Slab Contribution for Over-strength factor in Capacity-based Design

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Abstract— **Capacity design principles are adopted in earthquake resistant design of framed buildings by which the beams are allowed to yield whereas the columns are designed to remain elastic during an event of seismic occurrence. A pre-determined desirable failure mechanism can be achieved by designing the columns stronger than the beam and by preventing the occurrence of brittle shear failure anywhere in the structure. Thus, the designated yielding members (beams) alone need to be designed as ductile. Although the concept seems to be simple, various factors need to be considered before finalizing the design capacities. Many buildings withstood earthquakes of magnitudes several times higher than the design force. This can be attributed to the presence of reserve strength in structures which were not accounted for in the design. It was observed that floor slab also contributes to the lateral load resistance even though it is not usually considered in the structural design. The present paper discusses the various sources of over-strength in framed buildings and also quantifies the role of floor slabs in lateral load resistance by suggesting the magnification required for the over-strength factor.**

Keywords—capacity design; over-strength; yield; ductility; plastic hinges

I. INTRODUCTION

In capacity-based design, the structures are designed in such a way that plastic hinges can form only in predetermined positions and in predetermined sequences. The concept of this method is to avoid brittle mode of failure. This is commonly achieved by designing the brittle modes to have higher failure load than the ductile modes.

Capacity design principles are employed in structural design codes to ensure ductile response and energy dissipation in seismic resisting systems. In the event of an earthquake, the so called "deformation-controlled" components are expected to yield and sustain large inelastic deformations such that they can absorb the earthquake´s energy and soften the response of the structure. To ensure that this desired behavior is achieved, the required design strength of other components (capacity-designed components) within the structure is to exceed the strength capacity of the deformation-controlled components [1].

While the basic concept of capacity design is straightforward, its implementation requires consideration of many factors related to the variability in component strengths, overall inelastic system response, seismic hazard and tolerable probability of system collapse.

II. SOURCES OF OVERSTRENGTH

A critical examination of the factors that contribute to the reserve strength is necessary to use appropriate over-strength factor [2]. The important among them are,

- The difference between the actual strength of the material used in construction and that used in calculating capacity
- Effect of using discrete member sizes in steel structures and the use of limited bar sizes and arrangement in concrete structures
- Effect of non-structural elements such as infill walls
- Effect of structural elements that are not included in the prediction of capacity like contribution of reinforced concrete slabs, contribution of columns in flat plate structures with shear wall, increased resistance due to concrete confinement and reduced stiffness due to concrete cracking.

The factors contributing to over-strength are not always favourable. Flexural over-strength in the beams of moment-resisting frames may cause storey collapse mechanisms or brittle shear failure in beams. Nonstructural elements also may cause shear failure in columns or soft storey failure [3]. Moreover, the over-strength factor varies widely according to the period of the structure, the design intensity level, the structural system and the ductility level assumed in the design. This compounds the difficulties associated with evaluating this factor [4].

III. CAPACITY BASED DESIGN

The procedure starts with the estimation of design forces (bending moment (BM) and shear force (SF)) in all components. This is usually done by performing a linear static analysis using any standard method for all load combinations given in IS 456: 2000 [5] or IS 1893: 2002[6]. Beams (designated yielding members) are designed for the forces obtained from this linear analysis. Design bending moments are multiplied by the over-strength factor to estimate their capacity. This enhanced moment is carried over to the columns meeting at the joint to get the design forces for columns. This is called capacity design, which aims to protect the structure from the development of unwanted inelastic mechanisms.

Special measures are required to prevent unintended plastic hinges at locations where adequate detailing for ductility has not been provided. It should also be ensured that inelastic shear displacements which are accompanied by rapid strength degradation do not occur [7]. Required strength at these locations, for actions other than flexure is found from capacity design considerations. Basic strengths S_E corresponding to the first mode force distribution are amplified by an over-strength factor φ^o to account for maximum feasible flexural over capacity at the plastic hinge locations, and by a dynamic amplification factor ω to represent the potential increase in design actions due to higher mode effects. The relationship between design strength S_D and basic strength S_E is thus

$$\phi_s S_D = S_R = \phi^o \omega S_E \quad (1)$$

where S_R is the required dependable strength of design action S_E, and φ_s is the corresponding strength reduction factor. A value of $\varphi_s = 1$ should be adopted for flexural design of plastic hinges. $\varphi_s < 1$ is appropriate for other actions and locations.

1V. CAPACITY DESIGN OF AN EXAMPLE FRAME

A 4-storeyed commercial building located in Zone V as per IS 1893:2002 is selected for the study. It is assumed to have a storey height of 3.3m and two bays of width 6m in both the plan directions. Importance factor (I) is taken as 1.0 and response reduction factor (R) is taken as 5 assuming special moment resisting frame.

The fundamental period of the building is calculated as

$$T = 0.09 \frac{h}{\sqrt{d}} \quad (2)$$

where h is the height of the building and d is the dimension of the building along the direction of lateral force. Corresponding to the estimated period T, the spectral acceleration coefficient (S_a/g) is obtained from IS 1893: 2002. The base shear calculated is distributed as per the code and is shown in fig.1.

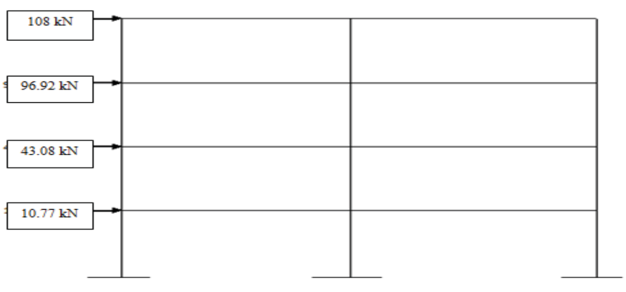

Fig. 1 Lateral Load distribution in frame

The frames are analysed for the various load combinations using SAP and the strength requirement for beams are directly obtained from the analysis. The sum of moments in columns and in beams are calculated separately and are shown in Table 1 for a typical frame.

Table 1 Column – Beam strength in joints

Joint	Seismic direction	∑M columns @joint (in kNm) (1)	∑M beams at joint with an overstrength factor 1.35 (in kNm) (2)	Check for (1) ≥ (2)	Moment magnification factor required
Exterior 4th floor	x	176.3	94.5	OK	1.00
	y	176.3	278.8	Not OK	1.58
Interior 4th floor	x	179.8	352.1	Not OK	1.96
	y	179.8	352.1	Not OK	1.96
Exterior 3rd floor	x	402.7	192	OK	1.00
	y	402.7	368.4	OK	1.00
Interior 3rd floor	x	439	532.2	Not OK	1.21
	y	439	532.2	Not OK	1.21
Exterior 2nd floor	x	458.8	302	OK	1.00
	y	458.8	473	Not OK	1.03
Interior 2nd floor	x	553	713.3	Not OK	1.29
	y	553	713.3	Not OK	1.29
Exterior 1st floor	x	560.7	292	OK	1.00
	y	560.7	464.1	OK	1.00
Interior 1st floor	x	641	655.7	Not OK	1.02
	y	641	655.7	Not OK	1.02

An over-strength factor of 1.35 is recommended in [4, 8]. If sum of column moments meeting at a joint is more than 1.35 times the sum of beam moments at that joint, the design moments are fixed as those obtained. But, in some joints, sum of column moments are found to be less than the required capacity; then the design of columns are performed such that the design column strength is at least 1.35 times the design beam strength.

IV. EVALUATION OF SLAB CONTRIBUTION IN OVER-STRENGTH FACTOR

As mentioned earlier, in-plane stiffness of floor slab is one of the causes of over-strength. To get a quantitative measure of this additional lateral resistance, three sets of frames (having 4 bays) are selected with 4-, 5-, and 6- storeys. The bay width and storey heights are respectively 6.0 m and 3.3m. They were designed for a live load of 4 kN/m^2 and the seismic base shear is calculated as per IS 1893: 2002 for zone V. Each set of frame consists of two models, i.e., with and without floor slabs. SF indicates a frame modelled with floor slab (Fig. 2) and BF indicates a bare frame without floor slab (Fig.3). The numeral indicates the total number of storeys. All the frames were modeled in SAP 2000 NL and are analysed and designed for the load combinations given in IS 1893:2002.

For the present study, the program developed by Chugh [9] was used to calculate the M-θ relation for beams and columns. It includes the effects of confinement, bond-slip and axial force, but the effect of shear is not considered. Although the axial force interaction is considered for column flexural hinges, the rotation values are calculated corresponding to gravity load alone. The stress-strain curves of concrete as per Modified Mander model [10] and that of steel as per IS 456:2000 are adopted.

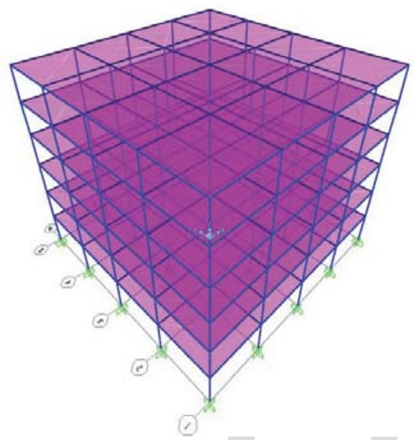

Fig. 2 Frame SF6

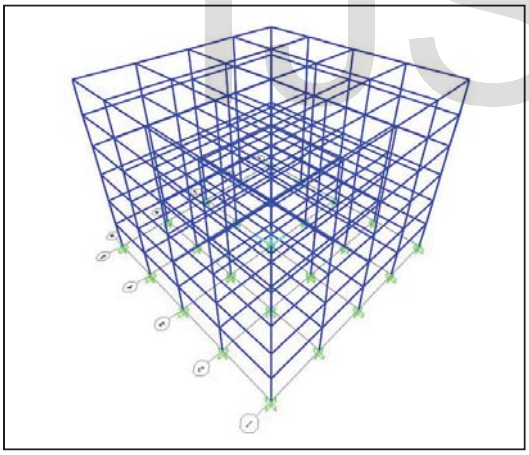

Fig.3 Frame BF6

The sectional details of beams and columns are shown in Tables 2 and 3. Pushover analyses are carried out on all the six frames. Displacement-controlled lateral loads are applied in 100 steps, until the maximum roof displacement reaches a drift of 2% of building height [11] or until the structure collapses. The applied lateral load distribution is as per IS 1893:2002. Performance levels are fixed as immediate occupancy (IO), life safety (LS) and collapse prevention (CP) corresponding to 0.1, 0.5 and 0.9 of ultimate plastic rotation respectively [11].

TABLE2. SECTIONAL DETAILS OF BEAMS

Frame	Level	Size (mm)	Top steel	Bottom steel	Stirrup
4-storeyed	Floors 1 & 2	230 x 450	4- 18 □	4-16 □	8□ @230 c/c
	Floors 3 & 4	230 x 450	2- 18 □	2-16 □	8□ @230 c/c
5-storeyed	Floor 1	230 x 450	4-20 □	4-18 □	8□ @230 c/c
	Floors 2 & 3	230 x 450	4-20 □	4-18 □	8□ @230 c/c
	Floors 4 & 5	230 x 450	2-16 □	2-14 □	8□ @230 c/c
6-storeyed	Floors 1 &2	230 x 450	4-25 □	4-22 □	8□ @230 c/c
	Floors 3,4	230 x 450	4-22 □	4-22 □	8□ @230 c/c
	Floors 5,6	230 x 450	4-16 □	4-16 □	8□ @230 c/c

TABLE 3. SECTIONAL DETAILS OF COLUMNS

Frame	Level	Size (mm)	Reinforcement	Lateral Ties
4-storeyed	Storey 1,2	300 x 500	16- 22 □	8□ @150 c/c
	Storey 3,4	300 x 500	16- 18 □	8□ @150 c/c
	Storey 1	500 x 500	16-25 □	8□ @150 c/c
	Storey 2,3	500 x 500	16-20 □	8□ @150 c/c
5-storeyed	Storey 4,5	500 x 500	16-16 □	8□ @150 c/c
	Storey 1,2	500 x 500	16-25 □	8□ @150 c/c
	Storey 3,4	500 x 500	16-22 □	8□ @150 c/c
6-storeyed	Storey 5,6	500 x 500	16-18 □	8□ @150 c/c

VI. RESULTS AND DISCUSSION

Referring to Figures 4 and 5, which show the hinge formation in various frame elements at the end of a particular load step, it can be inferred that the state of hinges (IO, LS or CP) as well as the number of hinges, are reduced when slabs are also modeled. This gives a physical interpretation for the contribution of slab in resisting lateral load.

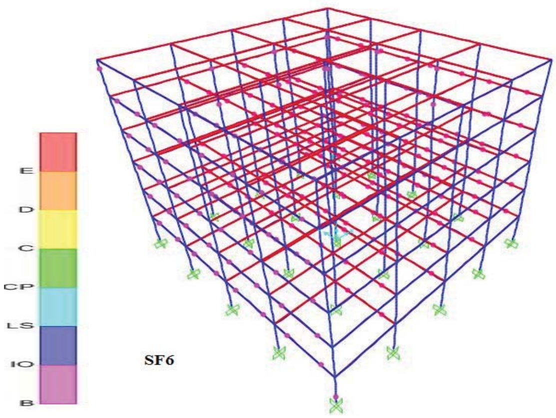

Fig. 4 Hinge Formation in SF6

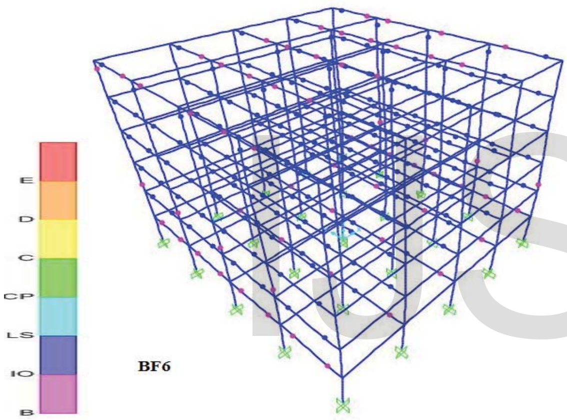

Fig. 5 Hinge Formation in BF6

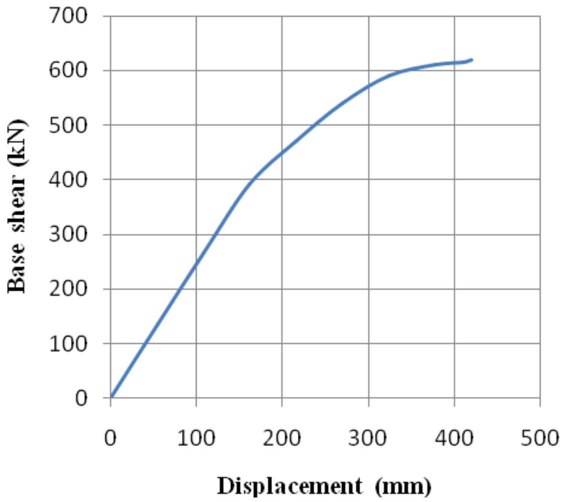

Fig. 6 Capacity curve for SF6

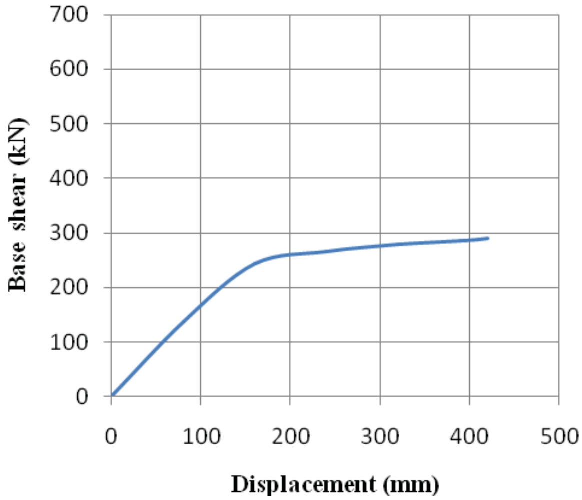

Fig. 7 Capacity curve for BF6

Figure 6 and Figure 7 show the quantitative representation of the slab contribution in resisting lateral loads. The base shear capacity of frames modeled with slab (SF) is significantly more than that of frames modeled without slab (BF). A comparison of the two values for the three sets of frames is shown in Table 4.

Slab Contribution for over-strength factor is the ratio of base shear capacity of frame modelled with slab to that of bare frame.

Table 4 Evaluation of slab contribution

Sl.No.	Frame	Base shear capacity of SF (kN)	Base shear capacity of BF (kN)	Slab Contribution For Over Strength Factor
1	4 storey	442	220	2.01
2	5 storey	590	265	2.23
3	6 storey	620	290	2.14
Average multiplication factor				2.10

This multiplication factor should be included along with the code-specified over-strength values of 1.2 to 1.35 so that column failures, by all means, can be prevented. Thus the over strength factor can come as high as 2.7. Even though it seems to be slightly expensive to provide column strength magnified by a higher value, the saving in large quantities of shear reinforcement (otherwise that have to be provided in columns if capacity design was not adopted) will compensate for it.

VII. CONCLUSIONS

Capacity based design principles are formulated based on the strong column-weak beam concept. Based on the research conducted in this area, over-strength factors ranging from 1.2 to 1.35 is generally recommended, which is the ratio of column bending moment capacity to beam bending moment capacity at a joint. Undesirable failure mechanism (column hinging) was reported to occur in a few structures even when designed with this concept. This necessitates for a re-thinking on the procedure.

The pictures of structures collapsed during earthquakes shows that floor slabs remain almost intact with very little damage. The in-plane stiffness of these slabs provides additional resistance to the horizontal structural members (beams) against lateral deformations. This needs to be accounted for while calculating over-strength factor.

A numerical study on 3-D frame models with number of storeys ranging from four to six, with and without slabs, is presented in this paper. Non-linear static analyses (pushover analysis) carried out on three regular frame models showed a magnification of 2.1 for the conventional over-strength factor.

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