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SEISMIC EVALUATION OF EXISTING BUILDINGS IN KARNATAKA

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ABSTRACT

Seismic strength evaluation of existing building is assuming increasing importance. Recent earthquakes all over the world, have demonstrated the disastrous consequences and vulnerability of inadequate structure.

In the present study evaluation of existing buildings in Karnataka were investigated. Most of the buildings existing in Karnataka were designed only for dead load and live loads (gravity loads), loads due to earthquake were not considered in the design. The project includes the evaluation of building by analyzing an existing building in Karnataka, considering the seismic loads along with gravity loads. The buildings were analyzed using SAP 2000

NOMENCLATURE

A_{st} = Area of Main Reinforcement
V_{us} = Strength of Shear Reinforcement
A_{sv} = Total Cross Sectional Area of Stirrup Legs
S_v = Spacing of Stirrups
P_u = Factored Axial Load on Member
B = Width
D = Effective Depth

1. INTRODUCTION

The word earthquake is used to describe any seismic event whether natural or caused by humans that generate seismic waves. Earthquakes are measured using observations from seismometers. The buildings which do not fulfil the requirements of seismic design, may suffer extensive damage or collapse if shaken by a severe ground motion. The seismic evaluation reflects the seismic capacity of earthquake vulnerable buildings for the future use.

According to the Seismic Zoning Map of IS: 1893-2002, India is divided into four zones on the basis of seismic activities. They are Zone II, Zone III, Zone IV and Zone V. Karnataka lies in Zone II.

A large number of existing buildings need seismic evaluation due to various reasons. However, the existing structure in the earthquake region has to be provided by some rehabilitation to sustain the expected performance level. Before rehabilitation work, it is necessary to understand the capacity of the existing building to check if it meets the intended performance level.

2. SEISMIC EVALUATION

Seismic evaluation methods are preliminary investigation and detailed evaluation.

2. 1 Preliminary Investigation

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability.

2.2 Detailed Evaluation

Detailed evaluation includes linear and nonlinear analysis. Linear and nonlinear analysis includes static and dynamic analysis. Linear static analysis is equivalent static analysis and where as Linear dynamic analysis includes response spectrum and time history methods. Nonlinear static analysis is pushover method and corresponding dynamic analysis is inelastic time history analysis.

3. CAPACITY DETERMINATION FROM DCR

In detailed evaluation full building analysis is performed. The detailed evaluation procedure is based on the analysis and design philosophy of IS 1893 (Part1):2002[15]. This involves equivalent static lateral force procedure, load with response reduction factors and Demand Capacity Ratio (DCR) for ductility as in IS 13920[16].

The values(moment and shear for columns and beams) obtained by adopting the calculations mentioned below is the capacity for the corresponding member and the demand is obtained from the SAP after modeling and analysis.

Demand Capacity Ratio= Max. Demand / Capacity. If the value of DCR<1 then the members is safe i.e. it can take the moment induced by seismic loading, else it will fail.

3.1 Moment of Resistance in Hogging

From the reinforcement details of the existing building calculate the A_{sc}. The equation to find the moment of resistance is given in Eqn. (1).

$$M_U = .36 f_{ck}\, bx(d - 0.416x) + (f_{sc} - .44 f_{ck})A_{sc}(d - d') \quad (1)$$

The moment obtained is the capacity of members from the available reinforcement details.

Moment of Resistance in Sagging

The contribution of the compression steel is ignored while calculating the sagging moment capacity as T-beam. Referring to Annex G of IS: 456-2000, sagging moment capacity for $x_u < D_f$ and $x_u < x_{umax}$, may be calculated as given below.

$$M_u = 0.87 f_y A_{st}\, d \left[1 - \frac{Ast\ fy}{bd\ f_{ck}}\right] \quad (2)$$

Vertical Beam Shear on Face of Face of Joint

Shear reinforcement provided in the existing beam at support section noted. Calculate P_t ,using $P_t = \frac{100\, A_{st}}{}$,

from the beam reinforcement. Using IS 456:2000 find out . Using clause 40.4 of IS 456:2000

$$V_{us} \quad \frac{0.87\, f_y A_{sv}\, d}{s_v} \qquad (3)$$

$$V_{u1} = V_{us} + \tau_c\, bd \qquad (4)$$

Eqn.(4) will give the maximum shear force i.e, the demand.

Flexural Capacity of Column

From the available reinforcement details of the building, the main reinforcement provided in the existing column at support section noted. From the main reinforcement, is calculated using the equation . Calculated and Referring to SP:16 find the value of , By cross multiplying b and , we will get i.e. The moment of resistance of the section.

Shear Capacity of Column

Considering the steel in one face will be in tension and calculate . From Table 19 of IS 456:2000 we get the value of . From Clause 40.4 of IS 456:2000 we calculate andas done in case of beams.

Calculated V_{u2} i.e. shear from moment capacity as per Is 13920. Maximum from V_{u1} & V_{u2} is taken i.e. shear resisted. If shear resisted is more than maximum shear from Sap the member is pass and Vice Versa.

Joint Shear

The joint shear equilibrium is shown in Figure 1.

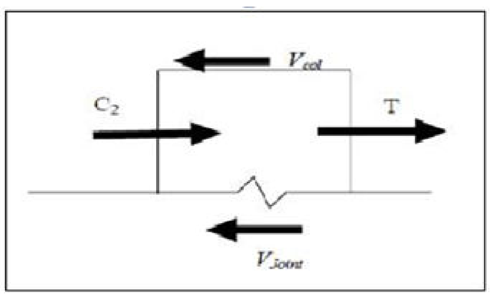

FIGURE 1.JOINT SHEAR [17]

Column Shear

The column shear is as explained below.

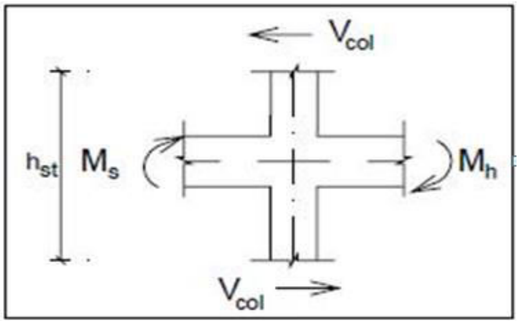

FIGURE 2. COLUMN WITHS WAY TO RIGHT [17]

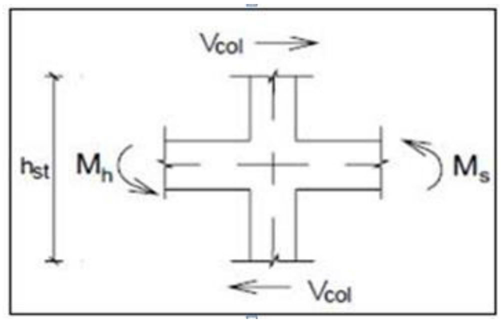

FIGURE 3. COLUMN WITH SWAY TO LEFT [17]

Force Developed in Beam Reinforcement

Force developed in the top bars

$T_1 = A_{st}$ x 1.25 x f_y (6)

Force developed in the bottom bars

$T_2 = A_{st}$ x 1.25 x $f_y = C_2$ (7)

Referring to Figure1.

$V_{joint} = T_1 + C_2 - V_{col}$ (8)

Maximum value of T_1 and minimum value of V_{col} are used in the above equation.

Check for Joint Shear Strength

The calculation of the effective width of the joint and the design shear strength of the joint is based on the draft revision of IS 13920:1993[17].

The effective width of the joint is lesser of the

i) $b_j = b_b + 0.5 \times h$ (9)

ii) $b_j = b_c$

Effective shear area of the joint = $A_c = b_j h$ (10)

The nominal shear strength of the joint shall not be taken greater than 1.5 $\sqrt{f_{ck}}\, A_c$ for joints confined on all four faces, 1.2 $\sqrt{f_{ck}}\, A_c$ for joints confined on three faces or on two opposite faces, and 1.0 $\sqrt{f_{ck}}\, A_c$ for others.

Check for Flexural Strength Ratio

At a joint in a frame resisting earthquake forces, the sum of the moment of resistance of the columns all be at least 1.1 times the sum of the moment of resista nce of the beams along each principal plane of the joint (Figure4).

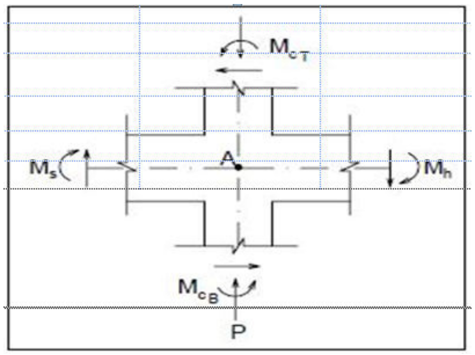

FIGURE 4. FLEXURAL STRENGTH OF JOINT[17]

3. MODELLING AND ANALYSIS OF BUILDING

A Ground with three storied (G+3) existing reinforced concrete building located in zone II of India was taken for the investigation. This existing building is designed and constructed using IS 456-1978 only for the gravity loads i.e., without considering the previous seismic code IS1893-1984.First the equivalent static analysis of building was done under gravity loads and seismic loads. Then compare the critical forces and moments of beams and columns at face of joint. The modeling and analysis was carrying out first in SAP 2000.Modeling consisted of following steps.

3.1 Structural Discretization

The beams and columns layout were available. Then the structure was discretized. Discretization includes fixing of joint coordinates and member incidences.

3.2 Property Specification and Support Condition

Properties were assigned to the members. The member properties were given. Then support conditions were given to the structure. The support condition given was fixed.

3.3 Loading

The self-weight of the members will be taken automatically by the software. Live loads of slabs were entered as floor loads. Live loads were considered as per IS: 875 (Part 2)-1987. The wall loads were provided to the beam based on provisions in IS: 875 (Part 1)-1987. Seismic loads were applied automatically by the software and is based on IS 1893 (Part I)-2002.

a) Live load- As per IS: 875(Part 2)-1987[11]
Rooms and Kitchen - 2 kN/m²
Toilet &Bathrooms - 2 kN/m²
Stair case - 3 kN/m²
b) Dead load- As per IS: 875(Part 1)-1987[10]
Dead load includes self-weight of columns, beams, slabs, brick walls, floor finish etc.
Total load on slab = 5.25 kN/m^2
Dead load on brick infill wall=10.67 kN/m
c) Wind load- As per IS: 875(Part 3)-1987[12]
Wind loads are determined using the following
Parameters
Basic wind speed = 33 m/s
Risk factor (50 years design life) $K_1 = 1.0$

Topography factor $K_3 = 1.0$
Terrain category & Building Class = 2 &B
Value of K_2 varies as per the building height, which is given in Table 1.

Height (m)	K_2 Value
5.2	0.98
8.2	0.98
11.2	0.988
14.2	1.029

3.4 Design Wind Pressure

Design wind speed, $V_z = V_b k_1 k_2 k_3$ (11)
Design wind Pressure, $P_z = 0.6\ V_z^2$ (12)

Length of the building in x direction, l_x = 1 1.71m
Length of the building in y direction, l_y = 7.48 m

TABLE 2. DESIGN WIND PRESSURE P_Z

Height (m)	V_b(m/s)	k_1	k_2	k_3	V_z (m/s)	Wind pressure P_z ,(N/m²)
5.2	33	1	0.98	1	32.34	627.52
8.2	33	1	0.98	1	32.34	627.52
11.2	33	1	0.988	1	32.59	637.46
14.2	33	1	1.029	1	33.01	675.09

Wind load in x direction, $W_x = P_Z$ x l_x x height = 113KN
Wind load in y direction, $W_y = P_Z$x l_y x height =167.56KN

d) Seismic load

The seismic load values were calculated as per IS 1893-2002[15]. Total seismic load, W =8742.4 KN

Design base shear, $V_B = A_h$ x W (13)
A_h = design horizontal acceleration spectrum value

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g} \quad (14)$$

Natural period of vibration, $T_a = 0.075h^{0.75} = 0.548S$

Design base shear, $V_B = A_h$ x W =367 kN (Z-II)

The value of design base shear is greater than the values of wind load in either direction. Hence, seismic force is considered as the critical.

3.5 Load Combinations

The different combinations used were:

1.5 DL + 1.5 LL
1.2 DL + 1.2 LL + 1.2 EQX
1.2 DL + 1.2 LL + 1.2 EQ(-X)

4. RESULTS

TABLE 3.THE DEMAND CAPACITY RATIOS FROM ANALYSIS RESULTS AND CAPACITY

1.2 DL + 1.2 LL + 1.2 EQZ 1.2 DL + 1.2 LL + 1.2 EQ(-Z) 1.5 DL + 1.5 EQX
1.5 DL + 1.5 EQ(-X)

1.5 DL + 1.5 EQZ
1.5 DL + 1.5 EQ(-Z)
0.9 DL + 1.5 EQX

0.9 DL + 1.5 EQ(-X)
0.9 DL + 1.5 EQZ

0.9 DL + 1.5 EQ(-Z)

building 1													Flex.Strg	
Jt														
no	Beam SF		Column Moment				Column SF		ColumnMu		Joint SF		Ratio	
			Mh		Ms				Mux	Muz	floors		floors	
	x	z	x	z	x	z	X	z	x	z	x	z	x	z
1	0.6	0.68	1.4	2.35	0.78	2.5	0.5	0.3	1.28	0.43	1.16	0.47	0.7	4
2	0.59	0.84	1.28	2.6	0.94	2.5	0.6	0.4	1.68	0.66	0.85	0.47	1.2	5.3
3	0.59	0.73	1.3	2.08	1.04	2.6	0.5	0.4	1.36	0.4	1.16	0.59	0.7	4.4
4	0.73	0.71	1.5	2.2	0.79	1.9	0.43	0.41	1.29	0.44	0.85	0.58	1.9	3
5	0.75	0.83	1.4	2.2	0.82	2.6	0.48	0.35	1.4	0.65	1.16	0.59	0.79	4.8
6	0.72	0.72	1.3	2	1.29	2.7	0.57	0.32	0.9	0.79	1.16	0.23	1.04	4.8
7	0.68	0.7	1.39	2.45	1.36	1.55	0.39	0.33	1.02	0.34	0.85	0.47	1.8	5
8	0.62	0.68	1.14	2.29	0.58	1.9	0.44	0.32	0.98	0.48	1.09	1.01	1.4	5.2
9	0.71	0.77	1.58	11.7	1.01	3.1	0.76	0.33	1.24	1.3	1.03	0.59	0.36	4
10	0.67	0.68	1.47	2.19	1.18	1.45	0.75	0.38	1.5	1.13	0.76	0.47	0.56	3.2
11	0.47	1.03	1.2	2.1	0.34	2.9	0.87	0.34	1.3	1.05	1.5	0.41	0.18	4.1
12	0.7	0.8	1.6	1.8	1.27	2.2	0.56	0.54	1.8	0.58	0.89	0.68	1.03	2.3
13	0.65	0.78	1.3	1.09	1.13	0.9	0.68	0.55	3.2	1.2	0.89	0.74	0.65	1.4
14	0.78	0.72	1.7	3.5	1.7	1.5	0.6	0.39	1.2	0.59	0.61	0.23	2	3.5
15	0.66	1.11	1.6	1.9	1.9	2.3	0.54	0.48	1.6	0.49	0.76	0.53	1.1	2.7
16	0.69	0.72	3.5	3.4	1.8	1.5	0.46	0.37	1.1	0.36	0.51	0.23	2.2	3.1
17	0.67	0.66	3.5	2.16	1.8	1.99	0.44	0.31	0.97	0.35	0.51	0.41	2.2	3.1
18	0.6	0.68	1.42	2.35	0.78	2.53	0.54	0.39	1.2	0.43	1.16	0.47	0.79	5
19	0.59	0.84	1.28	2.66	0.94	2.56	0.64	0.44	1.6	0.66	0.85	0.47	1.2	0.6

20	0.59	0.73	1.33	2.08	1.04	2.6	0.51	0.41	1.3	0.41	1.16	0.59	0.7	4.4
21	0.7	0.71	1.5	2.2	0.79	1.9	0.43	0.41	1.2	0.44	0.85	0.58	1.8	3
22	0.75	0.83	1.4	2.2	0.82	2.6	0.48	0.35	1.4	0.65	1.16	0.59	0.79	4.1
23	0.72	0.72	1.3	2	1.2	2.72	0.57	0.32	0.96	0.79	1.16	0.23	1.04	3.9
24	0.69	0.69	1.3	2.4	1.3	1.5	0.39	0.33	1.02	0.34	0.85	0.47	1.8	5.4
25	0.62	0.68	1.1	2.2	0.58	1.9	0.44	0.32	0.98	0.43	1.09	0.47	1.4	4
26	0.71	0.77	1.5	1.7	1.01	3.1	0.76	0.33	1.24	1.39	1.03	0.59	0.36	4
27	0.67	0.68	1.4	2.1	1.1	1.4	0.75	0.38	1.55	1.13	0.76	0.47	0.56	4.2

From the Table 3. it's clear that the DCR values for beam shear and column shear in x and z directions are less than than one, so they are safe in shear, so the reinforcement is is adequate.But the beam and column fails due to their moments exceeding the capacity, so the main reinforcement provided is not adequate for the seismic loading.

5. CONCLUSION

The results obtained from detailed analysis shows the deficiency of building towards the earthquake loads. The members may fail in case of seismic activities in future. Demand Capacity Ratio (DCR) is the main key to evaluate a member. If the demand is more than capacity of the member it will obviously fail. DCR values are calculated for Flexural and Shear capacities of beams and columns at face of joint.

After evaluation it was found that most of the framing beams and columns of the joint are safe in shear which shows enough shear reinforcement is present. The Flexural capacity of beams is checked for sagging and hogging moments. The result says almost all beams fail in sagging and hogging. In case of Columns most of columns fails in flexure.

As per the results obtained, my evaluation suggests that the frame needs to be strengthened and retrofitted.

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