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Correlation of Capacity Based Design and Force Based Design of Reinforced Concrete Structure

Owais Bashir Yatoo[1], and Er. Manish Kaushal[2]

[1]M. Tech Scholar, Department of Civil Engineering, RIMT University, Mandi Gobindgarh, Punjab, India
[2]Assistant Professor, Department of Civil Engineering, RIMT University, Mandi Gobindgarh, Punjab, India

Correspondence should be addressed to Owais Bashir Yatoo; owaisbashir776@gmail.com

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ABSTRACT- The main cause of failure of multi-storey multi-bay reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. If the frame is designed on the basis of strong column-weak beam concept the possibilities of collapse due to sway mechanisms can be completely eliminated. In multi storey frame this can be achieved by allowing the plastic hinges to form, in predetermined sequences only at the ends of all the beams while the columns remain essentially in elastic stage and by avoiding shear mode of failures in columns and beams. This procedure for design is known as Capacity Based Design (CBD) which would be safer than current design philosophy used for earthquake resistant design of multi storey multi bay reinforced concrete frames in India.
The present Dissertation work is an effort to understand Capacity Based Design Approach. In this Work a three storey workshop building a linear static analysis is carried out and then building is designed by force based method. Then for Capacity Based Design columns are designed for magnified moment by moment magnification factor. A mechanism is applied to subsequent failure of storey before vertical members by strengthening the columns with respect of beams. Apart from improving axial strength of columns, shear capacity of beams and columns also improved in capacity based method. Reinforcement obtained from this design compared to Force Based Design and concluded that CBD method is little conservative but it ensure to prevent column sway mechanism.

KEYWORDS- Capacity Based Design, Reinforced concrete, Ductility, Beams and Columns.

I. INTRODUCTION

Perhaps the most difficult aspect of structural engineering is designing buildings to withstand earthquakes. An earthquake's destructive, unexpected, and hard-to-characterize forces make it the most extreme threat a structure is likely to face. Because of this, even in nations with up-to-date building standards, earthquakes may inflict a lot of tragedy. Because of its superiority in meeting the seismic requirement of buildings, the Capacity Based Earthquake Resistant Design (CBERD) is gaining prominence in seismic design at the expense of the more traditional code-based design method. It has become the norm for earthquake safety in many industrialized nations. Response Reduction Factor (R), a measure of the structure's ductility and over strength, is used to account for the fact that, in strong earthquakes, the buildings will enter the inelastic region. However, this code-based approach does not account for the possibility of structural damage. However, the inelastic demand that is put on the structure during significant earthquake scenarios may be calculated using a capacity-based seismic engineering technique, which eliminates this drawback.

A. Seismic Analysis Procedures

Main features of seismic method of analysis based on Indian Standard IS 1893(part 1): 2002 are described as follows:

- Equivalent lateral force method
- Response spectrum analysis
- Elastic time history analysis

II. LITERATURE REVIEW

A. Capacity Design of Reinforced Concreteframes

The capacity design criteria that are now considered essential in order to prevent non-ductile failures and enable the building to survive earthquake attack without damage are outlined as the basis for the design of RC frames for fully ductile earthquake performance, which is applicable to both low buildings and major structures. This design principle is applicable to both low buildings and major structures.
Wangsadinata [1] show the load resistant factor technique was used to define the architectural ideas that go into creating structures that are earthquake resistant. The value of the ductility factor of the structure may be selected according to the demand as one type of performance-based design. The possible values for this factor range from the value for a fully elastic structure all the way up to the value for a fully ductile structure. This research provides the most recent revision of the seismic zoning map for Indonesia, as well as the elastic and reduced spectra for each of the zones. For the purpose of producing spectra for each zone.
Wiratman [2] used a three-dimensional building. He came to the conclusion that capacity-based design entails planning the flexural capabilities of a structural part; nevertheless, this also guarantees that the structure will not collapse under the influence of powerful earthquakes.

Tena-Colunga [3] provided an explanation of the notion of inelastic capacity design spectrum for base isolated structures, particularly those structures that make use of isolators that exhibit a bilinear hysteretic behaviour when subjected to the impact of seismic loading. The inelastic capacity design spectra related the peak nonlinear velocities, the displacements, the accelerations, and the effective isolated natural periods for the bilinear systems with a given post yield stiffness and the yield strength. This was done so that the spectra could be used to design inelastic devices. Since base isolators may be built for a set post-yield stiffness and yield strength, the inelastic capacity design spectra can be beneficial for designing base isolators with bilinear hysteretic behaviour.

Crisafulli et al. [4] cantilever infilled frames, in which yielding of the longitudinal reinforcement control was considered in order to achieve ductile behaviour at the base of the columns, a very new design approach was proposed by using the principles of capacity based design. This was done in order to achieve the goal of having ductile behaviour. The link between the foundation and the infill frames is intentionally made to be pre-cracked. In this location, simple round dowels need to be put so that the shear sliding may be controlled. This approach also provided a straightforward measurement of the lateral resistance of the structure, reducing ambiguities connected with the intricacy of the interaction between the panel and the frame. It has been suggested that tapered beam-column joints be used in conjunction with diagonal reinforcement in order to cut down on the opening of joints and to shift lateral pressures from the frame to the masonry panel.

Rahal [5] presented a very simple and rational method for calculating capacity as well as identifying the mode of failure of the membrane elements that had been subjected to combined in-plane shearing and normal stresses. This method was very successful in determining the mode of failure of the membrane elements. The non-iterative technique that has been given the term simple model for combined stress resultants (SMCS) is based on the simplification of findings derived from the modified compression field theory (MCFT). This simplification results in equations that have a similar format to the equations derived from the Plasticity Theory; the only differences are that the contribution of the reinforcement that is larger than the 'balanced' reinforcement is considered to be zero, and the maximum attainable shearing stress has been limited to a crushing strength that is based upon data from experimentation, from normal and high strength concrete membranes (100 N/mm2).

Panyakapo [6] given in the form of a graphical plot of strength capacity against the displacement of structures, whilst the damages caused by seismic systems are indicated in the language of damage indices. A nonlinear approach of static analysis known as the cyclic pushover method was used in order to get the seismic capacity diagram. In the Performance-Dependent Design (PBD) framework, there is a model that has been developed in which the seismic capacity is based on damage. A Seismic Damage-based Capacity Diagram is the name given to this technique to design. During the process of computing the damage index, the well-known Park-Ang damage model was used. In order to account for the hysteretic behavior of structures, a modified version of the Takeda model was used. The Cyclic Pushover Method was used to the investigation of a structure that was fifteen stories tall. It was discovered that the damages that occurred on the higher levels were far less severe than those that occurred on the lower floors. This was the primary reason for the accumulative damage that had occurred as a consequence of the absorption of hysteretic energy during the cyclic load reversal. In terms of the demand–capacity diagrams, the applicability to PBD has been examined. In the last step of this investigation, the performance point was appraised on the basis of damage. Nonlinear time history analysis is the approach that provides the most precise predictions of seismic demand and the most thorough assessments of the performance of structures.

Bajoria and Sangle [7] modeled a structure with the help of the FEM tool SAP2000. The nonlinear static pushover analysis is a straightforward way of analysis that has been used in place of the time history analysis. This analysis approach was chosen. The purpose of this study is to evaluate the performance of the structure by first calculating its strength and its capacity for deformation using nonlinear static analysis, and then comparing these capabilities with the requirements imposed by the various degrees of performance. In this particular research project, nonlinear pushover analyses were discovered to be an effective analysis tool for the structure of the storage rack. These analyses were found to provide very good estimates of base shear, the displacement, and the formation of plastic hinge at each specified load increment.

The seismic design of precast buildings may be based on the conventional capacity design criteria as long as it is assured that the connections will have a satisfactory behavior during earthquakes and will not break brittlely at an early stage. The knowledge of the seismic behaviour of the connections, such as in terms of the ductility resources, is currently nonexistent, and it is necessary to develop capacity design criteria for their proportioning. On the basis of such a need.

Biondini et al. [8] presented, in a well-organized manner, capacity design criteria to be applied for the various types of connections that may be situated in a typical precast concrete structure for industrial buildings, which was designed for earthquake resistance. These criteria were intended to be applied in order to ensure that the structures can withstand earthquakes..

George and Varghese [10] described the capacity design is to make the vertical components of a structure much more robust in comparison to the horizontal structural parts. The structures, which are developed in accordance with the idea of capacity-based design, do not generate any adequate failure mechanisms or the modes of inelastic deformation that might lead to the collapse of the structures. In capacity-based ERD structures, the components of the major lateral load resisting system are selected appropriately, designed, and then specified for energy dissipation under severe inelastic deformation. This is done so that the structures can withstand the loads they are planned to withstand. Following are some of the findings and conclusions that can be derived from a research that studied the performance of RC frames by employing the push-over analysis:

- The behaviour of the detailed RC frame construction is adequate as has been revealed by the intersection of capacity and the demand curves and the distribution of the hinges in columns and beams.
- The pushover analysis is a straightforward technique to investigate the non-linear behaviour of structures.

III. NECESSITY AND PURPOSE OF CAPACITY BASED DESIGN

Civil engineering structures are mainly designed to resist static loads. Generally the effects of dynamic loads acting on the structure are not considered. This feature of neglecting the dynamic forces sometimes becomes the cause of disaster, particularly in case of earthquake. The example of this category is Bhuj earthquake occurred on Jan.26, 2001. This has created a growing interest and need for earthquake resistant design of structures.

Conventional Civil Engineering structures are designed on the basis of strength and stiffness criteria. The strength is related to ultimate limit state, which assures that the forces developed in the structure remain in elastic range. The stiffness is related to serviceability limit state which assures that the structural displacements remains within the permissible limits. In case of earthquake forces the demand is for ductility. Ductility is an essential attribute of a structure that must respond to strong ground motions. Ductility is the ability of the structure to undergo distortion or deformation without damage or failure which results in dissipation of energy. Larger is the capacity of the structure to deform plastically without collapse, more is the resulting ductility and the energy dissipation. This causes reduction in effective earthquake forces. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The correct building components need to be made ductile. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than columns. This method of designing RC buildings is called the strong-column weak-beam design method.

Capacity design of structures seeks to use the advantages of ductile behavior in order for buildings to resist seismic loading. Certain structural elements are designed as ductile in order to exhibit inelastic behavior and prevent collapse under extreme loading. Additionally, these ductile elements are designed and detailed to fail prior to other brittle components of the structure. For a reinforced concrete member in flexure, this translates to tensile failure of the ductile steel reinforcement before the concrete, which is brittle, fails in compression. For the seismic design of larger structures, an engineer determines the plastic failure mechanism of a structure and carefully assigns which components will remain elastic and which ductile components will serve to dissipate energy through inelastic behavior with the formation of plastic hinges.

Most of the energy developed during earthquake is dissipated by columns of the soft stories. In this process the plastic hinges are formed at the ends of columns, which transform the soft storey into a mechanism. In such case the collapse is unavoidable. Therefore, the soft stories deserve a special consideration in analysis and design.

IV. METHODOLOGY AND RESULT

The entirety of this chapter discusses the characteristics of the materials that were used in the design process, the modelling process that was carried out, the calculation of base shear in accordance with the IS 1893: 2002 code, the procedure of capacity-based design, and finally, a summary of the entire structural modelling process is provided.

A. Structural Modeling

After the building has been designed in STAAD.Pro, it is then exposed to a variety of load combinations consisting of dead load, live load, and earthquake load. The reinforcement that was necessary for bearing the loads under the most difficult conceivable conditions was provided by the STAAD.Pro output file. The Base Shear is computed by using the rules described in Clause 7.5.3 of the IS 1893:2002 standard, and the resulting pattern is shown for future reference. The reinforcements that are produced as a consequence of the STAAD output file are compiled into a results file, where they are organised and stored. The action of the diaphragm takes into consideration the stiffness that is given by the slab. STAAD.Pro is to be used throughout the process of linear elastic analysis. In addition, the structure is intended to withstand not only dead load but also live load and seismic load. In accordance with IS 456: 2000, a number of different load configurations were tested, and the worst-case scenario was taken into consideration while designing the appropriate member. Self weight, brick load, and floor load are the components that make up dead load. Using the given density and size, an automated calculation was performed to determine the self-weight. The unit weight of a brick was determined to be 20 kN/m3 when measured over the beams.

Capacity Based Design of G+3 storey building

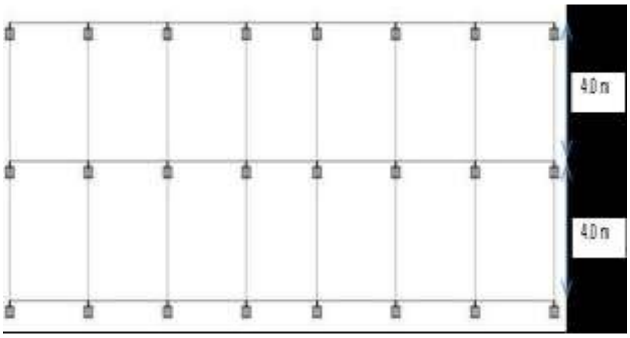
G+3 RC framed workshop building has been designed on the basis of capacity design philosophy by following IS 13920:1993. The basic concept of capacity design of structure is the spreading of inelastic deformation demands throughout the structures in such a way that the formation of plastic hinges take place at pre-determined positions at sequences.

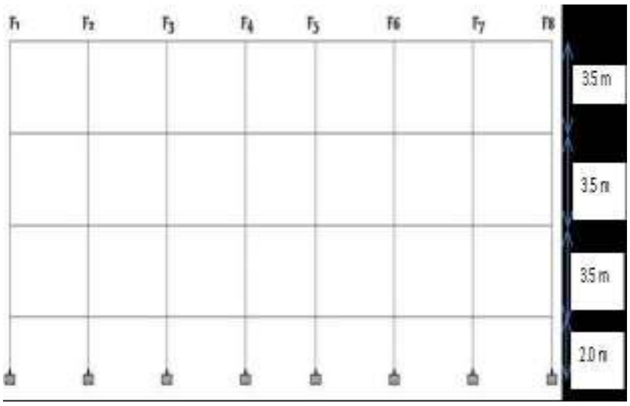
B. Building Details

A four storeyed (G+3) RC framed building is taken for analysis and design. The plan, elevation and side view are shown in Figure 4.1 (A, B, C), three-dimensional view of building is shown in Figure 4.2. The salient features of the building are:

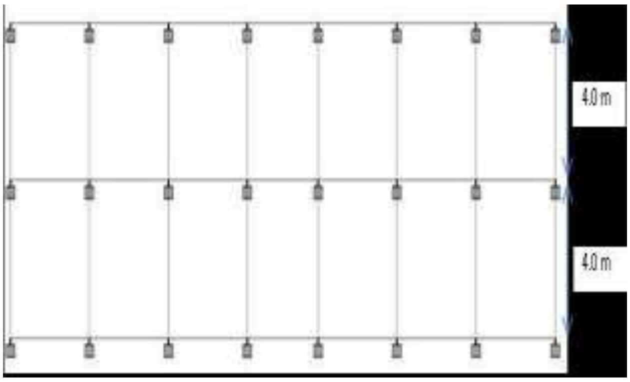
- Type of structure : multi storey rigid joint frame for workshop
- Seismic zone V
- Type of soil : Medium
- No. of stories : (G+3)
- Imposed Load : 5 KN/ m²
- Imposed Load at roof : 0.75 KN/m²
- Depth of slab : 100 mm

- Materials : M 20 concrete and Fe 415 steel
- Unit weight of RCC : 25 KN/m^3
- Beams ---
  : Plinth Beams - 300 × 500 mm
  : Ground storey beams – 300 x 600 mm
  : Second storey beams- 300 x 600 mm
  : Top storey beams- 300 x 500 mm
- Columns : 400 × 600 mm
- Clear cover of beam : 30 mm
- Clear cover of column : 60 mm
- Wall thickness : 230 mm





(A) levation



(B) Plan ide View

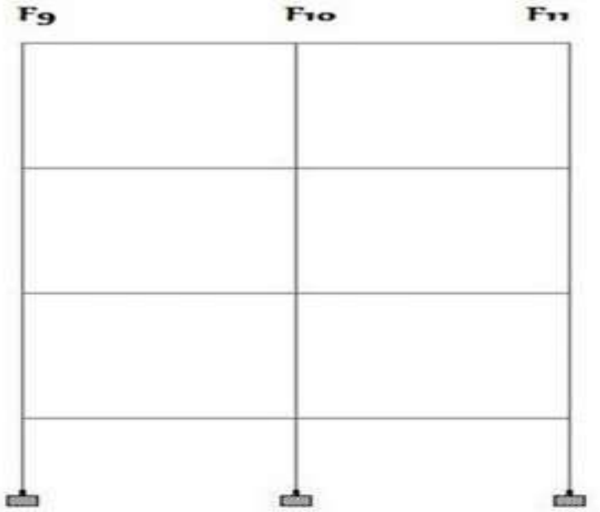


Figure 1: (A) Elevation, (B) Plan and (C) Side View of G+3 RC framed building

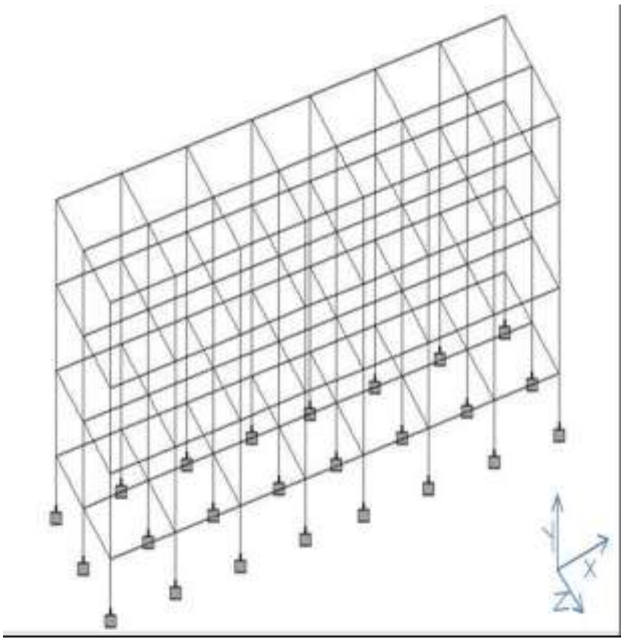


Figure 2: Three-dimensional view of G+3 Reinforced concrete building

C. Capacity Based Design of Structure

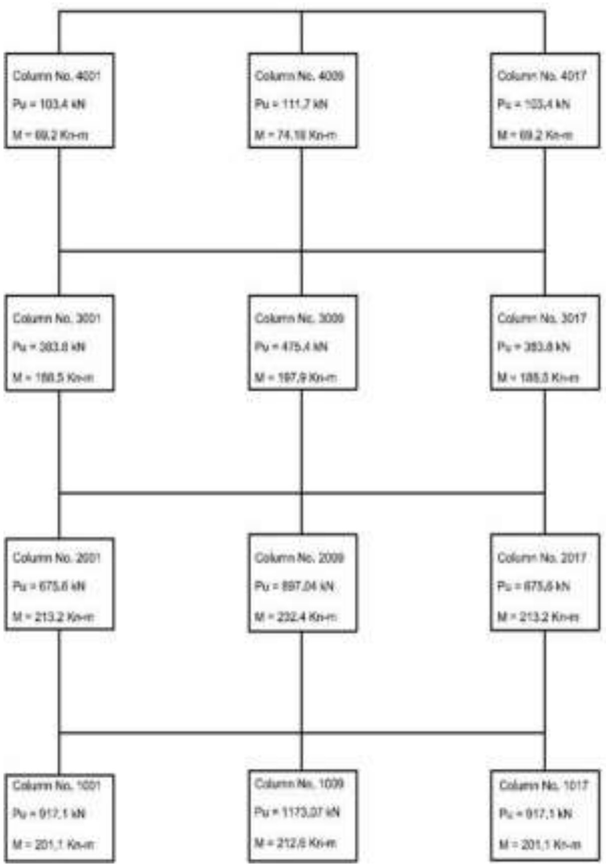
In capacity based design of this framed building sub frame are divided in two directions which are F1, F2, F3, F4, F5, F6, F7, F8 in XZ plane and F9,F10,F11 are in XY plane as shown in figure no.1. and 2 . Since F1 and F8, F2 and F7, F3 and F6, F4 and F5, F9 and F11 are symmetrical then in this chapter only F1, F2, F3, F4, F9, and F10 are designed. Design of remaining frames will be same as respective symmetrical frame.

1)Seismic Analysis of Frames

Seismic analysis of the frame for all load combination specified in IS 1893 (Part I):2002 are done. Maximum Design axial and biaxial bending forces for columns from all combinations are obtained. In figures Pu is used for maximum axial forces on each column of frames and column size is 300x600 mm2 throughout the building. Whole building is divided in two different types of frames

(i) planes in YZ direction and (ii) plane in XY direction. Figure no. 3 (A), 3(B), 3(C), 3(D) are showing the values of the maximum bending moment and axial force at each member of the frame in YZ plane and Figure no. 4 and Figure no. 5 are showing the values of the maximum bending moment and axial force at each member of the frame in XY plane. These results are obtained from the linear elastic a

(A)



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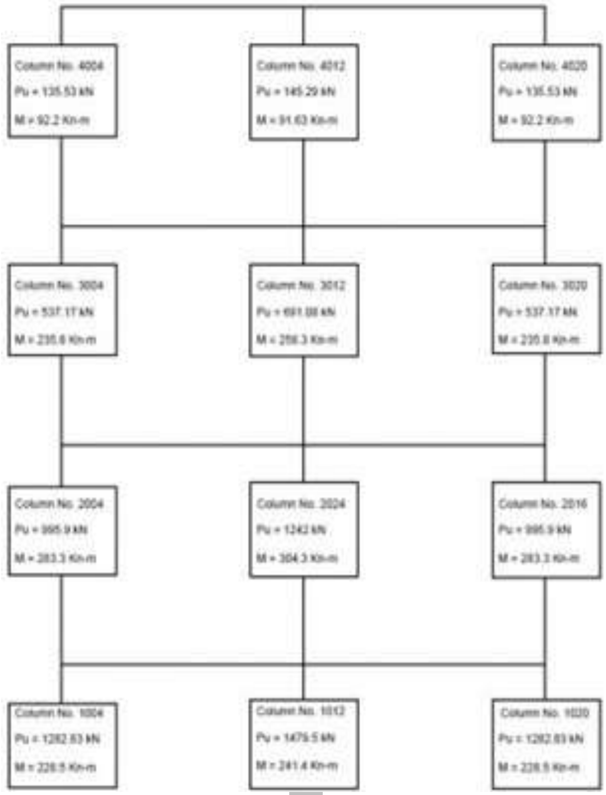


(C)

(D)

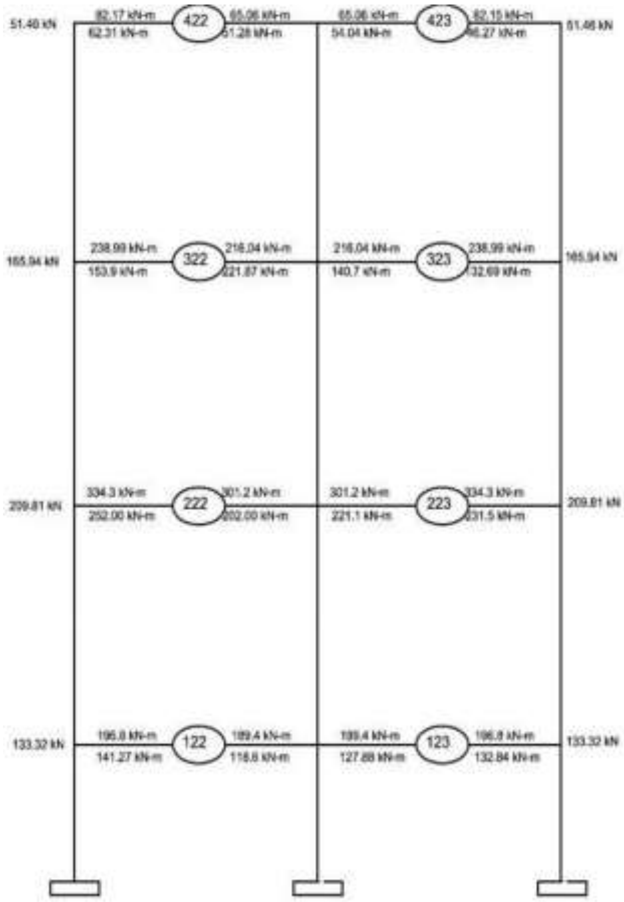
Figure 3: Max Design axial & biaxial bending forces for columns in frames (A) F1, (B)F2, (C) F3 and (D) F4

Figure 4: Max Design axial & biaxial bending forces for col. in frame F9 in XY plane

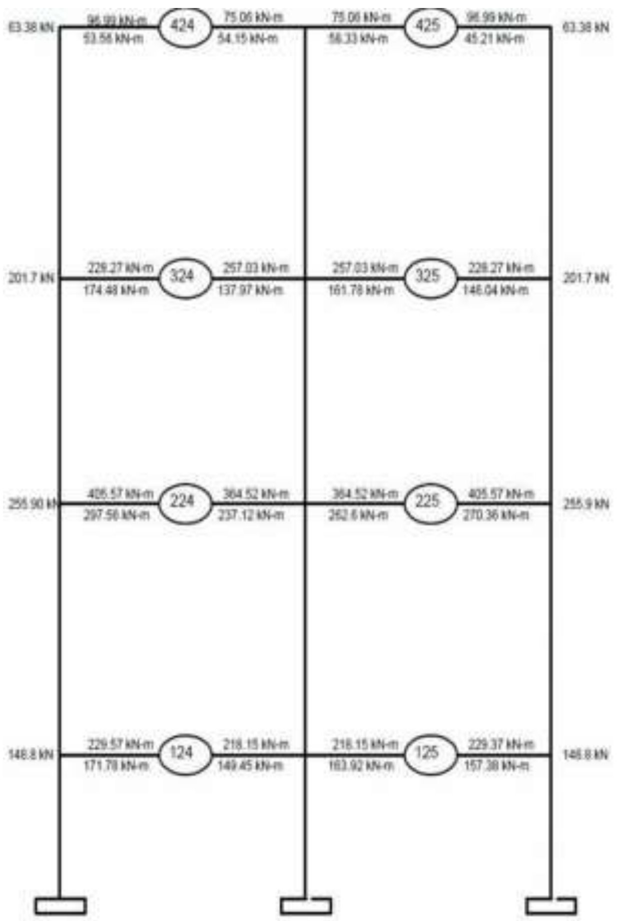
Figure 5: Max Design axial & biaxial bending forces for col. in frame F10 in XY plane

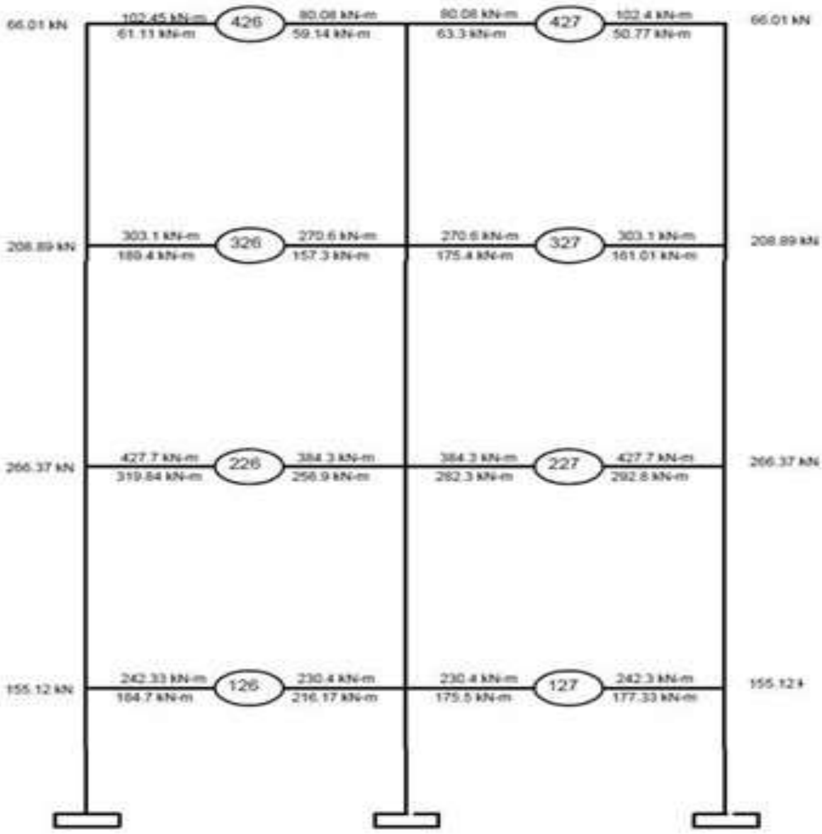
Determination of maximum design shear, maximum hogging and sagging bending moments.

The design forces are not completely based on linear elastic analysis, rather than they depend upon flexural capacities of the beams framing in to same joint. So that plastic hinges may not form at the base of column and at the top of the column below joint (Except at the base of the column of ground storey). From linear elastic analysis by STAAD.pro maximum design shear, maximum hogging bending moment and maximum sagging moment is obtained. In Figure no.6, Figure no.7. The obtained maximum value of shear force from all combination is written in side of end of beam column joints. This maximum shear is distributed same throughout of adjacent beam in longitudinal direction of first beam. Maximum hogging moment from all combinations is written on joint upside of beams and maximum sagging moments obtained from all combinations is written on beam column joint under the beam. Maximum hogging moment and maximum sagging moments are obtained for each beam at starting point and end point of beam.

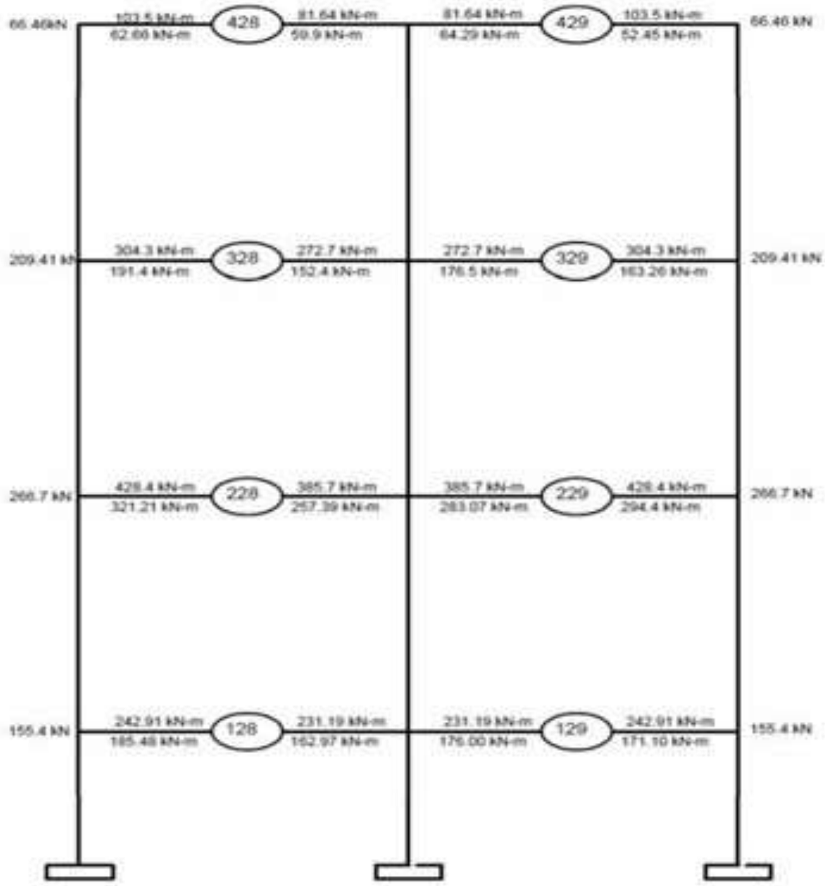


(A)





(C)



(D)

Figure 6: Max design shear, max hogging and sagging bending moment of beams of frame (A) F1, (B) F2, (C) F3 and (D) F4

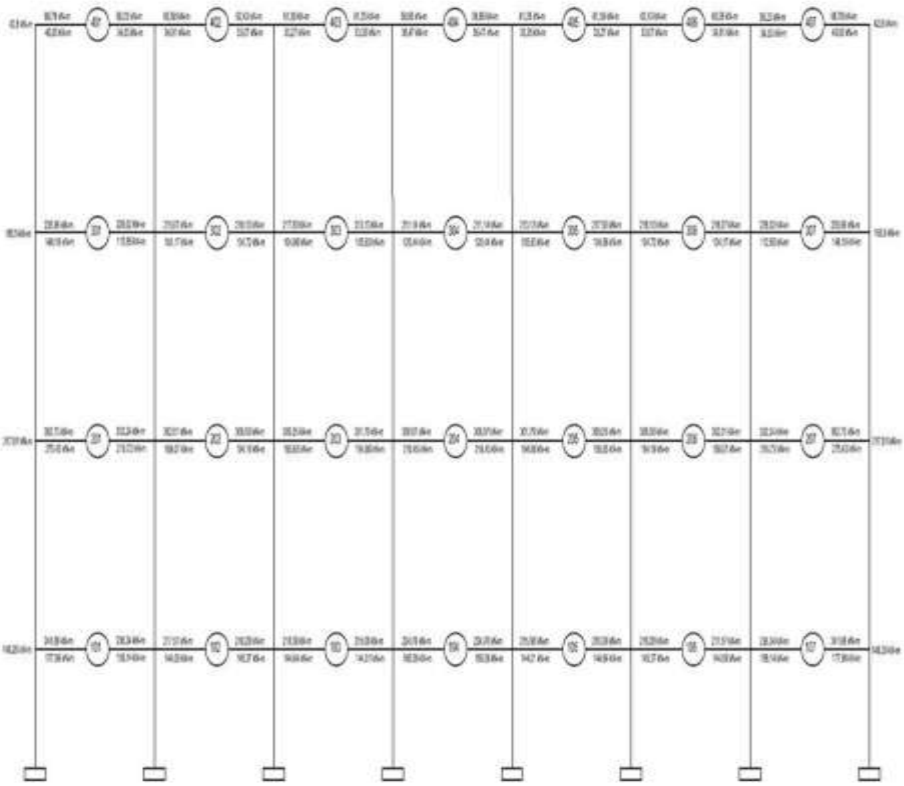


Figure 7: Max design shear, max hogging and sagging bending moment of beams of frame F9

V. COMPESRISON OF RESULTS BETWEEN CBDM AND FBDM

A. *Comperision of Required Reinforcement from Capacity Based Design and Force Based Design*

When the capacity based design of the structure was compared to the force based design of the structure, it was found that the needed reinforcement to the columns when using the capacity based design was more than the required reinforcement when using the force based technique. A comparison of required reinforcement is shown in figure NO.8 of all vertical structural members by both the methods. Also in capacity based design effect of shear in beams is not negotiable. Compression of shear reinforcement by capacity based method and force based method in beams of different floors is shown in figure no.9 . A tabular comparison of shear reinforcement of beam and required reinforcement by CBD and FBD methods is given below.

B. *Determination of Moment Capacity*

Here an example is calculating moment capacity at joint 101 of frame F1 is done. Provided top steel = 1570 mm2, Provided top steel = 1142 mm2,
beam dimension =300mmX600mm, d= 500-30 = 470 mm

1)Hogging Moment Capacity

MUlim= 0.36 * (XUmax/d)*[1-0.42*(XUmax/d)]*Bd2Fck MUlim=182.85 kN-m
Steel corresponding to MUlim is
Ast1= (0.48*0.36*20*300*470)/(0.87*415)
Ast1= 1350 mm2 Available Ast2= Ast - Ast1
Ast2= 1570-1350 = 220 mm2
Additional moment capacity due to available steel M2= (Asc*fsc)(d-d')/106 M2=177.37 kN-m
Ast2 required for provide Asc,
(Asc*fsc)/(0.87*415)=1116.5 mm2 (but available is 220 mm2 only) Hence M2= 0.87 fy*Ast2 *(d-d')/106 = 34.96 kN-m
Total Hogging Capacity = MUlim+M2 = 217.79 kN-m

2)Sagging Moment Capacity

Ast= 1142 mm2
bf = (l0/6)+ bw + 6Df = (.7*4000/6)+300+150 = 1666.6 mm
for Xu<df, the moment of resistance of T- beam is given by equation Xu= (.87 *415*Ast)/(.36*fck*bf) = 34.36 <150
Therefore MU= 0.87*415*Ast2*d*[1-(Ast*415)/(bf*d*fck)]/106 kN-m Totat sagging moment capacity Mu=187.92 kN-m

C. *Longitudinal Reinforcement in Columns*

Table 1: Longitudinal Reinforcement in Columns

	Column no	Required R/F ^{2}) as per CBD	Required R/F ^{2}) as per FBD
4	4001,4008,4017,4024	1869	1920
4	4002,4007,4018,4023	2052	1920
4	4003,4006.4019,4022	2736	1920
4	4004,4005,4020,4021	2827	1920
4	4009,4016	2189	1920
4	4010,4015	2781	1920
4	4011,4014	2827	1920
4	4012,4013	2827	1920
3	3001,3008,3017,3024	3237	3346
3	3002,3007,3018,3023	7205	4455
3	3003,3006,3019,3022	5016	4310
3	3004,3005,3020,3021	8162	4601
3	3009,3012	3465	3518
3	3013,3016	5882	4710
3	3014,3015	5928	4710
3	3010,3011	5836	3518
2	2001,2008,2017,2024	3557	3534
2	2002,2007,2018,2023	7205	5454
2	2003,2006,2019,2022	7296	5241
2	2004,2005,2020,2021	8162	5531
2	2009,2012	3648	3811
2	2013,2016	7752	5945
2	2014,2015	7296	5945
2	2010,2011	7980	5900
1	1001,1008,1017,1024	3557	3534
1	1002,1007,1018,1023	7205	5454
1	1003,1006,1019,1022	7296	5241
1	1004,1005,1020,1021	8162	5331
1	1009,1011	3648	3811
1	1014,1013	7752	5945
1	1015,1012	7296	5945
1	1016,1010	7980	5900

D. *Shear Reinforcement in Beams*

Table 2: Shear Reinforcement in Beams

Stoery	Beam no	Shear R/F provided by CBD.	Shear R/F provided by FBD.
4	422,423	2L- 6Φ @100 mm	2L- 6Φ @100 mm
4	424,425	2L- 6Φ @100 mm	2L- 6Φ @100 mm
4	426,427	2L- 6Φ @100 mm	2L- 6Φ @100 mm
4	428,429	2L- 6Φ @100 mm	2L- 6Φ @100 mm

4	401 to 407	2L- 6Φ @100 mm	2L- 6Φ @100 mm
4	408 to 414	2L- 6Φ @100 mm	2L- 6Φ @100 mm
4	415 to 421	2L- 6Φ @100 mm	2L- 6Φ @100 mm
3	322,323	2L- 8Φ @95 mm	2L- 8Φ @335 mm
3	324,325	2L- 8Φ @75 mm	2L- 8Φ @330 mm
3	326,327	2L- 8Φ @75 mm	2L- 8Φ @335 mm
3	328,329	2L- 8Φ @80 mm	2L- 8Φ @340 mm
3	301 to 307	2L- 8Φ @100 mm	2L- 8Φ @335 mm
3	308 to 314	2L- 8Φ @75 mm	2L- 8Φ @335 mm
2	222,223	2L- 8Φ @65 mm	2L- 8Φ @ 215mm
2	224,225	2L- 8Φ @75 mm	2L- 8Φ @220 mm
2	226,227	2L- 8Φ @55 mm	2L- 8Φ @215 mm
2	228,229	2L- 8Φ @65 mm	2L- 8Φ @215 mm
2	201 to 207	2L- 10Φ @85 mm	2L- 8Φ @210 mm
2	208 to 214	2L- 10Φ @72 mm	2L- 8Φ @215 mm
1	122,123	2L- 8Φ @100 mm	2L- 8Φ @390 mm
1	124,125	2L- 8Φ @80 mm	2L- 8Φ @390 mm
1	126,127	2L- 8Φ @80 mm	2L- 8Φ @385 mm
1	128,129	2L- 8Φ @80 mm	2L- 8Φ @390 mm
1	101 to 107	2L- 8Φ @90 mm	2L- 8Φ @390 mm
1	108 to 114	2L- 8Φ @70 mm	2L- 8Φ @395 mm

The longitudinal reinforcement provided in the columns of structure in all 4 floors and also the shear reinforcement provided in beams of structure at all the 4 floors is mentioned in table 1 and table 2 respectively The main purpose of the longitudinal reinforcement is the absorption of bending tensile stresses in the longitudinal direction of the main support direction of the structural component .The purpose of shear reinforcement is to prevent failure in shear, and to increase beam ductility and subsequently the likelihood of sudden failure will be reduced.

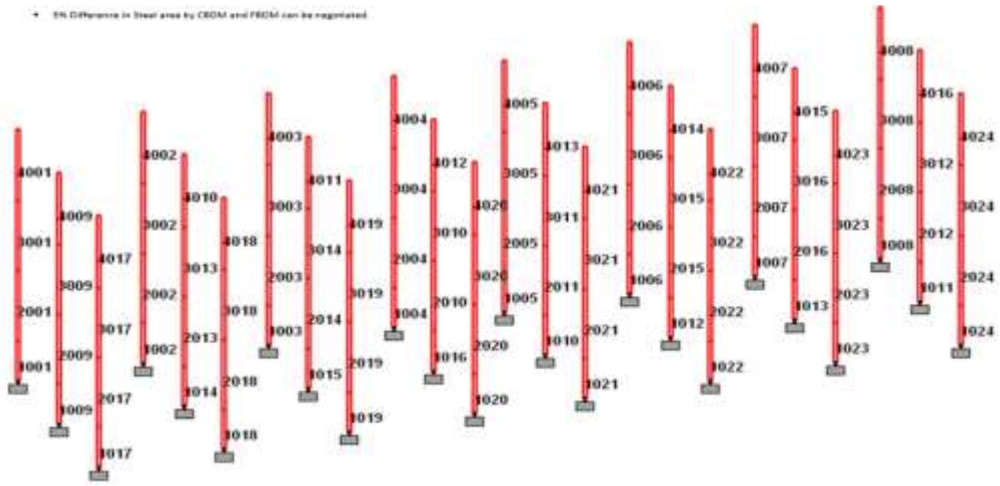
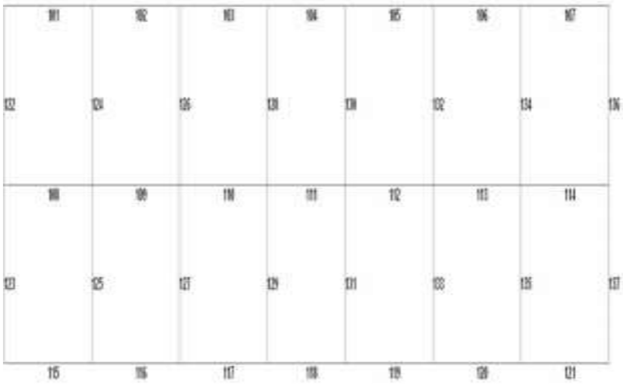
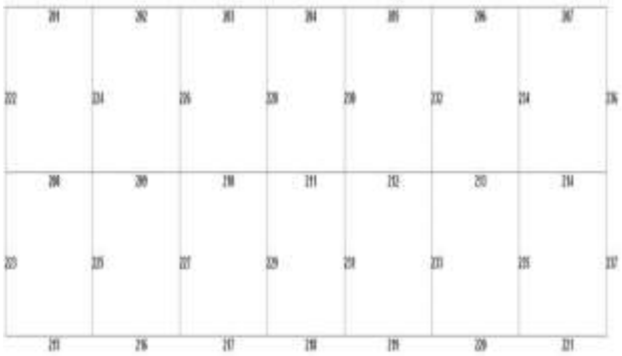


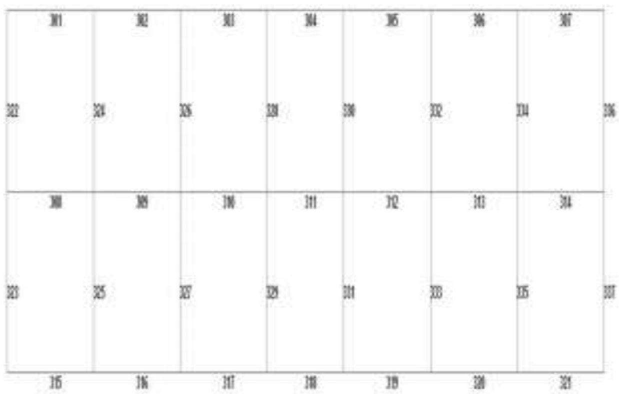
Figure 8: Difference in steel area in (mm2) of columns by both methods (CBDM and FBDM)



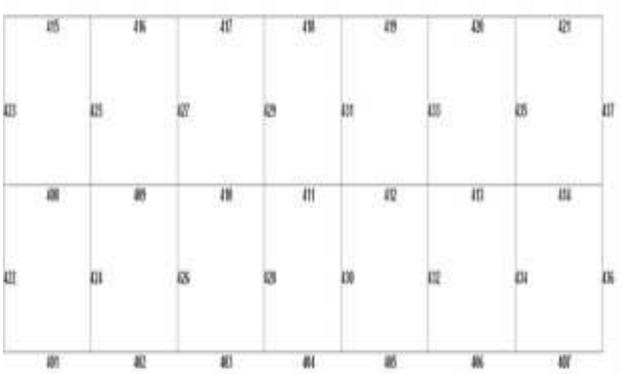
(A)1st floorplan



(B) 2nd floor plan



(C) 3rdfloorplan



(D) Roof plan

Figure 9: Shear Reinforcement in beams by both methods (CBDM and FBDM)

VI. CONCLUSION

In the present study, the Capacity Based Design (CBD) of a (G+3) four storey RCC frame using STAAD.Pro under the seismic loadings. After design, comparison has been carried out with the intention to study the relative steel provided in case of CBD and Force Based Design (FBD). The main observations and conclusions drawn are summarized below:

- Capacity-based design is a more forward-thinking approach to the design of reinforced concrete (RC) structures, particularly for multi-bay, multi-storied RC buildings, since it is more earthquake-resistant than Force-based design. The idea behind capacity-based design is to limit the formation of plastic hinges in the beams only. As a result, the collapse of the structure happens through the beam mechanism only. This localizes the failure, which results in less property damage and fewer fatalities.
- Collapse due to sway mechanism can result in the sudden failure of an entire storey or the entire structure. The column sway mechanism may be eliminated using the CBD technique, which involves making the columns stronger than the beams.
- The shear capacity of elements is increased in the capacity based design process, which avoids the likelihood of shear failure mode (which is brittle by nature and so failure happens rapidly). This is accomplished by raising the moment capacity of components.
- When compared to the force-based design technique for earthquake-resistant buildings, this method is a bit more expensive, but it is more successful in withstanding the forces that are generated by an earthquake.
- Because the calculations are based on the given reinforcement and the over strength of the structure, which takes into consideration the reserve strength beyond the elastic limit, this technique of design is more realistic.
- Because the structure may be repurposed after sustaining just little damage in the event of an earthquake, this strategy for building construction need to be used for public utility structures like schools, universities, and hospitals, amongst others.
- By using this approach of strong column weak beam, it is possible to avoid the risk of an unexpected collapse in the structure.
- The findings of this research indicate that the technique of designing based on capacity, as opposed to designing based on force, is highly conservative.

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