"SEISMIC UPGRADATION OF A G+III STOREYED EXISTING RESIDENTIAL BUILDING LOCATED IN JAMMU"

Thesis Report submitted in partial fulfillment of requirement for the degree of

Master of Technology

In

Structural Engineering

under the Supervision of

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By

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Certificate

This is to certify that project report entitled "Seismic Upgradation of A G+III Storeyed Existing Residential Building Located in Jammu", submitted by Nikita Gupta in partial fulfilment for the award of degree of Master of Technology in Structural Engineering to Jaypee University of Information Technology, Waknaghat, Solan has been carried out under my supervision.

This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

Date: **Signature:**

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Abstract

Past earthquakes in India have revealed that majority of the buildings are not designed Earthquake Resistant. Generally, buildings are designed taking into account only the gravity loads. Also, the current design seismic codes are not fully practiced while designing a building. Hence, a higher degree of damage is expected during an earthquake if the seismic resistance of the building is inadequate. Thus, for these types of buildings, Retrofitting gives out the best possible way in order to minimize the future damage caused due to vibrations produced by an earthquake. This thesis describes the seismic retrofitting of an existing G+III residential building located in Jammu. Since the building lies in J&K which is under severe seismic zone i.e. Zone IV and V, it needs to be designed seismically resistant. Static and dynamic analysis of the building is performed and the required and provided area of steel of all the structural members are compared and checked. Retrofitting is being done for the deficient columns of the building. The step by step retrofitting techniques are incorporated in this. The existing building is retrofitted using various techniques like jacketting of columns and addition of shear walls. Jacketting methods include RC jacketting, Steel plate Jacketting and FRP Jacketting. All these jacketting techniques are compared based on the parameters of safety, economy and onsite-feasibility. FRP jacketing, being light in weight, does not increase seismic weight of building but it improves lateral strength considerably. Steel jacketing, has a comparable cost with RC jacketing but giving better results. It is lighter in weight too. Hence, FRP and steel jacketing prove to be the best technique for retrofitting of weak concrete columns. FRP and steel jacketting provides more protection whereas RC jacketing is more economical.

CHAPTER-1

INTRODUCTION

1.1 General

In India, about 50-60% of the total area is vulnerable to the seismic activity. Thus, the knowledge of earthquake resistant building of structures has become a need. Jammu and Kashmir lies in zone IV and V respectively. Considering the past Jammu & Kashmir Earthquake of 2005, a strong need for the retrofitting of the existing buildings has been felt. The probable reasons may be as follows:

1. Buildings have not been designed and detailed to resist seismic forces.
2. Buildings may have designed for seismic forces, before the publication of current design seismic codes.
3. The lateral strength of the building does not satisfy the seismic forces as per the revised seismic zones or designed base shear.
4. Construction is apparently of poor quality.
5. There have been additions of change of use of building with increased vulnerability.

There are various methods of retrofitting. [12] Retrofit specifically aims to enhance the structural capacities (strength, stiffness, ductility, stability and integrity) of a building that is found to be deficient or vulnerable. There is no need to demolish a building to improve its lateral resistance if the damage or deterioration is less. The retrofit is intended to mitigate the effect of s future earthquake. There are two techniques of retrofitting: Global and the local techniques. The global retrofitting technique targets the seismic resistance of the building. It includes adding of infill wall, adding of shear wall, adding of steel bracings and base isolation. Adding of infill wall in the ground storey is a viable option to retrofit buildings with soft storey. Shear walls can be introduced in a building with flat slabs or flat plates. A new shear wall should be provided with an adequate foundation. Steel braces can be inserted in frames to provide lateral strength, stiffness, ductility, and energy dissipation. These can be provided in the exterior frames with least disruption of the building use. Local retrofitting technique targets the seismic resistance of a member, without significantly affecting the overall resistance of the building. The local retrofit technique includes the concrete, steel or Fibre reinforced polymer jacketting to the structural members like beams, columns, beam column joint, foundation. Concrete jacketing involves adding a new layer of concrete with longitudional reinforcement and closely spaced ties. The jacket increases both the flexural strength and the shear strength of the beam or the column.

The following are the advantages of retrofitting:

1. It increases the seismic resistance of the building.

2. It increases the ductile behaviour of the structure.

3. It increases the lateral load capability of the building.

4. Strength and stiffness of the building is improved.

1.2 Objective

In Jammu, most of the residential buildings have been designed only for dead and live loads. People are not aware of the seismic design of buildings. But, since Jammu (J&K) lies in zone IV, the buildings located in this state needs to be seismic resistant. In this thesis, an existing building has been undertaken for retrofitting. Consider a G+III storeyed residential existing building. It is a framed structure with total three stories above ground level. The ground level is an open storey being utilized as parking. It thus makes up a soft storey. On the roof, there is a water tank too. This building has already been designed for the dead and the live loads only. Thus, two main problems are identified in this building with respect to the seismicity of the building. Firstly, the building has not incorporated in it the Earthquake loads. Secondly, no provisions have been made up for the existing soft storey. So, ground storey needs to be given special attention.

CHAPTER-2

LITERATURE REVIEW

Retrofit specifically aims to enhance the structural capacities (strength, stiffness, ductility, stability and integrity) of a building that is found to be deficient or vulnerable. Till now an extensive research in this field has been conducted. Some of the previous work done in the study of retrofit of existing structures has been concluded below.

[1]The paper titled, "Seismic retrofitting technique using fibre composites" authored by Abhijit Mukherjee and Mangesh V. Joshi and is published in The Indian Concrete Journal, December, 2001. In this paper a novel technique for repair and retrofitting of structures with emphasis on earthquake is described using fibre composites. This technique has been successfully applied in the earthquake-affected Gujarat; it introduces high strength non-metallic fibres along with polymeric resins in repair. The author has described some conventional methods that were used like: section enlargement, polymer modified concrete filling and polymer grouting. The methods which involve concrete in strengthening are:

1. Time consuming, dusty and laborious.

2. Require a long time to implement, a longer period of evacuation.

3. They also increase the dead load on the structure.

These techniques often apply steel reinforcements that remain exposed to environmental attack. Thus, a new material known as FRC (fibre reinforced composites) was invented. FRPC are used in the form of plates, sheets and bars. A complete method of applying FRC has been elaborated: glass fibre, resins (optimum viscosity), preparation of substrate, fibre sheet wrapping. Also, a brief of strengthening of beam and beam column joint (Figure 1) has been discussed. Strengthening of beams has been categorised into Flexural strengthening (Flexural strengthening of beams and slabs is necessary when the tension steel has yielded or it has deteriorated due to corrosion) and Shear strengthening.

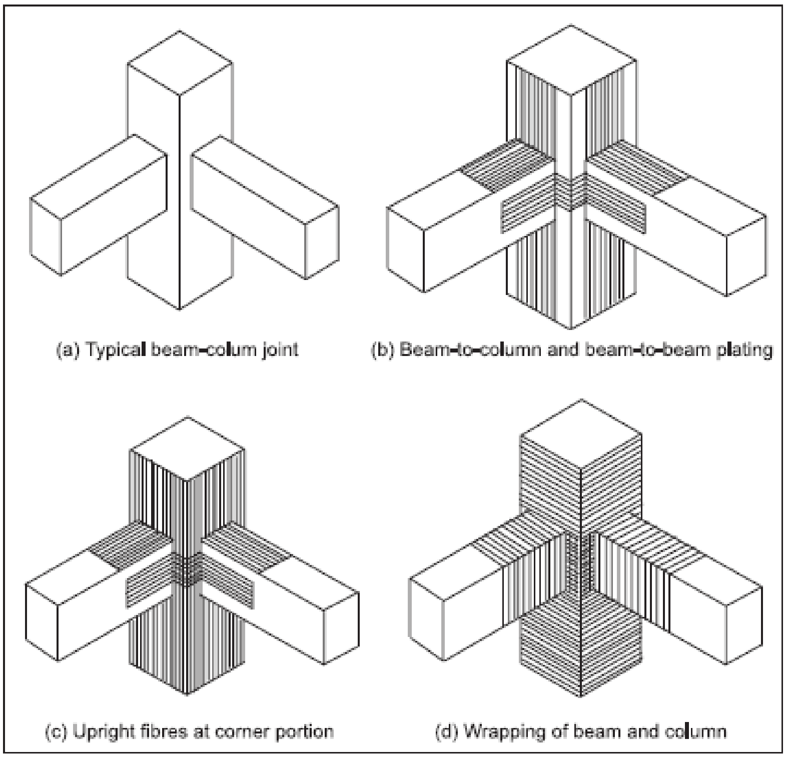


Figure 1: Strengthening procedure in beam column joint.

[2]A paper titled, "seismic analysis and retrofit of existing multi-storeyed building-An overview with a case study", written by Amlan K SENGUPTA, CHEMURU S. Reddy, Badari Narayanan and Asokan A, published in the 13th World Conference on Earthquake Engineering, Vancouver, B.C. , Canada , August 1-6, 2004. The paper reviews to assess the seismic vulnerability of reinforced concrete three to ten storeyed, residential and commercial buildings and to propose retrofit measures for the structurally deficient buildings particularly those located in the urban areas of Earthquake zones V, IV, III. A case study has been undertaken considering a four storeyed residential, ordinary moment resisting RC framed building, located in zone III, along with the use of M20 concrete grade and steel grade Fe 415. Then, preliminary and detailed analysis was done. It was confirmed that after the comparison of base shears of the building with and without infill, a number of beam sections were found to have deficient flexural capacity. For ground storey, demand exceeds capacities. The ground storey has higher drift. From pushover analysis, the building has larger strength than the factored base shear. But the building did not

reach the target displacement of 172mm. Thus, the building does not have adequate ductility. Thus, two schemes that are economically feasible were presented. Scheme 1, to enhance the capacities of the columns in the ground storey, local retrofitting of the columns by concrete jacketing was adopted. Scheme 2 was to reduce stiffness and hence lateral force-for this, Selected ground storey infill wall made non-integral by drilling gaps at the corners and filling with caulking materials. It enhances strength and ductility.

[3]A Paper titled, “Flexural strengthening of RC beams using precast SIFCON laminates”, by S. Balaji and G.S. Thirugnanam, is published in the Journal of Structural Engineering, Vol. 40, August-September2013. A brief of Slurry infiltrated fibrous concrete (SIFCON) is given. SIFCON is a special type of high performance fibre reinforced concrete composite, fabricated by slurry infiltration technique with high fibre volume content. It is cast using a preplacing technique in which fibres are placed in the mold and infiltered with cement based slurry. An experimental programme was set up and 12 beam specimens of size 100mm x 150mm x 1700mm were casted and tested under cyclic loading. The beams were tested with laminate confinement at the bottom face, side and on three faces as shown in the table 1. A test programme was set up using a two point flexural loading system. All the specimens were tested to cyclic loading. The load was increased and decreased in stages up to the final failure of specimen and at each stage; deflection was noted at the mid span. The results observed were as follows:

1. Load carrying capacity: 1st crack load on three face confinement beam was 21 KN, values for single face and two face were 15KN and 18 KN. Ultimate load carrying capacity of three face confinement was 55KN, values for single and two face were 33 KN and 36 KN.

2. The confinement of the beams with SIFCON laminates on three faces increases the load carrying capacity, ductility nearly twice that of conventional beams.

Table 1: Beams with laminate confinement

SNO.	Beam designation	Laminate confinement	Number of beams
01	RCB	No confinement	03
02	RCB1	Bottom face confinement	03
03	RCB2	Side face confinement	03
04	RCB3	Three face confinement(bottom and side faces)	03

[4]A paper titled, "Seismic retrofit of reinforced concrete buildings-A review and case study", authored by M C Griffith and A V Pinto and published in the 12th World Conference on Earthquake Engineering, 2000. In this paper, specific details of a 4-storey, 3-bay reinforced concrete frame test structure with unreinforced brick masonry infill walls are described along with estimates of its likely weaknesses with regard to seismic loading. The purpose of this study was to investigate possible retrofit options for use in a project using full scale pseudo-dynamic tests at the European Laboratory for Structural Assessment (ELSA) in Italy. The main issues to be addressed by potential seismic retrofit schemes are:

1. The large difference in strength and stiffness between the concrete frame and the brick masonry infill.

2. The strength of the bare concrete frame above level 2 is estimated to be only 60% of the strength of the frame below level 2.

3. The seismic behaviour of the bare concrete frame is likely to be that of a "weak-column, strong-beam" mechanism under ultimate deformation conditions.

4. The stirrup curtailment detail which uses 90° hooks is unlikely to withstand repeated cycle of large deformation.

Three retrofit strategies were employed. Option one was the replacement of unreinforced masonry infill with damped bracing using K-bracing. Option second was the composite jacketing of columns and selected masonry infill. Option third was to retrofit only the frame elements.

[5]A paper titled "Research on the Upgrade of Traditional Seismic Retrofits for Ancient Buddhist Temples in the Region of Spiti and Kinnaur in the Western Himalayas" written by Sandeep Sikka and Charu Chaudhry. The Spiti and Kinnaur region in the northern Indian state of Himachal Pradesh has some remarkable ancient adobe Buddhist temples. These had load bearing walls structures and lies in highly seismic zones. This paper puts forward the results of the study and describes the condition of historic earthen structures in the region after the earthquake of 1975. It describes the traditional seismic retrofits existing in the structures and explains the retrofitting techniques. Some of the flaws that were noticed aftermath of earthquake were: there is tremendous outward movement and deformation in the upper portion of load bearing walls, displacement of masonry not uniform, out of plane movement of walls. Also, because of application of extra mud layers (that were added in response to precipitation and regular rainfall) this led to further outward horizontal movement in the walls and which further led to separation of joints at corners and hence cracks were formed. Vertical ties are either missing or discontinuous. Ceilings with beams and rafters rest directly on load bearing walls without any steel plates and hence, minor and major cracks below ceiling level, thus, ingress of water. Wooden members also affected by the induced humidity and water ingress. Lack of ventilation caused dry and wet rot in these and thus diminishing their structural safety. Various retrofitting methods used: (1) study models for designing roof loads and then the compressive strength of walls. (2) Design of wall plates (3) design of wall ties (4) natural fibre jackets (5) adding diagonal bracing and buttresses.

[6]The paper titled "State of research on seismic retrofit of concrete buildings structures in the US" and written by Jack P. Moehle. This paper describes some of the causes of failure and hence their retrofitting techniques. Typical frame details in pre-1976 buildings in the western US shows the following defects: discontinuous

longitudional reinforcement in beams, transverse reinforcement not proportional to take up shear forces, open stirrups, hoops with 90 degree bends. Failure of columns, beam-column joint, slab-column failures have been observed during the past earthquakes. Two different rehabilitation approaches have been described: The global and the local retrofitting approaches. The global modification includes adding of reinforced concrete wall to reduce overall drifts; pre cast panels for frame infill, wing walls for weak columns and deep spandrels.

[7]The paper titled "Research and application of seismic retrofit technologies in Canada", written by Simon Foo, Nove Naumoski & Moe Chung, proposed various retrofitting techniques starting from friction dampers in 1980s to carbon fibre reinforced plastic in the 1990s to the external pre-stressing in the 2000s. Research has been carried by Filiatrault et al (1986), using a shake table test to investigate the performance of a 3-story steel frame, using friction dampers. Similar test were conducted by Aiken et al (1988), on a 9 –story steel frame. These provided a very satisfactory performance. Further, friction dampers were used in existing buildings of Harry Stevens, East Memorial, and Pump House. In this paper, they also proposed the use of carbon fibre reinforced plastic for the existing buildings of Port Alberni Federal Building. Also, for the non-structural elements, two sets of series were conducted using the shake table testing and also, some concept was guided in context to the floor response spectra and raised floor system.

[8] "Retrofiitng of exixsting RCC buildings by the method of jacketting" written by Bhavar Dadasaheb o. et all. In this paper, A health building in the heart of Nashik city is being strengthened to overcome the future disorders. 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. Some causes of the damage of the buildings were given. These were like: Old decaying buildings predating modern construction practices, New Buildings not being designed to Indian earthquake codes, Lack of knowledge, understanding or training in the use of these codes by local engineers, Unawareness that Gujarat is a highly seismic region, Buildings erected without owners seeking proper engineering advice, Improper detailing of masonry and reinforced structures. Retrofitting of health building at Nasik was described. This building was proposed in 1984 and accordingly

was designed for B + G + 4 storeys. The building during its life span at the end in 2008 was found completely deteriorated and was not capable to sustain further loads. And was predicted that it may fail. The reasons responsible for the failure of columns and roof were mentioned. Though it was planned to construct 6 storey in all, only 2 storey (i. e. Basement + G) structure was constructed in actual when structural details were for 6 storey. The built up area for basement is 1236 sq.mt. And that of the ground floor is 1178 sq. mt. This group had a smooth functioning up to 2004. Then in 2005 the administration of the hospital was changed. New administrative body utilized the same infrastructure up to 2008. The faults that were concluded were as below.

a) The high ground water table in the region.

b) Eccentricity of all the columns was a serious problem observed.

c) Improper techniques that were followed during mixing, placing & compacting the concrete.

d) There were many loose pockets found in the concrete.

e) Sufficient concrete cover at different stages to the steel reinforcement was not maintained.

f) The terrace slabs are provided with unwanted thickness of IPS flooring at the top.

Finally, retrofitting was decided for this building. The steps that were adopted were:

(A) The entire flooring at the basement to be removed & to be provided again with a raft below it. To overcome the uplift pressure that could be generated due to the higher ground water table it was suggested to provide a raft below the basement floor.

(B) All the columns should be jacketed.

(C) The loose pockets in the concrete in the beams are also removed and re-casted in M25 grade.

(D) The unwanted heavy loading of water proofing (IPS flooring) damaged the slab to greater extent.

(E) Five slabs are entirely opened, strengthened with additional reinforcement and are casted with 150 mm thick M25 grade concrete layer.

[9]A paper titled “A survey of methods and techniques used for seismic retrofitting of RC buildings” by Vijayakumar, Venkatesh Babu. In this, Pushover analysis of the structures (Lakshmanan D, 2006) using SAP 2000 has been highlighted for evaluating

the various repair strategies. Case studies were conducted on 150 year old buildings of Ganga Mahal located in Assi Ghat and Sanskrit University of kasha. Also, this paper concludes to find out the best efficient method for retrofit of an open existing ground storey RC frame. Three methods were used for the retrofit (1) concrete jacketing of columns in the ground storey (2) RC structural walls in the ground storey (3) Brick masonry infill in the ground story. Out of all these methods of retrofit, use of RC structural walls in the ground storey gave the maximum strength and ductility. Studies were also conducted for two more buildings which were damaged during Bhuj earthquake 2001: (a) open ground story with infills (b) partial open ground storey with infills. Various retrofitting techniques were used. Combination of column jackettting in ground storey and shear wall throughout the height of the building with proper strengthening of upper stories gave the most economical and desirable performance.

CHAPTER-3

BUILDING DATA

3.1 Introduction

This chapter includes the building data taken for my thesis project. I have taken up a G+III storey existing residential building located in Jammu. This is a framed structure with masonry infill walls. This unit consists of the building plan and the elevation view. Further, it also consists of the sizes and the reinforcements of various slabs, beams, columns and foundation provided.

3.2 Building Data

In my thesis project, I have taken up a G+III storey existing residential building located in Jammu. This is a framed structure with masonry infill walls. It contains an open ground storey referred to as soft storey. The roof consists of an overhead water tank with a capacity of 2700 LTS. The details of the building are as follows:

The total area of the land = 150.037 m^2

The ground coverage = 73.458 m^2

Total covered area = 293.832 m^2

Parking space =1 NO

Stair head room area= 13.44 m^2

Height of the building= 12.3 m

3.2.1 Framed system

The data for the drawings of the building is given in appendix A.

3.2.1.1 Column

The sizes and the reinforcements of each column are given in the table number 2. In total, there are 12 columns.

Table 2: Size of columns

Column no	Foundation to 1st level		1st to 3rd fl level		Above 3rd fl lvl	
	Size	Reinforce-ment	Size	reinforcement	Size	Reinforce-ment
C1,C4,C9	250X300	4-16ø +4-12ø 8 ø @150 c/c	250X300	4-16ø 8 ø @150 c/c	250X300	4-16ø 8 ø @150 c/c
C2,C3,C5,C8,C10,C11,C12	250X350	8-16ø 8 ø @150 c/c	250X350	8-12ø 8 ø @150 c/c	250X350	8-12ø 8 ø @150 c/c
C6,C7	250X400	8-16ø 8 ø@150 c/c	250X400	8-12ø 8 ø @150 c/c	250X400	8-12ø 8 ø @150 c/c

3.2.1.2 Beam

The beams are categorised into 4 types: B1, B2, B3, and B4. The sizes and the reinforcements of each are given in the table number 3.

Table 3: Size of the beams

Beam Type	**Beam Size**	**Mid span**			**Support**		
		top	**bottom**	**stirrup**	**top**	**bottom**	**stirrup**
B1	250x350	3-16ø	2-16ø	8ø@200c/c	3-16ø	2-16ø	8ø@120c/c
B2	250x350	2-16ø	2-16ø	8ø@200c/c	2-16ø+ 2-16ø	2-16ø	8ø@120c/c
B3	250x350	2-20ø	3-16ø	8ø@200c/c	2-20ø+ 2-16ø	3-16ø	8ø@120c/c
B4	250x300	2-12ø	2-16ø	8ø@100c/c	2-12ø	2-16ø	8ø@100c/c

3.2.1.3 Slab

The sizes and the reinforcements of the slab provided are given in the table number 4.

Table 4: Size of the slab

PANEL TYPE	SLAB THICKNESS	SHORTER DIRECTION		LONGER DIRECTION	
		TOP	BOTTOM	TOP	BOTTOM
S0	100MM	8 ø @200c/c	8 ø @200 c/c	8 ø @200 c/c	8 ø @200 c/c

3.2.1.4 Foundation

The sizes and the reinforcements of the foundation provided are given in the table number 5.

Table 5: Foundation data

Foundation Type	Foot Size		Col Position	Fd Depth		Pedestal Size: Lxbxd	Reinforcement	
	L	B		H	H		Shorter	longer
F1	2000	2000	C4,C9	250	500	450X400X150	12ø@150 c/c	12ø@150 c/c
F2	2200	2200	C1	250	500	450X400X150	12ø@130 c/c	12ø@130 c/c
F3	2200	2200	C2,C3	250	500	500X400X150	12ø@130 c/c	12ø@130 c/c
F4	2400	2400	C10,C11,C12	250	550	500X400X150	12ø@120 c/c	12ø@120 c/c
F5	2600	2500	C6	300	550	500X400X150	12ø@100 c/c	12ø@100 c/c
F6	2600	2500	C5,C8	300	550	550X400X150	12ø@100 c/c	12ø@100 c/c
F7	2700	2600	C7	300	550	550X400X150	12ø@100 c/c	12ø@100 c/c

3.2.2 Walls and other information

Main infill walls are 9”thick .These include the external walls and the parapet walls. However, the internal walls are 4.5” thick.

Grade of concrete = M20

Grade of steel = Fe415

Water tank = 1.2 m

CHAPTER-4

STRUCTURAL ANALYSIS

4.1 Introduction

In this chapter, a brief of the problems identified by observing the building plan are given. Also, the static and the dynamic analysis are done. After doing the analysis, the output file is considered for comparing the reinforcements of the structural members.

4.2 Problems identified by observing the building plan

4.2.1 Soft Storey

It is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. In my thesis project, the ground storey consists of columns and beams with no infill walls. It contains the parking area and is considered as a soft storey. There should be special provisions made for the soft storey as far as the seismic loads are concerned. As per [10] IS 1893:2002, Dynamic analysis of building is carried out including the strength and stiffness effects of infill and inelastic deformations in the members, particularly, those in the soft storey, and the members designed accordingly. Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storey:

a) The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads specified in the other relevant clauses.
b) Besides the columns designed and detailed for the calculated storey shears and moments ,shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible; to be designed exclusively for 1.5 times the lateral storey shear force calculated as before.

4.2.2 Water Tank

As shown in the elevation view, there is a water tank over the mumty .It may create a problem during the earthquake waves. [15]In the past earthquakes, it has been identified that the reinforced concrete water tanks under lateral earthquake loads have been extremely susceptible, so that in some cases, these tanks had suffered collapse. During the earthquake, water in the tank get vibrates. Due to this vibration water exerts impulsive & convective hydrodynamic pressure on the tank wall and the tank base in addition to the hydrostatic pressure. The effect of impulsive & convective hydrodynamic pressure should consider in the analysis of tanks. For small capacity tanks, the impulsive pressure is always greater than the convective pressure, but it is vice-versa for tanks with large capacity. Magnitudes of both the pressure are different. The effect of water sloshing must be considered in the analysis. Free board to be provided in the tank may be based on maximum value of sloshing wave height. If sufficient free board is not provided, roof structure should be designed to resist the uplift pressure due to sloshing of water.

4.3 Analysis of existing frame.

The building is then analysed using STAAD PRO V8i software (Figure 2).Both the static and the dynamic analysis have been done for this building. Firstly, Dead and the live loads are found out separately. The unit weights of the material are taken from IS 875: Part 1. The live loads of residential building, accessible and inaccessible roof is taken from [11] IS 875 Part 2. Then, the wind loads are calculated taking the wind speed of Jammu as 39 metre per second and the constants of wind proportionality as unity. Also, the seismic loads are taken out taking the help from IS 1893:2002. Analysis and the design of the concrete building are then carried out.

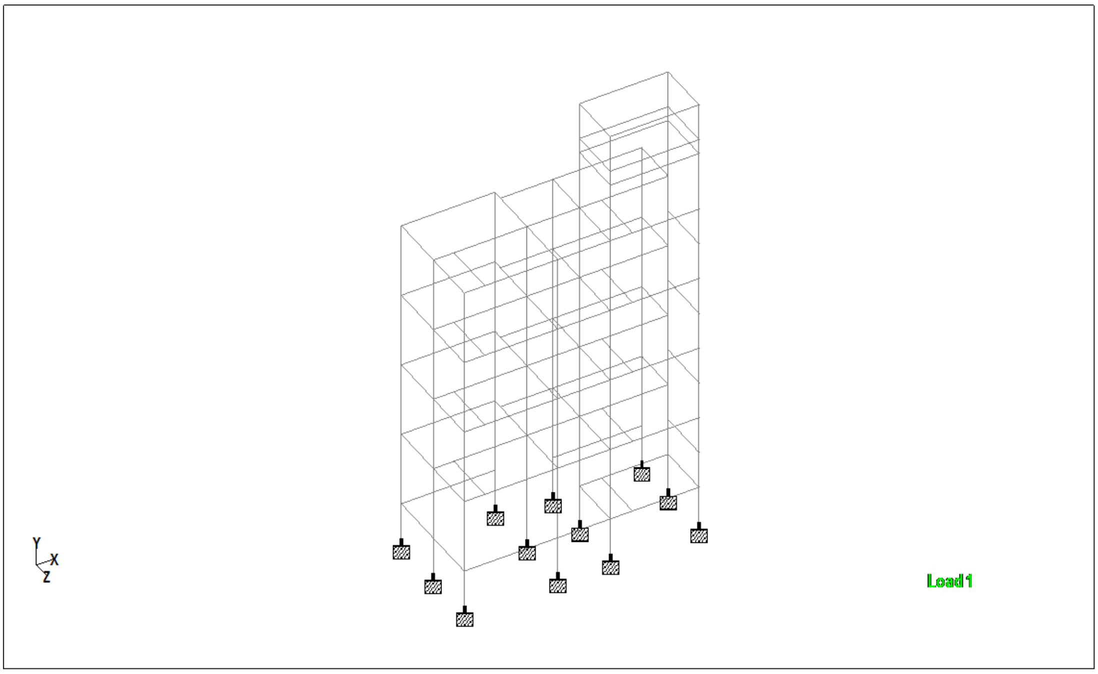


Figure 2: 3D Model of the building

4.4 Problems identified after the analysis

After performing the analysis of the structure, the output file was considered for producing the results. The provided and the required reinforcements are compared for each of the structural member. That is for the columns, beams, slab, and the foundation.

4.4.1 Column

Many columns were there in which the in which required reinforcement was more than the provided reinforcement (Table 6). The yellow colour highlight the table shows the major shortfall of area of steel where as the green colour shows the minor shortfall. For example: column number 2007 in the 2nd floor level (figure 3), the required reinforcement came out to be 2326.61mm^2 However, the provided was 1608mm^2. So there was a shortfall of reinforcement in this column. Likewise if we talk about column number 3004 (figure 4), the required reinforcement was 1304.9 mm^2 and the provided one was 804 mm^2

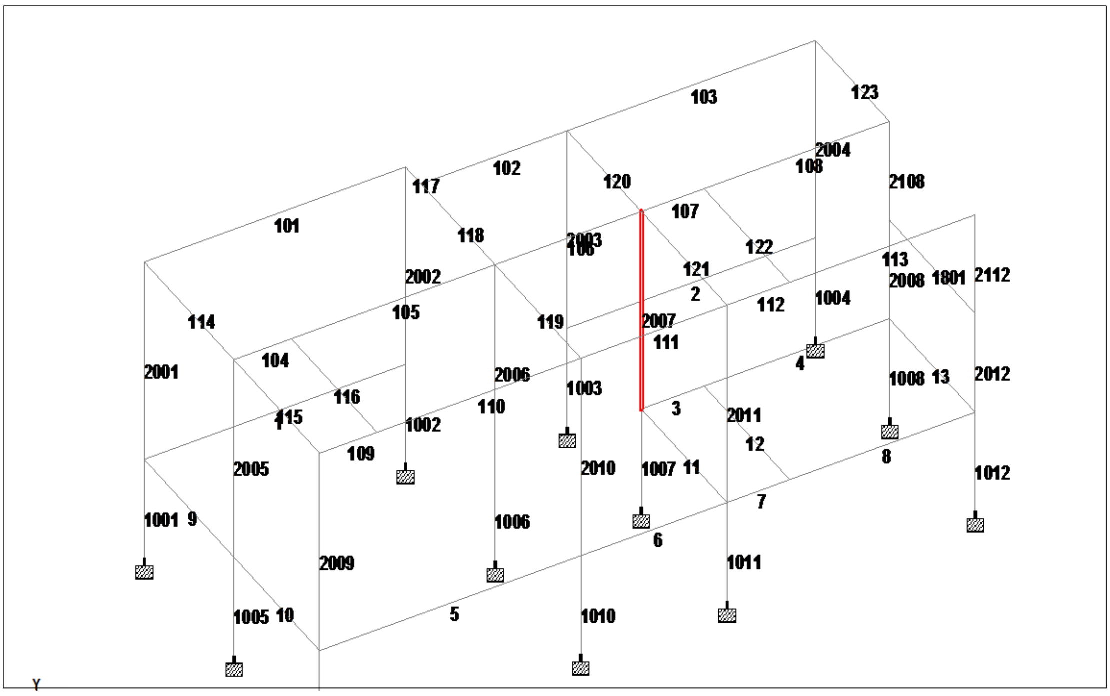


Figure 3: First floor column no 2007 (shortage of reinforcement)

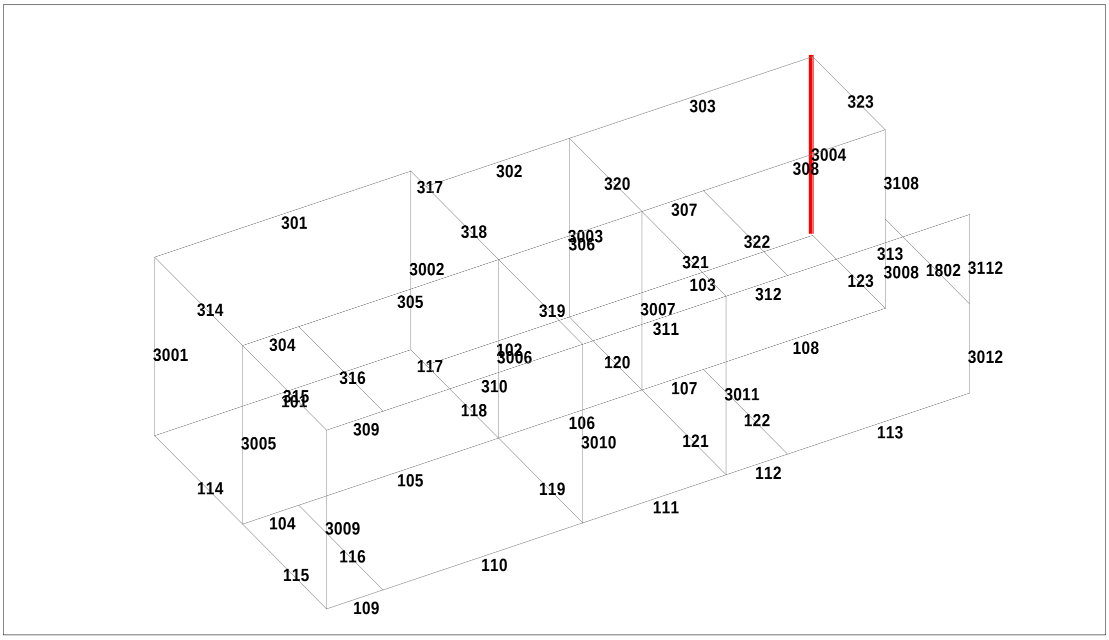


Figure 4: Second floor column no 3004 (shortage of reinforcement)

Table 6: Required and provided area of steel

Col no	Column Size	Area Of Steel(Fd to gd) (1000)	Area Of Steel(gd to 1^{st}) (2000)	Area Of Steel(1^{st} to 2^{nd}) (3000)	Area Of Steel(2^{nd} to 3^{rd}) (4000)	Area Of Steel (3^{rd} to roof) (5000)	Area Of Steel(roof to wt) (6000)
1	250.0X300.0	611.51	1308.27	1243.00	1031.39	1052.22	Nil
provided		4 -16ø +4 - 12ø=1256	4 -16ø +4 - 12ø=1256	4 -16ø=804	4 -16ø=804	4 -16ø=804	
2	250.0X350.0	914.84	1113.22	1013.47	765.11	904	Nil
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	
3	250.0X350.0	900.52	752.90	546.09	269.08	623	Nil
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	
4	250.0X300.0	600.36	1010.90	1304.29	1177	1137	Nil
provided		4 -16ø +4 - 12ø=1256	4 -16ø +4 - 12ø=1256	4 -16ø=804	4 -16ø=804	4 -16ø=804	
5	250.0X350.0	950.88	1680.19	1510.48	950	891	Nil
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	
6	250.0X400.0	944.32	813.40	1234.62	556	726	Nil
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	
7	250.0X400.0	2000.00	2326.61	1203.49	523	425	724/693
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	8-12ø=904
8	250.0X350.0	2100.00	1820/1996	1410.77/745	607/426	339/461	890/848/840
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	8-12ø=904
9	250.0X300.0	486	1370.46	1489.37	1384	1203	Nil
provided		4 -16ø +4 - 12ø=1256	4 -16ø +4 - 12ø =1256	4 -16ø =804	4 -16ø =804	4 -16ø =804	
10	250.0X350.0	967.85	1306.09	1182.45	735.00	847	Nil
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	
11	250.0X350.0	1928.30	1960.00	1446.84	720	519	644/854/835
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	8-12ø=904
12	250.0X350.0	1820.00	2030/1481	1266.18/1481	714/496	510/372	480/913/862
provided		8-16ø=1608	8-16ø=1608	8-12ø=904	8-12ø=904	8-12ø=904	8-12ø=904

4.4.2 Beam

Each floor level beams are considered and their reinforcements are compared as shown in Table 7.

Table 7: Beam Schedule

Beam no	Size	1st support		Mid span		2nd support		Stirrups
		Top	Bottom	Top	Bottom	Top	Bottom	
301	250.0X350.0	829.50	85.00	0.00	480.23	964.55	223.16	8@ 120
Prov.		1030	603	628	603	1030	603	8@120
302	250.0X350.0	259.23	163.34	0.00	163.34	613.44	163.34	8 @ 120
Prov.		804	402	402	402	804	402	8@120
303	250.0X350.0	864.75	121.67	0.00	396.57	787.07	35.11	8 @ 120
Prov.		804	402	402	402	804	402	8@120
304	250.0X350.0	1204.83	471.06	536.88	163.34	0.00	539.96	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
305	250.0X350.0	0.00	431.55	0.00	443.02	1080.64	352.97	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
306	250.0X350.0	492.84	200.15	163.34	163.34	459.43	217.60	8@ 120
Prov.		804	402	402	402	804	402	8@120
307	250.0X350.0	1051.69	322.92	416.33	163.34	0.00	457.13	8 @ 120
Prov.		804	402	402	402	804	402	8@120
308	250.0X350.0	0.00	401.13	0.00	326.75	822.02	77.95	8@ 120
Prov.		804	402	402	402	804	402	8@120
309	250.0X350.0	1093.24	365.44	470.10	163.34	0.00	523.01	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
310	250.0X350.0	0.00	418.05	0.00	443.32	1106.63	379.60	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
311	250.0X350.0	626.10	163.34	0.00	163.34	540.04	163.34	8@ 120
Prov.		1030	603	628	603	1030	603	8@120
312	250.0X350.0	1056.60	327.94	366.23	163.34	0.00	465.82	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
313	250.0X350.0	0.00	428.99	0.00	374.98	928.18	179.82	8 @ 120
Prov.		1030	603	628	603	1030	603	8@120
314	250.0X350.0	383.36	163.34	0.00	166.47	395.79	163.34	8 @ 120
Prov.		603	402	603	402	603	402	8@120
315	250.0X350.0	262.09	163.34	0.00	163.34	257.38	178.76	8 @ 120
Prov.		603	402	603	402	603	402	8@120
316	250.0X300.0	182.48	0.00	0.00	292.10	177.69	0.00	8 @ 100
Prov.		226	402	226	402	226	402	8@100
317	250.0X350.0	593.00	163.34	367.87	262.13	189.24	445.15	10 @ 100
Prov.		603	402	603	402	603	402	8@120
318	250.0X350.0	163.34	247.74	0.00	163.34	456.03	163.34	8 @ 120
Prov.		603	402	603	402	603	402	8@120
319	250.0X350.0	386.98	163.34	0.00	163.34	320.33	163.34	8 @ 120
Prov.		603	402	603	402	603	402	8@120
320	250.0X350.0	359.15	193.09	0.00	163.34	355.69	163.34	8@ 120
Prov.		603	402	603	402	603	402	8@120
321	250.0X350.0	330.25	163.34	0.00	163.34	301.60	167.19	8@ 120
Prov.		603	402	603	402	603	402	8@120
322	250.0X300.0	163.60	0.00	0.00	210.70	173.69	137.74	8@ 100
Prov.		603	402	603	402	603	402	8@120
323	250.0X350.0	508.23	183.35	0.00	167.88	366.46	350.42	8 @ 120
Prov.		603	402	603	402	603	402	8@120

4.4.3 Slab

For calculating the reinforcement that is required in the slab, it has been divided it into 9 panels. Using [13] 1S 456:2000 and [14] SP 16:1980, calculated the reinforcements required. The following table shows the complete reinforcement data given in Table 8. The reinforcement required and provided was safe.

Table 8: Slab Schedule

Case	Panel	ly	lx	ly/lx	Wu	-αx	+αx	-αy	+αy	-Mx	-My	+Mx	+My	-Astx	-Asty	+Astx	+Asty
4	S1	5.5	3.9	1.4	13.5	0.071	0.047	0.053	0.035	**15**	**10**	**11**	**7**	10mm @230	10mm @250	10mm @250	10mm @250
1	S2	5.5	4.7	1.2	13.5	0.043	0.032	0.032	0.024	**13**	**9**	**9**	**7**	10mm @250	10mm @250	10mm @250	10mm @250
3	S3	5.7	5.5	1.0	13.5	0.04	0.037	0.03	0.032	**16**	**15**	**12**	**13**	10mm @210	10mm @230	10mm @250	10mm @250
3	S4	6.4	5.5	1.2	13.5	0.048	0.037	0.036	0.028	**20**	**15**	**15**	**11**	10mm @170	10mm @230	10mm @230	10mm @250
4	S5	6.4	5.5	1.2	13.5	0.057	0.047	0.043	0.035	**23**	**19**	**17**	**14**	10mm @150	10mm @170	10mm @190	10mm @240
4	S6	5.5	4.5	1.2	13.5	0.061	0.047	0.046	0.035	**17**	**13**	**13**	**10**	10mm @200	10mm @250	10mm @250	10mm @250
2	S7	5.5	3.9	1.4	13.5	0.056	0.037	0.042	0.028	**11**	**7**	**8**	**6**	10mm @250	10mm @250	10mm @250	10mm @250
1	S8	5.5	3.4	1.6	13.5	0.057	0.032	0.043	0.024	**9**	**5**	**7**	**4**	10mm @250	10mm @250	10mm @250	10mm @250
7	S9	4.7	2.4	1.9	13.5	0.095	0	0.072	0.043	**8**	**0**	**6**	**3**	10mm @250	10mm @250	10mm @250	10mm @250

4.4.4Foundation

Table 9 and 10 shows the required and the provided reinforcement. There was not much difference in the required and provided reinforcement for the foundation.

Table 9: Foundation detail as provided

GrpID	Length m	Width m	Thickness m	Shorter		Longer	
				Reinf(Mz)	Reinf(Mz)	Reinf(Mx)	Reinf(Mx)
				Bar	Spacing	Bar	Spacing
1	2.2	2.2	0.5	12 mm	130mm	12 mm	130mm
2	2.2	2.2	0.5	12 mm	130mm	12 mm	120mm
3	2.2	2.2	0.5	12 mm	130mm	12 mm	120mm
4	2	2	0.5	12 mm	150mm	12 mm	150mm
5	2.6	2.5	.55	12 mm	100 mm	12 mm	100 mm
6	2.6	2.5	0.55	12 mm	100 mm	12 mm	100 mm
7	2.7	2.6	0.55	12 mm	100 mm	12 mm	100 mm
8	2.6	2.5	.55	12 mm	100 mm	12 mm	100 mm
9	2.6	2.5	.55	12 mm	100 mm	12 mm	100 mm
10	2.4	2.4	0.55	12 mm	120 mm	12 mm	120 mm
11	2.4	2.4	0.55	12 mm	120 mm	12 mm	120 mm
12	2.4	2.4	0.55	12 mm	120 mm	12 mm	120 mm

Table 10: Foundation Detail as Per Staad Foundation (Required)

GrpID	Length (m)	Width (m)	Thickness (m)	Slope End Thickness (m)	Shorter		Longer	
					Reinf(Mz)	Reinf(Mz)	Reinf(Mx)	Reinf(Mx)
					Bar	Spacing	Bar	Spacing
1	2.15	2.15	0.40	0.16	12 mm	226.44 mm	12 mm	185.27 mm
2	2.30	2.30	0.40	0.21	12 mm	198.91 mm	12 mm	168.31 mm
3	2.30	2.30	0.40	0.21	12 mm	198.91 mm	12 mm	156.29 mm
4	2.00	2.00	0.35	0.16	12 mm	236.00 mm	12 mm	209.78 mm
5	2.40	2.40	0.45	0.21	12 mm	208.00 mm	12 mm	190.67 mm
6	2.45	2.45	0.45	0.26	12 mm	194.83 mm	12 mm	179.85 mm
7	2.90	2.90	0.55	0.36	12 mm	164.00 mm	12 mm	154.89 mm
8	2.55	2.55	0.45	0.26	12 mm	187.54 mm	12 mm	162.53 mm
9	2.15	2.15	0.40	0.16	12 mm	203.80 mm	12 mm	203.80 mm
10	2.40	2.40	0.45	0.21	12 mm	176.00 mm	12 mm	163.43 mm
11	2.95	2.95	0.55	0.36	12 mm	149.37 mm	12 mm	149.37 mm
12	2.70	2.70	0.50	0.31	12 mm	184.86 mm	12 mm	161.75 mm

CHAPTER-5

RETROFITTING STRATEGIES

5.1 Introduction

This chapter contains the various retrofitting strategies used for this project. These include adding of shear walls, jacketting and the use of Fibre reinforced polymer for the retrofitting of the building. Their technical considerations, constructional considerations and limitations have been elaborated briefly.

5.2 Different options used for the retrofitting

5.2.1 Adding new shear walls

One of the most common methods to increase the lateral strength of the reinforced concrete building is to make a provision for additional shear walls (figure 5). The technique of infilling/adding new shear walls is often taken as the best and simple solution for improving seismic performance. Therefore, it is frequently used for retrofitting of non-ductile reinforced concrete frame buildings. The added elements can be either cast-in-place or pre-cast concrete elements. New elements preferably are placed at the exterior of the building; however it may cause alteration in the appearance and window layouts. Placing of shear walls in the interior of the structure is not preferred in order to avoid interior mouldings.

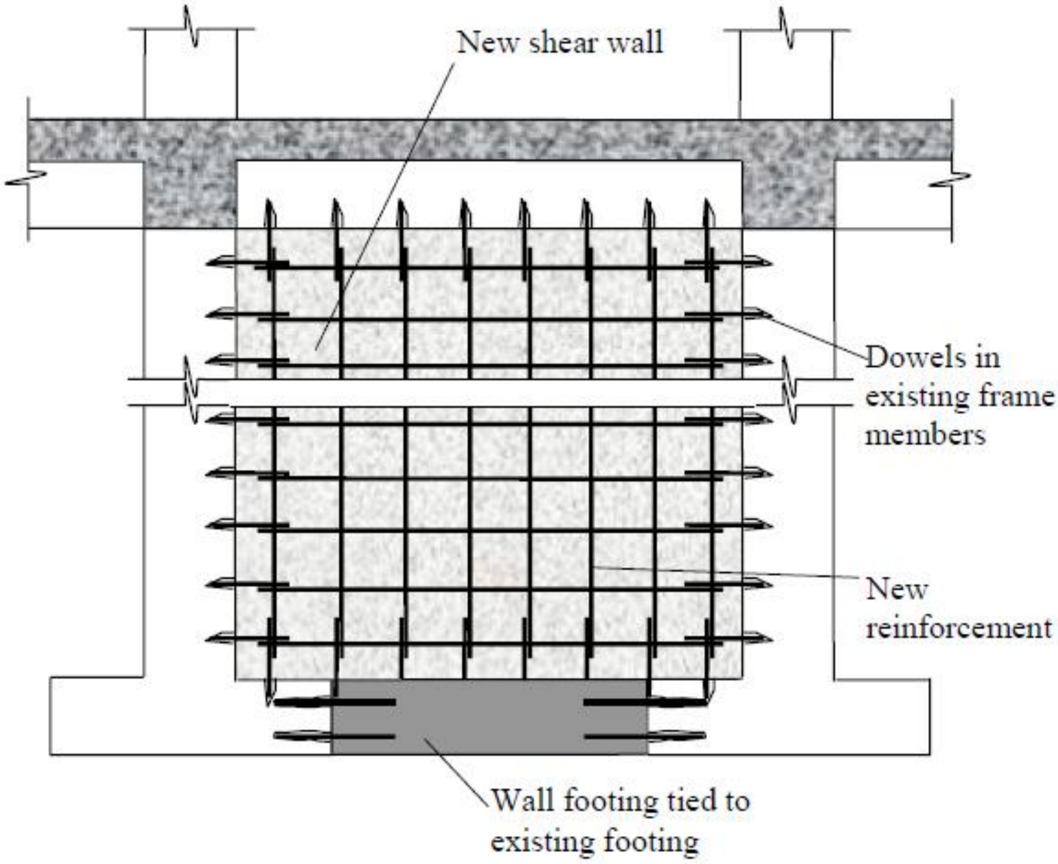


Figure 5: Adding of new shear wall

5.2.1.1 Technical consideration

The addition of new shear walls to existing frame has many technical consideration which may be summarized as (a) determining the adequacy of existing floor and roof starts to carry the seismic forces: (b) transfer of diaphragm shear into the new shear walls with dowels; (c) adding new collector and drag members to the diaphragm; (d) increase in the weight and concentration of shear by the addition of wall which may affect the foundation.

5.2.1.2 Constructional consideration

The first consideration during construction is to find locations where walls can be added and well located which may align to the full heights of the building to minimize torsion. It is often desirable to locate walls adjacent to the beam between columns so that only minimum slab demolition is required with connections made to beam at the sides of columns. The design of the shear wall may be similar to new constructions. The longitudinal reinforcement must be placed at the ends of the wall running continuously through the entire height. In order to realize this end, the reinforcement has to pass through holes in slabs and around the beams to avoid interference. To achieve both conditions , boundary elements can be used .Although it would also be convenient to have continuous shear reinforcement but in its absence, the walls must be adequately connected to the beams , slabs and columns ensuring proper shear transfer through shear connectors .Wall thickness also varies from 15 to 25 cm (6 to 10 inch) and is normally placed externally .This retrofitting system is only adequate for concrete structures, which bring forth a big increase in the Internal capacity and stiffness. A reasonable structural ductility may be achieved if the wall is properly designed with a good detailing. The connection to the existing to the structure has to be carefully designed to guarantee shear transfer.

5.2.1.3 Limitations

The main limitations of this method are: (i) increase in lateral resistance but it is concentrated at a few places, (ii) increased overturning moment at foundation causes very high uplifting that needs either new foundations or strengthening of the existing foundations, (iii) increased dead load of the structure, (iv) excessive destruction at each floor level results in functional disability of the buildings, (v) possibilities of adequate attachment between the new walls and the existing structure ,(vi) closing of formerly open spaces can have major negative impact on the interior of the building or exterior appearance.

5.2.2 Jacketing/confinement

Jacketing is the most popularly used methods for strengthening of building columns. The most common types of jackets are steel jacket, reinforced concrete jacket, fibre reinforced polymer composite jacket, jacket with high tension materials like carbon fibre, glass fiber etc. The main purposes of jacketing are :(i) to increase concrete confinement by transverse fibre/ reinforcement, especially for circular cross – sectional columns, (ii) to increase shear strength by transverse fibre / reinforcement, (iii) to increase flexural strength by longitudinal fibre/ reinforcement provided they are well anchored at critical section . Transverse fibre should be wrapped all around the entire circumference of the members possessing close loops sufficiently overlapped or welded in order to increase concrete confinement and shear strength .This is how members with circular cross-section will get better confinement and member with rectangular cross-section .Where square or rectangular cross –sections are to be jacketed, circular/oval/elliptical jackets are most often used and the space between the jacket and column is filled with concrete . Such types of multi-shaped jackets provide a high degree of confinement by virtue of their shape to the splice region proving to be more effective. Rectangular jackets typically lack the flexural stiffness needed to fully confine the concrete .However ,circular and oval jackets may be less desirable due to (i) need of large space in the building potential difficulties of fitting in the jackets with existing partition walls, exterior cladding , and non-structural elements and (ii) where an oval or elliptical jacket has sufficient stiffness to confine the concrete along the along dimension of the cross-section is open to question(figure 2) The longitudinal fibres similar to longitudinal reinforcement can be effective in increasing the flexural strength of member although they cannot effectively increase the flexural capacity of building frames because the critical moments are located at beam-column ends where most of the longitudinal fibres are difficult to pierce through to get sufficient anchorage.

5.2.2.1 Technical considerations

The main objective of jacketing is to increase the seismic capacity of the moment resisting framed structures. In almost every case, the column as well as beams of the existing structures has been jacketed .In comparison to the jacketing of reinforced concrete columns, jacketing of reinforced concrete beams with slabs is difficult yielding good confinement because slab causes hindrance in the jacket. In structures with waffle slab, the increase in stiffness obtained by jacketing columns and some of the ribs, have improved the efficiency of structures .In some cases, foundation grids are strengthened and stiffened by jacketing their beams. An increase in strength, stiffness and ductility or a combination of them can be obtained .There may be several options for the jacketing of members. Usually the existing member is wrapped with a jacket of concrete reinforced with longitudinal steel and ties or with welded wire fibre steel plate , similar to other strengthening schemes, the design of jackets should also include the structure may lead to a change in the lateral forces induced by an earthquake .Jacketing serves to improve the lateral strength and ductility by confinement of compression concrete .It should be noted that retrofitting of a few members with jacketing or some other enclosing techniques might not be effective enough to improve the overall behaviour of the structure, if the remaining members are not ductile.

5.2.2.2 Jacketing of Columns

Jacketing of Columns consist 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 columns and strength of 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 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 and (ii) steel jacketing

5.2.2.3 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: (i) increase in the shear capacity of columns in order to accomplish a strong column-weak beam design and (ii) to improve the column’s flexural strength by the longitudinal steel of the jacket made continuous through the slab system and 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 shown in figure 6.

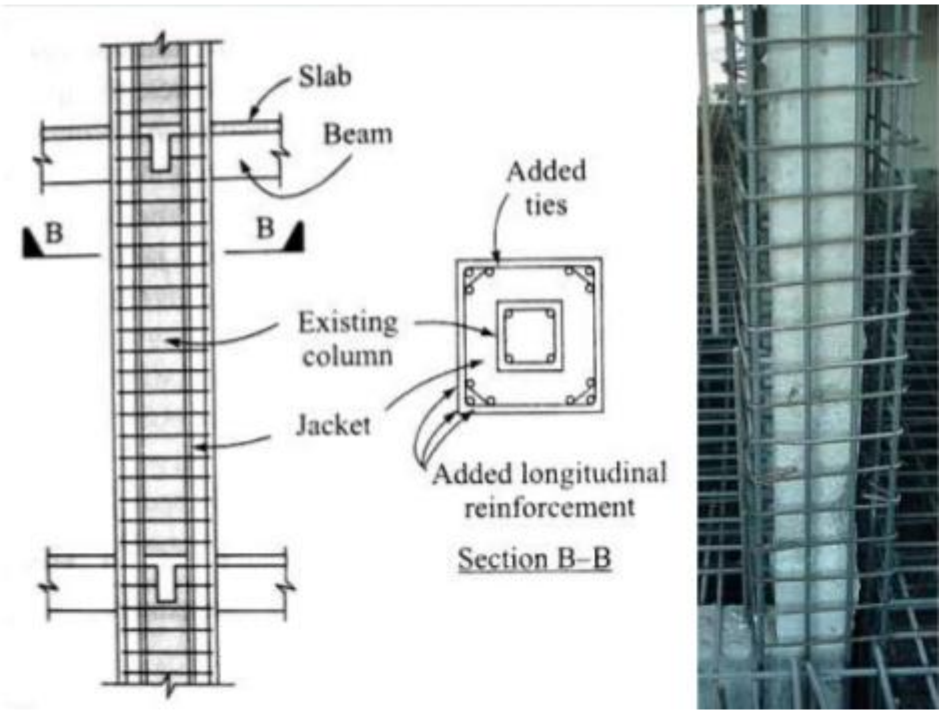


Figure 6: Jacketting of the columns

Rehabilitated sections are designed in this way so that the flexural strength of columns should be greater than that of the beams. Transverse steel above and below the joint has been provided with details, which consists of two L-shaped ties that overlap diagonally in opposite corners. The longitudinal reinforcement usually is concentrated in the column corners because of the existence of the beams where bar bundles have been used. It is recommended that not more than 3 bars be bundled together. Windows are usually bored through the slab to allow the steel to go through as well as to enable the concrete casting process. In some cases, jacketting has been applied only within the storey as a local strengthening.

5.2.2.4 Beam-column joint jacketing

A joint may be defined as the part of the column that is located through the depth of the beams, and which intersect that column. This critical region should have enough confinement and shear capacity. However, due to lack of space in the joint region it is difficult enough to provide an adequate confinement. Alcocer (1992) has assessed experimentally the behaviour of several beam columns sub-assemblages, where the point is confined with a steel cage. Test results have indicated that jacketing has been effective in rehabilitating the joint, with improving the strength, stiffness and energy dissipation characteristics of the existing joint. In these specimens, the dissipation of energy has been mainly concentrated at the beam's ends. It is also very important to point out the need to have a very strong column as compared to the beam to avoid driving of the column or joint into significant inelastic behaviour.

5.2.2.5 Slab column connection

The most critical type of structural damage is the slab column connection which results in the punching shear failure due to the transfer of unbalanced moments. The retrofitting of slab column connection is beneficial for the prevention of punching shear failures. A considerable amount of research carried out in this regard.

5.2.2.6 Limitations

There are some disadvantages associated with the column jacketing technique as well, (i) in some cases the presence of beams may require majority of new longitudinal bars to be bundled into the corners of the jacket; (ii) with the presence of the existing column it is difficult to provide cross ties for new longitudinal bars which are not at the corners of the jacket; (iii) jacketing is based mostly on engineering judgment as there is a dearth of guidelines.

5.2.2.7 Beam jacketing

Jacketing of beams is recommended for several purposes as it gives continuity to the column and increases the strength and stiffness of the structure. While jacketing a beam, its flexural resistance must be carefully computed to avoid the creation of a strong beam-weak column system. In the retrofitted structure, there is a strong possibility of change of mode of failure and redistribution of forces as a result of jacketing of column, which may consequently causes beam hinging. The location of the beam critical section and the participation of the existing reinforcement should be taken into consideration. Jacketing of beam may be carried out under different ways. The most common are one sided jackets. At several occasions, the slab has been perforated to allow the ties to go through and to enable the concrete. The beam should be jacketed through its whole length. The reinforcement has also been added to increase beam flexural capacity moderately and to produce high joint shear stresses. Top bars crossing the orthogonal beams are put through holes and the bottom bars have been placed under the soffit of the existing beams, at each side of the existing column. Beam transverse steel consists of sets of U-shaped ties fixed to the top jacket bars and of inverted U-shaped ties placed. Through perforations in the slab, closely spaced ties have been placed near the joint region where beam hinging is expected to occur. Martinez et al, 1994 has reported that adding concrete capitals or steel plates on both sides of slab can prevent punching shear failures.

5.2.3 Steel Jacketing

[16] Local strengthening of columns has been frequently accomplished by jacketting with steel plates. A general feature of steel jacketting is mentioned in Table 11.

5.2.4 FRP Jacketing

Several researchers have investigated the possibility and feasibility of fibre reinforced polymer composite jackets for seismic strengthening of columns winding them with high strength carbon fibres around column surface to add spiral hoops. The merits of this method are : (i) carbon fibre is flexible and can be made to contact the surface tightly for a high degree of confinement is of high ; (ii) confinement is of high degree because carbon fibres is of high strength and high modules of elasticity are used ; (iii) the carbon fibre has light weight and rusting does not occur.

Table 11: Jacketting with steel plates

Steel plate thickness	**At least 6mm**
Height of jacket	1.2 to 1.6 times splice length in case of flexural columns
	Full height of columns in case of Shear columns
Shape of jackets	Rectangular jacketing, prefabricated two L-Shaped panels The use of rectangular jackets has proved to be successful in case of small size columns up to 36 inch that have been successfully retrofitted with ¼' thick steel jackets combined with adhesive anchor bolt, but has been less successful on larger rectangular columns. On larger columns, rectangular jackets appear to be incapable to provide adequate confinement
Free ends of jackets	Welded throughout the jacket, size of weld ¼'
Bottom clearance	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
Gap between steel jacket and concrete column	25 mm (1 inch) fill with cementations grout
Size of Anchor Bolt	25 mm (1 inch) in diameter and 300 mm (12") long embedded in 200 mm (8") into concrete column
	Bolts were installed through pre-drilled holes on the steel jacket using an epoxy adhesive
Number	Two anchor bolts are intended to stiffen the steel and improve confinement of the splice

CHAPTER -6

NUMERICAL MODELLING USING ANSYS

6.1 Introduction

This chapter gives a brief introduction of the finite element analysis of the deficient columns using ANSYS v12. Numerical modelling and design of the columns are performed. Material properties and reinforcement detailing of each of the column is given. Meshing properties are also defined in this unit.

6.2 Numerical modelling using ANSYS

[17] The finite element method is a numerical technique for finding approximate solutions to boundary value problems for partial differential equations. The finite-element program ANSYS v12 Workbench is used for the numerical modelling of columns. The element details of each material are presented subsequently in table number 12. Cross-section of columns can be seen from table 6. The finite element analysis is an assembly of finite elements which are interconnected at a finite number of nodal points. In the present study, discrete modelling approach is used to model the behaviour of reinforced concrete columns using ANSYS software. In this approach, concrete columns are modelled by Solid65 elements while the reinforcement (steel) is modelled by Link8 elements. The nonlinearity is derived from the nonlinear relationships in material models and the effect of geometric nonlinearity is not considered. The parameters to be considered for Solid65 element are material number, volume ratio and orientation angles (in X and Y direction). The parameters to be considered for Link8 element are cross sectional area and initial strain. The columns are designed for the static loading.

Table 12: Element Details

COLUMN NUMBER	**HEIGHT (m)**	**AREA OF STEEL**	**STIRRUPS**	**LOAD, PU (KN)**	**MOMENT, MZ (KN-M)**	**MOMENT, MY (KN-M)**
1007	1.5	8-16 Φ	8-150 c/c	1528.68	72.33	39.92
2007	3	8-16 Φ	8-150 c/c	1589.70	95.41	51.50
3007	3	8-12 Φ	8-150 c/c	1325.15	79.78	44.20
4009	3	4-16 Φ	8-150 c/c	1040.01	33.73	42.09
5009	3	8-12 Φ	8-150 c/c	1005.61	46.57	36.55

Concrete: To model the concrete an eight-node solid element, Solid65, is used. This solid element has eight nodes with three degrees of freedom at each node with translations in the nodal x, y, and z directions. Plastic deformation, cracking in three orthogonal directions and crushing capability can be utilized by the element

Reinforcing Steel: Steel reinforcement in the analytical modelled of column is designed with typical fy-415 MPa bars. The steel for the finite-element model was assumed to be an elastic–perfectly plastic material with identical properties in tension and compression. A Link8 element is used to model the steel reinforcement. Two nodes are required for this element. Each node has three degrees of freedom, which are translations in the nodal x, y, and z directions

6.2.1 Material Properties

The following material properties (Table 13) are used for the present Finite element analysis for static structure under static loading. Grade of concrete used is M20 and grade of steel is fy 415.

Table 13: Properties of Concrete, Steel and FRP

S.NO	**Type of Jacketting**		**PROPERTIES**		
		Density (Kg/m^3)	Youngs modulus E (GPa)	Poissons ratio	Tensile yield Strength (GPa)
1	CONCRETE	2300	30	0.18	0
2	STEEL	7850	200	0.3	0.25
3	CARBON FRP	1960	517	0.3	1.86

6.2.2 Meshing and Reinforcement detailing

The geometric design and reinforcement detailing of columns is shown in Figure 7 and figure 8. To obtain good a result from the Solid65 element, a square mesh is used (Figure 9). Therefore, the mesh is setup such that square or rectangular elements are created. The volume sweep command of ANSYS v12 is used to mesh the support. This properly sets the width and length of elements in the concrete support and makes it consistent with the elements and nodes in the concrete portions of the model. In the analysis, the specimen was modelled with square concrete elements by using a 50 mm mesh configuration. The maximum layer of meshing is 5 and the Transition ratio is 0.272.

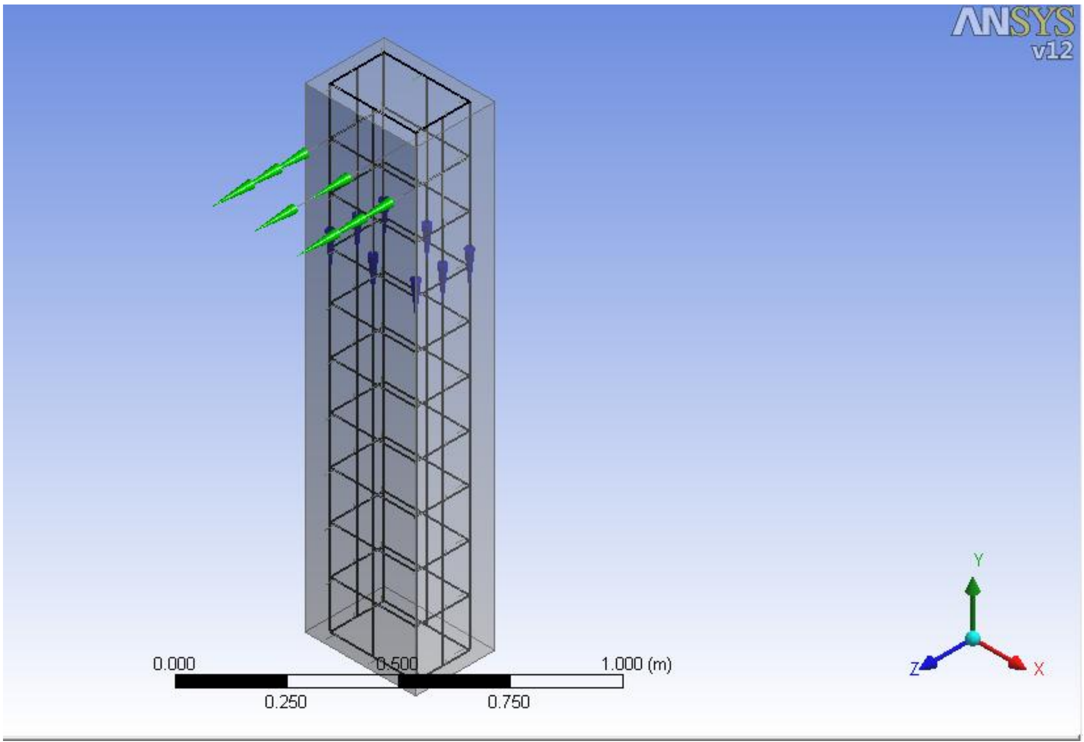


Figure 7: Geometric model and reinforcement detailing of Col 3007 (ISO view)

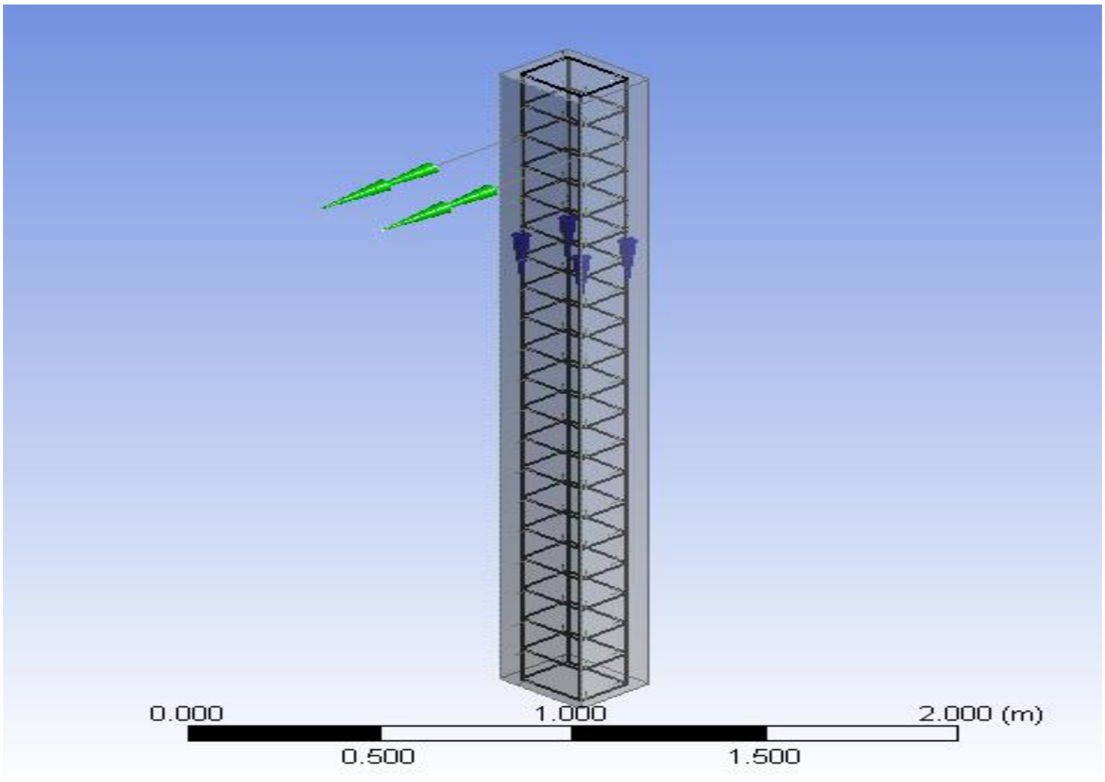


Figure 8: Geometric model and reinforcement detailing of Col. 5009 (ISO view)

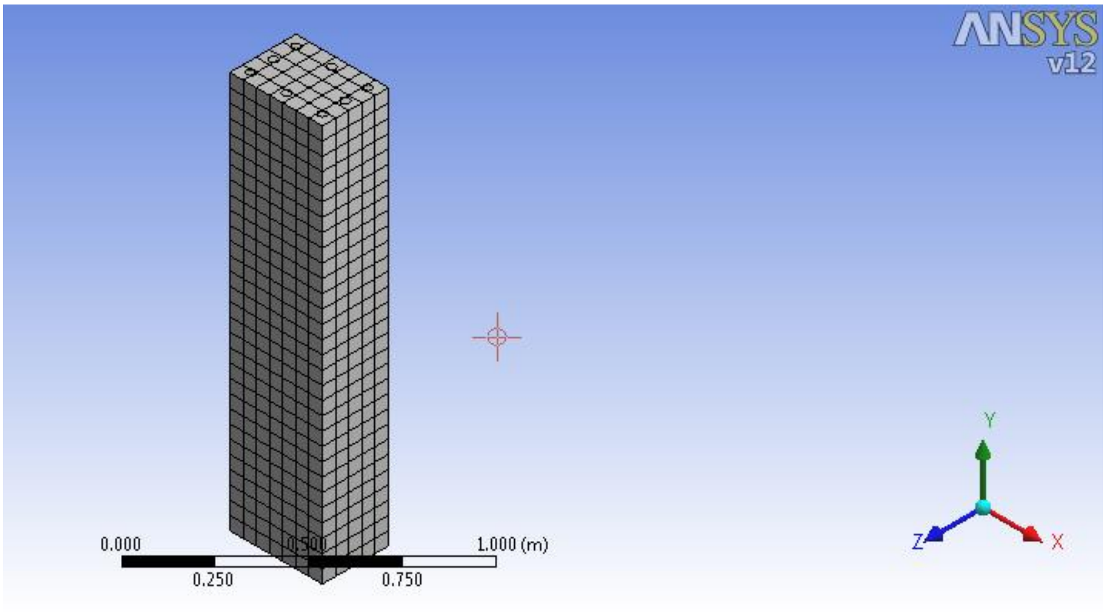


Figure 9: Square meshing of Column 3007

CHAPTER -7

PERFORMANCE OF BUILDING AFTER COLUMN JACKETTING

7.1 Introduction

This unit contains the RC retrofitting of the existing columns of the building. The columns at each floor level have been retrofitted. RC jacketting is done with the help of IS 15988:2013, Seismic Evaluation And Strengthening Of Existing Reinforced Concrete Buildings –Guidelines, Bureau of Indian Standards, New Delhi, 2013. The structural drawings are shown with the details of RC jacketing. The deficient columns are also modeled and designed using ANSYS v12. Also, RC jacketing is performed and Total deformation, Maximum principal stress and Maximum principal stress is calculated.

7.2 Introduction to IS 15988:2013

Column jacketing is carried out as per recommendations of Indian standard code IS 15988 (2013): Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines published By Bureau of Indian Standards [18]. Reinforced concrete jacketing improves column flexural strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure as per article 8.5.1.1 of the code for reinforced concrete jacketing is as follows:

1. The seismic demand on the columns, in terms of axial load P and moment M is obtained.
2. The column size and section details are estimated for P and M as determined above.
3. The existing column size and amount of Reinforcement is deducted to obtain the Amount of concrete and steel to be provided in the jacket.
4. The extra size of column cross-section and Reinforcement is provided in the jacket.
5. Increase the amount of concrete and steel actually to be provided as follows to account for losses, $A_c = (3/2)\, A'_c$ and $A_s = (4/3)\, A'_s$

Where A_c and A_s= actual concrete and steel to be provided in the jacket; and A'_c and A'_s = concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

The minimum specifications as per article 8.5.1.2 of the code for jacketing Columns are:

a) Strength of the new materials shall be equal or greater than those of the existing column. Concrete strength shall be at least 5 MPa greater than the strength of the existing concrete.

b) For columns where extra longitudinal reinforcement is not required, a minimum of 12φ bars in the four corners and ties of 8φ @100 c/c should be provided with 135° bends and 10φ leg lengths.

c) Minimum jacket thickness shall be 100 mm.

d) Lateral support to all the longitudinal bars shall be provided by ties with an included angle of not more than 135°.

e) Minimum diameter of ties shall be 8 mm and not less than one-third of the longitudinal bar diameter.

f) Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the Joints within a length of ¼ of the clear height shall not exceed 100 mm. preferably, the spacing of ties shall not exceed the thickness of the jacket or 200 mm whichever is less.

a) Strength of the new materials shall be equal or greater than those of the existing column. Concrete strength shall be at least 5 MPa greater than the strength of the existing concrete.

b) For columns where extra longitudinal reinforcement is not required, a minimum of 12φ bars in the four corners and ties of 8φ @100 c/c should be provided with 135° bends and 10φ leg lengths.

c) Minimum jacket thickness shall be 100 mm.

d) Lateral support to all the longitudinal bars shall be provided by ties with an included angle of not more than 135°.

e) Minimum diameter of ties shall be 8 mm and not less than one-third of the longitudinal bar diameter.

f) Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the Joints within a length of ¼ of the clear height shall not exceed 100 mm. preferably,

the spacing of ties shall not exceed the thickness of the jacket or 200 mm whichever is less.

7.3 Design Example for Column Jacket as per IS 15988:2013

Details of existing column (1007) are as follows:

Height of the Column=1500mm, Cross-Section= (250X400) mm, Effective Cover=40mm

Grade of Concrete =20 N/mm2 and Grade of steel=415 N/mm2

Load, P_u=1528.68 KN, Moment, M=72.33 KN-m, Reinforcement provided=8-16mmØ bars

Procedure:

P_u=0.4 x f_{ck} x A_c +0.67 x f_y x A_{sc}

According to the provisions provided in to 8.5.1.2 (a) of IS 15988: 2013, Concrete strength shall be at least 5 MPa greater than the strength of the existing concrete.

Thus, taking value of f_{ck} =25 N/mm2 and assuming A_{sc}= 0.8% A_c

1528.68 x 103=0.4 x 25 x A_c + 0.67 x 415 x (0.8 % A_c) or 1528 x 103= 12.22 A_c or A_c=125096.56mm2

According to 8.5.1.1 (e) of IS 15988:2013, A_c=1.5 A'_c

Thus, A_c=187644 mm2

Assuming the cross sectional details as:

B=400mm, D=187644/400=500mm

Jacketting details of cross section:

B = (400-250)/2=75mm, D = (500-400)/2=50mm

However, According to the code specified above, Minimum jacket thickness shall be 100 mm as per 8.5.1.2 (c) of IS 15988:2013

Thus, New size of the column:

B = 250+100 +100=450mm,

D =400 +100 +100=600mm

New concrete area=450x600=270000mm2 > A_c=125096.56mm2

Area of steel, A_s=0.8% x 450 x 600=2160mm2

But according to 8.5.1.1 (e) IS 15988:2013, A_s= (4/3) A'_s

A_s= (4/3) x 2160=2880 mm2

Assuming 16mm Ø bars,

Thus, number of bars, N =2880 x 4/ (π x 162) = 16 bars

Provide 16 NO. -16mm Ø bars for jacketed section.

Therefore, revised jacketed section will be 450mm x 600 mm. The details of RC jacketting are provided in Figure 10.

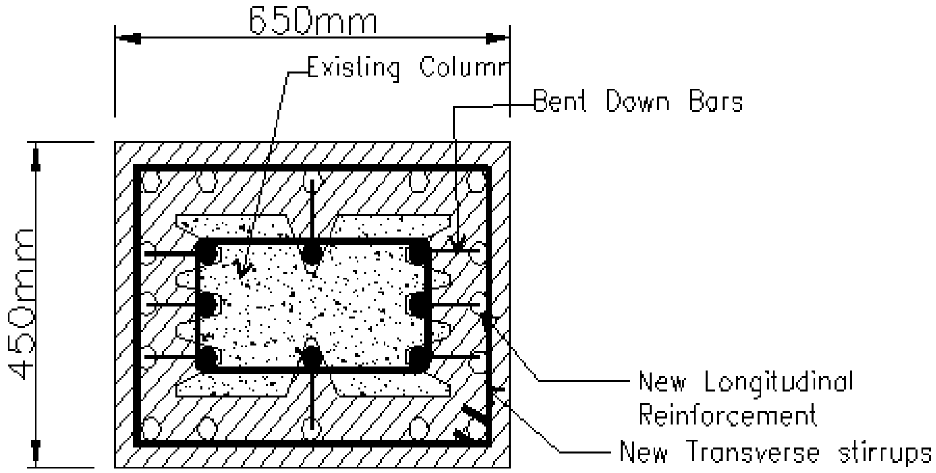


Figure 10: RC Jacketting of existing Column

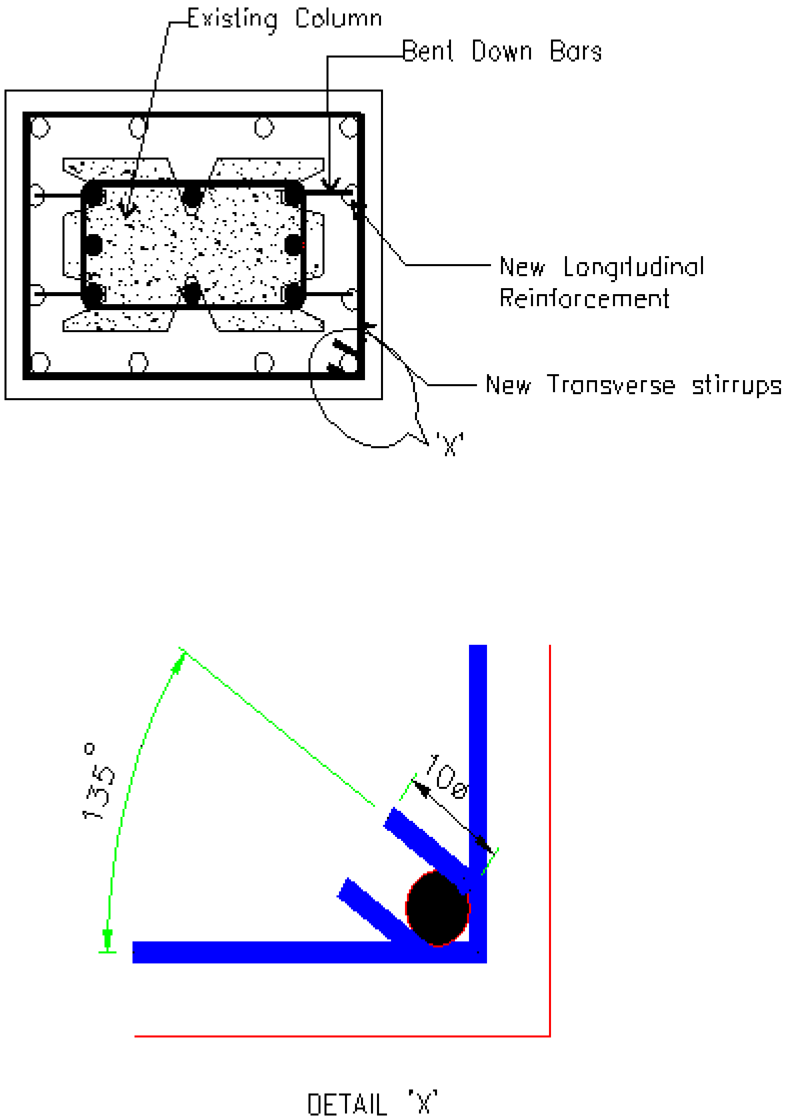


Figure 11: Reinforcement Jacket detailing for minimum level of retrofit.

Design of Lateral Ties

As per 8.5.1.2 (e) of IS15988: 2013, Minimum diameter of ties shall be 8 mm and not less than one-third of the longitudinal bar diameter.

Diameter of bar =1/3 of Ø of largest longitudinal bar = 1/3 x 16 = 6mm ….take 8mm

Spacing of ties as per 8.5.1.1 (f) of IS 15988:2013- The code suggests that the spacing, s of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y \; d_h^2}{\sqrt{f_{ck}} \quad t_j}$$

Where

f_y = yield strength of steel, f_{ck} = cube strength of concrete,

d_h= diameter of stirrup, and t_j = thickness of jacket

$s=\frac{415 \ x \ 16^2}{\sqrt{2\,5} \, x \, 200}$, s=110mm

Provide 8mm Ø @110mm c/c.

However, For columns (Figure 11) where extra longitudinal reinforcement is not required, a minimum of 12φ bars in the four corners and ties of 8φ @100 c/c should be provided with 135° bends and 10φ leg lengths.

7.4 Design Details of Column Jacketting

The deficient columns are highlighted in the figure 12 as shown below. Thus, these columns are retrofitted.

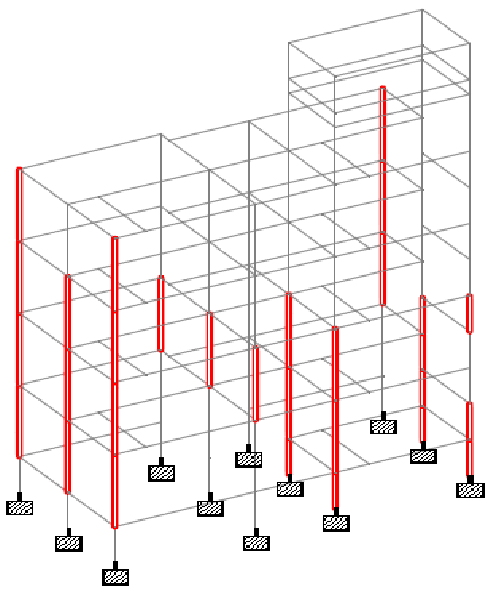


Figure 12: Pattern of deficient columns as per STAAD file.

All the deficient columns are retrofitted using IS 15988:2013. The jacket details are given in table 14.

Table 14: RC jacketting of columns.

COLUMN NO	Col size in mm		Pu KN	Ac cm^2	Adopt B2 mm	D2 mm	Jacketting B (mm)	Jacketting D (mm)	B3 (mm)	D3 (mm)	As (mm^2)	Dia of bars (mm	No of bars req	No of bars prov
	B1	D1												
1007	250	400	1529	1876	400	469	100	100	450	600	2880	16	14	16
1001	250	300	862	1057	400	264	100	100	450	500	2400	16	12	16
1008	250	350	1485	1822	400	456	100	100	450	550	2640	16	13	16
1011	250	350	1425	1749	400	437	100	100	450	550	2640	16	13	16
1012	250	350	1426	1750	400	437	100	100	450	550	2640	16	13	16
2001	250	300	1075	1319	400	330	100	100	450	500	2400	16	12	16
2007	250	400	1590	1951	400	488	100	100	450	600	2880	16	14	16
2008	250	350	1422	1745	400	436	100	100	450	550	2640	16	13	16
2011	250	350	1359	1667	400	417	100	100	450	550	2640	16	13	16
2012	250	350	1507	1849	400	462	100	100	450	550	2640	16	13	16
3001	250	300	1064	1306	400	327	100	100	450	500	2400	16	12	16
3002	250	350	1079	1324	400	331	100	100	450	550	2640	16	13	16
3004	250	300	1085	1332	400	333	100	100	450	500	2400	16	12	16
3005	250	350	1248	1532	400	383	100	100	450	550	2640	16	13	16
3007	250	400	1200	1472	400	368	100	100	450	600	2880	16	14	16
3008	250	350	1292	1585	400	396	100	100	450	550	2640	16	13	16
3009	250	300	1053	1292	400	323	100	100	450	500	2400	16	12	16
3011	250	350	1175	1441	400	360	100	100	450	550	2640	16	13	16
3012	250	350	1272	1561	400	390	100	100	450	550	2640	16	13	16
4001	250	300	990	1215	400	304	100	100	450	500	2400	16	12	16
4004	250	300	1053	1293	400	323	100	100	450	500	2400	16	12	16
4005	250	350	1111	1363	400	341	100	100	450	550	2640	16	13	16
4009	250	300	1018	1250	400	312	100	100	450	500	2400	16	12	16
5001	250	300	991	1216	400	304	100	100	450	500	2400	16	12	16
5002	250	350	1081	1326	400	332	100	100	450	550	2640	16	13	16
5004	250	300	1008	1236	400	309	100	100	450	500	2400	16	12	16
5009	250	300	1055	1294	400	324	100	100	450	500	2400	16	12	16

7.5 Performance of building after Jacketting

After performing jacketting, the columns are checked for safety against lateral load. The storey drifts and the lateral load carrying capacity of the building is significantly improved. Strong column and weak beam concept is followed for each of the beam – column joint.

7.5.1 Storey drifts

A comparision of the storey drifts of the existing building before and after retrofitting is checked. At each storey level, storey drifts are checked. Hence, it is found that the storey drifts of the retrofitted building have reduced as compared to the storey drift of the building before retrofitting. Thus, the lateral strength of the building is increased considerably. Table 15 shows the comparision of storey drifts before and after jacketting.

Table 15: Comparision of storey drifts before and after Jacketting

Storey	Height (m)	Load Combination	Before Jacketting Drift (X) mm	After Jacketting Drift (X) mm	% Reduction in storey drift	Before Jacketting Drift (Z) mm	After Jacketting Drift (Z) mm	% Reduction in storey drift
1	1.5	27	0.923	0.570	38	-	-	-
		28	0.847	0.470	44.5	-	-	-
		29	-	-	-	1.202	0.626	47.9
		30	-	-	-	1.196	0.600	49.8
3	4.5	27	2.58	1.825	29.26	-	-	
		28	2.25	1.504	33.15	-	-	
		29	-	-	-	3.09	0.072	97.6
		30	-	-	-	2.59	1.755	32.23
5	7.5	27	2.434	2.048	15.85	-	-	
		28	1.871	1.751	6.413	-	-	
		29	-	-	-	2.468	1.674	32.17
		30	-	-	-	1.361	0.772	43.27
7	10.5	27	1.919	1.696	11.60	-	-	
		28	1.475	1.37	7.12	-	-	
		29	-		-	1.568	1.5	4.336
		30	-		-	0.468	0.123	73.717
9	13.5	27	0.982	0.953	2.95	-		
		28	0.854	0.77	9.8	-		
		29	-	-	-	0.188	0.09	52
		30	-	-	-	0.650	0.55	15.83
10	15.9	27	2.311	2.111	8.65	-	-	-
		28	1.297	1.09	15.9	-	-	-
		29	-	-	-	4.23	4.1	3
		30	-	-	-	3.48	3.1	11
11	16.5	27	0.136	0.129	5.14	-	-	-
		28	0.107	0.001	99	-	-	-
		29	-	-	-	0.230	0.012	94.78
		30	-	-	-	0.147	0.10	31.09
12	18	27	0.055	0.015	72.72	-	-	
		28	0.031	0.021	32.25	-	-	
		29	-	-		0.141	0.132	6.38
		30	-	-		0.093	0.006	93.55

7.5.2 Lateral Resistance

The lateral load carrying capacity of each column is checked. It was found that loads and the moments in z and y direction were found to be significantly increased with the help of jacketting. Table 16 shows the loads and the moments in z and y direction before and after jacketting. All the deficient columns are tabulated which shows that the lateral resistance of the columns have appreciably improved.

Table 16 : Comparision of moments and loads before and after jacketting

Column Number	**Before Jacketting**	**After Jacketting**	**Before Jacketting**	**After Jacketting**	**Before Jacketting**	**After Jacketting**
	Mz (KN-m)	Mz (KN-m)	My (KN-m)	My (KN-m)	P (KN)	P (KN)
1001	28	141.38	23	126.55	861.65	2203.19
1007	72.33	240.07	39.92	176.19	1528.61	2762
1008	59.9	199	38.5	161	1485	2522
1011	48.69	200	31	161	1425.27	2548
1012	49.99	200.11	32	161.10	1425.39	2528
2001	48	133	38	119	1075	2185.87
2007	95	234	51	172	1589	2723
2008	64	195	41	157	1422	2493
2011	70	198.12	45	159.8	1443	2508
2012	69	196.87	43	158	1506	2498
3001	54	107	42	96.7	1064	2141
3002	62	138	40	112	1081	2365.71
3004	55	117	42	105	1085	2157
3005	79	148.20	42	120	1284	2381.99
3007	79	213	44	157	1325	2660
3008	64	176	41	143	1291	2434
3009	59	109	46	97.79	1119	2143
3011	69	181	44	146	1315	2445
3012	60	179	39	145	1272	2440
4001	48	70	39	63	990	2104
4004	55	101	43	90	1053	2212
4005	69	129	44	104	1140	2479.13
4009	53	81	42	73	1040	2153
5001	46	76	36	68	1006	2258
5002	56	83	36	67	1096	2442
5004	49	95	38	85	1029	2323
5009	46	83	36	74	1005	2283

7.5.3 Strong column-weak beam concept

After performing jacketting, the performance of the building should be such that strong column weak beam concept is followed. This concept is checked for the following columns and the beams.

1. Column 3007, Size: 600 x 450 mm

Percentage of reinforcement, p1 = $\frac{904+3216}{450 \text{ x } 570}$ = 1.6 %

Beams adjoining: 306, 307, 106, And 107,

Maximum Ast = 804 mm^2

Percentage of reinforcement, p2= $\frac{804}{250 \text{ x } 320}$ = 1 %

Since p1 >p2,

Strong Column-weak Beam concept is followed.

2. Column 4004, Size: 500 x 450

Percentage of reinforcement, p = $\frac{804+3216}{450 \text{ x } 470}$ = 1.90 %

Beams adjoining = 403, 423, 303, 323

Maximum Ast (303, 403) = 804 mm^2

Percentage of reinforcement, p = $\frac{804}{250 \text{ x } 320}$ = 1 %

Strong Column-weak Beam concept satisfied.

3. Column 5002, Size: 550 x 450

Percentage of reinforcement, p = $\frac{904+3216}{450 \text{ x } 520}$ = 1.76 %

Beams adjoining = 501, 517, 516, 417

Maximum Ast (501, 517) = 804 mm^2

Percentage of reinforcement, p = $\frac{1030}{250 \text{ x } 320}$ = 1.28%

Strong Column-weak Beam concept satisfied.

7.6 Reinforcement detailing for Columns

Figure 13 shows the Autocadd drawings representing the RC jacket for each column.

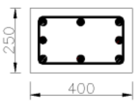
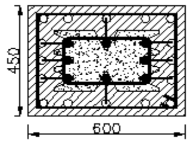
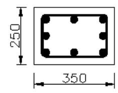
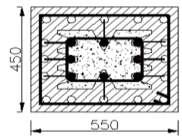
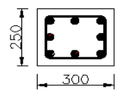
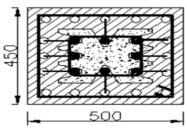
COLUMN NO	Original Col size in mm		Longitudinal Reinforcement (original)	Col size after Jacketting in mm		Longitudinal Reinforcement (after Jacketting)
	B1	D1		B2	D2	
C6,C7	250	400	 4-16Ø + 4-12Ø	400	600	 16-16Ø
C2,C3,C5,C8, C10,C11,C12	250	350	 8-16Ø	400	550	 16-16Ø
C1,C4,C9	250	300	 8-16Ø	400	500	 16-16Ø

Figure 13: Autocadd drawing of RC jacket

7.7 FEM modelling using ANSYS

Columns are modelled and designed using ANSYS v12. The above mentioned properties and reinforcement detailing is provided. A reinforced concrete (RC) jacket of 100mm thickness each is provided at all four sides of the column. After giving the jacketting properties, result values are obtained in terms of total deformation, Maximum principal stress and Maximum principal strain. Reinforced concrete jacketting of the column is done in order to retrofit it. Total deformation, maximum principal stress and maximum principal strain are calculated both before and after RC jacketting. It is found that the deformation is considerably reduced from 17.192 mm to 0.051 mm in case of column number 3007. Also, stress and strain values have also decreased. Figure 14 shows the RC jacketting of column number 3007. The green colour highlights the existing column and the rest part of column shows the RC jacket on all four sides. Figure 15 shows the arrangement of the reinforcing bars after performing the jacketting. Figure 16 and 17 shows the RC jacket (Reinforcement detailing) provided over the existing column (ISO view) for the column 3007 and 5009. The deformation values before and after jacketting is also shown in Figures 18, 19 and 20 for column number 3007. Also, the result values are compared and tabulated in table number 17.

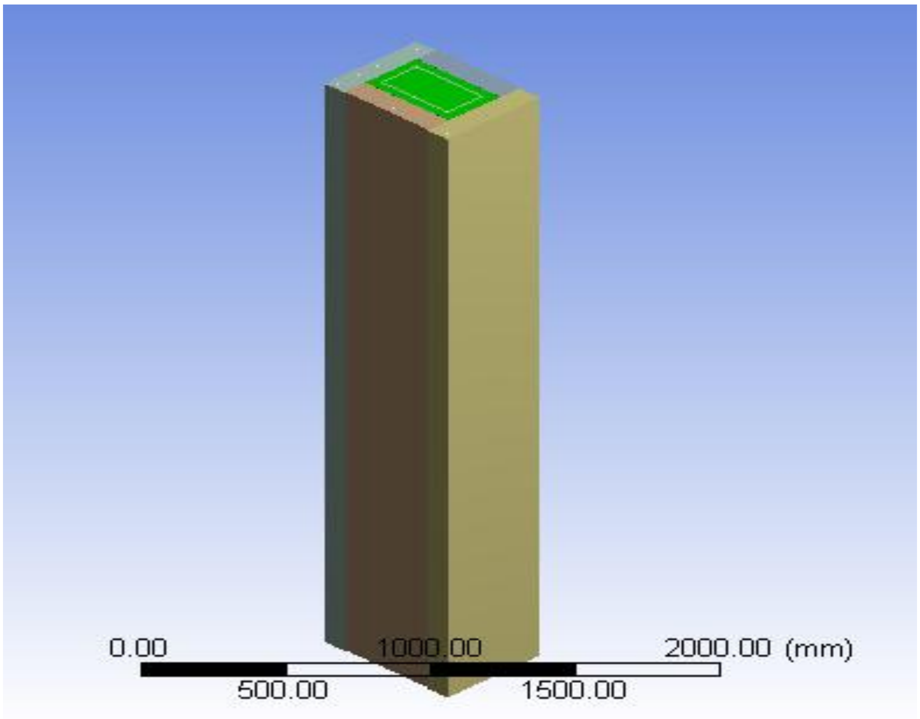


Figure 14: Column number 3007 with RC Jacketting (ISO View)

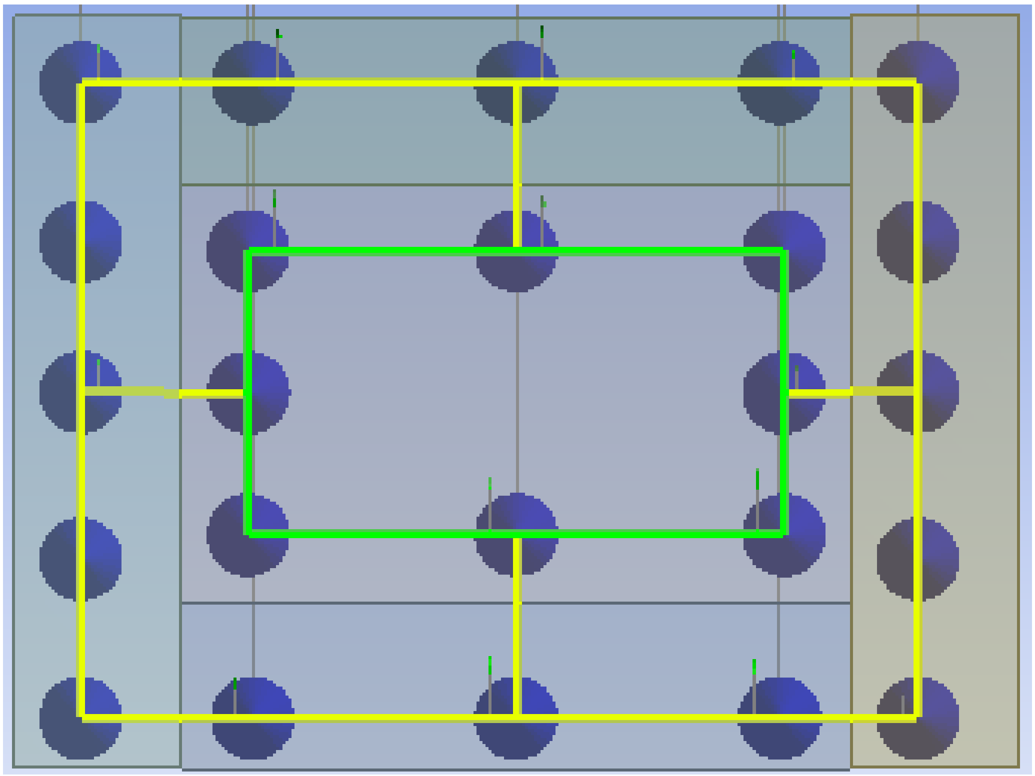


Figure 15: Column number 3007 with RC jacketting (Top View)

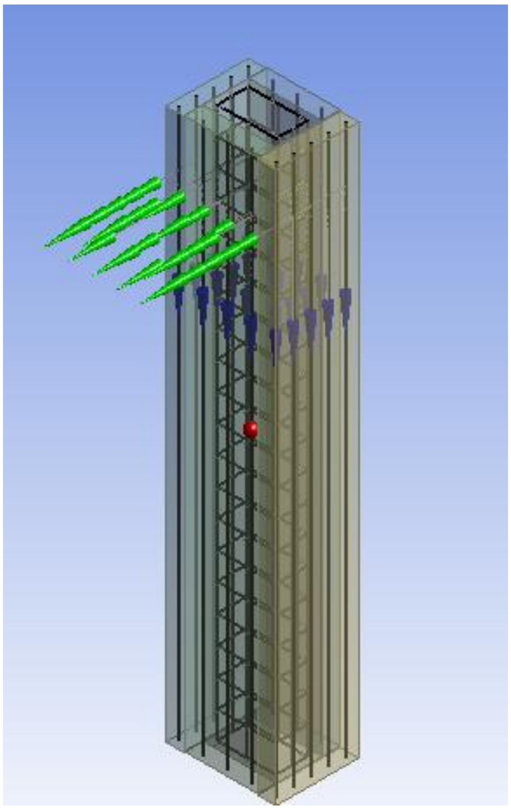


Figure 16: Concrete jacketting of column number 3007

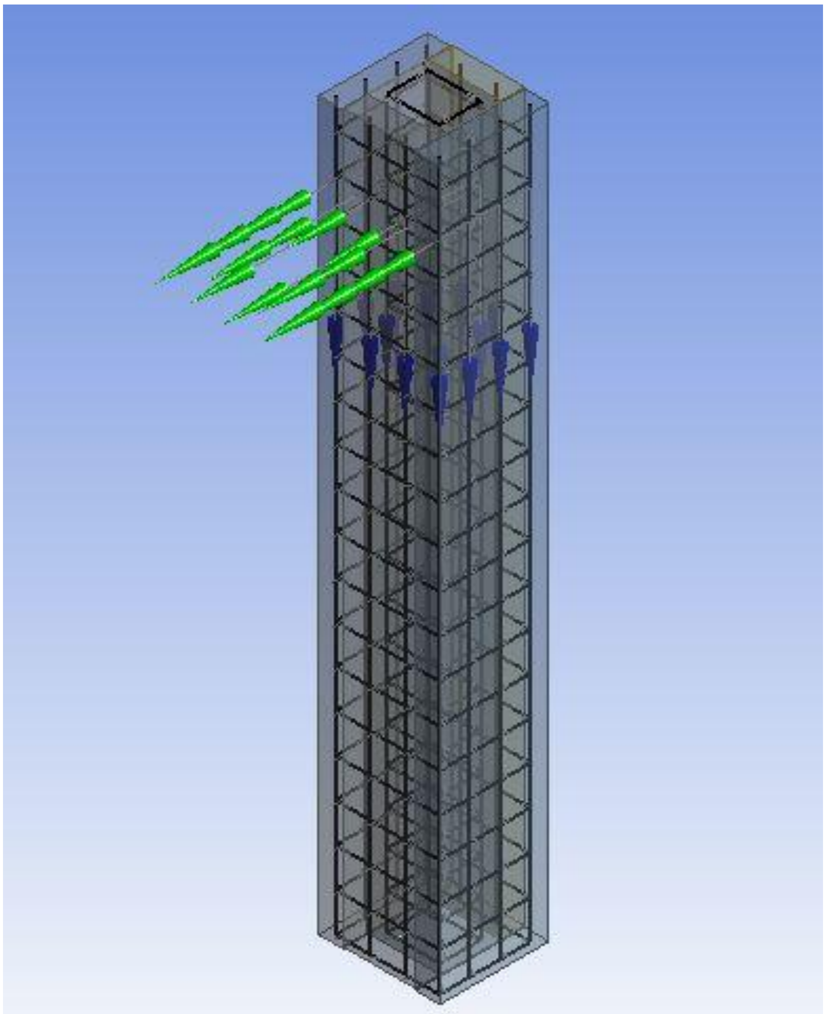


Figure 17: Concrete jacketting of column number 5009

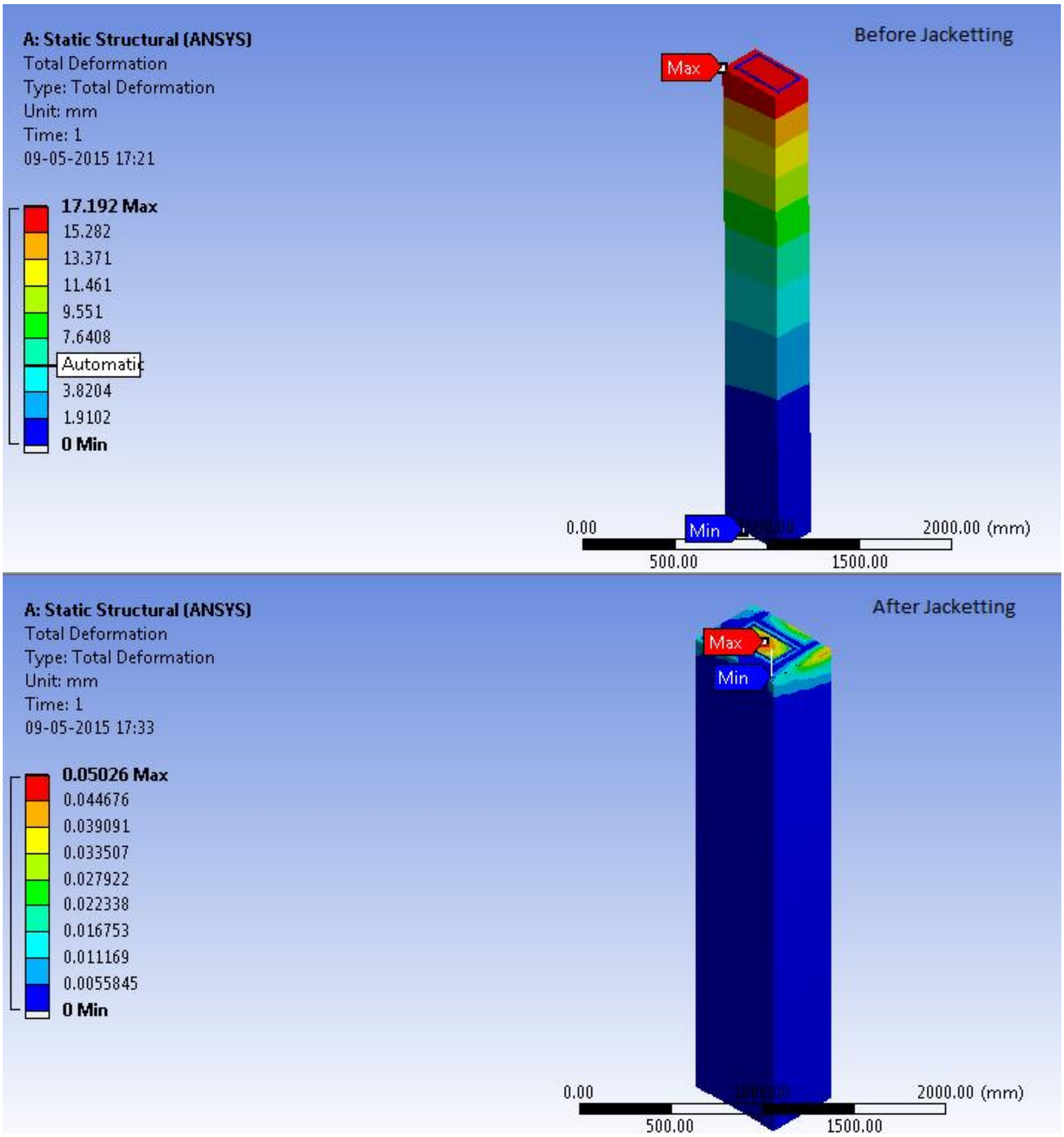


Figure 18: Total Deformation for Column 3007

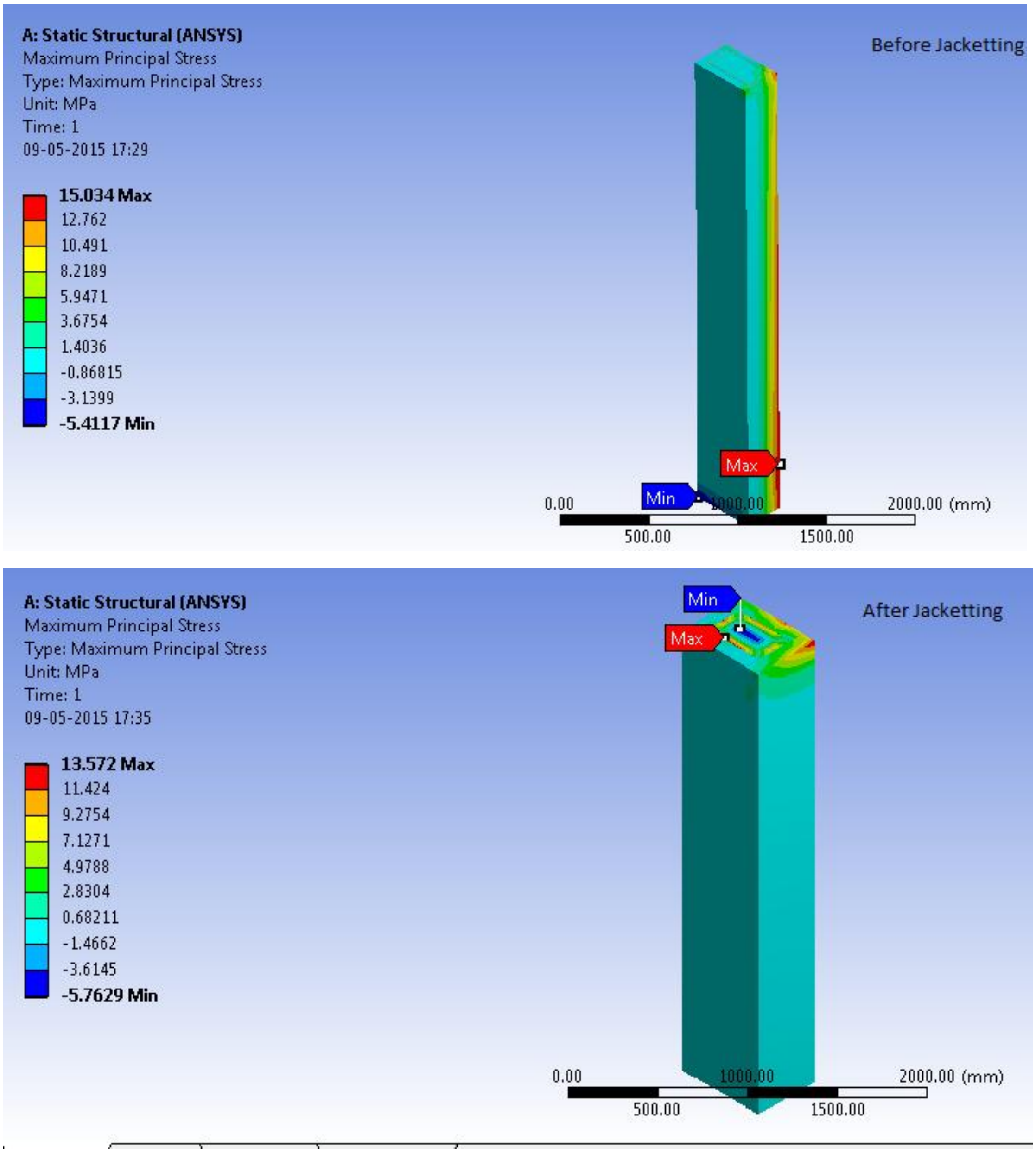


Figure 19: Maximum Principal Strain for Column 3007

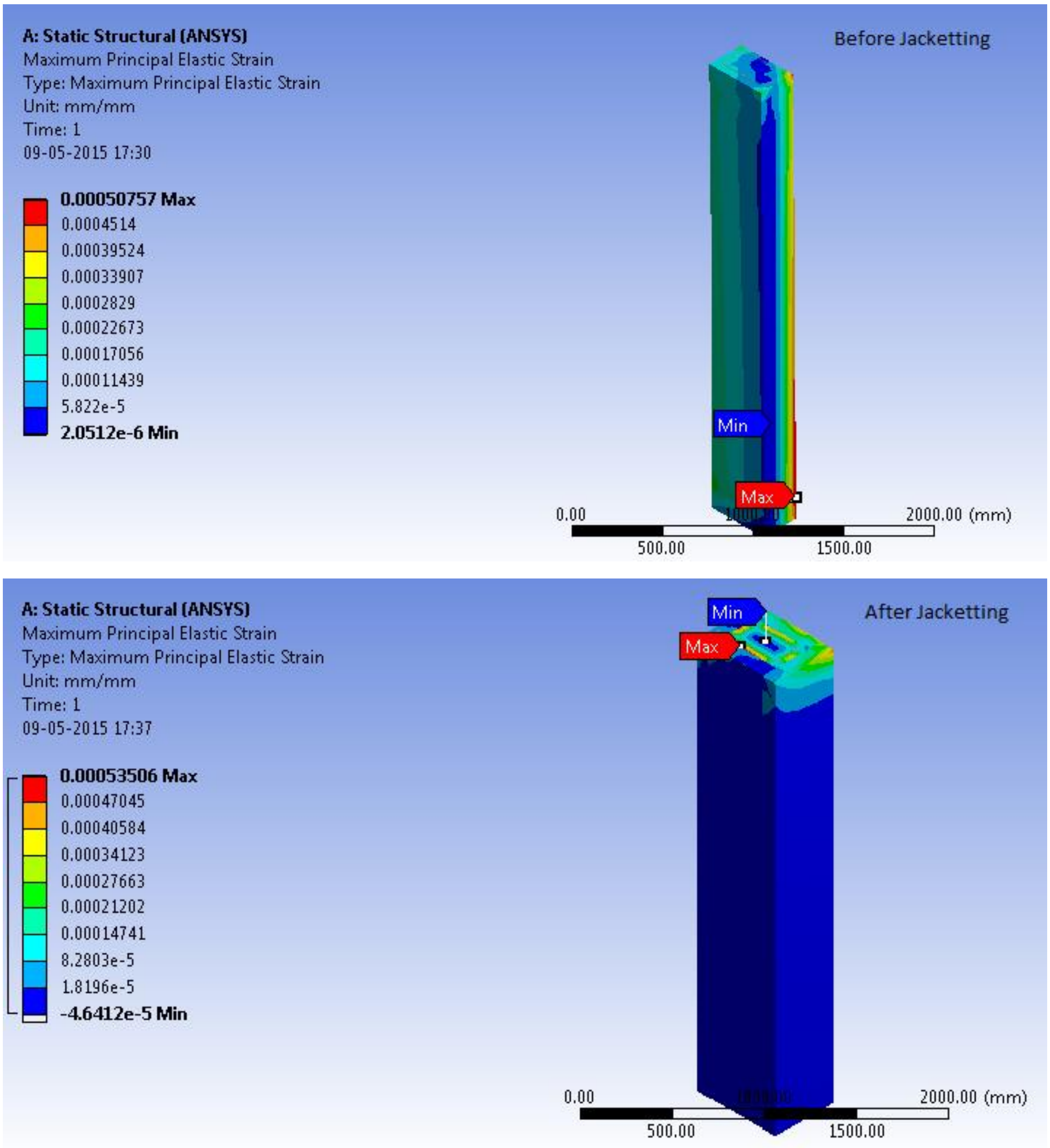


Figure 20: Maximum Principal Elastic Strain for Column 3007

Table 17: Concrete jacketting results before and after jacketting

Column number	Before Jacketting			After Jacketting		
	Deformation (mm)	Stress (MPa)	Strain (mm/mm)	Deformation (mm)	Stress (MPa)	Strain (mm/mm)
1007	2.96	59.4	0.00020	1.670	52.192	0.0003
2007	21.326	20.575	0.000647	0.231	31.43	0.002
3007	17.192	15.034	0.000507	0.051	13.572	0.000530
4009	13.408	8.879	0.0003	4.291	22.463	0.0001
5009	12.399	14.009	0.000406	0.0573	14.407	0.000483

It can be seen that total deformation, Principal maximum stress and strain values have been considerably reduced after jacketting.

CHAPTER -8

PERFORMANCE OF BUILDING AFTER ADDING SHEAR WALL

8.1 Introduction

This chapter includes the retrofitting of the building by adding shear wall. One of the most common methods to increase the lateral strength of the reinforced concrete building is to make a provision for additional shear walls. The performance of the building is analysed using STAAD PRO v8i.

8.2 STAAD model

The building is modelled and designed in STAAD PRO v8i. Shear wall is modelled as a surface element in the 5m and 4.75m bay as shown in the figure 21. Shear wall is added to the building as a surface plate. It is added on all the four sides of the building. The building is analysed and designed by IS 456:2000, Indian Standard Plain and Reinforced Concrete - Code of Practice.

8.2.1 Best location of adding shear wall

Using STAAD Pro v8i, shear walls were added in the different bays of the building. For example, it was added in the 2.8m bay on both the sides of the building. However, it did not improve Earthquake resistance of the building. Then the shear wall was added in the 5m and 4.75m bay simultaneously in order to maintain symmetry and for uniform distribution of forces. Figure 21 and 22 shows the location of shear wall in the building. The blue lines highlighted in Figure 22 shows the location of shear wall.

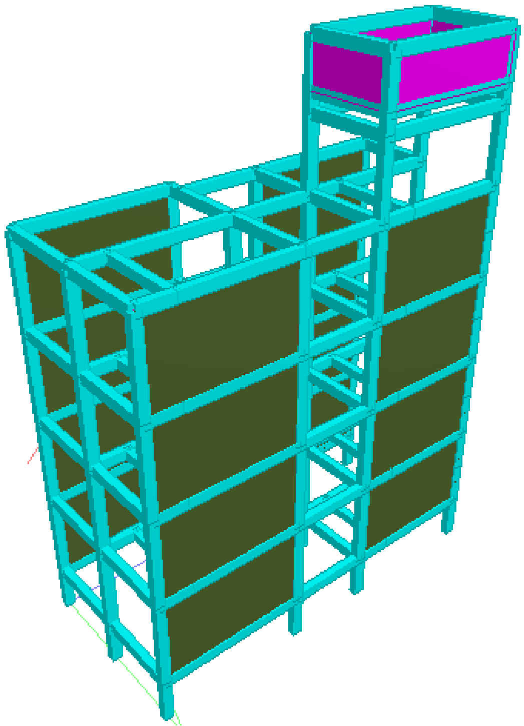


Figure 21: Addition of shear wall

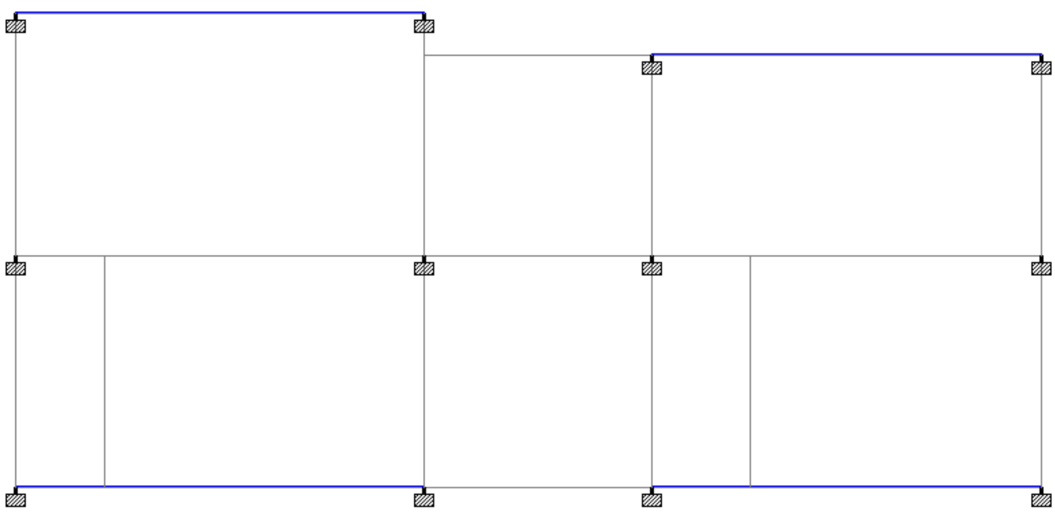


Figure 22: Plan of the building showing location of shear wall

8.3 Analysis results and comparision with the original building

The provided reinforcements of the existing columns came out to be more than the required reinforcements. Hence, now the design is safe for all the columns and beams of the building. The storey drifts are calculated and compared with the storey drifts of the existing building. A comparision of the reinforcements are tabulated in the table 18. A comparision of the storey drifts before and after jacketting is shown in the table 19. For example, Column number 1007 which was earlier found to be deficient is now safe in terms of area of steel provided. The provided reinforcement comes out to be 1425.92 mm^2. However, the required area of steel is 1608 mm^2. After the comparision of the storey drifts at each floor level, it was found that after performing jacketting the storey drift was considerably reduced. Percentage reduction is checked in each case. For example at storey height 1.5 m and load combination 27, the percentage reduction in storey drift came out to be 23.52 %. Similarly, at storey height 4.5m and load combination 27, the percentage reduction in storey drift came out to be 99%.

Table 18: Required and provided area of steel after addition of shear wall

Sno	Column Size	Area Of Steel	Area Of Steel	Area Of Steel	Area Of Steel	Area Of Steel	Area Of Steel
1	250x350	634.57	495.13	830.51	616.32	800	Nil
Prov.		1256	1256	804	804	804	
2	250x350	639.06	285.33	626.44	500.26	900	Nil
Prov.		1608	1608	904	904	904	
3	250x350	765.46	290.11	583.87	506.83	789.98	Nil
Prov.		1608	1608	904	904	904	
4	250x300	530.43	237.13	821.82	747.63	800	Nil
Prov.		1256	1256	804	804	804	
5	250x350	1127.72	1510.25	900	659.99	900	Nil
Prov.		1608	1608	904	904	904	
6	250x400	700.02	683.79	504.23	317.05	557.70	Nil
Prov.		1608	1608	904	904	904	
7	250x400	1425.92	1600	903.29	461.28	307.86	862.14/ 46.53
Prov.		1608	1608	904	904	904	904
8	250x350	1600.00	1327.93/1513.30	847.99/500.44	416.01/368.81	287.03/270.75	900.88/42.73
Prov.		1608	1608	904	904	904	904
9	250x300	588.34	501.10	800	708.49	800	Nil
Prov.		1256	1256	804	804	804	
10	250x350	745.57	291.52	435.53	421.50	759.84	Nil
Prov.		1608	1608	904	904	904	
11	250x350	1470.00	444.21	448.71	470.30	786.56	534.02/124.16/42.37
Prov.		1608	1608	904	904	904	904
12	250x350	1610.00	805.41/307.63	520.25/475.01	527.24/505.97	586.70/371.06	472.36/169.30/43.46
Prov.		1608	1608	904	904	904	904

Table 19: Required and provided area of steel after addition of shear wall

Storey	Height (m)	Load Combination	Before Jacketting Drift (X) mm	After Jacketting Drift (X)mm	% Reduction in storey drift	Allowable mm (x) (0.004 times the storey height)	Before Jacketting Drift (Z)mm	After Jacketting Drift (Z)mm	% Reduction in storey drift
1	1.5	27	0.923	0.706	23.52	6	-	-	-
		28	0.847	0.672	20.67	6	-	-	-
		29	-	-	-	6	1.202	0.784	34.77
		30	-	-	-	6	1.196	0.826	30.94
3	4.5	27	2.58	0.022	99	12	-	-	
		28	2.25	0.052	97	12	-	-	
		29	-	-	-	12	3.09	0.33	97.6
		30	-	-	-	12	2.59	0.2810	32.23
5	7.5	27	2.434	0.072	97	12	-	-	
		28	1.871	0.090	95	12	-	-	
		29	-	-	-	12	2.468	0.302	87.7
		30	-	-	-	12	1.361	0.238	82.51
7	10.5	27	1.919	0.130	93.22	12	-	-	
		28	1.475	0.136	90	12	-	-	
		29	-		-	12	1.568	0.264	83
		30	-		-	12	0.468	0.40	14
9	13.5	27	0.982	0.244	75	12	-		
		28	0.854	0.193	77	12	-		
		29	-	-	-	12	0.188	0.17	10
		30	-	-	-	12	0.650	0.135	79
10	15.9	27	2.311	2.00	13	12	-	-	-
		28	1.297	1.15	11.3	12	-	-	-
		29	-	-	-	12	4.23	2.200	47
		30	-	-	-	12	3.48	3.01	13.5
11	16.5	27	0.136	0.110	19.10	12	-	-	-
		28	0.107	0.09	16	12	-	-	-
		29	-	-	-	12	0.230	0.161	30
		30	-	-	-	12	0.147	0.120	18
12	18	27	0.055	0.040	27.27	12	-	-	
		28	0.031	0.014	55	12	-	-	
		29	-	-		12	0.141	0.006	95
		30	-	-		12	0.093	0.070	25

CHAPTER -9

PERFORMANCE OF BUILDING AFTER STEEL JACKETTING

9.1 Introduction

This chapter includes the retrofitting of the building using steel jacketting. In order to prevent loss of human life and property due to future earthquakes, the steel jacket retrofit has been used as a method to enhance the shear strength and ductility of reinforced concrete (RC) columns.

9.2 Analysis and Design using ANSYS V 12.0

The deficient existing RC columns are modelled and designed in ANSYS v 12.0. The properties of the steel jacket provided as input in the ANSYS file is given in table 13. The thickness of the steel plate varied from 4mm to 10mm. However, steel plate with 5mm thickness provided the best result in terms of total deformation, Maximum Principal Stress and Maximum Principal Strain. Figure 23 show the top view of column 3007 in which green colour lines shows the location of 5mm steel plates, white line shows the location of transverse reinforcing bar, and the grey colour indicates concrete. Figure 24 show the steel jacketting in ISO view.

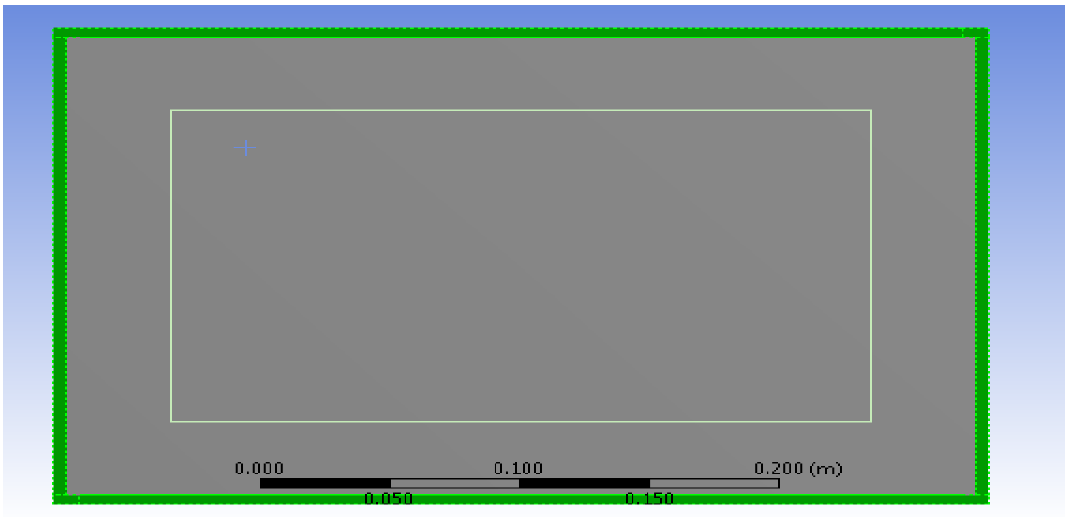


Figure 23: Column number 3007 with steel jacketting

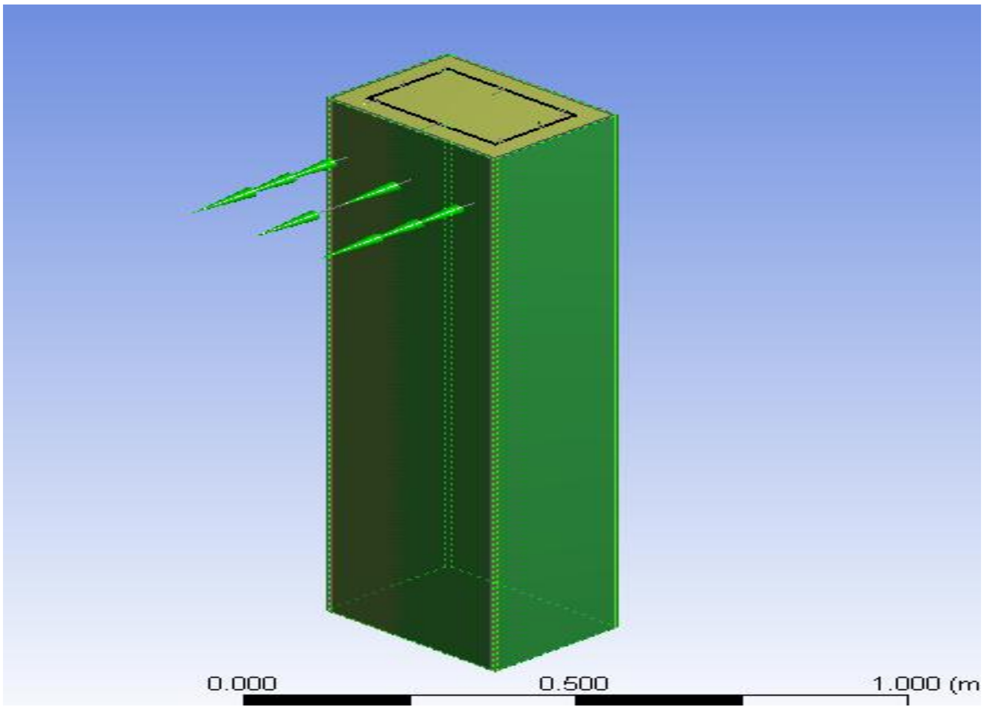


Figure 24: Column with steel jacketting (Iso view)

9.3 Results

After performing the steel jacketting for the deficient columns, the lateral load carrying capacity is significantly improved. It can be observed that the total deformation value before jacketting was 17.192 mm in column number 3009. However, after applying 5mm steel jacket to all four sides of the column, deformation value got reduced to 0.0681. The values of total deformation, maximum principal stress and strain are tabulated in table 20. The result values of column number 3007 before and after steel jacketting is shown in Figures 25, 26 and 27.

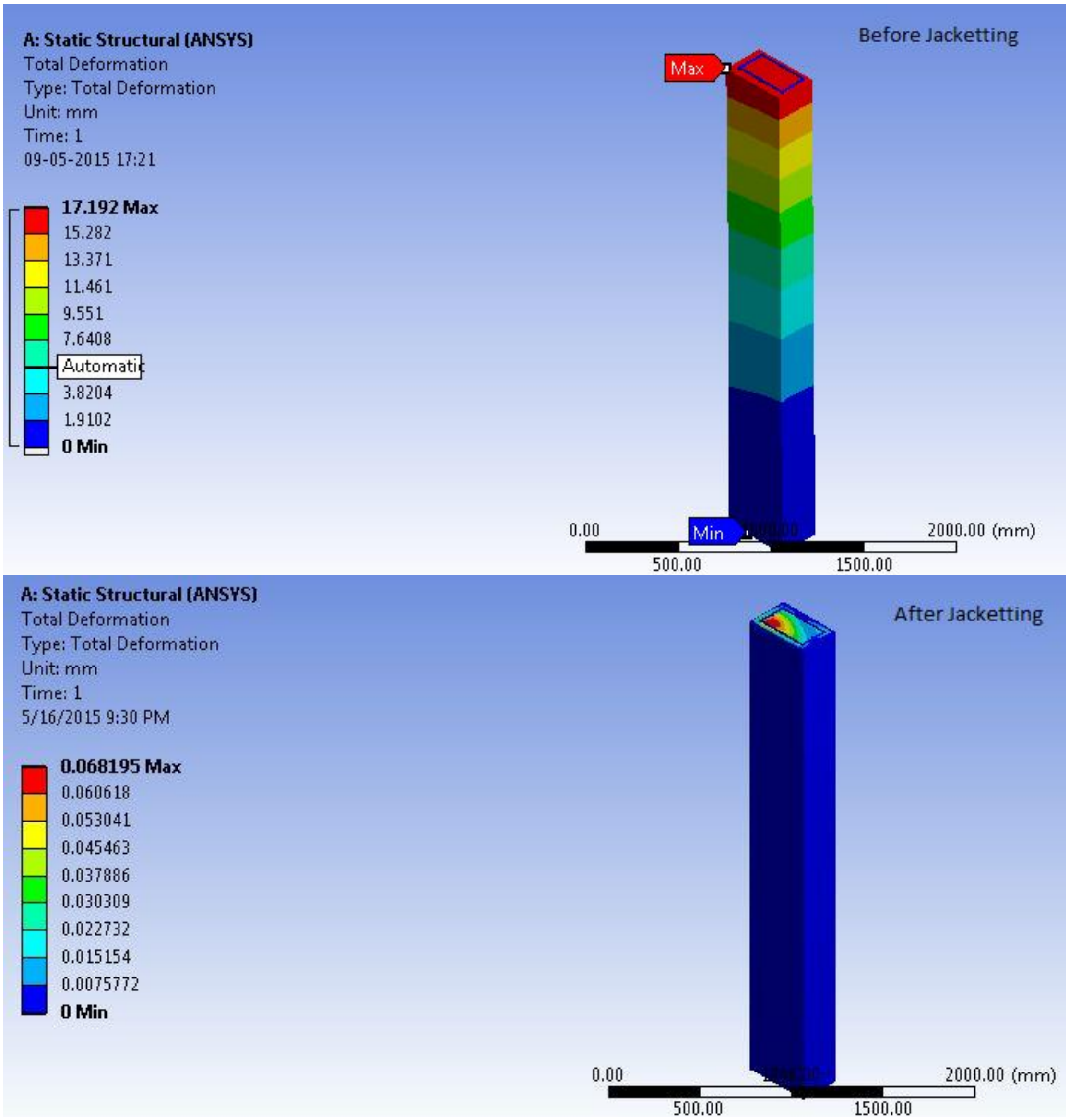


Figure 25: Deformation value before and after steel jacketting (column 3007)

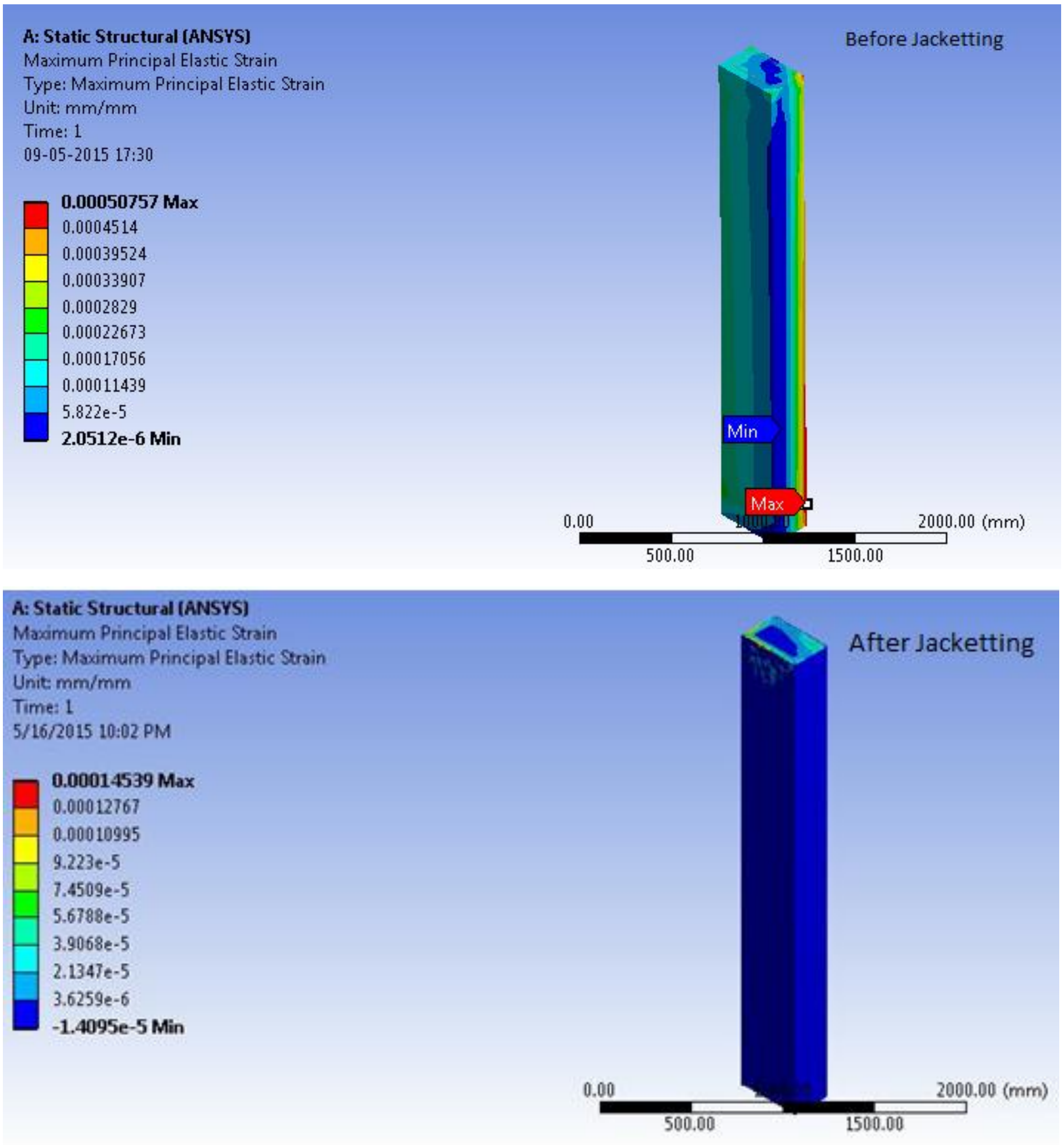


Figure 26: Maximum Principal Elastic Strain after steel jacketting (Column 3007)

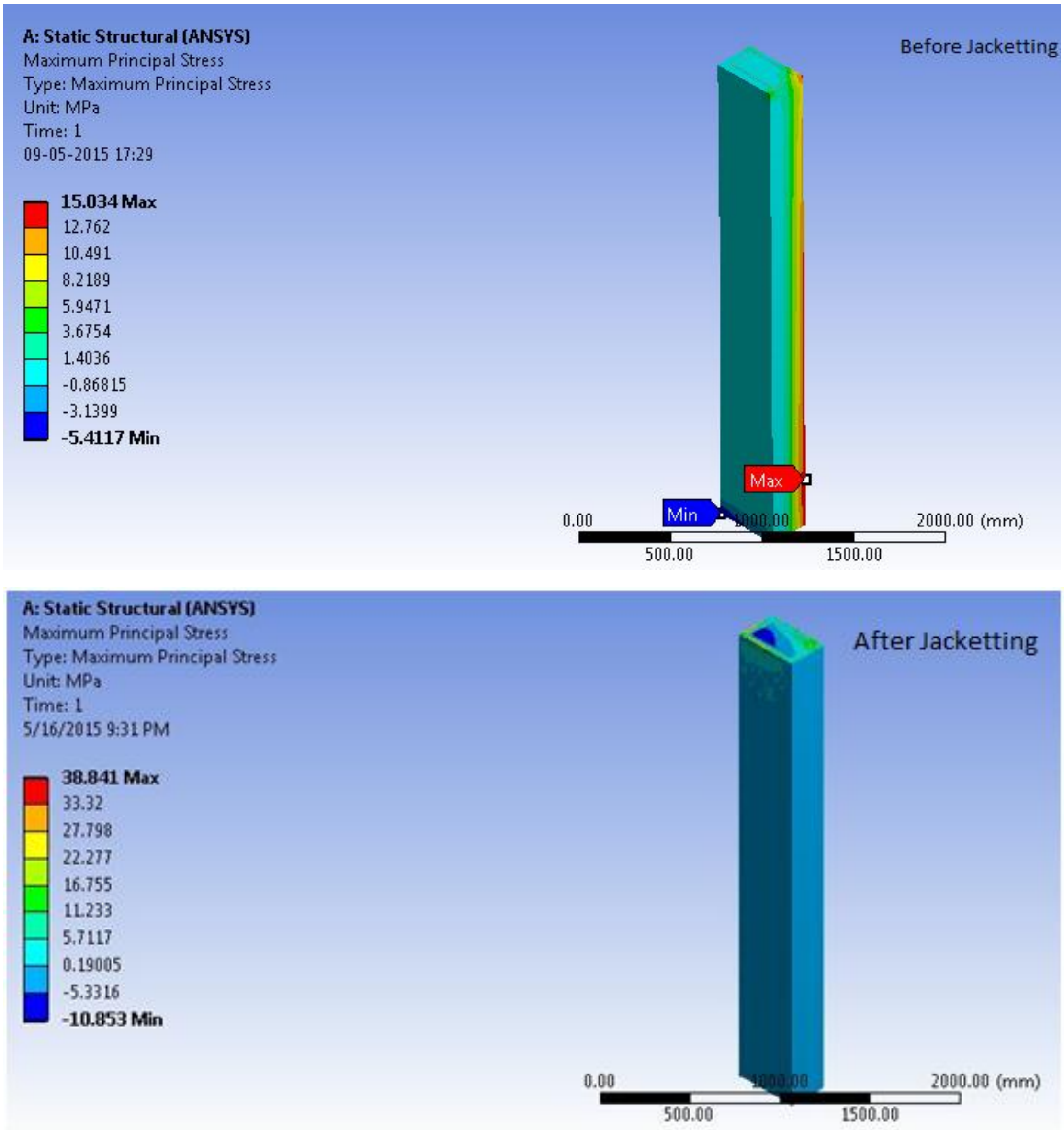


Figure 27: Maximum Principal Stress for Column 3007 after steel jacketting

Table 19: Steel jacketting results before and after jacketting

Column number	**Before Jacketting**			**After Jacketting**		
	Deformation (mm)	**Stress (MPa)**	**Strain (mm/mm)**	**Deformation (mm)**	**Stress (MPa)**	**Strain (mm/mm)**
1007	2.96	59.4	0.00020	1.489	58.501	0.0002
2007	21.326	20.575	0.000647	0.145	28.139	0.0017
3007	17.192	15.034	0.000507	0.051	13.572	0.000530
4009	13.408	8.879	0.0003	4.73	17.172	0.0003
5009	12.399	14.009	0.000406	0.0573	14.407	0.000483

CHAPTER -10

PERFORMANCE OF BUILDING AFTER FRP JACKETTING

10.1 Introduction

This chapter includes the retrofitting of the building using Fibre reinforced polymer jacketting. In order to prevent loss of human life and property due to future earthquakes, the steel jacket retrofit has been used as a method to enhance the shear strength and ductility of reinforced concrete (RC) columns.

10.2 Analysis and Design using ANSYS V 12.0

The deficient existing RC columns are modelled and designed in ANSYS v 12.0. The properties of the FRP jacket provided as input in the ANSYS file is given in table 13. The thickness of the FRP plate is chosen as 5mm. 5mm FRP thickness provides the best result in terms of total deformation, Maximum Principal Stress and Maximum Principal Strain when compared with 3mm and 6mm FRP sheet. Figure 28 show the top view of column 3007 in which green colour lines shows the location of 5mm FRP sheet, white line shows the location of transverse reinforcing bar, and the grey colour indicates concrete. Figure 29 show the FRP jacketting in ISO view.

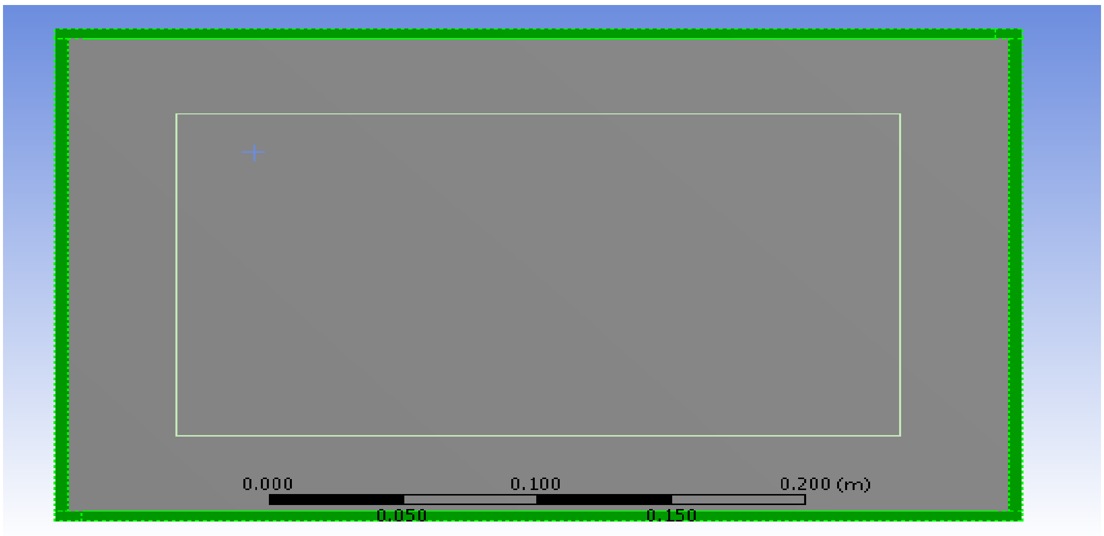


Figure 28: Column number 3007 with FRP jacketting

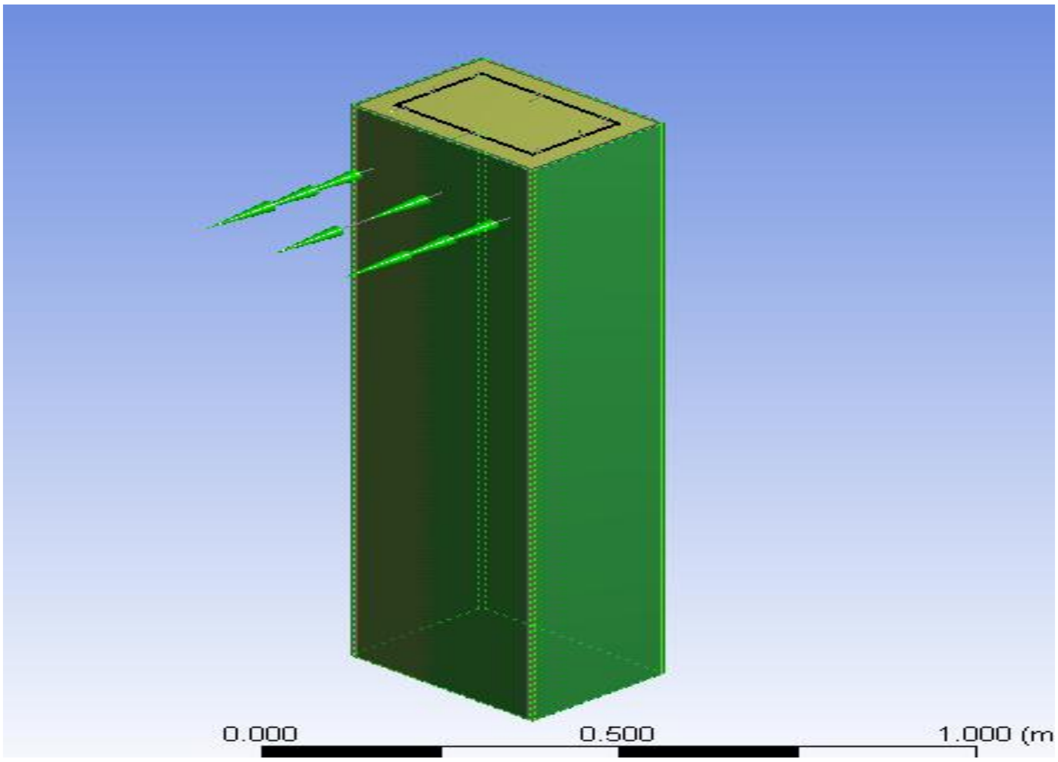


Figure 29: Column with FRP jacketting (Iso view)

10.3 Results

After performing the FRP jacketting for the deficient columns, the lateral load carrying capacity is significantly improved. It can be observed that the total deformation value before jacketting was 17.192 mm in column number 3007. However, after applying 5mm FRP sheet to all four sides of the column, deformation value got reduced to 0.00681 mm. It can be said that after using FRP sheet, deflection almost goes to zero and is effectively reduced. The values of total deformation, maximum principal stress and strain are tabulated in table 21. The result values of column number 3007 before and after steel jacketting is shown in Figures 30, 31 and 32.

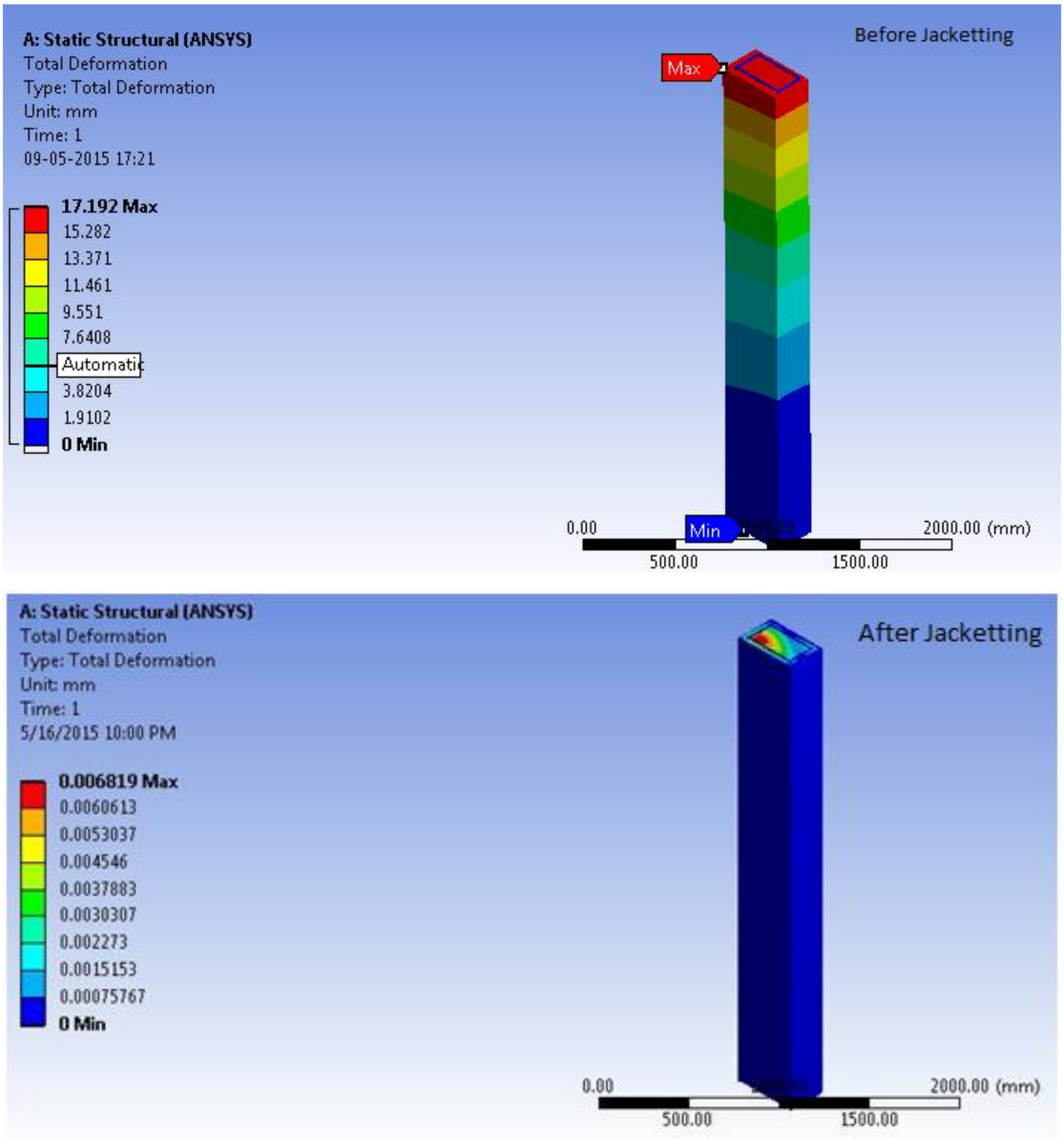


Figure 30: Deformation value before and after FRP jacketting for column 3007

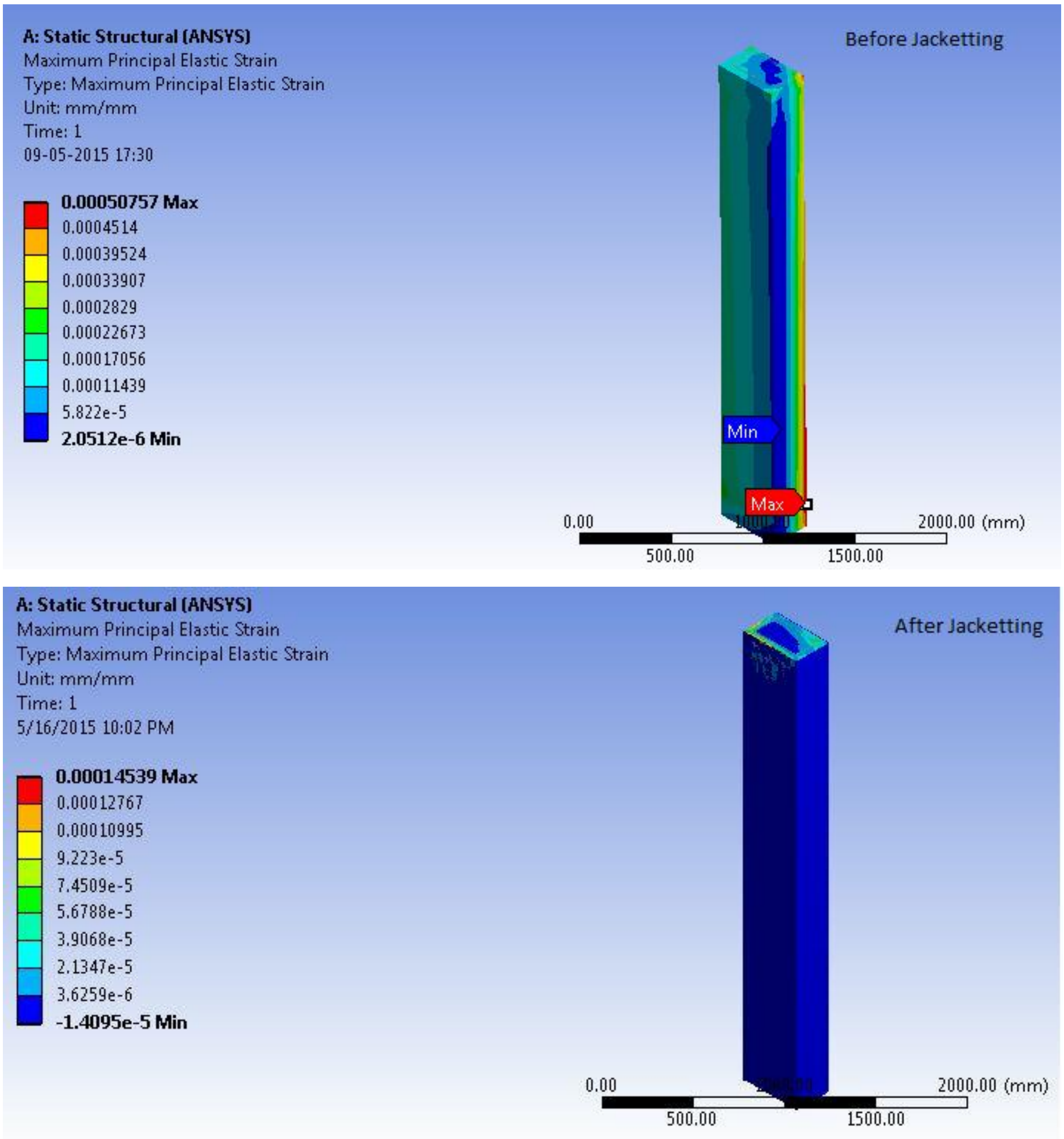


Figure 31: Maximum Principal Elastic Strain for Column 3007 after FRP jacketting

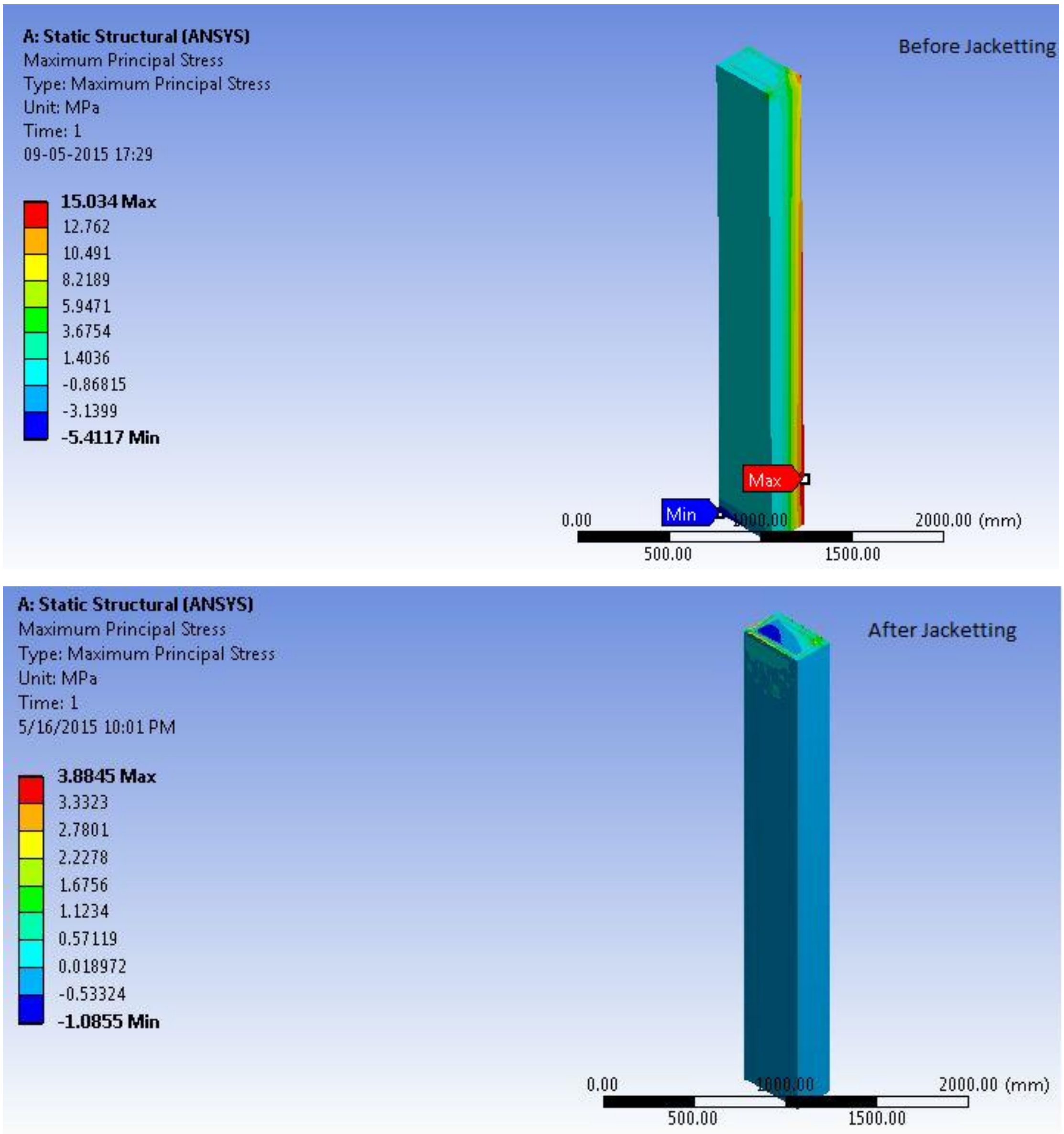


Figure 32: Maximum Principal Elastic Strain for Column 3007 after FRP jacketting

Table 20: FRP jacketting results before and after jacketting

Column number	**Before Jacketting**			**After Jacketting**		
	Deformation (mm)	**Stress (MPa)**	**Strain (mm/mm)**	**Deformation (mm)**	**Stress (MPa)**	**Strain (mm/mm)**
1007	2.96	59.4	0.00020	0.9119	44.77	0.00097
2007	21.326	20.575	0.000647	0.0817	21.793	0.00174
3007	17.192	15.034	0.000507	0.051	13.572	0.000530
4009	13.408	8.879	0.0003	2.240	9.870	0.0002
5009	12.399	14.009	0.000406	0.0573	14.407	0.000483

CHAPTER -11

COMPARISON OF RETROFITTING TECHNIQUES

11.1 Introduction

A comprehensive comparison of all techniques discussed in above chapters is given here. These techniques include RC jacketting, steel plate jacketting, FRP jacketting and addition of shear wall. For the above said building, best technique in terms of earthquake safety, economy and on-site feasibility is suggested.

11.2 Safety Comparision

The comparision of all the above techniques is made in terms of Average storey drifts, load and moment carrying capacities, deformations, Maximum principal elastic stress and Maximum principal strain.

11.2.1 Average Storey drifts

The average storey drift in x-direction after performing RC jacketting came out to be 27.2%. However, after the addition of shear wall, the average storey drifts in x direction is 56.82%. The storey drifts are effectively controlled by adding shear wall.

11.2.2 Loads and Moment Carrying Capacities

After performing jacketting, the load and moment carrying capacity has been significantly increased. Considering Column number 3007, the load carrying capacity before jacketting was 1325.15 KN. However, the capacity after performing RC jacketting increased to 2660.79 KN. Moreover, the moment carrying capacity (Mz) before jacketting was 79.78 KN-m. However, the capacity after performing jacketting increased to 213.55 KN-m. Thus, it can be concluded that jacketting the building or by using any such retrofitting method can guarantee the safety of the building and its occupants.

11.2.3 Deformations

The most important factor while designing a building earthquake resistant should be the control of storey drifts. It can be seen from the results of ANSYS files, that there is a significant control of storey drifts after performing RC, steel and FRP jacketting. In column number 3007, it can be seen that the deformation almost get reduced to zero in case of FRP jacketting. The value gets reduced from 17.192 *mm* to 0.0068 *mm*. Moreover, after applying steel jacket, the deformation value decreased from 17.192 *mm* to 0.0681 *mm*.

11.2.4 Maximum Principal strain and Maximum Principal Stress

The value of Maximum principal stress gets reduced from 15.034 MPa to 3.8845 MPa after applying FRP jacket for column 3007. Also, Maximum principal strain is decreased from 0.000507 to 0.00145. It is observed that in column number 5009, the value of maximum principal stress before jacketting was 14.009 MPa. However, the value after RC jacketting came out to be 14.407 MPa which showed no significant reduction. Jacketting with steel and FRP however did gave noteworthy results.

11.3 Cost Comparison

Quantities of concrete, steel, shear wall, FRP and steel have been calculated separately. Total quantity of concrete in columns for the existing building and the building with RC jacketting is taken from the results of STAAD file. However, the quantity of steel has been calculated manually. This is well elaborated in table number 22 and 23. Total quantity of steel calculated for the original file was 1400 Kg. However, after RC jacketting, the value increased to 3300 Kg. The quantity of concrete and the steel used in shear wall has been taken from STAAD file results. The Various methods proposed for Retrofitting of Framed Structure is given in Table 24. Approximate quantity of steel in one shear wall is 350 kg and Total quantity of reinforcement in 16 shear walls will be 350x16 = 5600kg. For cost of concrete refer [21] clause 5.1.2 (page 96) of Delhi schedule of rates (DSR), Government of India, Central Public works department, 2014 = Rs 6230/cum For cost of reinforcement refer [21] clause 5.22.6 (page 99) = Rs 68.10/kg. For cost of steel jacketting, refer clause 10.1 @ Rs 67.70. After getting all the rates of each item, A Comparison after applying various methods for Retrofitting of Framed structure is made and shown in table 25. Cost of application and fixing of FRP sheet is taken from the Construction Company Fosroc as Rs 4800/m^2

Table 21: Column steel calculated for the original file

COLUMN STEEL CALC. TABLE

Col. No.	size	STEEL PROVIDED							
		Fdn -1st Fl.		1st Fl. - 2nd Fl.		2nd Fl. - 3rd Fl.		3rd - Roof	
		12ø	16ø	12ø	16ø	12ø	16ø	12ø	16ø
C1	250x300	4	4	0	4	0	4	0	4
C2	250x350	0	8	8	0	8	0	8	0
C3	250x350	0	8	8	0	8	0	8	0
C4	250x300	4	4	0	4	0	4	0	4
C5	250x350	0	8	8	0	8	0	8	0
C6	250x400	0	8	8	0	8	0	8	0
C7	250x400	0	8	8	0	8	0	8	0
C8	250x350	0	8	8	0	8	0	8	0
C9	250x300	4	4	0	4	0	4	0	4
C10	250x350	0	8	8	0	8	0	8	0
C11	250x350	0	8	8	0	8	0	8	0
C12	250x350	0	8	8	0	8	0	8	0
		12	84	72	12	72	12	72	12
	length	4.5	4.5	3	3	3	3	3	3
	wt. /m	0.89	1.58	0.89	1.58	0.89	1.58	0.89	1.58
	weight	48.06	597.24	192.2	56.88	192.24	56.88	192.2	56.88

Table 22: Column steel calculated after RC jacketting

COLUMN STEEL CALC. TABLE (after jacketting)

Col. No.	size	STEEL PROVIDED							
		Fdn -1st Fl.		1st Fl. - 2nd Fl.		2nd Fl. - 3rd Fl.		3rd - Roof	
		12ø	16ø	12ø	16ø	12ø	16ø	12ø	16ø
C1	250x300	4	20	0	20	0	20	0	20
C2	250x350	0	8	8	16	8	0	8	16
C3	250x350	0	8	8	0	8	0	8	0
C4	250x300	4	4	0	20	0	20	0	20
C5	250x350	0	8	8	16	8	16	8	0
C6	250x400	0	8	8	0	8	0	8	0
C7	250x400	0	24	8	16	8	0	8	0
C8	250x350	0	24	8	16	8	0	8	0
C9	250x300	4	4	0	20	0	20	0	20
C10	250x350	0	8	8	0	8	0	8	0
C11	250x350	0	24	8	16	8	0	8	0
C12	250x350	0	24	8	16	8	0	8	0
		12	164	72	156	72	76	72	76
	length	4.5	4.5	3	3	3	3	3	3
	wt. /m	0.89	1.58	0.89	1.58	0.89	1.58	0.89	1.58
	weight	48.06	1166	192.2	739.44	192.24	360.2	192.2	360.24

Table 23: Various methods proposed for Retrofitting of Framed Structure

S.no	Type of structure	Concrete (m^3)			Reinforcement (Kg)			FRP (m^2)	Steel (5*mm* plate)
		Column	Shear Wall	Total	Column	Shear Wall	Total		
1	Framed structure (original)	15.6	---	15.6	1400	---	1400	---	---
2	Structure with RC Col. Jacketting (proposed)	30.4	---	30.4	3300	---	3300	---	---
3	Structure with Shear wall (proposed)	15.6	35.1	50.7	1400	5600	7000	---	---
4	Structure with FRP (proposed)	15.6	---	15.6	1400	---	1400	85.8	---
5	Structure with Steel Jacketting (proposed)	15.6	---	15.6	1400	---	1400	---	$0.43m^3$ or 3368 Kg

Table 24: Comparison after applying various methods for Retrofitting of Framed structure

S.no	Type of structure	Tentative Cost (Rs)				Total Cost (Rs)
		Concrete	Reinforcement	FRP	Steel	
1	Framed structure (original)	97188	95340	---	---	1,92528
2	Structure with Col. RC Jacketting (proposed)	189392	224730	---	---	4,14,122
3	Structure with Shear wall (proposed)	97188	476700	---	---	5,73,888
4	Structure with FRP (proposed)	97188	95340	411840	---	6,04,368
5	Structure with Steel Jacketting (proposed)	97188	95340	---	228013.6	4,20,541.6

11.4 On-site Feasibility

[19] Today there are several types of FRP strengthening systems namely a) wet lay-up systems; b) systems based on prefabricated elements; c) special automated wrapping systems.

a) Wet lay-up process (Figure 33) represents the most commonly used technique, in which unidirectional fibre sheets or woven fabric sheets are impregnated with resins and wrapped around columns, with the main fibres oriented in the hoop direction. Installation on the concrete surface requires saturating resin, usually after a primer has been applied. Two different processes can be used to apply the fabric (i) the fabric can be applied directly into the resin which has been applied uniformly onto the concrete surface, (ii) the fabric can be impregnated with the resin in a saturator machine and then applied wet to the sealed substrate. The wrapping can be realized continuously around the entire element or partially, using sheets of FRP disposed in spiral or in distinct sections. There can be applied variable number of layers (from same material or distinct ones), obtaining different thicknesses of the confining layer, depending on the required element strength.

Figure 33: Wet up confining FRP system

b) When prefabricated FRP jackets are used, the jackets are fabricated in half circles or half rectangles and circles with a slit or in continuous rolls, so that they can be opened up and placed around columns. This can be considered as technical most elaborated system, but the major problems emerge in the closure area of the composite layer because of insufficient overlapping.

Figure 34: Confining system based on pre-fabricated FRP elements

c) The FRP automated wrapping technique through winding of tow or tape was first developed in Japan in the early 90s and a little later in the USA. The technique, shown in Fig. 6, involves continuous winding of wet fibres under a slight angle around columns by means of a robot. Key advantage of the technique, apart from good quality control, is the rapid installation.

Figure 35: Automatic RC column wrapping

The practical technique of surrounding the old and existing columns with a new RC jacket involves a sequence of actions. [20] There are few steps which are to be followed before applying jacketting. First step is to repair the surface of old and existing column. This can be done by removing the deteriorated concrete by hand chipping, jack hammering or any other method that causes micro cracking of substrate (concrete of existing Column). This is then followed by sand blasting or water demolition technique, which makes the surface of column rough. The third and an important step is to use a bonding agent like epoxy resin. After the resin application, steel connectors are used. This is then followed by temporary shoring of existing RC columns. Finally, adding of longitudional and transverse reinforcement with steel connectors. This is how RC Column jacketting is given a practically shape.

CHAPTER -12

CONCLUSIONS AND REFERENCES

12.1 General

In India, almost all civil construction works are ne. But after decades of years from now, these constructions may needs to be strengthened and retrofitted. Old buildings, monuments, Important buildings will demand repair and retrofitting. Moreover, Retrofitting will also become essential for the following situations:

1. Buildings, which have not been designed and detailed to resist seismic forces.
2. Buildings, which were designed as per old seismic codes.
3. If the lateral strength of the building does not satisfy the seismic forces as per the revised seismic zones.
4. If the construction is apparently of poor quality.
5. If there have been changes in the occupancy of the building which may increase its vulnerability.

Buildings lying in severe seismic zones like J&K, Himachal Pradesh, Delhi etc need to have good earthquake resistant design features. The weak existing buildings of zones IV and V need retrofitting. IS 1893: 2002 must be inculcated along with IS 456:2000 to make an Earthquake resistant Building. IS 15988:2013 is the code for seismic strengthening of buildings, which is a new code, but designer should be well aware of its guidelines before designing a RC jacket of column.

12.2 Conclusions

After performing RC, Steel and FRP jacketting, the merits and de-merits of each of these techniques can be fully interpreted. RC jacketting though increases the column size but at the same time, increases the lateral load carrying capacity and significantly decreases the storey drifts at each floor but, some damage to concrete cover is inevitable in this work. However, in case of FRP jacketting, no damage to the existing building is required. There is no significant increase in the size of the column by FRP jacketting. This technique controls the deflection, stress and strain up to a maximum extent. Steel jacketting involves welding of the steel plates to the reinforcement of existing concrete column. The technique of adding new shear walls is often taken as a simple solution for improving seismic performance. Therefore, it is frequently used for retrofitting of non-ductile reinforced concrete frame buildings. A reasonable structural ductility may be achieved if the wall is properly designed with a good detailing. The connection to the existing to the structure has to be carefully designed to guarantee shear transfer. However, shear wall causes addition of dead load on the structure. Indian Standard code is available for RC jacketting of the columns. However, research is still going on in the field of FRP. FRP jacketting reduces the deformation, stress and strain to a significant value when compare with RC and steel jacketting. In column number 3007, it can be seen that the deformation almost get reduced to zero in case of FRP jacketting. The value gets reduced from 17.192 *mm* to 0.0068 *mm*. The value of Maximum principal stress gets reduced from 15.034 MPa to 3.8845 MPa after applying FRP jacket. Also, Maximum principal strain is decreased from 0.000507 to 0.00145. However, this is not true with steel jacketting in which stress increased from 15.034 MPa to 38.841 MPa. The value of deformation get reduced from17.192 mm to 0.0681 mm. But the value of maximum principal strain came out to be same as that in case of FRP jacketing. It is observed that in column number 5009, the value of maximum principal stress before jacketting was 14.009 MPa. However, the value after RC jacketting came out to be 14.407 MPa which showed no significant reduction. Jacketting with steel and FRP however did gave noteworthy results. FRP Jacketing, being light in weight, does not increase seismic weight of building but it improves lateral strength considerably. Steel jacketing, have a comparable cost with RC jacketing but giving better results. It is lighter in weight too. Hence, FRP and steel jacketing prove to be the best technique for retrofitting of weak concrete columns. FRP and steel jacketting provides more protection whereas RC jacketing is more economical.

12.3 Scope for future work

Retrofitting of beam column joint is equally important in terms of retrofitting of the existing building. Along with the retrofitting of existing columns, strengthening of beam-column joint is likewise vital.

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APPENDIX-A

DRAWING DETAILS

APPENDIX-B

PUBLICATIONS FROM THIS THESIS

[1] Nikita Gupta, Poonam Dhiman, Anil Dhiman, *"Design and Detailing of RC Jacketting for Concrete Columns"*, IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE) e-ISSN: 2278-1684, p-ISSN: 2320-334X. PP 54-58, 2015.

[2] Nikita Gupta, Poonam Dhiman, Dr. Ashok Kumar Gupta, *"Case Study: Retrofitting of an Existing Residential Building by Using Shear Wall"*, Journal of Basic and Applied Engineering Research, ISSN. 2350-0255, PP582-586, 2015.

[3] Nikita Gupta, Aditya Tiwary Poonam Dhiman, Dr. Ashok Kumar Gupta *"Advanced Retrofitting Techniques for Reinforced Concrete Buildings"*, Journal of Basic and Applied Engineering Research, ISSN. 2350-0255, PP 587-592, 2015.

[4] Nikita Gupta, Aditya Tiwary, Poonam, "*Preliminary Seismic Survey of Nepal Earthquake"* (Communicated)

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