Challenges in earthquake engineering

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It is often believed that providing adequate strength to be the only means of assuring satisfactory performance of structures. For example buildings are designed to resist wind forces by providing adequate stiffness. For large span prestressed balance cantilever bridge excessive creep deflections can be minimized by balancing a much greater proportion of dead load than what would be dictated by strength considerations.

But seismic resistant design is much more complex. Every seismic event has provided a new learning experience for the engineering fraternity. Every major earthquake has made engineers rethink and introduce new design guidelines. The problem is further complicated by the fact that every seismic event shows different types of input ground motions. Seismic design unlike other types of loading includes a wide variation in demands of structural performance.

It is well known that static seismic shears specified in the codes of practice are based on observations of performance of structures during earthquake. It has been recognised that assumption of linear elastic behaviour of structures results in very large seismic shears. There is considerable energy dissipation due to elasto-plastic behaviour of structures. For common residential or commercial buildings, it would be uneconomical to design the structures in such a way that during a major earthquake the structure remains in elastic range.

A prime consideration for earthquake resistant design is to create structures that are capable of deforming in a ductile manner when subjected to several cycles of loading well into inelastic range. Neglecting participation of non structural elements such as cladding and partition walls is a conservative approach for wind loads. But this is not so for seismic design because of unwelcome increase in the stiffness of structures. Performance criteria for power plants, nuclear reactors, hospital buildings and other public buildings which have to remain serviceable even after a seismic event, have to be radically different from that of residential, commercial and industrial buildings.

Although equivalent forces have been used for a variety of loads (vehicular and wind), the actual loads that occur do not exceed the adopted equivalent loads by a large margin as in the case of seismic loads.

Ductility of reinforced concrete structures

As we all know the current design practices for earthquake resistant designs though strength based are in practice governed by the ability of the structures to deform. The reduction factors recommended in the codes are dependent on the ductility of the structural system

One of the most important aspects of designing of reinforced concrete structures is to provide ductility in the structural system. Initial yielding of reinforcing steel does not mean the ultimate structural strength of the cross-section is reached. As the moment increases beyond the yield moment there is a redistribution of compressive stresses in concrete, and the additional reserve strength is due to slight increase in the lever arm. Final failure occurs when the concrete strain reaches 0.35%.

When members are proportioned on the basis of balanced design, simultaneous yielding of reinforcement and crushing of concrete occurs with very little ductility leading to sudden collapse. This can be avoided by limiting the amount of tensile reinforcement leading to the use of under reinforced sections. For sound earthquake resistance and to withstand earthquake induced oscillations, ductility has to be improved. This can be done in the following ways:

- Limiting the tensile reinforcement
- Increasing the compressive reinforcement
- Increasing concrete strength
- Limiting the yield stress in reinforcement
- Increasing extreme fibre strain in concrete by providing confining reinforcement

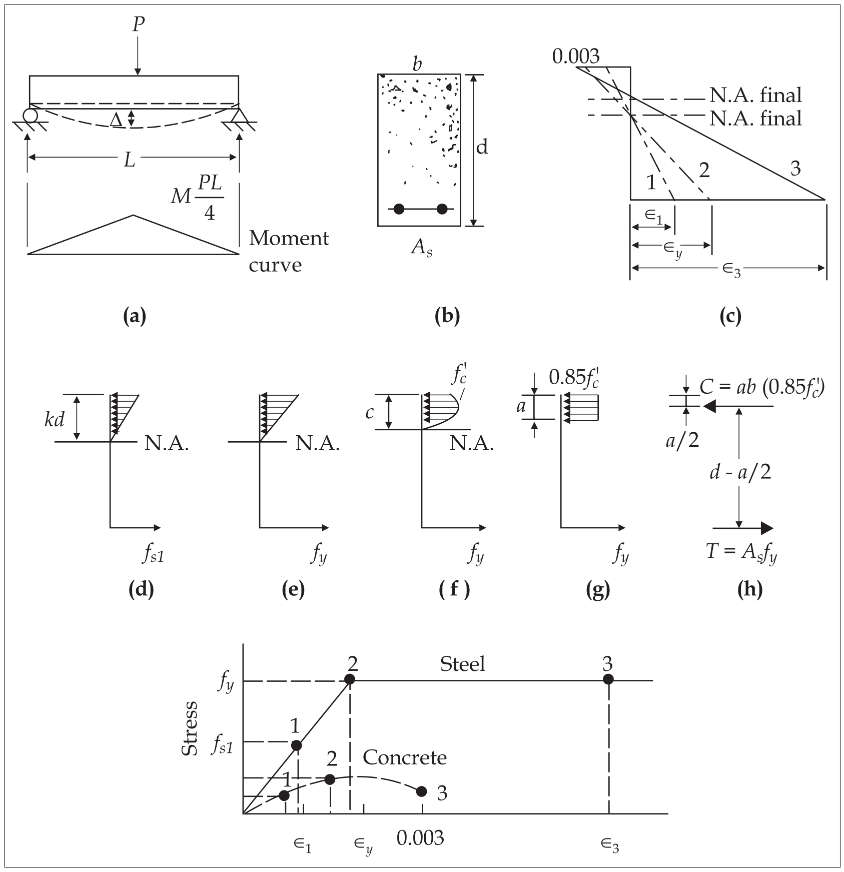

Figure 1. Changes in stress distribution as failure approaches

The effect of confining concrete with reinforcement is two-fold. It increases the strength of concrete and also the ultimate strain at the same time, Figure 2.

It must however, be noted that the concrete outside the transverse reinforcement is not confined. This cover concrete can be expected to have stress strain characteristics different from concrete confined by transverse steel. In such situations cover concrete, generally commences to spall on reaching the unconfined strength, particularly if the transverse steel reinforcement is high. The contribution of cover concrete should be ignored at high strains. But if transverse steel reinforcement content is low the cover concrete will not spall easily, and will continue to participate with the confining concrete.

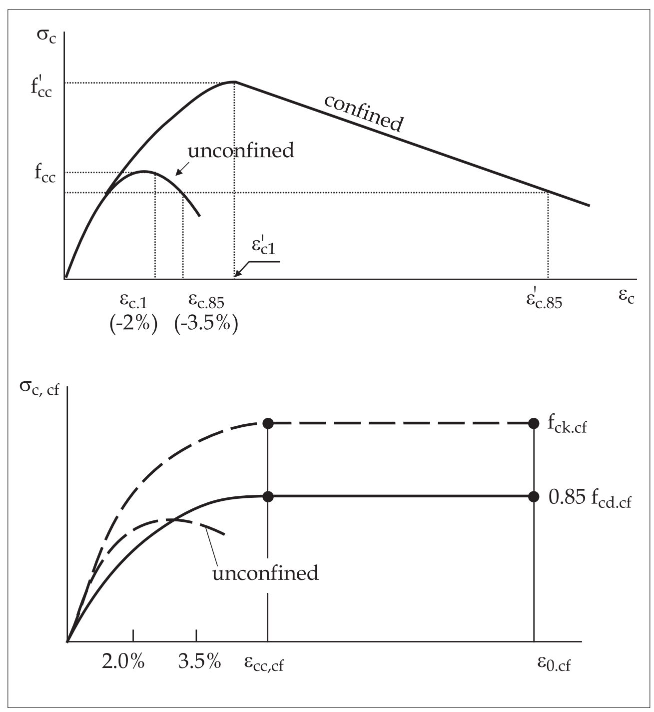

Figure 2. Effect of confining concrete with reinforcement

We must also remember that ductilities obtained for different types of displacement parameters are not

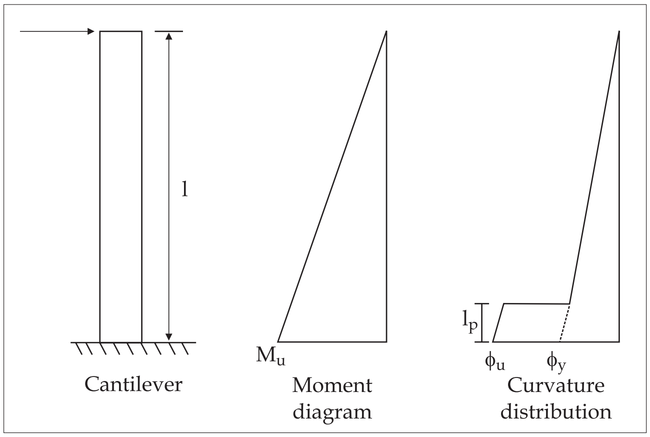

Figure 3. Cantilever deflection at ultimate condition

quite comparable. In the order of decreasing ductility we have:

- strain ductility (σ ~ ε)
- curvature ductility (M ~ Ø)
- rotation ductility (M~θ)
- displacement ductility (P ~ δ)

Displacement ductility

It must be clearly recognised that there is a considerable difference between displacement ductility factor and rotation ductility factor.

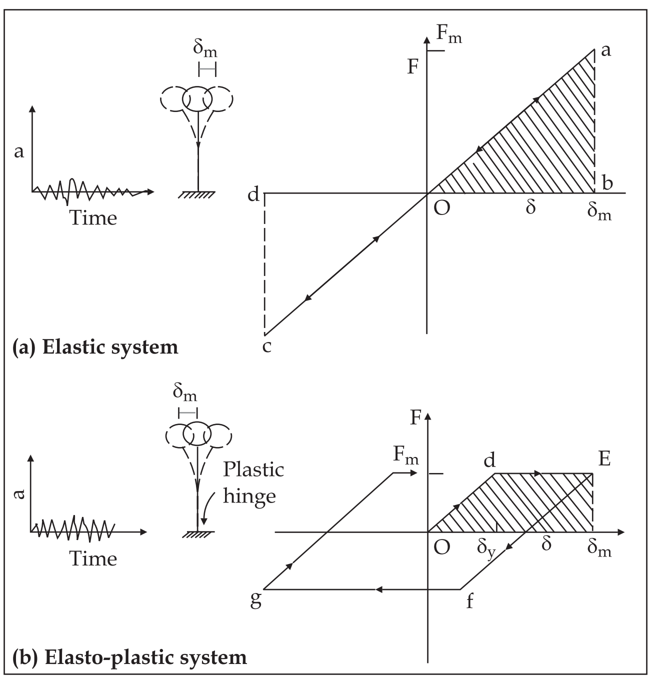

Figure 4. Energy dissipation with elasto-plastic behaviour

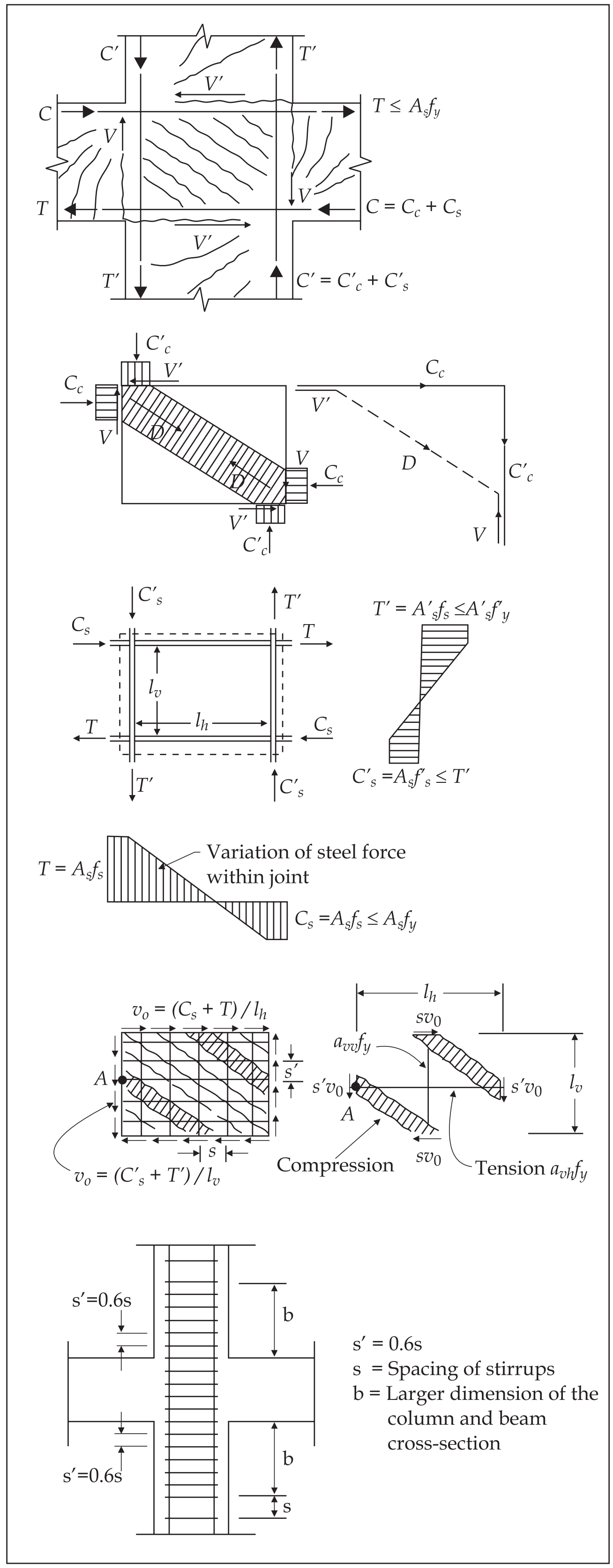

Figure 5. Beam column junction behaviour

The displacement of the free end of the cantilever clearly depends on the length of the plastic hinge which varies from 0.5 to 1 times the depth of the member

Energy dissipation

In the elastic systems there is some energy dissipation because of damping. But in inelastic systems there is considerable energy dissipation which reduces the strength demand for seismic design as shown schematically in Figure 4.

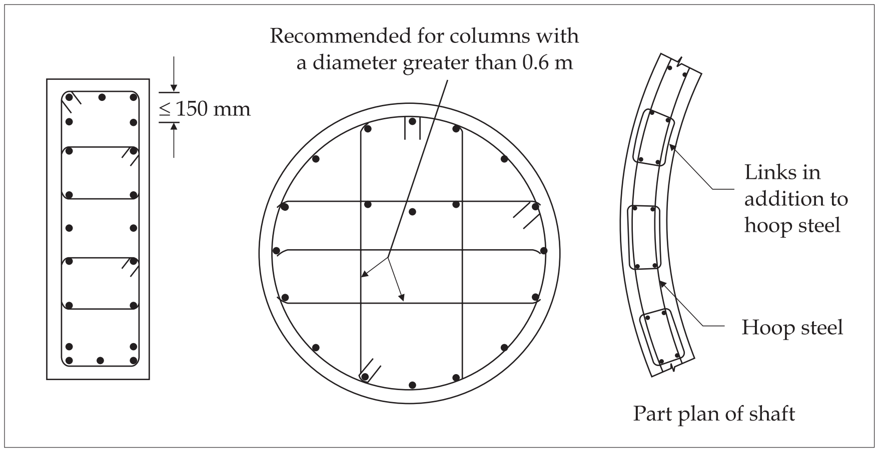

Figure 6. Some aspects of detailing

Detailing

For non seismic loads the bending moments required to be transferred from beams to columns rarely exceed the design values. Joint performance is not a very critical factor. Under seismic loads the moments required to be transferred to column are close to the deign values or may even exceed them. A good detailing practice is therefore of paramount importance. Figure 5 shows strut and tie modelling for understanding joint behaviour.

Another important aspect is requirement of column links within the beam depth which is required to prevent reinforcing steel from buckling. These are often omitted at site because of practical difficulties in terms of interference with beam reinforcement. These problems can be easily overcome by providing U shaped stirrups introduced after placing beam reinforcement

Stocky cylindrical shafts supporting water tanks are common in our country. They have low ductility because of the presence of substantially high axial loads. The transverse steel on inner face does not provide any restraint against buckling of vertical reinforcement. Both ductility and restraint against buckling of vertical reinforcement on inner face can be improved by providing additional closed stirrups.

Displacement based design

In the emerging trends, displacement related quantities are going to be used to judge performance acceptability. For example, specifications of the drift limit under seismic loading is one such measure. For framed structures, where shear deformation is predominant inter storey drift is a governing parameter. But, when

As regards provision of links in columns, IS 456 provisions lead to severe congestion for heavily reinforced columns. European and American practices are much better in this regard as can be seen in Figure 6.

For circular columns the size and spacing of circular rings recommended in codes of practice is independent of the column size. For large diameter columns (especially bridge piers) the hoop forces and associated deformations are considerably larger. FIP recommendations for circular columns larger than 0.60 m in diameter call for additional rectangular links, Figure 6.

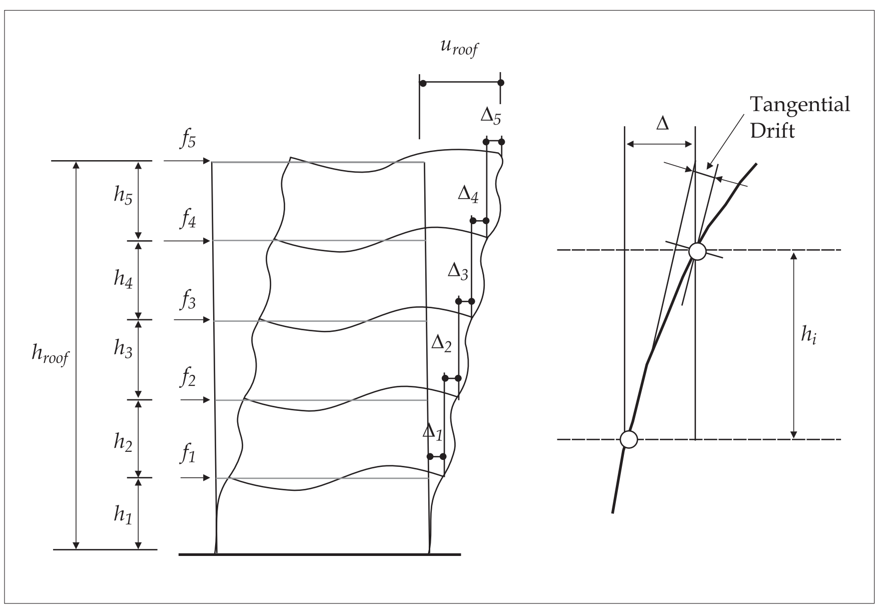

Figure 7. Controlling displacement

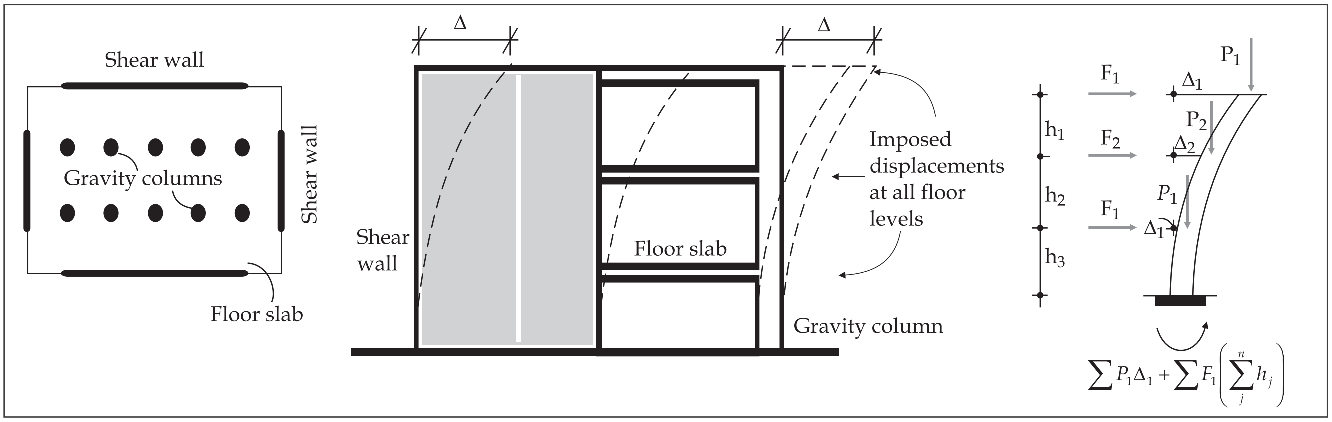

Figure 8. Flat slab building with shear walls

flexural deformations are important the drift is be measured with respect to the tangent of deflection diagram at the bottom storey. Controlling displacement could be one of the following:

- Global displacement (SDOF)
- Roof displacement
- Inter-storey drift
- Drift ratio
- Average drift

Calculation of inter-storey drift is straightforward inelastic regime. But during strong motion earthquakes the structures undergo inelastic deformations and it becomes necessary to appreciate non linear behaviour of the structures. Inelastic behaviour results in large displacement, permitting considerable energy dissipation, thereby reducing the seismic shear forces and required design strengths.

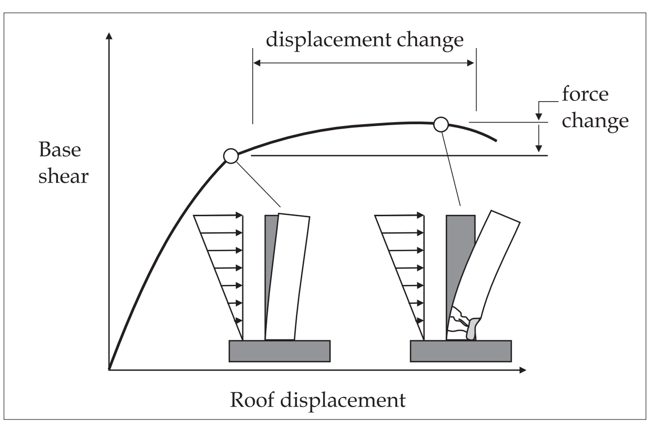

Figure 9. Displacement changes in slender shear walls

Importance of displacement based design can be readily appreciated when dealing with structures where some of the columns do not contribute to seismic resistance. For instance in a flat slab building, traditionally the columns are designed only for gravity loads as they do not resist any seismic shears. Yet for reasons of compatibility they too suffer large displacements during a major seismic event. It is important that these should be so proportioned that they do not loose the vertical load carrying capacity due to imposed displacements during a strong motion earthquake.

FIB Bulletin No. 25 – Displacement-based seismic design of reinforced concrete buildings

This an excellent document on displacement-based design for seismic resistance of reinforced concrete buildings. The following pages present an overall view of the methodologies considered in this bulletin.

Importance of displacement based design

For reinforced concrete walls, cracking and yielding occurs at relatively low displacements. Structural change occurs through formation of flexural plastic hinges. The extent of damage is related to the amount of plastic deformation in plastic hinge, which in turn, can be related to the displacement. Forces acting on the wall are of much lesser measure as these do not change significantly with increasing displacement.

When we have systems, composed of dissimilar elements such as shear walls and frames as in the case of a weak brittle frame retrofitted by a strong ductile wall, only a displacement based design will focus our attention to the problem of possible loss of vertical load carrying capacity of the weak frame (which can happen say if such structure is retrofitted by a ductile frame)

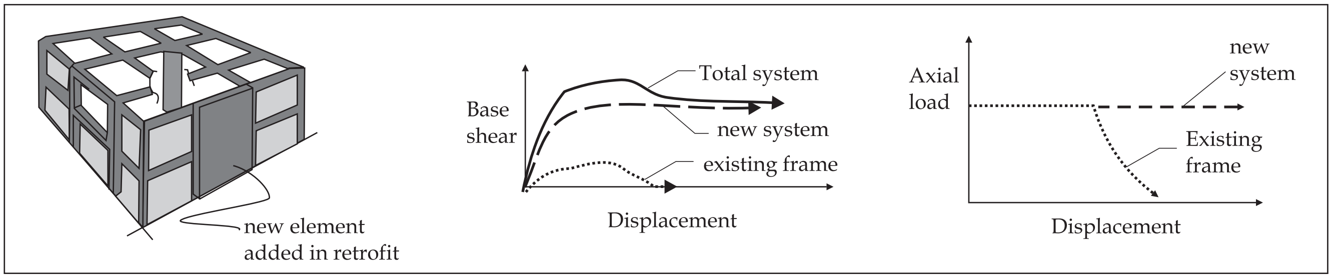

Figure 10. Dissimilar elements for retrofitting

Only displacement, based methods will adequately address the needs of structural protection. The displacement limits on structural systems will provide improved protection for non structural components.

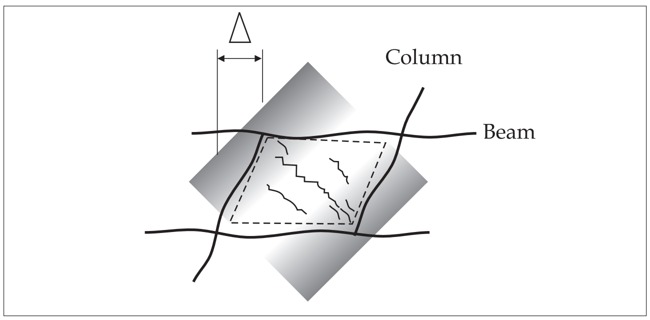

Figure 11. Deformations of non-structural components

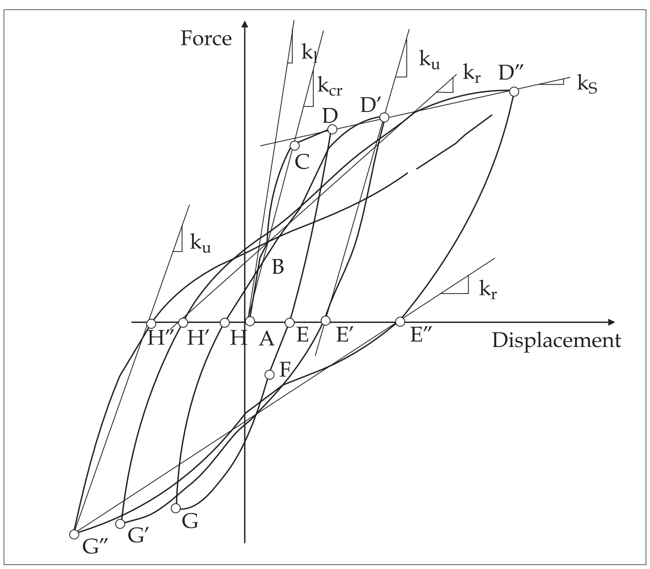

Figure 12. Non-linear behaviour of structures

Non-linear behaviour of reinforced concrete structures.

Typical non-linear behaviour of reinforced concrete structures is illustrated in Figure 12 . There is a steady reduction in stiffness of reinforced concrete members first on cracking and then on yielding of reinforcement. On unloading there is a residual displacement.

The model that is used in the analysis is a simplified version shown in Figure 11. Displacement imposed corresponds to a linear elastic system until the yield displacement u_y is reached when the yield force is F_y. Thereafter, the force remains constant till the reversal of displacement occurs at u_m This is the point of zero velocity, and if the reversal of displacement does not occur before the ultimate displacement capacity u_u is reached, there will be a failure of the structure.

Displacement ductility demand is the maximum excursion of the structure in inelastic range which depends on the mechanical properties of the system and is related to a particular ground motion. Displacement ductility capacity on the other hand is the maximum displacement the element can sustain which depends on the mechanical properties of the system.

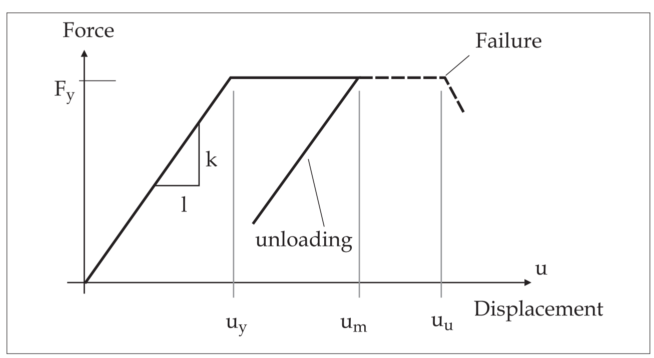

Figure 13. Simplified force displacement relationship

Estimating displacement

Single degree of freedom (SDOF)

The positive side of the response of SDOF system assuming elastic and inelastic response is shown Figure 14.

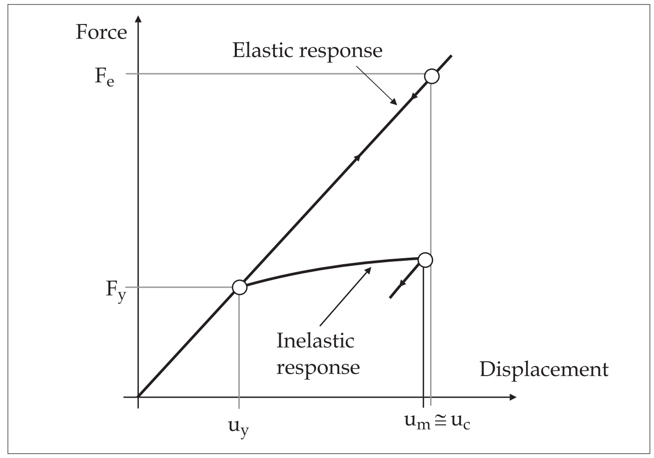

Figure 14. Elastic and inelastic response of SDOF

Muto (1960) concluded that the displacement demands for both the systems are more or less same. This also known as 'equal displacement rule'. Ductility reduction factors are but a simplified reflection of this rule. Newmark and Hall (1982) demonstrated that this rule was valid only for long natural periods. For intermediate range, energy was found to be equal for both the systems.

Newmark's conclusions are:

- For long periods, total displacements for any ductility demands are essentially same (equal displacement rule)
- For short periods systems, the maximum acceleration for any ductility demands is same (strength demand is same)
- For intermediate range, the maximum velocity and therefore the energy absorption for elastic and inelastic is the same.

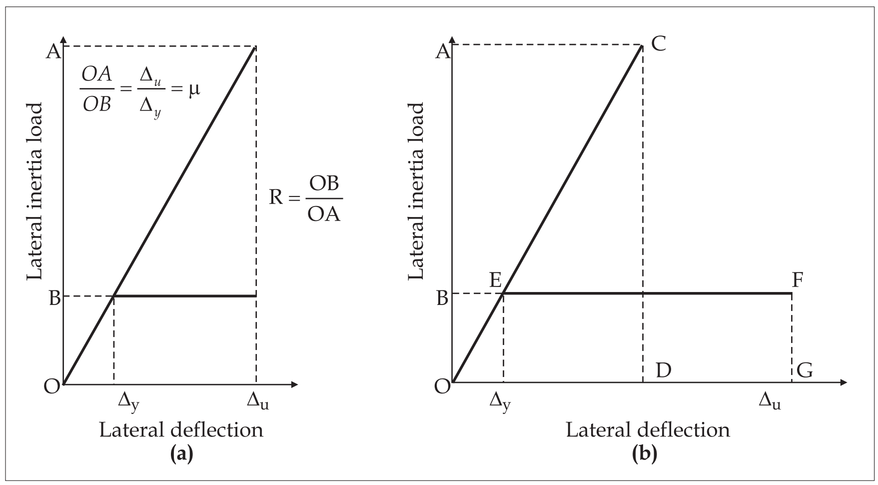

Figure 15. Equal displacement rule and equal energy rule

Tripartite diagrams developed by Newmark can not be used for elasto-plastic systems without some modifications. Two different spectra

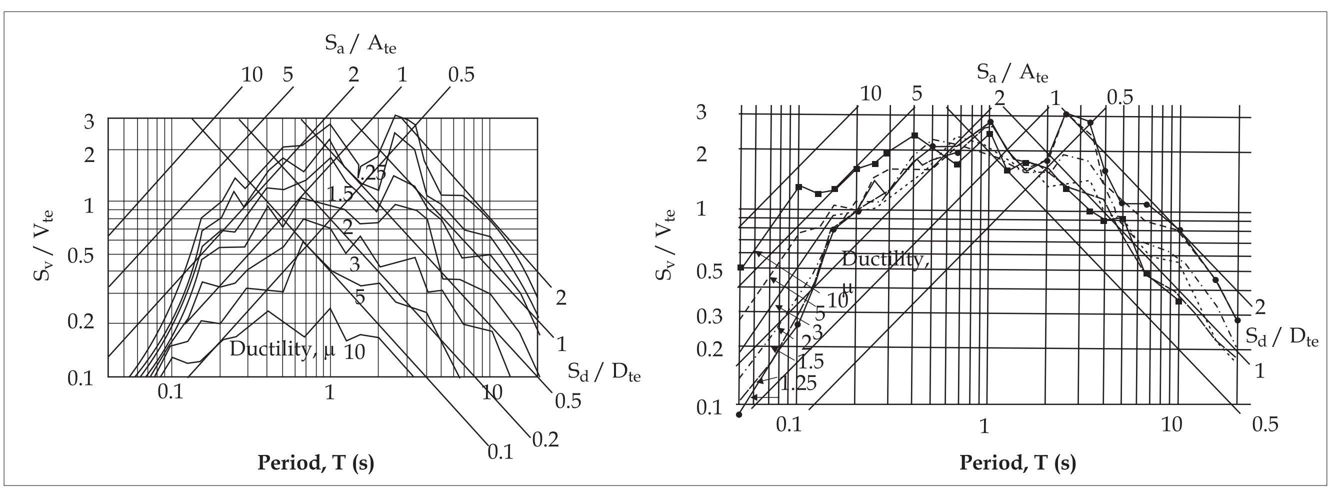

Figure 16. Maximum acceleration and displacement spectrum

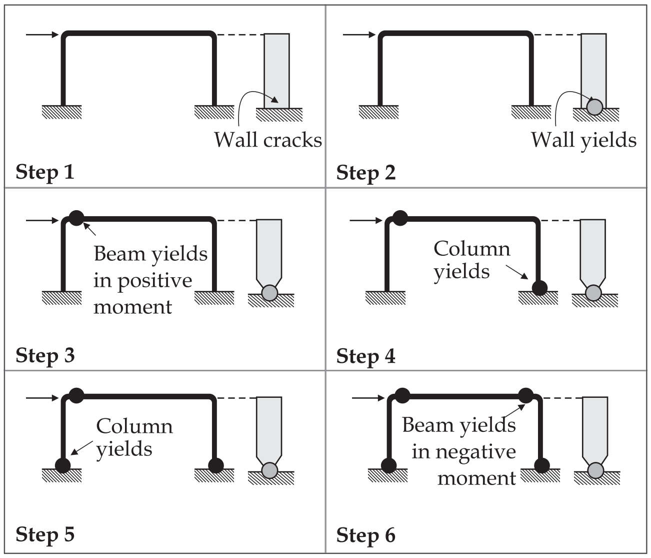

Figure 17. Pushover analysis

have to be drawn. Maximum acceleration spectrum, which gives true acceleration in which only the elastic component of displacement is obtained and total displacement spectrum, where the total displacements are shown but accelerations read in the chart have no meaning whatsoever. Figure 16 shows the maximum acceleration and displacement spectrum for two per cent, damping for various displacement ductilities.

Pushover analysis

This analysis establishes the relationship between displacement and base shear as the latter is increased gradually. The base shear distribution is based on first mode of vibration. As the shear is increased, hinges are formed and further increase in shear is resisted by the

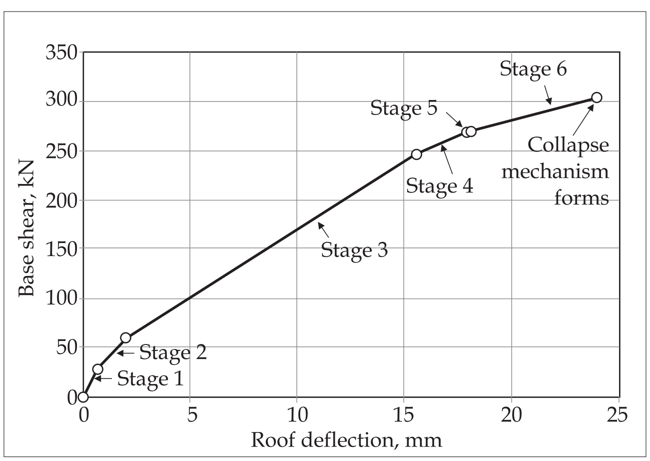

Figure 18. Displacement - shear relationship in mixed systems by pushover analysis

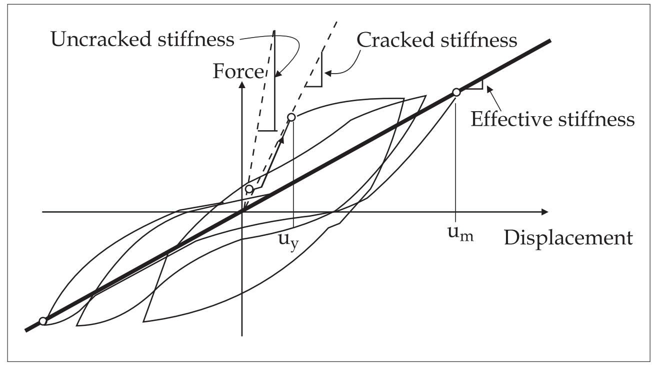

Figure 19. Secant stiffness approach

modified structural system till a collapse mechanism is reached.

This analysis provides the basis of a more accurate force displacement relationship in inelastic range especially when structures include dissimilar elements.

Secant stiffness approach (Gulkan and Sozen, 1974)

In this method stiffness used in the linear elastic analysis is replaced by effective stiffness defined as the ratio of cracked stiffness divided by the ductility. This is combined with a substitute damping which depends on the ductility of the structure.

$$\xi s = 0.2(1-1/\sqrt{\mu}) + 0.02 \qquad ......(1)$$

Capacity spectrum approach

Displacement demands on the system is determined by superimposing the acceleration of displacement spectra on capacity, push-over curve.

Effective damping depends on the reliability of hysteric behaviour.

$$\xi eff = \lambda.\xi_o+0.05 \qquad ......(2)$$

where
$\lambda = 0.3$ (unreliable hysteric behaviour)
$\lambda = 1$ for well detailed elements

Multi degree of freedom system (MDOF)

What used to be a supercomputer is now available as a desktop. We will see time history analysis of complex frames performed as a routine task. It is going to being a real challenge to still be able to make a good engineering judgement.

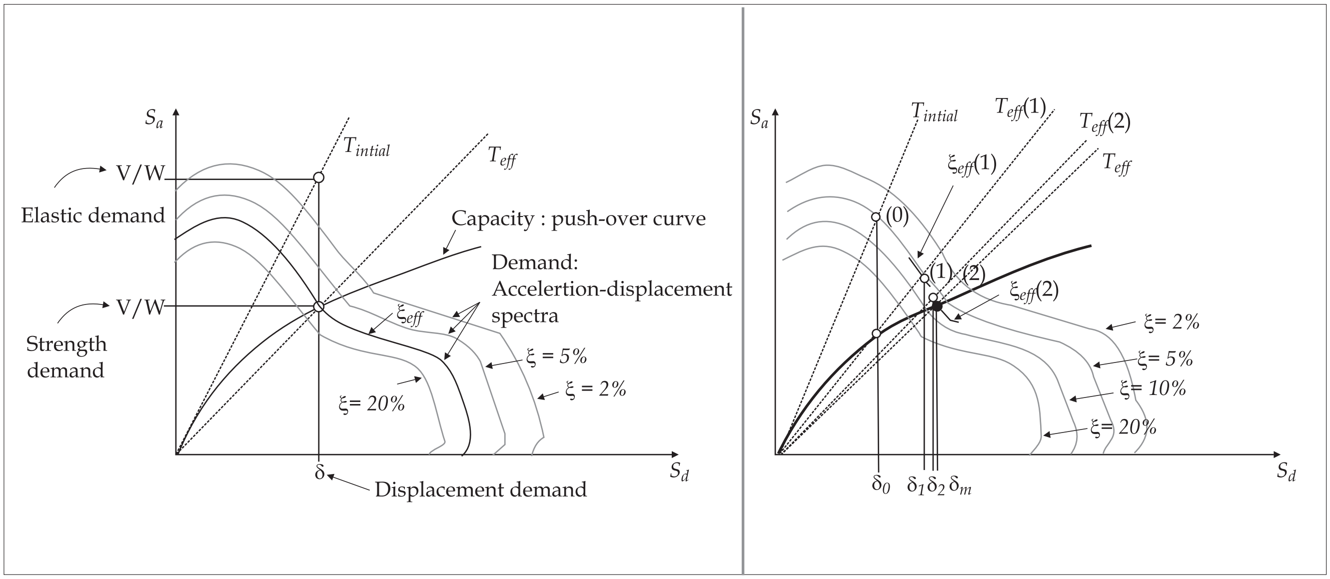

Figure 20. Capacity spectrum approach

Substitute structure

This is an extension of secant stiffness approach based on linear elastic response. The stiffness and damping properties are related to the original structure but differ due to inelastic response as follows:

(EI)i, substitute = (EI)I,actual / µi ......(3)

where
µi depends on the acceptable damage of a particular element. For instance, beams and columns would have different acceptable damage values.

Basic categories of displacement based procedures

These may be grouped into the following categories:

Deformation calculation based procedures (DCB)

In this procedure, the maximum displacement for an already designed structural system is computed. The structural system is so detailed in such a manner that displacement capacity exceeds the maximum calculated displacement. There is no change in the structural system

Iterative deformation specification based procedures (IDSB)

Unlike in DCB a limit is set to the maximum displacement which will be enforced. Changes are made in the structural system so that displacement are kept below the specified limits. This is an iterative procedure.

Direct deformation specified based procedures (DDSB)

The starting point is a pre-defined target displacement. The design of the structure then progress such that the end result is the required strength and hence the stiffness under the design level earthquake. This procedures is not iterative.

Seismic input

Spectra as input

These are averaged in elastic response normalised and averaged. Average plus standard deviation are termed as 84 % confidence level values. Newmark and Hall have provided amplification parameters for a range of damping ratios. European practice is however based on attenuation relationships which gives smooth acceleration spectra. Different spectra are provided for vertical and horizontal accelerations.

To account for the non linear behaviour of structures linear elastic response values are scaled-down by response modification factors.

Substantially larger behaviour factors are proposed in the USA than in Europe. As a result similar structures designed according to these codes are likely to suffer different levels of damage during earthquakes. It has been reported that higher behaviour factors do not necessarily lead to lighter structures because other loading conditions may take precedence.

Time history analysis

The following options are available for time history analysis:

- Natural earthquake records
- Artificial acceleration signals
- Earthquake records from mathematical simulations

Experimental database for inelastic displacements

As we all know, there could be considerable variation in calculation of displacements depending on the code of practice we adopt. We can follow a broad unified approach by examining the already existing test results available in various laboratories. Based on such studies simple relationships have been presented in FIB Bulletin No 25. Perhaps in the near future we may see such relationships being adopted in the codes of practice.

Current scenario in India

NICEE has rendered yeoman service to the civil engineering fraternity by discussing the code provisions currently in vogue and providing an excellent commentary and recommending modifications. There has been considerable improvement in understanding the problems related to earthquake resistant design.

As far as evolving of the optimal structural solutions are concerned there has been very little progress. The building shapes are governed by development control rules. One finds find all kinds of building shapes, which are strongly discouraged in seismic codes.

While computing the floor space index cantilever balconies are partly exempted because these are treated as non-dwelling areas. After the building is complete and occupied the balconies are routinely enclosed anyway thus increasing dwelling space. Cantilever balconies are often a ploy to increase the saleable area. If there are going to be no cantilever overhangs one could have developed different structural systems and perhaps provide better seismic resistance without increasing the structural costs.

The current practice is that an architectural plan is given to the structural engineer and he has somehow to manage to make the building structurally safe. Architects and builders have immense dislike for columns. Builders see expensive and decorative finishes giving them good returns but any increase in quantities of concrete and reinforcement is severely questioned by making comparisons with other buildings which are not properly designed for lateral loads. In such a scenario, how do we get strong columns and weak beams that are required for good seismic resistance?

Strategies for superior seismic performance

What we need is neat structural systems with perhaps longer spans which is possible if no distinction is made between balconies/flower beds/ and other usable areas for the purposes of calculation of FSI. Light partitions and participation of external frame will certainly improve matters. The development rules need to exclude area of load bearing columns and walls for FSI calculation.

American Concrete Institute Practice recommends the following strategy for 'strong columns weak beam' by adopting different capacity reduction factors as given below.

- 0.90 for tension controlled beams
- 0.75 for shear and torsion in beams
- 0.70 for columns with spirals
- 0.65 for other columns

There is also a requirement that moment capacity of the columns exceed the moments to be transferred from beams exceed by at leat 20 %. ($\sum Mnc \geq 1.2\sum Mnb$)

Conclusion

The progress in the field of seismic resistant design of structures has been truly extraordinary. Implementation of such latest developments into building practice poses a major challenge and many hurdles have to be overcome.

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