SEISMIC RETROFITTING OF DAMAGED TWO STOREY RC FRAMED SCHOOL BUILDINGS

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

DISASTER MITIGATION AND MANAGEMENT

By

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MAY, 2014

CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in the dissertation entitled, "**SEISMIC RETROFITTING OF DAMAGED TWO STOREY RC FRAMED SCHOOL BUILDINGS**" in partial fulfillment of requirement for the award of the degree of **Master Of Technology** in **Disaster Mitigation and Management**, submitted in the Centre of Excellence in Disaster Mitigation and Management, IIT Roorkee, Roorkee, is an authentic record of my own work carried out under the supervision of **Dr. D. K. PAUL,** Emeritus Fellow, Department of Earthquake Engineering, IIT Roorkee, Roorkee.

The matter embodied in this report has not been submitted by me for the award of any other degree or diploma.

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CERTIFICATE

This is to certify that above statement made by the candidate is correct to the best of my knowledge.

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ACKNOWLEDGEMENT

I wish to express my deep sense of gratitude and sincere thanks to my dissertation supervisor **Prof. & Dr. D. K. Paul**, Emeritus Fellow, for their intuitive and careful guidance, timely suggestions and constant encouragement throughout the course of the dissertation work. His every intervention has motivated me towards working hard and achieving my goals during the course of work.

I wish to thank **Dr. Ajay Gairola**, Head, Centre of Excellence in Disaster Mitigation and Management, IIT Roorkee, for his help in best possible manner for carrying out my work.

I would like to thank Mr. K. S. Chidambram and Mr. Dhiraj Raj for their help and valuable knowledge sharing.

I am also thankful to the entire faculty, my family and my friends for helping me whenever I needed.

ABSTRACT

Many existing RC frame buildings in India have not been designed at all according to Indian standards IS 1893-2002. To achieve required performance level of building during earthquake, earthquake resistant design features need to be considered. These buildings lack in strength, stiffness and ductility. In past earthquakes many buildings have collapsed and became life threatening to occupants of buildings. It is not economically feasible to demolish all such buildings and construct earthquake resistant building in their place. Retrofitting is one of the techniques by which strength, ductility and stiffness of building can be enhanced. Retrofitted structures performed well during earthquakes.

School Building have given special occupancy classification according to IS 1893:2002. These buildings are important from life safety point of children and due to post earthquake importance of these structures. In post earthquake phase, school building serve as relief centers or used as temporary shelter for disaster affected peoples. But many of these school buildings are not designed as per Indian standards to resist seismic forces. Retrofitting of these buildings is necessary to ensure safety of such buildings against future earthquake forces.

As a case study of retrofitting of RC frame school building blocks Kendriya Vidyalaya, Gangtok, has been considered in this dissertation. These building blocks were constructed in 1985. The blocks are asymmetric in plan and constructed on the hill terrain. Slopes have retained by retaining walls. Extensive cracks have appeared in some blocks due to lateral movement of soil. Major cracks have appeared in some of the masonry infill and columns of building blocks after 18th September 2011 earthquake. Since Gangtok has also experienced frequent earthquakes in recent past, building blocks safety has been evaluated against the Design Basis Earthquake (DBE) and Maximum Credible Earthquake (MCE).

A three dimensional analytical model for the building blocks have been developed in SAP 2000, software for simulation of behaviour under gravity and earthquake loadings. To evaluate the effect of infill on building during earthquake infilled frame modelling and analysis is also carried out. Load and load combinations have been considered as per relevant IS codes. Building blocks have been analyzed for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE). Nonlinear Static Pushover Analysis has been done to find performance of the buildings. Building blocks have Immediate Occupancy (IO) level at Performance Point under DBE. For MCE plastic hinges have been observed in the range of IO-LS while

masonry infill panels show cracks. Hinges have been found in columns of blocks which show deficiency of columns against lateral forces. Since building blocks are non-ductile reinforced concrete structures so columns are found to be deficient. After observing results of performance point for earthquakes, retrofitting techniques have been suggested.

To enhance lateral load capacity of blocks global retrofitting techniques such as addition of shear wall can be done. Since most of the columns are damaged so member retrofitting can also achieve IO level performance for MCE level seismic demand. Member level retrofitting is done in the form of Fibre Reinforced Polymer (FRP) jackets and thickness of jacket for each damaged column has been worked out.

In this study, it has been observed that contribution of infill in lateral load resistance is substantial. Strength and stiffness of infill have effect on performance point and time period of buildings. Increased performance has been obtained due to inclusion of masonry. To improve in plane strength of building repair methods for infill are also suggested in the study.

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DEFINITIONS

Retrofitting: It is a technique to up-grade performance of building against seismic forces by enhancing strength, ductility and stiffness.

Design Basis Earthquake (DBE): Ground motion with a 10 % chance of being exceeded in 50 years having a return period of 475 years.

Maximum Considered Earthquake (MCE): Ground motion with a 5 % chance of being exceeded in 50 years having a return period of 2475 years.

Performance Point (PP): It is intersection point of capacity curve and demand curve, representing damage state of structure.

Immediate Occupancy (IO): Damage state where risk of life is very low, no permanent drift occurs and structure substantially retains original strength and stiffness.

Life Safety (LS): Moderate damage occurs, risk of life is high and some residual strength and stiffness left in all stories.

Collapse Prevention (CP): Severe damage, greater risk of life, large permanent drift, and little residual strength and stiffness.

Chapter 1

INTRODUCTION

1.1 GENERAL

In our country a large number of buildings have built without earthquake resistant features. Whenever earthquake of moderate to large intensity hit the area, these buildings become threat to life safety of people. Many of the past earthquake events have recognized the need of retrofitting of these buildings as a measure to mitigate risk of life from building collapse. Among all type of occupancy of buildings, retrofitting of school buildings is serious concern for society, since life safety of children is a major issue during severe earthquake. In pre and post earthquake scenario, it has also been seen that school buildings require better performance than other structures. During the earthquake disaster children are more vulnerable and they are unable to save their lives from impact of seismic events. In post disaster scenario these school buildings are used as relief distribution centers. These structures also used as a temporary shelter for people who lost their house during earthquake. Unfortunately past events of earthquake have shown poor performance of school building. Bhuj earthquake of 2001 resulted in roof collapse of one third of school buildings [11]. It is now very well known that "Earthquake do not kill the people, Building collapse results in loss of lives". Loss of lives of children due to collapse of school buildings can be reduced through up-gradation of these structures.

In 2011 Sikkim earthquake, around 23 school buildings were completely damaged [14] and many of the school buildings were partially damaged. Many of the buildings constructed before 1960 were not designed according to earthquake resistant design features. And before 2002, many of the earthquake resistant criteria like ductile detailing were not included in the Indian Standard code. To ensure safety of these buildings which were designed as per earlier code, retrofitting is necessary. Retrofitting techniques can enhance strength, stiffness and ductility of the buildings and desired performance level can be achieved against seismic forces.

1.2 BUILDING STUDIED

Studied School building is Kendriya Vidyalaya, Gangtok, consisting of three blocks namely Block II, Block III (Primary School) and Block IV. These building blocks are constructed in 1985. All the school building blocks i.e. II, III and IV are double storey. Design drawings of blocks II and IV are almost identical but uses are different. Block III is different in plan from block II and IV. In Block II and IV, cracks have appeared in columns and masonry walls. In Primary school building i.e block III minor cracks have been seen in masonry walls. Reason of appearance of these cracks may be the event of 18th September 2011 earthquake. But during site investigation it has also been observed that distress in blocks can also be due to lateral movement of retaining walls which has caused foundation spreading of column foundation. Since the nature of crack appeared in blocks are not similar so detailed modelling and analysis of each block has been done.

According to IS 1893, Gangtok comes under seismic zone IV. It means Peak Ground Acceleration (PGA) for maximum considerable earthquake is 0.24g. On 18th September 2011, city has experienced earthquake of magnitude M_w 6.9[25]. In this event variation of PGA was above 0.15g to 0.20g. During this earthquake, building blocks have suffered damages in form of cracks in masonry infills, columns and beams of many locations.

1.3 OBJECTIVES

1. To develop 3D analytical models of Block II, Block IV and Primary School Block.
2. To study structural behaviour of each building blocks under earthquake loading.
3. To study seismic response of building blocks obtained from Modal Analysis and Nonlinear Static Pushover Analysis. Also compare the actual performance of building during earthquake with analytical result.
4. To study effect of infill panel on strength and stiffness as compared to bare frame for each block.
5. To find out performance of school buildings for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE).
6. To identify reason of damages by the study of location of hinges and their performance levels.
7. To suggest the retrofitting measures for building blocks to ensure seismic safety from future earthquakes.

1.4 SCOPE OF STUDY

School building blocks have been modelled based on specification given in design drawings and Indian standards. No detailed investigation of in-situ materials has been done. Building blocks has been analyzed for DBE and found safe. However cracks have been observed in building blocks. Reason of damages can be foundation spreading due to movement of retaining wall, poor quality of construction material, or higher value of spectral acceleration during the earthquakes compare to DBE. During site investigation, it has been observed that lateral ties are not provided at location in some of the column. So detailed analysis is also done for MCE, and damages are observed in the columns. And some of the columns are found deficient in providing demand to achieve Immediate Occupancy (IO) level performance due to yielding beyond IO level. Nonlinear Static Pushover analysis has been done for the damage assessment and evaluation of strength. Based on analysis result, retrofitting is suggested in form of FRP jacketing.

LITERATURE REVIEW

2.1 GENERAL

In the various literatures it has been described that seismic deficient structures suffer minor to extensive damage depending on severity of earthquakes. To retrofit these existing structures many conventional and unconventional methods are described in literature. Based on available literature, damages in past earthquakes and issues related with different retrofit methods are reviewed and presented herein.

2.2 DAMAGES IN PAST EARTHQUAKES

Mishra [24] had reported huge damages to residential and institutional buildings in Bhuj 2001 earthquake. In Bhuj 2001, numbers of collapsed and damaged dwellings units were 215,255 and 928,369 respectively [24]. EERI special earthquake report [23] had mentioned that 2 district hospitals (at Bhuj and Gandhidham), 21 Community Health Centres, 48 Primary Health Centres, 227 sub-centres were completely destroyed, about 1,884 school buildings collapsed, and 9,593 primary school damaged or destroyed [24]. According to Murthy and Seth [25], about 600 school buildings have suffered extensive damage or collapse.

2.3 SEISMIC EVALUATION OF EXISTING STRUCTURES

Freeman [13] had proposed Capacity Spectrum Method (CSM) to obtain performance point of structure under seismic load.

ATC-21 [5] mentioned about Rapid Visual Screening (RVS) method for quick assessment of building. Through RVS potential damage areas or member can be identified by external viewing of structures based on about fourteen structural criteria.

FEMA-178 [14] had proposed quick method of assessment for structural vulnerability.

FEMA-273 [15] provided guidelines for rehabilitation of structures based on performance objectives.

ATC-40 [6] provided complete guidelines for evaluation of existing structures based on performance objectives and also described retrofit strategies according to required performance level.

Bracci *et al.* [7] had proposed an advanced capacity spectrum method to compute maximum seismic demand and capacity using concept of adaptive pushover analysis.

Fajfar [12] had described complete review on evaluation of CSM and developed N2 method in which performance of structure is defined by comparing the capacity curve with inelastic response spectra of seismic event and related structural inelastic dissipation.

FEMA 356 [16] defines the performance level using force deformation curve of a member and accordingly their limit for acceptance in evaluation.

SAP 2000 analysis software uses global approach for design and consisted of nonlinear behaviour of section where deformation are expected to be high, called as Plastic Hinges and their behaviour is represented by moment rotation curve which is a main basis of evaluation of structural strength and deformation capacity.

Panagiotakos and Fardis [26] had proposed two methods to evaluate hinge rotation by quantifying ultimate and yielding rotation.

In different codes like EC-8 [10] adopted simplified method of CSM or N2 method to plot Capacity curve of structure using Nonlinear Static Pushover Analysis.

2.4 RETROFITTING METHODS

In Reinforced Concrete buildings structural level retrofit strategies include two approaches: Conventional method and Non-conventional method.

Additional of shear wall at suitable location along the full height of building increases lateral capacity, ductility and stiffness provided connection to existing structure are well designed for shear transfer [31].

Addition of steel bracing to the structures where large opening are required is effective solution for retrofitting, effective connection between steel bracing and column brings high strength and stiffness to existing structures [3].

E1-Dakhakhni *et al.* [11] mentioned that strengthening of existing moment resisting frame with addition of strong masonry wall may results in failure of existing column.

Delfosse and Delfosse [9] introduced the concept of seismic base isolation to reduce the experienced base shear forces of structure as non-conventional retrofit method.

In non-conventional retrofitting, horizontal seismic forces are also reduced by use of supplemental damping devices like viscous damper, visco-elastic damper, and hysteric dampers [3].

Member level strengthening is done to strengthen seismically deficient members [3]. In this retrofit strategies concrete or steel or fibre reinforced polymer jacket are provided to columns, beams, joints and foundation to increase the confinement of concrete and lateral load capacity of member [3].

Sugano [30] had described different types of concrete and steel jackets for column which can enhance strength, stiffness and ductility.

Details of steel jacketing which is used to provide local strengthening of column are given by Aboutaha *et al.* [1].

Seismic shear strengthening using FRP jacketing method and approach is described by Priestley *et al.* [28].

Lam and Teng [32] had found out the stress- strain model for FRP-confined concrete based on their experimental results and earlier literature.

An FRP jacket in horizontal direction improves the confinement of concrete under flexure and also it enhances ductility [32].

THE BUILDING

3.1 LOCATION OF SITE

Kendriya Vidyalaya school buildings are situated in Gangtok. Gangtok is the capital and largest town of Sikkim state. It is situated in the eastern Himalayan range at an altitude of 1650m. The town lies on one side of a hill. Roro Chu and Ranikhola are two streams which flanked the city in eastern and western part respectively. According to Bureau of Indian Standard, Sikkim falls on Zone IV [21]. Kendriya Vidyalaya is located in Tadong area of East Sikkim spread over several levels of hill terraces.

3.2 SEISMICITY OF THE REGION

Sikkim state is crossed by Main Boundary Thrust (MBT) and Main Central Thrust (MCT). These two thrusts are main reasons for frequent earthquakes in state. Other than these two faults, Gangtok and Teesta lineaments have also caused many earthquakes in the region [25].

A great earthquake of magnitude M_w 8.1 hit the area in January 1934. But that earthquake is not important for this study since building blocks were constructed in 1985. After 1985, significant earthquakes occurred in area are Bihar Nepal earthquake of magnitude M_w 6.5 in August 1988, Sikkim earthquake of magnitude M_w 5.3 in February 2006, Bhutan earthquake of magnitude M_w 6.1 in September 2009. Recently earthquake of magnitude M_w 6.9 struck near Sikkim Nepal border on 18th September 2011.

3.3 DETAILS OF BUILDING BLOCKS

Building blocks are reinforced concrete frame structures with infill material as masonry brick wall. These schools blocks have steel roof trusses covered with sheets (Fig 3.1). Block II, Block III (Primary School) and Block IV have overall plan dimension of 50.18m x 9.95m, 35.3m x 30.3m and 57.68m x 9.95m respectively. Plan and elevation of block II, III and IV are shown in Appendix B. These building blocks are constructed at several levels along the slope of soil. Slope of soil is stabilized by retaining walls (Fig 3.6). Block II, III and IV are 2-story buildings. Building Blocks are irregular in plan. Block II and IV have two parts separated by crumple joint of 0.10m. Watertank is installed on roof top of one part of the block. Block III (Primary school) building has an open ground in the middle and three sides of this building have double storey covered with steel truss roof, while front side has 1-storey with open terrace (Fig 3.3). Tables 3.1and 3.2 give the details of columns sizes and reinforcement. Building blocks are located on hill slope and having natural aesthetic appearance. Typical layout of Block II, III and IV are shown in figs 3.10 to 3.12.

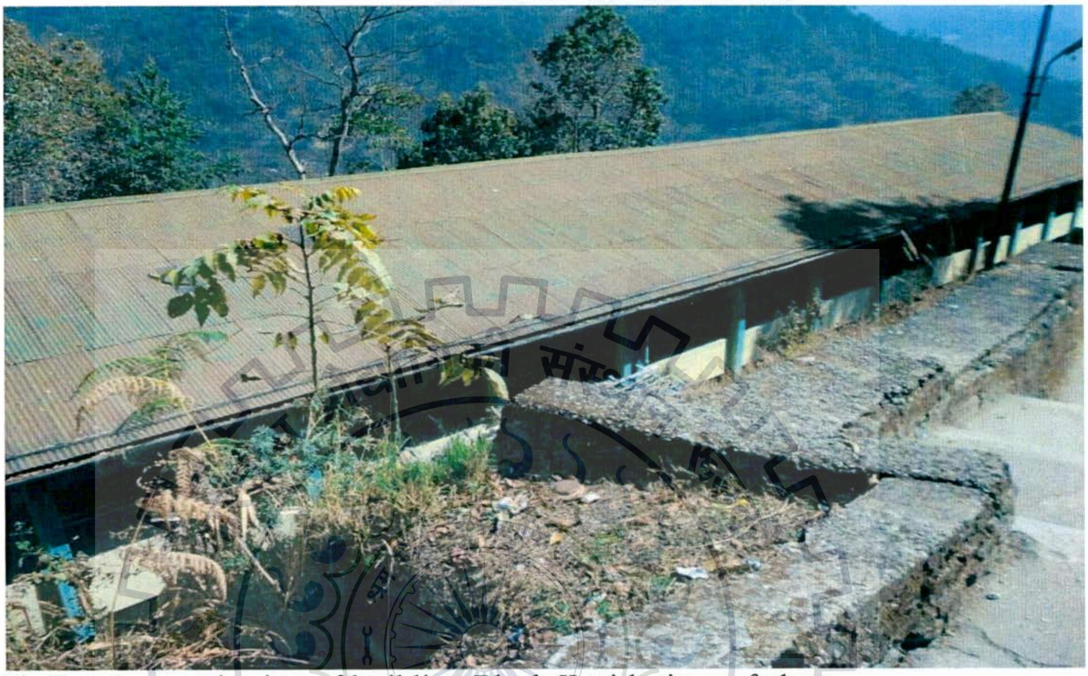

Fig 3.1: Isometric view of building Block II with tin roof sheet

Fig 3.2: Isometric view of Block III (Primary School): 3-sides covered with roof trusses and 1-side open terrace

Fig 3.3: Back view of Block II showing soil retained by high retaining wall

3.4 THE DAMAGE AND DISTRESS

Damages occurred in the blocks may be due to high stresses caused by earthquake forces. Distresses are mostly in the form of cracks in columns and masonry walls (Figs 3.6 & 3.7). Damages are observed in masonry wall of classrooms and exterior wall (Fig 3.4). Bathroom and Toilet portion of the blocks are damaged caused due to dampness (Fig 3.8). Spalling of concrete from column, beam and roof slab has also been observed at several locations. Reason of distress in columns and beams may also be due to spreading of retaining wall (Figs 3.5 & 3.9). After spalling of concrete from column it has also been seen that transverse reinforcement are not provided.

Fig 3.4: Damage in masonry infill panel of classrooms and exterior wall

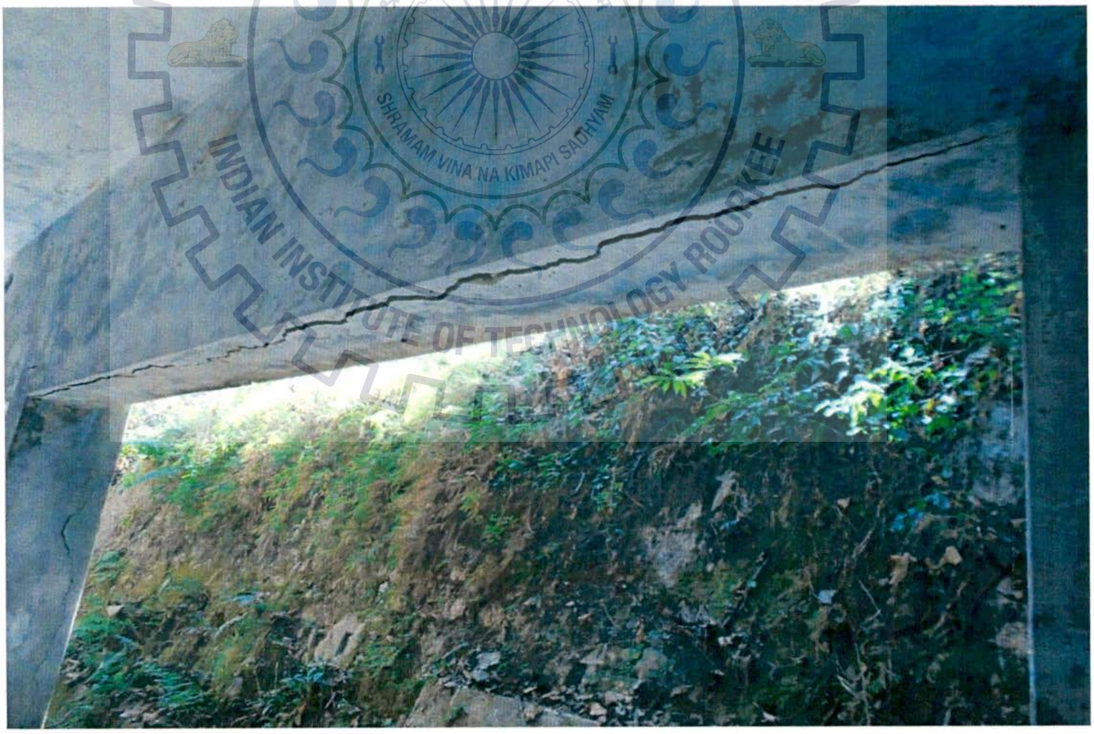

Fig 3.5: Crack in Beam due to corrosion of reinforcement

Fig 3.6: Spalling of concrete cover showing missing stirrups

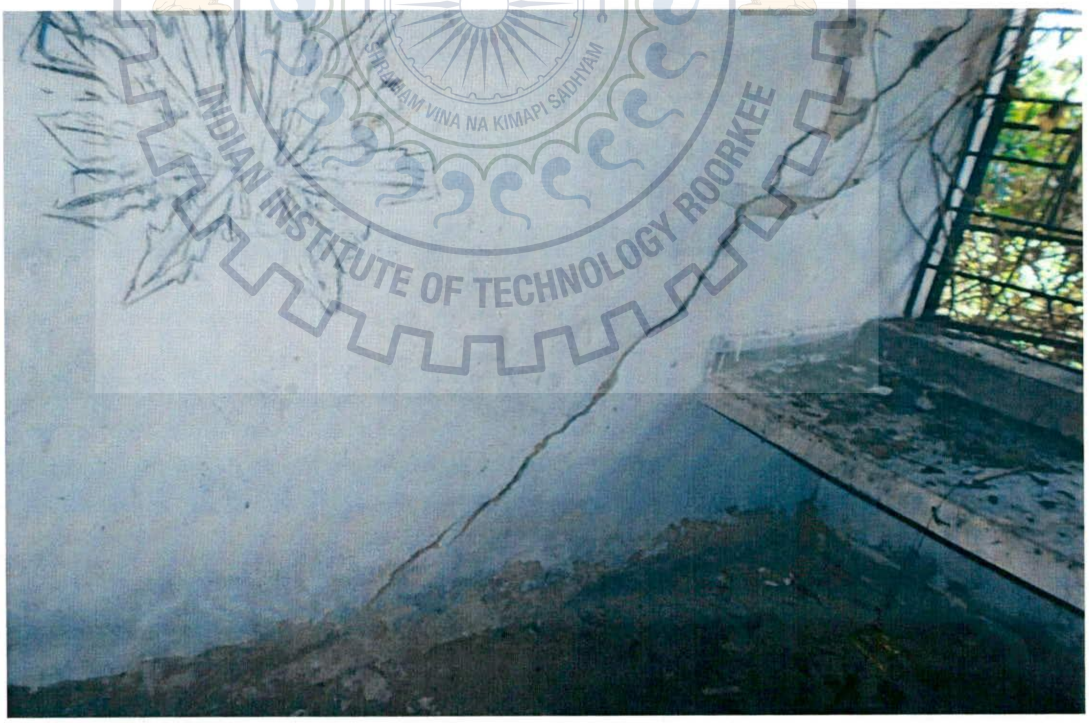

Fig 3.7: Crack in masonry wall

Fig 3.8: Damages due to dampness and appearance of cracks in column

Fig 3.9: Damages in column due to movement of retaining wall

Table 3.1 Column reinforcement details for Block II and IV

Column Id.	Shape	Size		Main Steel		Lateral steel		
		B (mm)	D (mm)	No.	Bar dia.(mm)	No. of sets	Bar dia. (mm)	Spacing (mm)
C1	Rectangular	200	400	6	20	2	6	150
C2	Rectangular	200	350	4	16	3	6	150
C3	Rectangular	400	200	6	12	2	6	150
C4	Rectangular	350	200	4	16	2	6	150
C5	Rectangular	200	350	6	16	3	6	150
C6	Rectangular	200	400	6	12	4	6	150

Note: Grade of concrete and steel are M15 and Fe250 respectively.

Table 3.2 Column reinforcement details for Block III

Column Id.	Shape	Size		Main Steel		Lateral Steel		
		B (mm)	D (mm)	No.	Bar dia.(mm)	No. of sets	Bar dia. (mm)	Spacing (mm)
C1	Rectangular	200	300	6	16	2	6	150
C2	Rectangular	200	200	4	12	2	6	150

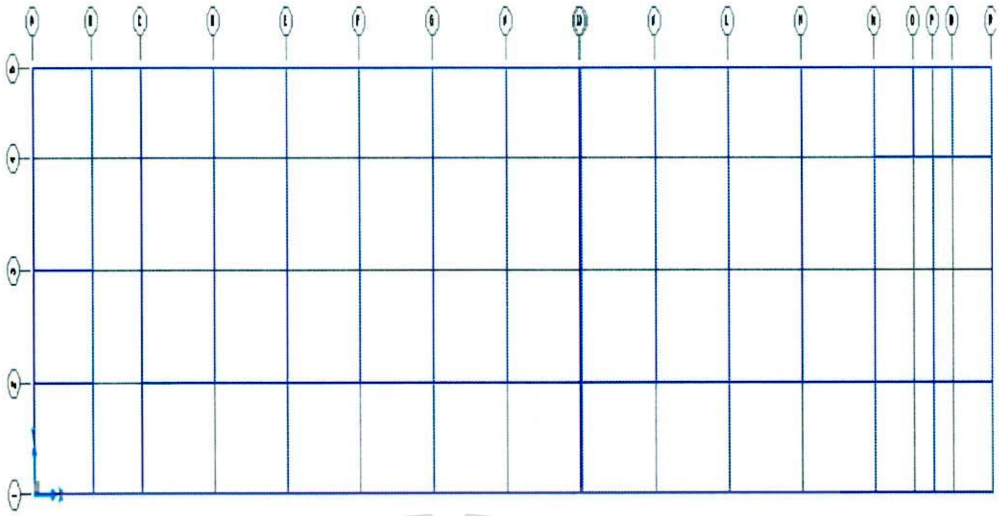

Fig 3.10: Typical layout of Block II and grid pattern

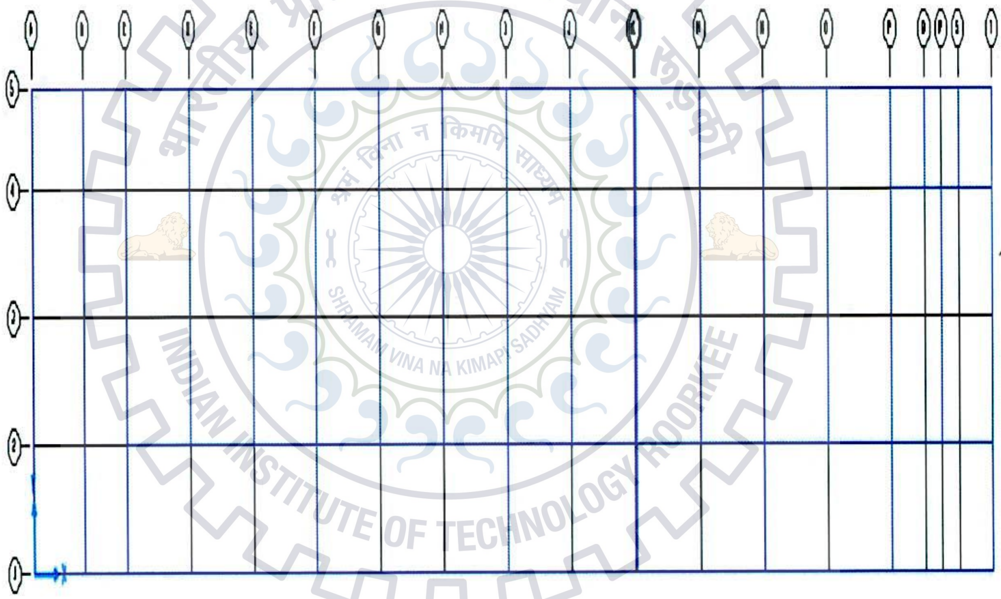

Fig 3.11: Typical layout of Block IV and grid pattern

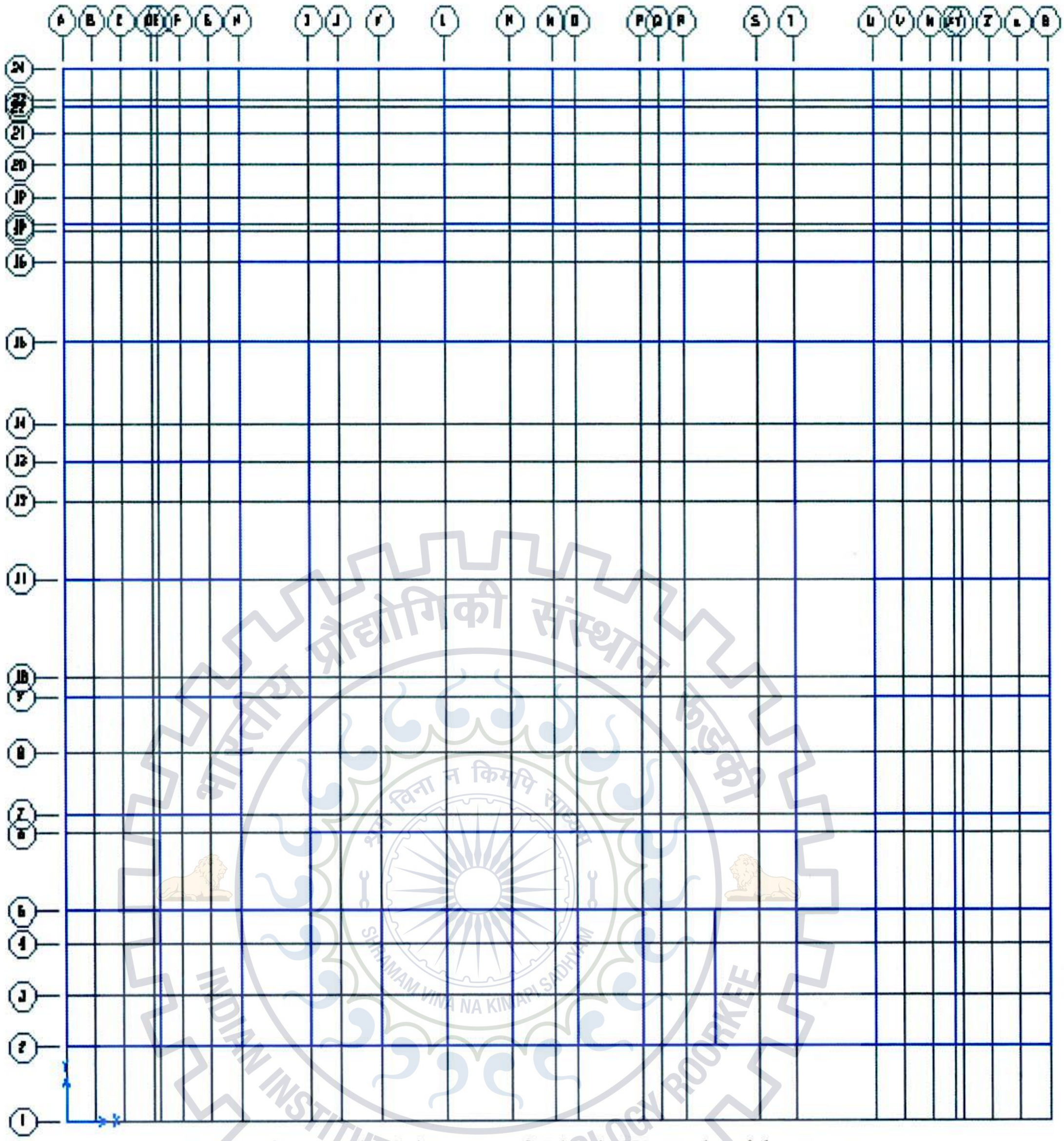

Fig 3.12: Typical layout of Block III and grid pattern

Chapter 4

BEHAVIOUR UNDER EARTHQUAKE FORCES

4.1 LOADS AND LOAD COMBINATIONS

4.1.1 DEAD LOAD

The weight of floor slabs, corridor slabs, staircase slabs, roof truss members, columns, beams, external wall, and partition walls have been considered for dead weight calculation. The dead weight of watertank has been taken for full capacity of storage i.e. 400gals. Thicknesses of slabs have been taken as 130mm, 120mm and 100mm (as given in design drawings). Dead loads of slab are distributed to surrounding beams according to trapezoidal and triangular load distribution as per yield line pattern [17]. Thickness of exterior walls and partition walls has been taken as 230mm and 150mm respectively. Dead weight of masonry walls are uniformly distributed to beams. Walls are constructed with brick masonry having unit weight of $20kN/m^3$. Density of concrete is taken as $25kN/m^3$.

4.1.2 LIVE LOAD

Live load on buildings has been taken as per IS: 875(Part 2)-1987 [19] as shown in Table 4.1.

4.1.3 EARTHQUAKE LOAD

Earthquake loads are calculated as per IS: 1893-1975 [20]. Gangtok falls under Zone IV of the seismic zoning map of India, therefore Zone factor (Z) is considered as 0.24. Value of Importance Factor (I) is taken as 1.5 and Response Reduction factor (R) is considered as 3 since ductile detailing have not been used in the building blocks design. The earthquake loads have been applied using response spectrum. Response spectrum is taken for medium soil according to Seismic Zone IV [21]. Scale Factor for spectra is calculated according to Eq 4.1.

$$\text{Scale Factor} = \frac{Z}{2}\frac{I}{R}\,g \tag{4.1}$$

Using Eq 4.1 scale factor is calculated as 0.5886. Earthquake forces are applied in X as well as in Y direction.

Table 4.1: Live load on building as per IS: 875 (part 2)-1987

Sl. No.	Occupancy Classification	UDL (kN/m^2)
1	Class Room	3.00
2	Staircase	4.00
3	Corridor	4.00
4	Bathroom & Toilets	2.00
5	Libraries	6.00
6	Reading Room with separate storage	3.00
7	Store Room of educational building	5.00

4.1.4 LOAD COMBINATIONS

All possible load combinations have been considered to evaluate the safety of structure. The following load combinations have been considered.

(i) 1.2(DL+LL±EQX)

(ii) 1.2(DL+LL±EQY)

(iii) 1.5(DL±EQX)

(iv) 1.5(DL±EQY)

(v) 0.9DL±1.5EQX

(vi) 0.9DL±1.5EQY

Where DL = Dead Load

LL = Live Load

EQX = Earthquake force in X-direction

EQY = Earthquake force in Y-direction

4.2 MODELLING

I. Modelling of each school building block has been performed in SAP 2000 version14 software and 3D computer model of the R.C frame building is generated for bare and infilled frames to evaluate the seismic behaviour of blocks. Since blocks are asymmetric in plan, 3D models for blocks are developed (Figs 4.1-4.6). The beams and columns have been modelled as frame elements. The slabs have been assumed as rigid diaphragms; therefore ground floor and first floor are modelled as rigid diaphragms.

II. Steel trusses and purlins are modelled as frame elements.

III. Base of 3D frame model building blocks are fixed to calculate the response of structures against earthquake forces.

IV. The effects of seismic forces on building blocks have been simulated using software.

4.3 MODELLING OF INFILL

Infill in-plane elastic stiffness has been considered in the model by modelling of compression strut of width (a) as a diagonal member. The width of equivalent strut is calculated by Eq 4.2 [4].

$$a = 0.175\,(\lambda_1\, h_{col})^{-0.4}\; r_{inf} \tag{4.2}$$

$$\lambda 1 = \left[\frac{E_{me}\, t_{inf}\;\; Sin2\theta}{4\, E_{fe}\;\; I_{col}\;\; h_{inf}}\right]^{0.25} \tag{4.3}$$

where,

h_{col} = Effective height of column between centrelines of beam

h_{inf}= Effective height of infill panel

E_{fe} = Expected modulus of elasticity of frame material

E_{me} = Expected modulus of elasticity of infill material

I_{col} = Moment of inertia of column

L_{inf} = Length of infill panel

r_{inf} =Diagonal length of infill panel

t_{inf}= Thickness of infill panel and equivalent strut

θ = Angle whose tangent is the infill height to length aspect ratio, in radians

λ_1= Coefficient used to determine equivalent strut width of infill strut.

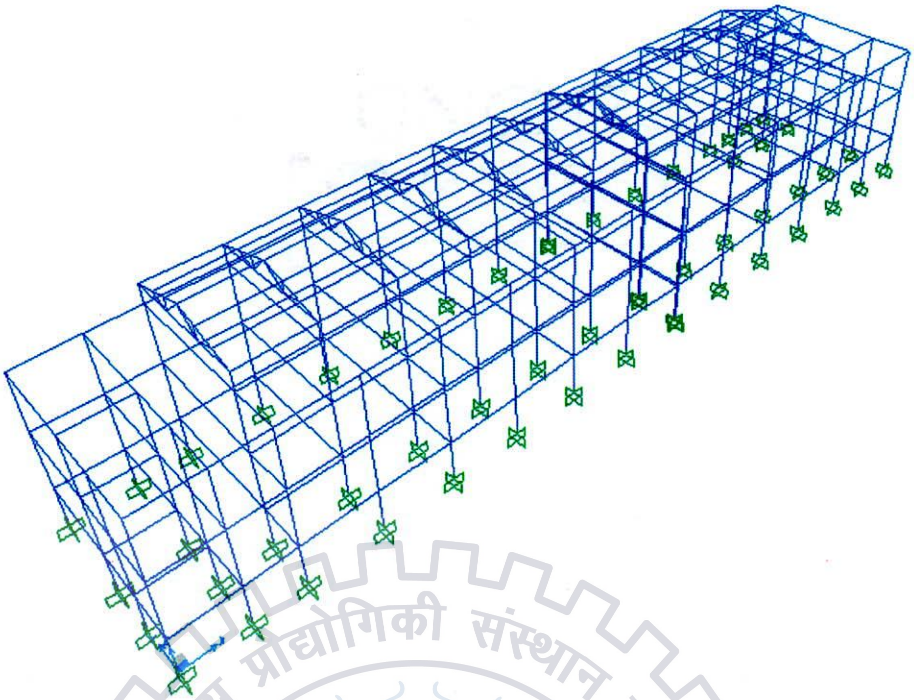

Fig 4.1: 3D Bare Frame model of Block II

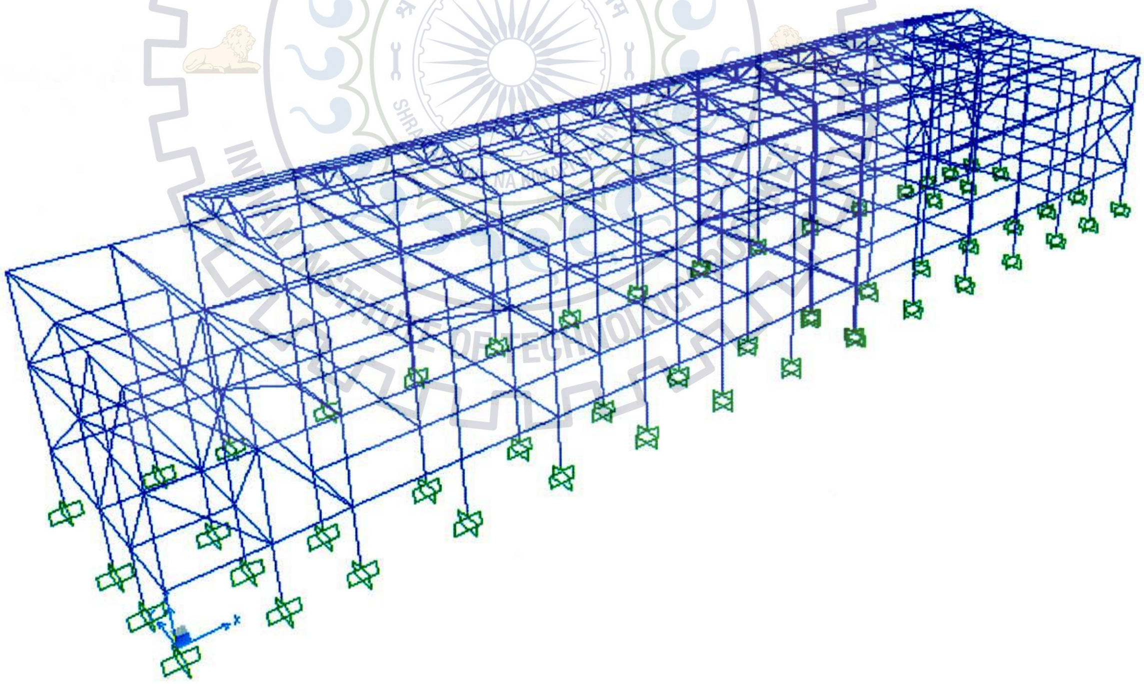

Fig 4.2: 3D Infilled Frame model of Block II

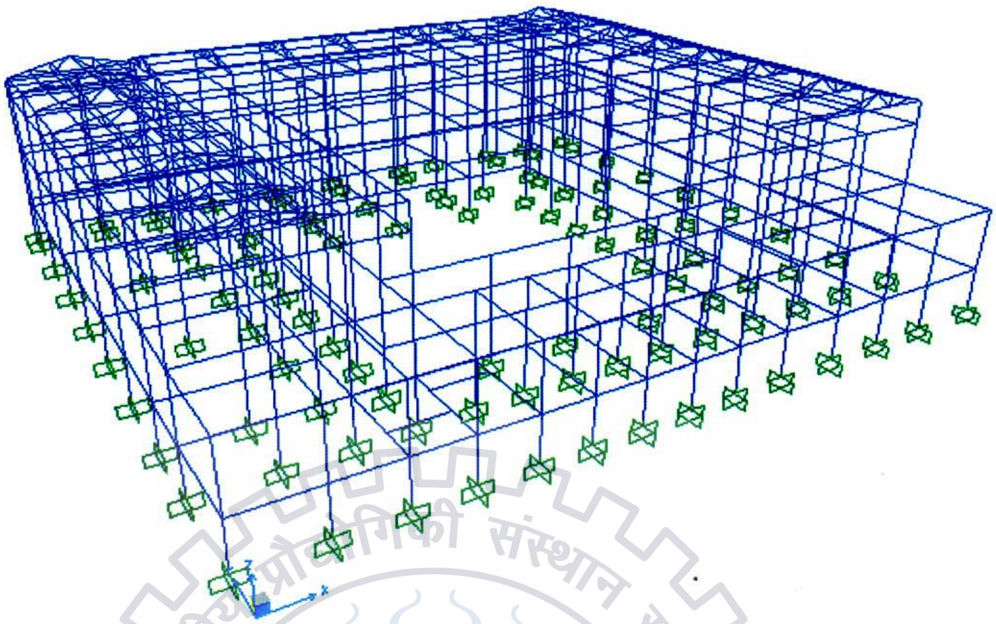

Fig 4.3: 3D Bare Frame model of Block III (Primary School)

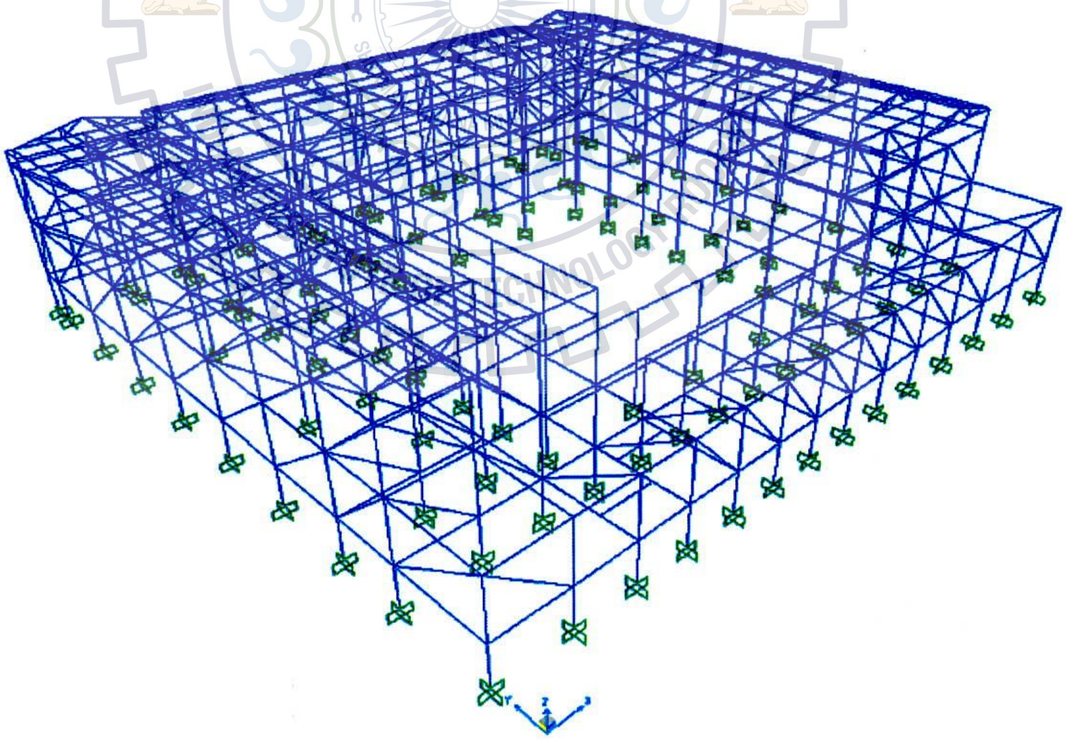

Fig 4.4: 3D Infilled Frame model of Block III (Primary School)

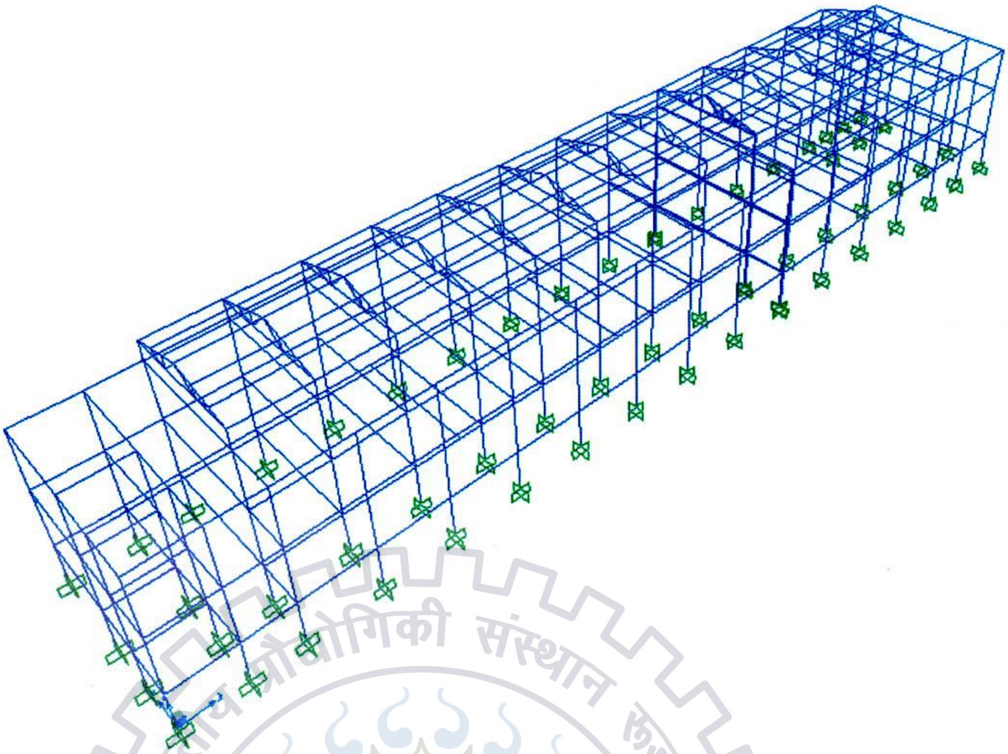

Fig 4.5: 3D Bare Frame model of Block IV

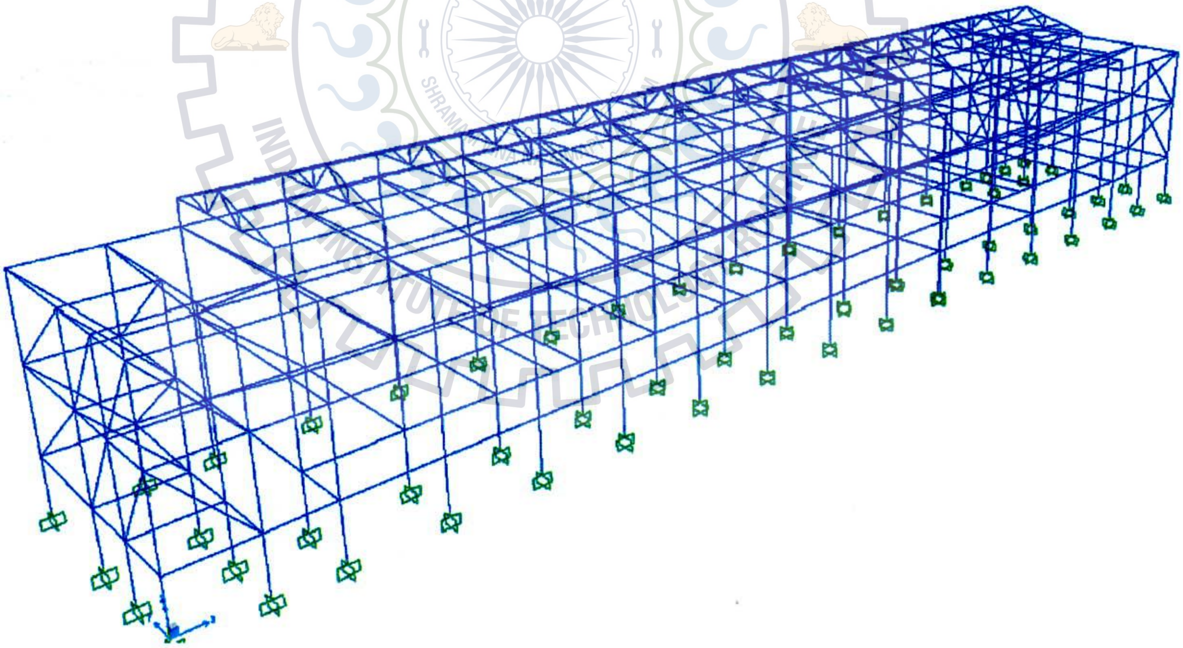

Fig 4.6: 3D Infilled Frame model of Block IV

4.4 MODAL ANALYSIS

To study the dynamic response of building blocks modal analysis has been done using SAP software. Time periods of the blocks assuming fixed at base is evaluated for first three fundamental vibration modes (Table 4.2) for bare frame and infilled frame.

Table 4.2: Time Periods (in sec) for blocks

Mode No.	Block II		Block III		Block IV	
	Bare Frame	Infilled Frame	Bare Frame	Infilled Frame	Bare Frame	Infilled Frame
1	0.769	0.516	0.678	0.396	0.732	0.418
2	0.584	0.445	0.666	0.386	0.541	0.377
3	0.558	0.402	0.617	0.345	0.504	0.331

4.5 SUMMARY

Modal analysis results as shown in table 4.2 for bare frame and with infill frame for Block II, III and IV. In all the blocks time period has reduced significantly for infill frame as compare to bare frame which shows infill has substantial stiffness contribution which need to be considered in model.

NONLINEAR STATIC PUSHOVER ANALYSIS

5.1 NONLINEAR STATIC ANALYSIS

The seismic performance of building blocks has been evaluated by same 3D computer model using SAP 2000 through nonlinear analysis. A nonlinear static procedure focuses on Capacity Spectrum Method [6]. Pushover curve is plotted in the method which is a graphical representation of global force- displacement curve of structures [6]. Pushover curve is compared with response spectra (which represent seismic demand) and performance point of existing building is evaluated. Intended performance objective is compared with performance point using FEMA-356 [16]. Performance point of three building blocks has been evaluated using Force- deformation relation of ASCE-41[4].

5.2 HINGE PROPERTIES

Using the guidelines of FEMA-356 [16] and ASCE-41 [4], nonlinear plastic hinges are assigned to beams and columns. The flexural hinges (M3) are assigned for the beam at two ends. The interacting P-M2-M3 frame hinges have been assigned to all columns at lower and upper ends. Expected yield strength of concrete and steel has been used for nonlinear analysis. Compressive strength of infill is taken as per ASCE-41 [4], for fair quality of masonry infill. Expected modulus of elasticity and shear strength is taken following ASCE-41 [4] guidelines. The infill panel shear behaviour has been considered by deformation controlled axial plastic hinges assigned at mid length of equivalent diagonal struts.

5.3 ANALYSIS OF RESULTS

Nonlinear analysis has been done for all the three school building blocks of the bare frame and their respective infilled frame. Pushover analysis has been performed using software along X and Y directions. Base shear and roof displacement at performance point are found out using ATC-40 [2] (Table 5.1 to 5.3). Pushover Curves are plotted in both directions for blocks II, III and IV for infill frame as well as bare frame (Figs.5.1 to 5.12). It has been observed that masonry infill panel has large lateral stiffness and strength. Due to inclusion of infill panel in model performance point has improved for all the three blocks. The saw-tooth portion of the curve represents damage or life safety state of infill panel and after that strength also decreases. The status of performance point has also been evaluated in terms of number of hinges for DBE and MCE level of earthquakes (Table 5.4 to 5.6). It has been observed that blocks are found to be safe for DBE. But at performance point, plastic hinges are formed in columns at MCE level demand. The performance point reaches beyond Immediate Occupancy (IO) level (Figs 5.7 to 5.12).

- Block II

  Under DBE demand almost all ground floor infill panel have yielded beyond the IO level limit (Fig 5.13). Columns around the staircase also yield but remains within IO level. Under MCE level ground floor masonry panels have shown to undergo to collapse state which can also be seen in photographs of damages in chapter-3 (Fig 3.4). Columns of corridor and bathroom area yielded beyond IO level which corresponds to damages in the building (Fig 3.8).

- Block III

  DBE level demand has suffered by yielding of ground floor and some of the first floor infill panel but no damage has been observed in the beams and columns. Some of the masonry infill panel of ground panel have collapsed or failed under MCE level demand, all the beams are safe and some of the columns of corridor area have yielded beyond IO level.

- Block IV

  Analysis shows that all the ground floor panel masonry will yield under DBE level demand but beam and column will be within safe limit i.e A to B or B to IO level. Under MCE level some of the ground floor masonry has collapsed and 1st floor masonry has started yielding. And under MCE level demand the entire corridor column yielded beyond IO level and some of the ground floor infill panel have collapsed.

Table 5.1: Base Shear and Roof Displacement at Performance Point in X and Y directions under different earthquake levels for Block II

Direction	Earthquake Level	Bare Frame		Infilled Frame	
		Base Shear V_b(kN)	Roof Displacement (mm)	Base Shear V_b(kN)	Roof Displacement (mm)
X	DBE	1756	30	1901	21
	MCE	2117	54	2223	38
Y	DBE	2321	22	2391	15
	MCE	3177	54	3323	23

Table 5.2: Base Shear and Roof Displacement at Performance Point in X and Y directions under different earthquake levels for Block III

Direction	Earthquake Level	Bare Frame		Infilled Frame	
		Base Shear V_b (kN)	Roof Displacement (mm)	Base Shear V_b (kN)	Roof Displacement (mm)
X	DBE	2872	27	3252	12
	MCE	3211	47	3721	24
Y	DBE	2800	30	3845	15
	MCE	3146	50	4611	24

Table 5.3: Base Shear and Roof Displacement at Performance Point in X and Y directions under different earthquake levels for Block IV

Direction	Earthquake Level	Bare Frame		Infilled Frame	
		Base Shear V_b (kN)	Roof Displacement(mm)	Base Shear V_b (kN)	Roof Displacement (mm)
X	DBE	1657	30	2243	12
	MCE	2026	58	2861	31
Y	DBE	2345	21	2772	12
	MCE	3096	48	3500	17

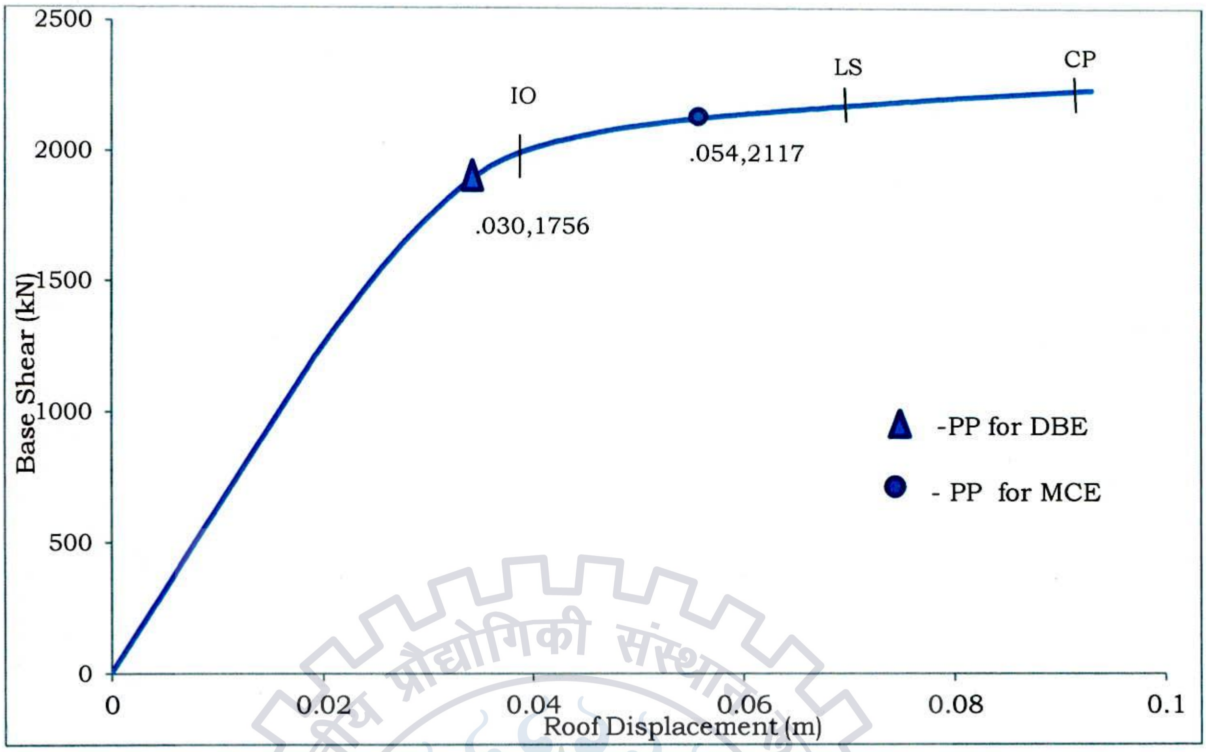

Fig 5.1: Nonlinear Pushover curve of Block II (bare frame) along X (longitudinal)-direction

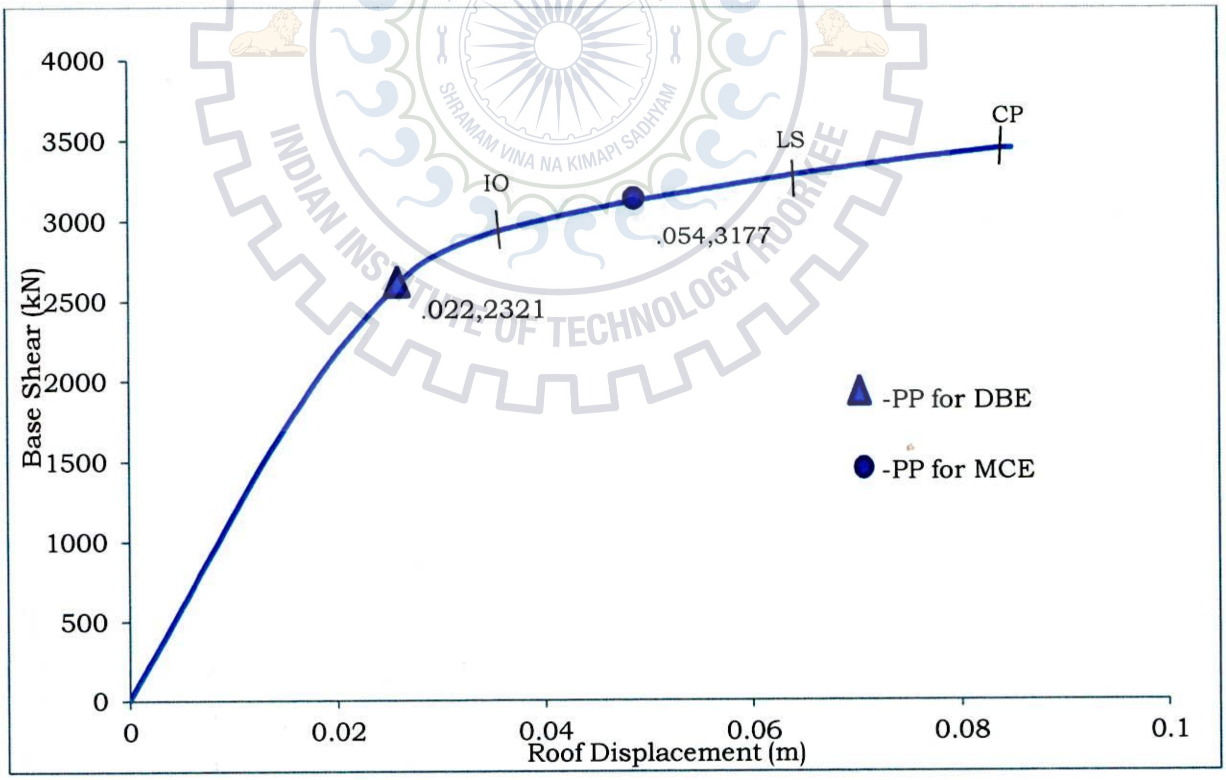

Fig 5.2: Nonlinear Pushover curve of Block II (bare frame) along Y (transverse)-direction

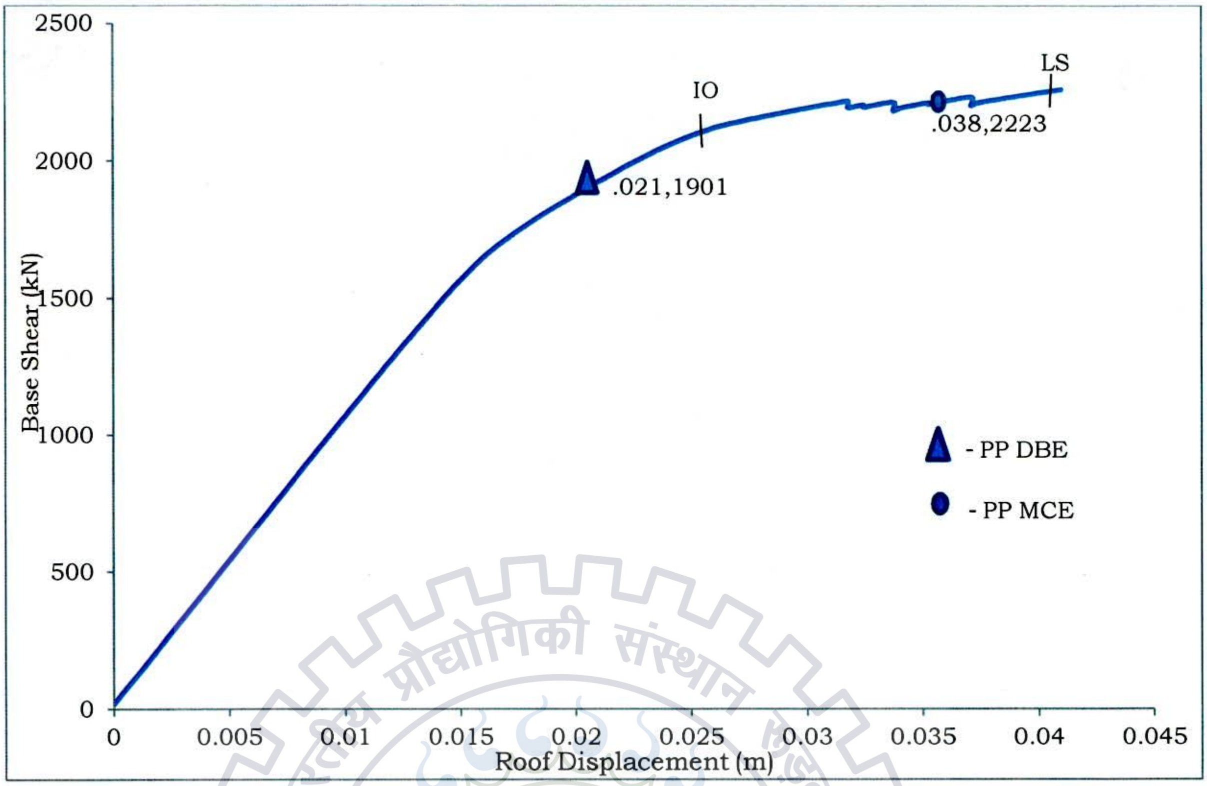

Fig 5.3: Nonlinear Pushover curve of Block II (Infilled frame) along X (longitudinal) direction

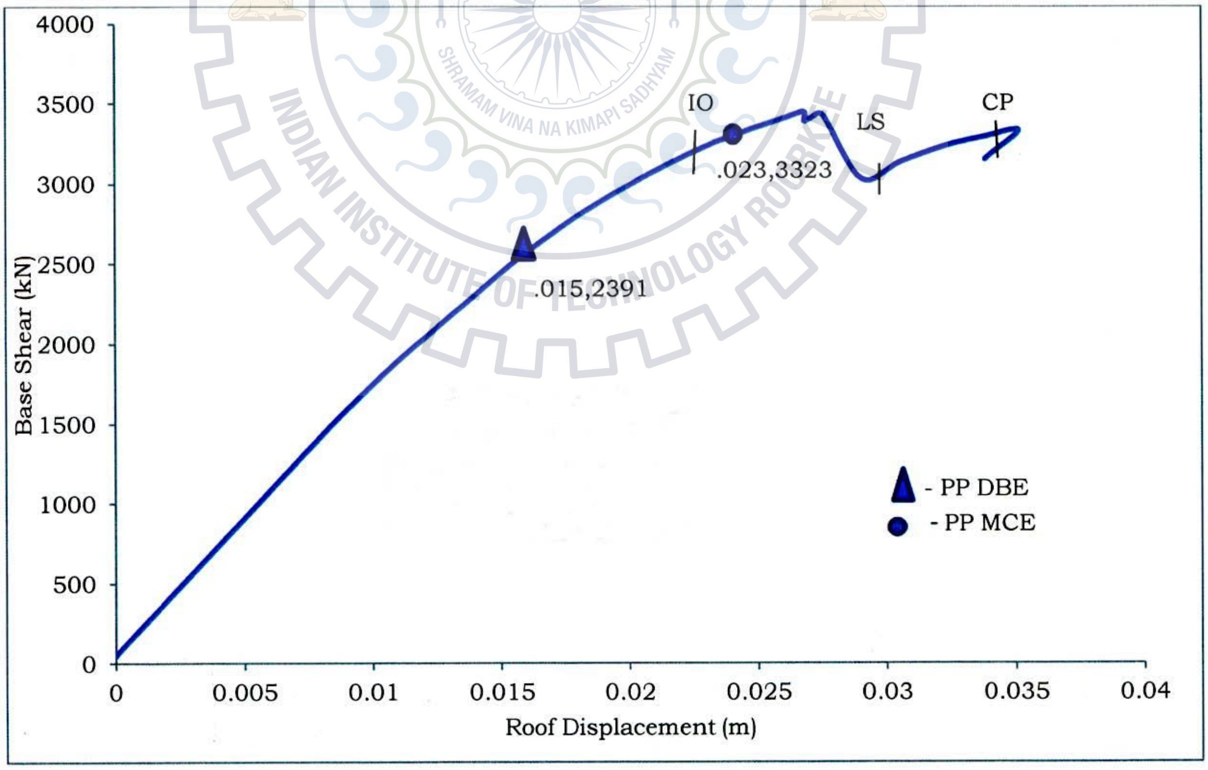

Fig 5.4: Nonlinear Pushover curve of Block II (Infilled frame) along Y (transverse)-direction

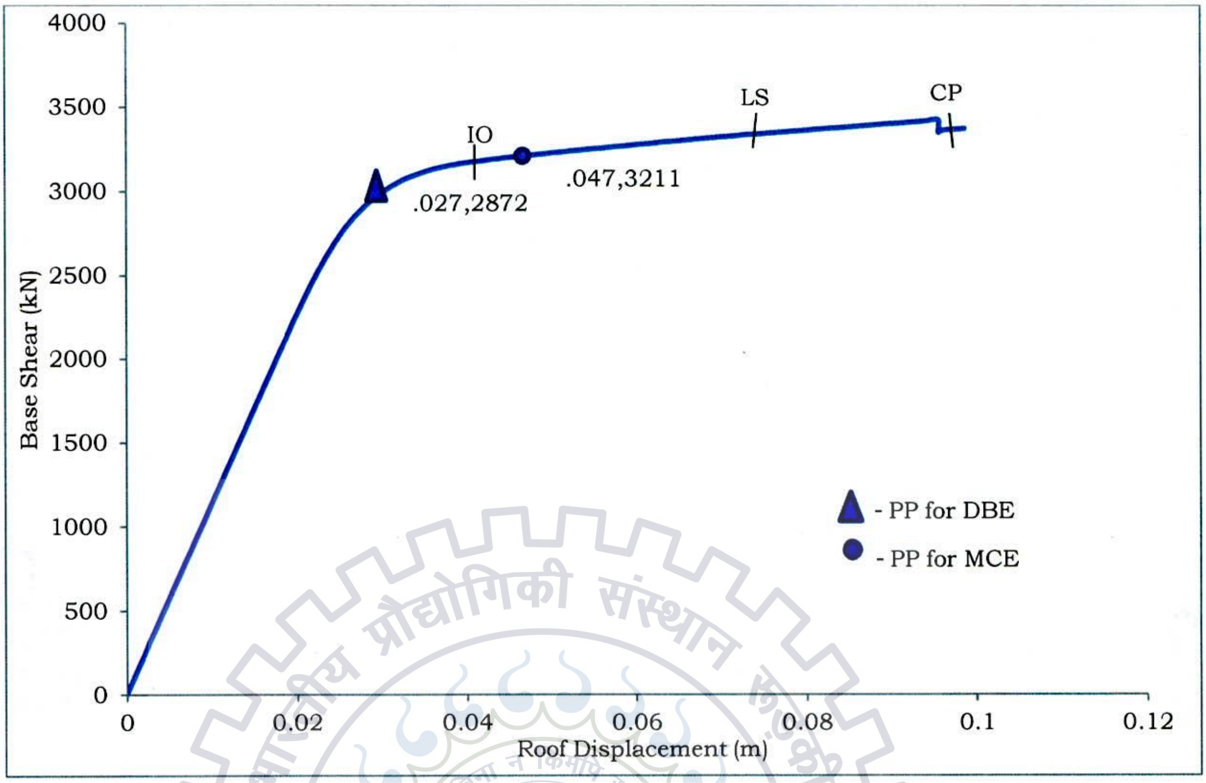

Fig 5.5: Nonlinear Pushover curve of Block III (bare Frame) along X (longitudinal)-direction

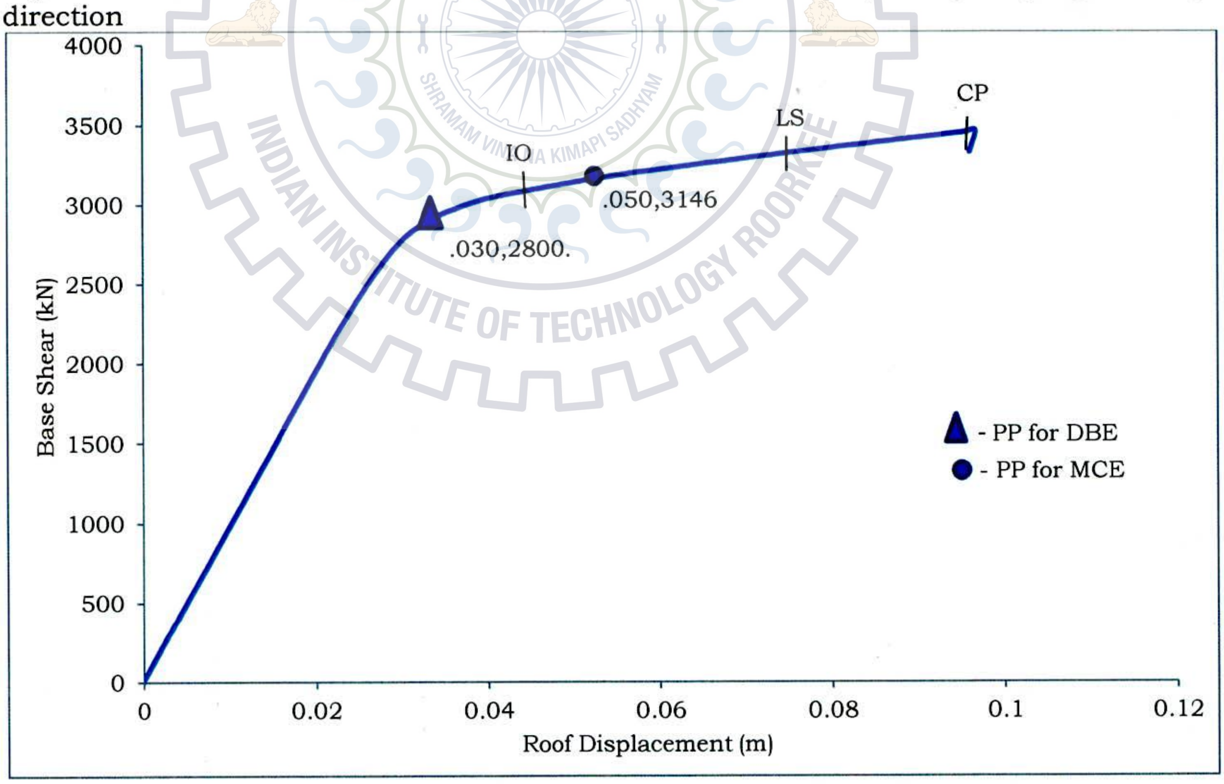

Fig 5.6: Nonlinear Pushover curve of Block III (bare Frame) along Y (transverse)-direction

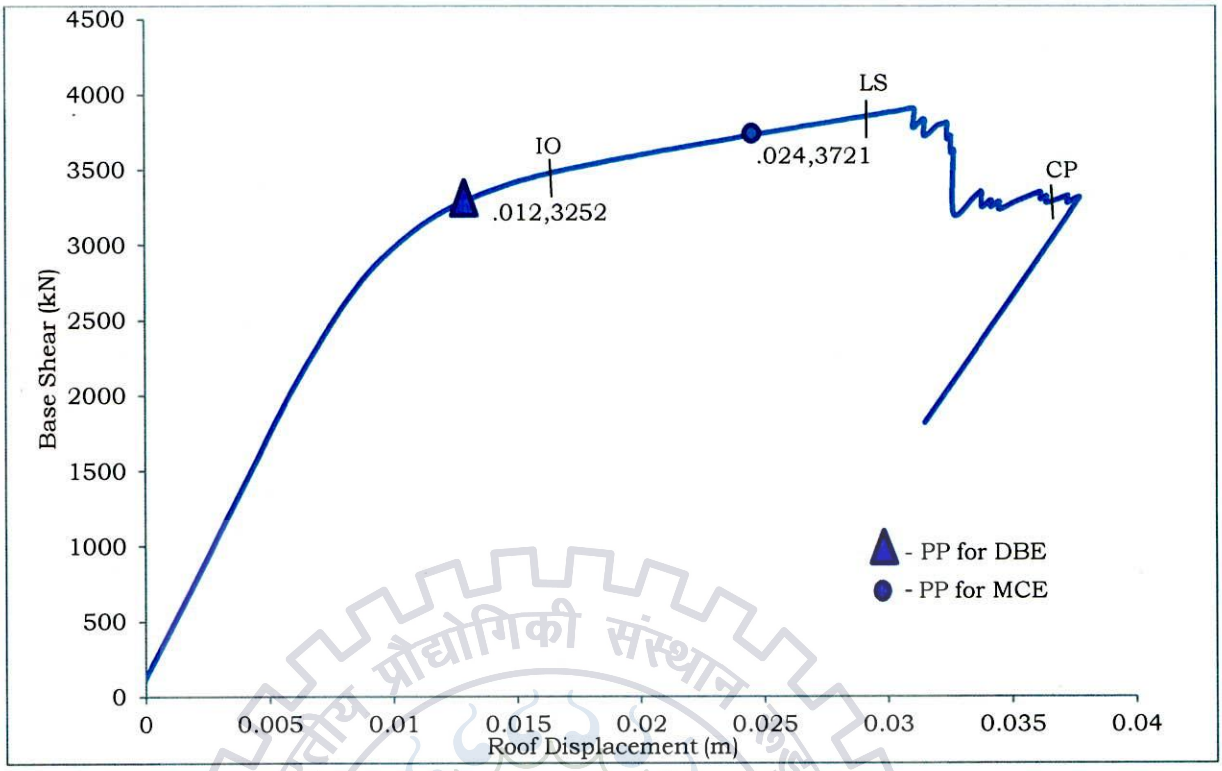

Fig 5.7: Nonlinear Pushover curve for Block III (Infilled Frame) along X (longitudinal)-direction

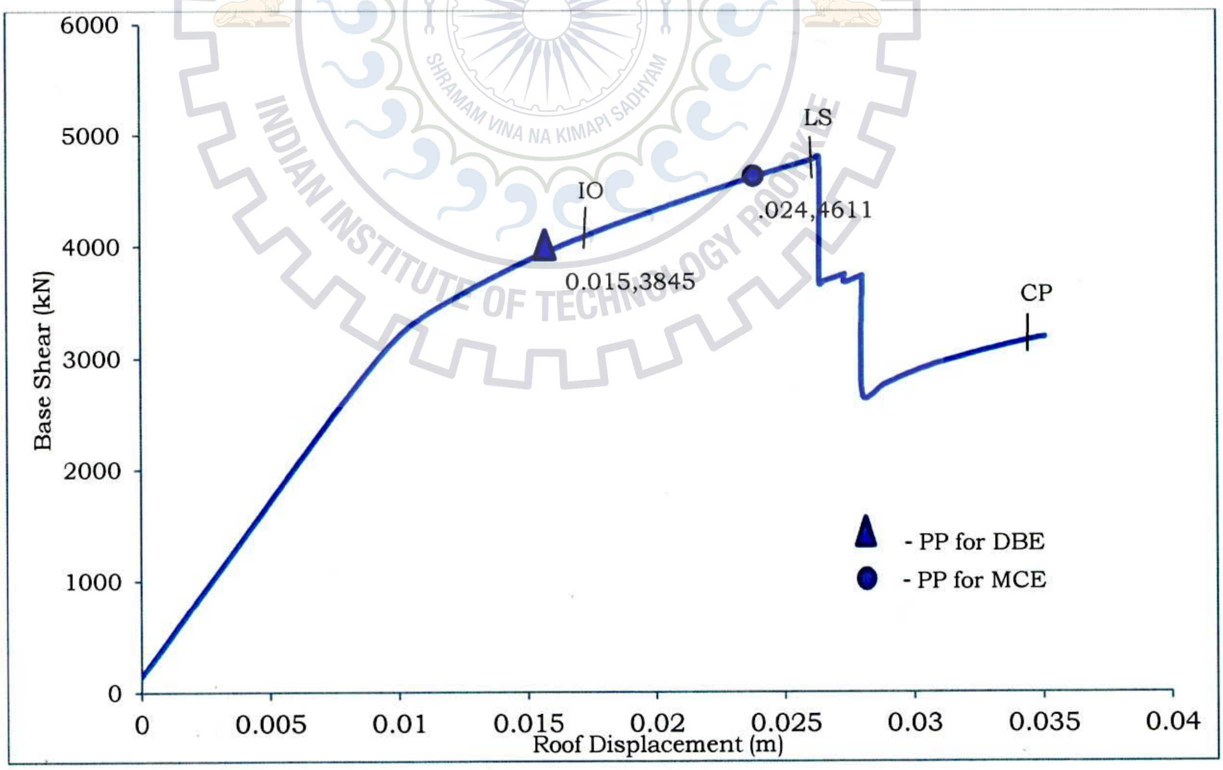

Fig 5.8: Nonlinear Pushover curve of Block III (Infilled Frame) along Y (transverse)-direction

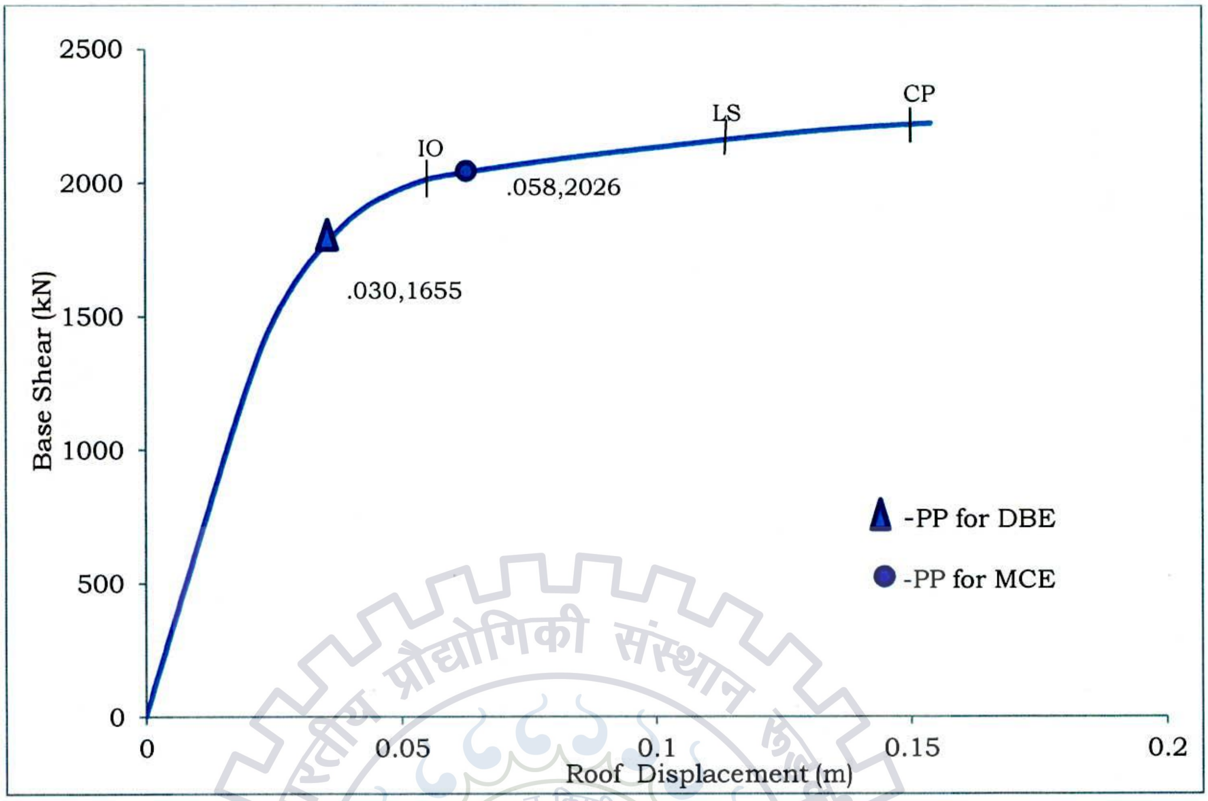

Fig 5.9: Nonlinear Pushover curve of Block IV (Bare Frame) along X (longitudinal)-direction

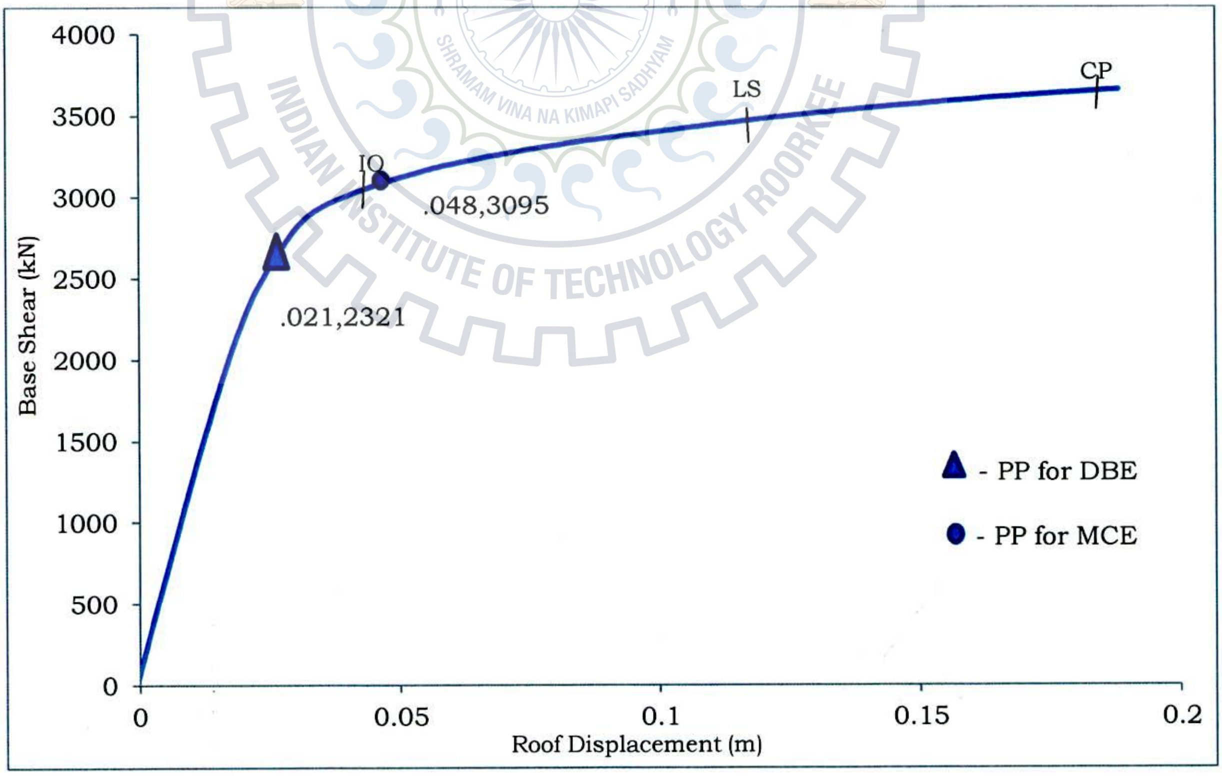

Fig 5.10: Nonlinear Pushover curve of Block IV (Bare Frame) along Y (transverse)-direction

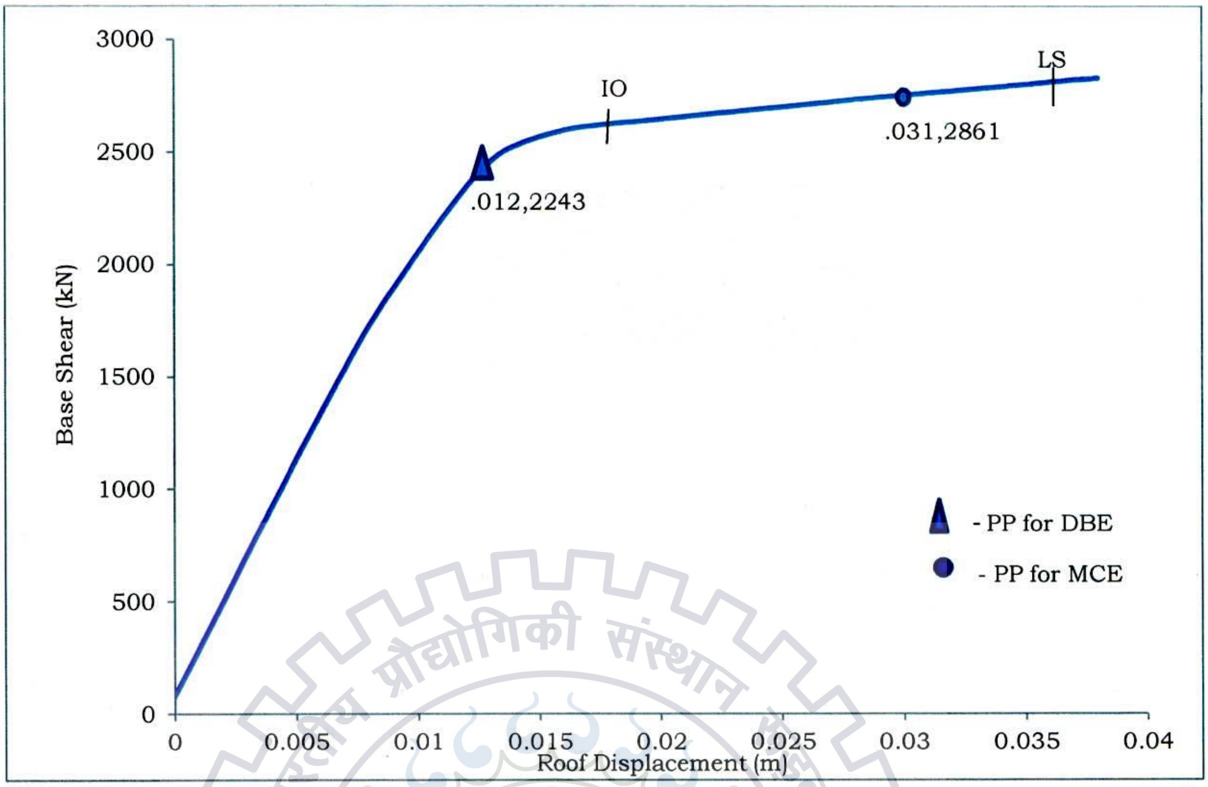

Fig 5.11: Nonlinear Pushover curve of Block IV (Infilled Frame) along X (longitudinal)-direction

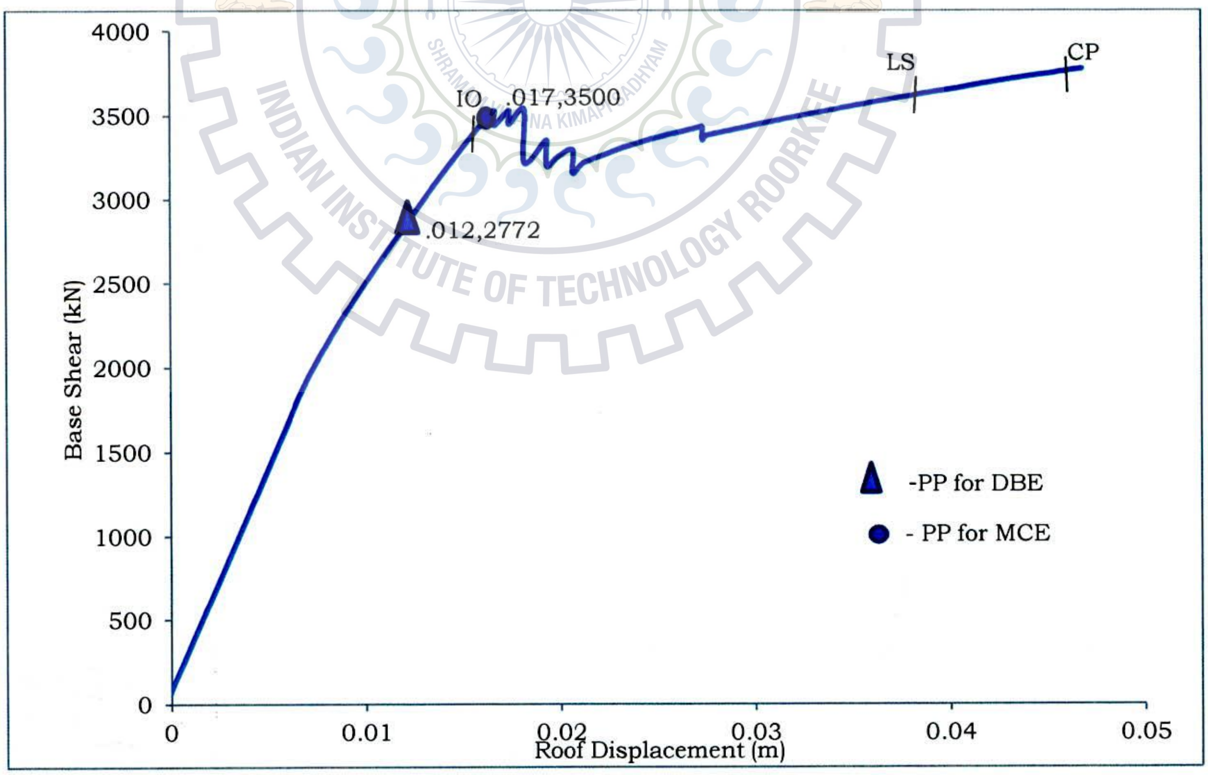

Fig 5.12: Nonlinear Pushover curve of Block IV (Infilled Frame) along Y (transverse)-direction

Table 5.4: Number of Hinges in each range at performance point for Block II (Bare Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	743	71	0	0	0	0	0	0	814
	MCE	559	187	68	0	0	0	0	0	814
Y	DBE	765	49	0	0	0	0	0	0	814
	MCE	660	120	34	0	0	0	0	0	814

Table 5.5: Number of Hinges in each range at performance point for Block II (Infilled Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	789	81	6	0	0	0	0	0	876
	MCE	703	115	57	0	0	0	0	1	876
Y	DBE	841	29	6	0	0	0	0	0	876
	MCE	752	98	23	3	0	0	0	0	876

Table 5.6: Number of Hinges in each range at performance point for Block III (Bare Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	1483	159	0	0	0	0	0	0	1642
	MCE	1274	324	44	0	0	0	0	0	1642
Y	DBE	1427	215	0	0	0	0	0	0	1642
	MCE	1239	346	57	0	0	0	0	0	1642

Table 5.7: Number of Hinges in each range at performance point for Block III (Infilled Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	1537	174	26	0	0	0	0	0	1737
	MCE	1433	256	58	0	0	0	0	0	1737
Y	DBE	1606	100	81	0	0	0	0	0	1737
	MCE	1520	182	24	11	0	0	0	0	1737

Table 5.8: Number of Hinges in each range at performance point for Block IV (Bare Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	735	139	0	0	0	0	0	0	874
	MCE	538	305	31	0	0	0	0	0	874
Y	DBE	805	69	0	0	0	0	0	0	874
	MCE	670	198	6	0	0	0	0	0	874

Table 5.9: Number of Hinges in each range at performance point for Block IV (Infilled Frame)

Direction	Earthquake Level	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
X	DBE	930	62	2	0	0	0	0	0	994
	MCE	856	0	138	0	0	0	0	0	994
Y	DBE	957	25	12	0	0	0	0	0	994
	MCE	891	91	8	2	0	0	0	0	994

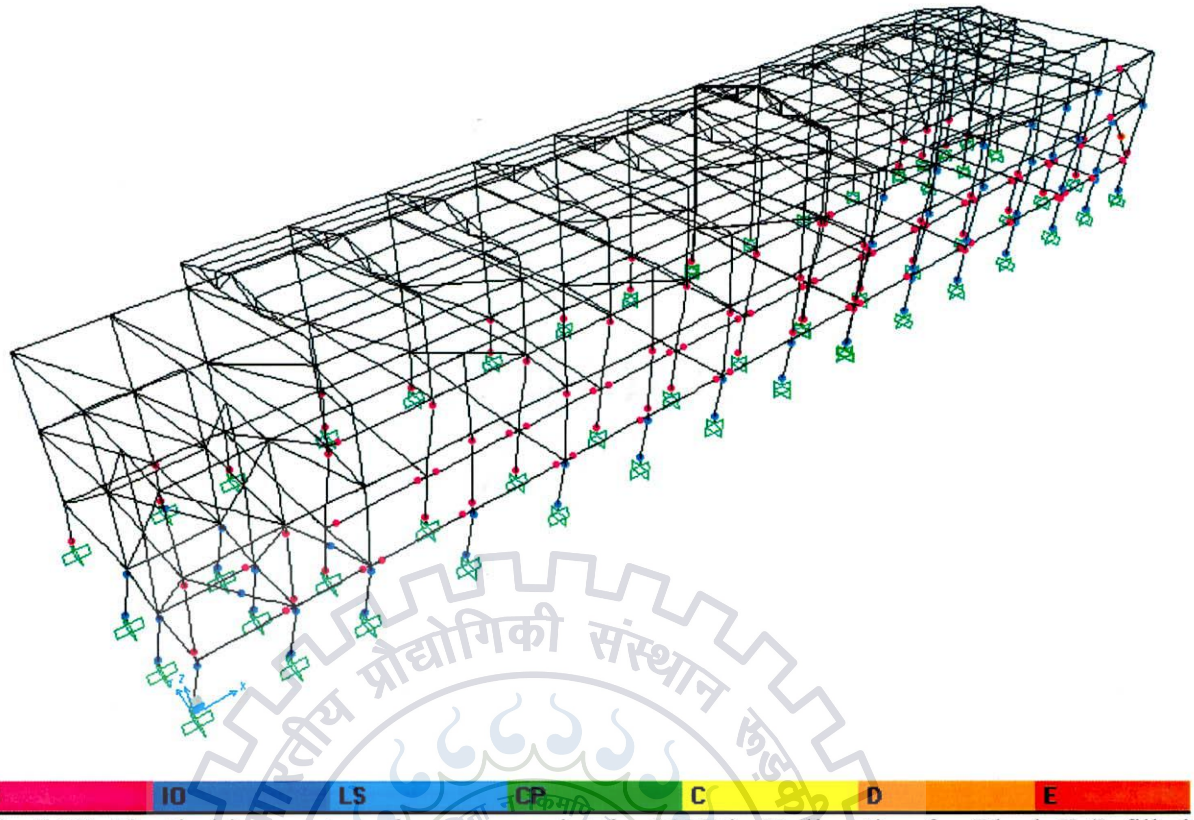

Fig 5.13: Plastic hinges at performance point for MCE in X-direction for Block II (Infilled Frame)

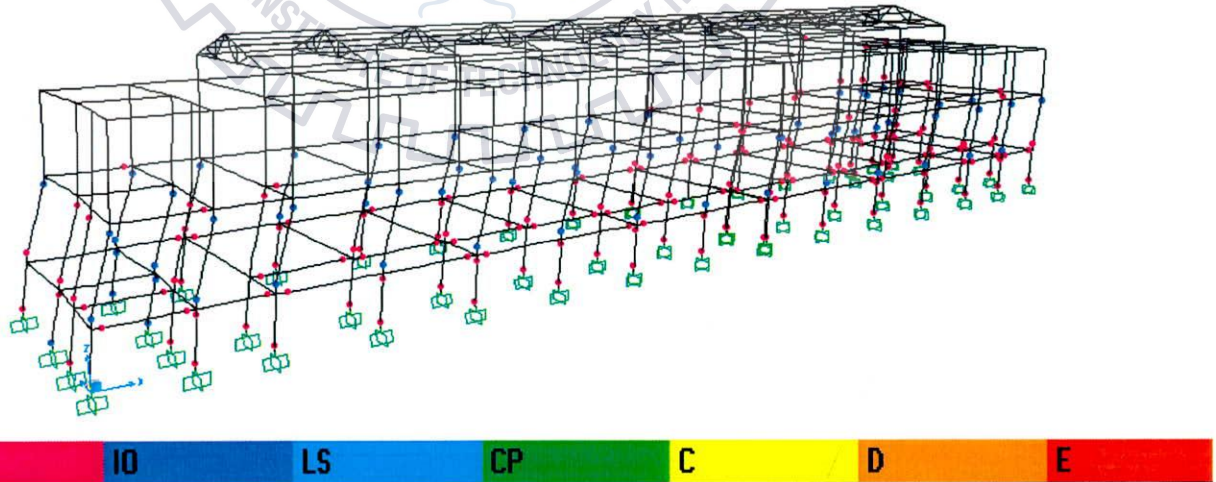

Fig 5.14: Plastic hinges at performance point for MCE in X-direction for Block II (Bare frame)

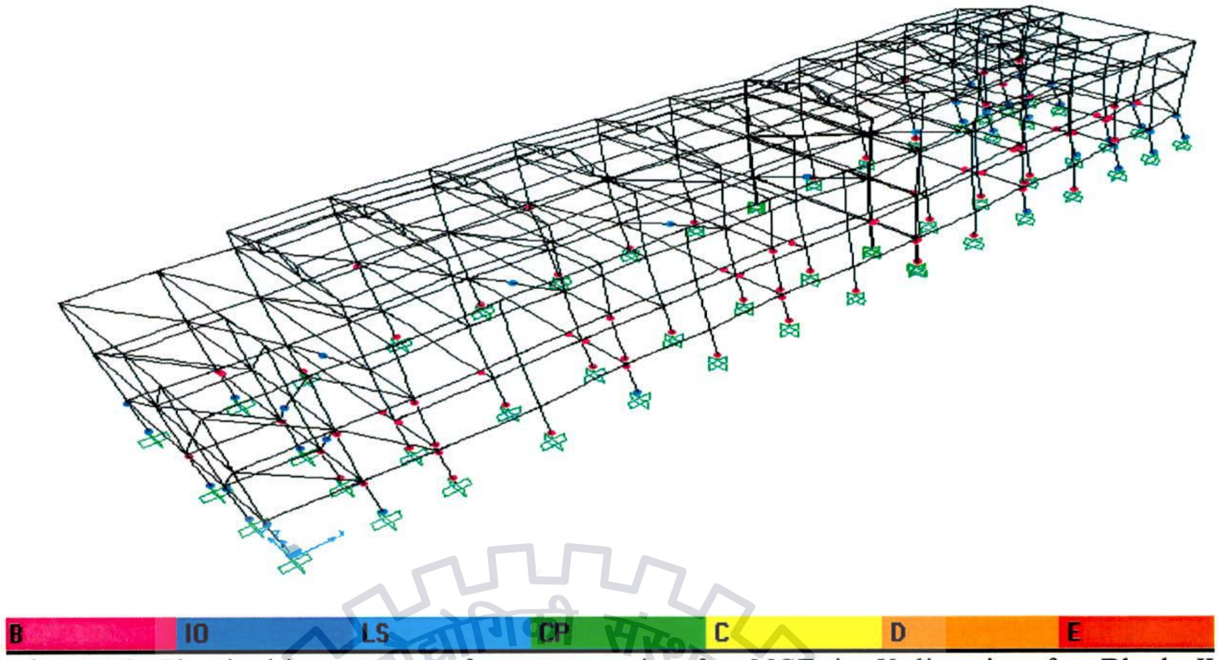

Fig 5.15: Plastic hinges at performance point for MCE in Y-direction for Block II (Infilled Frame)

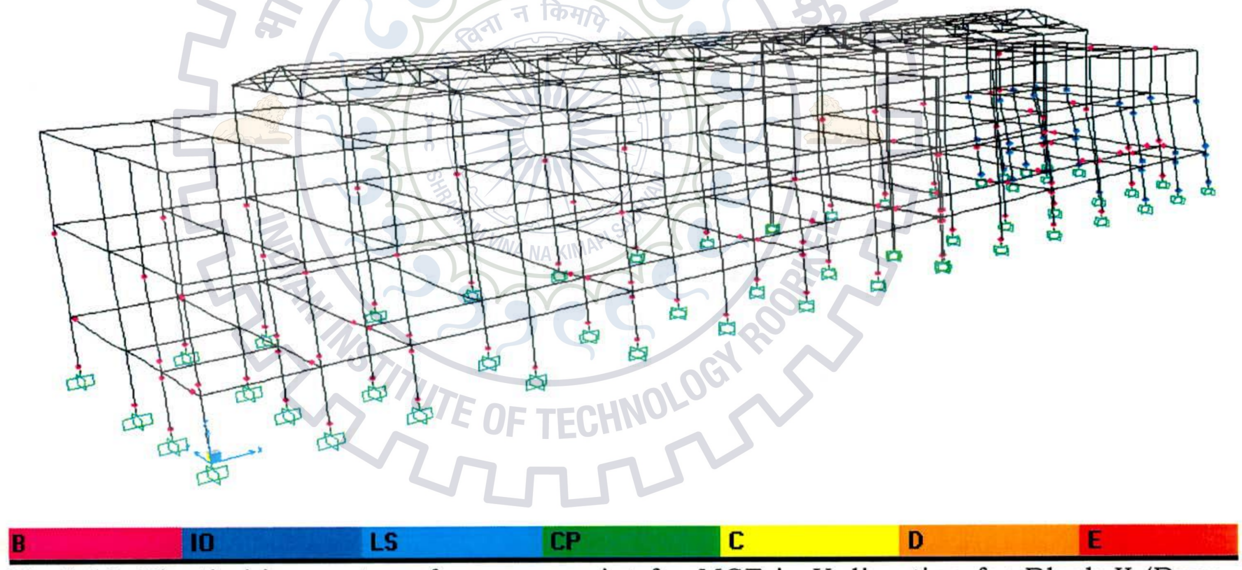

Fig 5.16: Plastic hinges at performance point for MCE in Y-direction for Block II (Bare Frame)

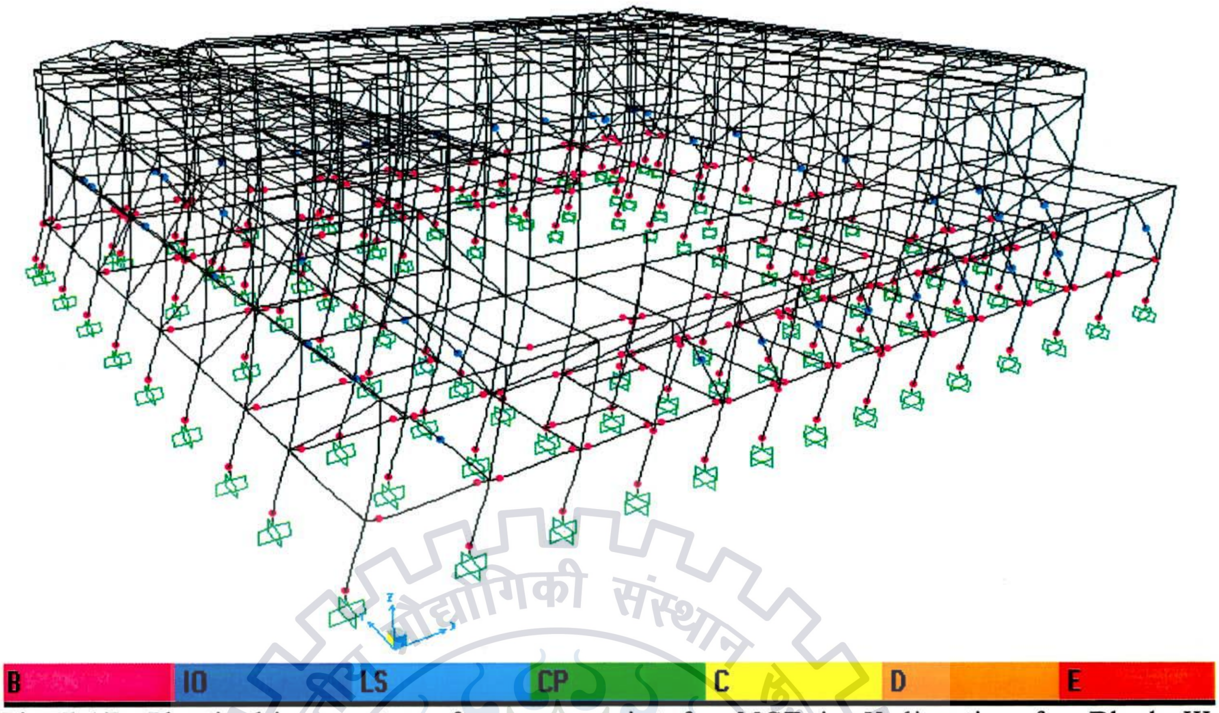

Fig 5.17: Plastic hinges at performance point for MCE in X-direction for Block III (Infilled Frame)

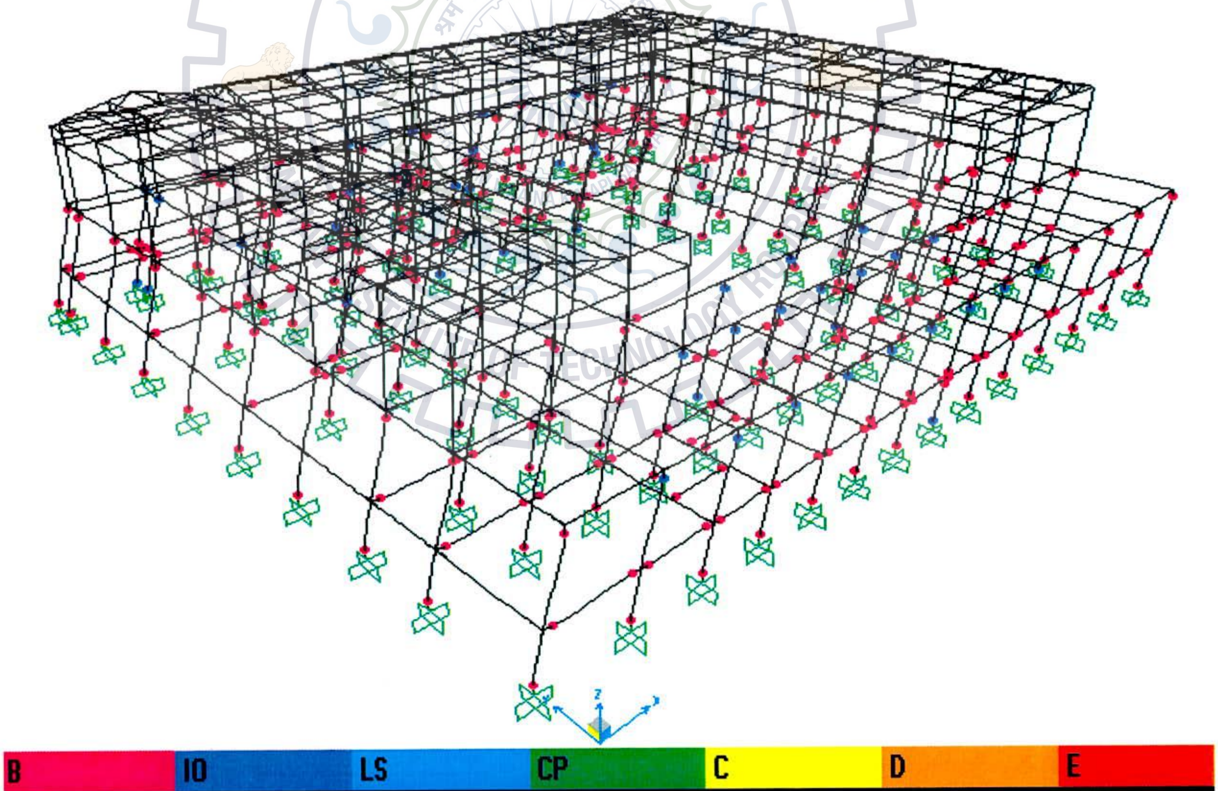

Fig 5.18: Plastic hinges at performance point for MCE in X-direction for Block III (Bare Frame)

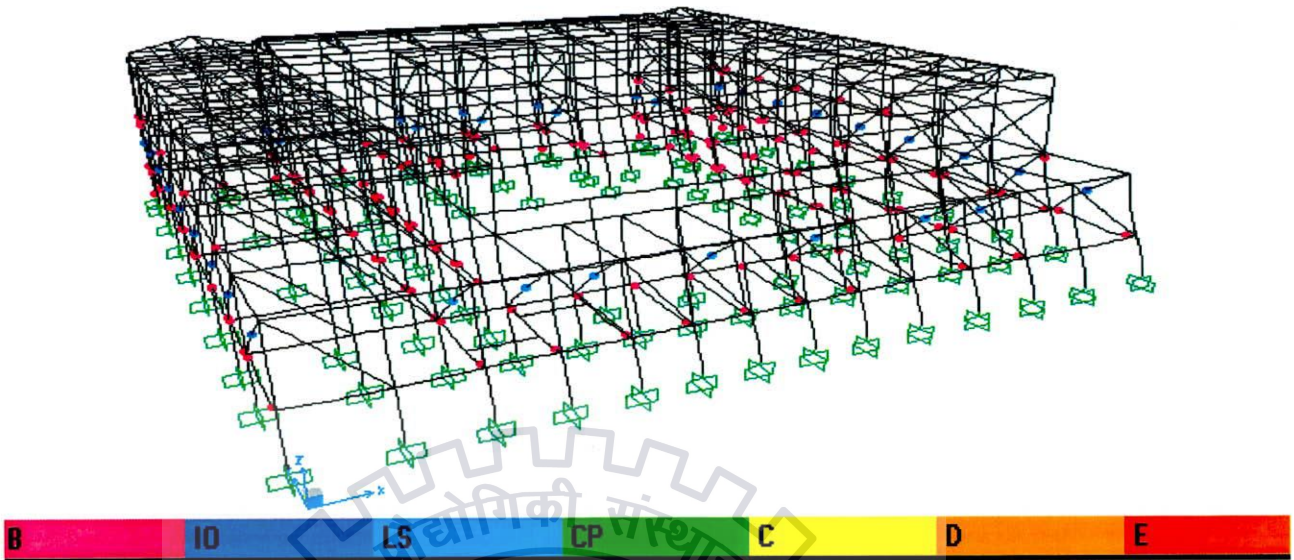

Fig 5.19: Plastic hinges at performance point at MCE in Y-direction for Block III (Infilled Frame)

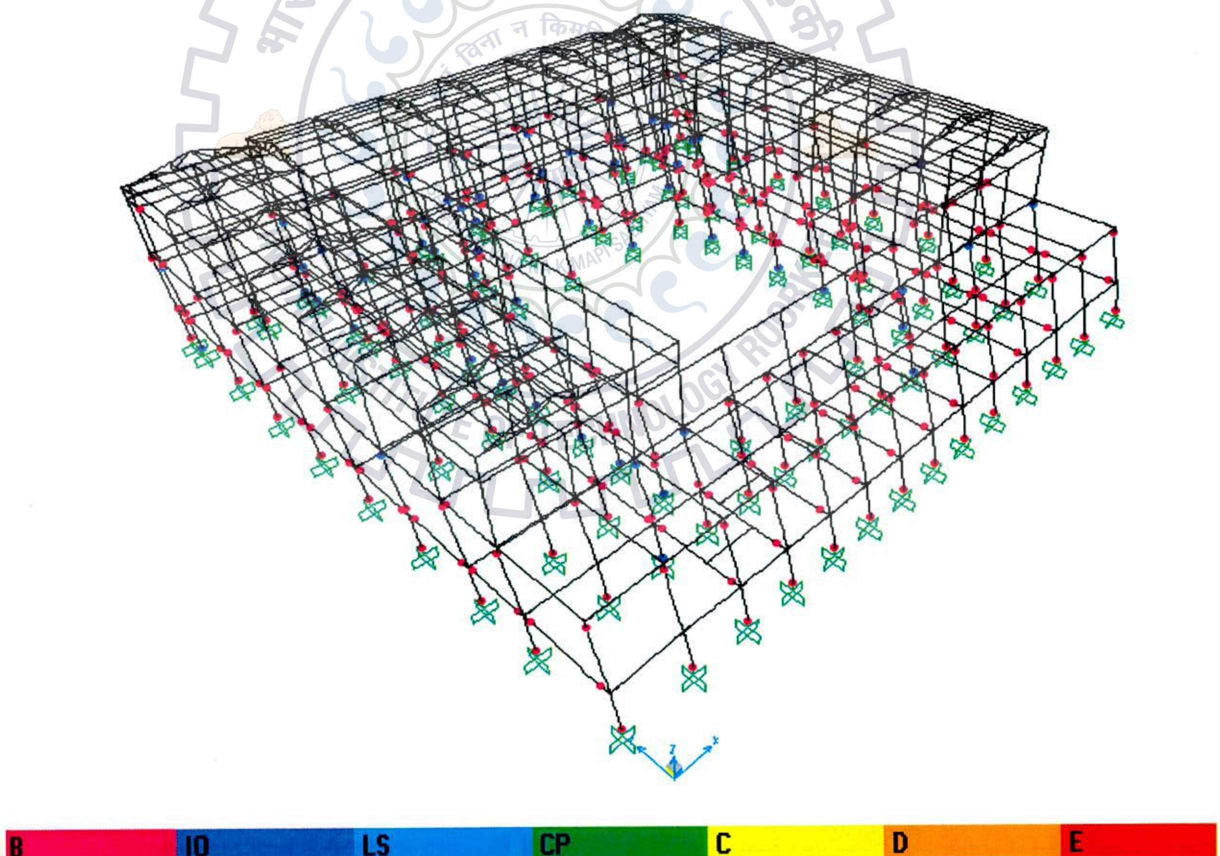

Fig 5.20: Plastic hinges at performance point at MCE in Y-direction for Block III (Bare Frame)

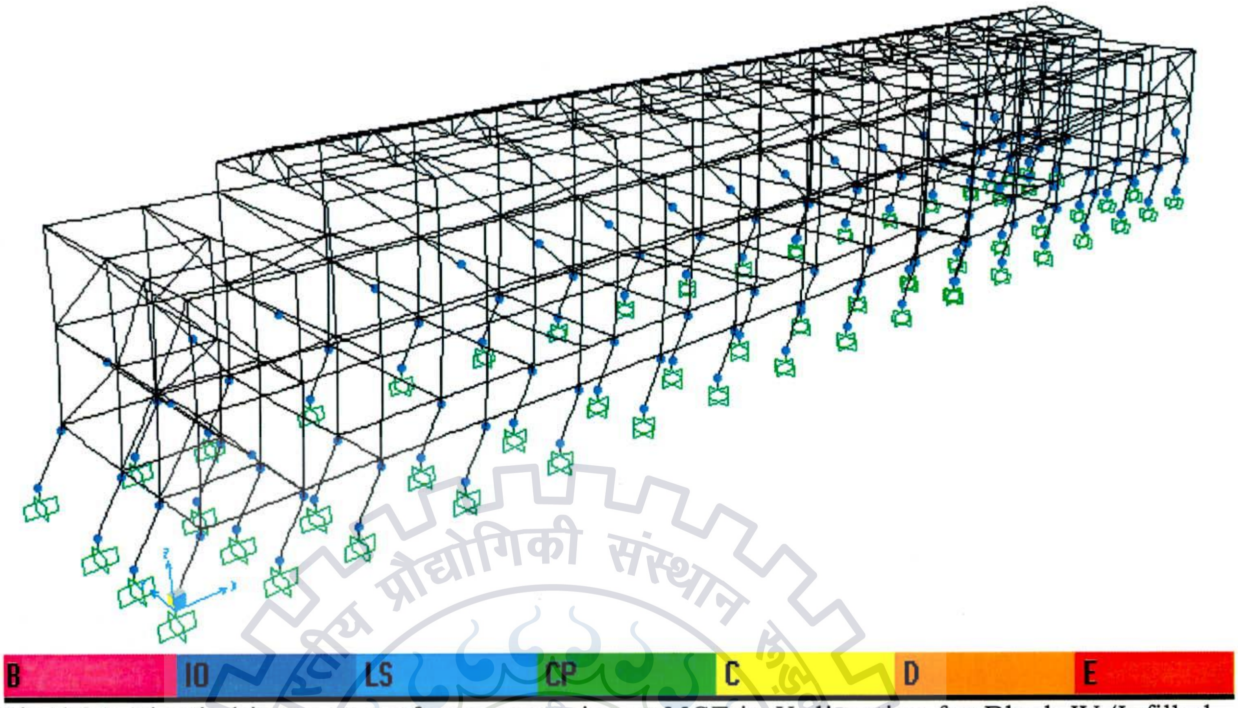

Fig 5.21: Plastic hinges at performance point at MCE in X-direction for Block IV (Infilled Frame)

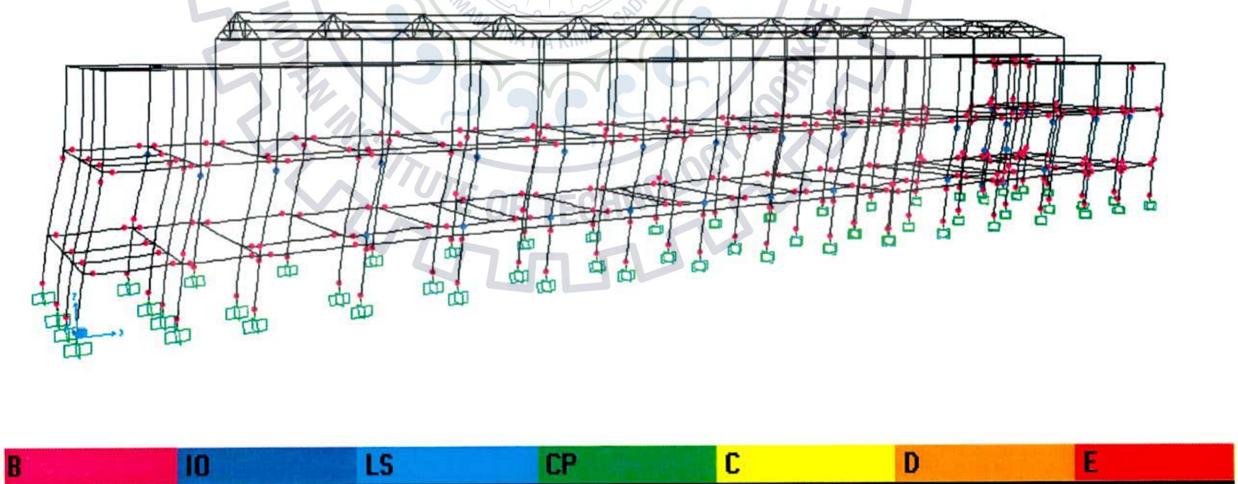

Fig 5.22: Plastic hinges at performance point at MCE in X-direction for Block IV (Bare Frame)

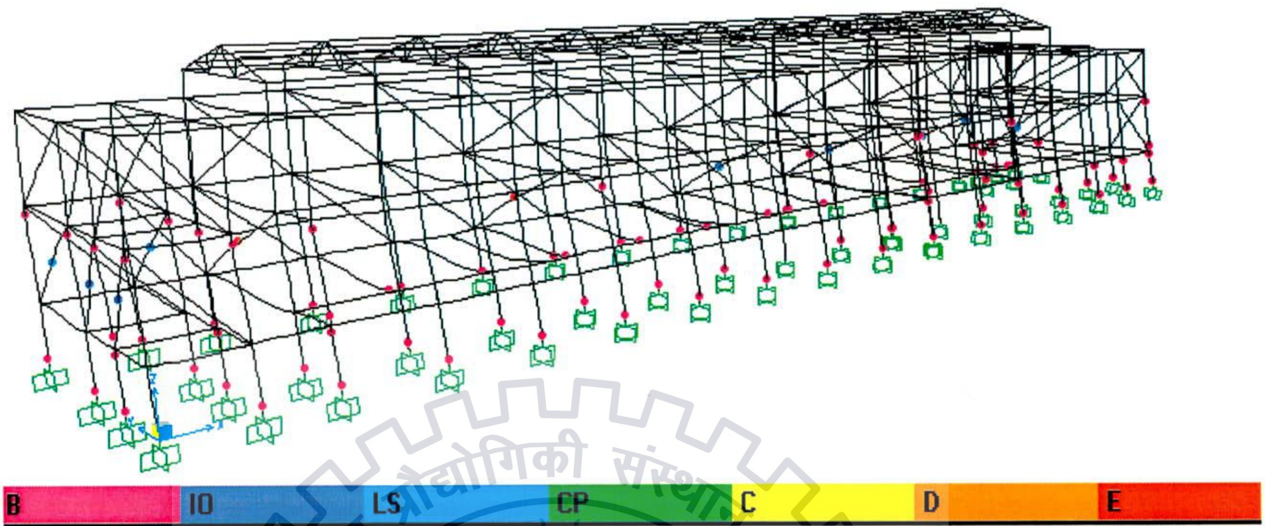

Fig 5.23: Plastic hinges at performance point at MCE in Y-direction for Block IV (Infilled Frame)

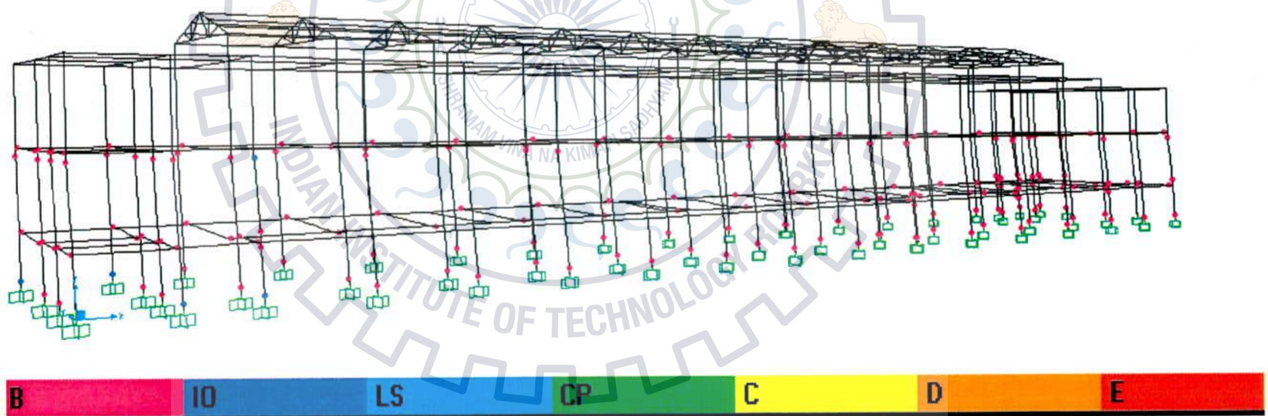

Fig 5.24: Plastic hinges at performance point at MCE in Y-direction for Block IV (Bare Frame)

5.4 SUMMARY

From the result of nonlinear pushover analysis it is evident that masonry infill panels resist seismic forces before the infill panel undergo damage. Once the crack appears in the masonry panels inelastic deformation capacity decreases. Figures 5.1 to 5.12 show capacity curve of blocks II, III and IV for with infill and without infill. Figures 5.13 to 5.24 show the hinge patterns of different blocks at performance point under MCE demand for bare as well as with infill panel.

Figures 5.3, 5.4, 5.7, 5.8 and 5.12 show saw-tooth curves which represent sudden drop in lateral force caused by failure of infill panels. And in these curves lateral force again quickly increases with displacement due to stiffness of infill.

Tables 5.1 to 5.3 represent base shear and displacement at DBE and MCE for different blocks in longitudinal and transverse directions respectively. It has been observed that due inclusion of infill frame strength increases. Tables 5.1, 5.2 and 5.3 show that base shear is large as compare to bare frame and displacement is small, which is due to contribution of infill strength and stiffness.

Table 5.4 to 5.8 represent state of hinges under DBE and MCE level. In Block II, hinges in the state beyond E is due to collapse of masonry under MCE level and same infill panel have suffered huge damage in the earthquake of 18th September 2011(Fig 3.4). Table 5.6 shows hinge state for Block III, at performance point no hinges formulation in IO to LS under DBE level for bare frame. In Table 5.7, IO-LS hinge show yielding of infill and corridor columns and LS-CP hinges are due to failure of masonry under MCE. In Block IV, IO to LS range hinges are more as compared to bare frame of the same block, which shows yielding of masonry infill panel and in the Table 5.9, 2 hinges are in the range of LS-CP level representing collapse of masonry in MCE level.

RETROFITTING OF BLOCKS

6.1 GENERAL

It has been found that blocks have Immediate Occupancy level performance under DBE. At performance point for demand of MCE level of earthquakes, state of hinges in the columns are in the range of Immediate Occupancy (IO) to Life Safety (LS) Level for Blocks II, III and IV (shown in Figs 4.7 to 4.18). For earthquake resistant design, columns should be stronger than beams and hinges should appear in beam first i.e. Strong Column Weak Beam frame system [20]. But during analysis it has been observed that plastic hinges are formed in column before its appearance in beam which shows deficiencies in design method and vulnerability of failure of structure during major earthquakes. Since building blocks are school buildings therefore required performance level for major earthquake should be IO level. So, retrofitting measures should be such that plastic hinge formation occurs in beam first and no hinges falls in range of IO-LS level.

6.2 RETROFITTING TECHNIQUES

i) Addition of shear wall:

It is one of the most commonly used retrofitting techniques for non-ductile RC frame structures, in which seismic performance of building is enhanced by addition of shear wall [3]. Added shear wall increases the lateral strength of structures.

ii) Jacketing of columns:

Jacketing of column is one of the member level retrofit techniques. It improves the axial and the shear strength of column in uniform and distributed way [3]. To improve lateral load capacity of building jacketing is generally carried out in the form of reinforced concrete jackets or steel jackets [3].

iii) Overlaying of FRP (Fibre Reinforced Polymer) on columns:

Winding of high strength glass/carbon fibre around column surface enhance seismic strength of columns [3]. GFRP (Glass Fibre Reinforced Polymer) is widely used for retrofitting due to its properties of high degree of confinement, lightweight, and flexibility in application.

6.3 TECHNIQUE USED FOR RETROFITTING

6.3.1 Strengthening of Column

After nonlinear static pushover analysis it has been observed that damages are mostly in columns and building performance can be enhanced by using only member level retrofitting techniques. GFRP wrapping on damaged column have suggested as retrofitting measure for all the blocks. The thickness of GFRP layer has been calculated according to method suggested by Teng [32] based on study of Priestley and Seible (1995), Priestley *et al.* (1996) and Seible *et al.* (1997) [32]. In this method, seismic strength of column is provided by V_C (shear strength provided by concrete), V_N (shear strength due to axial load) and V_S (contribution of transverse steel reinforcement in shear strength). V_C, V_N and V_S are given by eqns 6.1, 6.2 and 6.3 respectively.

$$V_C = K\sqrt{f_C}\,A_e \tag{6.1}$$

$$V_N = (d - x) * N / 2l \tag{6.2}$$

$$V_S = A_{sv}\,f_{ye}\,D' / S_v \tag{6.3}$$

Where, K = factor dependence on ductility [32]

f_C = Cylindrical compressive strength of concrete

A_e = Effective shear area taken as 0.8 times of gross area

N = Axial Load

d = Section depth in lateral direction

x = Neutral axis depth

L = Shear span of column

A_{sv} = Area of transverse Reinforcement

f_{ye} = Expected Yield strength of steel

D' = Distance between centres of stirrups in the lateral direction

S_v = Spacing between stirrups

To avoid de-bonding between concrete and GFRP sheet, allowed tensile stress $f_{frp,e}$ will be limited to .004 tensile strain [32]. Shear strength of FRP jacket V_{frp} for rectangular columns is given by eqn 6.4.

$$V_{frp} = 2\, f_{frp,e}\, t_{frp}\, d\, \cot\theta \tag{6.4}$$

If shear demand of column in the plastic hinges based on flexural overstrength is V_o and ϕ_v is shear strength reduction factor (taken as 0.85) then required thickness of FRP jacket is calculated by

$$t_{frp} = V_o / \phi_v - (V_c + V_N + V_S) / 2\, f_{frp,e}\, d \cot\theta \tag{6.5}$$

using the method described above and formulas, required thickness of FRP jacket is calculated for identified yielded columns beyond limit of IO for different blocks and tabulated below (Using GFRP of $f_{frp,e}$= 500MPa);

Table 6.1: Thickness of FRP jacket in hinge and non-hinge region for Block II

Column ID	Main Grid	Floor Level	$t_{frp,non-hinge}$ (mm)	$t_{frp,hinge}$ (mm)
C-5 200*350	A-1	GF & FF	0.34	0.43
C-5 200*350	B-1	GF & FF	0.34	0.43
C-2 200*350	C-1	GF	0.15	0.27
C-2 200*350	D-1	GF	0.15	0.27
C-2 200*350	E-1	GF	0.15	0.27
C-2 200*350	F-1	GF	0.15	0.27
C-2 200*350	G-1	GF	0.15	0.27
C-2 200*350	H-1	GF	0.15	0.27
C-2 200*350	I-1	GF	0.15	0.27
C-2 200*350	J-1	GF	0.15	0.27
C-2 200*350	L-1	GF	0.15	0.27
C-2 200*350	N-1	GF	0.15	0.27
C-2 200*350	P-1	GF	0.15	0.27
C-2 200*350	R-1	GF	0.15	0.27
C-5 200*350	A-2	GF	0.54	0.66
C-5 200*350	B-2	GF	0.54	0.66
C1 200*400	J-2	GF & FF	0.72	0.82
C1 200*400	K-2	GF & FF	0.72	0.82
C1 200*400	L-2	GF & FF	0.72	0.82
C1 200*400	M-2	GF & FF	0.72	0.82
C1 200*400	N-2	GF & FF	0.72	0.82

C-5 200*350	P-3	GF & FF	1.17	1.26
C-5 200*350	R-3	GF & FF	1.17	1.26
C-5 200*350	A-4	GF & FF	0.54	0.66
C-5 200*350	B-4	GF & FF	0.54	0.66
C-6 200*400	A-5	GF & FF	1.4	1.5
C-6 200*400	B-5	GF & FF	1.4	1.5
C-3 400*200	N-14	GF & FF	0.72	0.82
C-3 400*200	O-15	GF & FF	0.72	0.82
C-3 400*200	Q-17	GF & FF	0.72	0.82
C-3 400*200	R-17	GF & FF	0.72	0.82

Table 6.2: Thickness of FRP jacket in hinge and non-hinge region for Block III

Column ID	Main Grid	Floor Level	$t_{frp,non-hinge}$ (mm)	$t_{frp,hinge}$ (mm)
C-1 200*300	I-6	GF	0.41	0.45
C-1 200*300	L-6	GF	0.41	0.45
C-1 200*300	N-6	GF	0.41	0.45
C-1 200*300	R-6	GF	0.41	0.45
C-1 200*300	T-6	GF	0.41	0.45
C-1 200*300	I-15	GF	0.41	0.45
C-1 200*300	L-15	GF	0.41	0.45
C-1 200*300	N-15	GF	0.41	0.45
C-1 200*300	R-15	GF	0.41	0.45

Table 6.3: Thickness of FRP jacket in hinge and non-hinge region for Block IV

Column ID	Main Grid	Floor Level	$t_{frp,non-hinge}$ (mm)	$t_{frp,hinge}$ (mm)
C-5 200*350	A-1	GF	0.37	0.46
C-5 200*350	B-1	GF	0.37	0.46
C-2 200*350	C-1	GF	0.37	0.46
C-2 200*350	D-1	GF	0.37	0.46
C-2 200*350	E-1	GF	0.37	0.46
C-2 200*350	F-1	GF	0.37	0.46
C-2 200*350	G-1	GF	0.37	0.46
C-2 200*350	H-1	GF	0.37	0.46
C-2 200*350	I-1	GF	0.37	0.46

C-2 200*350	K-1	GF	0.37	0.46
C-2 200*350	L-1	GF	0.37	0.46
C-2 200*350	M-1	GF	0.37	0.46
C-2 200*350	N-1	GF	0.37	0.46
C-2 200*350	O-1	GF	0.37	0.46
C-2 200*350	P-1	GF	0.37	0.46
C-2 200*350	R-1	GF	0.37	0.46
C-2 200*350	T-1	GF	0.37	0.46
C-5 200*350	A-2	GF	0.54	0.66
C-5 200*350	B-2	GF	0.54	0.66
C-1 200*400	C-2	GF	0.37	0.45
C-1 200*400	D-2	GF	0.37	0.45
C-1 200*400	E-2	GF	0.37	0.45
C-1 200*400	F-2	GF	0.37	0.45
C-1 200*400	G-2	GF	0.37	0.45
C-1 200*400	H-2	GF	0.37	0.45
C-1 200*400	I-2	GF	0.37	0.45
C-1 200*400	J-2	GF	0.37	0.45
C-1 200*400	K-2	GF	0.37	0.45
C-1 200*400	L-2	GF	0.37	0.45
C-1 200*400	M-2	GF	0.37	0.45
C-1 200*400	N-2	GF	0.37	0.45
C-1 200*400	O-2	GF	0.37	0.45
C-1 200*400	P-2	GF	0.37	0.45
C-5 200*350	A-3	GF	0.59	0.67
C-5 200*350	B-3	GF	0.59	0.67
C-5 200*350	A-4	GF	0.58	0.66
C-5 200*350	B-4	GF	0.58	0.66
C-4 350*200	P-4	GF	0.50	0.55
C-4 350*200	R-4	GF	0.50	0.55
C-4 350*200	T-4	GF	0.50	0.55
C-6 200*400	A-5	GF	0.37	0.47
C-6 200*400	B-5	GF	0.37	0.47
C-1 200*400	C-5	GF	0.37	0.47
C-1 200*400	D-5	GF	0.37	0.47

C-1 200*400	E-5	GF	0.37	0.47
C-1 200*400	F-5	GF	0.37	0.47
C-1 200*400	G-5	GF	0.37	0.47
C-1 200*400	H-5	GF	0.37	0.47
C-1 200*400	I-5	GF	0.37	0.47
C-1 200*400	J-5	GF	0.37	0.47
C-1 200*400	K-5	GF	0.37	0.47
C-1 200*400	L-5	GF	0.37	0.47
C-1 200*400	M-5	GF	0.37	0.47
C-1 200*400	N-5	GF	0.37	0.47
C-1 200*400	O-5	GF	0.37	0.47
C-3 400*200	P-5	GF	0.60	0.65
C-3 400*200	Q-5	GF	0.60	0.65
C-3 400*200	S-5	GF	0.60	0.65
C-3 400*200	T-5	GF	0.60	0.65

For better confinement and effectiveness of FRP jacket, edges of rectangular column should be made rounded to give elliptical shape [32]. Before wrapping of FRP jacket, primer epoxy should be applied on column to make surface smooth Primer Epoxy coating also filled of honeycomb gaps of column surface (Fig 6.2). After one day of primer epoxy coating, FRP are wrapped on column and saturated with epoxy resin so that effective bond between concrete and FRP layer (Fig 6.3) can be achieved. FRP jacket should be wrapped at distance of 1 *inch* from joint to avoid joint failure. The length of hinge region is same as plastic hinge length(L_p) which can be found out by eqn 6.6 [27] or typically taken as 0.5 times of section depth (Fig 6.1).

$$L_p = 0.08L + 0.022\, d_b\, f_y \quad (6.6)$$

Where, d_b= diameter of bar

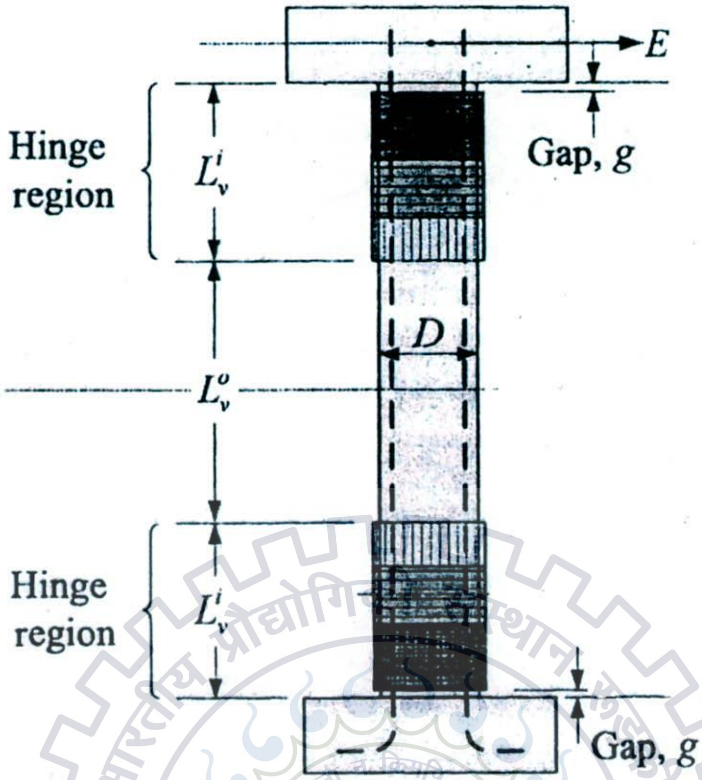

Fig 6.1: Hinge and non-hinge region of column

Fig 6.2: Primer coated column before wrapping of FRP

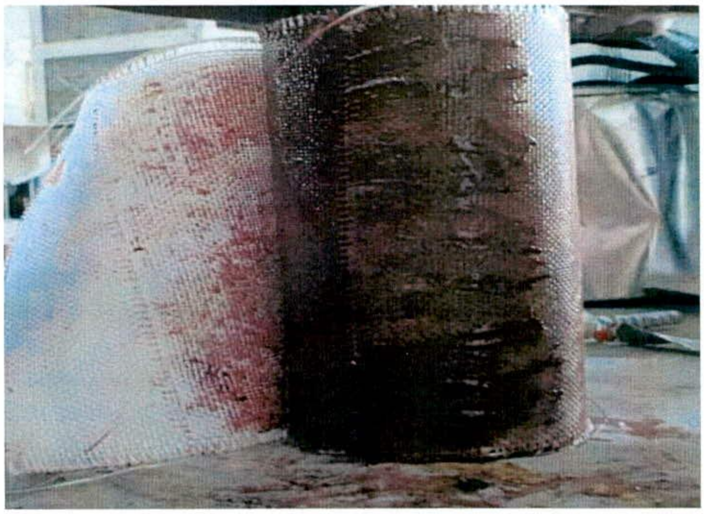

Fig 6.3: wrapping of FRP and saturating with epoxy resin

6.3.2 Repair of Damaged Masonry

Masonry wall which has collapsed or yielded beyond acceptable limit should be replaced by new masonry wall. Masonry infill panel contributes substantially in lateral load resistance in-plane, so yielded masonry wall should be strengthened by wire mesh fixing on both sides of wall. And minor crack within infill panel should be grouted using shrinkage compensated cement slurry.

6.3.3 Repair of Cracked Concrete

Crack should be filled using polymer resin grouts. The commonly used polymers are polyester, epoxy, vinyl ester and acrylic. These grouts come in two component i.e. liquid resin content and curing agent hardener. Polymer grouts are injected by pre-mixing the resins and the hardener. Mixture is injected through a pressure gun fitted with a nozzle. For wide crack filler should also be used. Figs 6.4 and 6.5 show operation and fixing of grouting machine.

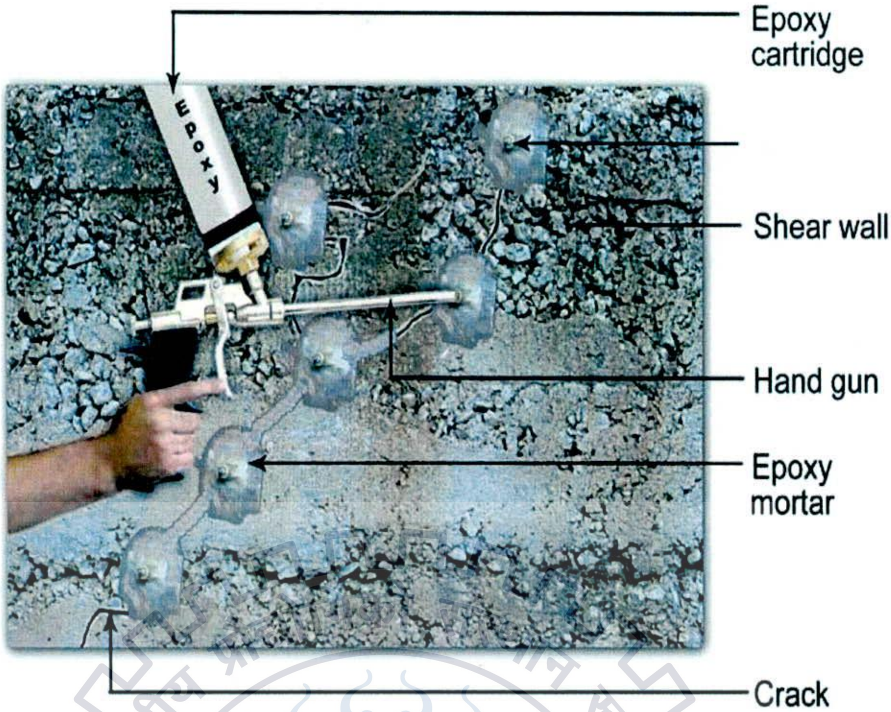

Fig 6.4: Hand epoxy Grouting Machine

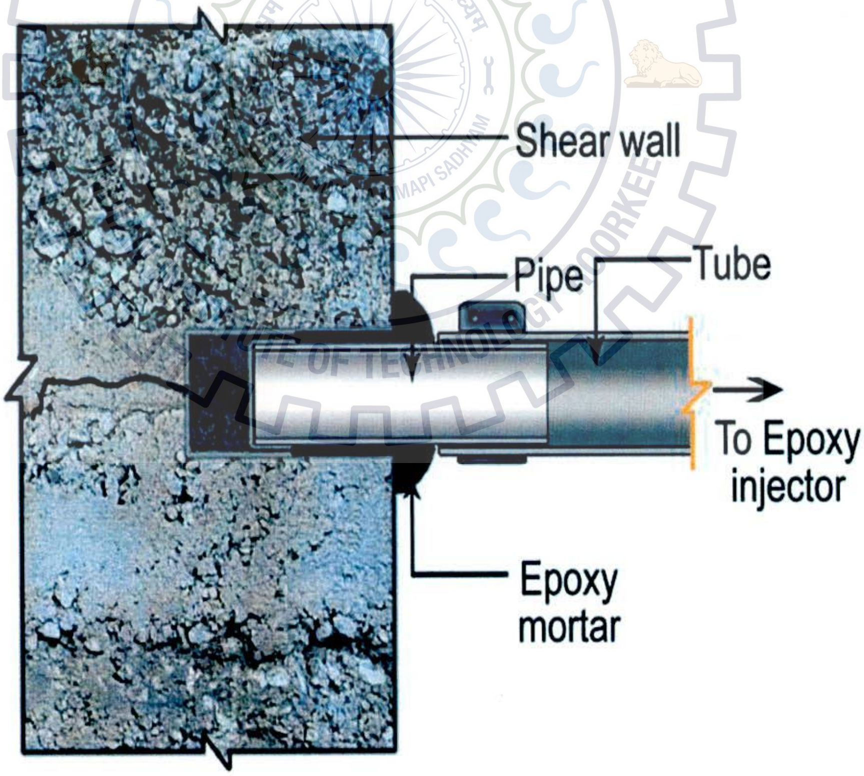

Fig 6.5: Fixing of ports for grouting

6.4 SUMMARY

It is very important to strengthen each identified vulnerable component of building to achieve desired performance objective. Shear strengthening of column is done using FRP jackets. Also, masonry infills need to be repaired by wire meshing or to be replaced by new walls. Concrete cracks can be filled by grouting technique using shrinkage compensating cement slurry grout.

CONCLUSION

A procedure of seismic retrofitting suitable to school building has been developed considering Kendriya Vidyalaya, Gangtok as an example. School is located on a hill slope having several building blocks at various elevations. On September 18, 2011 earthquake caused damage to almost all the blocks. Seismic retrofitting of these building blocks was required to be under taken using the state of art technology.

As a first step site visit was undertaken and extent of damage was studied and damages were recorded in the form of photographs. Blocks II, III and IV have undergone damages which are taken as an example for seismic retrofitting.

Plan and elevation of the blocks are not similar, so strength evaluation of block II, III and IV has been done individually. 3D Modelling of three school building blocks with and without masonry infill panel have been done using SAP 2000 Version14. Properties of materials has been taken as specified in design drawings, detailed in-situ investigation of material property has not been done. Loads and load combination are taken as per Indian standards. For damage assessment, building blocks have been evaluated for DBE and MCE level of earthquakes. To compare the infill strength and stiffness under seismic load infilled model has been developed for DBE and MCE conditions. Dynamic response of blocks has been found out by modal analysis for infill as well as for the bare frame. Time periods for first three fundamental modes have been calculated. It has been observed that infill reduces the time period that means infill makes the building frame stiffer.

Performance level is evaluated for each block by nonlinear analysis under DBE and MCE levels. Nonlinear static analysis has been performed for same 3D computer model. Expected yield strength of steel and concrete has been taken for pushover analysis. And force-deformation relationships for plastic hinges are assigned as per ASCE-41 [4]. Pushover curves are plotted for each block in both principal directions. Seismic demand is provided for DBE and MCE level of earthquakes. And performance point has been found out using capacity spectrum method as per ATC-40 [2]. Number of hinges at each stage of performance level at performance point has also been found. It has been observed that Performance point is within Immediate Occupancy level (IO) at DBE since no hinges are formed in beyond IO level in blocks for bare frame. Hinges beyond IO level in infilled frames are appeared under DBE level demand due to yielding of masonry walls. Performance points are also calculated for demand of MCE level earthquake. Analysis of infilled frame shows that masonry infill panel increases the lateral load capacity and stiffness of frame. After collapse of set of infill panel capacity decreases and again quickly regains the capacity. It has been seen that columns fail earlier the failure of beams which shows Strong-Beam Weak-Column deficiencies. Blocks are non-ductile RC frames which may lead to formation of hinges in columns.

After damage analysis and evaluation of performance point for each block, up-gradation of block has been suggested. An analysis result shows that all the blocks hinges have formed in columns in retrofitting. Therefore, member level retrofitting has been suggested for columns with FRP overlaying or RC Jacketing or Steel jacketing, to increase the seismic resistance capacity of columns.

Here, for damaged column retrofitting is done using GFRP (Glass Fibre Reinforced Polymer). Thickness of FRP jacket in hinge and non-hinge region is calculated. For walls and cracked concrete repair methods are also suggested. For the masonry walls, repair techniques are suggested in the form of wire meshing on both side of wall and replacement with new wall for failed masonry. Cracked concrete can be filled by grout using Automatic Grouting Machine. Retrofitting and repairing of damaged member can bring building blocks to required IO level performance in future earthquakes.

APPENDIX- A

Sample Calculation of Thickness of GFRP jacket

Column ID: C1 200*400

Grid Point Location: J-2

k= 0.083 (hinge region)

=0.17 (Non-hinge region)

A_e = 0.8*200*400 = 64000 mm²

d = 200 mm

x = 130 mm

N = 448.42 kN

l = 3200 mm

f_{ye} = 312.5 N/mm²

f_c = 12 N/mm²

A_{sv}= 28.27 mm²

D' = 120 mm

S_v = 150 mm

$V_o = 165.2\ kN$ and $\phi_v = 0.85$

For Hinge region,

$V_C \ = K\sqrt{f_C}\,A_e = 0.083 * \sqrt{12}\ * 64000 = \ 18.40\ kN$

$\mathrm{V_N} = (d - x) * N/2l = \ (200 - 130) * 448.42\ /\ 2 * 3200\ = 4.90\ kN$

$V_S\ = A_{sv}\, f_{ye}\, D'/\ S_v\ = 28.27 * 312.5 * 120/150 = 7.07\ kN$

$V_c\ +\ V_N\ +\ V_S$=30.37 kN

$t_{frp}\ = V_o/\ \phi_v$ - $(V_c\ +\ V_N\ +\ V_S)/\ 2\, f_{frp,e}\, d\cot\theta = ((165.2/0.85) - 30.37))/2 * 500 * 200 * \cot 45$

$= \ 0.819\ mm\ \ \cong 0.82\ mm$

For Non-Hinge Region,

$V_C = \ K\sqrt{f_C}\,A_e = 0.17 * \sqrt{12}\ * 64000 = \ 37.69\ kN$

$\mathrm{V_N} = 4.90\ kN$

$V_S\ = 7.07\ kN$

$V_c\ +\ V_N\ +\ V_S = 49.66\ kN$

$t_{frp}\ = V_o/\ \phi_v$ - $(V_c\ +\ V_N\ +\ V_S)/\ 2\, f_{frp,e}\, d\cot\theta = ((165.2/0.85) - 49.66))/2 * 500 * 200 * \cot 45$

$= \ 0.72\ mm$

APPENDIX- B

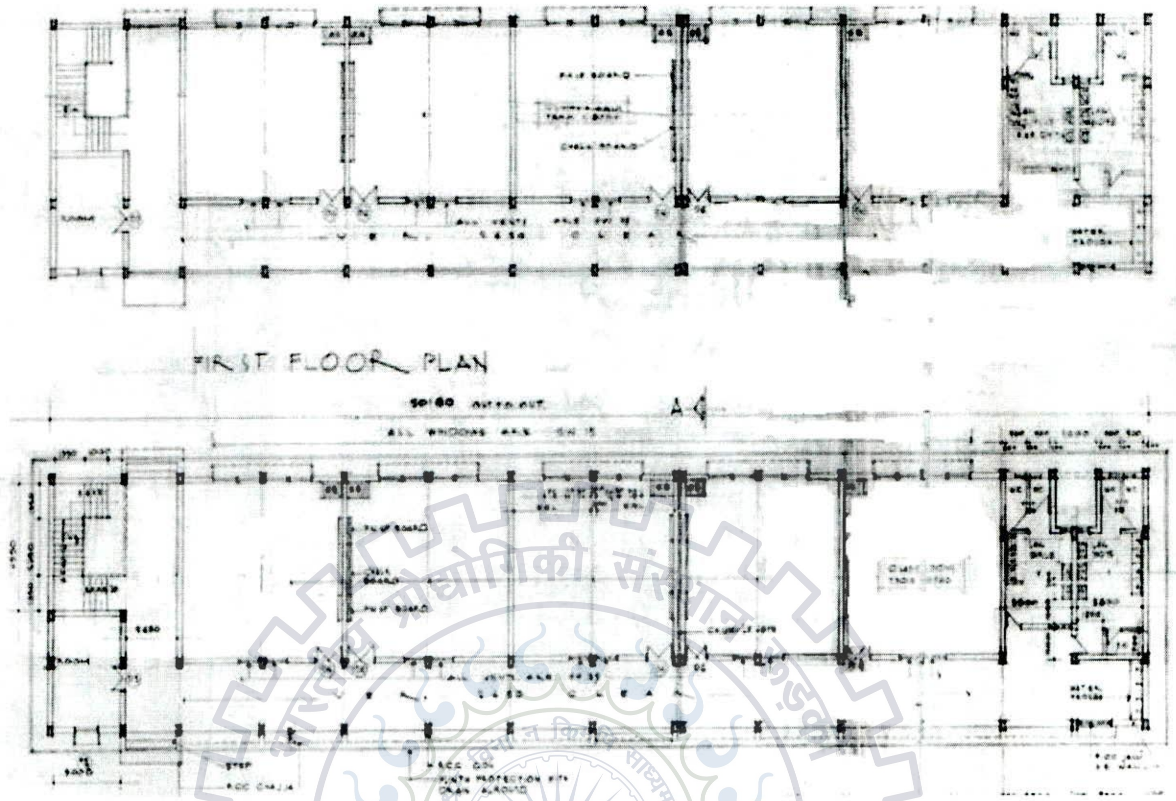

Fig B-1: Plan of Block II

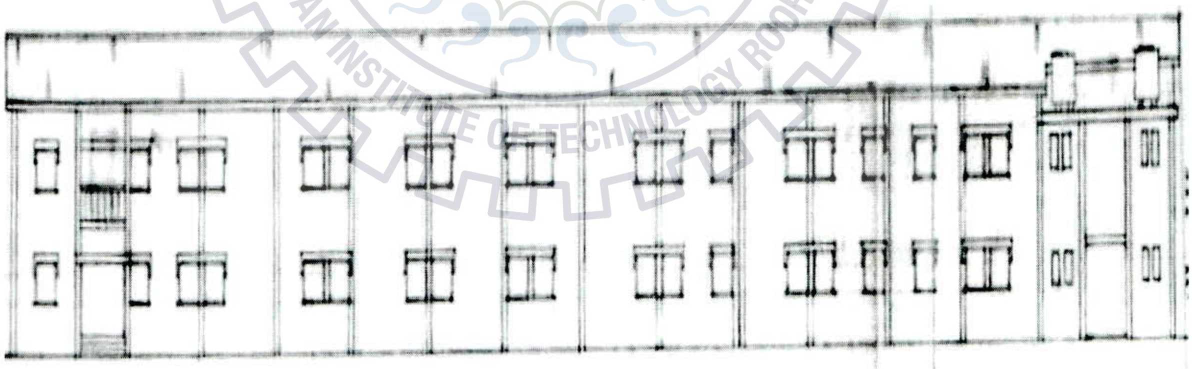

Fig B-2: Typical elevation of Block II and IV

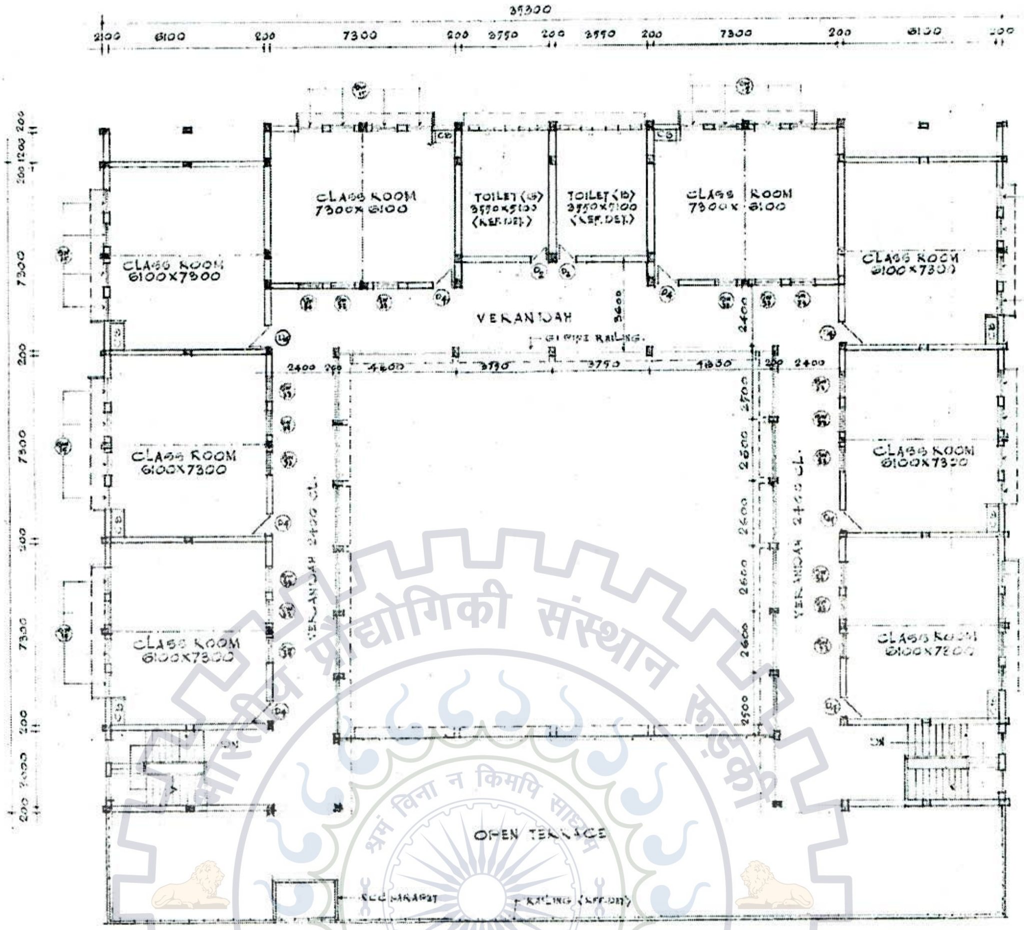

Fig B-3: Plan of Block III

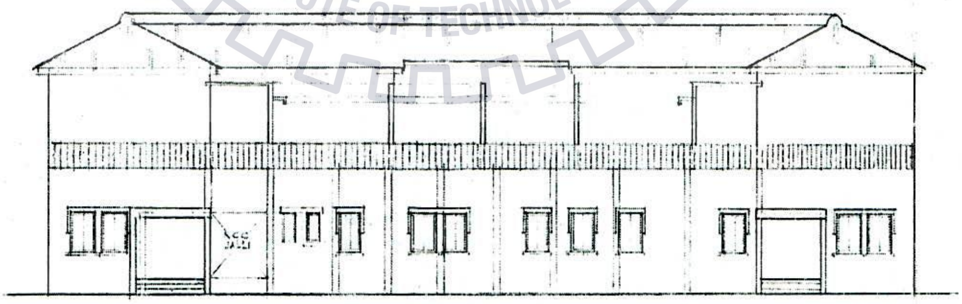

Fig B-4: Elevation of Block III

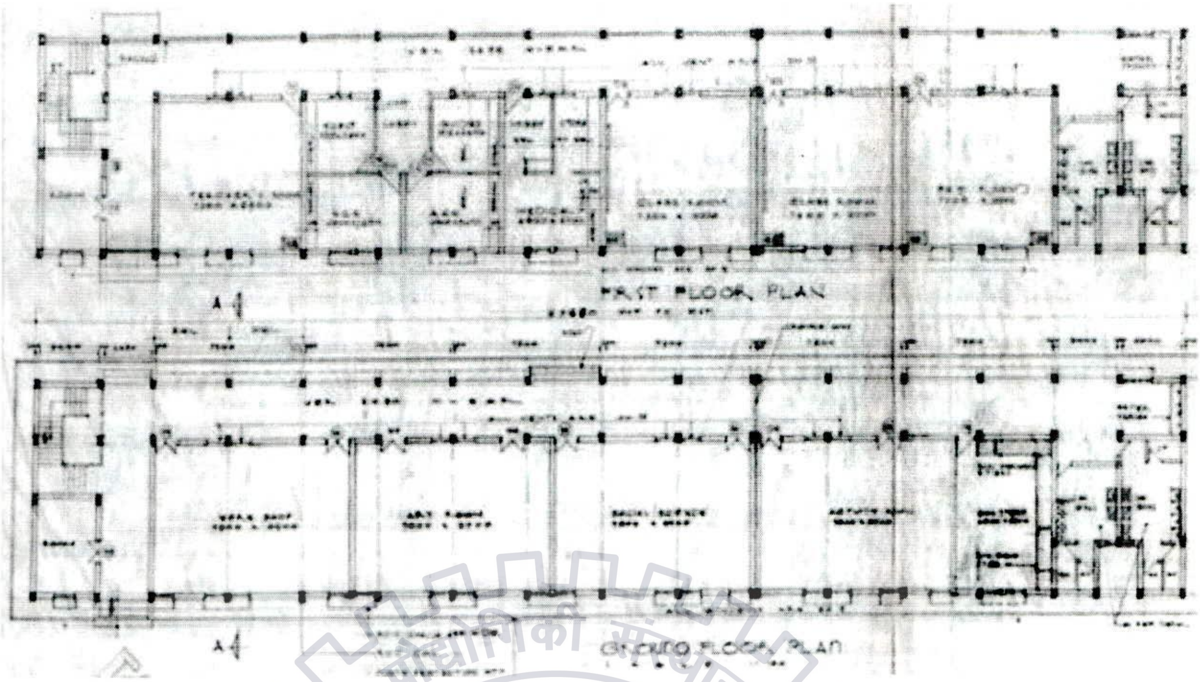

Fig B-5: Plan of Block IV

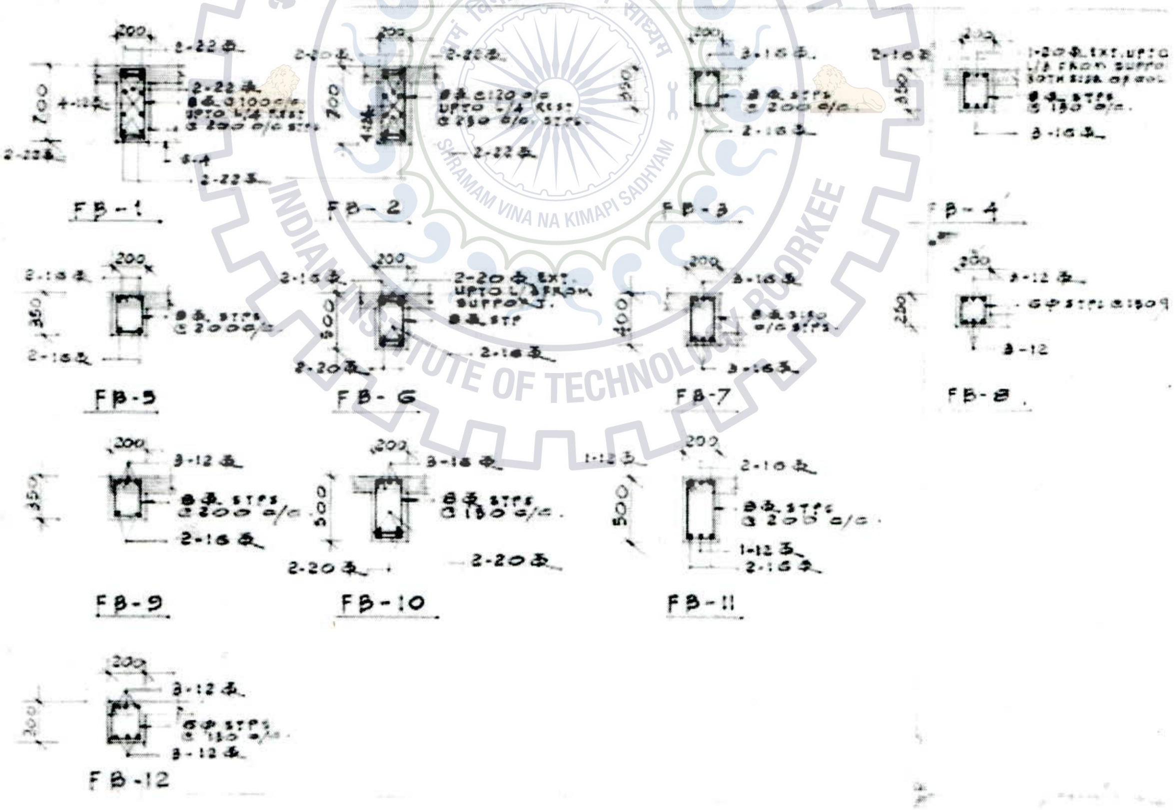

Fig B-6: Detailing of beams for Block II and IV

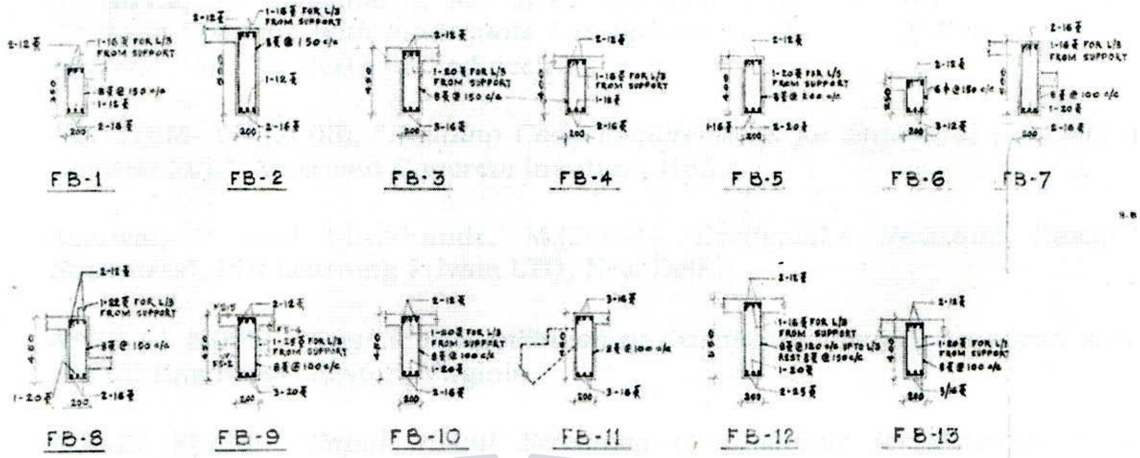

Fig B-7: Detailing of beams for Block III

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