45 EARTHQUAKE RESISTANT DESIGN OF STEEL STRUCTURES

1.0 INTRODUCTION

Earthquakes are natural phenomena, which cause the ground to shake. The earth's interior is hot and in a molten state. As the lava comes to the surface, it cools and new land is formed. The lands so formed have to continuously keep drifting to allow new material to surface. According to the theory of plate tectonics, the entire surface of the earth can be considered to be like several plates, constantly on the move. These plates brush against each other or collide at their boundaries giving rise to earthquakes. Therefore regions close to the plate boundary are highly seismic and regions farther from the boundaries exhibit less seismicity. Earthquakes may also be caused by other actions such as underground explosions.

The Indian sub-continent, which forms part of the Indo-Australian plate, is pushing against the Eurasian plate along the Himalayan belt. Therefore, the Himalayan belt is highly seismic whereas peninsular India, which is not traversed by any plate boundary, is relatively less seismic. Earthquakes became frequent after the construction of Koyna dam and this is regarded as a classic case of man-made seismicity. However, the Latur earthquake of 1993, which occurred in what was previously considered to be the most stable region on the earth implies that no region is entirely safe from devastating earthquakes.

Earthquakes cause the ground to shake violently thereby triggering landslides, creating floods, causing the ground to heave and crack and causing large-scale destruction to life and property. The study of why and where earthquakes occur comes under geology. The study of the characteristics of the earthquake ground motion and its effects on engineered structures are the subjects of earthquake engineering. In particular, the effect of earthquakes on structures and the design of structures to withstand earthquakes with no or minimum damage is the subject of earthquake resistant structural design. The secondary effects on structures, due to floods and landslides are generally outside its scope.

The recent earthquake in Kutch, Gujarat on 26 Jan 2001 has not only exposed the weaknesses in the Indian construction industry but also the lack of knowledge about earthquake engineering among all concerned. Taking advantage of the fear caused by the earthquake in the minds of both the common people and the engineering community, a number of people who have no knowledge about earthquake engineering have made totally absurd statements with regard to earthquake resistant design. Examples are given below:

- Tall buildings (exceeding 27 metres) should NOT be built.
  (Comment: There is no scientific basis for this limitation).

- A building of U or L shape (viz. Irregular shapes) is preferable.

  (Comment: Buildings of U or L shapes are under enhanced risk of earthquake damage due to torsional mode of motions).

- Adjacent buildings should be rigidly interconnected.

  (Comment: It is inappropriate to connect adjacent buildings rigidly as re - entrant corners in such cases might undergo distress).

- Steel and Wood are ductile.

  (Comment: Wood is NOT ductile).

It will be wise to verify the credentials of the so called "experts" with a view to ascertain if they are qualified to provide advice on Earthquake resistant Design, before believing or accepting their advice.

An important thing to remember is that as of now, earthquakes cannot be predicted by any means scientific or otherwise. Claims that earthquakes can be predicted by Astrology, by Astronomical or other observations of sun-spots, by observing animals including rats and bandicoots or by observing unnatural phenomena such as sudden rise in the water table are all methods **which have no scientific basis and should be regarded as extremely risky.**

Earthquake load differs from other loads in many respects, which makes it more difficult to design for it. An important characteristic of earthquake loading is the uncertainty associated with its amplitude, duration, and frequency content. Structures are normally designed to withstand gravity loads acting vertically with adequate factor of safety. Therefore the lateral loads arising due to horizontal earthquake ground motion can cause severe damage unless special provisions are made to resist them. The third characteristic of earthquake ground motion is that it is cyclic and induces reversal of stresses. Therefore axially loaded members may have to resist both tension and compression while beam cross-sections will have to resist both positive and negative bending moments. The fourth characteristic is that the loading is dynamic and produces different degree of response in different structures. Dynamic analysis requires the consideration of inertia and elastic forces as well as energy dissipating mechanisms like damping (Clough and Penzien 1993). These characteristics make seismic analysis and design extremely difficult and time-consuming and so simplified procedures are often used in practice.

2.0 DESIGN PHILOSOPHY AND METHODOLOGY

Severe earthquakes have an extremely low probability of occurrence during the life of a structure. If a structure has to resist such earthquakes elastically, it would require an expensive lateral load resisting system, which is unwarranted. On the other hand, if the structure loses its aesthetics or functionality quite often due to minor tremors and needs repairs, it will be a very unfavourable design. Therefore, a *dual strategy*, akin to the limit state design, is adopted. The usual strategy is:

> *'to ensure elastic behaviour under a moderate earthquake which has a return period equal to the life of the structure and prevent collapse under the extreme probable earthquake'.*

For example, if the expected life of the structure is fifty years, then it is designed to remain elastic under an earthquake, which is likely to occur at least once in fifty years. Thus, no major repair will be necessary as a consequence of such earthquakes during the life of the structure. However, structures are designed to prevent collapse and loss of life under the most severe earthquake. The reason for adopting such a strategy is that it is extremely expensive to design structures to respond elastically under severe earthquakes, which may not occur during their expected life. Thus, it is well worth the risk to let them get damaged beyond repair in case the severe earthquake occurs, the chances of which are low.

The important properties of structures, which contribute to their elastic resistance under moderate earthquakes, are its yield strength and elastic stiffness. During a severe earthquake, the structure is likely to undergo inelastic deformations and has to rely on its *ductility* and *hysteretic energy dissipation capacity* to avoid collapse.

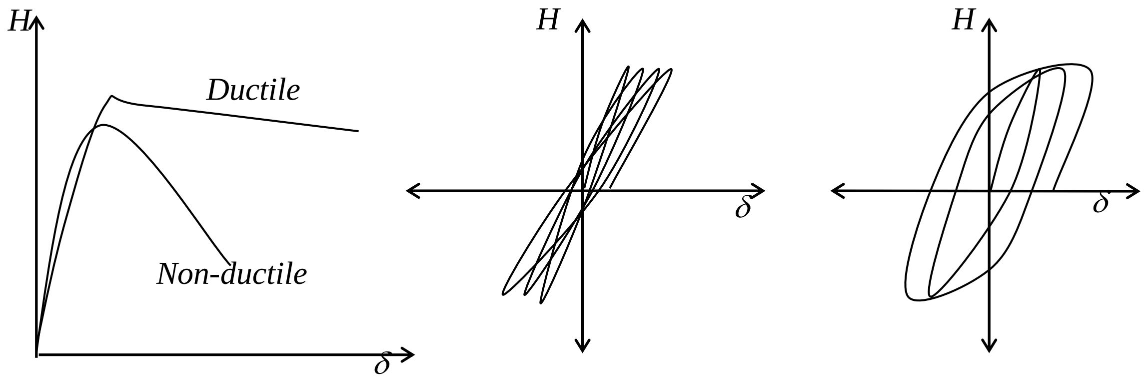
Ductility is the property, which allows the structure to undergo large plastic deformations without significant loss of strength [see Fig. 1(a)]. Ductility μ is defined as the ratio of the ultimate deformation δ_u at an assumed collapse point, to the yield deformation δ_y. It may be noted that the collapse point may be assumed to lie on the descending branch of the load-deformation curve. This is still safe because earthquake loading is transient and will cease to act after a short time and so the structure will not be toppled.

Hysteretic energy is the energy dissipated by inelastic cyclic deformations and is given by the area within the load-deformation curve also called the hysteretic curve. In structures having low hysteretic energy dissipation capacities, even if the deformations are well below the ultimate deformation, the structure is likely to collapse due to *low-cycle fatigue* effect as described below.

The degradation of strength and stiffness under repeated inelastic cycling is called *low-cycle fatigue*. Ensuring that the structure is able to dissipate a large amount of hysteretic energy in each cycle can minimise low-cycle fatigue effect. The area enclosed by the force-deformation loops gives the hysteretic energy. Larger area implies more dissipation of hysteretic energy as shown in Fig. 1(c). One way of ensuring good ductility and energy dissipation capacity in steel structures is to use thicker sections thereby avoiding early local buckling. Thus, plastic and compact sections are preferred over semi-compact and slender sections. Other parameters, which control ductility, are slenderness ratio and axial load ratio of the members.

Since earthquake loading produces large deformation as well as low-cycle fatigue, both ductility and energy dissipation capacity are required to resist severe earthquakes. Experimental studies have shown that these two capacities are interrelated and a large demand on one tends to decrease the other.

With reference to framed structures, it has been found that some collapse mechanisms ensure larger energy dissipation capacities compared to some other collapse mechanisms. The technique of ensuring a preferred collapse mechanism by suitably adjusting the capacities of the members is called *Capacity Design*. In practice, due to the difficulties associated with inelastic analysis and design, no attempt is made to calculate the actual capacities in relation to seismic demand and it is only ensured that the members and joints of the structure have adequate ductility and energy dissipation capacities and the structure as a whole will fail in a preferred collapse mechanism.



(a) ductile and non -ductile behaviour *(b) poor energy dissipation capacity* *(c) good energy dissipation capacity*

Fig. 1 Earthquake Resistant Properties

In addition to strength requirements at the ultimate load, structures are also designed to have adequate stiffness in the lateral direction under service loads. This is usually ensured, by limiting the relative displacement between successive floors, known as the *storey drift*. For buildings, a maximum allowable storey drift of *0.004* times the storey height is normally used under moderate earthquakes.

Although all of the above mentioned concepts are important for ensuring the safety of structures during a severe earthquake, one should keep in mind the great uncertainty associated with the seismic behaviour of structures. Past earthquakes have demonstrated that simplicity is the key to avoid unforeseen effects and so attention given at the planning stage itself can go a long way in ensuring safety and economy in seismic design. Some of the factors to be considered at the planning stage are described below.

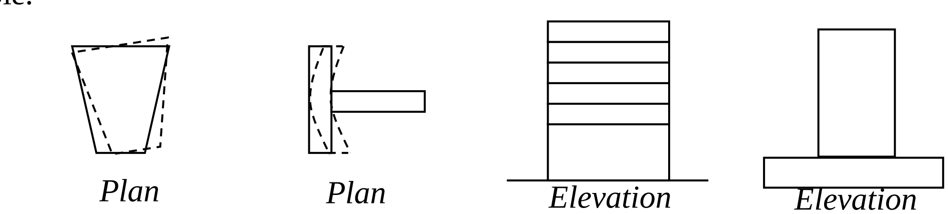
Although the selection of a suitable site and foundation is not within the scope of this chapter, some guidelines are given in view of the wrong advice given by several 'experts' in the aftermath of the Gujarat earthquake. It is common sense to select a site where the bedrock is available close to the surface so that the foundations can be laid directly on the rock. Conversely, where such a condition is not available it is a simple matter to say that the site should be avoided. However the engineering problem arises when, due to practical reasons, it is not possible to avoid a site where bedrock is not available close to the surface. For example, in the great Indo-gangetic plain or the Rann of Kutch, where will one find a suitable rock stratum for laying the foundations? It is precisely in these situations that well qualified 'experts' are required who can give guidelines about the type of foundations to be adopted.

Most Civil engineers are aware that in expansive clays one can select under-reamed piles to prevent differential settlement. A similar concept can be used for earthquake engineering wherein one chooses for raft foundations over weak soils to avoid differential settlements. If the loads are high, one should select pile foundations and provide a strong pile cap. Where spread footings are decided upon, it may be advantageous to have adequate plinth or tie beams to prevent the deleterious effects of differential settlement. **An important phenomenon, which should be guarded against is liquefaction of sands**. Sands have adequate bearing capacity under normal conditions but tend to liquefy and lose their shear strength during an earthquake especially when the water table rises. In dry or desert areas, where there is little chance for the water table to rise one need not worry about liquefaction. The other alternative is to provide sufficient drainage paths, which can dissipate the excess pore water pressure and thereby prevent liquefaction. Different types of foundations, foundations at different levels as on a hill-slope and foundations on different soil types should also be avoided.

Several other techniques such as erecting the building on a bed of sand or round stones, providing rockers below the columns, providing some kind of sliding mechanisms and so on are also suggested by various engineers. However, **these methods could prove highly dangerous unless they are based on sound engineering principles backed by experimental evidence.** A variety of methods which have been tested and proven to be effective both in the laboratory and field conditions are given at the end of this chapter.

The key for good seismic design is simplicity in plan and elevation. The plan should be symmetric and there should be a uniform distribution not only of mass but also stiffness. Columns and walls should be arranged in grid fashion and should not be staggered in plan. There should be a uniform distribution of columns and walls in the plan and weaknesses such as ducts and open-to-sky should be avoided. The openings in the walls should be located centrally and be small enough so that the wall is not unduly weakened. Openings from column to column as also corner windows ahould be avoided. Both wall dimensions and openings should conform to the provisions given in IS 4326 – 1993. In the elevation, sudden changes in stiffness as in stilt floors, long cantilevers and floating columns should be avoided. In case, it is not possible to avoid them, they should be designed by a qualified structural engineer who understands structural dynamics and earthquake resistant design. Appendages like sun-shades (Chajjas), water tanks and staircase covers should be designed for higher safety levels.

Structures, which have more than one axis of symmetry and have uniform distribution of strength and stiffness and are free from reentrant corners, are said to be *regular structures*. Structures, which do not satisfy one or more of the above requirements, are said to be irregular. Some common types of irregularities are shown in Fig. 2. Irregular structures exhibit special problems during earthquakes and should be avoided as far as possible.



(a) Un-symmetric Plan *(b) Reentrant Corners* *(c) Soft Story* *(d) Plaza Type Buildings*

Fig. 2 Irregular Structures

An unsymmetrical plan as shown in Fig. 2(a) leads to torsional effects, especially if the mass centre and the shear centre of the lateral load resisting systems do not coincide. Reentrant corners, as in Fig. 2(b), lead to differential deformations in the wings and consequent cracking at the corners. Therefore all L-shaped, H-shaped, U-shaped, W-shaped and any other alphabet shaped, except O-shaped buildings, should be avoided. It is advantageous to split such plans into separate rectangles with a crumple zone in between as explained in IS 4326-1993. A flexible first storey, also known as the soft storey shown in Fig. 2(c) leads to excessive ductility demands on the columns in the first storey and should be avoided. Sudden changes in stiffness in the elevation, as in the plaza-type building shown in Fig. 2(d), should also be avoided. Connections and bridges between buildings should be avoided and buildings with different sizes and shapes should have adequate gap between them to avoid pounding. The revised draft of IS 1893-2000, gives more details about irregularities and design methods for such buildings.

Masonry and infill (non-structural) walls should be reinforced by vertical and horizontal reinforcing bands to avoid their failure under a severe earthquake. It should be noted that wood is not ductile and needs to be reinforced with steel to withstand severe earthquakes. Also other non-structural elements should be carefully designed so that they do not cause injury to people. Hugh book shelves and almirahs should be tied to the walls so that they do not topple, Wall clocks and picture frames should not be put over exit doors as they can fall on the head of the person trying to escape. Roof tiles should be tied by a steel wire or some sort of sheeting should be used below them to prevent their falling down.

Reinforced Concrete elements should be detailed as per IS 13920-1993 which requires extra stirrups at potential hinging locations and extra anchorage lengths. It should be remembered that steel structures perform better than RC structures and should be adopted for all important buildings such as hotels, multi-storied buildings and hospitals. Pre-cast elements should be tied securely so that they don’t get dislodged during the earthquake.

3.0 SEISMIC ANALYSIS AND DESIGN VERIFICATION

Structures are usually designed for gravity loads and checked for earthquake loading. In conformity with the design philosophy, this check consists of two steps – the first ensures elastic response under moderate earthquakes and the second ensures that collapse is precluded under a severe earthquake. Due to the uncertainties associated in predicting the inelastic response, the second check may be dispensed with, by providing adequate ductility and energy dissipation capacity. In this section, the various methods of performing these checks are described.

The important factors, which influence earthquake resistant design are, the geographical location of the structure, the site soil and foundation condition, the importance of the structure, the dynamic characteristics of the structure such as the natural periods and the properties of the structure such as strength, stiffness, ductility, and energy dissipation capacity. These factors are considered directly or indirectly in all the methods of analysis.

3.1 Elastic Response Analysis

Elastic response analysis is invariably performed as a part of the usual design procedure. The primary aim of elastic analysis is to ensure serviceability under moderate earthquakes. For simple and regular structures, the seismic coefficient method is normally used. Structures such as multi-storeyed buildings, overhead water tanks and bridge piers are usually designed by the response spectrum method while for more important structures such as nuclear reactors, time-history response analysis is usually adopted. In what follows the seismic coefficient method is explained in detail while the response spectrum method and time history analysis are described briefly since understanding of these methods requires some knowledge of structural dynamics.

3.1.1 The Seismic Coefficient Method

This is the simplest of the available methods and is applicable to structures, which are simple, symmetric, and regular. In this method, the seismic load is idealised as a system of equivalent static loads, which is applied to the structure and an elastic analysis is performed to ensure that the stresses are within allowable limits. The sum of the equivalent static loads is proportional to the total weight of the structure and the constant of proportionality, known as the seismic coefficient, is taken as the product of various factors, which influence the design and are specified in the codes (IS 1893 – 1984).

Typically, the design horizontal seismic coefficient α_h is given by

$$\alpha_h = \beta \cdot I \cdot \alpha_o \tag{1}$$

where, α_o is the basic horizontal seismic coefficient, β is a coefficient depending upon the soil-foundation system and I is the importance factor. The factor α_o, also known as the zone factor, takes care of the geographical location of the structure. The site soil condition and the type of foundation also modify the ground motion locally and are taken into account by means of the coefficient β. The importance of a structure is determined based on its destructive potential or its role in the post-earthquake scenario. Thus dams, which can cause flooding, are given the maximum importance while hospitals, which may be required following the earthquake, are given relatively higher importance than ordinary buildings.

The total horizontal load, also known as the base shear is then taken as,

$$V_B = KC\alpha_h W \tag{2}$$

where, K = performance factor which takes into account the ductility and energy dissipation capacities, C = coefficient taking into account the fundamental natural period T and W = total of dead load plus an appropriate amount of live load.

For multi-storey buildings, the natural period in seconds may be calculated as $T = 0.1n$, where, n is the number of storeys.

The base shear calculated above is then distributed along the height of the building using the formula,

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \tag{3}$$

where, Q_i is the lateral force at the top of floor i, W_i is the total of dead and appropriate amount of live load at the top of floor i, h_i is the height measured from the base of the building to the top of floor i, and n is the number of storeys.

The seismic coefficient method gives conservative results but has the advantage of being simple and easy to use. It ignores the effect of higher modes and cannot accommodate irregularities in the structure. It is used for checking against moderate earthquakes since the emphasis is on resisting the earthquake loads by virtue of elastic strength rather than inelastic behaviour. Therefore the only safeguards that can be provided against severe earthquakes is by following a design procedure, as in capacity design, along with a set of detailing rules, which will ensure some degree of ductility and energy dissipation capacity.

3.1.2 The Response Spectrum Method

Although the response spectrum method requires more calculations than the seismic coefficient method, it has the advantage that, it can account for irregularities as well as higher mode contributions and gives more accurate results. Therefore, this is the most widely used method in seismic analysis. Numerous attempts have been made to extend the applicability of the method for inelastic response under severe earthquakes.

The *response spectrum* is a plot of the maximum response (usually the acceleration S_a) of single-degree-of-freedom (SDOF) systems as a function of their natural period T (see Fig. 3). For design purposes, the smoothed average of a number of elastic response spectrums corresponding to various possible earthquakes at a particular site, known as the *smoothed elastic design response spectrum* (SEDRS), is used. The SEDRS is further simplified so that it can be represented by a set of equations corresponding to different period ranges. SEDRS are usually specified for different soil conditions.

Most structures, such as multi-storeyed buildings are multi-degree-of-freedom (MDOF) systems whose response can be approximated by considering only the first few natural modes. This fact is used to great advantage in *modal spectral analysis*, where the first few natural vibration mode shapes are calculated as a first step. Each mode can then be considered to represent the vibration shape of an SDOF with a corresponding natural period and so its maximum response can be directly determined from the response spectrum. The total response of the structure can then be calculated as a combination of these individual responses. A variety of ways are available to combine the individual responses considering the fact that these maximum responses occur at different instants of time. When the natural periods are sufficiently apart, the most common way of combining the maximum responses is by taking the square root of the sum of the squares (SRSS) method (Clough and Penzien 1993).

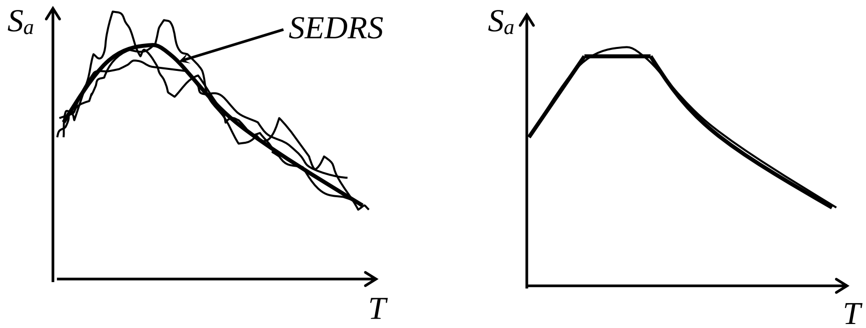


Fig. 3 Developing the Design Response Spectrum

3.1.3 Time-history Analysis Method

For important structures, both linear and non-linear responses can be obtained by carrying out detailed time-history analysis for one or more design accelerograms. These design accelerograms may be either natural accelerograms recorded at the site or at similar sites or they can be artificial accelerograms generated in such a way as to be compatible with the design response spectrum. A variety of numerical time-stepping methods are available for calculating the response time-history. Detailed discussions of these methods are beyond the scope of the present chapter and the reader is referred to books on structural dynamics (Clough and Penzien 1993).

3.2 Inelastic Response Analysis

Some structures may be more ductile than others may and the designer may wish to take advantage of ductility to reduce the design loads. In such cases, inelastic response analysis will be required to ensure safety under severe earthquakes.

3.2.1 Inelastic Response Spectra

The concept of elastic response spectrum can be extended to the inelastic range with the tacit assumption that damage is proportional to the maximum inelastic displacement. Thus, the inelastic response spectrum for a given earthquake accelerogram can be plotted by carrying out an inelastic time-history analysis for each SDOF system period and picking up the maximum value of the response as will be explained in the next subsection.

For design purposes, an inelastic design response spectrum can be specified based on an assumed force-deformation relationship. Alternatively, the ratio of the ordinate of the Elastic Design Response Spectrum (EDRS) to that of the Inelastic Design Response Spectrum (IDRS), known as the *force-reduction-factor (FRF)* or the *q-factor* can be specified for various time periods T. The corresponding inelastic design strength F_y can be obtained as the elastic design strength F_e divided by the *FRF* (see Fig. 4).

Assuming an *elastic-perfectly-plastic (EPP)* force-deformation relationship that is typical for structural steel, and analysing for a large number of earthquake accelerograms, Newmark and Hall (1982) arrived at the following values for the force-reduction-factors.

$$
\begin{aligned}
&FRF = 1 && \text{for short period structures } (T < 0.1\,\text{sec}) \\
&FRF = \sqrt{2\mu - 1} && \text{for medium period structures } (0.1 \le T \le 0.6\,\text{sec}) \\
&FRF = \mu && \text{for long period structures } (T > 0.6\,\text{sec})
\end{aligned}
\tag{4}
$$

Note that as T tends to zero, the acceleration of the inelastic system will be the same as the acceleration of the elastic system and so $FRF = 1$. As T tends to infinity, the displacement of the inelastic system will be μ times the displacement of the elastic system and so $FRF = \mu$. For systems with intermediate period ranges, considering the equivalence of absorbed energy (*i.e.* equating the areas under OAB and OAC), the expression for *FRF* can be derived as $\sqrt{(2\mu-1)}$.

Some researchers have used other types of force-deformation relationships to arrive at various expressions for *FRFs* (Krawinkler and Nassar 1992, Miranda 1993, Kumar and Usami, 1996). Based on these studies, it has been found that the division of the entire spectrum of time periods into three ranges leads to highly conservative results at the upper end and unconservative results at the lower end of each range. Therefore, it would be more appropriate to specify *FRFs* as smooth functions of time period. Also when the earthquake accelerogram has a predominant period close to the system period, *FRFs* in excess of μ can be obtained.

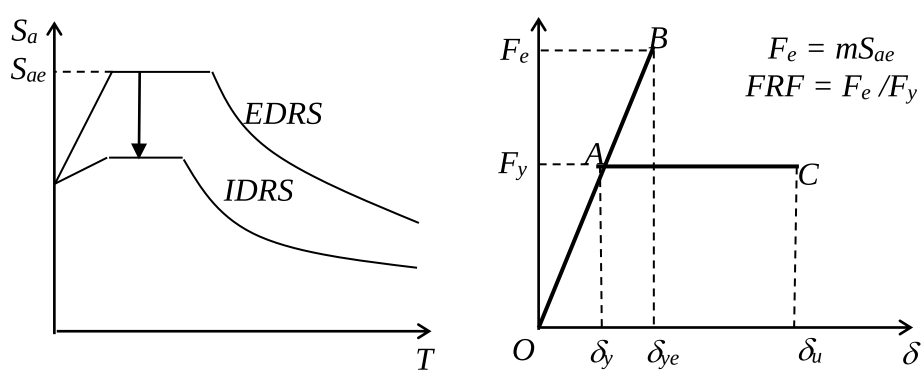


Fig. 4 Inelastic Design Response Spectrum

Strictly speaking, modal spectral analysis is not applicable in the inelastic range since superposition of the modal responses is not valid. Further, it is difficult to calculate the *FRF*s for MDOF structures as it depends on several factors such as the local ductility of critical sections, amount of redistribution of forces possible, the type of collapse mechanism developed and also on the characteristics of the earthquake accelerogram. Therefore, the codes prefer to specify the IDRS by using a behavioural factor or the q-factor, which is similar to the *FRF* (Eurocode 8 1993, AISC 1997). The behavioural factor assumes a certain level of ductility and energy dissipation capacity depending on the type and topology of the structure. For example, eccentrically braced frames or moment resisting frames are assigned a larger q-factor compared to concentrically braced frames in Eurocode 8.

3.2.2 Inelastic Time-history Analysis

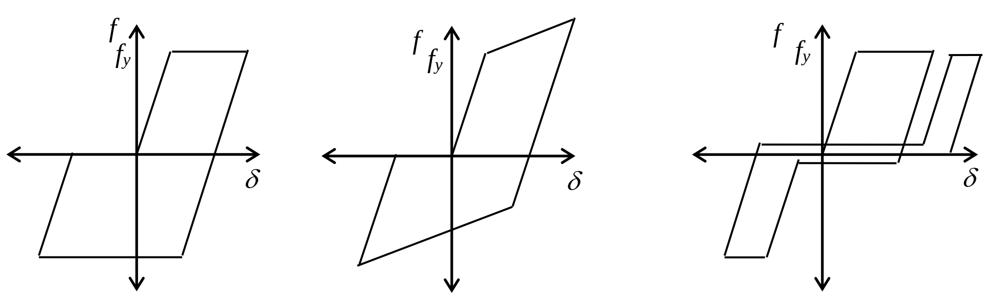
Inelastic time-history analysis is basic for plotting the inelastic response spectrum and requires a hysteretic model giving the cyclic force-displacement relationship. In addition to the maximum displacement plotted in the inelastic response spectrum, time-history analysis also provides information such as the number and magnitude of inelastic excursions and the residual displacement at the end of the earthquake. The accuracy of the analysis will depend on the numerical method used and the correlation between the assumed hysteretic model and the actual behaviour. Some of the commonly assumed hysteretic models for steel structures and members will now be described.

The simplest and the most commonly used hysteretic model is the elastic-perfectly-plastic (EPP) model [Fig. 5(*a*)]. The model comes directly from plastic theory and consists of an elastic branch up to the yield deformation followed by a perfectly plastic range at the actual or equivalent yield load. It neglects the effect of strain hardening and is incapable of simulating the strength and stiffness degradation due to low-cycle fatigue.

A slight modification of the EPP model yields the elastic-plastic-hardening (EPH) model [Fig. 5(*b*)], which takes into account the gradual plastification and strain hardening effects. The most common hardening rule used in this model is the kinematic hardening rule wherein an increase in the yield load in one direction is accompanied by a corresponding decrease in the yield load in the other direction so that the elastic range remains constant.

For bolted connections, the slip model shown in Fig. 5(c) is commonly used. This is because, once the bolts elongate in tension and separation takes place between the connected plates, the connection offers little resistance to load reversal until the connected plates come back in contact and bear against each other in compression.

For cross-sections undergoing local buckling such as wide-flanged beams and thin-walled sections, the strength and stiffness degradation are pronounced under cyclic loading and cannot be neglected. Therefore, more complicated hysteretic models employing a set of degradation rules derived empirically or evolutionary-degrading hysteretic models based on a damage index are used (Cosenza and Manfreidi 1992, Kumar and Usami 1996).



(a) EPP model ***(b) EPH model*** ***(c) Slip model***

Fig. 5 Hysteretic models

The hysteretic behaviour can also be obtained directly from experiments and used in time-history analysis simultaneously. Such tests are called pseudo-dynamic tests or hybrid tests (Kumar and Usami 1996).

Apart from the hysteretic model, which defines the behaviour of a member or system, the characteristics of the ground motion also dictate the type of response. Therefore, standard accelerograms are usually specified for use in time history analysis. Where a number of analyses are required a set of accelerograms may be generated artificially so as to be compatible to the design spectrum.

3.2.3 Damage Evaluation and Damage Spectra

Since a number of aspects of the response contribute to the damage of the member or structure, a damage index is used to evaluate and compare different response histories. The damage index is a number, which indicates the amount of damage sustained and the reserve capacity left after the structure or member is subjected to a particular earthquake. Typically, it is normalised to have a value of zero for elastic response and attains a value of unity at an assumed collapse point. Various damage indices are available in the literature and a few important ones are described in this section. The use of the damage index in design, by means of damage spectra, is also explained.

The damage sustained by a structure under cyclic loading is reflected by a number of parameters. The parameters may be stated as (1) the maximum deformation; (2) the low-cycle fatigue; (3) the distribution of cycles; (4) the order of cycles and (5) the structural parameters and loading conditions. Depending on the parameters considered, damage indices of various types and complexities exist.

The simplest damage index is the ductility damage index, which considers only the damage due to maximum deformation. The ductility damage index is given by

$$D = \frac{\delta_{\max} - \delta_y}{\delta_u - \delta_y} \tag{5}$$

where, δ_{max} is the maximum deformation, δ_u is the ultimate deformation and δ_y is the yield deformation.

A damage index, which considers only the damage due to low-cycle fatigue, may be defined as

$$D = \sum_{i=1}^{N} \left(\frac{\delta_i - \delta_y}{\delta_u - \delta_y} \right)^c \tag{6}$$

where, δ_i is the maximum deformation in the *i*-th half-cycle, N is the total number of half-cycles and c is a structural parameter such that $c \geq 1$.

Combining the above expressions, a comprehensive damage index considering both deformation damage and low-cycle fatigue effect can be defined as

$$D = (1-\beta)\left(\frac{\delta_{\max} - \delta_y}{\delta_u - \delta_y}\right)^c + \beta \sum_{i=1}^{N}\left(\frac{E_i}{f_y(\delta_u - \delta_y)}\right)^c \qquad (7)$$

where, β and c are structural parameters and E_i is the hysteretic energy dissipated in the i-th half-cycle. Note that for an elastic-perfectly-plastic system, $E_i = f_y(\delta_i - \delta_y)$.

The damage index can be used to quantify the damage sustained and thus enables the comparison of the different types of responses. However, the damage index can be put to greater use if one realises that inelastic response spectra or force reduction factor spectra can be plotted for constant damage index rather than for constant ductility values. In this way the low-cycle fatigue effect can also be taken into account in design. Such damage spectra have been plotted by Cosenza and Manfreidi (1992) and Kumar and Usami (1996) among others.

4.0 SEISMIC BEHAVIOUR OF STEEL STRUCTURES

Steel structures have been known to perform well under earthquake loads provided certain guidelines are followed in design. Some of these guidelines are discussed qualitatively at the material, member and structural levels.

4.1 Seismic Behaviour of Structural Steel

Steel being a ductile material, equally strong in compression and tension, is ideally suited for earthquake resistant structures. The common grades of mild steel have adequate ductility and perform well under cyclic reversal of stresses. High strength steels provide higher elastic limits but have less ductility. Another disadvantage in using high strength steels is that they require less areas of cross-section as compared to mild steels and thereby get more prone to instability effects.

Steel as a material is produced with high quality control, which aids in Capacity Design. The sequence of formation of plastic hinges is important in capacity design and so it is necessary to be able to predict the actual yield stress accurately. If the actual strength of members is larger than their design strength, plastic hinges may develop in other members first. In order to avoid such a situation, some codes introduce a factor, which is the ratio of the expected yield strength to the specified minimum yield strength for various grades of steel. This factor is also used to ensure that members or connections that must withstand the development of plastic hinges in other members have sufficient strength.

4.2 Seismic Behaviour of Bracing Members

Bracing members are used either as part of a lateral load resisting system or to increase the stiffness of a frame in the lateral direction. They may be either pin-ended or fixed-ended. Pin-ended braces will be subjected to only axial forces and usually fail by global buckling under compressive load. After the initial buckling, the buckling strength gets reduced in subsequent cycles due to non-straightness of the brace. However, the maximum tensile strength remains relatively unchanged during cycling and presents a ductile behaviour. Therefore, pin-ended braces are usually used in pairs, as in X-bracing so that at least one brace will be effective for loading on either side. Another advantage of using braces is that it becomes possible to dissipate energy without damaging the main structure and it is easy and economical to replace the braces after an earthquake.

The hysteretic behaviour of a slender brace subjected to incremental amplitude cycling is shown in Fig. 6. In the first cycle on the compression side, a drastic change in geometry takes place because of inelastic buckling. During reloading on the tension side, the loops are well rounded because the fibres on the inside of the buckle undergo plastic yielding. Subsequent cycles have a trapezoidal shape with a plastic limb on the tension side and another on the compression side. The limb on the tension side has a constant strength close to the yield strength while the limb on the compression side has the degraded strength. Unloading and reloading from tension to compression side is almost instantaneous while that from compression to tension side takes place with a degraded stiffness.

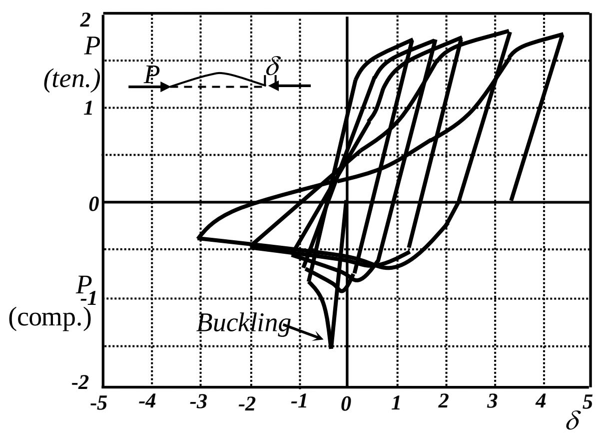


Fig. 6 Hysteretic behaviour of a slender brace

To ensure adequate energy dissipation capacity, the slenderness is normally limited to a value such that the strength of the brace under static compressive loading is about one-half of that under tension. Ductility on the compression side is low but it reflects the ductility of the material on the tension side and ductility values as large as twenty can be obtained on the tension side. Since loading is predominantly axial, members with solid cross-sections such a rods and bars are preferred and so local buckling is usually not a consideration except in single angle braces.

4.3 Seismic Behaviour of Beam-Columns

Frame members such as beams and columns are normally expected to develop ductile plastic hinges at critical sections. Therefore such members should have plastic cross-sections. However, since plastic cross-sections are not always very efficient there is a tendency to use compact and sometimes semi-compact sections in which case adequate ultimate moment capacity, ductility or rotation capacity and also the hysteretic energy dissipation capacity should be ascertained. Since local buckling is the controlling factor in the evaluation of these quantities, the most reliable methods are empirical in nature. Various methods of evaluating these quantities are available in the literature but are beyond the scope of the present chapter. In the following, a qualitative description of the cyclic behaviour of beam-columns with various cross-sections is described in terms of the characteristics of the hysteretic loop shapes, the degradation, and the failure mode. Understanding these aspects will help the designer to choose an appropriate cross-section for the various members.

4.3.1 Seismic Behaviour of I-sections

Steel I-sections have good ductility and energy dissipation capacities and usually fail by local buckling of the flanges, which is sometimes followed by initiation of cracks at the flange-web junction or in the case of built-up sections, at the weldments. Steel I-sections have been tested by Bertero and Popov (1965), Krawinkler and Zhorei (1984) and Ballio and Castiglioni (1994) among others.

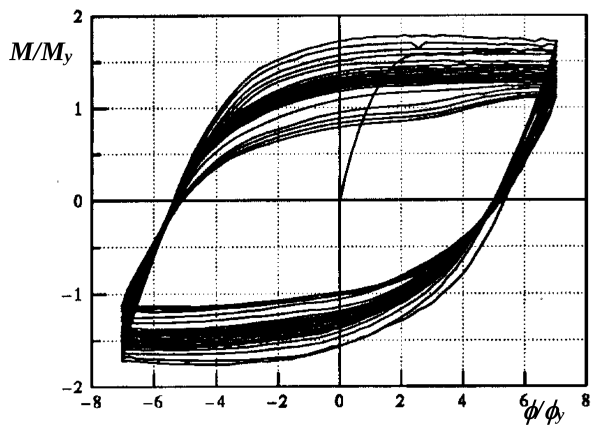


Fig.7 Hysteretic behaviour of an I-section

A typical hysteretic curve is shown in Fig. 7 for constant amplitude cycling. The three ranges of response, namely, the degradation due to local buckling, the gradual stabilisation, and subsequent *pinching* of the hysteretic curve after crack initiation can be observed. As the sections become more and more compact, the middle range becomes smaller and the tendency for cracking is increased which means that highly compact sections, coupled with rigid connections, may not be able to provide the required rotations. The damage accumulation in these sections can be modelled using the low-cycle fatigue approach.

4.3.2 Rectangular Hollow Sections (RHS)

Rectangular hollow sections, either hot-rolled or fabricated by welding four plates are used in buildings and bridge piers. The sections used in buildings have relatively low width-thickness ratios of component plates but are subjected to higher axial load ratios (ratio of applied axial load to squash load) as compared to the sections used in bridge piers. These sections score over I-sections due to the fact that there are no outstands and consequently the ultimate strengths are higher and the post-local buckling performance is also better. However beam-to-column connections are relatively more difficult to fabricate using rectangular hollow sections.

Rectangular hollow sections have been tested by Ballio and Calado (1994) and Kumar and Usami (1996). A typical hysteretic curve under incremental amplitude cycling is shown in Fig. 8. The hysteretic loop shape is similar to that of I-sections but in the case of sections with higher width-thickness ratios, the degradation in strength with cycling is considerable, particularly under increased amplitudes. Therefore the damage accumulation calculations need to take into account the deformation damage in addition to the low-cycle fatigue damage. Both ductility and strength are affected by the axial load ratio and the ductility decreases rapidly with increase in the axial load ratio.

RHS sections used in buildings are invariably unstiffened and the failure is by inward local buckling of the flange followed by outward buckling of the web plates.

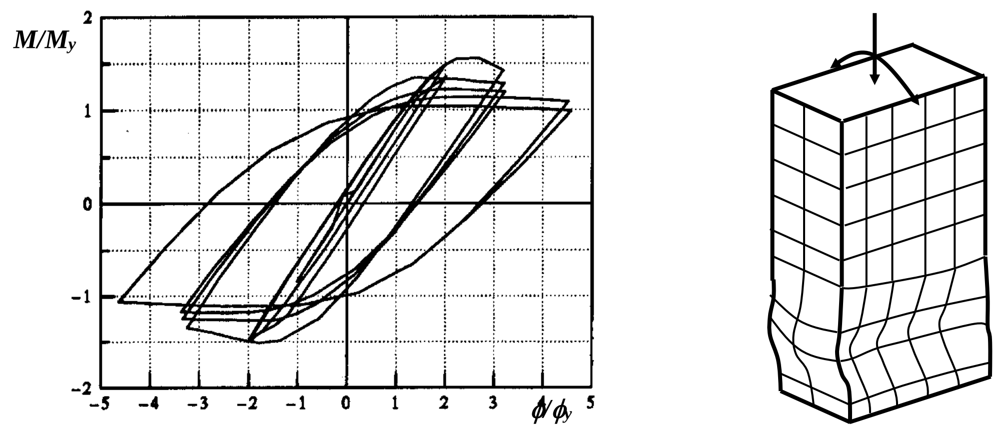


Fig. 8 Hysteretic behaviour of rectangular hollow section

4.3.3 Circular Hollow Sections (CHS)

Due to their pleasing appearance, circular hollow sections are used as columns in buildings and in bridge piers. Unlike RHS, CHS are more difficult to connect to I-section beams and may require elaborate external stiffeners to achieve a ductile connection.

Circular hollow sections have been tested by Kumar et al (1998) and others. A typical hysteretic curve under incremental amplitude cycling is shown in Fig. 9. It can be seen from the figure that the hysteretic loops are more rounded and have negative slopes in the plastic regions. This is due to the type of local buckling and the way it spreads around the circumference as described below.

When a CHS section is subjected to bending in one direction, the part of the section under compression develops local buckling outwards. As the bending moment is increased, the local buckling spreads along the circumference of the tube, since there are no webs to arrest the spread like in RHS sections. If a bending moment is applied in the opposite direction, a similar local buckling develops and finally merges with the previously developed local buckling to form what is known as a *ring type* local buckling. The formation of a ring type local buckling not only gives smaller ductility values for CHS as compared to RHS but also causes progressive collapse under moderate but long duration earthquakes. This is because, when cycled at small amplitudes, the side, which buckles first, tends to have a smaller maximum strength as compared to the opposite side. As a result, the deformations on that side increase progressively leading to collapse.

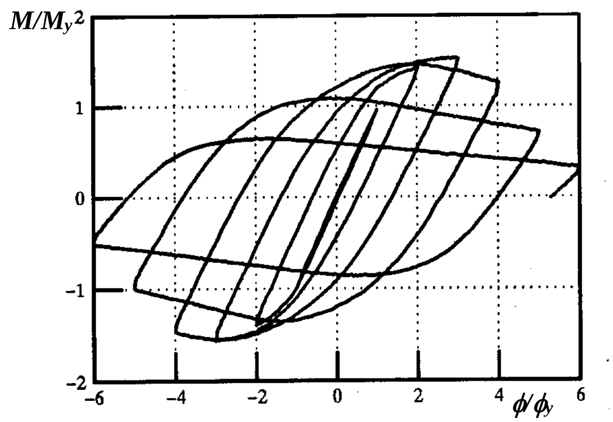


Fig. 9 Hysteretic behaviour of circular hollow section

4.3.4 Concrete-Filled Tubes (CFTs)

Hollow sections filled with concrete have several advantages such as increased axial load carrying capacity and no necessity for formwork. Their earthquake performance is also excellent due to the fact that their bending strengths are high, the concrete being in a confined state dissipates more energy and also enhances the local buckling strength of the steel section in some cases.

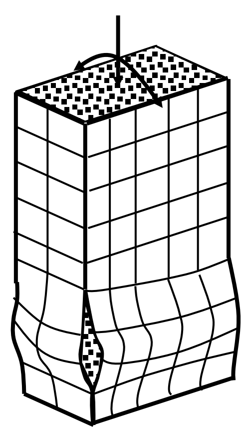


Fig. 10 Failure Mode of a Concrete-filled RHS

In the case of concrete-filled rectangular sections, the concrete effectively prevents the tendency of the flanges to buckle inwards thereby delaying the local buckling and increasing strength, ductility and energy dissipation capacity of the member. Concrete-filled rectangular sections have been tested by Ge and Usami (1994) and others. Some pinching may be observed in the hysteretic curve due to separation or slip between concrete and steel but its effect on energy dissipation capacity is negligible. Failure is by outward buckling of both flange and web plates and consequently a crack develops and propagates in the longitudinal direction, separating the plates.

Circular sections tend to develop local buckling outwards and so filling concrete inside is not likely to increase their ductility although some improvement in strength may be expected.

4.4 Connections

Connections are the most vulnerable in steel structures. Bolted connections using black bolts tend to slip, which reduces their energy dissipation capacity under cyclic loading and so are to be avoided. HSFG bolts perform better. All bolted connections exhibit pinching of the hysteretic loops, which reduces their energy dissipation capacities. Brittle welding failures are common due to low-cycle fatigue and so special care needs to be taken to reduce stress concentrations at welds. The cost of various types of connections dictates the lateral load resisting system used in steel framed structures.

Simple connections are not expected to carry any moments and so only rigid and semi-rigid connections will be discussed. Rigid connections are usually strengthened to an extent that their rotations /deformations are negligible compared to that of the members being connected. This is because in conformity with the capacity design philosophy, it is advantageous to ensure the development of plastic hinges in beams away from the beam-column connection.

Limited test results are available for the cyclic behaviour of beam-to-column connections (Mazzolani and Piluso 1996 and Calado et al 1998). It was found that fully welded connections with column web stiffeners at the level of the beam flanges [Fig. 11(a)] provide a well-developed hysteretic curve similar to that shown for I-sections. Extended end plate connections to rigid column stubs using bolts [Fig. 11(b)], also provides fairly good hysteretic loops but leads to abrupt failures after a few cycles. Bolted angle connections with top and seat angles as well as web cleats [Fig. 11(c)] leads to pronounced pinching of hysteretic loops due to deformation of the angles (Fig. 12). Connections with three splice plates welded to columns and bolted to beam flanges and web [Fig. 11(d)] give some degree of pinching but provision of diagonal column web stiffeners does not improve the situation. It should be noted that the above tests were conducted in a quasi-static manner.

Observations on steel moment resisting connections damaged during the Northridge and Kobe earthquakes indicate that such connections develop brittle fractures under high strain rates (Bruneau et al 1998) and so it may be advantageous to provide some degree of flexibility in the connections and go for semi-rigid connections. Semi-rigid connections for composite beams may be obtained by providing top reinforcement. .

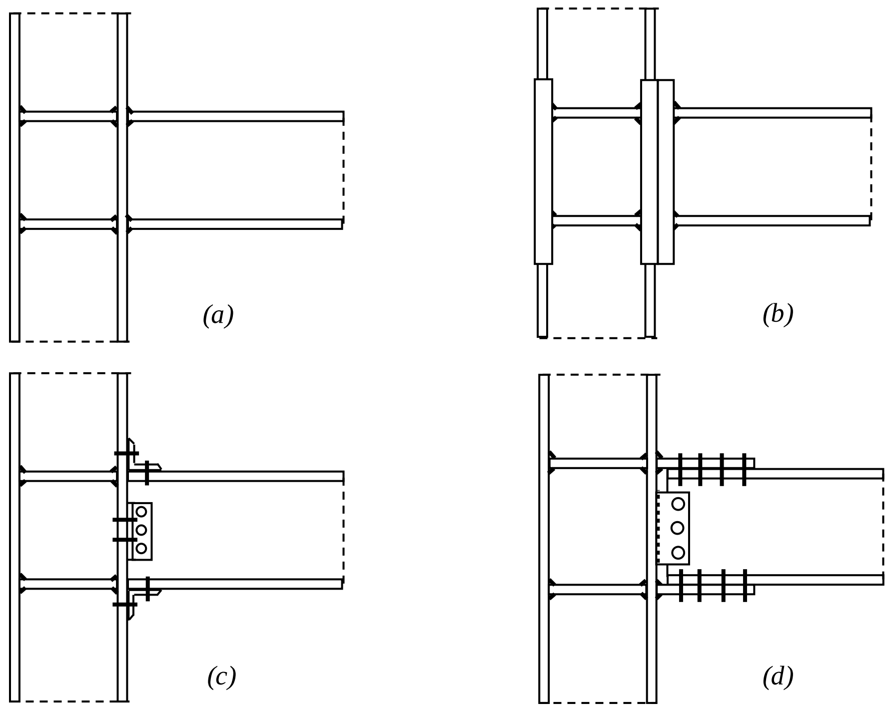


Fig. 11 Types of moment connections

4.5 Steel Frames

Steel Frames can be classified as *sway frames* and *non-sway frames* depending upon their sensitivity to second-order effects in the elastic range. Both types of frames can be either *braced* or *unbraced* even though braced frames normally fall under non-sway category. Braced frames can be classified as concentrically braced or eccentrically braced.

Concentric bracing may be designed to resist either the entire seismic load or as a supplementary system in a moment resisting frame. In the former case the bracing is used in combination with simple beam-to-column connections (shear connections).

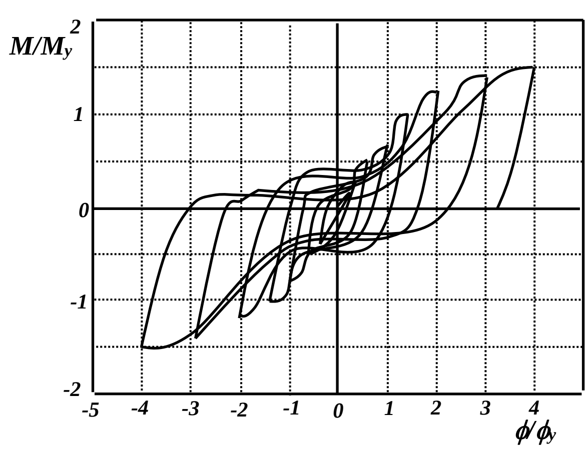


Fig. 12 Hysteretic behaviour of bolted angle connection

In *concentrically braced frames (CBF)* having simple connections, it is assumed that the centroidal axes of the members meet at a common point at each joint and so the members carry essentially axial loads. Various bracing configurations such as diagonal bracing [Fig. 13 (a)], Cross or X-bracing [Fig. 13 (b)] and Chevron bracing [Fig. 13 (c)], are possible. Concentric bracing has not been found to perform well due to premature buckling of the braces, which limits their energy dissipation capacity. Further, due to their higher stiffness, they tend to attain a larger seismic force. Their performance can be improved by limiting the tendency of the brace to develop global or local buckling and ensuring proper connections. They may also interfere with aesthetics and functionality of the buildings.

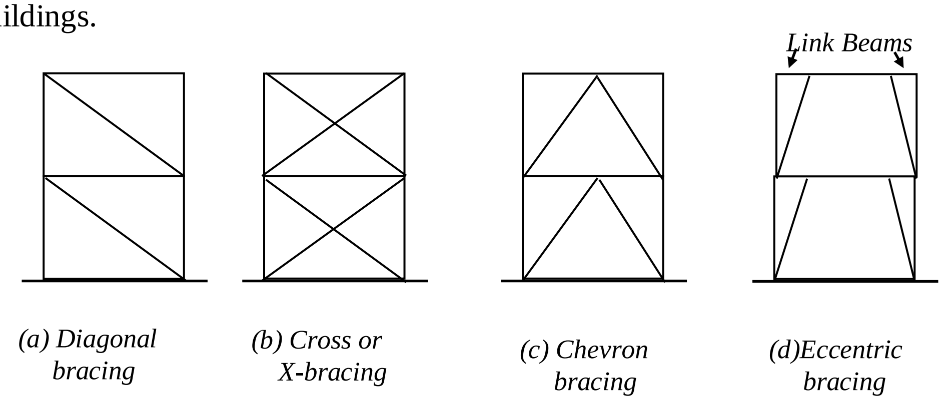


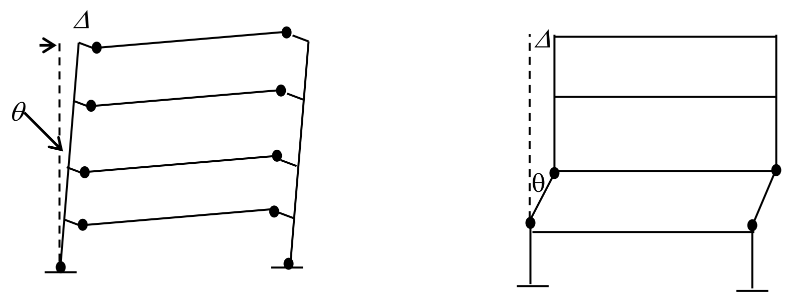
Fig. 13 Bracing systems in Steel Frames

Eccentrically braced frames (EBF) are designed by assuming the bracing member to be pin-ended but the beam-column connection to be a moment-resisting connection. The bracing provides increased stiffness in the lateral direction and thus helps in controlling the drift. The short part of the beam, between the bracing and the column is known as the link (see Fig. 13(d)) and most of the energy is dissipated in the link by yielding in shear or flexure. Therefore, eccentrically braced frames perform better than concentrically braced frames. They are also functionally convenient in some situations.

Moment resisting frames (MRF) rely on the ability of the frame to act as a partially or fully rigid jointed frame while resisting the lateral loads. Due to their flexibility, moment resisting frames experience a large drift especially in multi-storeyed buildings. The frames can be designed either to dissipate energy by the formation of plastic hinges at the beam-ends or to dissipate energy in the connections. The former is preferred over the latter due to the complexities associated with connection analysis and design. However, in both cases it is necessary to ensure a strong and ductile connection. Measures taken to improve the performance of connections include the use of column-web stiffeners in I-beam to I-column connections to stiffen the panel zones. Beams ends may also be haunched to ensure the formation of the plastic hinge away from the connection and thereby obtain better performance.

5.0 CAPACITY DESIGN

The type of collapse mechanism developed largely dictates the overall ductility and energy dissipation capacity of the frame and so capacity design is invariably carried out. In capacity design, the type of collapse mechanism required is pre-decided and attempts are made to make sure that no other mechanism develops. In multi-story frames, the strong column-weak beam mechanism is preferred since this mechanism requires the formation of many plastic hinges and also the plastic rotation capacity required at the hinges is less (Fig. 14). The term plastic rotation capacity is used in place of ductility when talking about the moment-rotation curve instead of the usual force-deformation curve (this is explained in the chapter on plastic analysis).



(a) Desired *(b) Undesired*

Fig. 14 Desired and Undesired Collapse Mechanisms in Capacity Design

6.0 SPECIAL DEVICES AND SYSTEMS

In addition to the above mentioned design guidelines, the structural engineer has the option of using a variety of devices or systems to ensure safety and serviceability of the structure under strong ground motion (Kwok 1991). These devices or systems either isolate the structure from ground vibration or act as energy dissipating mechanisms thereby reducing the vibration amplitudes and damage to the structure. A brief overview of some of the available systems will be given in this section. These are finding increasing applications in modern designs.

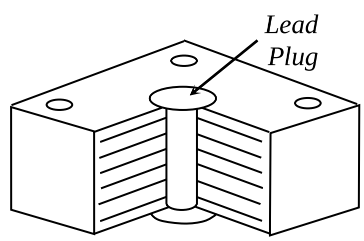


Fig. 15 Laminated rubber bearing pad

The simplest way of reducing the vibration of the structure and hence its design loads is by isolating it from the ground by means of springs or special rubber pads. This technique is called *base isolation*. The idea behind base isolation is to put a spring or similar device, which makes the structure-spring system flexible thereby increasing the time period. Looking at the response spectrum in Fig. 3, it can be realised that an increase in time-period implies a reduction in the response. Base isolators may be either coiled springs or laminated rubber-bearing pads. The pads are made of alternate layers of steel and rubber and have a low lateral stiffness [Fig. 15].

The technique of reducing the vibration by means of special devices mounted on the structure is called vibration control. When the device merely absorbs the energy during vibration without any energy input from outside, the control is said to be passive. On the other hand, when the system opposes the vibration by means of an external energy source, the control is said to be active.

Visco-elastic dampers are commonly used as sliding supports of beams and trusses. They consist of a visco-elastic material, sandwiched between two steel plates, which undergoes shear deformation and thus dissipates energy.

Tuned mass dampers (TMD) are extra masses attached to a structure by a spring-dashpot system and tuned to vibrate out of phase with the structure. Energy is dissipated by the dashpot due to the relative motion between the mass and the structure.

Tuned liquid dampers (TLD) are essentially water tanks mounted on structures and dissipate energy by the sloshing of the water. The motion of the liquid may also be hindered, by baffles or orifices, to get additional energy dissipation.

Hydraulic actuators can be used to provide active vibration control and are expensive but effective in controlling the vibration. They require a sensor to sense the vibration and activate the actuator so as to counter it.

Weak links, stoppers or other devices, made of ductile material can be used to dissipate energy by hysteretic behaviour *(Inelastic dampers)* and thereby increase the overall damping in the structure.

7.0 SUMMARY

The characteristics of earthquake loads were described. The dual strategy of ensuring elastic response under moderate earthquakes and preventing collapse under a severe earthquake was explained. The properties of the structure, particularly ductility and hysteretic energy dissipation capacity, which aid in resisting earthquake loads, were pointed out. The architectural considerations, which can simplify the design process and assure good seismic performance, were described.

The elastic and inelastic response prediction methods such as seismic coefficient, response spectrum and time-history analysis were explained. The background concepts on which most codal provisions are based were also explained. Guidelines to improve the seismic behaviour of steel structures were given at the material, member and structure levels. In particular, the hysteretic behaviour and collapse modes of bracing members and flexural members with various cross-sections were described in detail. The behaviour of lateral load resisting systems such as bracings and moment resistant frames was described. The concept of capacity design, which aims at maximising the energy dissipation capacity of moment resisting frames by choosing an appropriate collapse mechanism, was explained. Finally, an overview of special devices and systems, which can be used to control the response and thus reduce the design forces for members, was given.

REFERENCES

1. AISC, ‘Seismic Provisions for Structural Steel Buildings’, 1997.
2. Ballio G and Castiglioni C, ‘Seismic behaviour of steel sections’, *Jnl of Construct. and Steel Research* 1994.
3. Bertero V and Popov E, ‘Effect of large alternating strains on steel beams’, ASCE. ST1, 1965.
4. Breneau M and Uang , ‘Ductile design of Steel Structures’, McGraw Hill, New York, 1998.
5. Calado L et al, ‘Behaviour of steel beam-to-column joints under cyclic reversal loading: An experimental study’, in Stability and Ductility of Steel Structures, Ed. by Usami and Itoh, Elsevier, London, 1998.
6. Clough R W and Penzien J, ‘Dynamics of Structures’, McGraw Hill, New York, 1993
7. Cosenza E and Manfreidi G ‘Seismic analysis of degrading models by means of damage functions concept’, in Nonlinear Seismic Analysis of Reinforced Concrete Structures, Ed. by Fajfar P and Krawinkler H, Elsevier Applied Science, 1992.
8. Eurocode 8, ‘Structures in Seismic Regions’, Committee of the European Communities, Draft of Oct 1993.
9. IS 1893-1984, ‘Criteria for Earthquake Resistant Design of Structures’.Bureau of Indian Standards, New Delhi.
10. Krawinkler H and Nassar A A, ‘Seismic design based on ductility and cumulative damage demands and capacities’, in Nonlinear Seismic Analysis of Reinforced Concrete Structures, Ed. by Fajfar P and Krawinkler H, Elsevier Applied Science, 1992.
11. Krawinkler H and Zhorei M, ‘Cumulative damage in steel structures subjected to earthquake ground motions’, Computers and Structures, Vol.16, No.1-4, 1984.
12. Kumar S et al ‘Nonlinear dynamic response of thin-circular tubes’, in Stability and Ductility of Steel Structures, Ed. by Usami and Itoh, Elsevier, London, 1998.
13. Kumar S and Usami T, ‘ Inelastic seismic response analysis of thin-walled steel bridge piers’, NUCE research report No. 1000, Nagoya, 1996.
14. Kwok, K C S, ‘Damping and control of structures subjected to dynamic loading’, in Structures Subjected to Dynamic Loading, Ed. by Narayanan and Roberts, Elsevier Applied Science, London, 1991.
15. Mazzolani F M and Piluso V, ‘Theory and design of seismic resistant steel frames’, E & FN Spon, London, 1996.
16. Miranda ED-Model, ‘Site-dependent strength design factors’, ASCE, Vol. 119, ST12, 1993.
17. Naeim F, The Seismic Design Handbook, Van Nostrand Reinhold, New York, 1996.
18. Usami T and Ge H B, ‘Ductility of concrete-filled steel box columns under cyclic loading’, ASCE, Vol.120, ST7, 1994.
19. IS 4326-1993‘Earthquake Resistant Design and Construction of Buildings’, BIS, New Delhi.
20. IS 1893- 2000 ‘Criteria for Earthquake Resistant Design of Structures’, BIS, New Delhi.
21. IS 13920-1993 ‘ Ductile Detailing of RC Structures subjected to Seismic Loading’, BIS, New Delhi.