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SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDING – A CASE STUDY

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ABSTRACT

Earthquakes are a huge concern for structures; causing losses of peoples' lives, damages and collapse of homes. The main objective of the present study is to analyze and strengthen an existing old building by using proper and innovative retrofitting techniques and collected data from previous studies. The selected building is a two-storey of Backward class Boys Hostel in Akola consists of prefabricated concrete. The present study evaluates the performance of G+1 hostel building using equivalent seismic analysis of an existing building. In the next step the study evaluates the performance of the selected building for an additional floor. It is observed that the present building is inadequate to withstand the seismic loading for an additional building construction. The column member sizes failed on the addition of an extra storey and hence objective of the study is to develop technique to protect the existing structure. The study recommends jacketing of failed column members by increasing the column sizes and reinforcement requirement of the modified structure. The study also noted that the provided member sizes and reinforcement are sufficient to withstand the load of an additional two storey floors to meet the requirement of the occupancy at the hostel building.

Keywords: seismic analysis, jacketing, retrofitting, reinforced concrete building, demand capacity ratio.

I. INTRODUCTION

The aftermath of an earthquake manifests great devastation due to unpredicted seismic motion striking extensive damage to innumerable buildings of varying degree i.e., either full or partial or slight. This damage to structures in its turn causes irreparable loss of life with a large number of casualties. As a result, frightened occupants may refuse to enter the building unless assured of the safety of building from future earthquakes. It has been observed that majority of such earthquake damaged buildings may be safely reused if they are converted into seismically resistant structures by employing a few retrofitting measures. This proves to be a better option catering to the economic considerations and immediate shelter problems rather than replacement of buildings. Moreover, it has often been seen that retrofitting of buildings is generally more economical as compared to demolition and reconstruction even in the case of severe structural damage. Therefore, seismic retrofitting of building structures is one of the most important aspects for mitigating seismic hazards especially in earthquake prone countries. Various terms are associated to retrofitting with a marginal difference like repair, strengthening, retrofitting, remoulding, rehabilitation, reconstruction etc. but there is no consensus on them. The method of retrofitting principally depends on the horizontal and vertical load resisting system of the structure and the type of materials used for parent construction. It also relies on the technology that is feasible and economical. The understanding of mode of failure, structural behaviour and weak and strong design aspects as derived from the earthquake damage surveys exercise considerable influence on selection of retrofitting methods of buildings. Usually, the retrofitting method is aimed at increasing the lateral resistance of the structure. The lateral resistance includes the lateral strength or stiffness and lateral displacement or ductility of the structures. The lateral resistance is often provided through modification or addition of retrofitting elements of an existing structure in certain areas only. The remaining elements in the structure are usually not strengthened and are assumed to carry vertical load only, but in an earthquake, all components at each floor, retrofitted or not, will undergo essentially the same lateral displacements. While modified or added elements can be designed to sustain these lateral deformations, the remaining non-strengthened elements could still suffer substantial damage unless lateral drifts are controlled. Therefore, caution must be taken to avoid an irregular stiffness distribution in the strengthened structure. Thus, the ability to predict initial and final stiffness of the retrofitted structure need clarification and quantification. Consequently, it is suggested that the design of retrofitted schemes should be based on drift control rather than on strength consideration alone. The use of three-dimensional analysis is recommended to

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identify and locate the potential weakness of the retrofitted building. It was seen that a number of institutions, researchers, academicians, industrialists and many agencies working in some field directly or indirectly related to the construction industry have put in enormous efforts and finances to work towards the improvement of knowledge and encouraged engineers to change the state-of-the-art practices and switch to modern and revalued techniques. The literature available has also inculcated a sense of responsibility in the construction industry to face hazards like earthquakes with confidence and without fear with such a large ocean of information and expertise being made available. RC jacketing was selected as the retrofitting technique by [1] and employed to the weak member and later the member in the structure was compared with the bending moment value before and after providing retrofitting. It was determined that RC jacketing strengthened the structure, which was vulnerable to seismic activity. The case studies of actual buildings where traditional and innovative retrofitting methods have been applied was presented by [2]. Among all the methods of seismic retrofitting, particular attention is devoted to the method which is based on stiffness reduction. A review of the seismic upgrading techniques was studied by [3] which target reinforced concrete (RC) buildings, as such buildings constitute a large portion of the existing building stock. A design procedure to evaluate the mechanical characteristics of hysteretic Energy Dissipation Bracing (EDB) systems for seismic retrofitting of existing reinforced concrete framed buildings was studied by [4]. The proposed procedure, aiming at controlling the maximum interstorey drifts, imposes a maximum top displacement as function of the seismic demand and, if needed, regularizes the stiffness and strength of the building along its elevation. The study by [5] highlights the principles of assessing and retrofitting of structure against seismic events. A three-dimensional R.C. frame designed with linear elastic dynamic analysis using response spectrum method. The computer software package STAAD Pro is used for dynamics analysis technique is used to assess the performance of a reinforced concrete building. The effect of infill strength and stiffness in the seismic analysis of multi storey building was performed by [6]. The analysis procedure is applied for the evaluation of existing design of a reinforced concrete bare frame, frame with infill and frame with infill and external shear wall. In order to examine the performance of these models, the pushover analysis for seismic evaluation of existing buildings was performed. The global and local retrofitting techniques are discussed by [7]. Conventional techniques (Local and global) of retrofitting were compared with modern technique (Fiber Reinforced Polymers). The study by [8] compared the behavior of flat slab with old traditional two-way slab along with effect of shear walls on their performance. The seismic retrofitting of frame structures using hysteretic dampers is a very effective strategy to mitigate earthquakeinduced risks. However, its application in current practice is rather limited since simple and efficient design methods are still lacking, and the more accurate time-history analysis is time-consuming and computationally demanding [9]. The introduction of energy dissipation devices within a building structure creates a number of analysis and design problems that must be considered by the structural engineer, but which are not directly addressed in code-based documents. Some of these problems have been discussed by [10]. The structure without retrofit shows more storey displacement, storey drift and storey shear. Different retrofit techniques such as RC column jacketing method, steel jacketing method and bracing methods are adopted by [11]. The step-by-step retrofitting of buildings by using steel plate shear walls (SPSWs) with the aid of SAP2000 programme was demonstrated by [12]. One type of reinforced concrete building is selected for evaluation. This building represents the most used forms of residential buildings in the Sudan, in terms of geometric form, and dimensions. The use of carbon fiber as retrofitting material was proved by [13] after detailed study it was found that these techniques have a problem of debonding of CFRP from concrete. The debonding behavior and strength was discussed by loading tests and analyses. A five storeyed RC framed building was considered by [14] to evaluate the seismic performance through static non-linear pushover analysis. Damages of building frame are found out for different scenarios using energy approach methods. At the end, retrofitting measures are done to increase the base shear capacity and reduce damage. The base shear capacity of the frame increased due to jacketing of bottom columns from 0.5 × 0.5 m to 0.6 × 0.6 m, 0.7 × 0.7 m are 30 % and 85 % respectively. Thus, from the above-mentioned studies following objectives are derived for the present study.

1) To study different retrofitting Techniques.
2) To study Typical Loading on Frame for different load cases.

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3) To study the analysis of existing building (Model 1), evaluate the performance of a proposed Building with additional floor. (Model 2), perform column jacketing of failed structure (Model 3) and finally check the hostel Building with modified G+3 floors (Model 4) using Staad Pro.

In the next section, the methodology adopted in the present study is elaborated.

II. METHODOLOGY

For seismic analysis, IS 1893 (Part 1):2002 has three methods namely, Equivalent Static Analysis (ESA), Response Spectrum Analysis (RSA) and Time History Analysis (THA). Equivalent Static analysis is empirical, simple but approximate and uses empirical formula for time period. Static seismic loads are obtained using approximate fundamental time period. Response spectrum is the plot of maximum response (acceleration, velocity or displacement) as a function of natural frequency of single degree of freedom system, for a given earthquake base excitation and of course, for a given damping. This response spectrum is used in the RSA. IS 1893, gives smooth design response spectrum of acceleration. Response spectrum is given for three different types of soil and factors for different damping are given. In RSA, modal properties of structures are used. In this method, first modal properties are obtained using free vibration analysis and then response is obtained for each mode. The modal responses are combined using modal combination rule. In response spectrum analysis static seismic loads are obtained, however, they are based on natural modes and time period of buildings. In time history analysis, dynamic response of a structure is obtained. For routine and regular type of buildings, equivalent static analysis is used. For irregular buildings, response spectrum analysis is suggested. For structures like Nuclear Power Plants, site specific spectra and time histories of ground acceleration are to be used. For such cases, time history analysis is used. In the present study, the steps involved in modeling and retrofitting analysis of hostel building in Akola as shown in figure 1 using the STAAD software is explained. To perform dynamic analysis in STAAD following steps must be followed:

i. Geometric Modeling ii. Sectional Properties & Material Properties iii. Supports : Boundary Conditions iv. Loads & Load combinations (Dynamic) v. Analysis and Design Specification

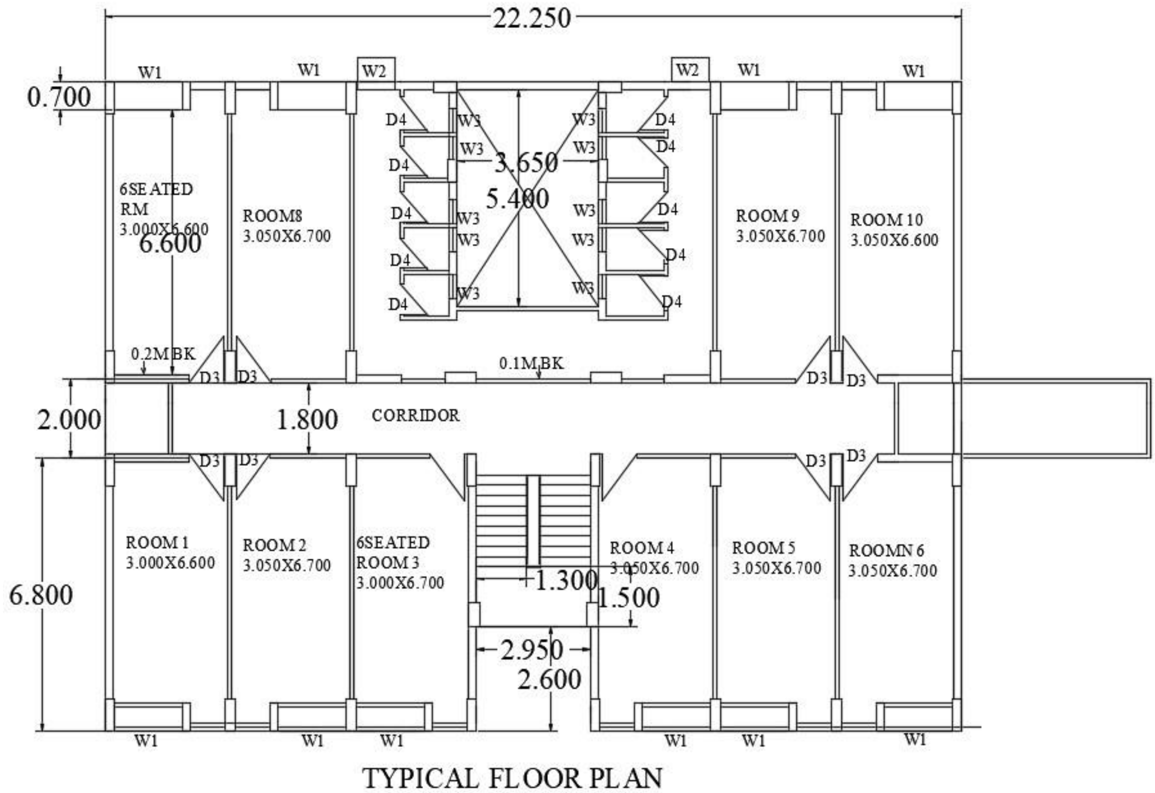

Figure 1. Typical floor plan of the hostel building, Akola, Maharashtra.

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III. MODELING AND ANALYSIS

Geometric Modeling

To model any structure in STAAD the first step is to specify the nodal coordinate data followed by selection of elements from element library. For the present work beam elements are selected to model the structure.

Sectional & Material Properties

The element selected for modeling is then assigned the properties if the element is beam the cross section of beam is assigned. For plate elements thickness is assigned. After assigning the sectional property to the member it is important to assign it with member properties. Material properties include modulus of elasticity, poisson's ratio; weight density, thermal coefficient, damping ratio and shear modulus.

Support and boundary condition

After assigning the sectional and material properties, boundary condition is assigned to the structure in form of fixed, hinged and roller support to structure. In the present work boundary condition is assigned in form of fixed support.

Load and load combination

Loads are a primary consideration in any building design because they define the nature and magnitudes of hazards are external forces that a building must resist to provide a reasonable performance (i.e., safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (size and shape) and location (climate and site conditions).Ultimately, the type and magnitude of design loads affect critical decisions such as material collection, construction details and architectural configuration. Thus, to optimize the value (i.e., performance versus economy) of the finished product, it is essential to apply design loads realistically. In the present project works following loads are considered for analysis.

Dead Loads (IS- 875 PART 1):

Dead loads consist of the permanent construction material loads compressing the roof, floor, wall, and foundation systems, including claddings, finishes and fixed equipment. Dead load is the total load of all of the components of the components of the building that generally do not change over time, such as the steel columns, concrete floors, bricks, roofing material etc. In STAAD Pro assignment of dead load is automatically done by giving the property of the member. In load case we have option called self-weight which automatically calculates weights using the properties of material i.e., density. In the present study the following loads are taken under dead load.

1. Columns

a. Wt. of column at level 1 = 3.15/2 x .25 x .8 x 25 x 32 = 252 kN

b. Similarly, Wt of columns at level = 504 kN

c. And Wt. of columns at level 3 = 372 kN

2. Beams

a. Wt. of beams at level 1

i. 3.125 x 0.25 x (0.4-0.1) x 25 x 8 = 46.875 kN

ii. 3.15 x 0.25 x (0.4-0.1) x 25 x 8 = 47.25 kN

iii. 2.45 x 0.25 x (0.4-0.1) x 25 x 4 = 18.375 kN iv. 2.75 x 0.25 x (0.4-0.1) x 25 x 4 = 20.625 kN

v. 4.25 x 0.25 x (0.4-0.1) x 25 x 2 = 15.94 kN

vi. 6.2 x 0.3 x (0.6-0.1) x 25 x 16 = 372 kN vii. 2.6 x 0.25 x (0.25-0.1) x 25 x 8 = 19.5 kN

Total beam weight at storey 1 540.565 kN

b. Wt. of beams at level 2= 540.565 kN

c. Wt. of beams at level 3= 540.565 kN

Slab

d. Wt. of slab at level 1

i. 0.1 x 6.2 x 3.125 x 25 x 4 = 193.75 kN ii. 0.1 x 2.6 x 3.125 x 25 x 2 = 40.625 kN

iii. 0.1 x 6.2 x 3.15 x 25 x 4 = 195.3 kN iv. 0.1 x 2.6 x 3.15 x 25 x 2 = 40.95 kN v. 0.1 x 2.45 x 6.2 x 25 x 4 = 151.9 kN vi. 0.1 x 2.6 x 2.45 x 25 x 2 = 31.85 kN vii. 0.1 x 4.25 x 2.6 x 25 x 1 = 27.63 kN Total = 682 kN

e. Wt. of slab at level 2 = 682 kN

f. Wt. of slab at level 3= 0

3. Walls

a. Wt. of walls at level 1 = 956.8 kN

b. Wt. of walls at level 2 = 2918.24 kN

c. Wt. of walls at level 3 = 2918.24 kN

Live Loads (IS 875 PART 2):

Live loads are produced by the use and occupancy of a building. Loads include those from human occupants, furnishings, no fixed equipment, storage, and construction and maintenance activities. As required to adequately define the loading condition, loads are presented in terms of uniform area loads, concentrated loads, and uniform line loads. The uniform and concentrated live loads should not be applied simultaneously on a structural evaluation. Concentrated loads should be applied to a small area or surface consistent with the application and should be located or directed to give the maximum load effect possible in end- use conditions. In the present project the following loads are taken under live load.

1. Live Load at level 1 = 0 kN

§ 7.3.2 of IS-1893 2002

2. Live Load at level 2 = 1320 kN

3. Live Load at level 3 = 0 kN

§ 7.3.2 of IS-1893 2002

After assigning the primary and generated load case to the structure the combination of loads is assigned. Table 1 shows primary and load combination assigned to the structure. The lateral loads calculated are then applied to the structure at each column node in the respective storey. For the analysis following load combinations specified by the IS 1893: 2002 are used. The basic load combinations given by the code as per clause 6.3.1.2 are as follows.

Table 1 Primary and Load combination

Type	L/C	Name
Primary	1	DL
Primary	2	LL
Primary	3	EQX+
Primary	4	EQX-
Primary	5	EQZ+
Primary	6	EQZ-
Combination	7	1.5(DL+LL)
Combination	8	1.5(DL+EQX+)
Combination	9	1.5(DL+EQX-)
Combination	10	1.5(DL+EQZ+)
Combination	11	1.5(DL+EQZ-)
Combination	12	1.2(DL+LL+EQX+)
Combination	13	1.2(DL+LL+EQX-)

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Combination	14	1.2(DL+LL+EQZ+)
Combination	15	1.2(DL+LL+EQZ-)
Combination	16	0.9DL+1.5EQX+
Combination	17	0.9DL+1.5EQX-
Combination	18	0.9DL+1.5EQZ+
Combination	19	0.9DL+1.5EQZ-

Seismic Load Calculation

• **Design Seismic Base Shear**

As per the draft code on Seismic Evaluation and Strengthening of Existing Building the design seismic base shear is modified considering the existing conditions of the building. The modified base shear is given as equation 1

$V_{bm} = A_{hm} \times W$ ... (1)

$= (U \times A_h) \times W$

Where,

U - Factor for reduced usable life of the building

A_h - Design horizontal seismic coefficient, given as equation 2

$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{Sa}{g}$ ... (2)

The building has the same dimensions in both directions, also the effect of infill is not considered. Hence, the time period in both directions is given by equation 3 ,

$T = 0.075 \times h^{0.75}$ ... (3)

$= 0.075 \times 9.3^{0.75}$

≈ 0.4

Corresponding to T = 0.4, medium soil site and 5% damping, $\frac{Sa}{g} = 2.5$ § Figure 2 of IS-1893 2002

The building is situated in area of medium seismicity i.e. Zone III.

Z = 0.16 § Table 2 of IS-1893 2002 The importance factor for the considered building,

I = 1.50 § Table 6 of IS-1893 2002

The response reduction factor for the building, R = 3.0

Thus,

$A_h = \frac{0.16}{2} \times \frac{1.50}{3} \times 2.5$

$= 0.1$

$A_{hm} = 0.67 \times Ah$ ... (4)

$= 0.67 \times 0.1 = 0.067$

The elevation of the hostel building is depicted in figure 2. The summary of the seismic weight calculation is depicted in table 2 and 3 and the values are assigned in STAAD software. Total seismic load of the building = 12226.975 kN Hence,

Modified Seismic Base Shear, V_{bm} = 1348 kN At any level the seismic load is calculated using equation 5

$Q_i = V_B \times \frac{W_i h_i^2}{\sum W_i h_i^2}$ ... (5)

Where,

Q_i = Design lateral force at floor i, W_i = Seismic weight of floor i, h_i = Height of floor i measured from base n = Number of storeys in the building is the number of levels at which the masses are located.

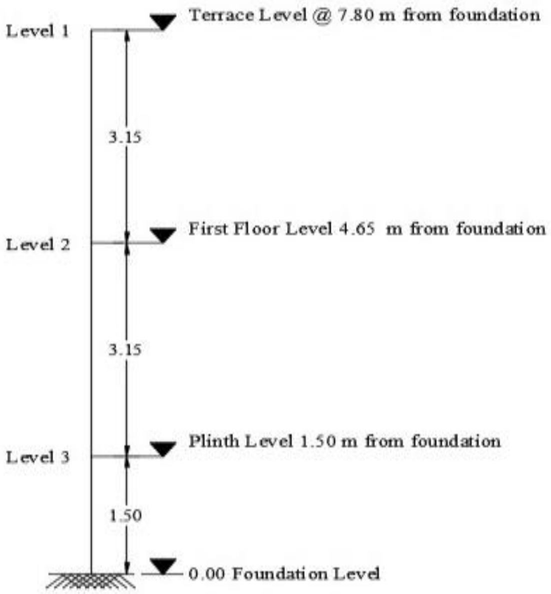

Figure 2. Elevation of the hostel building model.

Table 2 Seismic Load Calculations

Load	**Member**	**At Level 1** (in kN)	**At Level 2** (in kN)	**At Level 3** (in kN)
Dead Load				
	Columns	252	504	372
	Beams	540.565	540.565	540.565
	Slab	682	682	0
	Walls	956.8	2918.24	2918.24
Live Load				
	Slab	0.00	1320	0.00
Total		**2431.365**	**5964.805**	**3830.805**

Using STAAD software four building models are analyzed. Model is G+1 hostel building as shown in figure 2 and properties as tabulated in table 4.

Table 3 Vertical Distribution of Lateral Forces by Static Method

Level	**W_i**	**h_i** (in m)	**Wi*Hi2**	**Q_i** (in kN)	**Q_i / Column** (in kN)	**Base Shear** (in kN)
1	2431.365	7.8	147924.25	698.39	21.82	698.39
2	5964.805	4.65	128973.99	608.92	19.03	1298.31
3	3830.805	1.5	8619.31	40.69	1.27	1339
			285517.55	**1339**		

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Table 4 Summary of the member sizes of the existing Hostel building, Akola.

1	Number of Storey	G+1
2	Plinth Height	1.5m
3	Floor to Floor Height	3.15m
4	Type of Structure	Residential Structure
5	Grade of Concrete	M20
6	Yield Strength of Steel	Fe 415
7	Footing Depth	1.5m
8	Slab Thickness	0.1m
9	External Wall Thickness	230mm
10	Internal Wall Thickness	100mm
11	Column Dimension	(250X800) mm
12	Beam Dimension	(300X600) mm, (250X400) mm
13	Live Load (IS 875 – Part 2)	4 kN/m^2
14	Wall Load	14.03N/m

The above-mentioned loads are applied to model I shown in figure 3 is the existing structure without any modifications in its structural configuration or section sizes.

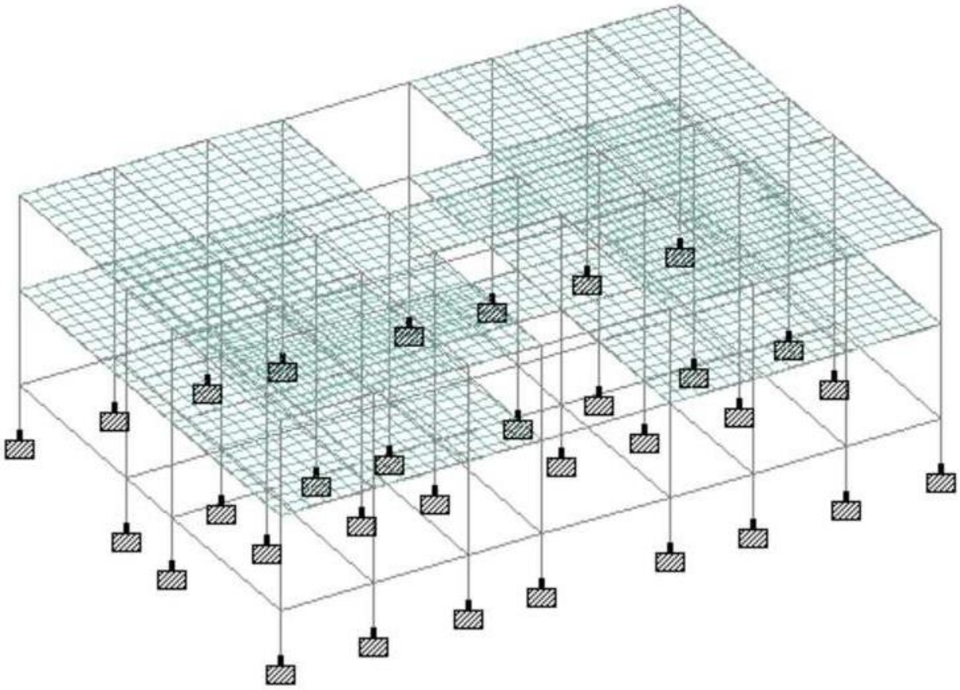

Figure 3. Existing Building Model. (Model 1)

Model II in Figure 4 is modified by adding additional floor on structure. To check the capacity of the structural members.

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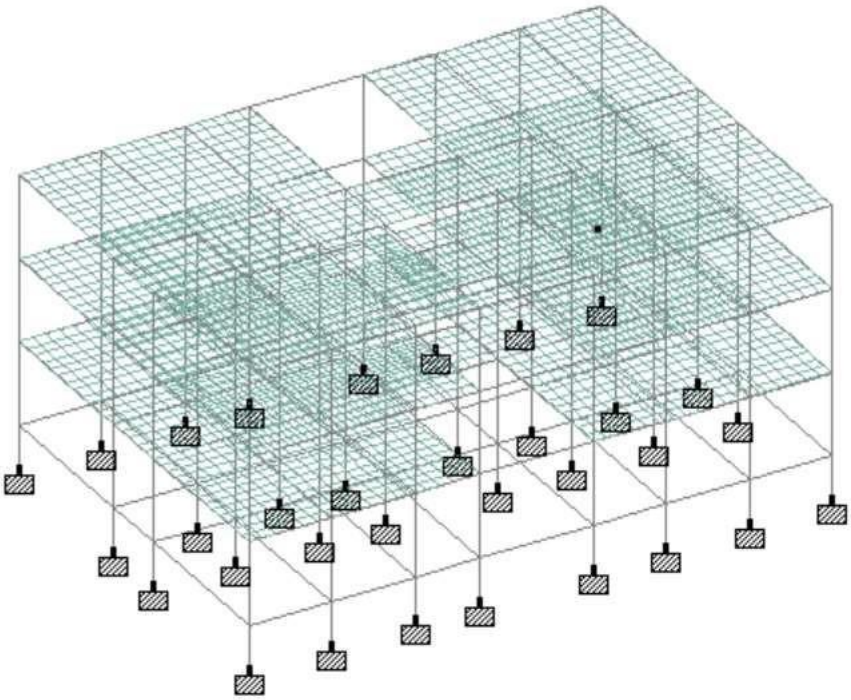

Figure 4. Proposed Building with additional floor. (Model 2)

Whereas in Model III is the retrofitted structure provided jacketing to the failure column as shown in figure 5. The study also checked the performance of G+3 building as shown in figure 6 with the jacketed columns and observed that the modified column sizes are suitable and safe for the building.

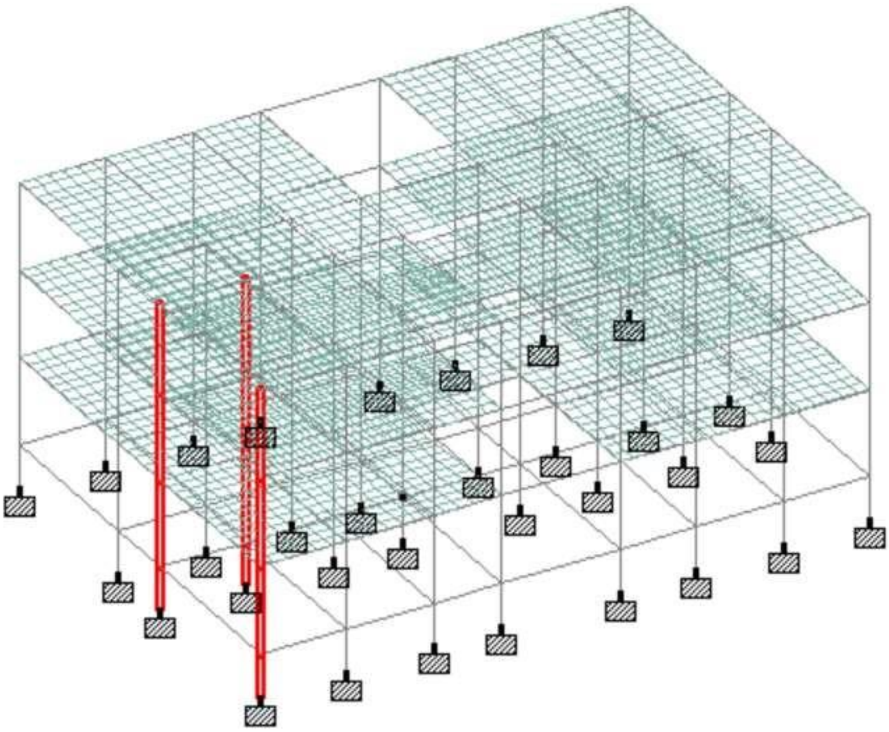

Figure 5. Column Jacketing of failed structure. (Model 3)

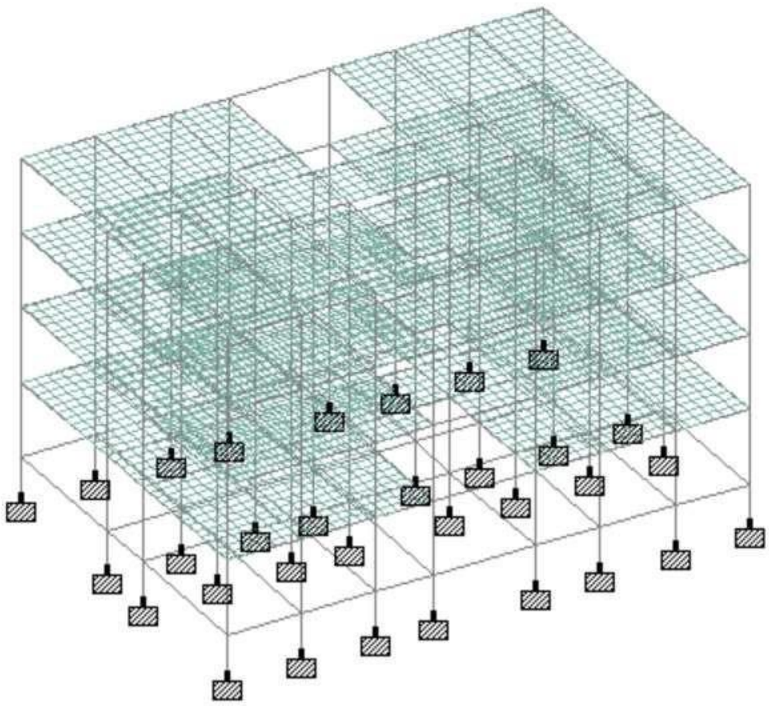

Figure 6. Hostel Building with modified G+3 floors (Model 4)

The structure is modelled using the STAAD pro software considering a rigid diaphragm and the layout of the building is plotted in figure 7. The analysis will give the demand in each of the member. This will be due to one of the combination cases specified. The results thus obtained would support and confirm reasoning during the preliminary observations and further analysis with application of the calculated seismic forces will quantify the requirement of retrofitting. In the next section the results and discussion of the present study is discussed.

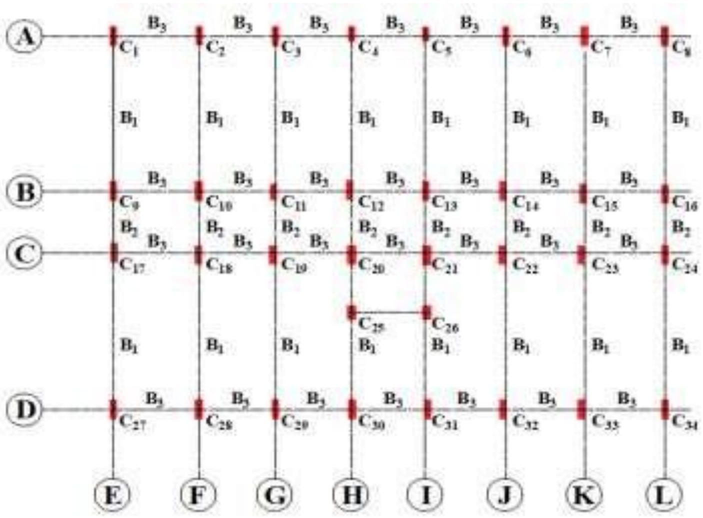

Figure 7. Layout of the Hostel Building.

RESULTS AND DISCUSSION

After assigning sectional properties, support conditions, static seismic loads along with combination of loading. In the present study, retrofitting technique is implemented to evaluate the performance of existing of Hostel building in Akola. The analysis is carried out using STAAD software. The existing hostel building is evaluated for seismic zone

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3. The existing building model is found to be safe for the provided reinforcement and member sizes. In the next step an extra floor is constructed to the existing building and checked for building safety. It is observed that many members sizes failed and a need to retrofit the failed member sizes is required. The study retrofits the failed column sizes with the help of model 3 using the jacketing technique by increasing the reinforcement requirement of the building. The study also added one more floor to the jacketed building model and observed that the revised member sizes and reinforcement is adequate for the additional building floor (G+3). The bending moment results for model 1 and model 2 is as shown in figure 8 and 9 respectively. It is observed that increasing an additional storey to the existing building increases the bending moment on the columns and make them vulnerable to failure. The analysis of results show that column fail due to an additional storey and a need to retrofit them using jacketing technique is essential. The failed column details are tabulated in table 5. The columns are important elements in any reinforced concrete framed structure. The integrity and strength of a structure depend on the robustness of a column. The deficient columns were identified during detailed evaluation of building. Members requiring strengthening or enhanced ductility can be jacketed by reinforced concrete jacketing, steel profile jacketing, steel encasement or wrapping with FRPs. Jacketing can be applied in cases of heavily damaged frame members or in cases of insufficient member strength, which is a case with the columns of the building considered. They need to be strengthened for both aspects. Reinforced concrete jacketing is adopted for the purpose of retrofitting the columns; the procedure given in the draft code will be adopted for the same. The design method for concrete jacketing in the draft code is based on provisions in Eurocode, UNIDO/UNDP documents and Indian Standards.

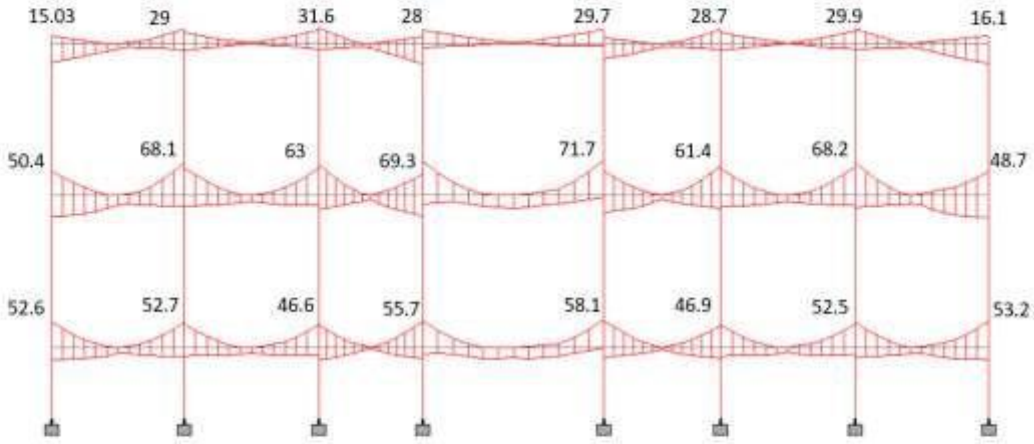

Figure 8. Bending Moment in kN-m for frame D (Model 1).

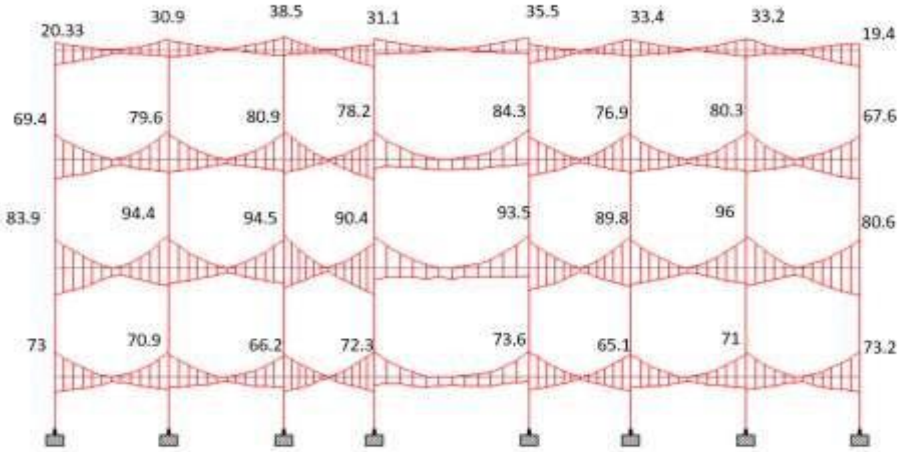

Figure 9. Bending Moment in kN – m for frame D (Model 2).

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Table 5 Failure Column Details

Column ID	Dimension mm	Area of Steel mm²	Main Reinforcement Provided	Tie Reinforcement Provided
154	250X800	1980.91	12 nos. 16 mm	8 mm @ 250 mm c/c
3500	250X800	3858.19	-	-
3501	250X800	2051.60	12 nos. 16 mm	8 mm @ 250 mm c/c
3508	250X800	1969.11	12 nos. 16 mm	8 mm @ 250 mm c/c

Member flexural strength increases with the enlargement of the concrete area and by adding new longitudinal reinforcement. Shear strength, and especially ductility, is improved by better confinement with close transverse reinforcement – ties or steel strips. The enlarged sections of repaired or strengthened members can result in considerable stiffness change of the different members, causing a redistribution of seismic moments and affecting the seismic forces in different parts of the building structure. As done in previous sections the demand in the two critical frames, one supporting the long cantilever and the other subjected to heavy concrete overlay, are obtained. The amount of excess concrete and steel needed is calculated and the provision for concrete jacketing will be provided. One sample calculation for the same is given; the details of jacketing for the other columns are obtained in the similar fashion. The moment carrying capacity for such a section with 1000 kN of axial load is just 38 kNm. The maximum moment carrying capacity of such a section is about 55 kNm which reduces the axial load carrying capacity to a nominal of 240 kN. The loads subjected to these columns is however much greater. Two combinations first of maximum axial load and the other of maximum moment are considered. The combination that accounts for greater section and reinforcement is given. The details of jacketing are as given in Figure 10.

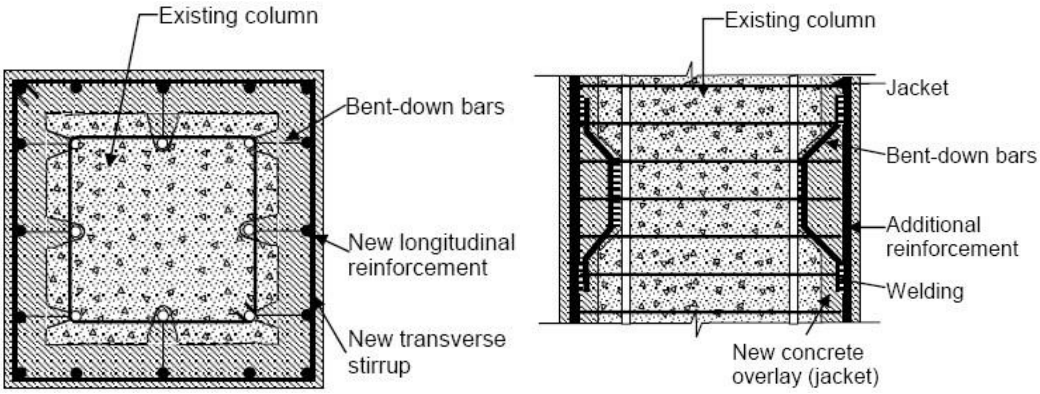

Figure 10. Typical details of a Reinforced Concrete Jacketing

Jacketing is one of the most frequently used technique used to strengthen RC columns with this method, axial strength, bending strength and stiffness of the original column increased. The column no 154 was chosen as the failure in the first storey. It showed a maximum bending moment value of 134kN-m. The revised column dimension are tabulated in table 6

Height of Column = 3150mm

Width of Column = 250mm

Depth of Column = 800mm

Area of Steel Provided = 1980.91mm²

F_y = 415 N/mm²

F_{ck} = 20 N/mm^2

Axial Force = 2398.73 kN.

Factored Axial Force = 3598.1 kN

$P_u = 0.4 \times f_{ck} \times A_c + 0.67 \times fy \times A_{sc}$

According to Clause 8.5.12 of IS 15988:2013

A_{sc} = 0.8% of A_c

i.e., $3598.1 \times 10^3 = 0.4 \times 20 \times A_c + 0.67 \times 415 \times 0.8\% A_c$

Therefore A_c = 351913.1 mm^2

$Ac^1 = 1.5A_c$ = 1.5 x 351913.1 = 527869.61 mm^2

Taking B = 400mm

D = 1320mm Approximately 1400mm.

As per IS 15988:2013, minimum Jacketing thickness to provide is 100mm. Therefore, Width of the column B = 400mm Depth of the Column D = 1400mm Hence provide a column of size 400mmX1400mm

Table 6. Revised Column Details

Failed Column	Original Dimension	Proposed Dimension
Column - 154	250X800	400X1400
Column - 3500	250X800	400X1400
Column - 3501	250X800	400X1400
Column - 3508	250X800	400X1400

The Demand Capacity Ratio (DCR) = Required Area of Steel / Provided Area of steel.

The column reinforcement details of the existing building (Model 1) is tabulated in table 7 and all the column of the model 1 is passed in the designed.

Table 7. Details of columns for frame on gridline D in Model I

Column	Condition	Forces			Load Case	Section Required in mm²	Steel Required in mm²	Steel Provided in mm²
		P_u	M_y	M_z				
C27	Pu max	2346.94	70.09	271.19	14	250X900	1600	1810
	M max	210	28.4	67.6	13			
C28	Pu max	2210.20	67.54	252.85	5	250x850	1210	1360
	M max	338	15.9	76	5			
C29	Pu max	2346.94	74.25	286.3	12	250X900	1600	1810
	M max	326	15.8	73.9	5			
C30	Pu max	2210.2	75.75	283.22	5	250x850	1210	1360
	M max	566	19.5	128	5			
C31	Pu max	2346.94	81.17	310.09	5	250X900	1600	1810

	M_{max}	437	15.1	87.5	5			
C32	Pu max	2210.2	66.82	250.49	5	250x850	1210	1360
	Mu max	325	17.2	74.9	5			
C33	Pu max	2346.94	77.61	298.76	5	250X900	1600	1810
	Mu max	345	17.5	76.2	5			
C34	Pu max	2210.2	65.39	245.67	12	250x850	1210	1360
	Mu max	315	5.42	69.9	5			

Constructing an additional floor to the existing building resulted in following column reinforcement details as tabulated in table 8 leading to column failure.

Table 8 Reinforcement detailing of column for frame on gridline D (Model 2).

Column	**Forces**			**Load Case**	**Section Required in mm²**	**Steel Required in mm²**	**Remark**
	P_u	M_y	M_z				
C25	2346.94	80.67	308.5	14	250X800	1600	Fail
C26	2529.25	94.10	376.91	12	250X800	1985.1	Pass
C27	2529.25	85.96	344.40	16	250X800	2129.7	Pass
C28	2529.25	95.85	383.90	14	250X800	2167.1	Pass
C29	2529.25	95.18	380.56	12	250X800	1966.4	Pass
C30	2529.25	86.49	346.34	18	250X800	2049.3	Pass

The failed columns were retrofitted using jacketing technique to protect the failed columns. The reinforcement details using the jacketing technique is tabulated in table 9

Table 9 Details for jacketing of columns for frame on gridline D in model 3.

Column	**Forces**			**Section Required in mm²**	**Steel Required in mm²**	**Details of Concrete Jacketing**				
	P_u	M_y	M_z			A_{sc} in mm²	*Thickness in mm*	A_{st} in mm²	*Main Steel Provided*	*Stirrups Provided*
C25	2346.94	80.67	308.5	400X1400	4480	0.1225	100	4480	16 nos. 20 mm	10 mm@ 80 mm c/c
C26	2529.25	94.10	376.91	250X800	1985.1	0.1225	100	1600	10 nos. 16 mm	10 mm @ 80 mm c/c
C27	2529.25	85.96	344.40	250X800	2129.7	0.1225	100	1600	10 nos. 16 mm	10 mm @ 80 mm c/c

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C28	2529.25	95.85	383.90	250X800	2167.1	0.1225	100	1600	10 nos. 16 mm	10 mm @ 80 mm c/c
C29	2529.25	95.18	380.56	250X800	1966.4	0.1225	100	1600	10 nos. 16 mm	10 mm @ 80 mm c/c
C30	2529.25	86.49	346.34	250X800	2049.3	0.1225	100	1600	10 nos. 16 mm	10 mm @ 80 mm c/c

It was intended to obtain the most suitable solution for the purpose of retrofitting the considered building. Three models were studied and the demand in each was obtained through a three-dimensional analysis. With reference to these we discussed the feasibility the structural model with some changes in its lateral load resisting system. The various critical members were studied for retrofitting the existing structure the results obtained from these can be applicable for the other members and frames too. Further constructing an additional floor (model 4) proved that all the columns passed and the proved column sizes and reinforcement is adequate to safeguard the structure from any failure. The reinforcement details of the model 4 are tabulated in table 10. It is seen that whichever may be the case the behavior of the Model III has lesser demand due to reduced due to retrofitting the structure with column jacketing. The conclusion on the study performed will be discussed in the next section

. **Table 10** Reinforcement detailing of column for frame on gridline D Model IV.

Column	Forces			Load Case	Section Required in mm²	Steel Required in mm²	Remark
	P_u	M_y	M_z				
C25	2346.94	85.05	327.29	14	250X800	1600	Pass
C26	2529.25	92.87	370.49	16	250X800	1991.58	Pass
C27	2529.25	91.05	361.92	16	250X800	2130.61	Pass
C28	2939.46	111.48	489.84	14	250X800	2458.33	Pass
C29	2529.25	93.20	372.33	16	250X800	1991.62	Pass
C30	2529.25	91.46	363.62	18	250X800	2065.39	Pass

IV. CONCLUSION

It was intended to evaluate an existing structure for the purpose of seismic retrofitting, a task considered of utmost important in seismic engineering today. The investigation, evaluation and the strengthening of the critical members of the structure and their procedure is studied. The purpose of this study was to suggest the most suitable model which will have the least demand to capacity ratio. The amount of retrofitting needed for the considered critical member for each of the model is calculated which shows the suitability of one of the structural models considered. In the present study retrofitting technique in the form of jacketing column is studied. Four different models namely actual building (model 1), proposed G+2 building (mode 2l), jacketing columns for the failed columns in model 2 (model 3) and proposed G+3 building (model 4) are evaluated Some major conclusions drawn from the study are as follows:

1) The evaluation reveals that the structure is not only deficient for seismic considerations but for gravity loads itself.
2) There was not much coordination between the executing and designing authority which led to blunders that could have been easily avoided.

3) If the existing structure is to be retrofitted for seismic forces it would result in a need for bulky sections and thus a greater financial input for obtaining the required performance.
4) The demand in beams in Model III is lesser than Model II.
5) In the critical frames considered for retrofitting of columns. To overcome the failure of column as shown in Model II.
6) As the Model III has the least demand to capacity ratio, the retrofitting needed will be smaller and will inturn ask for the least financial input for the purpose of retrofitting as compared to the other models.
7) Model III has an improved overall performance needed for the resistance for withstanding seismic forces.
8) Thus, the study suggests to incorporate Model III as the most feasible and suitable option for the seismic retrofitting of the existing Hostel building.

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