General Concepts of Capacity Based Design

Sujin S. George, Valsson Varghese

Abstract- An earthquake resisting building is one that has been deliberately designed in such a way that the structure remains safe and suffers no appreciable damage during destructive earthquake. However, it has been seen that during past earthquakes many of the buildings were collapsed due to failure of vertical members. Therefore, it is necessary to provide vertical members strong so as to sustain the design earthquake without catastrophic failure. Capacity designing is aiming towards providing vertical members stronger compared to horizontal structural elements. A structure designed with capacity design concept does not develop any suitable failure mechanism or modes of inelastic deformation which cause the failure of the structures. In capacity design of earthquake resisting structures, elements of primary lateral load resisting system are chosen suitably and designed and detailed for energy dissipation under severe inelastic deformation.

Keyword- Capacity Design, Hinges, Pushover Analysis

I. INTRODUCTION

"Capacity design is a concept or method of designing flexural capacities of critical member sections of a building structure based on behavior of the structure in responding to seismic actions". 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. As far as design is concerned, a key feature is to avoid undesirable modes of failure. Capacity design procedure which sets aside the results of analysis and aims at establishing a favorable hierarchy of strength in the structures by ensuring that strength of columns is higher than that of adjacent beams, with possible allowance for beam over strength. The area of greatest uncertainty of response of capacity design structures is the level of inelastic deformations that might occur under strong ground motions.

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II. CAPACITY DESIGN OF STRUCTURES

During earthquakes many of the buildings were collapsed due to improper strength hierarchy. Many of the buildings were collapsed in Ahmedabad during "2001 Bhuj earthquake" due to improper strength.

Fig.1 Formation of Column Sway Mechanism

Fig.1 shows the photographs of the collapse of the building during Bhuj earthquake. Reference of these photographs is Nicee. However, for specific situations, the applications of capacity design concept were already implied in some codes. In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. The critical regions of these members, often termed as *plastic hinges*, are detailed for inelastic flexural action.

A.Capacity Design Concept

To highlight the simple concept of capacity design philosophy, the chain shown in Fig. 2 will be considered.

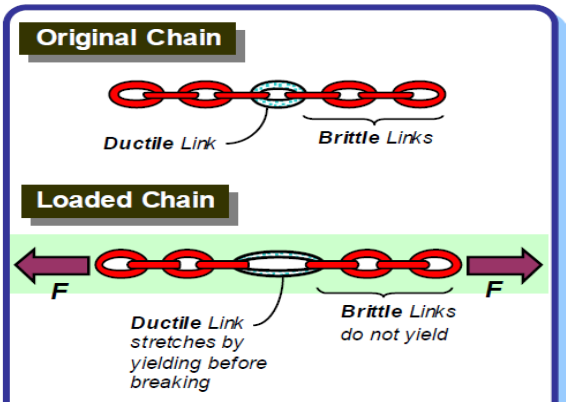
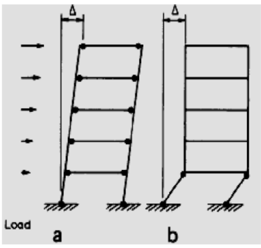


Fig. 2 Capacity Design Concept Illustrated with Ductile Chain

The chain consists of links made of brittle and ductile materials. Each of these links will fail when elongated. Now hold the last link at either end of the chain and apply a force 'F'. Since the same force F is being transferred through all the links, the force in each link is the same i.e. F. As more and more force is applied, eventually the chain will break when the weakest link in it breaks. If the ductile link is the weak one (i.e. its capacity to take loads is less), then the chain will show large final elongation. Instead, if the brittle link is the weak one, then the chain will fail suddenly and show small final elongation. Therefore if we want to have such a ductile chain, we have to make the ductile link to be the weakest link.

B. Strong Column-Weak Beam Concept

It must be recognized that even with a weak beam strong column design philosophy which seeks to dissipate seismic energy primarily in well-confined beam plastic hinges, a column plastic hinges must still form at the base of the column. In structure with strong column weak beam concept, beam yield first than column. So column sway mechanism is avoided in the structure. Example of two frames are given below. Frame of Fig. 3(a) With strong column weak beam concept and frame of fig. 3(b) Without strong column weak Beam concept.



Comparison of Energy Dissipating Mechanism with and without Strong Column - Weak Beam Concept

Fig 3

A comparison of the two example frames in Fig. 3 shows that for the same maximum displacement Δ at roof level, plastic hinges rotations θ1in case (a) are much smaller than those in case (b), θ2. Therefore the overall ductility demand, in terms of the large deflections Δ, is much more readily achieved when plastic hinges develop in all the beams instead of only in the first storey column. The column hinge mechanism, shown in Fig. 3.(b), also referred to as a soft-storey, may impose plastic hinge rotations, which even with good detailing affected regions, would be difficult to accommodate. This mechanism accounts for numerous collapses of framed buildings in recent earthquakes. In the case of the Fig. 3, frame with strong column weak beam prohibit formation of column sway mechanism and only beam sway mechanism can be developed.

A capacity design approach is likely to assure predictable and satisfactorily inelastic response under conditions for which even sophisticated dynamic analysis techniques can yield no more than crude estimates.

III. PLASTIC HINGE ZONES

Lateral load analysis systems of the structures, dissipate energy under severe imposed deformations through critical regions of the members, often termed as "plastic hinges". Location of plastic hinges in the structures is important, because plastic hinges cause excessive deformation. In plastic hinge regions, rotations of the member is very high which leads to failure. In capacity design of structures for earthquake resistance, distinct element of primary lateral force resisting systems are chosen and suitably designed and detailed for energy dissipation under several imposed deformations. So these critical regions are well detailed. In capacity design concept, potential plastic hinge regions within structure are clearly defined. These are designed to have dependable flexural strengths as close as practicable to the required strength.

Subsequently, these regions are carefully detailed to ensure that estimate ductility demands in these regions can be reliably accommodated. This is achieved primarily closed-spaced and well anchored transverse reinforcement.

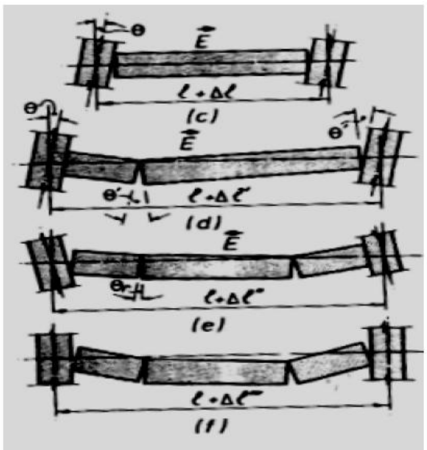
A. Types of Plastic Hinges

Location of plastic hinges in beams must be closely identified since special requirements are needed in inelastic regions of beams of frames subjected to earthquake forces. Types of plastic hinges on the base of the locations.

(1) NEGATIVE PLASTIC HINGES
(2) POSITIVE PLASTIC HINGES

(1) Negative Plastic Hinge

Plastic hinges in the beams of frames, the design which is dominated by seismic actions, commonly developed immediately adjacent to the side of column arecalled negative plastic hinge. Negative plastic hinges are formed in short span of beam. This hinges are formed adjacent to the face of the column or the maximum negative moment regions. As shown in fig.4 the plastic hinges formed at the adjacent side of the column, rotation of the beams at the plastic hinges is θ and due to this rotation length is increased Δ l

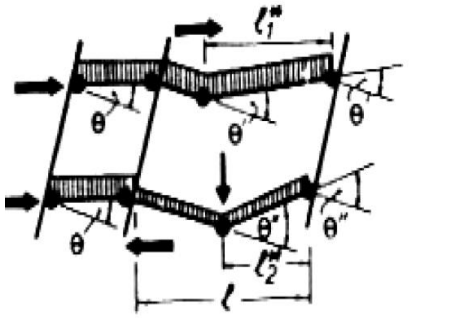


Beam Potential Negative Plastic Hinge Pattern

Fig.4

(2) Positive Plastic Hinge

Plastic hinges in the beams of the frame, developed in the maximum positive moment regions are called positive plastic hinges zones. Positive plastic hinges generally develop in the long span beams, in which gravity load is dominating. Fig.5 shows that, the rotations of the negative plastic hinges at the end or adjacent to the columns is θ . Positive plastic hinges formed at the distance l* from right end of beam. Therefore rotation of the positive plastic hinges is, θ ' = (ll *)θ as shown in Fig.5.



Beam Potential Positive Plastic Hinge Pattern

Fig.5

IV. PUSH OVER ANALYSIS

In the In the pushover analysis, the structure is represented by a 2-D or 3-D analytical model. The structure is subjected to a lateral load that represents approximately the relative inertia forces generated at locations of substantial masses such as floor levels. The static load pattern is increased in steps and the lateral load-roof displacement response of the structure is determined until a specific target displacement level or collapse is reached.

The internal forces and deformations computed at the target displacement levels are estimates of strength and deformation capacities which are to be compared with the expected performance objectives and demands. The sequence of component cracking, yielding and failure as well as the history of deformation of the structure can be traced as the lateral loads (or displacements) are monotonically increased. A typical lateral load-roof displacement performance relationship for a structure obtained from the pushover analysis is shown in Fig.6.

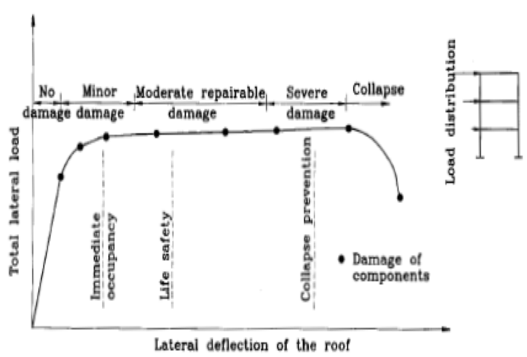


Fig.6 Typical performance curve from pushover analysis

A.Performance levels of elements

An idealized Load - Deformation curve is a piece wise linear curve defined by five points given below.

1. Point “A” corresponds to unloaded condition
2. Point “B” corresponds to the onset of yielding
3. Point “C” corresponds to the ultimate strength
4. Point “D” corresponds to the residual strength. Residual strength can be assumed to be 20% of the yield strength.

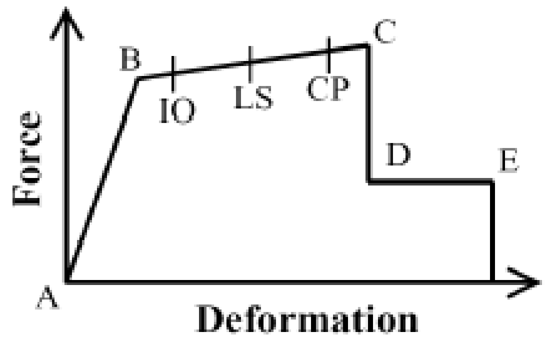


Fig.7 Load Deformation Curve

5. Point “E” corresponds to the maximum deformation capacity with the residual strength. To maintain computational stability, a high value of deformation capacity equal to 15Δy can be assumed.

A building performance level is a combination of the performance levels of the structure and non-structural components. The performance levels are in discrete damaged states identified from a continuous spectrum of possible damaged states.

The structural performance states are as follows:

1. Immediate Occupancy (IO) : Transient drift is 1% with negligible permanent drift

2. Life Safety (LS) : Transient drift is 2% with 1% permanent drift

3. Collapse Prevention (CP) :4% inelastic drift, transient or permanent

In the absence of test data, following values may be adopted.

1. Immediate Occupancy (IO) : 0.2Δ from point B
2. Life Safety (LS): 0.5Δ from point B
3. Collapse Prevention (CP):0.9Δ from point B

Here Δ is the plastic plateau.

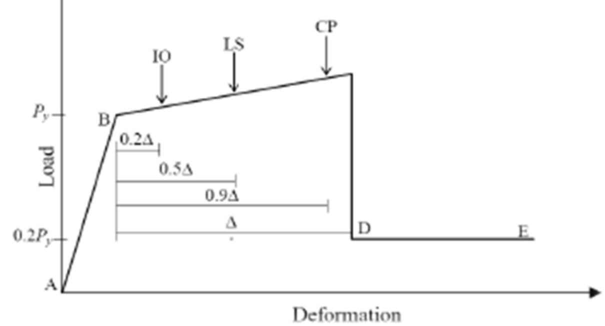


Fig.8 Load Deformation Curve indicating performance states

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B.Lateral Load Distribution

Currently, two types of load distribution are used. They are fixed load distribution and variable load distribution.

(1) Fixed load distribution

In the fixed load distribution, the distribution is determined prior and remains unchanged during the pushover. Some of the fixed distributions used are as follows:

- A single concentrated horizontal force at the top
- Uniform load distribution on all floors
- Triangular or standard code load distribution
- A load distribution proportional to the product of the mass vector and the fundamental mode shape
- Lateral force distribution based on a linear elastic dynamic analysis or response spectrum analysis of the building

(2) Variable load distribution

The load distribution changes as the building is deformed to larger and larger displacements. The following are some of the variable load distributions.

- A distribution proportional to the product of the mass vector and the fundamental mode shape is used initially until first yielding takes place. Then, for each load increment beyond yielding, the forces are adjusted to be consistent with the deflected shape in the inelastic state. The load distribution is based on the product of the current floor displacements and masses.
- A distribution based on mode shapes derived from secant stiffness's at each load step.
- A distribution proportional to storey shear resistances at each step

C.Determination of Target Displacement

1. Single degree of freedom (SDOF) approach:

Traditionally, the target displacement is determined based on the seismic response of an equivalent SDOF system due to the assumption that the building will respond in a single mode. The load-deflection curve of the resulting SDOF system takes the form of the capacity curve of the building.

2. Capacity spectrum approach:

The base shear and roof displacement values at each point on the capacity curve are transformed into spectral acceleration and spectral displacement values to obtain the capacity spectrum. A reduced response spectrum is created by adjusting the response spectrum to a damping value appropriate to the level of anticipated deformation. The capacity spectrum is superimposed on the reduced response spectrum curve of the design level earthquake. The intersection of these twocurves gives an estimate of the target displacement.

3. Elastic dynamic analysis approach:

This approach is based on the equal seismic displacement principle between elastic and inelastic systems. The analysis may take the form of time-history analysis or response spectrum analysis, depending on the form of the input ground motion information available.

D.Purpose of Pushover Analysis

Response characteristics that can be obtained by Pushover Analysis include; Estimates of the deformation demands on elements that have to deform inelastic ally, in order to dissipate energy.

1. Identification of the critical regions, where the inelastic deformations are expected to be high.
2. Consequences of strength deterioration of particular elements on the overall structural stability.
3. Identification of the strength irregularities in plan or elevation that causes changes in the dynamic characteristics in the inelastic range.
4. Estimates of inter-storey drifts, accounting for strength and stiffness discontinuities. In this way, damage on nonstructural elements can be controlled
5. Sequence of members yielding and failure and the progress of the overall capacity curve of the structure.
6. Verification of the adequacy of the load path, considering all the elements of the system, both structural and nonstructural.
7. To provide approximate evaluation of deformation demands in critical elements.
8. Expose undesirable strength and stiffness discontinuities in structure.
9. Expose potentially brittle elements
10. Expose regions of large deformation demands requiring proper detailing.
11. Assess stability of structural system.

V. CONCLUSIONS

The performance of reinforced concrete frames was investigated using the push over analysis. These are the conclusions drawn from the analysis:

- The pushover analysis is a relatively simple way to explore the non-linear behavior of buildings

- The behavior of properly detailed reinforced concrete frame building is adequate asindicated by the intersection of the demand and capacity curves and the distribution ofhinges in the beams and the columns.

- The causes of failure of reinforced concrete during the Bhuj earthquake may beattributed to the quality of the materials used and also to the fact that most of buildings constructed in that region are of strong beam and weak column type and not to the intrinsic behavior of framed structures.

- It would be desirable to study more cases before reaching definite conclusions about the behavior of reinforced concrete frame buildings.

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