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Capacity Based Design of Ten Storey Building Frame And Comparison With Conventional Design

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Abstract**:** *Capacity Based Earthquake Resistant Design is based on the concept that spreading of inelastic deformation demand in a structure takes place in such a manner so that the plastic hinges formed at predefined positions and sequences. The main objective of this study is to determine the advantage of capacity based design as compare to conventional design. In this work the building of ten storey have been analyzed and designed by capacity based design method. Building have been analyzed in STAAD Pro. V8i software and design of column moments, beam shear, column shear has been done. The building is designed by capacity based design method for earthquake zone IV. It has been proved that the value of column moment, column shear, beam shear obtained from capacity based design method are more than the value obtained from conventional method.*
Keywords: *Capacity based design method, Strong column-weak beam, Plastic hinge, Moment magnification factor, Plastic hinge.*

I. Introduction

Capacity design is a method of designing flexural capacities of critical member of a building based on behavior of the building in responding to seismic forces. This behavior is reflected by the assumptions that the seismic action is of a static equivalent nature increasing gradually until the structure reaches its state of near collapse and critical regions occur simultaneously at predetermined locations to form a collapse mechanism simulating ductile behavior. Ductility and energy dissipation of structure under an event of earthquake depends upon the vertical member (column) of the structure.

A short definition of capacity based according to Paulay (1992) is as follows: -

In structures designed for ductile seismic response the location of potential plastic hinge regions is deliberately chosen to enable the development of a suitable plastic mechanism.

Basic concept of capacity-based design is that, in the yielding condition, the strength of weaker member is related to the capacity of stronger member.

In multi storey buildings this can be achieved by formation of plastic hinges at the end regions of nearly all the beams in all stories of the building while vertical members (columns and walls) remain essentially elastic in all stories, with the exception of the base of the bottom storey

Capacity based design working on generally two concepts:

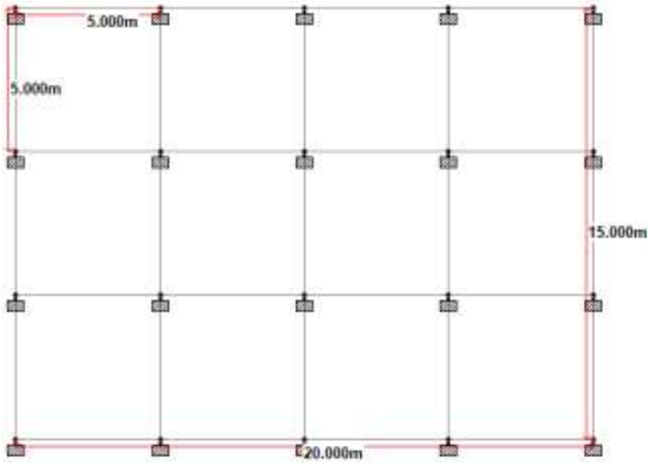
- Ductile chain concept
- Strong column weak beam concept

II. Analytical Modelling

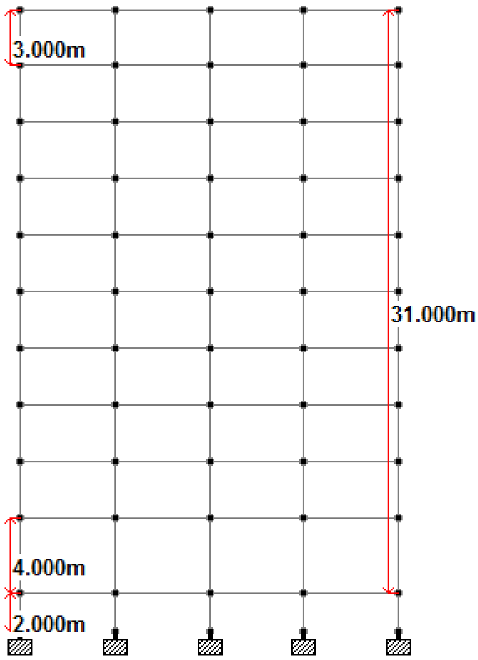
In this work systematic analysis is done for a ten storey building frame. Plinth beams are provided for the ten storey building frame which helps to control seismic demand in RC frame buildings. Analysis of ten storey building frame is carried out by using structural software (STAAD Pro. V8i). In this study, building frame is assumed in zone IV (IS 18932016) to obtain the maximum value of seismic forces. The building frame is then designed by using capacity-based design method for the forces obtained from STAAD Pro. V8i.

A. Geometry of the Building

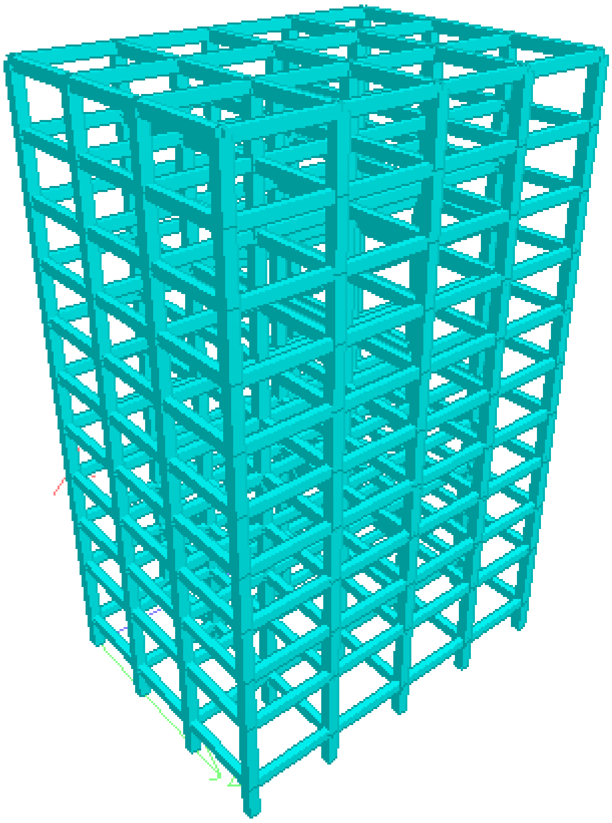
Plan of the building- 15m x 20m
Height of the building- 31m
Distance between two columns- 5m



"Fig. 1" Plan of the Ten Storey Building



"Fig. 2" Elevation of the Ten Storey Building Frame



"Fig. 3" Isometric 3-D view of Ten Storey Building

B. *Building Properties*

Table 1 Site Properties

Details of building	G+9
Wall Thickness 1. Outer wall thickness 2. Inner wall thickness 3. Parapet wall thickness	 230mm 115mm 115mm
Depth of foundation	2m
Floor Height 1. Ground floor 2. All floors other than ground floor	 4m 3m
Total height of building	31m

Table 2 Seismic Properties

Seismic zone	IV
Zone factor	0.24
Importance factor	1.2
Response reduction factor	3
Soil condition	Medium
Damping	5%

C. *Material Properties*

Material grades of M30 and Fe 415 are used for the design

D. *Size of Members*

Table 3 Size of Members

Beam	400mm X 600mm
Column	550mm X 550mm
Slab	150mm

E. Loading on Structure

Table 4 Loading on Structure

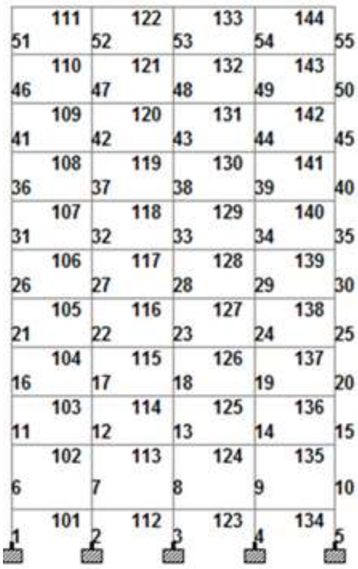
Dead Load	
1. Self-Weight	
2. Wall Load on Ground Floor	
i) Outer wall load	
ii) Inner wall load	15.64KN/m
3. Wall load on Floors other than Ground floor	7.82KN/m
i) Outer wall load	
ii) Inner wall load	
4. Parapet wall load on roof	
5. Floor load	11.04KN/m
	5.52KN/m
	2.3KN/m
	3.75KN/m2
Live Load	
1. Live load on floors	3.5KN/m2
2. Live load on roof	1.5KN/m2

F. Load Combinations for Analysis

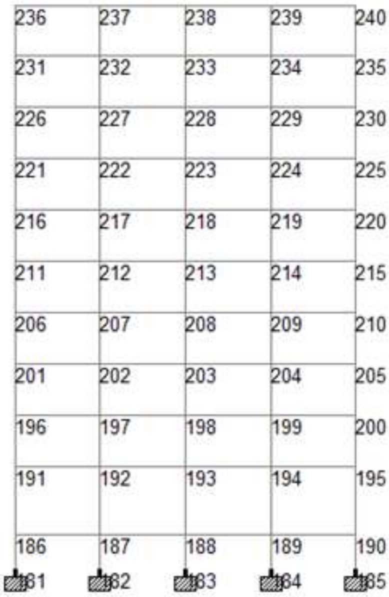
1. 1.5(DL+LL)
2. 1.2(DL+LL±EQ-X)
3. 1.2(DL+LL±EQ-Z)
4. 1.5(DL±EQ-X)
5. 1.5(DL±EQ-Z)
6. 0.9DL±1.5EQ-X 15
7. 0.9DL±1.5EQ-Z

III. Capacity Based Design Of Ten Storey Building Frame

Analysis of ten storey building frame is done by STAAD Pro V8i software. From the analysis result maximum value for moments and shear forces are taken. Figure 4 and 5 shows the beam, column and node number of frame for which designing is carried out. Analysis of ten storey building frame is done by using IS Code 1893 (Part 1): 2002. Design of ten storey building frame is done in STAAD Pro. V8i by using Code IS 13920: 1993.



"Fig. 4" Frame showing Beam and Column Number



"Fig. 5" Frame showing Node Number

G. Seismic Analysis of Building

Seismic Analysis of ten storey building is done in STAAD Pro. V8i for all load combination as per IS 1893(Part-1):2002.

H. Determination of Flexural Capacities of Beams

The flexural capacities of the beams under hogging and sagging condition for the provided reinforcement are determined

I. Establishing a Strong Column Weak Beam Mechanism

To eliminate the possibility of a column sway mechanism during the earthquake, it is essential that the plastic hinges should be formed in beams (except at the base of the columns of ground storey).

J. Moment Magnification Factor for Columns

Moment capacities of columns are to be checked for the sum of the moment capacities of beams at the joint with an over strength factor of 1.35 (adopted from Euro code, EC 8). If the sum of capacities of columns is less than the sum of moment capacities of beams multiplied by over strength factor, the column moments should be magnified by the factor by which they are lacking in moment capacity over beams. If the sum of column moments is greater than sum of beam moments, there is no need to magnify the column moments. In such cases the multiplying factor is taken as unity. After obtaining the moment magnification factors, the column flexural strengths are to be increased accordingly at every joint and the maximum revised moment from the top and bottom joints to be taken for design

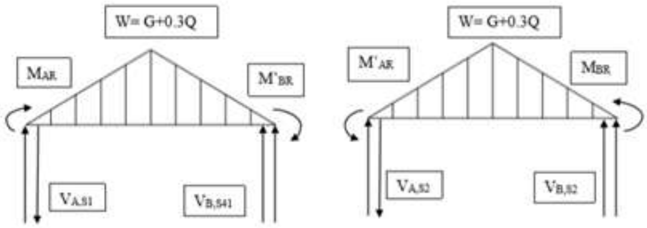
Table 5 Moment Magnification Factor for Columns

Joint No.	Seismic Direction	Sum of Resisting Moment of top and bottom Column at joint (1)	Sum of Resisting Moment of left and right beam at joint with an over strength factor 1.35(2)	Check (1)≥(2)	Moment Magnification Factor
191,195	1	330.78+209.27=540.05	1.35(0+485.25)=655.08	Not Ok	1.21
	2	330.78+209.27=540.05	1.35(0+485.25)=655.08	Not Ok	1.21
192,194	1	406.75+324.06=730.81	1.35(428.03+346.50)=1046.02	Not Ok	1.43
	2	406.75+324.06=730.81	1.35(346.80+428.02)=1046.02	Not Ok	1.43
193	1	401.14+317.97=719.11	1.35(428.03+301.49)=984.84	Not Ok	1.37
	2	401.14+317.97=719.11	1.35(301.48+428.02)=984.85	Not Ok	1.37
196,200	1	209.27+205.43=414.69	1.35(0+428.03)=577.84	Not Ok	1.39
	2	209.27+205.43=414.69	1.35(0+428.02)=577.83	Not Ok	1.39
197,199	1	324.06+310.27=634.33	1.35(395.99+295.16)=933.05	Not Ok	1.47
	2	324.06+310.27=634.33	1.35(295.16+395.98)=933.05	Not Ok	1.47
198	1	317.97+305.96=623.94	1.35(395.99+278.11)=910.04	Not Ok	1.46
	2	317.97+305.96=623.94	1.35(278.11+395.98)=910.03	Not Ok	1.46
201,205	1	205.43+201.53=406.96	1.35(0+428.03)=577.84	Not Ok	1.42

	2	205.43+201.53=406.96	1.35(0+428.02)=577.84	Not Ok	1.42
202,204	1	310.27+302.37=612.64	1.35(388.28+267.33)=885.08	Not Ok	1.44
	2	310.27+302.37=612.64	1.35(267.33+388.28)=885.08	Not Ok	1.44
203	1	305.96+297.85=603.82	1.35(388.28+267.33)=885.08	Not Ok	1.47
	2	305.96+297.85=603.82	1.35(267.33+388.28)=885.08	Not Ok	1.47
206,210	1	201.53+195.68=397.20	1.35(0+395.99)=534.58	Not Ok	1.35
	2	201.53+195.68=397.20	1.35(0+395.98)=534.58	Not Ok	1.35
207,209	1	302.37+289.07=591.44	1.35(370.72+258.63)=849.63	Not Ok	1.44
	2	302.37+289.07=591.44	1.35(258.63+370.72)=849.63	Not Ok	1.44
208	1	297.85+284.76=582.62	1.35(370.72+258.63)=849.63	Not Ok	1.46
	2	297.85+284.76=582.62	1.35(258.63+370.72)=849.63	Not Ok	1.46
211,215	1	195.68+186.15=381.83	1.35(0+388.28)=524.18	Not Ok	1.37
	2	195.68+186.15=381.83	1.35(0+388.28)=524.18	Not Ok	1.37
212,214	1	289.07+268.63=557.71	1.35(352.84+232.17)=789.76	Not Ok	1.42
	2	289.07+268.63=557.71	1.35(232.17+352.84)=789.76	Not Ok	1.42
213	1	284.76+264.43=549.19	1.35(352.84+232.17)=789.76	Not Ok	1.44
	2	284.76+264.43=549.19	1.35(232.17+352.84)=789.76	Not Ok	1.44
216,220	1	186.15+172.10=358.25	1.35(0+346.80)=468.19	Not Ok	1.31
	2	186.15+172.10=358.25	1.35(0+346.81)=468.19	Not Ok	1.31
217,219	1	268.63+239.52=508.16	1.35(334.64+278.42)=827.75	Not Ok	1.63
	2	268.63+239.52=508.16	1.35(278.52+334.63)=827.75	Not Ok	1.63
218	1	264.43+235.60=500.03	1.35(316.11+218.72)=722.03	Not Ok	1.44
	2	264.43+235.60=500.03	1.35(218.72+316.11)=722.03	Not Ok	1.44
221,225	1	172.10+152.33=324.43	1.35(0+301.49)=407.01	Not Ok	1.25
	2	172.10+152.33=324.43	1.35(0+301.48)=407.01	Not Ok	1.25
222,224	1	239.52+200.47=439.99	1.35(295.16+158.81)=612.86	Not Ok	1.39
	2	239.52+200.47=439.99	1.35(158.81+295.16)=612.86	Not Ok	1.39
223	1	235.60+197.19=343.54	1.35(295.16+177.53)=638.13	Not Ok	1.47
	2	235.60+197.19=343.54	1.35(177.53+295.16)=638.13	Not Ok	1.47
226,230	1	152.33+127.43=279.76	1.35(0+258.63)=349.16	Not Ok	1.25
	2	152.33+127.43=279.76	1.35(20+58.63)=349.16	Not Ok	1.25
213	1	284.76+264.43=549.19	1.35(352.84+232.17)=789.76	Not Ok	1.44
	2	284.76+264.43=549.19	1.35(232.17+352.84)=789.76	Not Ok	1.44
216,220	1	186.15+172.10=358.25	1.35(0+346.80)=468.19	Not Ok	1.31
	2	186.15+172.10=358.25	1.35(0+346.81)=468.19	Not Ok	1.31
217,219	1	268.63+239.52=508.16	1.35(334.64+278.42)=827.75	Not Ok	1.63
	2	268.63+239.52=508.16	1.35(278.52+334.63)=827.75	Not Ok	1.63
218	1	264.43+235.60=500.03	1.35(316.11+218.72)=722.03	Not Ok	1.44
	2	264.43+235.60=500.03	1.35(218.72+316.11)=722.03	Not Ok	1.44
221,225	1	172.10+152.33=324.43	1.35(0+301.49)=407.01	Not Ok	1.25
	2	172.10+152.33=324.43	1.35(0+301.48)=407.01	Not Ok	1.25
222,224	1	239.52+200.47=439.99	1.35(295.16+158.81)=612.86	Not Ok	1.39
	2	239.52+200.47=439.99	1.35(158.81+295.16)=612.86	Not Ok	1.39
223	1	235.60+197.19=343.54	1.35(295.16+177.53)=638.13	Not Ok	1.47
	2	235.60+197.19=343.54	1.35(177.53+295.16)=638.13	Not Ok	1.47
226,230	1	152.33+127.43=279.76	1.35(0+258.63)=349.16	Not Ok	1.25
	2	152.33+127.43=279.76	1.35(20+58.63)=349.16	Not Ok	1.25
227,229	1	200.47+149.85=350.32	1.35(238.84+156.45)=533.64	Not Ok	1.52
	2	200.47+149.85=350.32	1.35(156.45+238.84)=533.64	Not Ok	1.52
228	1	197.19+146.35=343.54	1.35(238.84+156.45)=533.64	Not Ok	1.55
	2	197.19+146.35=343.54	1.35(156.45+238.84)=533.64	Not Ok	1.55
231,235	1	127.43+84.79=212.23	1.35(0+177.53)=239.66	Not Ok	1.13
	2	127.43+84.79=212.23	1.35(0+177.53)=239.66	Not Ok	1.13
232,234	1	149.85+90.62=240.47	1.35(184.48+156.45)=460.26	Not Ok	1.91
	2	149.85+90.62=240.47	1.35(156.45+184.48)=460.27	Not Ok	1.91
233	1	146.35+93.37=239.71	1.35(184.48+156.45)=460.27	Not Ok	1.92
	2	146.35+93.37=239.71	1.35(156.45+184.48)=460.27	Not Ok	1.92
236,240	1	0+84.79=84.79	1.35(0+156.45)=211.21	Not Ok	2.49
	2	0+84.79=84.79	1.35(0+156.45)=211.21	Not Ok	2.49
237,239	1	0+90.62=90.62	1.35(156.45+156.45)=422.43	Not Ok	4.66
	2	0+90.62=90.62	1.35(156.45+156.45)=422.43	Not Ok	4.66
238	1	0+93.37=93.37	1.35(156.45+156.45)=422.43	Not Ok	4.52
	2	0+93.37=93.37	1.35(156.45+156.45)=422.43	Not Ok	4.52

K. Capacity Design for Shear in Beams

The design shear forces in beams are corresponding to the equilibrium condition of the beam under the appropriate gravity load (permanent dead load + % of live load) and to end resisting moments corresponding to the actual reinforcement provided, further multiplied by a factor γRd.



"Fig. 6" Equilibrium Condition for Determination of Sher Force

where, MAR, M'AR, MBR, M'BR are the actual resisting moments at the hinges accounting for the actual area of the reinforcing steel (all positive) and γRD the amplification factor taking into account the reduced probability that all end cross sections exhibit simultaneously the same over strength. G, dead load, Q, live load acting on the beam. For seismic direction 1

VAS1 = Wl/2 – γRD ({MAR + M'BR}/l)

VBS1 = Wl/2 + γRD ({MAR + M'BR}/l)

For seismic direction 2

VAS2 = Wl/2 + γRD ({M'AR + MBR}/l)

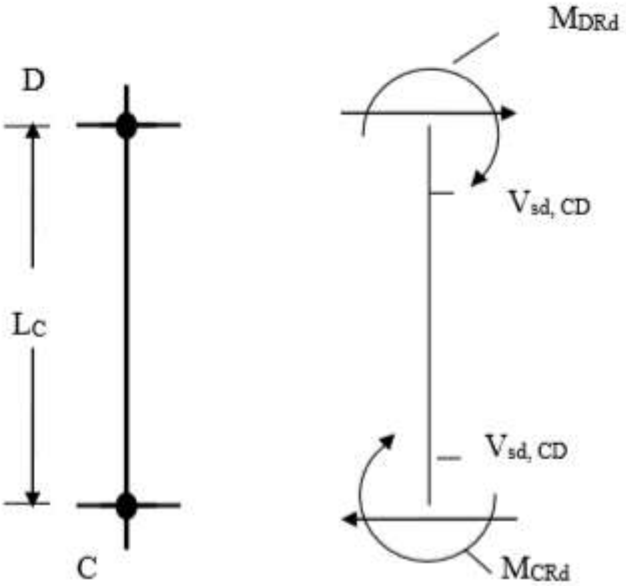
VBS2 = Wl/2 - γRD ({M'AR + MBR}/l)

Table 6 Capacity Design for Shear force beams

Beam No.	Moment capacity of beam	Load (kN)	Shear (VAS1)	Shear (VAS2)	Shear (VBS1)	Shear (VBS2)
102,135	485.246 346.8058	72.6	-135.413	280.613	280.613	-135.413
113,124	428.0273 301.4878	72.6	-109.779	254.9788	254.9788	-109.779
103,136	428.0273 295.1604	72.6	-108.197	253.3969	253.3969	-108.197
114,125	395.9891 278.1138	72.6	-95.9257	241.1257	241.1257	-95.9257
104,137	428.0273 267.3317	72.6	-101.24	246.4398	246.4398	-101.24
115,126	388.2853 267.3317	72.6	-91.3042	236.5042	236.5042	-91.3042
105,138	395.9891 258.635	72.6	-91.056	236.256	236.256	-91.056
116,127	370.7211 258.635	72.6	-84.739	229.939	229.939	-84.739
106,139	388.2853 232.167	72.6	-82.5131	227.7131	227.7131	-82.5131
117,128	352.8378 232.167	72.6	-73.6512	218.8512	218.8512	-73.6512
107,140	346.8058 278.5162	72.6	-83.7305	228.9305	228.9305	-83.7305
118,129	334.6355 218.7202	72.6	-65.7389	210.9389	210.9389	-65.7389
108,141	301.4878 158.8121	72.6	-42.475	187.675	187.675	-42.475
119,130	295.1604 177.529	72.6	-45.5723	190.7723	190.7723	-45.5723
109,142	258.635 156.4547	72.6	-31.1724	176.3724	176.3724	-31.1724
120,131	238.8372 156.4547	72.6	-26.223	171.423	171.423	-26.223
110,143	177.5298 156.4547	72.6	-10.8961	156.0961	156.0961	-10.8961
121,132	184.4828 156.4547	72.6	-12.6344	157.8344	157.8344	-12.6344
111,144	156.4547 156.4547	47	-31.2274	125.2274	125.2274	-31.2274
122,133	156.4547 156.4547	47	-31.2274	125.2274	125.2274	-31.2274

L. Capacity Design for Shear in Columns

Capacity design shear forces are evaluated by considering the equilibrium of the column under the actual resisting moments at its ends.



"Fig. 7" Capacity Design Values of Shear Forces Acting on Columns

VSD,CD = γRD (MDRd + MCRd)/lc

where, MDRd and MCRd are the flexural capacities of the end sections as detailed, lc is the clear height of the column and γRD = 1.35.

Table 7 Capacity Based Design for Shear in Columns

Column No.	Mz	Storey height	Capacity based shear of column
6,10	401.2378	4	270.8355
7,9	582.1904	4	392.9785
8	549.3698	4	370.8246
11,15	291.5931	3	262.4338
12,14	476.6694	3	429.0025
13	463.7795	3	417.4016
16,20	291.6901	3	262.5211
17,19	456.3824	3	410.7442
18	448.4853	3	403.6368
21,25	286.1468	3	257.5321
22,24	436.837	3	393.1533
23	436.5975	3	392.9378
26,30	268.632	3	241.7688
27,29	415.2639	3	373.7375
28	415.2708	3	373.7437
31,35	255.5531	3	229.9978
32,34	437.5866	3	393.8279
33	381.8241	3	343.6417
36,40	224.9139	3	202.4225
37,39	390.1682	3	351.1514
38	347.3834	3	312.6451
41,45	191.1028	3	171.9925
42,44	305.383	3	274.8447
43	306.3019	3	275.6717
46,50	159.0592	3	143.1533
47,49	286.8126	3	258.1313
48	280.9979	3	252.8981
51,55	211.2139	3	190.0925
52,54	422.4277	3	380.1849
53	422.4277	3	380.1849

IV. Results

Design results of both the method (conventional design and capacity design) are summarized in tables. And then comparison of design results is done and graph is drawn showing the variation in results. By graph we point out the advantage of capacity-based design and conclude the applicability of the capacity-based design. Design results which are compare-

- Moment in columns
- Shear in beams
- Shear in columns

M. Comparison Between Bending Moment in Columns

Table 8 Comparison Between Bending moment in columns

Column No.	Maximum Moment in columns by conventional based design (kNm)	Maximum Moment in columns by capacity-based design (kNm)
6	330.78	401.2378
7	406.75	582.1904
8	401.138	549.3698
9	406.75	582.1904
10	330.78	401.2378
11	209.269	291.5931
12	324.06	476.6694
13	317.975	463.7795
14	324.06	476.6694
15	209.2699	291.5931
16	205.43	291.6901
17	310.268	456.3824
18	305.963	448.4853
19	310.268	456.3824
20	205.43	291.6901
21	201.526	286.1468
22	302.371	436.837
23	297.853	436.5975
24	302.371	436.837
25	201.5236	286.1468
26	195.679	268.632
27	289.073	415.2639
28	284.763	415.2708
29	289.073	415.2639
30	195.679	268.632
31	186.152	255.5531
32	268.633	437.5866
33	264.429	381.8241
34	268.633	437.5866
35	186.152	255.5531
36	172.103	224.9139
37	239.523	390.1682
38	235.604	347.3834
39	239.523	390.1682
40	172.103	224.9139
41	152.332	191.1028
42	200.474	305.383
43	197.192	306.3109
44	200.474	305.383
45	152.332	191.1028
46	127.43	159.0392
47	149.846	286.8126
48	146.349	280.9979
49	149.846	286.8126
50	127.43	159.0392
51	84.796	211.2139
52	90.621	422.4277
53	93.366	422.4277
54	90.621	422.4277
55	84.796	211.2139

N. Comparison Between Shear Force in Beams

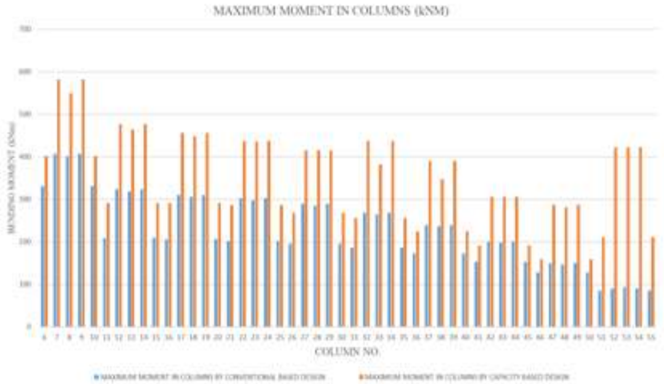
Table 9 Comparison between Shear Force in Beams

Beam No.	Shear force in beams by conventional design (kN)	Shear force in beams by capacity-based design (kN)
102,135	225.828	280.633
113,124	212.94	254.978
103,136	211.504	253.39
114,125	204.448	241.126
104,137	206.871	246.44
115,126	200.365	236.5
105,138	201.223	236.25
116,127	195.605	229.44
106,139	192.875	227.71
117,128	188.139	218.85
107,140	181.065	228.93
118,129	177.366	210.93
108,141	165.219	187.67
119,130	162.85	190.77
109,142	145.032	176.37
120,131	144.334	171.42
110,143	120.65	156.09
121,132	122.931	157.83
111,144	68.556	125.22
122,133	67.723	125.22

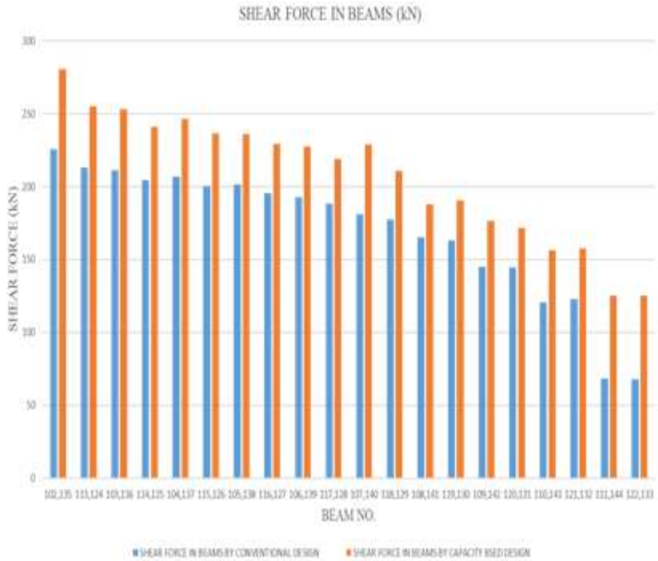
O. Comparison Between Shear Force in Columns

Table 10 Comparison between Shear Force in Beams

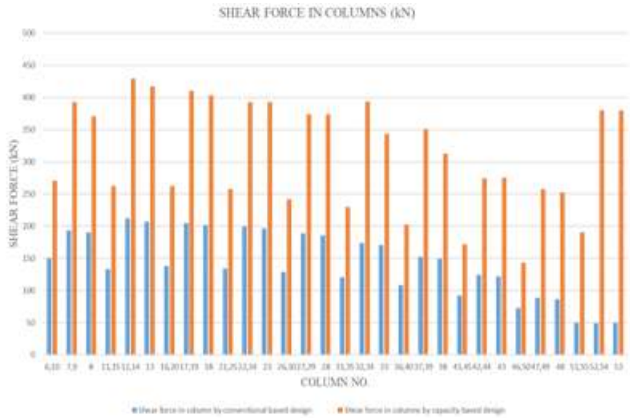
Column No.	Shear force in column by conventional based design (kN)	Shear force in columns by capacity-based design (kN)
6,10	150.043	270.8355
7,9	193.416	392.9785
8	190.197	370.8246
11,15	133.653	262.4338
12,14	212.125	429.0025
13	207.168	417.4016
16,20	138.094	262.5211
17,19	204.656	410.7442
18	201.971	403.6368
21,25	134.418	257.5321
22,24	199.195	393.1533
23	196.104	392.9378
26,30	128.883	241.7688
27,29	188.926	373.7375
28	186.052	373.7437
31,35	120.353	229.9978
32,34	173.632	393.8279
33	170.811	343.6417
36,40	108.477	202.4225
37,39	152.334	351.1514
38	149.68	312.6451
41,45	92.641	171.9925
42,44	124.297	274.8447
43	121.994	275.6717
46,50	73.029	143.1533
47,49	88.944	258.1313
48	86.718	252.8981
51,55	50.282	190.0925
52,54	48.923	380.1849
53	50.345	380.1849



“Fig. 8” Comparison of column Moment



“Fig. 9” Comparison of Shear Force in Beams



“Fig. 10” Comparison of Shear Force in Columns

V. Conclusions

- From the comparison it can be seen that column moments obtained from capacity-based design are more than the column moments obtained from conventional design method. It can be noticed that increase in column moments is more for interior columns then exterior columns. Due to increase in column moments by capacity-based design method increase the capacity of columns so that the formation of plastic hinges in the columns can be avoided.

- In ground floor increment in column moment is 21.3005% (for exterior column) to 43.1.224% (for interior column).
- From the comparison it can be seen that beam shear obtained from capacity-based design are more than the beam shear obtained from conventional design method. Due to increase in beam shear by capacity-based design method increase the shear capacity of beam to avoid the brittle failure.
- In sixth floor increment in shear force is 13.588% (for exterior beams) to 17.14% (for interior beams).
- From the comparison it can be seen that column shear obtained from capacity-based design are more than the column shear obtained from conventional design method. Due to increase in column shear by capacity-based design method increase the shear capacity of column.
- In third floor increment in shear force is 91.6% (for exterior columns) to 97.4% (for interior columns). The increase in column shear is significant for exterior and interior columns.
- For earthquake resistant design capacity-based design method is more appropriate and realistic method of design.
- In this method design of building is based on the provided reinforcement.
- In this method structure’s over strength takes its reserve strength beyond its elastic limit.
- This method of earthquake resistant design is costlier than the conventional method due to increase in column moment, column shear and beam shear but it is very effective for resisting earthquake forces.

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