned}$$

$$\begin{aligned} \text{Mass of beam -II} &= (0.60 \times 0.75 \times 25 \times (15.23 \times 4 + 4.65 \times 1)) \\ &= 737.6625 \text{ KN} && (4.5) \text{ (ii)} \end{aligned}$$

$$\begin{aligned} \text{Mass of beam -III} &= (0.23 \times 0.60 \times 25 \times ((58.81 \times 2) + (19.88 \times 2) + (5.64 \times 3) + \\ &\quad + (4.65 \times 3)) + 26.38 + 1.98 + 1.98 + 2.288 \\ &= 713.9016 \text{ KN} && (4.5) \text{ (iii)} \end{aligned}$$

Mass of Exterior wall = (Width \* Height \* Unit weight of brick \* length of wall \* No. of sides)

$$\begin{aligned} \text{Mass of Exterior wall} &= (0.25 \times 3.1 \times 20 \times (19.88 \times 2 + 58.81 \times 2)) \\ &= 1550 \text{ kN} && (4.6) \end{aligned}$$

Mass of Interior wall = (Width \* Height \* Unit weight of brick \* length of wall \* No. of sides)

$$\begin{aligned} \text{Mass of Interior wall} &= (0.15 \times 3.1 \times 20 \times (19.88 \times 2 + 58.81 \times 2)) \\ &= 1860 \text{ kN} && (4.7) \end{aligned}$$

Columns

Mass of columns = (Width \* Depth \* Unit weight of concrete \* Height of columns \* No. of columns in each floor)

$$\begin{aligned} \text{Mass of column -I} &= (0.75 \times 0.75 \times 30 \times 3.1 \times 14) \\ &= 732.375 \text{ KN} && (4.8) \text{ (i)} \end{aligned}$$

$$\begin{aligned} \text{Mass of column -II} &= (0.60 \times 0.60 \times 30 \times 3.1 \times 14) \\ &= 468.72 \text{ KN} && (4.8) \text{ (ii)} \end{aligned}$$

$$\begin{aligned} \text{Mass of column-III} &= (0.30 \times 0.70 \times 30 \times 3.1 \times 14) \\ &= 273.42 \text{ KN} && (4.8) \text{ (iii)} \end{aligned}$$

$$\text{Total} = 1474.515 \text{ KN} \quad (4.8)$$

Live load calculation

Roof (Terrace)

Mass of slab = (Given live load \* Length of slab in X-Direction \* Length of slab in Y- direction)

$$\begin{aligned} \text{Mass of slab} &= (1.5 \times 19.88 \times 58.81) \\ &= 1753.7142 \text{ kN} && (4.9) \end{aligned}$$

$$\begin{aligned} \text{Seismic mass of slab} &= 25 \% (\text{mass of slab}) \\ &= (0.25 \times 1753.7142) = 438.428 \text{ kN} && (4.10) \end{aligned}$$

Remaining Floors

Mass of slab = (Given live load \* Length of slab in X-Direction \* Length of slab in Y- direction)

$$\begin{aligned} \text{Mass of slab} &= (4 \times 19.88 \times 58.81) \\ &= 4676.5712 \text{ kN} && (4.11) \end{aligned}$$

$$\begin{aligned} \text{Seismic mass of slab} &= 50 \% (\text{mass of slab}) \\ &= (0.50 \times 4676.5712) = 2338.2856 \text{ kN} && (4.12) \end{aligned}$$

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Total loads on Roof (Terrace)

$$\begin{aligned} \text{Total loads on roof (W1)} &= (\text{Dead load of slab} + \text{Mass of beams} + \text{Mass of parapet wall} + \text{Live load of slab}) \quad \text{W1} = \\ &= (3.1) + (3.2) + (3.3) + (3.10) \\ &= 6722.57 + 1956.93 + 737.66 + 713.901 \\ \text{W1} &= 10131.017 \text{ kN} \quad (4.13) \end{aligned}$$

Total loads on Remaining each floor (Except terrace) Total loads on remaining floors (W2) = (Dead load of slab + Mass of beams + Mass of exterior wall + Mass of Interior wall + Live load on slab + column weight in single floor)

$$\begin{aligned} \text{W2} &= (4.4) + (4.5) + (4.6) + (4.7) + (4.12) + (4.8) \\ &= 6722.5711 + 3408.5016 + 2436.39 + 1463.934 + \\ &\quad + 1474.515 + 2338.2856 \end{aligned}$$

$$\text{W2} = 17844.1973 \text{ kN} \quad (4.14)$$

Total loads on all Remaining floors (Except terrace)

$$\text{Total number of remaining floors except terrace} = 5$$

$$\begin{aligned} \text{Total loads on all remaining floors (W3)} &= (\text{W2}) * (5) \\ &= (17844.1973) * (5) \\ &= 89220.9865 \text{ kN} \quad (4.15) \end{aligned}$$

Total seismic weight of Existing building

$$\begin{aligned} \text{Total Seismic weight of building} &= (\text{W1}) + (\text{W3}) \\ &= (3.13) + (3.15) \\ &= (10131.0176) + (89220.9865) \\ &= 99352.0041 \text{ kN} \end{aligned}$$

## VI. BASE SHEAR CALCULATION FOR EXISTING BUILDING

Equivalent Lateral force method is used to know the structure behaviour by applying the lateral loads acting at each story in X and Y directions manually. These lateral loads are calculated by considering the various parameters like the Response reduction factor (R), Zone factor (Z), Importance factor (I), Horizontal acceleration coefficient (Ah), Structural response factor (Sa/g) and Total seismic weight of building (W) as per the IS code 1893-2002.

Calculation of Lateral forces at each floor for a Model 1 building

$$\begin{aligned} \text{Weight in Terrace} & \\ \text{Weight of Slab (DL+LL)} &= 8476.28 \text{ KN} \\ \text{Weight of beams} &= 3408 \text{ KN} \\ \text{Total} &= 11884.7816 \text{ KN} \end{aligned}$$

Weight in Remaining Floors

$$\begin{aligned} \text{Weight of Slab (DL+LL)} &= 11399.1412 \text{ KN} \\ \text{Weight of beams} &= 3408.5016 \text{ KN} \\ \text{Weight of Columns} &= 1474.515 \text{ KN} \\ \text{Total} &= 16282.1578 \text{ KN} \end{aligned}$$

Considering zone as zone III with value 0.16, importance factor as 1, Response reduction factor as specially moment resisting frame as 5 from table 7 of IS 1893, spectral acceleration coefficient from clause 6.4.5 of IS 1893.

$$\begin{aligned} \text{Fundamental natural time period} &= 0.075 * h^{0.75} \\ &= 0.075 * 24.2^{0.75} \\ &= 0.8183 \text{ sec} \end{aligned}$$

From time period value by interpolating in IS 1893 of clause 6.4.5 we get Sa/g as 1.85

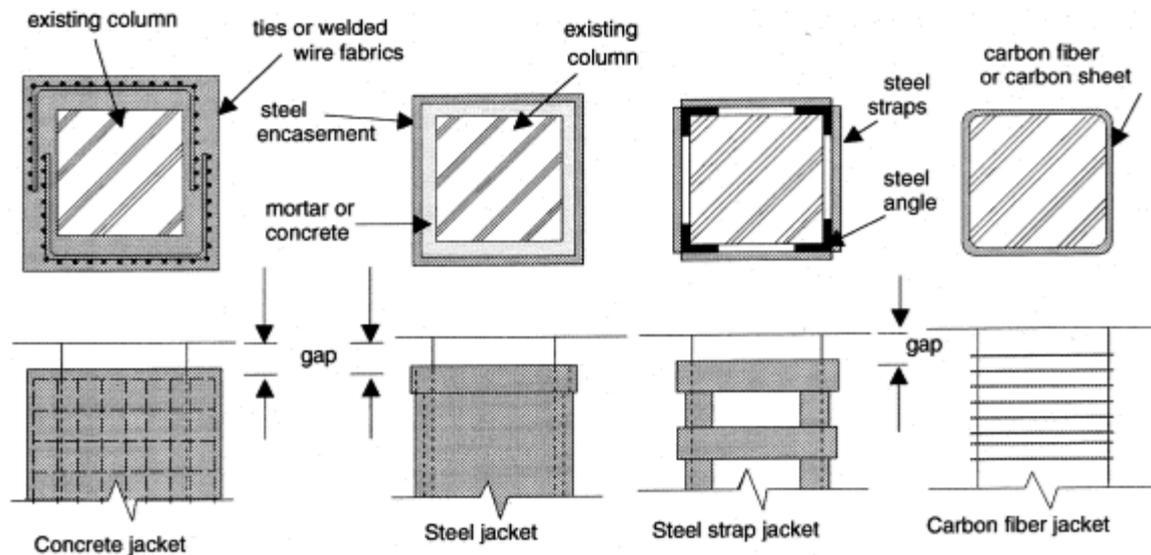
$$\begin{aligned} \text{Ah} &= (Z/2) * (I/R) * (\text{Sa/g}) \\ \text{Ah} &= (0.36/2) * (1/5) * (1.85) \\ &= 0.01583 \end{aligned}$$

$$\begin{aligned} \text{Base Shear of Building} &= A_h * W && (\text{W from equation 4.16}) \\ &= 0.01583 * 99352.0041 \\ &= 1572.7422 \text{ Kn} \end{aligned}$$

### VII. JACKETING OF COLUMNS

Jacketing of columns consists of added concrete with longitudinal and transverse reinforcement around the existing columns. This type of strengthening improves the axial and shear strength of columns while the flexural strength of column and strength of the beam-column joints remain the same. It is also observed that the jacketing of columns is not successful for improving the ductility. A major advantage of column jacketing is that it improves the lateral load capacity of the building in a reasonably uniform and distributed way and hence avoiding the concentration of stiffness as in the case of shear walls. This is how major strengthening of foundations may be avoided. In addition the original function of the building can be maintained, as there are no major changes in the original geometry of the building with this technique. The jacketing of columns is generally carried out by two methods:

- (i) reinforced concrete jacketing
- (ii) Steel jacketing



#### REINFORCED CONCRETE JACKETING:

Reinforced concrete jacketing can be employed as a repair or strengthening scheme. Damaged regions of the existing members should be repaired prior to their jacketing. There are two main purposes of jacketing of columns: Increase in the shear capacity of columns in order to accomplish a strong column-weak beam design and To improve the column's flexural strength by the longitudinal steel of the jacket made continuous through the slab system are anchored with the foundation. It is achieved by passing the new longitudinal reinforcement through holes drilled in the slab and by placing new concrete in the beam column joints as illustrated in figure 1. Rehabilitated sections are designed in this way so that the flexural strength of columns should be greater than that of the beams.

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## STEEL JACKETING

Local strengthening of columns has been frequently accomplished by jacketing with steel plates. A general feature of steel jacketing is mentioned in Table 2.

Tables of Details of Steel Jacketing

Steel plate thickness	At least 6mm
Height of jacket	1.2 to 1.5 times splice length in case of flexural columns. Full height of column in case of shear columns.
Free ends of jackets bottom clearance	Welded throughout the height of jacket, size of weld <sup>1</sup> 38 mm (1.5 inch), steel jacket may be terminated above the top of footing to avoid any possible bearing of the steel jacket against the footing, to avoid local damage to the jacket and/or an undesirable or unintended increase in flexural capacity.

## VIII.CONCLUSION

After an extensive bibliographical study and the revision of several Indian design projects for the retrofit of structures, it can be concluded that there are some outlines that help, not only in a qualitative but in a quantitative manner, the design of a retrofit scheme of a structure by means of concrete jacketing.

Although these guidelines can give a rational basis for a practical design, research still need to address critical aspects in the behaviour of jacketed elements .The change in behaviour in jacketed elements whose shear span/death ratios are significantly reduced, due to their jacketing, needs to be clarified.

In low to moderate seismicity regions, reinforced concrete buildings are designed to resist dead and live loads only. This result in joint cores with reinforcement detail that cannot fulfil seismic design criteria, thus making such building frames vulnerable to joint shear failure when subjected to lateral loading or ground excitation. Even in seismic regions, old buildings designed according to the then existing immature seismic design codes lack sufficient hoops inside the joint cores. The joint cores are the most critical components in such frames, and the ultimate failure of such frames under lateral loading would be due to the inadequate shear capacity of the joint core. Hence, such lightly reinforced joints need to be strengthened before exposing them to any form of lateral loading. Reinforced concrete (RC) jacketing is an effective method of retrofitting such connections. In this paper, the usefulness of RC jacketing technique to strengthen lightly reinforced beam-column joints is investigated experimentally.

A full-scale lightly reinforced concrete beam-column sub-assembly was strengthened by casting an RC jacket outside the column and the joint, and the improvement brought over by the retrofitting technique in the cyclic response of the specimen was verified experimentally. The joint of the original specimen was not adequately reinforced to fulfil seismic design requirements, and it was the weakest component of the sub-assembly. When subjected to cyclic lateral loading, the joint panel hence experienced severe damage due to excessive shear deformation while the beam and column remained virtually undamaged. The original specimen was vulnerable to joint shear failure. On the other hand, the retrofitted specimen failed after the formation of a plastic hinge in the beam, and the joint was no longer the weakest component of the sub-assembly. Apart from the increase in the capacity and deformability, the shear deformation of the joint panel reduced significantly after retrofitting. It is concluded that the RC jacketing method is effective in strengthening non-seismic RC frames with inadequately reinforced joints.

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