Retrofitting Process of an Existing Building With Respect To Seismic Consideration in Bangladesh

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Abstract— There might be many buildings in Bangladesh which do not meet the current seismic requirement and as a result may suffer much damage during the earthquake. Especially the older buildings which were constructed without the consideration of proper seismic forces should be evaluated for seismic load and retrofitted accordingly. If remedial measures are taken based on seismic evaluation, much damage can be overcome. The objective of the research here is to evaluate the existing building for earthquake performance. For applying earthquake loads, Equivalent Static Force Method is used according to BNBC 1993. Reinforcement details of the considered building were not available. For the purpose of study, in the first step an analysis is done applying only Dead and Live Loads according to BNBC 1993. The building is then designed for Dead Load and Live Load only without the consideration of seismic load. In the second step, the building is analyzed for seismic loading in addition to Dead Load and Live Load with proper load factor. Three dimensional analyses is done using design software STAAD-Pro. The Demand Capacity Ratio (DCR) is carried out for beams and columns in order to evaluate the member for seismic loads. Then retrofitting is carried out for the failed beams and columns. Steel Plating Retrofitting Method is applied for the beams and Concrete Jacketing Retrofitting Method is applied for the columns. It is recommended from this study that the buildings which were not built with seismic consideration can be evaluated and retrofitted following the research presented in this study.

Keywords: — Seismic Evaluation, Demand Capacity Ratio (DCR), Retrofitting, Bangladesh National Building Code, Equivalent Static Force Method, Base Shear

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1 INTRODUCTION

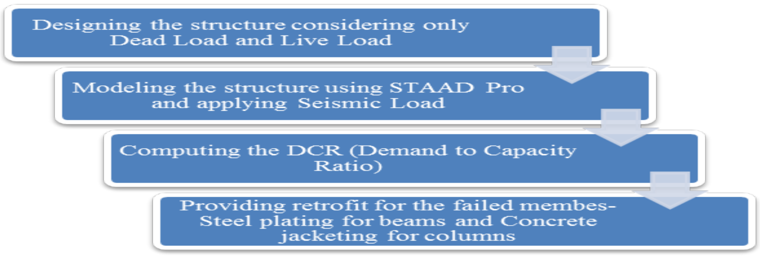
AN earthquake (also known as a quake, tremor or temblor) is the consequence of a sudden release of energy in the Earth's crust that generates seismic waves. The seismicity or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time (Calvi et al. (2002)).

Beam-column joint connections are a common structural weakness in dealing with seismic retrofitting. Prior to the introduction of modern seismic codes in early 1970s, beam-column joints were typically non-engineered or designed. Calvi et al. (2002) revealed that laboratory testing's have confirmed the seismic vulnerability of these poorly detailed and under-designed connections. Park, R. et al. (2002) found that failure of beam-column joint connections can typically lead to catastrophic collapse of a frame-building, as often observed in recent earthquakes. Durgesh C. Rai (2005) gave the guidelines for seismic evaluation and strengthening of buildingsThis document is particularly concerned with the seismic evaluation and strengthening of existing buildings and it is intended to be used as a guide. Devesh et al. (2006) agreed on the increase in drift demand in the tower portion of set-back structures and on the increase in seismic demand for buildings with discontinuous distributions in mass, strength and stiffness. Sadjadi et al. (2007) presented an analytical approach for seismic assessment of RC frames using nonlinear time history analysis and push-over analysis. Saptadip Sarkar (2010) studies the Design of Earthquake resistant multi stories RCC building on a sloping ground which involves the analysis of simple 2-D frames of varying floor heights and varying no of bays using a very popular software tool STAAD Pro.

The vulnerability of the structure can be assessed with a higher accuracy and better informed decisions can be made on the possible improvement of the seismic resistance of existing RC structures by Seismic Evaluation. For example, the critical components of the structure that are likely to sustain significant damages during future earthquake ground motions may be identified. Accordingly, the required immediate structural interventions may be designed to reduce the deformation demands on these components. Subsequently, the overall behavior of the structure may be improved to achieve a satisfactory overall seismic performance during a future earthquake.

2 METHODOLOGY

2.1 General: The methodology of this study can be shown by the following flow chart



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Seismic Evaluation is a major tool in earthquake engineering which is used to understand the response of buildings due to seismic excitations in a simpler manner. In the past the buildings were designed just for gravity loads and seismic analysis is a recent development. It is a part of structural analysis and a part of structural design where earthquake is prevalent.

2.2 Seismic Evaluation Methods

1. Preliminary Investigation
2. Detailed Evaluation

Preliminary Investigation:

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is an approximate method based on conservative parameters to identify the potential earthquake risk of a building and can be used for screening of buildings for detailed evaluation.

Detailed Evaluation:

There are different types of detailed earthquake analysis methods. Equivalent Static Analysis as per BNBC'93 is done in this research study.

I. Equivalent Static Analysis
II. Response Spectrum Analysis
III. Time History Analysis

Equivalent Static Analysis:

The Equivalent Static Analysis procedure is essentially an elastic design technique. It is, however, simple to apply than the multi-model response method, with the absolute simplifying assumptions being arguably more consistent with other assumptions absolute elsewhere in the design procedure.

The total design base shear in a given direction shall be determined from the following equation:

$$V = \frac{ZIC}{R} W \qquad (1)$$

Z= Seismic Zoning Coefficient

I= Structural Importance Coefficient

R= Response Modification Coefficient for structural systems

W=Total seismic dead load

C= Numerical Coefficient given by the equation:

$$C = \frac{1.25S}{T\,^\wedge(2/3)} \qquad (2)$$

S= Site Coefficient for Soil Characteristics

T= Fundamental period of vibration in seconds, of the structure for the direction under consideration

For all of the buildings the value of T may be approximated by the following formula:

$$T = Ct(\mathrm{hn})^{3/4} \qquad (3)$$

C_t= 0.073 for reinforced concrete moment resisting frames

h_n= Height in meters above the base to the level n

The total lateral force which is the base shear V, shall be distributed along the height of the structure in accordance with the following equation:

$$V = Ft + \sum_{i=1}^{n} Fi \qquad (4)$$

Ft= Lateral force applied at the storey level I and

Fi= Concentrated lateral force considered at the top of the building.

The concentrated force, Ft acting at the top of the building shall be determine by following equation:

$$Ft = 0.07TV \leq 0.25V \text{ when } T>0.7 \text{ second} \qquad (5)$$

$$Ft = 0.0 \text{ when } T\leq 0.7 \text{ second} \qquad (6)$$

The remaining portion of the base shear (V-Ft) shall be distributed over the height of the building including level n, according to the relation:

$$Fx = \frac{(V - Ft)Wxhx}{\sum_{i=1}^{n} Wihi} \qquad (7)$$

At each storey level-x, the force Fx shall be applied over the area of the building in proportion to the mass distribution at that level.

2.3 Seismic Retrofitting

2.3.1 Steel Plating

In the present study, a series of experiments were conducted attempting to retrofit deep reinforced concrete coupling beams using a bolted steel plate. In addition to the control specimen, the other specimens were bolted with a steel plate on the side face to improve the shear strength and inelastic behavior. A mechanical device was added to two specimens to restrain plate buckling. Moreover, the plate buckling-restrained specimen with a sufficient number of bolts in the anchor regions had a more stable response and better inelastic performance under reversed cyclic loads.

2.3.2 Concrete Jacketing

- ❖ Properties of Jackets:
  - Match with the concrete of the existing structure.
  - Compressive strength greater than that of the existing structures by 5 N/mm² or at least equal to that of the existing structure.
- ❖ Minimum Width of Jacket:
  - 10 cm for concrete cast-in-place and 4 cm for shortcrete.
  - If possible, four-sided jacket should be used.
  - A monolithic behavior of the composite column should be assured.
  - Narrow gap should be provided to prevent any possible increase in flexural capacity.

- Minimum Area of Longitudinal Reinforcement:
    - 3Afy, where, A is the area of contact in cm^2 and fy is in kg/cm^2.
    - Spacing should not exceed six times of the width of the new elements (the jacket in the case) up to the limit of 60 cm.
    - Percentage of steel in the jacket with respect to the jacket area should be limited between 0.015and 0.04.
    - At least, 12 mm bar should be used at every corner for a four sided jacket.
- Minimum Area of Transverse Reinforcement:
    - Minimum bar diameter used for ties is not less than 10 mm or 1/3 of the diameter of the biggest longitudinal bar.
    - The ties should have 135-degree hooks with 10 mm bar dia anchorage.
    - Due to the difficulty of manufacturing 135-degree hooks on the field, ties made up of multiple pieces, can be used.
- Connectors:
    - Connectors should be anchored in both the concrete such that it may develop at least 80% of their yielding stress.
    - Distributed uniformly around the interface, avoiding concentration in specific locations.
    - It is better to use reinforced bars (rebar) anchored with epoxy resins of grouts.

3 3D STRUCTURAL SOFTWARE ANALYSIS & RETROFITTING

3.1 Analysis

A 9 story residential building is considered in this research study. The building has two units. For simplification of work one unit is taken here. In Figure 1, 3D model of the building at STAAD Pro and in Figure 2 the layout of plan with Grid Line is shown. Beam size is same at all story. But there is difference in column sizes. In total six types of column sizes are used in the building. The Column Dimensions are shown in the following Table 1

Table 1: Column Dimensions

Location	Level 01 to 05	Level 06 to 09
Interior	585mmX585mm	381mmX381mm
Exterior	508mmX508mm	331mmX331mm
Corner	432mmX432mm	280mmX280mm

Dimension of beam: 305mmX559mm and 305mmX458mm

Building Parameters:

- Building type: Reinforced concrete frame.
- Grade of concrete, f_c= 21MPa
- Type of steel used- Mild Steel implies, f_y=345MPa
- Live load= 30 psf at roof (accessible) and 40psf at all other floors (BNBC'93).
- Brick load=0.5 k/ft

Load Combinations:

- DL
- LL
- 1.4 * DL+ 1.7 * LL
- 0.75(1.4DL+1.7LL+1.87EQ)
- 1.4(DL+LL+EQ)

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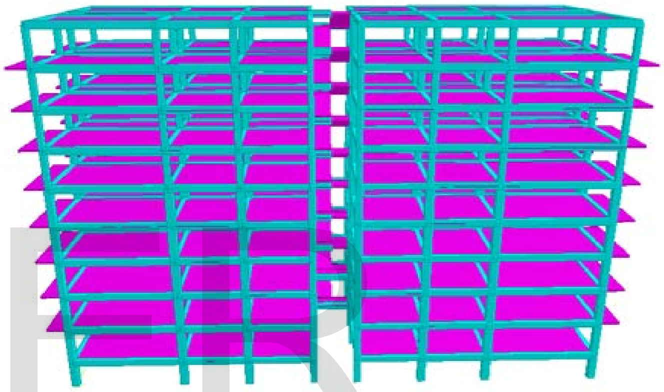

Figure 1: 3D Model of the building at STAAD Pro

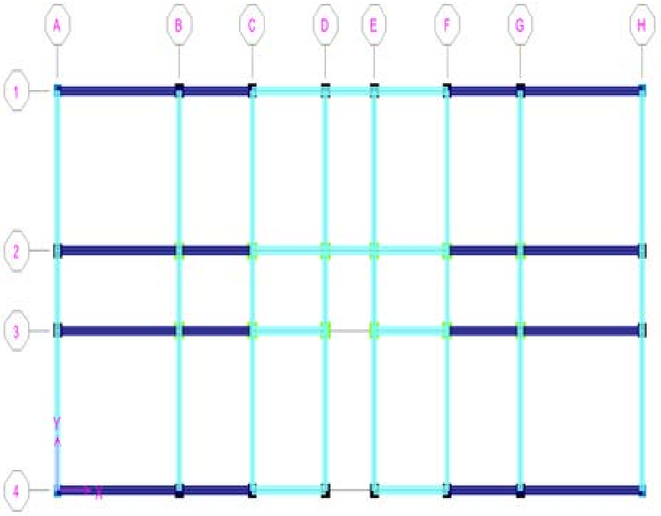

Figure 2: Layout of plan with grids

At Table 2 total seismic loads at different floors for each grid has been calculated as per BNBC'93. These loads were applied to each node of the grid at different floors of the building at STAAD Pro.

Table 2: Total Seismic Load Calculation at Different Floors

Floor	Hx(ft)	Fx (kip)				
		Grid A	Grid B	Grid C	Grid D	Total
8th	87	21.663	30.537	24.708	18.27	190.356
7th	77	19.173	27.027	21.868	16.17	168.476
6th	67	16.683	23.517	19.028	14.07	146.596
5th	57	14.193	20.007	16.188	11.97	124.716
4th	47	12.878	17.719	14.57	8.695	107.724
3rd	37	10.138	13.949	11.47	6.845	84.804
2nd	27	7.398	10.179	8.37	4.995	61.884
1st	17	4.658	6.409	5.27	3.145	38.964
GF	7	1.918	2.639	2.17	1.295	16.044

Sample Calculation of Level 01 Beam Check with Seismic Load:
Beam ID: A12 Beam No: 84 (According to STAAD Pro)
Maximum –ve moment: -1297.58 k-in or -108.13 k-ft (Capacity)
Maximum +ve moment: 1297.58 k-in or 108.13 k-ft (Capacity)
Maximum –ve moment: -143.99 k-ft (Demand) (From STAAD Pro.)
Maximum +ve moment: 58.71 k-ft (Demand) (From STAAD Pro.)
For +ve moment DCR= 58.71/108.13 = 0.543(DCR<1) [Pass]
For -ve moment DCR= 143.99/108.13 = 1.33(DCR>1) [Fail]
All beam of Level 01 with seismic loads are shown in the following Table 3

Table 3: Level 01 Beam Check with Seismic Loads

Beam ID	Beam No	DCR		Result	
		Max -Ve	Max +Ve	Max -Ve	Max +Ve
1AB	81	0.984	0.667	pass	pass
1BC	82	0.328	0.147	pass	pass
1CD	83	0.735	0.309	pass	pass
A12	84	1.33	0.543	**fail**	pass
B12	85	1.073	0.634	**fail**	pass
C12	86	1.073	0.625	**fail**	pass
D12	87	1.327	0.541	**fail**	pass
2AB	88	0.848	0.687	pass	pass
2BC	89	0.282	0.177	pass	pass
2CD	90	0.594	0.309	pass	pass
A23	91	1.315	1.046	**fail**	**fail**
B23	92	1.063	1.273	**fail**	**fail**
C23	93	1.063	1.309	**fail**	**fail**
D23	94	1.315	1.142	**fail**	**fail**
3AB	95	0.848	0.687	pass	pass
3BC	96	0.848	0.177	pass	pass
3CD	97	0.544	0.315	pass	pass
A34	98	1.319	0.520	**fail**	pass
B34	99	1.066	0.607	**fail**	pass
C34	100	1.066	0.614	**fail**	pass
D34	101	1.359	0.558	**fail**	pass
4AB	102	0.987	0.667	pass	pass
4BC	103	0.332	0.143	pass	pass
4CD	104	0.548	0.309	pass	pass

Sample Calculation of Level 01 Interior Column Check with Seismic Load:
Column ID: B3 Column No: 74 (According to STAAD Pro)
Nominal Axial load capacity, Pn = As*fy + 0.85 fc' * (Ag – As)
=14.72622*50+0.85*3*(529-14.72622)= 2047.71kip
Ultimate Axial Strength, P_{ult} = 0.8*0.7*Pn= 0.8*0.7*2047.71= 1146.72 kip
Maximum Load: 1146.72 kip (Capacity)
Maximum Load: 1260.51 kip (Demand) (From STAAD Pro.)
So, DCR= Demand / Capacity
= 1260.51/1146.72
= 1.09923(DCR>1) [Fail]
Column checks for all levels are shown in the following Table 4, Table 5 and Table 6. Check is done for one exterior, one interior and one corner column for each level

Table 4: Exterior Column A2 Check

Level	Demand(k)	Capacity(k)	DCR	Result
9	78.46	335.25	0.23403	pass
8	166.597	335.25	0.49693	pass
7	254.348	335.25	0.75868	pass
6	341.79	335.25	1.01951	**fail**
5	432.084	821.64	0.52588	pass
4	522.532	821.64	0.63596	pass
3	612.921	821.64	0.74597	pass
2	703.343	821.64	0.85602	pass
1	793.79	821.64	0.96611	pass

Table 5: Interior Column B3 Check

Level	Demand(k)	Capacity(k)	DCR	Result
9	122.31	532.6	0.22964	pass
8	243.87	532.6	0.45789	pass
7	366.927	532.6	0.68894	pass
6	491.46	532.6	0.92276	pass
5	635.046	1146.72	0.55379	pass
4	784.9	1146.72	0.68447	pass
3	939.469	1146.72	0.81927	pass
2	1098.346	1146.72	0.95782	pass
1	1260.51	1146.72	1.09923	**fail**

Table 6: Corner Column A1 Check

Level	Demand(k)	Capacity(k)	DCR	Result
9	55.38	236.7	0.23398	pass
8	120.081	236.7	0.50731	pass
7	184.326	236.7	0.77873	pass
6	248.181	236.7	1.0485	**fail**
5	314.978	579.65	0.54339	pass
4	382.237	579.65	0.65943	pass
3	449.209	579.65	0.77497	pass
2	516.065	579.65	0.8903	pass
1	582.65	579.65	1.00518	**fail**

3.2 Retrofitting Process of Deficient Members

Retrofitting of Beam by Steel Plating Method:
Beam ID: D34 (Level 01) Size: 305mm × 458mm
Original Capacity = 108.5 k-ft and Target Capacity = 147.45 k-ft. Steel plate of thickness 1.5 mm is added to both tension and compression face. Effective depth of beam, d =15.5 inch. Stress in steel plate in compression and tension, $f_{pc} = f_{pt} = 50$ ksi. Providing width of steel plate, b = 8 inch. Strength added by steel plating = compression side + tension side. Compression side = $f_{pc} \times A_{pc}\ (\frac{[illegible]}{[illegible]} + d)$ and Tension side= $f_{pt} \times A_{pt}\ (\frac{[illegible]}{[illegible]} + d_c)$. So, Strength added by steel plating = 72.24 k-ft. So, Capacity after steel plating = Original capacity + 72.24 k-ft = (108.5+72.24) k-ft = 180.74k-ft>Target capacity (147.45k-ft). So, it's OK
Retrofitting of Column by Concrete Jacketing Method:
Exterior Column: Column ID: A2 Level:06
Size of Column: 331mmX331mm
Extra gross area for jacketing, $A_g = 272$ inch^2. Capacity increased by concrete jacketing, $P_u = 496.83$ kip

Total capacity increased by concrete jacketing = Original capacity + 496.83 kip = (335.25 + 496.83) kip =832.08 kip > Demand (341.79 kip). So, it's OK.
Required reinforcement for concrete jacketing,
$A_{st}(\text{required}) = \frac{[illegible]}{[illegible]}\ 272 = 4.08$ inch^2 Use 6Φ25mm, $A_{st}(\text{provided}) = 4.56$ inch$^2 > 4.08$ inch^2 OK.
The detailed calculations of Concrete Jacketing as well as Capacity Check are shown in the following Table 7

Table 7: Concrete Jacketing

Position of Column	Level	Capacity Increased (k)	Original Capacity(k)	Total Capacity Increased (k)	Demand (k)	Check
Exterior	6	496.83	335.25	832.08	341.79	OK
Interior	1	789.083	1146.7	1935.8	1260.5	OK
Corner	1	613.731	579.65	1193.38	582.65	OK
Corner	6	438.379	236.7	675.079	248.18	OK

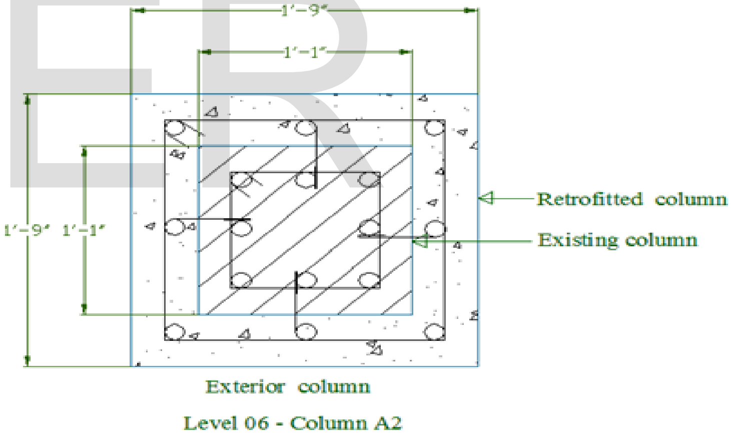

Figure 3: Concrete Jacketing of Exterior Column A2.

4 CONCLUSIONS AND RECOMMENDATIONS

Based on the seismic evaluation carried out in this study, the following important conclusions can be made-

- All of the beams and columns in one unit were checked for vulnerability due to seismic loads. In total, there are 216 beams in the building in one unit. Among them 64 beams are failed after applying earthquake force. It means 29.63% beams are failed.
- On the other hand there are 144 columns in the building in one unit. Among them 21 columns are failed af-

ter applying earthquake force. It means 14.58% columns are failed.

- Maximum DCR for beams is found to be 1.373 at Level 02 which is 37.3% greater than the capacity. Similarly maximum DCR for column is found 1.09923 at Level 01 which is 9.923% greater than the capacity.
- In case of retrofitting of beam by Steel Plating it is found that the capacity achieved by retrofitting method is 180.74 k-ft which is more than the target capacity of 147.45 k-ft. The capacity increase is 22.58%.
- On the other hand, in case of retrofitting of interior column by Concrete Jacketing, the capacity achieved by retrofitting method is 1935.80 kip which is more than the demand 1260.51 kip. The capacity increase is 53.57%.

Finally based on this research study, it is recommended that the buildings which were not built with seismic consideration can be evaluated and retrofitted following the research procedure presented in this study.

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