17

SEISMIC-RESISTANT DESIGN

INTRODUCTION

As discussed in Section 3.9 of Chapter 3, a huge amount of property damage and loss of life is due to natural hazards and the main culprits are the earthquakes, high speed winds, and floods [in India we do not have proper code for flood-resistant design, similar to the document available in USA (ASCE 24-05)]. We may use the same analysis procedures and software packages to arrive at the axial and shear forces and bending moments acting on structures subjected to lateral loads due to wind, earthquake, or for the combination of loads, as discussed in Section 3.15 of Chapter 3 (these lateral loads may be calculated by using the procedures described in Sections 3.12 and 3.13 of Chapter 3 and applied at the respective joints of a structure in the horizontal direction). Once the forces and moments are found out, we may proceed with the design as in the case of gravity loads.

However, we need to bear in mind that designing for earthquake is very different from designing for other loads such as dead load (DL), live load (LL), and wind load (WL). This is because, for loads other than earthquake loads, we design the structure for the actual load (though they are probabilistic, codes have specified the maximum probable loads based on measurements); however, it has to be noted that the wind loads specified in IS 875-Part 3:1987 have to be increased and rationalized as given in NICEE-GSDMA documents and ASCE 7-10. However, when we design any structure for earthquake loads, we consider about 1/6th to 1/10th of the earthquake load only [This is because, we do not consider the loads due to *maximum credible earthquake* or *maximum considered event* (MCE), but consider only the *design basis earthquake* (DBE), that is, an earthquake that the structure is required to safely withstand with repairable damage—see Section 3.12 of Chapter 3. In IS 1893 (Part 1):2002, DBE is taken as 50% of MCE. In addition, we divide the load by the *response reduction factor*]. It may be noted that as per UBC 1997 and IBC 2000, the DBE corresponds to an earthquake having 10% probability of being exceeded in 50 years, that is, a 475-year return period. [IS 1893(Part-1):2002 merely states that it is an earthquake that can reasonably be expected to occur at least once during the design life of the structure]. Even the *design-response spectrum* we consider is a smooth, broad-banded spectrum (needs to be site-specific in order to be more exact—in the design of important structures such as nuclear power stations, only *site-specific response spectrum* has to be used). The most important point to remember is that the earthquake-resistant design codes allow damage to the structure but disallow total collapse and death of occupants. Thus the codes rely on over-strength and ductility for the safety of the structure.

The combined effect of over strength, redundancy, and ductility is expressed in terms of *response reduction factor* (R) in the codes, which is used to arrive at the design horizontal seismic coefficient (see Section 3.12 and Table 3.22 of Chapter 3). It has to be noted that the precise estimation of *over-strength* is difficult to determine since many factors contributing to it involve uncertainties. The actual strength of materials, the contribution of non-structural elements, and the actual participation of some structural elements such as reinforced-concrete slabs are factors leading to high uncertainties (Humar and Ragozar 1996). In addition, not all factors contributing to over-strength are favourable. For example, flexural over-strength of members leads to increased shear forces when plastic hinges form, which may result in brittle shear failures. Non-structural elements, for example brick infills, could lead to shear failure due to short column effect or to soft storey failures (Park 1996). Moreover, the over-strength factor varies widely according to the period of the structure, the design intensity level, the structural system and the ductility level assumed in the design. This compounds the difficulties associated with evaluating this factor accurately (Elnashai and Mwafy 2002). Though conventional seismic design procedures in all modern seismic codes still adopt force-based design criteria (by reducing the anticipated seismic forces using a single reduction factor, to arrive at the design force level) it may be prudent to control the deformation during the design, which is the major cause of damage and collapse of structures subjected to earthquakes. Hence, a brief discussion about response reduction factor is provided.

The factors influencing seismic design are listed and briefly discussed. The survival of a building during an earthquake depends not only on structural analysis, design, and detailing, but also on architectural considerations. Proper planning is more important than design and detailing; regular structures may survive an earthquake but irregular structures may fail. There are many types of irregularities and these may be grouped as *plan irregularities* and *vertical irregularities*. These irregularities are to be avoided for better performance during earthquakes. Other aspects of planning and design in earthquake zones (for example, location of openings in walls, avoidance of long cantilevers and floating columns) and consideration of the vertical component of earthquakes (for designing horizontal projections and for structures close to the faults or with heavy mass at the top) are also discussed.

Several systems can be adopted to provide adequate resistance to seismic lateral forces. The most common are: moment-resisting frame, a combined system of moment frames and shear walls, braced frames with horizontal diaphragms, and a combination of all these systems. These systems and the concept of strong column and weak beams (which will result in the plastic hinges forming in beams, rather than in columns) are described.

Steel plate shear walls (SPSW) due to the reduced thickness, as compared with concrete shear walls, offer significant advantages in terms of cost, performance, and ease of design and erection. They are considered an alternative to braced frames and can provide equivalent strength and stiffness. National Building Code of Canada (1994), AISC 341-05, and FEMA 450-2004 introduced provisions for the design of SPSW. These provisions are described briefly along with the advantages and drawbacks of SPSW.

For better performance of the structure, the connections between columns and beams should not fail before the beams or columns. Extensive testing on these connections has resulted in three recommendations for their better performance: (a) a toughening scheme, (b) a strengthening scheme, and (c) a weakening scheme. These schemes are discussed along with the pre-qualified seismic moment connections, which are recommended in the AISC 341 code. A brief review of devices that can be used to reduce the effects of earthquakes is also provided.

17.1 RESPONSE REDUCTION FACTOR

As already discussed in Section 3.12 of Chapter 3, the code provisions allow the structure to be damaged in the case of severe shaking. It has to be noted that conventional seismic design procedures adopt force-based design criteria instead of a displacement-based criteria. The basic concept of displacement-based criteria is to design the structure for a target displacement rather than the specified strength (see Priestley et al. 2007 for more details of displacement-based seismic design). Hence, the deformation (which causes damage and collapse of structures during earthquakes), is controlled during the design. However, unfortunately, the traditional concept of reducing the seismic forces using a single reduction factor (called the *response reduction factor*), to arrive at the design force level, is still widely used. This is because structures designed as per this concept have performed satisfactory during full-scale tests and also during earthquakes. This reduction is justified in seismic codes based on reserve strength and ductility (see Fig. 17.1), which improve the capability of the structure to absorb and dissipate energy.

Thus, as per the code methodology, the structure is designed for a seismic force much less than that expected under strong earthquakes, if the structure were to remain linearly elastic. IS 1893(Part 1):2002 provides clauses for the calculation of the probable realistic force to be applied for an elastic structure, and then divides that force by a factor of 2R. [It has to be noted that for important structures the *Probabilistic Seismic Hazard Analysis Map* (PSHA map) of India, found at http://www.hpsdma.nic.in/ should be consulted, instead of the seismic zones map provided in IS 1893 code.] For example, if we consider a structure in zone V, as per IS 1893 (Part 1), $Z = 0.36$ gives a probable indication of the ground acceleration. For $T = 0.3$ s, $S_a/g = 2.5$ as per Fig. 3.24 of Chapter 3, which means that if the building remains elastic, it may experience a maximum horizontal force equal to 90% of its weight ($0.36 \times 2.5 = 0.90$). If we use an R factor of 5 and an importance factor of 1, then we have to design the building for a horizontal force equal to 0.09 times the weight [$0.90 / (2 \times 5)$]. It is clear from this example that the designer is going to design the building for only one-tenth of the maximum elastic force and hence should provide adequate ductility and quality control for good post-yield behaviour. In other words, the term R gives an indication of the level of over-strength and ductility that a structure is expected to have. The intent of the R factor is to simplify the structural design process such that only linearly elastic static analysis (i.e., equivalent static method) is needed for most building design (see Fig. 17.1). It is of interest to note that the R factor could be traced back to the *K factor*, which was used in the first edition of Structural Engineers Association of California's Blue book in 1959 (SEAOC Recommended Lateral Force Requirements and Commentary 1959). R values generally range from 1 for systems that have no ability to provide ductile response to 5 for systems that are capable of highly ductile response.

Thus, as per IS 1893 (Part 1), the structure can be designed for a much lower force than is implied by the strong shaking by considering the following contributing factors, which will prevent the collapse of the structure: over-strength, redundancy, and ductility (see Fig. 17.1). These factors are discussed briefly in the following sections.

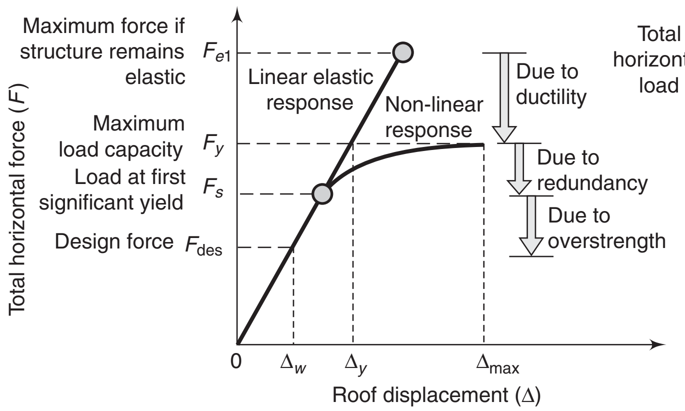


FIG. 17.1 Concept of response reduction factor (Jain and Murty 2004)
Source: Reprinted with permission from NICEE.

17.1.1 Over-strength

The factors that account for the yielding of the structure at loads higher than the design load are as follows.

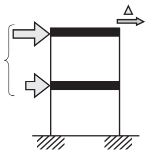
1. Partial safety factors
   (a) Partial safety factor for seismic load
   (b) Partial safety factor for gravity load
   (c) Partial safety factor for materials
2. Material properties
   (a) Provided member size larger than the required size as per design
   (b) Strain hardening in steel resulting in higher strength
   (c) Higher material strength under cyclic load (see Fig. 1.14 in Chapter 1)
3. Strength contribution of non-structural elements such as masonry infills
4. Special ductile detailing, especially at the joints, which also contributes to the increase in strength

17.1.2 Redundancy

It has to be noted that yielding at a location of a structure will not result in collapse. After the first yield, the forces are redistributed and the structure collapses only after forming a *mechanism* (see Section 8.4 of Chapter 8 for details). This type of action in a redundant structure provides an additional safety margin.

17.1.3 Ductility

As defined in Chapter 1, *ductility* is the capacity of materials/structures to absorb energy by deforming into the inelastic range. (Ductility μ may also be defined as the ratio of the ultimate deformation Δ_{max} at an assumed collapse point to the yield deformation Δ_y—see Fig. 17.1). This capacity of the structure to absorb energy, with acceptable deformations and



without failure, is a very desirable characteristic in any earthquake-resistant design. Ductility is often measured by the *hysteretic behaviour* of critical components such as the column-beam assembly of a moment-resisting frame. The hysteretic behaviour is usually predicted by the cyclic moment-rotation or force-deflection behaviour of the assembly as shown in Fig. 17.2. The slope of the curve represents the stiffness of the structure and the enclosed area, the dissipated energy. Perfect ductility is defined by the ideal elastic–perfectly plastic (also called elasto-plastic) curves, which are difficult to achieve in ideal elasto-plastic materials such as steel. *Hysteretic energy* is the energy dissipated by inelastic cyclic deformations and is given by the area within the load deformation curve shown in Fig. 17.2, also called the *hysteretic curve.* Under ideal conditions, *hysteresis loops* of the form shown in Fig. 17.2(a) result, where the energy absorbed will be about 70–80% of that of an equivalent elasto-plastic loop. Limited energy dissipation curves are shown in Fig. 17.2(b). The degradation of strength and stiffness under repeated inelastic cycling is called *low-cycle fatigue.*

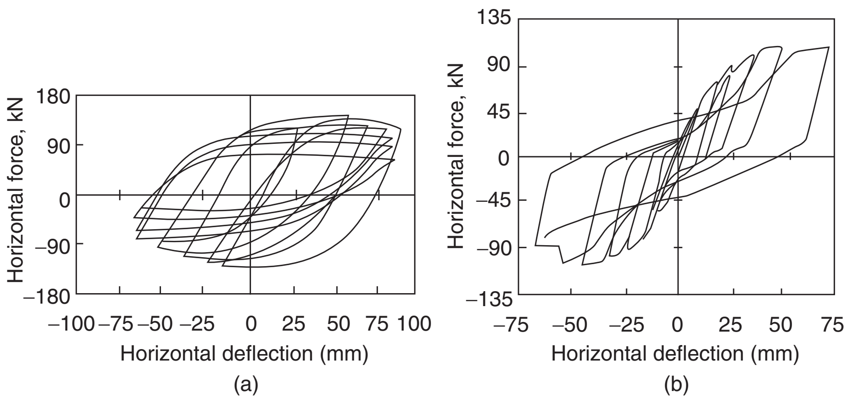


FIG. 17.2 Hysteretic behaviour (a) Curve representing large energy dissipation (b) Curve representing limited energy dissipation

When a structure yields, the following things happen:

1. There is more energy dissipation in the structure due to hysteresis.
2. The structure becomes softer and its natural period increases; due to this it has to resist a lower seismic force (see Fig. 3.24 of Chapter 3).

Thus, higher ductility indicates that the structure can withstand stronger earthquakes without complete collapse. The values prescribed in the code (IS: 1893 and IS: 800) for

R are based on the observed performance of buildings in past earthquakes, expected ductility, over-strength, practice in other countries, and include all the factors discussed in the previous paragraphs (Jain 1995). The *R* values for different types of steel structures are given in Table 3.22 of Chapter 3.

As already indicated, a large area enclosed by the force-deformation loops indicates more dissipation of hysteretic energy. One way of ensuring high ductility and energy dissipation capacity is to use slightly thicker sections thereby avoiding local buckling (however, thick welded sections will result in brittle fracture associated with lamellar tearing). Plastic and compact sections (see Chapter 8) are more ductile than semi-compact and slender sections. Slenderness ratio and axial load ratio of the members may also control ductility. Ductility and energy dissipation capacity are important factors in resisting severe earthquakes (note that these two quantities are interrelated and a large demand on one may tend to decrease the other).

17.2 FACTORS INFLUENCING SEISMIC DAMAGE

The seismic damage caused at a particular site is not only dependent on the earthquake but also influenced by the soil at site and the characteristics of the foundation and structural elements. These parameters influencing seismic damage are listed in Table 17.1.

TABLE 17.1 Parameters influencing seismic damage

Earthquake parameters (Section 3.11.1 of Chapter 3)	Soil at site and foundation parameters	Structural parameters
Amplitude	Soil properties	Natural period of the building
Magnitude	Natural period of the soil	Configuration of building (regular vs irregular)
Duration	Type of foundation	Type of lateral force-resisting system (MRF, braced frame, shear wall, etc.)
Frequency	–	Detailing of joints
Distance of site from epicenter or fault	Geographical conditions between the epicenter and the site(a building built on the top of the ridge may be subjected to intensified shaking)	Construction material (steel, concrete, wood, masonry) and quality control at site

The parameters concerned with the earthquake were discussed briefly in Section 3.11 of Chapter 3 (more details may be found in Bolt 2006). The other factors are discussed in the following subsections.

17.2.1 Influence of Soil Properties and Foundation Types

The seismic motion that reaches a structure on the surface of the earth is influenced by the local soil conditions. Earthquake shaking may be amplified, depending on the intensity of shaking, the nature of the rock and, above all, the surface soil type and depth of soil above the bedrock. [It has to be noted that the soil at site may be classified for earthquake response based on shear wave velocity, standard penetration resistance, and undrained shear strength of soil (see Table C.8 of Appendix C and also Table 3.1 of Eurocode 8 (EN 1998-1-2004)]. Thus, as the building vibrates under ground motion, its acceleration is amplified if the fundamental period of the building coincides with the period of vibration being transmitted through the soil. This amplified response is called *resonance.* [It has to be noted that normal single storey to 20 storey buildings typically have fundamental natural periods in the range of 0.1 to 2.0 s, while the natural periods of soil may be in the range of 0.4 to 2 s, depending on the nature of the soil on ground (see also Table C.7 of Appendix C). Hard ground or rock will experience short period vibration. Very soft ground may have a period of up to 2 seconds but, unlike a structure, it cannot sustain longer period motions except under certain unusual conditions (Arnold 2006). Approximate fundamental natural periods of other structures are as follows: suspension bridges: 6 s, elevated water tanks: 4 s, RC chimneys: 2 s, and large gravity dams: 0.8 s. Taller and more flexible buildings have natural periods in the range of 1–5 s or greater].

Special attention should be paid to soils that have very low values of shear wave velocity, low internal damping, and an abnormally extended range of linear behaviour, since they produce unusual seismic site amplification and soil-structure interaction effects (EN 1998-1:2004). A layer of soft and thicker soil above the bed rock may result in an amplification factor of 1.5–6 over the rock shaking. This amplification is most pronounced at longer periods, and may not be so significant at short periods. The amplification also tends to decrease as the level of shaking increases (Arnold 2006). As a result, earthquake damage tends to be more severe in areas of soft ground, as in the 1906 San Francisco earthquake, the 1989 Loma Prieta earthquake (Northern California), and the 1985 Mexico City earthquake (Mexico City's downtown area is about 400 km away from the 8.1-magnitude earthquake epicenter, and had silt and volcanic clay sediments of the bed of the historic Lake Texcoco, which were between 7–37 m deep and had a high water content. The earthquake caused the soft ground under the downtown buildings to vibrate for over 90 seconds at its long natural period of around 2 seconds. This caused buildings that were between about 6–20 storeys in height to resonate at a similar period, greatly increasing the accelerations within them. Taller buildings suffered little damage).

Soil liquefaction is another effect caused by earthquakes. A saturated, uniform, fine grained sand or silt, when subjected to repeated vibration, experiences an increase in pore water pressure due to a redistribution of its particles, with a consequent reduction in shear strength. This produces a 'quick sand' type condition, with a loss of bearing capacity, causing settlement and collapse of structures.

For better seismic response, proper precautions have to be taken at the planning stage itself. It is preferable to select a site where bedrock is available close to the surface, so that foundations can be laid directly on the rock. The differential movement of foundation due to seismic motions is an important cause of structural damage, especially in heavy, rigid structures that cannot accommodate such movements. Hence, if the foundation is on soft soil with spread footings, adequate plinth or tie beams should be provided to counter differential settlement. If the loads are heavy, pile foundations with strong pile caps may be provided. A raft foundation is ideal for resisting differential settlements, but may prove to be expensive. In sandy or silty soils, if the water table is near the foundation level, the following methods may be adopted to prevent liquefaction.

1. Drainage may be installed to lower the ground water table and remove the pore water (it should be checked weather the resulting settlement will not affect adjacent structures).
2. A porous overburden may be placed over the site to produce over-consolidation, which results in increased pore pressures being required before liquefaction can occur.
3. If there are no adjacent structures, pre-consolidation of the soil may be achieved by vibro-flotation techniques.
4. In order to increase the shear strength of soil, soil grouting or chemical injection may be employed.
5. All deleterious soil may be removed and replaced with better soil.
6. Pile foundation may be employed with piles extending and resting on to a sound soil layer below the unsatisfactory layer.

Appendix F of the draft IS:1893 gives a simplified procedure for evaluating the liquefaction potential of a soil deposit.

Different types of foundation in the same structure, foundations on different soil types for the same structure, and foundations at different levels (such as those found in hill slopes) should be avoided for better seismic performance.

17.3 RULES TO BE FOLLOWED FOR BUILDINGS IN SEISMIC AREAS

To perform well in an earthquake, a building should possess the following four main attributes: (a) simple and regular configuration, (b) adequate lateral strength, (c) adequate stiffness, and (d) adequate ductility. Thus, structural planning of steel buildings should be done in such a way that the beams yield prior to the columns, the strength of connections should be greater than the strength of beams and columns framing into the connection members, the connections should guarantee high strength, ductility, energy dissipation capacity, and excessive lateral sway is avoided.

Buildings with a simple regular geometry and uniformly distributed mass and stiffness in plan and elevation (regular structures), as shown in Fig. 17.3, have been found to suffer less damage in earthquakes than those with irregular structures. Hence, columns and walls should be arranged in grid fashion and should not be staggered in plan. The effect of asymmetry will induce torsional oscillations in structures and stress concentrations at re-entrant corners. Irregularities may be grouped as *plan irregularities* and *vertical irregularities*.

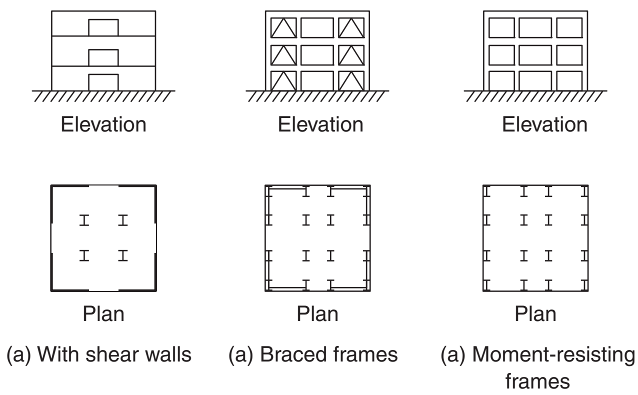


FIG. 17.3 Buildings with regular configuration

17.4 PLAN IRREGULARITIES

The several plan irregularities that should be carefully considered and avoided while designing buildings in seismic areas are briefly discussed in this section.

17.4.1 Irregularity due to Re-entrant Corners

Buildings with re-entrant corners (i.e., plans in the shape of L, H, V, +, Y, W, or any other letter, except the O-shape) should be avoided. Projection of the structure beyond any re-entrant corner should not be greater than 15% of its plan dimension in the given direction (see Fig. 17.4). It will be advantageous to split such plans into separate rectangles by using *seismic joints* at junctions of the individual wings. (Fisher 2005 and Saunders 2005)

17.4.2 Torsional Irregularity

When floor diaphragms (a *diaphragm* is a horizontal system that transmits lateral forces to the vertical resisting elements; for example, reinforced concrete floors or horizontal bracing systems that transmit lateral forces to the columns)

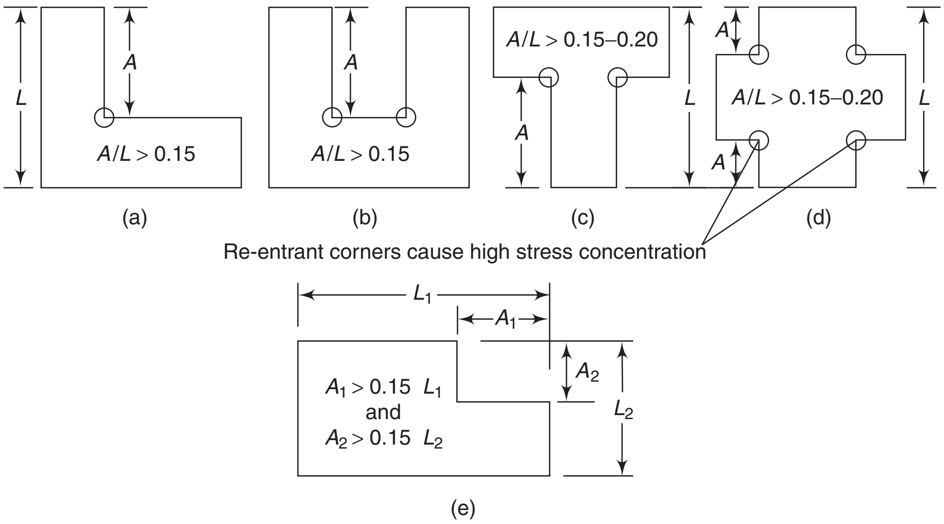


FIG. 17.4 Plan irregularities

cause severe damage (or even collapse) of structures, is provided by Özmen et al. (2014).

As shown in Fig. 17.5(b), seismic force-resisting systems, such as shear walls should not be placed asymmetrically or the masses (for example, heavy swimming pools at the top of buildings) should not be placed eccentrically. Torsion should be minimized by making the building symmetrical and regular in geometry and stiffness, and by providing lateral load-resisting elements at the building's perimeter as shown in Fig. 17.5(a).

It has to be noted that provisions are not available in the Indian codes (IS 456:2000 or IS 13920: 1993) for the design of diaphragms. Some guidelines and discussions are provided by Subramanian (2013) [see Section 10.10 of the book]. Various researchers have identified that the commonly employed *equivalent static analysis method* for the design of diaphragms, under-estimates the acceleration of floors, particularly in the lower levels of the buildings (see for example, Gardiner, et al. 2008).

are considered rigid in their own plane in relation to the vertical structural elements that resist the lateral forces, torsional irregularity must be considered. Torsion is present whenever there is an eccentricity between the *center of mass* (CM) and the *center of rigidity* (CR), also called the center of stiffness. When the maximum storey drift, computed with design eccentricity at one end of a structure transverse to an axis, is less than 1.2 times the average of the storey drifts at the two ends of the structure, torsional irregularity need not be considered. Thus, in the building plan shown in Fig. 17.5(c), the drift Δ_2 should not be greater than $1.2[(\Delta_1 + \Delta_2)/2]$. If Δ_2 is greater than $1.4[(\Delta_1 + \Delta_2)/2]$, it is considered to be a case of extreme torsional irregularity (in the Canadian code the factor is 1.7 instead of 1.4). More discussions on torsional irregularity, which is one of the most important factors and may

17.4.3 Diaphragm Discontinuity

The roof/floor deck or slabs respond to lateral loads like a deep beam. In the American code (ACI 318:2014), it is assumed that the deck or floor slab acts as the web of a continuous beam carrying shear, and the perimeter spandrel beams or walls act as the compression and tension chords (flanges)

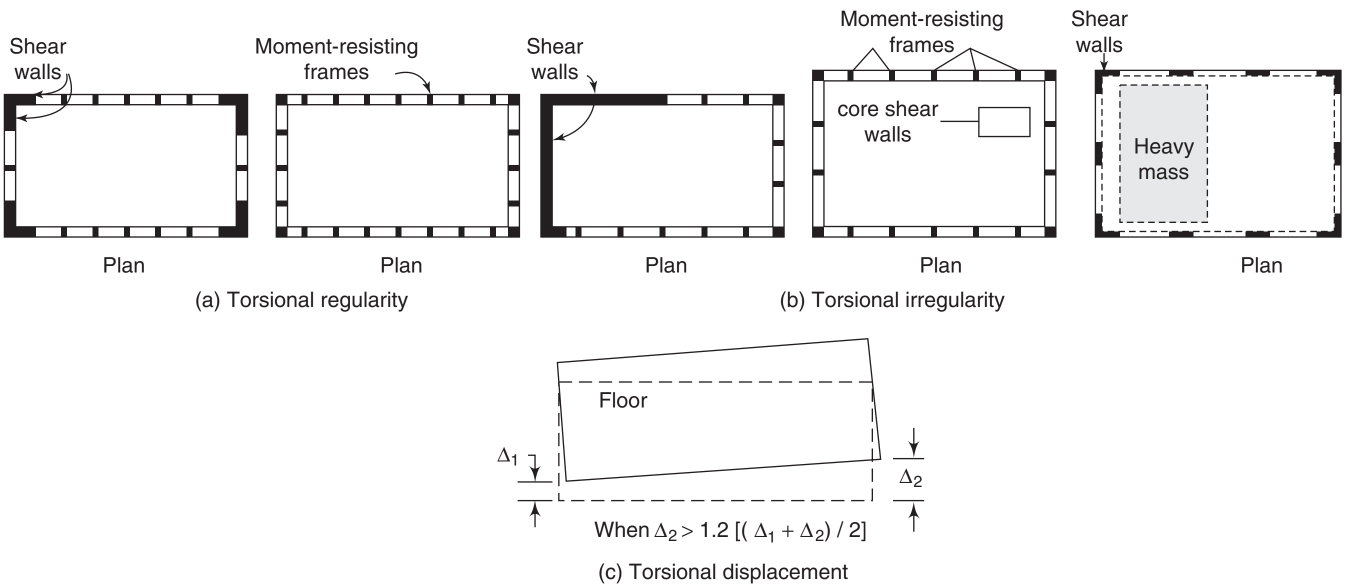


FIG. 17.5 Torsional irregularity

of the continuous beam. Thus, inappropriate location or large size of openings (due to stair or elevator cores, atriums, skylights, etc.) creates problems similar to those related to cutting a hole in the web of a beam. Such openings reduce the ability of the diaphragm to transfer the load and may cause failure.

Hence, for diaphragms with abrupt discontinuities or variations in stiffness, the cut outs or open areas as shown in Fig. 17.6 should not be greater than 50% of the gross enclosed diaphragm area. Similarly, there should not be changes in the effective diaphragm stiffness of more than 50% from one storey to the next. This is because diaphragm discontinuity changes the lateral load distribution to different elements, in contrast to a rigid floor diaphragm.

More details about the analysis, design, and constructional aspects of diaphragms may be found in the ACI code (Section 21.11), Thomas and Sengupta (2008), and Moehle et al. (2010). Details of the analysis of chord forces for slabs with large openings may be found in Sengupta and Shetty (2011).

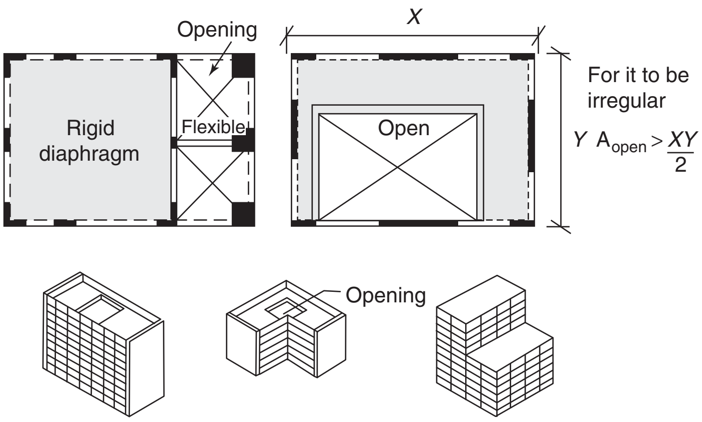


FIG. 17.6 Diaphragm discontinuity

17.4.4 Out of Plane Offsets

Out-of-plane offset as shown in Fig. 17.7 is a serious irregularity, since it imposes excessive vertical and lateral load effects on horizontal elements. It is recommended to carry all shear walls down to the foundation level without any offsets, especially in places that may experience moderate to severe earthquakes. When such an arrangement is adopted, it is very important to provide adequate stiffness to the columns supporting the upper storey shear walls and also confining reinforcement throughout the reinforced concrete columns, extending beyond the beam-column joints.

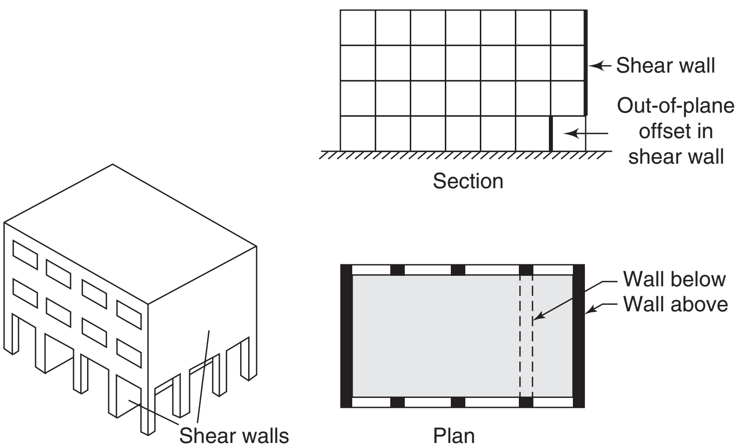


FIG. 17.7 Out-of-plane offsets

17.4.5 Non-parallel System

The vertical elements resisting the lateral forces should be parallel to or symmetrical about the major orthogonal axes or the lateral force resisting elements. When they are not parallel as shown in Fig. 17.8, additional load combinations are necessary (see also Section 3.15.1 of Chapter 3).

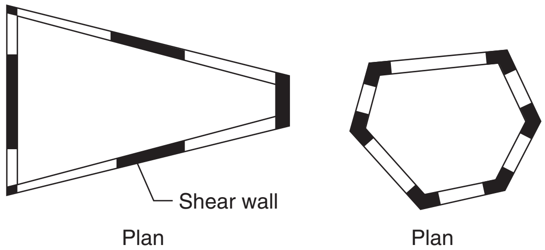


FIG. 17.8 Non-parallel systems

17.5 VERTICAL IRREGULARITIES

Similar to horizontal irregularities, various kinds of vertical irregularities should also be carefully considered and avoided in seismic zones. Some of these vertical irregularities are briefly discussed in the following subsections.

17.5.1 Stiffness Irregularity

Omitting the exterior walls or all the walls for parking lots in the ground floor leaves all the columns at the ground level as the only elements available to resist lateral forces, thus causing an abrupt change in stiffness at that level. In general, a storey with lateral stiffness less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above is considered a *soft storey* (see Fig. 17.9).

Buildings on stilts may have lateral stiffness less than 60% of that in the storey above or less than 70% of the average stiffness of the three storeys above and hence considered to have an *extreme soft storey*. Soft-storey buildings are known for their poor performance during earthquakes and many buildings have collapsed during the 2001 Bhuj earthquake. During California's Loma Prieta earthquake of 1989, nearly half of all homes became uninhabitable due to soft-storey failure.

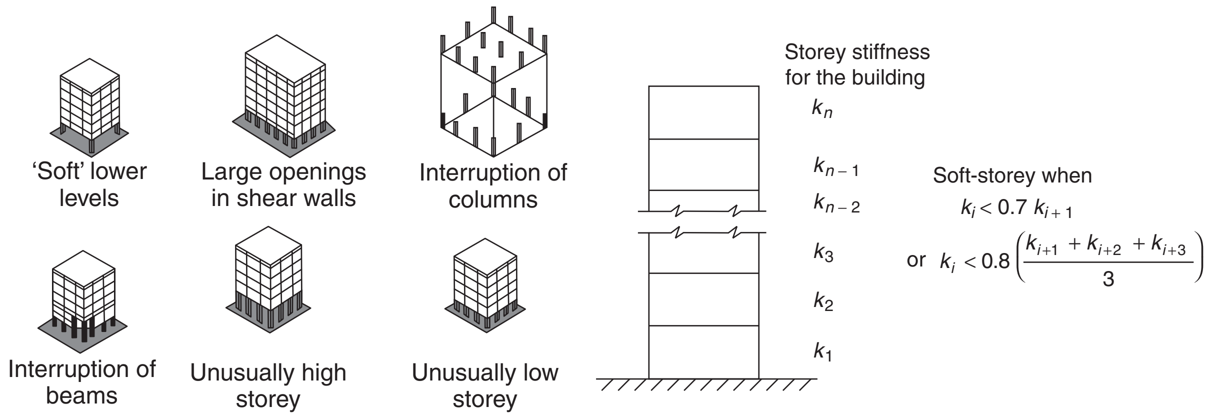


FIG. 17.9 Stiffness irregularity

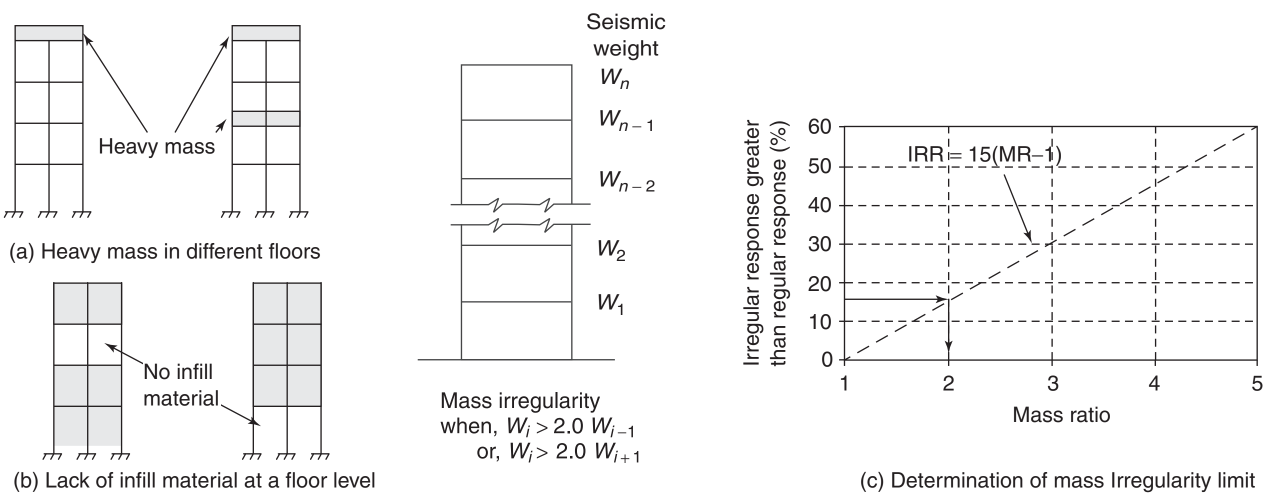
17.5.2 Mass Irregularity

Mass irregularity is induced by the presence of a heavy mass on a floor, for example, a roof-top swimming pool, a service floor having water tanks and heavy equipment for air conditioning and/or back-up power generator, as shown in Fig. 17.10. IS 1893 suggests that the mass irregularity should be considered when the seismic weight of any floor is more than 200% of its adjacent floors [NZS 1170.5 and the 2005 edition of the National Building Code of Canada (NBCC) consider 150% instead of 200%]. However, the code suggests that a roof that is lighter than the floor below need not be considered.

It has to be noted that this limit of 1.5–2.0 times the seismic weight for mass irregularity and also other limits for other types of irregularities have been specified in codes from engineering judgment and not based on rigorous quantitative analysis. A more meaningful comparison will be obtained if structures designed to a target drift are compared with the actual drift demand. A study by Sadashiva et al. (2009) found that the increased mass, when present at either the first floor level or at the roof, produced higher drift demands than when located at the mid-height. They also proposed a simple equation as shown in Fig. 17.10(c), where IRR represents the irregular response greater than regular response; and MR is the mass ratio. Thus, if it is decided that the mass irregularity should not produce more than 15% additional inter-storey drift, then as per Fig. 17.10(c), the mass ratio should be restricted to be less than 2.

17.5.3 Vertical Geometric Irregularity

Buildings with vertical offsets (e.g., setback buildings) as shown in Fig. 17.11, are not allowed, especially when a larger dimension is above the smaller dimension, as it acts like an inverted pyramid. Vertical geometric irregularity is considered to exist where the horizontal dimension of the lateral force-resisting system in any storey is more than 150% (130% in the Canadian code) of that in an adjacent storey.



(a) Heavy mass in different floors

(b) Lack of infill material at a floor level

(c) Determination of mass Irregularity limit

FIG. 17.10 Mass irregularity

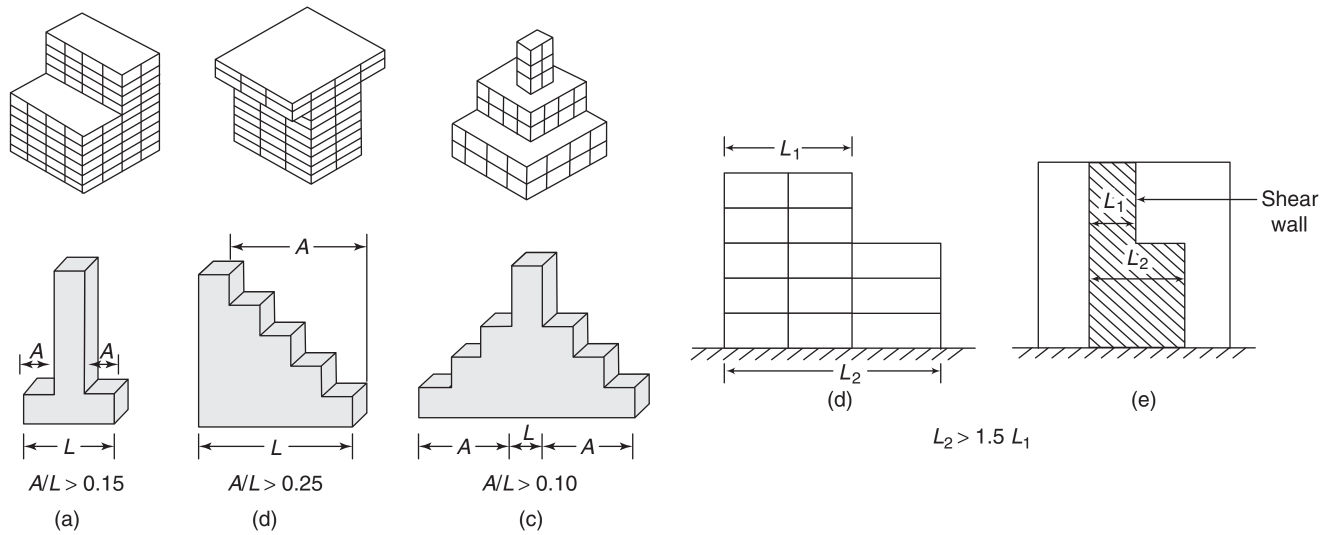


FIG. 17.11 Vertical geometric irregularity

17.5.4 In-plane Discontinuity in Vertical, Lateral Force-resisting Elements

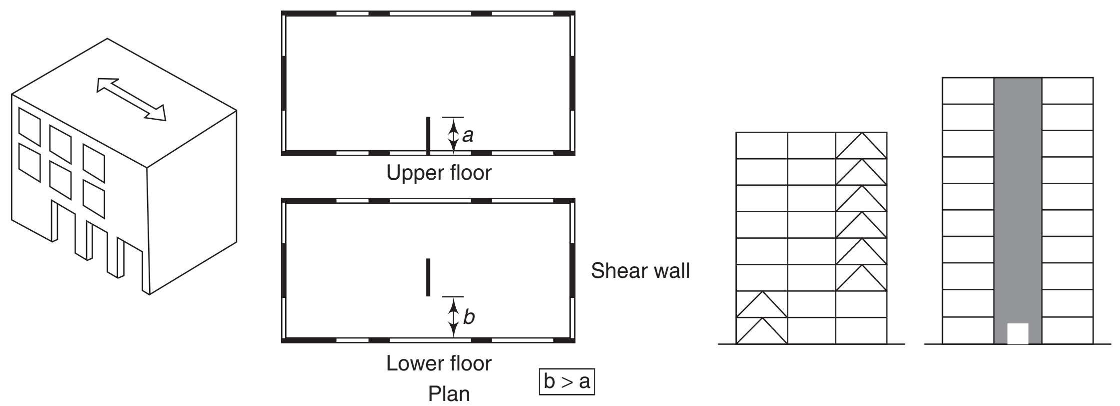
In-plane discontinuity in vertical, lateral force-resisting elements is considered to exist when there is an in-plane offset of a lateral load-resisting element (which is greater than the length of these elements) of the lateral force-resisting system or a reduction in lateral stiffness of the resisting element in the storey below (see Fig. 17.12).

17.5.5 Discontinuity in Capacity (Weak Storey)

Weak-storey configuration is often used in multi-storey apartment buildings and hospital buildings, in which not only the first floor is designed without walls (for using it as car parking), but also has a greater height than the rest of the floors. This irregularity can also be present at the first floor or at other intermediate floors (to provide meeting halls, restaurants, or machine rooms). There are numerous examples of many buildings presenting a combination of these types of irregularities, soft and weak storey, making them seismically vulnerable.

It has to be noted that the infilled brick walls in the upper storeys increase the lateral stiffness of the frame by a factor of three to four times than that of lower weak storey. This makes the much stiffer upper storeys to behave like a rigid block, and most of the horizontal displacement of the building occurs locally in the soft storey alone. Thus, the dynamic ductility demand during an earthquake gets concentrated in the soft storey and the upper storeys tend to remain elastic; the severely strained 'soft' storey causes total collapse of the building and much smaller damages occur in the upper storeys (unless the building collapses to the ground). In several past earthquakes the main cause of collapse is the ground floor soft storey (Bachmann 2002).

According to IS 1893 (Part 1):2002, a storey may be considered as a weak storey when the lateral strength of that



(a) In-plane offset of an element of the lateral force resisting system

(b) Reduction in lateral stiffness of an element in the storey below

FIG. 17.12 In-plane discontinuity in vertical, lateral force-resisting elements storey is less than 80% of the storey above (see Fig. 17.13). The storey lateral strength may be calculated as the total strength of all seismic force resisting elements sharing the storey shear in the considered direction. This irregularity is similar to that encountered in a soft storey, and may be taken care of by making the storey stiffer in comparison to the adjacent storeys.

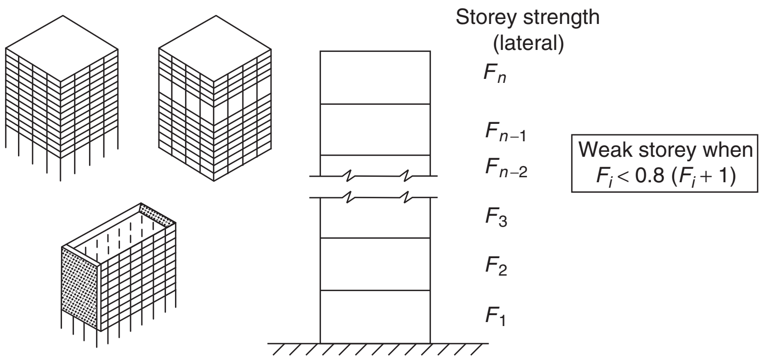


FIG. 17.13 Weak storey

More details about the architectural planning of buildings in order to reduce earthquake damage are provided by Arnold (2006), Arnold and Raitherman (1982), Ambrose and Vergun (1997), Bachmann 2002, FEMA 454 (2006), Mathews et al. (1977), Murty (*Earthquake Tips* 2004), and Subramanian (1994).

The irregularities discussed till now are found to have a detrimental effect on building behaviour—some of them affect the behaviour considerably and may lead to complete collapse. Stiffness or weight irregularities and torsionally stiff irregular buildings can be dealt with by using a dynamic analysis—which will overcome the shortcomings of applying a static load distribution that is based on a 'first mode' shape for uniform buildings. Torsionally flexile buildings can be dealt with by analysing the structure for an additional torsional moment load case. In-plane or out-of-plane offsets of the lateral load-resisting system (especially with offset walls), are more serious and a dynamic three-dimensional analysis may improve the force distribution, but may not solve the potential overload of the supporting structures, large force transfers in and out of the elements at the discontinuities, and large force transfers through the diaphragms (DeVall 2003). Hence, offsets or discontinuities of walls in tall buildings in high seismic zones are banned in the Canadian code. Similarly, the dangerous weak storeys are also banned in most seismic zones (DeVall 2003).

As per IS 1893 (Part 1): 2002, dynamic analysis must be performed for irregular buildings and regular buildings when they exceed certain heights in different earthquake zones (see Table 3.20 of Chapter 3). As per the Canadian code, equivalent static force procedure may be used for structures in any of the following situations: (a) relatively low seismic hazard, as defined by the short-period (0.2 s) design spectral acceleration; (b) regular structures less than 60 m in height and with fundamental lateral period less than 2 s; or (c) certain irregular structures less than 20 m in height and with fundamental lateral period less than 0.5s. For all other structures, especially irregular structures, dynamic analysis should be used.

17.6 OTHER ASPECTS OF PLANNING AND DESIGN IN EARTHQUAKE ZONES

The openings in walls should be located centrally and should be of small size so that the walls are not unduly weakened. Ventilators provided near the edges of walls, adjacent to columns, will create a short column effect and result in the failure of the columns. There will be a similar effect if openings are provided from column to column. Since the infill masonry wall is much stiffer than the columns, column hinges form in the column at the top as well as at the top of the infill masonry wall rather than at the top and bottom of the column (see Fig. 17.14). If the column flexural capacity is M_p, the shear in the columns increases by the factor H/h, where H is the height of the column and h is the height of infill masonry wall, resulting in non-ductile failure of the columns; several failures in the past earthquakes may be attributed to this effect (FEMA P750:2009, Bachmann 2002).

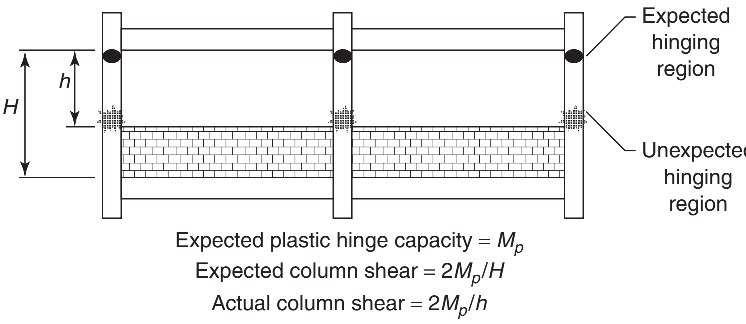


FIG. 17.14 Short-column effect (FEMA P750:2009)

Long cantilevers and floating columns should be avoided, as they have found to fail during earthquakes. Clause 7.12.2 of IS 1893 (Part 1):2002 states that vertical cantilever projections such as water tanks, parapets, and smoke chimneys projecting above the roof of buildings should be designed for five times the horizontal seismic coefficient, A_h, discussed in Section 3.12 (see Eqn 3.28) of Chapter 3. Similarly horizontal cantilever projections such as sunshades (*chhajjas*) and balconies should be designed for five times the *vertical seismic coefficient*. Clause 7.12.2 of IS 1893 (Part 1):2002 suggests that this value may be taken as $10A_h/3$. An excellent introduction to the earthquake protection of non-structural elements in buildings is provided by Murty et al. 2013b.

Concrete stairways often suffer seismic damage due to their inhibition of drift between connected floors. This can be avoided by providing a slip joint at the lower end of each stairway to eliminate the bracing effect of the stairway or by tying stairways to stairway shear walls [see Chapter 17 of Subramanian (2013) for more details].

Masonry and infill (non-structural) walls should be reinforced by vertical and horizontal reinforcing bands to avoid their failure under a severe earthquake. Other non-structural elements should be carefully detailed or tied so that they may not fall under severe shaking. Murty et al. (2013b) and FEMA 454 (see Chapter 9) provide information on earthquake protection of non-structural elements in buildings.

It has to be noted that failure of a beam causes localized effect whereas that of a column may affect the stability of the whole building. Hence it makes good sense to make columns stronger than beams. This can be achieved by appropriate sizing of the member and detailing. This concept is called *strong-column, weak-beam concept* (see Section 17.8.2).

When buildings are too close to each other, they may pound on each other. Connections and bridges between buildings should be avoided and buildings with different sizes and shapes should be adequately spaced. When building heights do not match, the roof of the shorter building may pound the mid-height of the columns of the taller one, resulting in dangerous consequences. The buildings or two adjacent units of the same building should be separated by a distance equal to R times the sum of the calculated storey displacements to avoid pounding (for the value of R, see Table 3.22 of Chapter 3). As per clause 7.11.3 of IS 1893 (Part 1): 2002, this value may be multiplied by a factor of 0.5 if the two units have same floor elevation.

17.6.1 Consideration of Vertical Component of Earthquake

Clause 6.1.1 of IS 1893 (Part 1):2002 suggests considering the vertical component of earthquake, in structures with large spans, those in which stability is a criterion for design or for overall stability analysis of structures. It also specifies that that special attention should be paid to the effect of vertical component of earthquake on pre-stressed or cantilever beams, girders, and slabs. As discussed in the previous section, IS 1893 suggests that the vertical component be taken as 2/3rd of the peak horizontal component. It has to be noted that the characteristics of the vertical component of earthquake are significantly different than those of the horizontal component. Observations from the damage patterns of the 1989 Loma Prieta, 1994 Northridge, 1995 Kobe, 1999 Chi Chi, 2010 Darfield, and 2011 Christchurch (where a vertical PGA of 2.21 was recorded) earthquakes emphasize the significance of vertical seismic effects, especially in near-field conditions. It was also found that the vertical-to-horizontal (V/H) spectral ratio is a strong function of natural period, source-to-site distance, and local site conditions (Dana et al. 2014). Hence, their effects cannot be ignored in design, especially for structures close to the faults or with heavy mass concentrated at the top, such as huge water tanks, bridges with heavy decks etc., where heavy damages are expected to the lower part of the structure.

The earthquake records show that the vertical component is richer in high frequency content than the horizontal component. This results in high vertical response spectral ordinates at short periods, especially at sites close to the fault. High vertical spectral acceleration at short periods can affect structural systems and components that have short vertical natural periods. In fact, based on the recorded structural response of twelve instrumented structures, Bozorgnia et al. (1998) identified that the vertical natural periods of several structural systems and components fall in the range of 0.075 to 0.26 s. Thus, a period-independent ratio of 2/3, adopted in several codes, is a grossly unconservative approximation of the V/H spectral ratio at short periods, and is a relatively conservative approximation at long periods, especially for sites with firm soil located near active faults (Bozorgnia and Campbell 2004a and 2004b).

Several researchers have concluded that certain compressive, tensile, shear, and flexural failures in structural members may be due to the high values of seismic vertical forces. In steel structures subject to near-field ground motions, the vertical ground motion may induce ultra-low cycle fatigue in connections. Furthermore, vertical frequencies in buildings are not influenced by changes in height or lateral stiffness of buildings. This can lead to resonance and significant component demands when the fundamental vertical frequencies of the structures match the vertical pulses of ground motions (Dana et al. 2014). Recent earthquakes and studies have shown that the vertical-to-horizontal (V/H) spectral ratios can reach a value of 1.7 for short periods and 0.7 for long periods. This implies that the commonly adopted code specified ratio of V/H = 2/3 is far exceeded, especially in the short-period range. This effect is pronounced in near-fields of high-frequency ground motions and in unconsolidated soil environments; hence in such situations, site specific spectra has to be used in the design for vertical ground motions (Dana et al. 2014).

Recent research has also shown that the vertical components of earthquakes have less pronounced effects in the perimeter/corner columns than in the interior columns. Perimeter/corner columns receive more forces from seismic horizontal components than those of interior columns as the perimeter/corner columns provide resistance to overturning (Dana et al., 2014). In addition, the contribution of gravity forces is larger for interior columns since the effect of overturning is negligible at interior columns. It has also been found that the vertical component of earthquake affects the forces in the upper storey columns than in the lower storey columns. A similar pattern was observed in beams. The effects due to the vertical components of earthquake are less in beams that have smaller forces due to horizontal component and having smaller tributary areas. The effects of the vertical component of earthquake were found to be more pronounced in the interior beams, upper storey beams and the long span beams. Hence care should be taken while checking the upper storey beams and columns of multi-storey buildings, especially at sites close to the fault.

17.7 SEISMIC FORCE RESISTING SYSTEMS

Several systems can be adopted to provide adequate resistance to seismic lateral forces. The most common systems are moment-resisting frames (though they consist of a three-dimensional space frame, for analysis purposes, they may be considered as two-dimensional in several cases); a combined system of moment frames and shear walls; braced frames with horizontal diaphragms; and a combination of these systems (see Fig. 17.15 and Table 17.2). Out of these, moment-resisting frames may be economical for buildings with only up to 30 storeys (the infill walls of non-reinforced masonry also provide some stiffness). Shear wall and braced systems (which are more rigid than moment-resisting frames) are economical up to 40 storeys (see also Fig. 2.15 of Chapter 2). When frames and shear walls are combined, the system is called *dual system.* A moment-resisting frame when provided with specified details for increasing the ductility and energy absorbing capacity of its components is called a *special moment-resisting frame (SMRF)* or *special moment frame* (*SMF*); otherwise it is called an *ordinary moment-resisting frame (OMRF)* or *ordinary moment frame (OMF).* Similarly, braced frames may be classified as *ordinary concentrically braced frames* (*OCBF*), *special concentrically braced frames* (*SCBF*), and *eccentrically braced frames* (*EBF*).

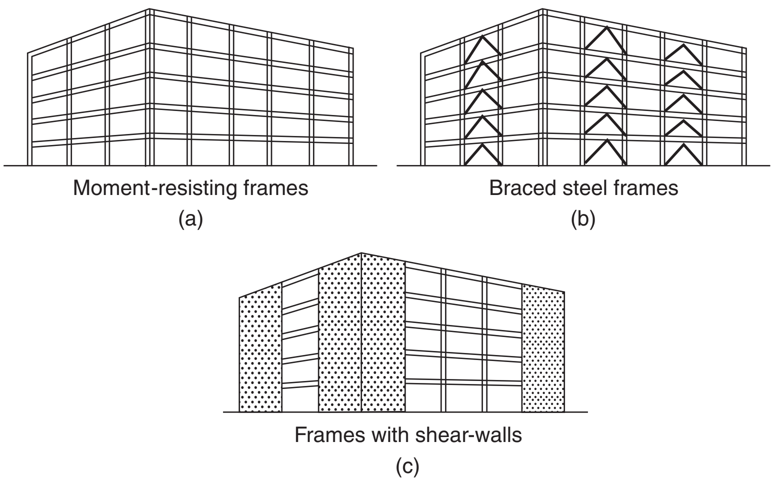


FIG. 17.15 Lateral force-resisting systems

The design engineer should not consider the structure as composed of a summation of different parts (such as beams, columns, trusses, and walls) but as a completely integrated system, which has its own properties with respect to lateral force response. Thus, he or she should follow the flow of forces through the structure into the foundation and make sure that every connection along the path of forces is adequate to maintain the integrity of the system. It is also necessary to provide adequate redundancy in the structure. When a primary system yields or fails, the redundancy allows the lateral forces to be redistributed to a secondary system to prevent *progressive collapse*. It has to be realized that the forces due to earthquake are not static but dynamic, (cyclic and repetitive) and hence the deformations will be well beyond those determined from the elastic design.

Recently, innovative design concepts have been developed to better protect structures, together with their occupants and contents, from the damaging effects of destructive environmental forces including those due to winds, waves and earthquakes. *Base isolation* is a passive structural control system, pioneered in New Zealand by Dr Bill Robinson during the 1970s, where some isolators are used to substantially decouple the building from its foundations resting on shaking grounds, thus protecting the structural integrity of the building. There are four types of base isolation – elastomeric, sliding, rolling, and a combination of these. With these types of base isolation, the drift of the building during an earthquake is greatly reduced.

Another concept is the use of *passive energy dissipation devices* or *dampers* which absorb or consume a portion of the input energy, thereby reducing energy dissipation demand on primary structural members and minimizing possible structural damage. Devices that are commonly used for seismic protection of structures include viscous fluid dampers, viscoelastic solid dampers, friction dampers and metallic dampers [Soong and Dargush (1997) and Symans et al. (2008)]. These innovative devices are briefly discussed in Section 17.14.

An important aspect of earthquake-resistant design is to understand the impact of earthquake on the structure, and the damage it suffers. While less building damage will result in low repair costs, significant damage (or collapse) will result in higher cost of repair/replacement and also significant time for repair. Thus, a successful seismic design should result in the selection of a structure that is not only economical but also suffer minimum damage with corresponding low post-earthquake repair cost. The behavior of each structural system will differ with the type and duration of earthquake and soil types (FEMA 454:2006). Systems with stable cyclic behavior, good energy dissipation, and controlled inter-storey drift will yield low repair costs (Fig. 17.16).

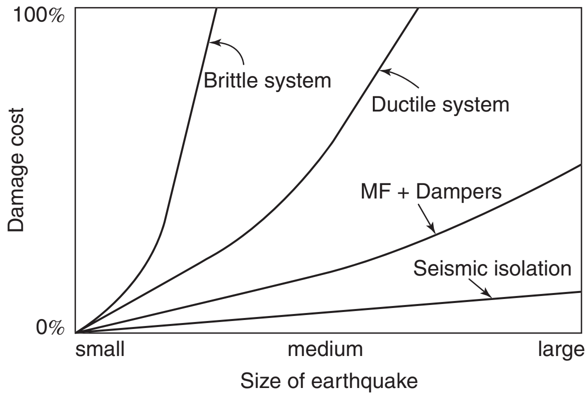


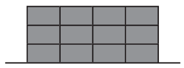
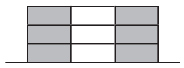
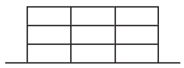
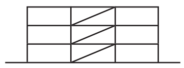
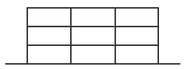
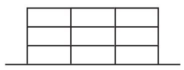
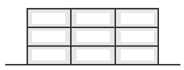
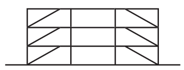
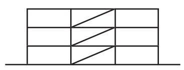
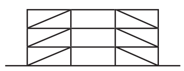
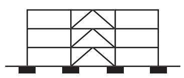
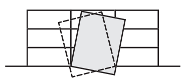
FIG. 17.16 Performance of different systems during earthquakes

Table 17.2 compares the different seismic system strategies discussed in this section (FEMA 454:2006). Some reinforced concrete systems are also included for comparison. The response reduction factors shown in the last column of the table are from IS 1893 (Part 1): 2002 and IS 800: 2007 (The values of *R* shown in brackets are from ASCE 7-10-They are not to be compared directly with the values given in Indian codes, as the formulation in these codes are different). It has to be noted that ASCE 7-10 provides *R* values for 85 different building systems, whereas IS 1893 (Part 1): 2002 provides only for 14 systems.

17.8 MOMENT-RESISTING FRAMES (MRFs)

Although steel 'special' moment frame is a relatively new concept used in building codes, steel moment frames have been used for more than one hundred years. The 10-storey,

TABLE 17.2 Comparison of different seismic system characteristics

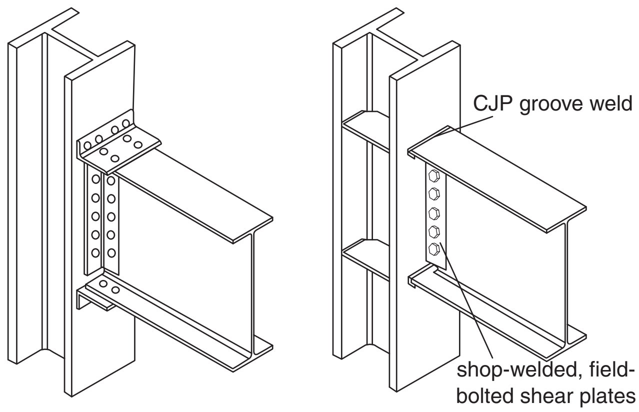
System		Non-linear drift	Cyclic behaviour	Energy dissipation	Post EQ repair cost	Response reduction factor, R (ASCE 7-10)
Steel frame with unreinforced masonry wall		Medium	Stable	Medium	Medium	1.5 (3)
Steel frame + *R*C shear walls		Medium	Stable	Medium	Medium	3–4 (6–7)
Non-ductile *R*C MF		Large to collapse	Unstable	Low	High	3 (3)
Steel concentrically braced frame		Large to collapse	Unstable	Low	High	4.5 (3.25–6)
*R*C Shear walls		Medium	Stable to Unstable	Medium to high	Medium to high	3–4 (5–6)
Steel OMF		Medium to Large	Unstable	Medium	High	4 (3.5)
Steel SMF		Medium to Large	Stable to semi-stable	Medium	Medium to high	5 (8)
Ductile *R*C MF		Medium to Large	Stable to semi-stable	Medium	Medium to high	5 (8)
Steel EBF		Low to medium	Stable	Medium to high	Low to medium	5 (8)
Damper + steel MF		Low to medium	Stable	Very high to high	Low	–(8.5*)
Buckling restrained braced frame		Medium	Stable	Medium to high	Low	– (8)
Seismic isolation		Low inter-storey drift	Stable	Very high	Very low	– (1.6–2.0*)
Rocking system		Large rocking motion	Stable	High	Low	– (8.5*)

*suggested in FEMA 454:2006

42 m tall Home Insurance Building in Chicago, built in 1884, is considered the first skyscraper with steel moment frames. In just 28 years, skyscraper technology progressed to the 60-storey Woolworth Building in New York. This and other tall buildings, built till the 1900s, were constructed with built up H-sections and the steel framing, at their perimeters, were completely encased by masonry, concrete, or a combination of both, to provide fire resistance. However, the designers of these early moment frame structures neglected the contribution of concrete/masonry encasements, which substantially stiffened and strengthened the steel framing and provided significant fixity at the framing connections for both gravity and lateral loads. By the early 1900s, the built-up sections were replaced by rolled H-sections (a good example being the Empire State Building in New York, USA; see Fig. 1.43(d) of Chapter 1). Most or all of the columns, interior and exterior, were part of the lateral load-resisting system. The weight of the walls, cladding, and floor systems of these buildings were so great that their column design was controlled by the gravity loads and not by the lateral loads.

The buildings built after World War II adopted modern glass and aluminum curtain wall systems instead of the infill perimeter masonry walls, for reasons of economy. These frames had connections without gussets, using angles or split tees to connect the top and bottom flanges to the steel H-columns, as shown in Fig. 17.17(a). When welding was introduced in the 1950s, the angles and split tees were replaced by flange plates that were shop welded to the column flanges, then riveted to the beam flanges (rivets were replaced by high strength bolts in the 1960s). During 1970–1994, designers used the connection as shown in 17.17(b), which had field-welded, complete joint penetration (CJP) groove welds to connect beam flanges to columns, and shop-welded, field-bolted shear plates joining beam webs to columns.

Earlier research by Professor Egor Popov at the University of California at Berkeley in the 1960s and 1970s showed that in order to obtain superior inelastic behavior of steel moment-resisting frames in strong earthquakes, it is necessary to adopt proper proportioning and detailing of these frames. These design criteria were introduced in the 1988 Uniform Building Code, and the frames designed as per these criteria were designated as *special moment-resisting space frames*, and were assigned the highest *R* factor. It is important to realize that after the 1980s, engineers adopted designs that minimized expensive site welding and economized their designs. This resulted in using fewer bays of moment-resisting frames, which had heavier beams and columns. In some extreme cases, tall structures were provided with only a single bay of moment-resisting framing on each side of the building, resulting in less redundant structures with more concentrated lateral force resistance (Hamburger et al. 2009).



(a) Riveted, unstiffened seat angle connection (b) Welded unreinforced flange-bolted web connection used during 1970-1994

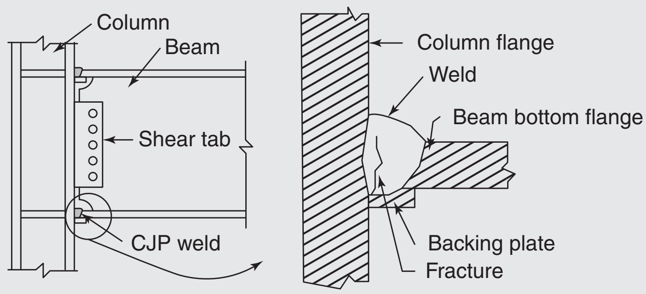
FIG 17.17 Earlier connections used in moment-resistant frames (Hamburger et al. 2009)

CASE STUDY

Damages during 1994 Northridge Earthquake

After the 1994 Northridge earthquake (Los Angeles, USA), engineers found that the welded moment connections in the special moment-resisting frames of over 200 buildings experienced brittle fractures in this magnitude 6.7 earthquake [Gioncu and Mazzolani (2002), Yang and Popov (1997)]. Until this earthquake, many engineers regarded these buildings as highly resistant to earthquake damage. These beam-to-column connections experienced rotation levels well below the plastic moment capacity of framing members, but still suffered damages. (None of these steel frame buildings collapsed, but the unexpected type and severity of damage proved the inadequacy of the building code provisions available at that time). Most of the failures occurred in the upper half to two-thirds of tall buildings and at all levels in low-rise buildings (fewer than six storeys).

The joints were designed assuming that the top and bottom flanges resisted the bending moments and the shear was resisted by the web connection, ignoring any eccentricity in the connection. Horizontal stiffeners (also called as *continuity plates*) were also provided in the columns. The failures included non-ductile fractures of the beam bottom flange-to-column complete-joint-penetration (CJP) groove welding (see figure in the next page) which propagated into the adjacent column flange and web and into the beam bottom flange, and also panel zone failures. This failure was accompanied in some instances by secondary cracking of the beam web shear plate and failure of the beam top flange weld. Even though a number of these different types of fractures were observed, the majority



(a) Damage consists of fractures or cracks that initiate in the welded joints of the beams to columns

(b) Fracturing of W14 column at welded beam-to-column connection in Northridge earthquake

Damages to beam-column connections during 1994 Northridge earthquake, USA; Severe stress concentrations inherent in the configuration of the connection were not considered in the design

Source: FEMA 354, FEMA 355D

of these fractures were found near the welded bottom beam flange joint. Failures occurred in connections with and without column-flange stiffeners as well as connections with and without return welds on the shear connection plates. Both wide-flange columns and built-up box sections appear to have been affected. The damage to steel buildings during the 1995 Kobe earthquake in Japan (in which damage occurred in near-field regions due to very high velocities) was even more disturbing: 10% of the steel buildings in Kobe designed to current Japanese building standards collapsed.

The factors that contributed to the damage include the following (Williams 2004):

1. Stress concentration at the bottom flange weld, due to the notch effect produced by backing strips left in place
2. Poor welding practices, including the use of weld metal of low toughness
3. Uncontrolled deposition rates
4. The use of larger members than those previously tested or the use of higher strength girders
5. Lack of control of basic material properties (large variation of member strength from the prescribed values)
6. Less system redundancy and higher strain demands on connections
7. Inadequate quality control during construction
8. The tri-axial restraint existing at the center of beam flanges and at the beam-column interface, which inhibited yielding

The FEMA (Federal Emergency Management Agency) formed in mid-1994 the *SAC Joint Venture* [consisting of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and Consortium of Universities for Research in Earthquake Engineering (CUREE)] and funded $12 million over eight years. Their goal was to develop reliable, practical, and cost-effective guidelines and standards of practice for the repair or upgrading of damaged steel moment frame buildings, the design of new steel buildings, and the identification and rehabilitation of at-risk steel buildings. Detailed analytical studies of components, sub-assemblages, and complete structural systems and experimental testing of over 120 full-scale sub-assemblages were conducted. This resulted in the publication of several reports- FEMA 350, 351, 352, 353, and 355 series, which formed the basis for the AISC 341-05 code (Now, AISC 341-10) on *Seismic Provisions for Structural Steel Buildings* (www.sacsteel.org). In addition to this code, AISC developed another ANSI (American National Standards Institute) approved standard, AISC 358-05, which presents materials, design, detailing, fabrication, and inspection requirements for a series of pre-qualified moment connections. AISC updates and reissues this standard from time to time, as and when additional research results are available.

17.8.1 Design Principles for Special Moment Frame

The design base shear equations provided in Clause 7.5.3 of IS 1893 (Part 1):2002 and Clause 12.3 of IS 800:2007 incorporate a factor of $2R$ in the denominator of the equation for determining the horizontal seismic coefficient A_h (see Section 3.12 and Eqn 3.28 of Chapter 3), which reflects the expected degree of inelastic response for design-basis earthquakes, and also the ductility capacity of the framing system. As per Table 23 of IS 800, steel special moment frame (SMF) is permitted to be designed using $R = 5$ [ordinary moment frame (OMF) should be designed with an R value of 4 only], and is expected to sustain multiple cycles of significant inelastic response when subjected to design-level ground motion. As mentioned earlier, several steel SMF may have substantial over-strength, due to a number of factors: columns and beams having more than the designed area and moment of inertia, due to the discrete available sections, oversized columns to meet strong-column/weak-beam criteria, use of oversize sections to provide sufficient stiffness for drift control, and variability in the strength of the steel material itself. As a result, several steel SMF may remain elastic during earthquake shaking.

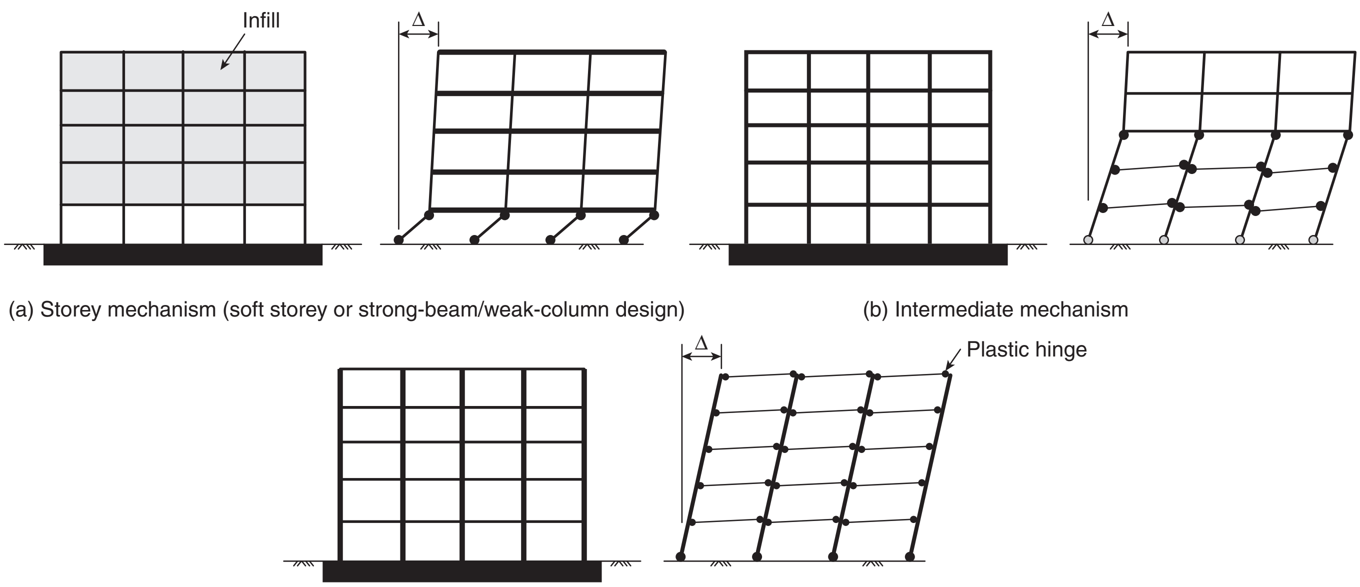
The proportioning and detailing requirements specified in Section 12.10 (for OMF) as well as Section 12.11(for SMF) of IS 800:2007 are intended to provide ductile inelastic response. The primary goals of these provisions are as follows (Hamburger et al. 2009):

1. Achieve a strong-column/weak-beam condition that distributes inelastic response over several stories
2. Avoid P-delta instability under gravity loads and anticipated lateral seismic drifts
3. Incorporate details that enable ductile flexural response in yielding regions

17.8.2 Strong-column, Weak-beam Concept

When buildings are subjected to earthquake loads, plastic hinges will be formed at the ends of the members where there are heavy bending moments (as they are often designed only for a fraction of the earthquake loads), and subsequently will fail when there are enough plastic hinges to form a mechanism. A few possible mechanisms in which a structure may fail is shown in Fig. 17.18. The distribution of damage over the height of the building depends on the distribution of lateral drift. If the building has weak columns or long columns in a particular storey, most of the inelastic portion of the structure's drift will be concentrated in these storeys, as shown in Fig. 17.18(a), resulting in very large P-Δ effects at those locations (note that in the current design practice, contribution masonry infill is considered in the mass of the building, but neglected in estimation of stiffness; thus the actual behavior of building is not captured in design). Such soft storey columns should generally be avoided for better performance of the structure. Due to the failure of several buildings having soft storey columns, clause 7.10.3(a) was included in IS 1893 (Part 1):2002, which states that the columns and beams of soft storey should be designed for 2.5 times the storey shears and moments calculated under seismic loads (though there is no theoretical or experimental justification for this clause). In addition, clause 7.10.3(b) stipulates that shear walls be placed symmetrically in both directions as far away from the center of the building as feasible and designed for 1.5 times the lateral storey shear force. On the other hand, if strong columns are provided throughout the building height, drift will be more uniformly distributed (Fig. 17.18c), and localized damage will be reduced. Buildings, with columns which have equal strength as beams, may result in an intermediate mechanism as shown in Fig. 17.18(b).

It is also important to recognize that the columns in a given storey support the weight of the entire building above those columns, whereas the beams only support the gravity loads of the particular floor; hence, failure of a column is of greater importance than the failure of a beam, as column failure will result in total collapse of the entire building. Recognizing this fact, building codes often specify that columns should be stronger than the beams that frame into them. This *strong-column/weak-beam principle* is fundamental to achieving safe behavior of frames during strong earthquake ground shaking. Buildings, following this principle, will fail in *beam-hinge mechanism* (beams yielding before the columns) and not in the *storey mechanism* (columns yielding before the beams). Storey mechanism must be avoided as it causes greater damage to the building). Therefore, column should be stronger than the beams meeting at a joint. Note that the beam-hinge



(a) Storey mechanism (soft storey or strong-beam/weak-column design)

(b) Intermediate mechanism

(c) Beam mechanism (strong-column/weak-beam design)

FIG. 17.18 Different failure mechanisms of moment-resisting frames mechanism also has a plastic hinge at the base of ground floor column (see Fig. 17.18c); hence it is important not to have any splicing at this location and to have ductile detailing, so that the required rotation capacity is achieved.

Murty et al. (2013a), based on parametric studies, found that an increase in the column to beam strength ratio increases (a) lateral load (base shear) capacity of the building and (b) lateral deformation and ductility capacity of the building (see Fig. 17.19). Even though this study is based on reinforced concrete frames, similar behaviour is likely in steel moment frames also. The sequence of hinge formation is critical in a building in addition to its capacity curve and the location of plastic hinges in the building. It is preferable to have hinges to form in beams before they are formed in columns. Such a possibility is found to occur when column to beam strength ratio is more than 3.6 (Murty et al. 2013a). As the column to beam strength ratio of about 3 to 4 is impractical in most practical cases, a lower strength ratio of 1.2 is adopted in clause 12.11.3.2 of IS 800: 2007 (see Eqn 17.1) at the joint, whereas Section E3 of AISC 341-10 adopts a column to beam strength ratio of 1.0 only at the joint. Due to this some column yielding associated with an intermediate mechanism, as shown in Fig. 17.18(b) is to be expected, and the columns must be detailed accordingly. Flexural yielding of columns at the base is permitted by AISC 341-10.

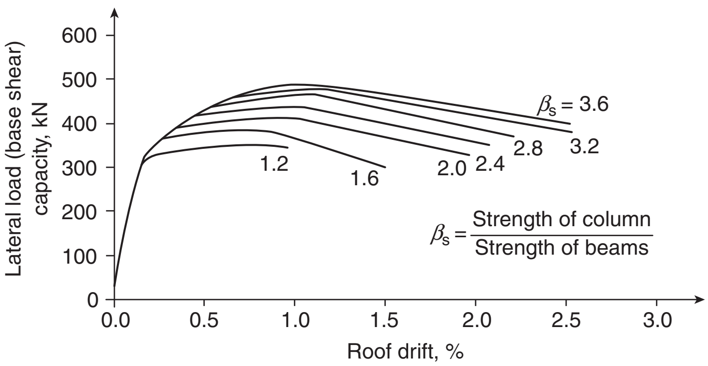


FIG 17.19 Increase in global ductility with increase in column to beam strength

Source: Murty et al 2012, Reprinted with permission from NICEE.

Clause 12.11.3.2 of IS 800:2007 stipulates that the sections selected for beams and columns should satisfy the following condition:

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2 \tag{17.1}$$

where, $\sum M_{pc}$ is the sum of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline, with a reduction for the axial force in the column (When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines may be used), and $\sum M_{pb}$ is the sum of expected flexural strengths of the beams at the intersection of the beam and column centerlines.

When determining available column flexural strength, it is important to consider the axial loads in the column, as they will reduce the moment capacity (see Section 8.10 of Chapter 8). Hence, AISC 341-10 suggests that the value of ΣM_{pc} may be calculated using

$$\sum M_{pc} = \sum Z_{pc}(f_{yc} - f_a) \text{ with } f_a = P_{uc}/A_g \geq 0 \tag{17.1b}$$

The value of ΣM_{pb} may be calculated using AISC 341-10 as

$$\sum M_{pc} = \sum (M_p + M_v) \tag{17.1 c}$$

where, A_g is the gross area of column, mm^2; f_{yc} is the specified minimum yield stress of column, MPa; Z_{pc} is the plastic section modulus of the column about the axis of bending, mm^3; and P_{uc} is axial force in the column, N; M_v (the additional moment due to shear amplification) = $V_p S_h$, V_p is the shear at plastic hinge location, N; and S_h is the distance from column center line to plastic hinge location, mm.

As per AISC 341-10, this requirement of Eqn (17.1) need not be met when $f_a < 0.3 f_{yc}$ for all load combinations, except for the two special factored load combinations specified in AISC 341-10 when the factored axial load on the column exceeds 40% of the nominal capacity and any of the following conditions hold:

1. The joint is at the top storey of a multi-storey frame
2. The joint is in a single-storey frame.
3. The sum of the available shear strengths of all exempted columns is less than 20% of the available shear strengths of all moment frame columns for a specific storey in the total frame and the sum of the available shear strengths of all exempted columns in a specific frame is less than 33% of the available shear strengths of all moment frame columns.

17.8.3 Provisions in IS 800:2007 for SMF

As per IS 800:2007, SMF can be used in any seismic zone and for any importance-factor value. According to Clause12.11of IS 800:2007, the beam and column sections used in SMF should be either plastic or compact; in addition at potential plastic hinge locations, plastic sections only should be used. It has to be noted that Table D1.1 of AISC 341 defines separate *b/t* limits for members subject to seismic loads and are parts of SMF; these limits are prescribed for highly ductile and moderately ductile members; these 'seismically compact' sections are expected to achieve a deformation ductility of at least 4. When the column-beam moment ratio as per Eqn (17.1) is above 2.0, AISC 341-10 suggests that the columns will remain elastic. Hamburger et al. (2009) cautions that the strong-column/weak-beam provisions of AISC 341-10 may not be adequate to avoid formation of storey mechanisms in all cases; hence, the designers may increase column sizes, beyond code requirements, to obtain better performance during severe earthquakes. Such an increase in sizes may reduce the need to provided expensive web stiffener or doubler plates in beam-column junctions, but will increase the total weight of steel used. Other provisions in IS 800:2007 are similar but are not as elaborate as those given in AISC 341.

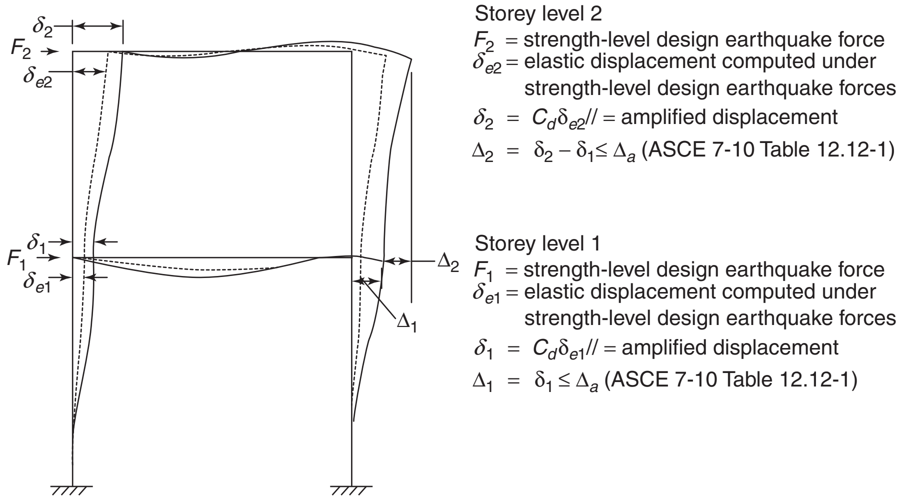


FIG. 17.20 Storey drifts determination in special moment frames, as per ASCE 7-10

17.8.4 Proportioning for Drift

It is important to realize that the size of beams in a steel SMF is controlled by the consideration of drift. Consequently, the size of columns is also drift controlled, because the strong-column/weak-beam concept demands larger columns when larger beams are provided. The only exception may be the end columns in SMF, which have high axial load demands, and hence may be controlled by strength design criteria (Hamburger et al. 2009).

Members of the seismic force-resisting system (SFRS) that are anticipated to undergo inelastic deformation have been classified as either *moderately ductile members* or *highly ductile members*. As per the code, during the design earthquake, highly ductile members in special moment frames (SMF) are anticipated to undergo significant plastic rotation of 0.04 radians or more without degradation in strength and stiffness below the full yield value (M_p). The member rotations result from either flexure or flexural buckling.

The checking of *storey drift* (storey drift may be defined as the displacement of one level relative to the other level above or below) for lateral loads should be as per IS 1893 (Part 1):2002. This code stipulates that the storey drift in any storey (due to the minimum specified lateral loads with a partial load factor of 1.0), should not exceed 0.004 times the storey height. For buildings located in seismic zones IV and V, it should be ensured that the structural components, that are not a part of the seismic force-resisting system in the direction of consideration, do not lose their vertical load-carrying capacity under the induced moments resulting from storey deformations equal to R times the storey displacements [see also Clause 7.11 of IS 1893 (Part 1):2002].

As per ASCE 7-10, the storey drift (Δ) is computed as the difference of deflections at centers of mass at top and bottom of storey (see Fig. 17.20). The deflection at level x at the center of mass (δ_x) is determined as follows:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{17.2}$$

where, C_d is the deflection amplification factor, as given in Table 12.2-1 of ASCE 7-10 (for SMF, $C_d = 5.5$ and for OMF $C_d = 3$; Generally C_d is 1/2 to 4/5 the value of R), δ_{xe} is the deflection determined by elastic analysis, and I is the importance factor. The factor C_d is used to adjust lateral displacements for the structure determined under the influence of design seismic forces to the actual anticipated lateral displacement in response to design earthquake shaking. Generally, the more ductile a system is, the greater will be the difference between the value of R and C_d .

It has to be noted that as per ASCE 7-10, when using equivalent lateral force procedure, the P-Δ effects on storey shears and moments drifts need not be considered if the stability coefficient (θ), as defined in Eqn (17.3a) is less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{17.3a}$$

where P_x is the total vertical design load at and above level x (kN); while computing P_x, no individual load factor need exceed 1.0, Δ is the design storey drift as defined earlier (see Fig. 17.20) occurring simultaneously with V_x (mm) V_x is the seismic shear force acting between levels x and $x - 1$ (kN), h_{sx} is the storey height below level x (mm) and C_d = the deflection amplification factor as per Table 12.2-1 of ASCE 7-10.

ASCE 7-10 also states that the stability coefficient (θ) should not exceed $\theta_{\max}$, which is defined as:

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \tag{17.3b}$$

where β is the ratio of shear demand to shear capacity for the storey between levels x and $x - 1$. This ratio may be conservatively taken as 1.0. When the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-Δ effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1 - \theta)$, to include P-Δ effects. When θ is greater than θ_{max}, the structure will become unstable and hence should be redesigned. When the P-Δ effect is included in an automated analysis, Eqn (17.3b) should still be satisfied, however, the value of θ computed from Eqn (17.3a) using the results of the P-Δ analysis is permitted to be divided by $(1 + \theta)$ before checking Eqn (17.3b). The computed drift has to be compared with the allowable drift as given in Table of 12.12-1 ASCE 7-10 [it ranges between 0.01 h_{sx} to 0.02 h_{sx}, where h_{sx} is the storey height below level x (mm)]. If the computed drifts are more than allowable, the stiffness of the members have to be increased, analysis and design performed again and the drifts compared again to check whether they are within allowable limits. It has to be noted that the response spectrum method (see Section 3.12.4 of Chapter 3) will result in more economical designs for steel SMF than the equivalent static method (Hamburger et al. 2009).

Joint Panels

It is important to realize that the storey drifts are caused by flexural and shear deformations in beams and columns and also by the shear deformations in joint *panel zones*, causing a shear (rocking) mode of drift, and by axial deformations in the columns causing flexural mode of drift (Hamburger et al. 2009). In general, the beam bending is the largest contributor, while column bending is the smallest. Panel zone shear deformations may contribute to about 15% to 30% to the total shear mode of drift. Clause 12.7.3b of ASCE 7-10 stipulates that the contribution of panel zone deformations to overall storey drift should be included, when checking drift limits. Hamburger et al. (2009) provide approximate equations to calculate storey drift due to beam flexure, column flexure and panel zone shear deformations. The effect of panel zone shear deformations can also be directly incorporated in the analytical model by using the *scissors elements* or a *panel zone parallelogram model*. Potential failure modes include shear buckling, and if doubler plates are included, fracture at the welds. Failure modes may also include column flange bending, web crippling, and web buckling. Checking of panel zones and the joint panel model proposed by Krawinkler (1978) are discussed briefly in Section 4.5.2 of Chapter 4. More details about these analytical models and the behaviour of panel zones can be found in Krawinkler (1978); FEMA-335C (2000); and Lee, et al. (2002).

Beam-column and Other Connections

As per Clauses 12.11 of IS 800:2007, all the beam-column connections of SMF should be rigid and should be designed to withstand a moment of at least 1.2 M_p of the connected beam. When reduced beam section is used (see Section 17.10), its flexural strength, determined at the column face, should be at least 0.8 M_p of the unreduced section, at a storey drift angle of 0.04 rad.

The beam-to column connection of SMFs should be designed to withstand a shear resulting from the load combination of 1.2DL + 0.5LL plus the shear resulting from the application of 1.2 M_p in the same direction at each end of the beam. In column connections along the strong axis, the *panel zone* has to be checked for shear buckling in accordance with clause 8.4.2 of IS 800. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling. Continuity plates (tension stiffener) should be provided in all strong axis welded connections, except in end plate connection. Depending on the type of connection used (see Section 17.9), the following failure modes have to be checked [Hamburger et al. (2009)]:

1. Net section fracture at bolts, shearing and tensile failure of bolts, bolt bearing, and block shear failures
2. Fracture in or around welds
3. Fracture in highly strained base material
4. Fracture in weld access holes

Failure modes of column splices are similar to those of beam-to-column connections. Failure modes of column bases may include anchor bolt failures or pull out, fracture in base plate, or in column-to-base plate connections, and local buckling of gusset plates.

17.8.5 Continuity Plates

In a rigid, fully welded connection, *continuity plates* of thickness greater than the thickness of beam flanges have to be provided and welded to the column flanges and the web. The individual thickness of column webs and doubler plates should be such that (see Fig. 17.21)

$$t \geq (d_p + b_p)/90 \tag{17.4}$$

where t is the thickness of column web or doubler plate, d_p is the *panel zone* depth between the continuity plates, and b_p is the panel zone width between column flanges.

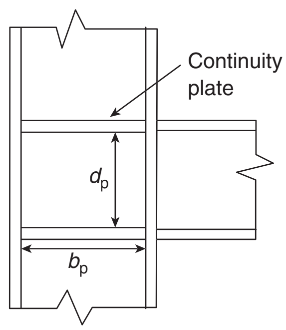


FIG. 17.21 Continuity plates

Protected Zones

As per AISC 341-10, the plastic hinging zones at the ends of SMF beams should be treated as *protected zones*. In general,

CASE STUDY

Torre Latinoamericana—The skyscraper that withstood two earthquakes

Torre Latinoamericana, Mexico

Source: Reprinted with permission from Mr Juan Pablo Ortiz Arechiga through www.flickr.com

The Torre Latinoamericana (English: Latin–American Tower) is a skyscraper in downtown Mexico City, Mexico, built during 1956 (height 188 m with 44 storeys). It is widely recognized internationally as an engineering and architectural landmark since it was the world's first major skyscraper successfully built on highly active seismic land.

The project was designed and executed by Dr Leonardo Zeevaert and his brother Adolfo Zeevaert, Mexican civil engineers born in Veracruz. Prof. Nathan M. Newmark, of the University of Illinois was the main consultant. Its design consists of a steel-frame construction and deep-seated piles, which were necessary due to the Mexico City's frequent earthquakes and muddy soil composition. Before construction, both engineers carried out a number of soil mechanics studies at the construction site, and designed the structure accordingly. Today this is a mandatory practice, but at that time it was quite an innovation. Foundation work started in 1949 with 361 numbers of 33 m deep pylons. While it was being built, detractors said that this huge building will not withstand Mexico City's earthquakes.

The tower gained international attention when it withstood the 7.9-magnitude earthquake in 1957, thanks to its outstanding design and strength. This performance resulted in the recognition in the form of the American Institute of Steel Construction Award of Merit for 'the tallest building ever exposed to a huge seismic force' (as is attested by plaques in the building's lobby and observation deck). Interestingly, this building experienced another bigger earthquake on 19 September 1985, with a magnitude of 8.1, which destroyed many buildings in Mexico City, especially the ones built in the downtown area, in the tower's neighborhood. The Torre Latinoamericana withstood the force produced by this earthquake also without problems, and has thus become a symbol of safety in Mexico City. Today, the tower is considered as one of the safest buildings in the Mexico city despite its potentially dangerous location.

During the September 1985 earthquake, the engineer Adolfo Zeevaert was inside his office on the 25th floor. From that vantage point he was able to witness the destruction taking place; collapse of several buildings and the dust cloud that followed—all the while feeling the movements inside the tower. It could arguably be said that it was the first time that a builder and designer of a tall building witnessed first-hand its behavior during a massive earthquake.

for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

Special moment frames have the advantages of architectural simplicity and relatively low base shear. Moment-frame column and beam sizes can be significantly heavier per linear foot than in braced frames due to their means of transferring forces and resisting lateral drift. Heavier sections lead to higher overall tonnages (material costs), and possibly the need for larger erection equipment. Doubler plates required at beam-columns connections (to strengthen the column web locally) result in additional cost and fabrication time. SMF may require more field-welding than braced frames, which leads to higher erection costs. A greater number of moment frames is often required over braced frames to provide enough stiffness in the building to accommodate drift requirements for serviceability. The other disadvantages include connection cost and connection testing.

17.8.6 Ordinary Moment Frames (OMF)

Ordinary moment frame (OMF) should not be used in severe seismic zones (IV and V) and for buildings with important factor greater than unity in seismic zone III (Clause 12.10 of IS 800:2007). As per IS 800:2007, during the design earthquake, moderately ductile members in ordinary moment frame (OMFs) are anticipated to undergo moderate plastic rotation of 0.02 radians without degradation in strength and stiffness below the full yield value (M_p). For OMFs semi-rigid moment connection is permitted in clause 12.10; however it is desirable to have rigid joints. As per Clauses 12.10 of IS 800:2007, OMFs with rigid moment connections should be designed to withstand a moment of at least 1.2 M_p of the connected beam. Similar to the case of SMF, rigid as well as semi-rigid connections should be designed to withstand a shear resulting from the load combination of 1.2DL + 0.5LL plus the shear resulting from the application of 1.2 M_p in the same direction at each end of the beam. In rigid, fully welded connection, continuity plates of thickness greater than the thickness of beam flanges have to be provided and welded to the column flanges and the web.

When semi-rigid connections are used, the connection should be designed to withstand a moment of at least 0.5 M_p of the connected beam or the maximum moment that can be delivered by the system, whichever is less. In semi-rigid joints, the design moment should be achieved within a rotation of 0.01 radians (The Fyre-Morris model given in Annex F of IS 800:2007 may be used to check the curvature). The stiffness and strength of semi-rigid connections should be considered in the analysis and design, in addition the overall stability of the frame should be ensured. It has to be noted that AISC 341 defines *intermediate moment frames (IMF)* also, which may fall between the requirements of SMF and OMF.

17.8.7 Designing Buildings with SMF/OMF

An initial choice must be made about the different members of the frames, based on past experience. After obtaining the seismic forces acting at different levels as explained in section 3.12 of Chapter 3, the forces and moments in different members can then be obtained by using any suitable standard computer program for the various load combinations specified in the code (see Section 3.15 of Chapter 3). If the software considers only elastic analysis, moments due to P-delta effects must also be determined. Then, the building should be checked for the overall drift, storey drift, and joint rotation specified in Section 12 of IS 800:2007. If they are not within the allowable limits, the member sizes have to be increased and the process repeated again till they are satisfied.

The size of columns and beams should also be checked for their required strength and strong-column weak-beam concept. The beam-column connections should also be detailed carefully and the necessary bracings should be designed and provided. The structure must also be designed to resist overturning effects caused by seismic forces. Provisions should be made for increase in shear forces on the lateral force resisting elements resulting from the horizontal torsional moment caused due to the eccentricity between the center of mass and center of rigidity [see Section 7.9 of IS 1893 (Part 1):2002 for these details]. IS 1893 also contains special provisions for buildings with soft or weak storey(s), torsional irregularity, non-structural elements, considerations of diaphragm flexibility, and modeling of brick infill panels.

An example of a twelve-storey moment-resisting steel frame as per the 2009 NEHRP Recommended Seismic provisions is provided by Charney et al. (2013). The cost of SMF can be minimized by using the following rules of thumb:

1. Avoid doubler plates by increasing column sizes.
2. Provide partial penetration welds in lieu of complete penetration welds at beam-to-column moment connections (can be used only in non/low-seismic zones).
3. Use fixed-column base connections to reduce drift. They provide more stiff frames at the first storey than pinned-column base connections do, resulting in reduced drift deflections.
4. Decrease fabrication and/or erection labor cost wherever possible, for example, by using higher sections will result in net a lower overall cost.

More details of the design, design guidance, and detailing and constructability issues of SMF may be found in AISC design guides 12 and 13 and in the NEHRP seismic design technical brief 2 (Hamburger et al. 2009).

17.9 SEISMIC MOMENT CONNECTIONS

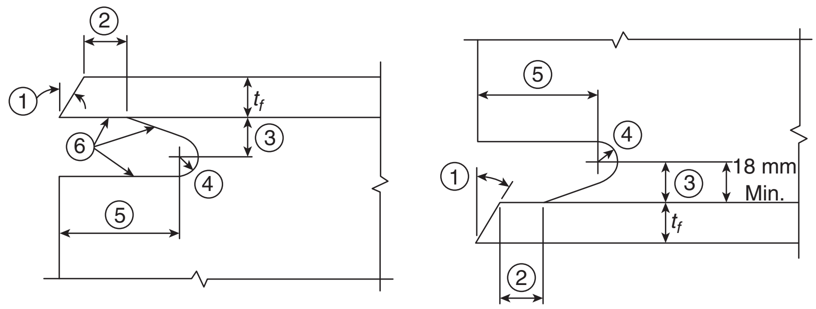
After the 1994 Northridge earthquake, the AISC 341 code requires that the performance of beam-column connections used in SMF should be demonstrated through testing. Hence full-size testing has to be conducted to ensure that the connections are capable of developing 0.04 radians of inter-storey drift without excessive strength loss, when subject to cyclic loading. As this testing is expensive and only a few laboratories are equipped to conduct such large scale testing, AISC 341 permits the use of prequalified connections (see Section 17.9.4).

Extensive testing has been done in the past on beam-column connections subjected to cyclic loading, and based on these tests the following three basic philosophies have been suggested to improve the connection behaviour (FEMA 350:2000): (a) a toughening scheme, (b) a strengthening scheme, and (c) a weakening scheme. Often all of these schemes are used in combination.

17.9.1 Toughened Connections

In toughened connection, attention is paid to the complete penetration weld details between the beam and the column. Notch-tough electrodes are specified, which have a Charpy V-notch value of 27 J at -18^{o} C and 54 J at 21^{o} C. In addition, backing strips are removed and replaced with reinforcing fillet welds to eliminate the notch effect and to remove any weld flaws (weld flaws are likely to occur at the bottom flange, where the beam web prevents continuous weld passes). At the top flange, a reinforcing fillet is added to secure the backing bar to the column flange. Test results showed that to achieve the inelastic rotation demands of SMF, the beam web to column connection should be a complete joint penetration weld. It was also realized that the size, shape, and finish of weld access holes should be detailed in such a way as to reduce stress concentrations in the region around the access holes (see Fig. 17.22).

Another important aspect is the provision of column continuity plates in all cases. The thickness of these plates should be at least equal to the thickness of beam flange (not including cover plates) or one half of the total effective flange thickness (including cover plate). Welding of continuity plates to column flanges should be with full penetration groove welds, while the plate to column-web may be with double sided



Notes: 1. Bevel as required for selected groove weld.
2. Large of t_f or 12mm (plus 0.5 t_f, or minus 0.25t_f)
3. 0.75 t_f to t_f, 20mm minimum (∓6mm)
4. 10mm minimum radius (plus not limited, minus 0)
5. 3 t_f (∓12mm)
Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°

FIG. 17.22 Weld access hole details

Source: FEMA 350:2000.

fillet welds. Notch-tough electrodes should be used in all cases, and welding in the *k* region of the column should be avoided.

Seismic Structural Design Associates has developed a proprietary system, called DSDA *SlottedWeb™ connection*, which has a horizontal slot in the web of the beam, near the flanges, which separates the beam's web and flange effectively as shown in Fig. 17.23. There is also a shear plate that is shop welded to the column flange and fillet welded to the beam at the site.

The slot in the SlottedWeb™ connection isolates the beam section in such a way that the shear is resisted entirely by the web and the flanges resisting only the bending moment.

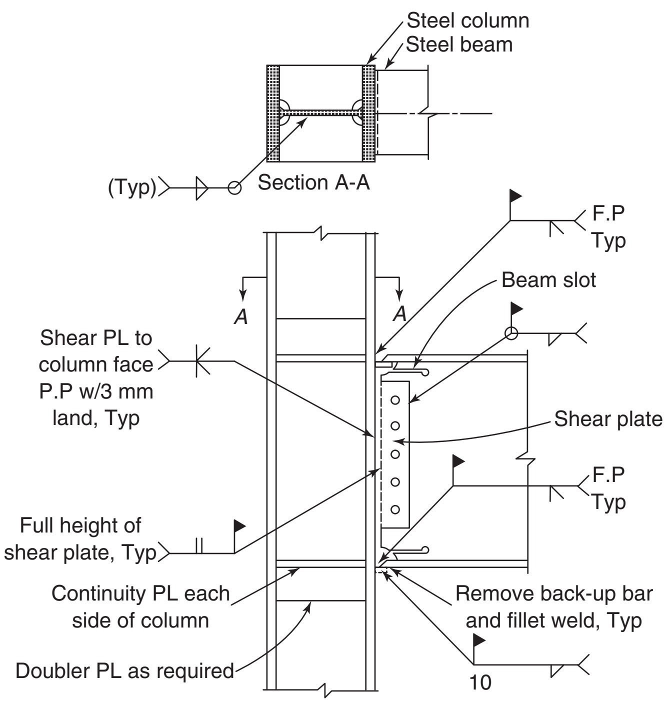


FIG. 17.23 SlottedWeb™ connection (www.slottedweb.com)

According to SSDA, the SlottedWeb™ connection results in controlled ductile beam yielding and also prolonged fatigue life due to the following:

1. The beam web slots eliminate the seismic shear in the beam flanges, which reduces the large stress and strain gradients across and through the beam flanges and at the weld access hole by permitting the flanges to flex out of plane. Typically, the elastic stress and strain concentration factors of the pre-Northridge connections (caused by the large beam flange shear that resulted in severe local beam flange and/or column flange distortions) were reduced from 4.0 to 5.0 down to 1.2 to 1.4.
2. The lateral-torsional buckling of beams, which is prevalent in non-slotted beams, is eliminated. The separation of the beam flanges and beam web allow the flanges and web to buckle independently nd concurrently, thus eliminating a the twisting mode of buckling. Elimination of this buckling mode is important for perimeter seismic moment frames that support the exterior cladding of the building. Even a small amount of twisting of these perimeter beams will be reflected as significant visible distortion of the exterior cladding.
3. The separation of the beam web and the flange results in biaxial rather than triaxial stress and strain states in the region of the connection, thus increasing the fatigue life of the connection.
4. Residual weld stresses are significantly reduced because of the long structural separation between the vertical web and horizontal flange welds.
5. Full scale low cycle fatigue tests have shown that the fatigue life was increased considerably (more than tripled) over the non-slotted connections that subject the beam flange/welds to a large portion of the beam seismic vertical shear.

Analytical and experimental studies have shown that SlottedWeb™ develops the full plastic moment capacity of the beam and does not reduce its elastic stiffness. According to SSDA, SlottedWeb™ is more economical than reduced beam section (RBS) moment connections or Kaiser bolted brackets, since SlottedWeb™ involves fewer additional parts and cuts, and the work is involved in the thinner web material (Average connection cost is about $1840 per connection, as against $2360 for RBS connection). SlottedWeb™ has been used in the Yerba Buena Tower, San Francisco, Hyatt San Diego and GAP Embarcadero, San Francisco. More details about this connection may be found in www.slottedweb.com.

17.9.2 Strengthened Connections

In this design philosophy, the portion of the beam adjacent to the column, where the maximum moment occurs during seismic loading, is strengthened. This will force the plastic hinge to form away from the joint, in the un-strengthened portion of beam. The strengthening may be done using cover plates, rib plates, side plates or haunches at the beam-to-column interface (see Fig. 17.24). However, the code requirement of strong-column/weak-beam condition is to be satisfied. It is also recommended that the connections be designed in such a way to ensure plastic hinge location at a distance of half the beam depth from the column face.

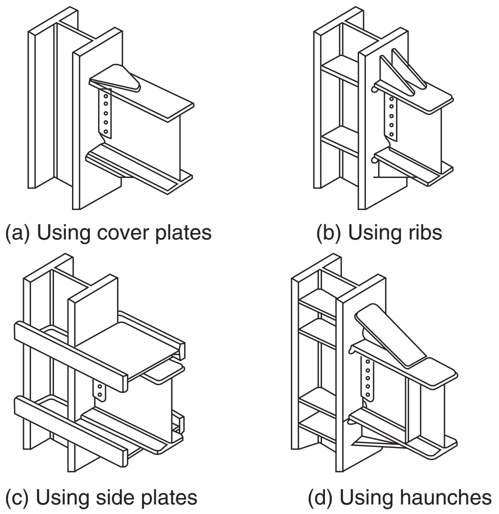


FIG. 17.24 Strengthened connections

When these connections are used, the effective section modulus of the beam at the connection is increased, decreasing the bending stress at the extreme fiber of the section, and also the total force resisted by the flange welds. The extrapolated moment near the column face, M_f, can now be well above the beam plastic moment, M_p, and hence must be considered in design (Tamboli 2009).

Cover-plated connections Experimental research on cover-plated connections has shown that these connections perform well in the inelastic range (FEMA 350:2000). However, when cover plates are used, proper detailing is to be followed for obtaining ductile behaviour. Typically, cover plates are fillet-welded to the beam flanges and groove-welded to the column flange. A common detail is shown in Fig. 17.25(a). Note that only the long sides of the cover plates are welded to the beam flanges. For ease of welding at site, the bottom cover plate is oversized and the top plate undersized, so that down-hand welding can be adopted at each location. When oversized top and bottom cover plates are used, the top plate is shop-welded to the beam and the bottom plate is field-welded. Cross-welds to the beam flanges at the end of the cover plates are not recommended, since they do not perform well in the inelastic region. Moreover at the column flange, both the cover plate and the beam flange should be connected by the same groove weld as shown in Fig. 17.25(b), and not by separate groove welds, since they introduce the 'notch' effect. Design of this connection with checks to satisfy FEMA requirements is provided by Tamboli (2009). See Example 14.20 of Chapter 14 for the design of cover plated connection, but the detailing should be as per Fig. 17.25 for earthquake resistance.

When haunched connections are used, the haunch may be located on the bottom flange only (since the top flange will support the floor slab). Haunches may be made from triangular cut sections of structural tee sections or by using plates, and stiffeners are provided in the column and beam webs, where the haunches end (see Fig. 17.24d). More details about this type of connection may be found in Yu et al (1997).

The action of vertical rib plate is similar to that of a cover plate or the haunch; it strengthens the section by increasing the section modulus while distributing the beam-flange force over a larger area of the column flange (see Fig. 17.24b). It is possible to use a single rib plate at the center of the beam flange, but it is better to use multiple ribs on each flange to direct the beam-flange force away from the center of the beam flange. Such a detail may also reduce the stress concentration at the center of the beam-flange

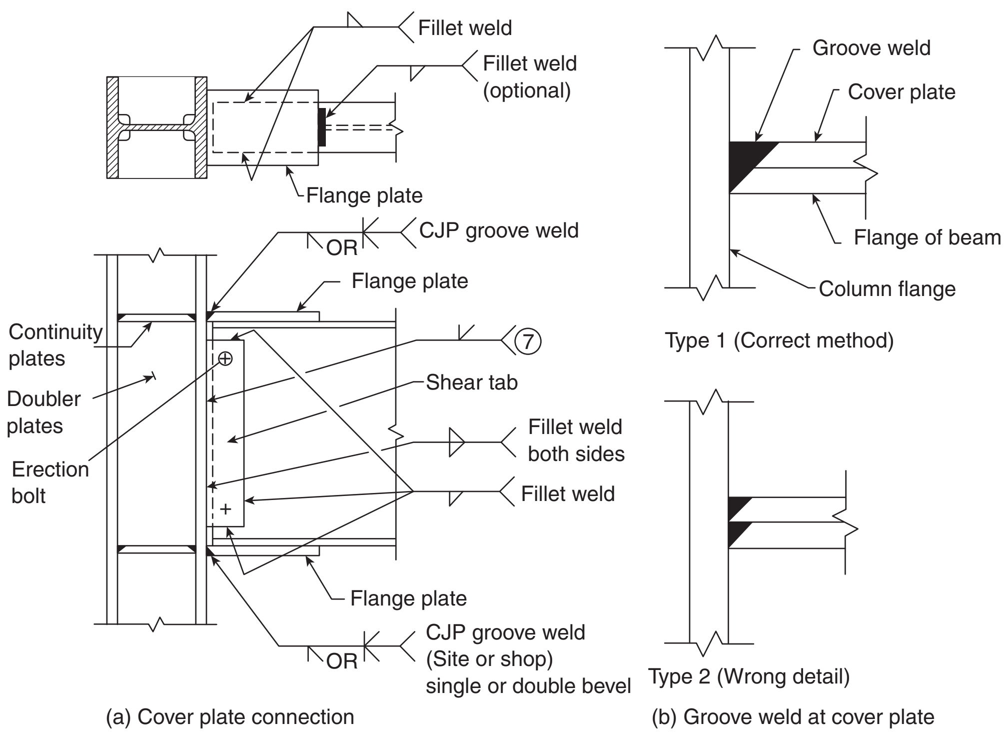


FIG. 17.25 Connection with cover plate as per FEMA 350:2000

groove weld. The ductile behaviour of such rib strengthened connections has been validated in a few experimental results.

17.9.3 Weakened Connections

A *weakened connection* is achieved by removing portions of the beam flange to create a reduced beam section (see Fig. 17.26). Hence this connection is also called *reduced beam section* (RBS) *moment connection*. Thus the designer chooses a location where the plastic hinge will occur by creating a weak link, or fuse, in the moment capacity of the beam. A proprietary reduced beam connection called 'dog bone' connection is shown in Fig. 17.26(a). The effect of 'dogbone' is similar to that of cover plate connections. With cover plates the connection is made stronger than the beam by strengthening the connection. In the *dogbone*, the connection is effectively made stronger than the beam by weakening the beam. The geometry of the RBS must be selected in such a way that the factored nominal moment capacity is not exceeded, at the critical beam section adjacent to the column. This method has potential benefits where the strengthening scheme had drawbacks—that is, the strong-column weak-beam concept and panel zone strength requirements are easily achieved. While producing the same effect of cover plates, the dogbone connection can be constructed with relatively simpler details, resulting in a more reliable and economic solution. However, the reduction in flange area may reduce the stiffness and stability of beam flange, and may increase the susceptibility of lateral torsional buckling of the beam in the reduced section. Hence additional lateral bracing is recommended in these locations. The earliest application of dogbone connection was made in 1969 (Iwankiw and Carter 1996 and Engelhardt et al. 1998) and the experimental validation of these connections is provided by Chen et al. (1996).

The shape, size and location of the reduced beam section may affect the performance of the connection. Various shaped such as straight-cut, taper-cut, arc-cut, and drilled flanges have been tried and tested (see Figs 17.26b and 17.26c). In general, arc-cut reduced beam sections have provided favourable results (Iwankiw and Carter 1996 and Iwankiw 1997). The design methodology presented by FEMA 350 (2000) and AISC 358-10 is applicable to RBS with curved arc cuts.

The distance of the RBS away from the column face, *a,* and the length *b* up to which the cut is made in the beam may be chosen as follows (FEMA 350:2000, AISC 358-10):

$$a = (0.5\,\text{to}\,0.75)b_{bf} \tag{17.5}$$

$$b = (0.65\ \text{to}\,0.85)d_b \tag{17.6}$$

$$c = \left(0.1\,\text{to}\,0.25\right)b_{bf} \tag{17.7}$$

where b_{bf} is the width of beam flange (mm), a is the horizontal distance from face of column flange to the start of an RBS cut (mm), b is the length of an RBS cut (mm), c is the depth of cut at center of the reduced beam section (mm), and d = depth of beam (mm). These limits are based on both stability and strength considerations.

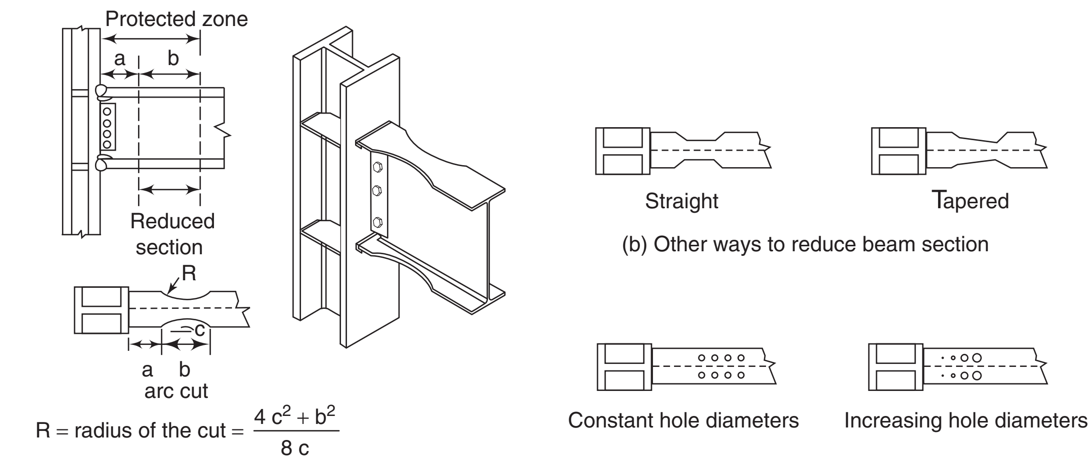
The design procedure, as per AISC 358-10, is as follows:

***Step 1*:** After choosing the values of a, b, and c, check whether the beams and columns are adequate for all load combinations specified in the code, including the reduced section of the beam, and that the design storey drift for the frame complies with applicable limits specified by the code. Calculation of elastic drift shall consider the effect of the reduced beam section. In lieu of more detailed calculations, AISC 358 allows to calculate the effective elastic drifts by multiplying elastic drifts based on gross beam sections by 1.1 for flange reductions up to 50% of the beam flange width. Linear interpolation may be used for lesser values of beam width reduction.

***Step 2*:** Compute the plastic section modulus at the center of the reduced beam section:

$$Z_{RBS} = Z_{pz} - 2ct_{bf}\left(d - t_{bf}\right) \tag{17.8}$$

where Z_{RBS} = plastic section modulus at center of the reduced beam section (mm^3), Z_{pz} = plastic section modulus about the



(a) Reduced beam section with arc cut

(b) Using holes to reduce beam section

FIG. 17.26 Weakened connections

z-axis, for full beam cross section (mm^3), t_{bf} = thickness of beam flange (mm),

Step 3**:** Compute the probable maximum moment, M_p, at the center of the reduced beam section:

$$M_p = C_p R_y f_y Z_{RBS} \tag{17.9}$$

where f_y is the specified minimum yield stress (MPa), R_y is the ratio of the expected yield stress to the specified minimum yield stress (see Table A3.1 of AISC 341-10), and C_p is a factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions,. AISC 358-10 suggests to take the value of C_p as follows:

$$C_p = \frac{f_y + f_u}{2f_y} \le 1.2 \tag{17.10}$$

where f_y is the specified minimum yield stress of the member (MPa), and f_u is the specified minimum tensile strength of the member (MPa).

Step 4**:** Compute the shear force at the center of the reduced beam sections at each end of the beam.

The shear force at the center of the reduced beam sections should be determined from a free body diagram of the portion of the beam between the centers of the reduced beam sections. This calculation shall assume the moment at the center of each reduced beam section is M_p and according to AISC 358 should include gravity loads acting on the beam based on the load combination of $1.2DL + 0.5IL + 0.2S$, as suggested in ASCE 7-10.

Step 5**:** Compute the probable maximum moment at the face of the column.

The moment at the face of the column shall be computed from a free-body diagram of the segment of the beam between the center of the reduced beam section and the face of the column, as shown in Fig. 17.27.

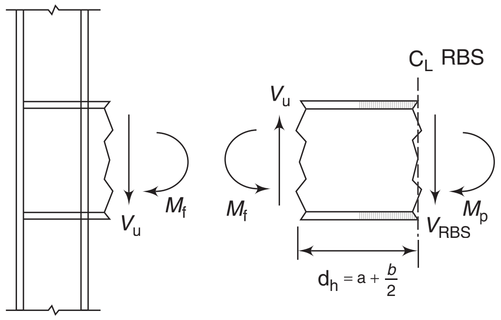


FIG. 17.27 Free-body diagram between center of RBS and face of column

Based on this free-body diagram, the moment at the face of the column is computed as follows:

$$M_f = M_p + V_{RBS} d_h \tag{17.11}$$

where M_f = probable maximum moment at face of column (Nmm), d_h = distance from face of the column to the plastic hinge (mm)= $a + b/2$, and V_{RBS} = larger of the two values of shear force at the center of the reduced beam section at each end of the beam (N).

It has to be noted that Eqn 17.11 neglects the gravity load on the portion of the beam between the center of the reduced beam section and the face of the column. If desired, the gravity load on this small portion of the beam may be included.

Step 6**:** Compute M_{pe}, the plastic moment of the beam based on the expected yield stress:

$$M_{pe} = R_y f_y Z_{pz} \tag{17.12}$$

Step 7**:** Check the flexural strength of the beam at the face of the column:

$$M_f = \phi_d M_{pe} \tag{17.13}$$

For ductile limit states $\phi_d = 1$

If Eqn17.13 is not satisfied, adjust the values of *c, a* and *b*, or adjust the section size, and repeat Steps 2 through 7.

Step 8**:** Determine the required shear strength, V_u, of beam and beam web-to-column connection and then check design shear strength of beam. The required shear strength, V_u may be calculated as

$$V_u = \frac{2M_p}{L_h} + V_{gravity} \tag{17.14}$$

where V_u is the required shear strength of beam and beam web-to-column connection (N), L_h is the distance between *plastic hinge locations* (mm), $V_{gravity}$ is the beam shear force resulting from $1.2D + 0.5L + 0.2S$ (N)

Step 9**:** Design the beam web-to-column connection.

Step 10**:** Check continuity plate requirements.

Step 11**:** Check column-beam relationship limitations.

17.9.4 Pre-qualified Seismic Moment Connections

Though the recent version of the code, IS 800:2007, contains provisions for design and detailing for seismic loads, it does not suggest the type of connections which are suitable for high or intermediate seismic zones. As mentioned earlier, as the testing of full scale specimens of the beam-column joints is expensive and time consuming, AISC 341 permits the use of prequalified connections [Subramanian (2010)]. These prequalified connections have been demonstrated by extensive testing and analysis and the expert review panel of AISC has approved them to be capable of providing code specified joint rotation without degradation in strength and stiffness below the full yield value.

These connections may be adopted in India also for better performance in strong or intermediate earthquakes.

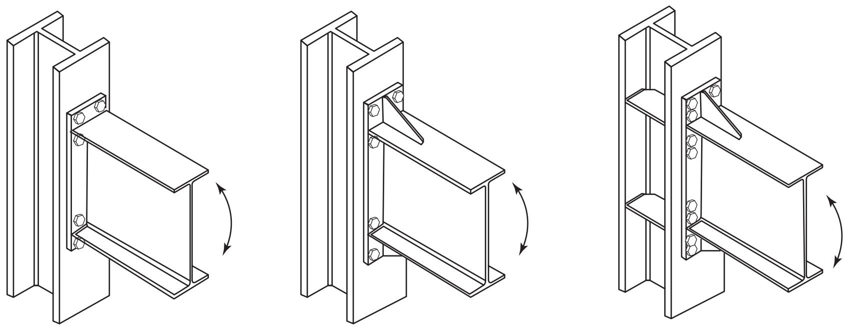
Types of Pre-qualified Moment Connections

AISC 358-2010 suggests 10 types of pre-qualified connections as given in Table 17.3 (the last two have been proposed in the 2016 version of the code):

TABLE 17.3 Prequalified moment connections as per AISC 358-10

Category	Connection description	Chapter in AISC 358-16	Acronym	Permissible systems
Welded, fully restrained	Welded unreinforced flanges, welded web	8	WUF-W	OMF, SMF
	Reduced beam section	5	RBS	OMF, SMF
	SidePlate moment connection	11	SidePlate	OMF, SMF
Bolted, fully restrained	Bolted unstiffened extended end plate	6	BUEEP	OMF, SMF
	Bolted stiffened extended end plate	6	BSEEP	OMF, SMF
	Bolted flange plates	7	BFP	OMF, SMF
Cast iron connection	Kaiser bolted bracket	9	KBB	OMF, SMF
	ConXtech ConXL moment connection	10	ConXL	OMF,SMF
Bolted, partly restrained	Double tee moment connection	13	DT	OMF, SMF
	Simpson strong-tie strong frame moment connection	12		OMF, SMF

A brief discussion about these prequalified connections is given here. More details about them and their methods of design may be found from AISC 358-2010.



(a) Four-bolt unstiffened, 4E (b) Four-bolt stiffened, 4ES (c) Eight-bolt stiffened, 8ES

FIG. 17.28 Bolted un-stiffened extended end-plate (BUEEP) and bolted stiffened extended end-plate (BSEEP) moment connections (From AISC 2010 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications with Supp. No. 1 and Supp. No. 2* Reprinted with permission from AISC. All rights reserved)

Reduced Beam Section Connection

In reduced beam section (RBS) moment connection (also known as the 'dog bone' connection), some portions of the beam flanges are removed in a pre-determined fashion, adjacent to the beam-column connection. In such a connection, yielding and plastic hinges are forced to form away from the connection at the reduced section of the beam. It has already been discussed in the previous section.

Bolted Unstiffened and Stiffened Extended End-plate Moment Connections

Bolted unstiffened extended end plate (*BUEEP*) *connections* and *Bolted stiffened extended end plate* (*BSEEP*) *connections* are made by welding the beam section to an end plate which is in-turn bolted to the column flange. The beam web and flanges are to be connected using either a CJP groove weld or a pair of fillet welds each having a size of 0.75 times the beam web thickness, but greater than 6 mm. Other specifications for protected zones, bolt pitch and spacing, etc. are provided in AISC 358-10.Three types of these connections are pre-qualified by AISC 358 (see Fig. 17.28). ACI 358 gives equations to check the various limit states of this type of connection such as flexural yielding of the beam section or end plates, yielding of column panel zone, shear or tension failure of the end-plate bolts, and rupture of the various welded joints. These provisions are intended to ensure inelastic deformation of the connection by beam yielding.

Extended end-plate moment connections in SMF systems with concrete structural slabs are prequalified subject to a few conditions, such as (a) the nominal beam depth is greater than 610 mm, (b) shear connectors are not present within 1.5 times the beam depth from the face of the connected column flange, and (c) the concrete structural slab is kept at least 25 mm from both sides of both column flanges. The AISC 358 code also gives a range of parameters that have been satisfactorily tested. These limitations make BUEEP and BSEEP suitable primarily for pre-engineered buildings.

Bolted Flange Plate Moment Connection

Bolted flange plate (*BFP*) *moment connections* consist of plates welded to column flanges and bolted to beam flanges as shown in Fig. 17.29. Identical top and bottom plates are used. Flange plates are connected to column flange by using CJP groove welds and beam flanges are connected to the plates by using high strength friction grip bolts. The web of the beam is connected to the column flange using a bolted single-plate

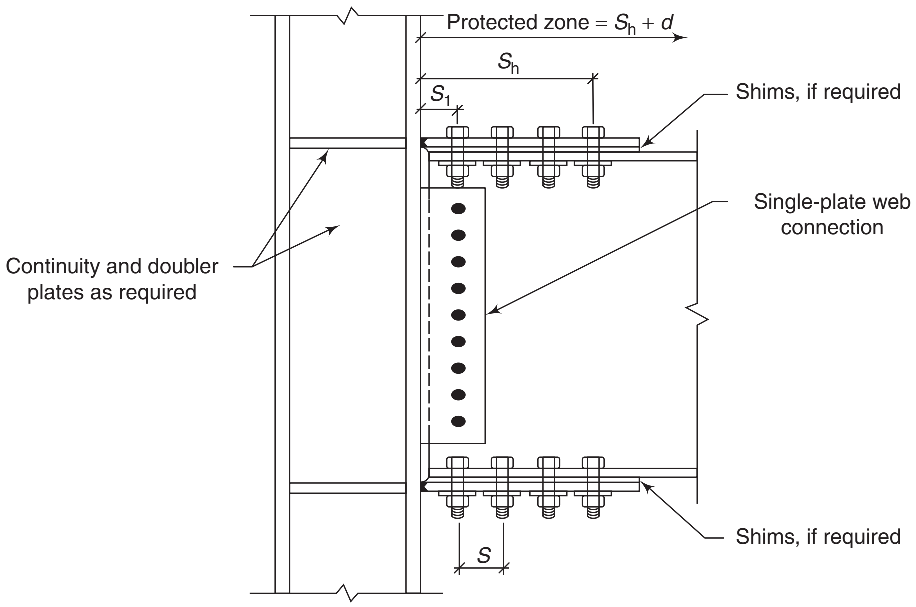


FIG. 17.29 Bolted flange plate (BFP) moment connection

Source: AISC 2010 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications with Supp. No. 1 and Supp. No. 2*. Reprinted with permission from AISC. All rights reserved

shear connection, with bolts in short-slotted holes. In this connection, yielding and plastic hinge formation are designed to occur in the beam near the end of the flange plates.

The flange plates and web shear plate are shop welded to the column flange and bolted to the beam flanges and web, respectively at site. Bolts shall be arranged symmetrically about the axes of the beam and shall be limited to two bolts per row in the flange plate connections. ASTM A490 or A490M bolts (maximum size 28 mm) with threads excluded from the shear plane should be used for the beam flange connections. The seismic behaviour of BFP momment connections were studied by Sato et al. (2008).

The experimental investigations on seismic behaviour showed the following (AISC 358-10):

1. Specimens achieved an inter-storey drift angle of 0.06 radians before failure
2. Initial yielding of the beam was at the last bolt away from the face of the column
3. Bolt slip of the flange plate bolts, which occurs at similar resistance levels to the initial yielding in the beam flange
4. Secondary yielding in the column panel zone, which occurs as the expected moment capacity and as strain hardening of beam hinge occurs
5. Limited yielding of the flange plate, which may occur at the maximum deformations

This sequence of yielding has resulted in very large inelastic deformation capacity for the BFP moment connection. However, the design procedure for this type of connection is more complex than other pre-qualified connections and is provided in AISC 358-10.

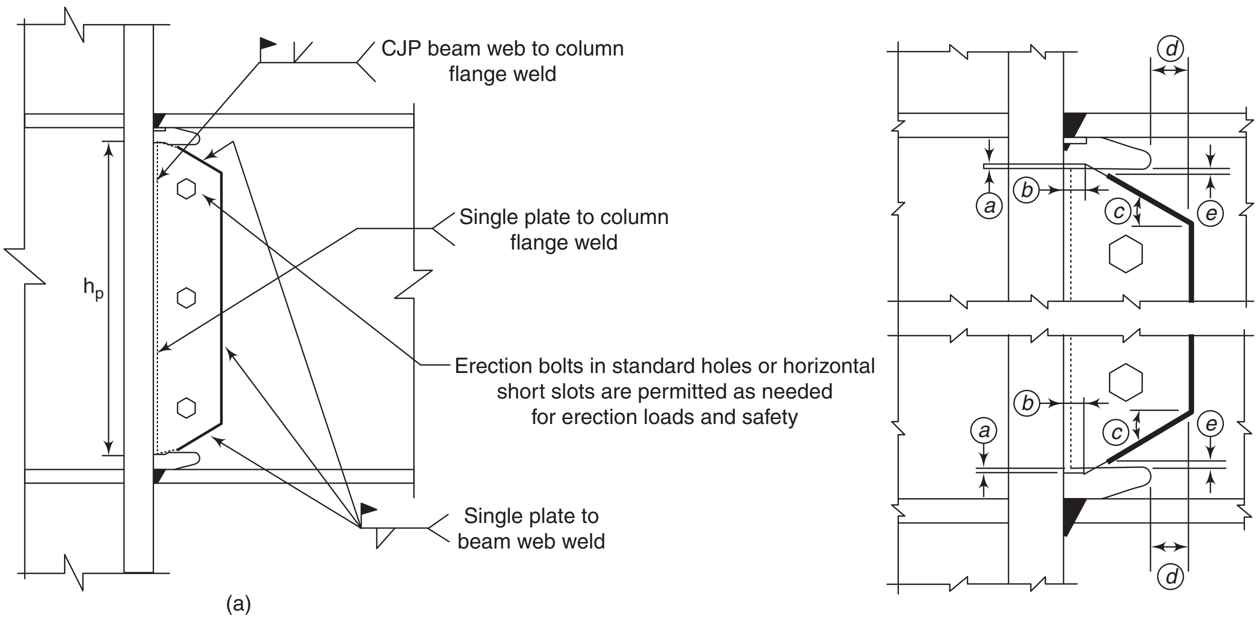
Welded Un-reinforced Flange-welded Web Moment Connection

Unlike other pre-qualified connections, in the *welded un-reinforced flange-welded web (WUF-W) connection*, the plastic hinge location is not moved away from the column face. Rather, the design and detailing features are intended to allow it to achieve SMF performance without fracture. Inelastic rotation is intended to occur in the beam in the region adjacent to the face of the column. In this connection the beam flanges are welded directly to the column flange using CJP groove welds. The beam web is bolted to a single-plate shear connection for erection. This plate is subsequently used as a backing bar for welding the beam web directly to column flange using CJP groove weld, which extends to the full depth of the web (that is, from weld access hole to weld access hole). A fillet weld is also used to connect the shear plate to the beam web, as shown in Fig. 17.30. The single plate connection adds stiffness to the beam web connection. A special seismic weld access hole and detailing, as shown in Fig. 17.30(b), are specified for the WUF-W moment connection, to reduce stress-concentration in the region around the access hole. The design procedure for the WUF-W moment connection is available in AISC 358-10. More details about the behavior of these connections are provided by Lee et al. (2005).

Kaiser Bolted Bracket Moment Connection

In Kaiser bolted bracket (KBB) moment connection, a cast steel (high-strength) bracket is fastened to each beam flange and bolted to the column flange as shown in Fig 17.31. The bracket can be either bolted (B-series) or welded (W-series) to the beam. The bracket is proportioned to develop the probable maximum moment strength of the beam, such that yielding and plastic hinge formation occurs in the beam at the end of bracket away from the column flange. This connection is designed to eliminate field welding and facilitate speedy frame erection. To provide erection tolerance, the bracket column bolt holes are vertically short-slotted and the column bolt holes are slightly oversized.

Several tests on this type bolted bracket connection were conducted at Lehigh University (Adan and Gibb 2009).Then it was patented with the United States Patent and Trademark Office by of Steel Cast Connection LLC. The advantage of using casting is that it will not have HAZ issues or residual stresses that would be found in welds.



Notes:
a. 6 mm minimum, 12 mm maximum.
b. 25 mm minimum
c. 30° (±10°)
d. 50 mm minimum
e. 12 mm minimum distance, 25 mm maximum distance, from end of fillet weld to edge of weld access hole.

FIG. 17.30 Welded un-reinforced Flange – welded Web (WUF-W) moment connection (a) connection, (b) Detailing of connection

Source: AISC 2010 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications with Supp. No. 1 and Supp. No. 2.* Reprinted with permission from AISC.

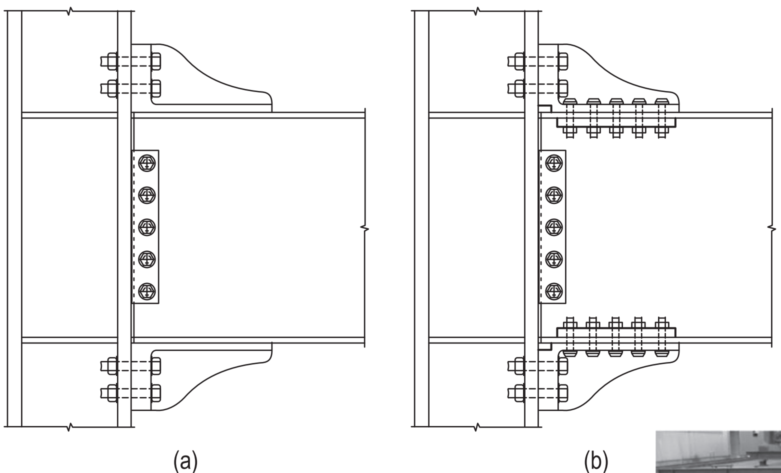


FIG. 17.31 Kaiser bolted bracket (KBB) moment connections (a) Beam welded to bracket, (b) beam bolted to bracket

Source: AISC 2010 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications with Supp. No. 1 and Supp. No. 2.* Reprinted with permission from AISC.

This bracket is available in USA, in various sizes and bolt patterns to match the demand required for strength and ductility. The bracket is shop welded to the beam and field bolted to the column. The brackets used in Wasatch property Management Corporate Headquarters building in Utah, USA are shown in Fig. 17.32. The use of these cast steel brackets resulted in a saving of $3,000 per joint, due to the avoidance of complete penetration field welding, doubler plates, continuity plates and ultra-sonic testing and also from the reduced beam tonnage(Cartwright,2006). The design procedure and detailing requirements for these connections are given in AISC 358-2010. More information about these KBB moment connections can be had from www.steelcastconnections.com.

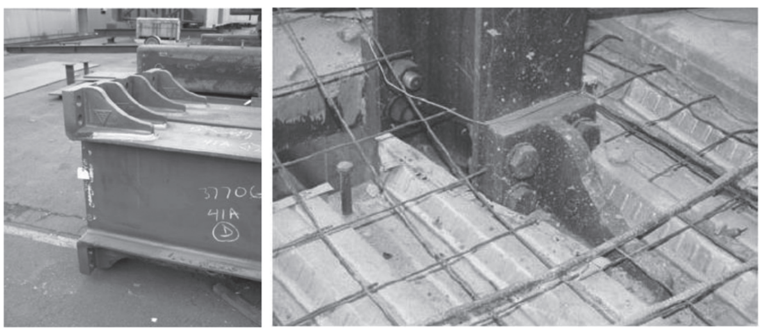


FIG. 17.32 Use of Kaiser bolted brackets in Wasatch Property Management Corporate Headquarters building in Utah, USA

Courtesy: William C. Gibb of Steel Cast Connections LLC

SidePlate Connections

SidePlate® is a patented, award winning SMF connection developed by SidePlate System Inc., Los Angeles, U.S.A. It can be used in moment frame and braced frames, for both uniaxial and biaxial framing applications. SidePlate™ uses a series of flange and web plates with horizontal shop and field fillet welds to create a rigid, fixed connection between wide-flange columns and beams (see Fig. 17.33). In the shop, beam flange cover plates are fillet welded to the top and bottom of the beam

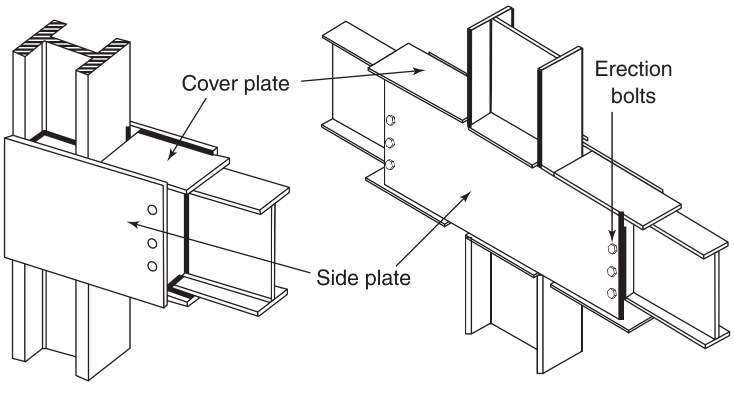
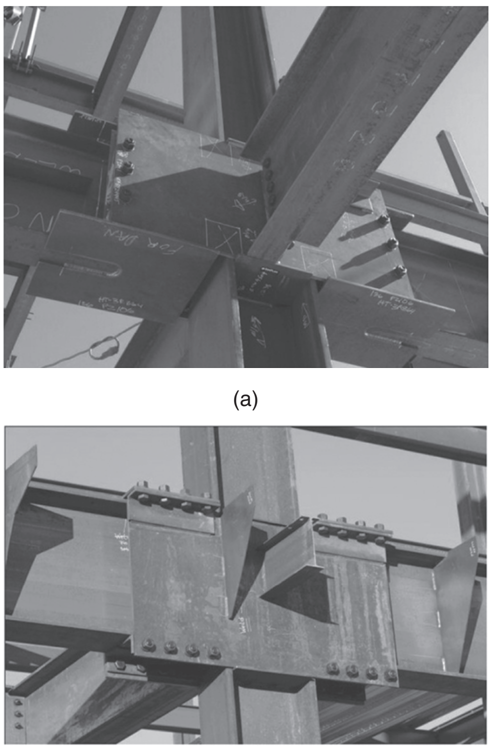


FIG. 17.33 SidePlate® Connection

Courtesy: Henry Gallart, President, SidePlate Systems, Inc.



(a)

(b)

FIG. 17.34 SidePlate® (a) Field fillet-welded version (b) Field-bolted version

Courtesy: Henry Gallart, President, SidePlate Systems, Inc.

and erection angles are fillet welded to the web. Column side plates and horizontal shear plates are fillet welded to the column web. Figure 17.34 shows the field fillet- welded version as well as the new field bolted version of SidePlate® connection.

In the field, column trees are erected and full-length beams are hoisted into place between the two pairs of column side plates. The beams are bolted to the column side plates (typically consists of four to six A325 bolts) and four horizontal fillet welds are applied to complete the beam column connection. Column design options include rolled or built-up 'H' wide flange sections, and tube steel or built-up box columns. Up to three storey-length of column trees can be fabricated, transported and erected and are typically transported by trucks or rail car to the job site.

In 2009, SidePlate System Inc. modified the original SidePlate® connection, which included a beam stub in a column tree assembly and required a field CJP beam splice. The new connection, referred to as the SidePlate FRAME™, has eliminated the beam stub, CJP splice, and reduced the fillet weld sizes by about half of the original connection [Cordova and Hamburger (2011)]. Thus, the new SidePlate FRAME™ is a 100% fillet welded connection and eliminates the need for CJP welds. These improvements, which resulted from extensive analytical and experimental studies, have reduced shop fabrication time and improved constructability [Cordova and Hamburger (2011)]. The cyclic rotational capacity of this moment connection system was tested at the Charles Lee Powell Structural Research Laboratories, University of California, San Diego, which showed that it exceeds all beam-to-column prequalification requirements of ANSI/AISC 341-05.

According to SidePlate System Inc., there is no limit on column or beam size; hence, deeper and lighter sections can be used reducing the total steel weight and cost. It also recommends that panel zones can be modeled as completely rigid and increased beam stiffness provided by the column side plates can be included in the analytical model; this will again result in savings in steel weight. The side plates of this system can also be extended to permit attachment of braces for dual systems, and dampers for energy dissipated structures. More information, on this patented connection can be had from www.sideplate.com.

The second supplement to ANSI/AISC 358s2-14 included the proprietary SidePlate® moment connection to the roster of prequalified connections and included the design procedure.

ConXtech's ConXL™ and ConXR™ Moment Connections

The American inventor Robert J. Simmons introduced the ConXR™ moment connection in 2004, through his company ConXtech. Two types of systems are currently available: ConXR™ and ConXL™. The ConXL connection is pre-qualified as a fully restrained moment connection of wide flange beams to concrete-filled 400 mm HSS or built-up box columns. The ConXL connection forms a compression collar around a square column using high-strength pretensioned bolts, with no field welding. The compression collars are made up of collar corner and collar flange assemblies. The collar corner assemblies, either forged or cast steel, are attached to the column and the forged steel collar flange assembly is attached to the end of a beam. The individual interlocking collar components are manufactured with contact surfaces milled with three-dimensional tapers designed to safely align and lock together for ease of erection during field assembly.

A column can accept up to four moment-connected beams, one at each face of the column (see Fig. 17.35). Note that each compression collar at a moment node (intersection of moment beams and moment column) must be completed with *flange collars* at each face of the column. At the building perimeter or building corners, collar *flange blanks* (collars with no beam attached) are used to complete the compression collar around the column (Cordova and Hamburger, 2011). All beams connecting to a moment node can have different weights, but are required to have the same nominal depth. Beams typically use a reduced beam section (RBS) to efficiently meet strong-column/weak-beam requirements, but an RBS is not a requirement. Four collar corner assemblies are required at each moment node.

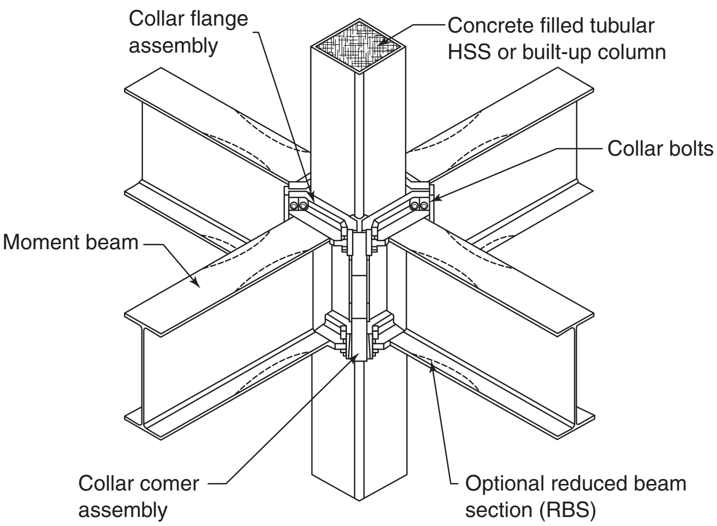


FIG. 17.35 Assembled ConXL™ moment connection

Courtesy: ConXtech Inc.

In a shop environment, *beam collar flanges* are welded into assemblies with a *web extension* connecting the top and bottom collar flanges. The *web extension* is manufactured in different depths allowing standardized collar flanges to be paired with beams of various beam depths. The column collar corners are similarly shop welded into assemblies with a collar corner middle piece of various lengths connecting the standardized top and bottom collar corners. The collar flange assemblies are then welded to the ends of wide flange beams, and *collar corner assemblies* are welded to the corners of the column at the proper floor framing elevations.

On the job site, beams are lowered into the tapered *collar corner assemblies* on the column. Once all beams (or *collar flange blanks* used at the perimeter and corners) are erected around a column, the *collar flanges* are bolted to one another around the column using high-strength bolts. The four *collar flanges* at the column faces forms a compressive collar around the column when the bolts are pretensioned. A ConXL moment node will have compression collars at the same elevation as the beam's top and bottom flange, which creates a load transfer path in bearing at the column that is designed to allow the beams to develop their full sectional moment capacities.

In addition to the ConXL connections, which are used in frames with spans of 5.5m or more, ConXtech also offers the ConXR connection which utilizes a 200 mm HSS column and can be used in shorter spans of 2.5m to 6m. In the United States, the ConXL moment connection has been prequalified for use as a special and intermediate moment frame connection by the AISC Connection Prequalification Review Panel and is published in the *AISC Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 358-s1-11) under Chapter 10 – ConXtech ConXL Moment Connection. The ConXR moment connection meets prequalification requirements for a SMF connection through full-scale cyclic testing as per Chapter K of the Seismic Provisions for Structural Steel Buildings (AISC 341-10). The ConXtech moment connections can also be used as an ordinary moment connection.

Both ConXtech connections can also be used in a bi-axial moment-resisting frame. They provide lateral resistance about 2 axes, resulting in reduced member sizes and foundation loads. A ConXtech design typically utilizes most of the column to beam connections as moment connections, the resulting redundancy also providing the key resistance to *progressive collapse*. The advantages of using this patented system are ease of system detailing, fabrication, erection and lack of field welding; all these factors leading to reduced time and cost. The design steps are provided in AISC 358-10 and experimental validation is provided by Seek and Murray (2005). More information on this system may be found at www.ConXtech.com.

The details of other moment connections approved for inclusion in the 2016 edition of AISC 358, i.e. *Double Tee and Simpson Strong Tie Strong Frame moment connection* may be found in AISC 358-16 and www.strongtie.com/products/strongframe/special_mf/intro.asp respectively. Another interesting all field bolted connection called the *Pin-Fuse Joint* has been developed by Skidmore, Owings & Merrill (SOM) which has started its prequalification process. Cordova and Hamburger (2011) provide details of Pin-Fuse Joint and also describe about a cast, high-strength steel connection called *Cast Connex* (www.castconnex.com).

17.10 BRACED FRAMES

These systems consist of steel frames with diagonal steel braces placed in selected bays. Floors are cast-in-place concrete slabs or metal deck and concrete (see Fig. 17.36). Braced frames provide resistance to lateral forces acting on a structure. The members of a braced frame act as a truss system and are subjected primarily to axial stress. Depending on the diagonal force, length, required stiffness and clearances, the diagonal members can be made of double angles, channels, tees, tubes or even wide flange shapes. Besides performance, the shape of the diagonal is often based on connection considerations. The braces are often placed around service cores and elevators, were frame diagonals may be enclosed within permanent walls. The braces can also be joined to form a closed or partially closed three dimensional cell so that torsional loads can be resisted effectively. A height-to-width ratio of 8-10 is considered to form a reasonably effective bracing system. Braced frames are most effective at the perimeter of the building, where they can control the torsional response of the building. Braced frames can also be an effective system for seismic retrofit due to their high stiffness and because they can be assembled from pieces of relatively small size and weight. Braced frames are inherently stiff. Thus, for the structures of the same height, braced frames will normally have a lower period than a moment frame.

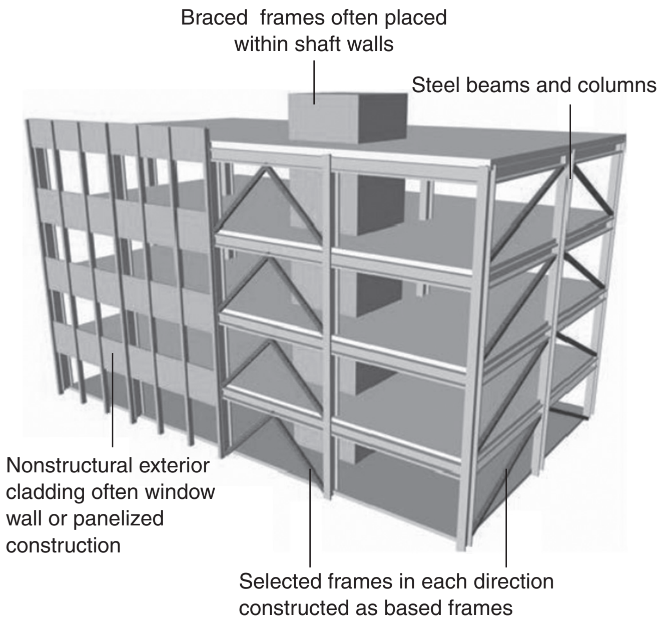


FIG. 17.36 Braced frames

Source: FEMA 454-06

Braced frames may be grouped into concentrically braced frames (CBFs), and eccentrically braced frames (EBFs), depending on their ductility characteristics. In addition, CBFs are subdivided into two categories, namely, ordinary concentrically braced frames (OCBFs) and special concentrically braced frames (SCBFs).

17.10.1 Concentrically Braced Frames (CBFs)

Concentrically braced frames (CBFs) are structures resisting lateral loads through a vertical concentric truss system, the axes of the members aligning concentrically at the joints. In recent years, typical steel construction in regions of high seismic risk has shifted from moment-resisting frames to concentrically braced frames. In CBFs the axes of all members, i.e. columns, beams and braces, intersect at a common point such that the member forces are axial. The Chevron bracing, cross bracing (X-bracing), and diagonal bracing (single diagonal or K-bracing) are classified as concentrically braced and are shown in Fig. 17.37(a) – (d).

Braces buckle in compression and yield in tension. The initial compressive buckling capacity is normally smaller than tension yield force, and for subsequent buckling cycles, the bucking capacity is further reduced by prior inelastic excursion. Hence, bracing systems should be balanced so that lateral resistance in tension and compression is similar in both directions. It requires that diagonal bracing must be used in matched tensile and compressive pairs. This balancing is directly achieved in X-bracing, multi-storey X-bracing and chevron bracing. X-bracing is most commonly used with light bracing on shorter structures (Sabelli et al. 2013).

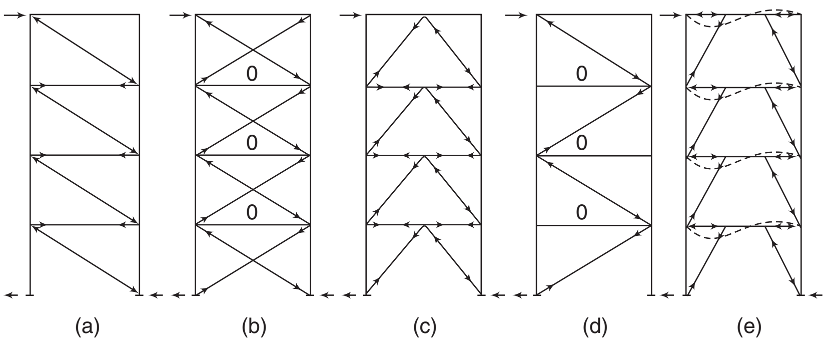


FIG. 17.37 Types of bracings and the load path (a) Single diagonal bracing (b) X-bracing (c) Chevron (inverted V) bracing (d) Single-diagonal, alternate direction bracing (e) Knee bracing (EBF)

On the other hand, EBFs utilize axial offsets to deliberately introduce flexure and shear into framing beams to increase ductility. For example, in the knee bracing shown in Fig. 17.37(e), the end parts of the beam are in compression/tension with the entire beam subject to double curvature bending. (Note that in all the frames shown in Fig. 17.37, a reversal in the direction of horizontal load will reverse all actions and deformations in each of the members). The EBFs are discussed in detail in the next section.

The inability to provide reversible inelastic deformation is the principle disadvantage of CBFs. After buckling, an axially loaded member loses strength and does not return to its original straight configuration. To reduce the possibility of this occurring, during moderate earthquakes, more stringent design requirements are imposed on bracing members (see clause 12.7 of the IS 800:2007). Thus OBCFs are not allowed in seismic zones IV and V and for buildings with an importance factor greater than unity (I >1.0) in zone III; a K-bracing is not permitted in earthquake zones by clause 12.7.1.2 of the code (the inelastic deformation and buckling of K-bracing members may produce lateral deflection of the connected columns, causing collapse) Similarly, tension-only bracing is also prohibited, for the reasons discussed earlier.

Given the variation of design forces with period stipulated in most building codes, and the lower response modification factor, *R,* associated with conventional CBFs (*R* for OCBF is 4 and SCBF is 4.5 as against SMF of 5), the minimum lateral design load stipulated for a concentrically braced frame is generally larger than for a moment-resisting frame. In spite of these higher forces, the inherently large lateral stiffness of braced frames is generally adequate to satisfy the lateral drift requirements in current codes without further increasing the member stiffness and strength (Uriz and Mahin 2008). OBCFs should have to be designed to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians without degradation in strength and stiffness, below the full yield value. The slenderness of bracing members should not exceed 120 and the required compressive strength shall not exceed 0.8 P_d, where P_d is the design strength in axial compression. The design tensile strength of the bracing members should be based on the criteria of gross section yielding (see section 7.6.1) and not on the net section rupture (see section 7.6.2).

Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will resist between 30 to 70% of the lateral load. This is to prevent an accumulation of inelastic deformation in one direction and to preclude the use of tension only diagonal bracing. Bracing members cannot have slender cross sections. Bolted connections are not permitted within the middle one-fourth of the clear brace length, by the code. For braces consisting of built-up sections, the tack fastener spacing should be provided in such a way that the unfavorable slenderness ratio of individual element between the fasteners is less than 0.4 times the governing slenderness ratio of the brace.

According to clause 12.7.3 of the code, the end connections in bracings should be designed to withstand the minimum of the following:

1. A tensile force in the bracing equal to $1.2 f_y A_g$
2. Force in the brace due to load combinations specified in clause 12.2.3 of the code
3. Maximum force that can be transferred to the brace by the system

In addition the connection should be designed for a moment of 1.2 M_p of the braced section about the buckling axis. The gusset plates should also be checked for buckling out of their plane.

It is to be noted that the code in clause 12.2.3 specifies that the frame must be analyzed for the following two additional load combinations, which are different from the load combinations discussed in section 3.15.

1. 1.2 Dead Load + 0.5 Imposed Load (LL) ± 2.5 Earthquake Load (EL)
2. 0.9 Dead Load (DL) ± 2.5 Earthquake Load (EL)

Although the frames may be designed assuming truss behaviour, the large gusset plate connections effectively create stiff, moment-resistant connection rather than a pinned connection. These moments effectively increase the resistance of the frame over that expected from the plane truss analysis. At the same time the moments also introduce unexpected yield and failure modes in the CBF and complicate the current understanding of braced frame behaviour (Roeder and Lehman 2008). Roeder and Lehman also stress that the design and detailing of the gusset plate connection is important. Some guidance is provided by AISC design guide 29.

Extensive damage to CBFs occurred during the 1985 Mexico City, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes. The most severe damage was observed in frames where braces were proportioned to resist tension only, where connections were weaker than the braces attached to them, where braces framed into columns, and where braces were inclined principally in one direction. Since then, the seismic provisions for structural steel buildings (AISC 341) have been updated to prohibit or restrict such conditions. Notwithstanding these improvements, the seismic performance of concentrically braced steel frames may still be less than desired. For example, conventional braces used in the United States have limited ductility capacity and are prone to fracture due to low-cycle fatigue. They also tend to lose compressive strength when loaded in the inelastic range, which leads to a concentration of damage in weakened stories. Prompted by these observations, seismic design requirements for braced frames changed considerably during the 1990s, and the concept of special CBFs was introduced in AISC codes. Additionally, researchers have undertaken a variety of investigations to develop ways to improve the performance of CBFs through:

1. The introduction of new structural configurations
2. The use of special bracing elements, including those utilizing
   (a) composite action
   (b) metallic yielding
   (c) high-performance materials
   (d) friction and viscous damping
3. The introduction of new behavior modes, such as uplifting foundations

Hence SCBFs are now increasingly used instead of CBFs in earthquake zones (Sabelli et al. 2013)

17.10.2 Special Concentrically Braced Frames (SCBFs)

As per clause 12.8 of the code, these frames should be designed to withstand inelastic deformation corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value. They are allowed to be used in any zone and for any building. The slenderness ratio of the bracing members should not exceed 160 and the required compressive strength should not exceed the design strength in axial compression, P_d. Along the line of bracing, braces should be provided such that for lateral loading in either direction, the tension braces resist 30 to 70% of the load. The bracing and column sections used in SCBFs should be plastic sections. The bracing members should be made of steel meeting the requirement of CVN impact value greater than 27J at – 30ºC.

These provisions are for X-braces only. For other types of bracings such as Chevron or V-type bracings, and for eccentrically braced frames, the code does not give any guideline. More information about the design of such bracings may be found in Becker (1995), Becker and Ishler (1996), Bruneau, et al. (1997), Bozorgnia and Bertero (2004), and Williams (2004).

The connections in a braced frame may be subjected to impact loading during an earthquake and, in order to avoid brittle fracture, must be designed to withstand the minimum of the following:

1. A tensile force in the bracing equal to $1.1 f_y A_g$
2. The force in the brace due to the following load combinations
   (a) 1.2 DL + 0.5LL ± 2.5 EL
   (b) 0.9 DL ± 2.5EL
3. The maximum forces that can be transferred to the brace by the system.

The connection should be checked to withstand a moment of 1.2 times the full plastic moment of the braced section about the buckling axis and for tension rupture, and block shear under the aforementioned loading. The gusset plates should be checked for buckling out of their plane, and sufficient length should be provided for plastic hinge formation.

Recent research has shown that the current practice of providing a linear clearance of twice the thickness of gusset plates (see Fig. 17.38a), leads to thicker and larger size of gusset plates. This creates a rotationally stiff joint, which limits the rotation of the connection and leads to extensive frame yielding. Based on the recent research, Roeder and Lehman (2008) suggest providing an elliptical clearance of eight times the thickness of gusset plate (see Fig. 17.38b). This will not only result in smaller, thinner and compact gusset plates, but also greater ductility and inelastic deformation of the system. Welds joining the gusset plate to the beam and column should be sized using the plastic capacity of the gusset plate rather than the expected resistance of the brace.

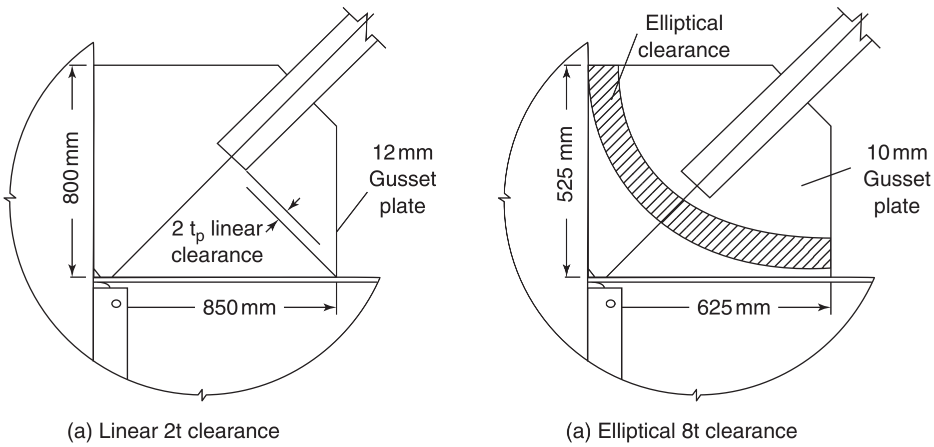


FIG. 17.38 Improved connection detail for CBFs

Source: Roeder and Lehman 2008. Reprinted with permission, STRUCTURE magazine, February 2008.

Similar to OCBFs, bolted connections are not permitted within the middle one-fourth of the clear brace length, by the code. For braces consisting of built-up sections, the tack fastener spacing should be provided in such a way that the unfavorable slenderness ratio of individual element between the fasteners is less than 0.4 times the governing slenderness ratio of the brace.

The code also specifies that the columns of SCBFs should be plastic sections and the splices should be located within the middle one third height of the column. The splices should be capable of resisting at least the nominal shear strength and 50% of the nominal flexural strength of the smaller connected section.

Conventional SCBF systems have inherent problems due to the vastly different compression and tension capacities of the braces. During a major seismic event the compression brace will most likely buckle resulting in the companion tension brace resisting the majority of the demand. Recent full scale testing of a SCBF by the University of California at Berkeley (Uriz and Mahin 2008) highlighted relatively high rate of failure of traditional CBFs in several recent earthquakes. It also pointed out a variety of failure modes such as connections and details unable to develop the full tensile capacity of the braces, and local buckling and fracture of plastic hinge regions at the mid-span of buckled braces. In addition, tests and post-earthquake reconnaissance investigations suggest that CBFs with relatively robust braces may be susceptible to a number of other failure modes, including fracture of the connection of the gusset plates to the supporting beams and columns, failures in columns or base plates. However, when we use *buckling-restrained braced frame* (which is a special case of a special concentrically braced frame), the high ductility of the braces results in very small probability of collapse, and negligible loss in lateral load capacity, even for very rare events (see Section 17.13 for the details of buckling-restrained braced frame). Moreover, replacing the BRB after a major earthquake and bringing back the system to the original condition is also considerably easier than in SCBFs.

Use of the *uniform force method* (AISC Design Guide 29, 2014) provides more compact gusset plates and less expensive designs. Using the lower bound theorem of limit analysis and the uniform force method, this guide addresses: brace-to-gusset connections, orthogonal and non-orthogonal connections, chevron or K-bracing, eccentric braces, connections at column base plates, both non-seismic and seismic conditions, and gusset plate stability. The guide also includes extensive, complete design examples considering every applicable limit state. Analysis and design guidance, connection design methods, and detailing and constructability issues of SCBFs may be found in the NEHRP Seismic design technical brief no. 8 (Sabelli et al., 2013).

17.10.3 Eccentrically Braced Frames (EBF)

The bracing member in an eccentrically braced frames (EBF) is connected to the beam so as to form a short link beam between the braces and the column or between two opposing braces (see Fig. 17.39). Thus the eccentric bracing is a unique structural system that attempts to combine the strength and stiffness of a braced frame with the inelastic behaviour and energy dissipation characteristics of a moment frame. The link beam acts as a *structural fuse* to prevent buckling of the brace from large overloads that may occur during major earthquakes. EBFs display better architectural versatility than CBF to provide space for openings.

When an EBF is subjected to lateral load, the axial force induced in the braces is transferred in the form of high levels of shear and bending moment in the link. However, the link is usually subjected to typically low levels of axial force. Consequently, links will normally experience shear and/or flexural yielding during an earthquake and undergo formation of shear or flexural plastic hinges. Thus the links in EBF, by undergoing plastic deformations, allow dissipation of seismic energy and act as fuse to prevent damage in other parts of the frame. In addition, eccentrically braced frames may be designed to control frame deformations and minimize damage to architectural finishes during seismic loading (Williams 2004).

The web buckling is prevented by providing adequate stiffeners in the link. Links longer than twice the depth of beam tend to develop plastic hinges while shorter links tend to yield in shear. Buildings using eccentric bracing are lighter than moment-resisting frames and, while retaining the elastic stiffness of concentrically braced frames, are more ductile. Thus, they provide an economical system in seismic zones. Premature failure of the link does not cause the structure to collapse, since the structure continues to retain its vertical load carrying capacity and stiffness. Although this system has a good seismic behavior, such links are not readily disposable elements -beams would need shoring, floor slabs might require repairs, etc. The design and other details of eccentrically braced systems are provided by Williams (2004) and Bruneau, et al. (1997).

17.11 DUAL SYSTEMS

Dual systems consist of steel frames with concrete slabs or concrete fill over metal deck. Shear walls are provided as vertical transportation cores, isolated walls in selected bays, or as a perimeter wall system (see Fig. 17.40). In this dual system, the shear walls resist most of the lateral force (since they are stiffer than frames). Hence, there is a concern that if these walls fail, the building may suddenly collapse. This concern arises due to the fact that most buildings have only a small number of shear walls with little redundancy. Hence the code stipulates that the moment-resisting frames have to be designed to resist

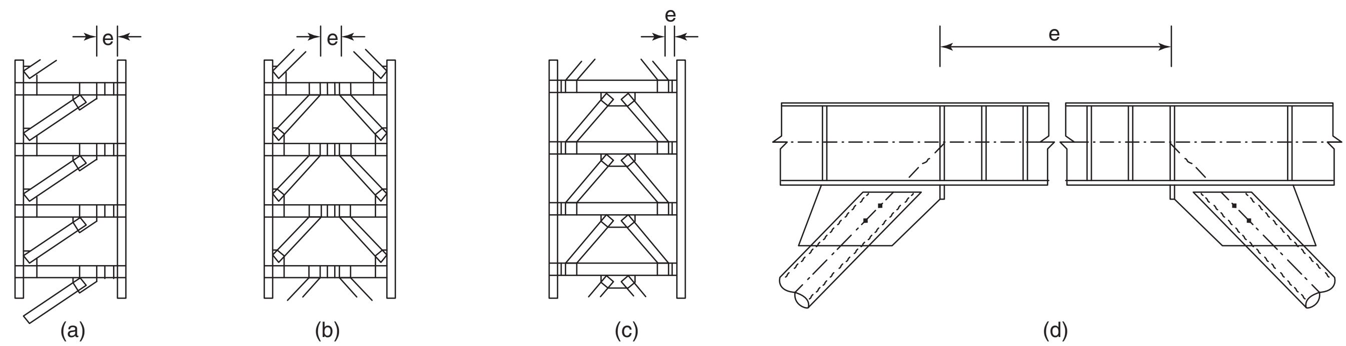


FIG. 17.39 Eccentric bracing system (a)–(c) Common types of bracing (d) Detail

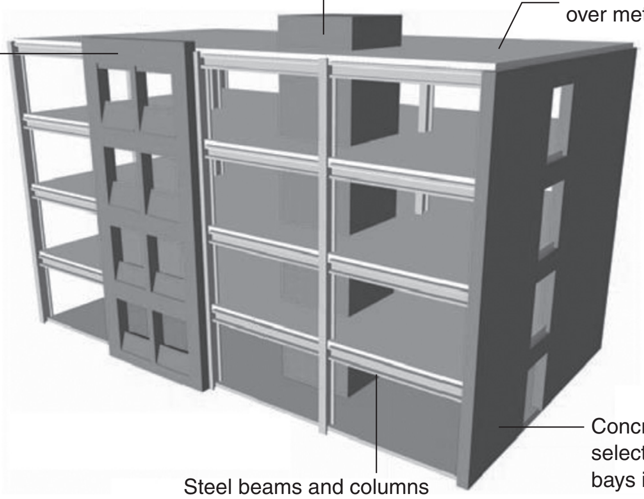


FIG. 17.40 Dual system of steel frames and shear walls

Source: FEMA 454-06

at least 25% of the design base shear, if the analysis indicates that the frames are taking less than 25% of total seismic load (see also the commentary of draft IS 875 code, clause C.4.9), in order to consider the system as a dual system and apply the appropriate *R* factor [ranging from 4.0–5.0, as per Table 7 of IS 1893 (part 1)]. This provision of designing the moment frame for resisting at least 25% of the total base shear provides a 'back-up' resistance to seismic forces.

In general, a dual system has a comparably higher value of *R* since a secondary lateral support system is available to assist the primary non-bearing lateral support system. However, in this case, the storey drifts should be calculated by analyzing the frame for the actual design force and not for 25% of design force (Jain 1995).

17.12 STEEL PLATE SHEAR WALLS (SPSW)

Steel plate shear walls (SPSW) due to the reduced thickness, as compared with concrete shear walls, offer significant advantages in terms of cost, performance and ease of design and erection. They are considered as an alternative to braced frames and can provide equivalent strength and stiffness. National Building Code of Canada (1994), AISC 341-10, and FEMA 450:2004 introduced provisions for the design of SPSW.

Although the post-buckling behaviour of plates under monotonic load has been investigated by several researchers for more than half a century, post-buckling strength of plates under cyclic loading has not been investigated till now (Caccese et al. 1993; Driver et al. 1998; and Kulak et al. 2001). The results of these investigations revealed that plates can be subjected to a few reversed cycles of loading in the post-buckling domain, without damage. However, steel plate shear walls (SPSW) have been used as the primary lateral load resisting system in buildings for more than three decades in United States, Canada and Japan. Some of these buildings include (Seilie and Hooper 2005) the following:

1. United States Federal Courthouse, Seattle, WA – 23-storey building with a height of 107 m
2. Sylmar Hospital, Los Angeles, CA – 6-storey building
3. Canam Manac Headquarters Expansion, St George, Quebec, Canada, 6-storey building. The shear walls had a width of 2.6 m centre-to-centre of columns and an overall height of 23 m. The infill plates are only 4.8 mm thick and the columns are W 250s. At the floor levels, double C 200 × 17 members were used to anchor the tension field in the infill plates as well as carry gravity loads around the shafts. (Driver and Grondin 2001). Figure 17.41 shows the planar SPSW the system adopted for this building.

FIG. 17.41 Planar SPSW system of the Canam HQ building, Canada

Source: Seilie and Hooper, *From Steel Plate Shear Walls: Practical Design and Construction*, Modern Steel Construction, April 2005. Reprinted with permission from AISC.

4. The 50-storey (171 m tall) Hyatt Regency Hotel at Reunion, Dallas, TX
5. The 35-storey (130 m tall) Kobe Office Building, Kobe, Japan
6. Shinjuku Nomura building, Tokyo Japan – 51-storey building having a height of 211 m

Out of these buildings the Kobe Office Building and Sylmar Hospital have withstood fairly significant earthquakes (Kobe earthquake in 1995 and Northridge earthquake in 1994, respectively) and survived without any structural damage (Seilie and Hooper 2005).

There are three different types of SPSW systems:

1. Unstiffened, thin SPSW
2. Stiffened SPSW
3. Composite concrete SPSW

In North America unstiffened, thin SPSW are common while in Japan, the stiffened SPSW system is often used. In this section we will confine our discussion on unstiffened thin SPSW system only.

17.12.1 Advantages of SPSW

The following are the advantages of SPSW system:

1. SPSW allow for less structural wall thickness in comparison to the thickness of concrete shear walls. This results in saving of rentable floor area.
2. Steel savings as much as 50% have been achieved in structures employing a steel plate shear wall system rather than a moment-resisting frame. When compared with reinforced concrete shear walls, the steel system offers reduced foundation costs. This feature makes SPSW more amenable for upgrading the lateral load resistance of an existing structure without overstressing the foundation. However, proper connection at the interface of the steel panel and concrete frame members has to be provided to transfer the interfacial forces. We must also consider compatibility between the nominal ductility of the existing frame and the ductile nature of the steel infill panel. A connection system using HSS tube collars has been developed at the University of Alberta, Canada and tested. This system is found to have several advantages over the more common drilled expansion or adhesive anchors. In addition this system and enhances the behaviour of existing concrete frames (Driver and Grondin 2005).
3. The use of SPSW system reduces construction time. It is not only fast to erect, but also involves no curing period.
4. As mentioned earlier, the thin plates have excellent post-buckling capacity.SPSW ductility is superior to braced frames and even moment frame systems. It has been found that the system can survive up to 4 to 5% drift without experiencing significant damage (which is more than that expected in many moment frame systems). Even though some pinching and tearing close to the corners of the panel were observed due to bending, it did not reduce the plate capacity and stiffness (Astaneh and Zhao 2002; Driver and Grondin 2001).
5. As pointed out earlier, at least two buildings that use SPSW as their primary lateral force-resisting system have experienced significant earthquakes and survived with insignificant structural damage (Astaneh and Zhao 2002).

Large-scale laboratory tests have been conducted to verify the behaviour of SPSW systems (Driver et al 1998). These tests confirmed the high initial stiffness, large energy dissipation capacity and great ductility of these systems, even after a large number of extreme load cycles. The Canadian Standard for structural steel design (CAN / CSA – S16 2001) is the first standard to include provisions for unstiffened SPSW design, though it included analysis and design provisions in the 1994 edition itself in a non-mandatory appendix. Similar provisions have been incorporated in the AISC seismic provisions for steel buildings, where it is denoted as *special plate shear wall* (ANSI / AISC 341 – 10).

However, the SPSW system has the following disadvantages:

1. SPSW systems are more flexible than concrete shear walls. Hence it may be necessary to provide additional flexural stiffness to SPSW. One way of improving the flexural stiffness may be to provide large concrete composite concrete infill steel pipe columns at all corners of the core wall. This type of stiffening will also improve its overturning capacity.
2. Excessive initial compressive force in the SPSW panel may delay the development of the tension-field action. Hence, the construction sequence may be so scheduled to avoid build up of excessive compression in the panel.

17.12.2 Analysis and Design of SPSW

A typical SPSW system consists of steel plate panels (web element), vertical boundary element (columns) and horizontal boundary elements (beams). Thus, in a SPSW system, the building columns correspond to the flanges in a plate girder and the floor beams correspond to the girder vertical stiffeners. However, unlike girders, sizeable top and bottom horizontal bracing elements are required in SPSW to anchor the significant tension fields that develop at these ends of the structural system. Also the use of wide flange structural shapes (which have substantial in-plane bending stiffness) for the vertical and horizontal boundary elements favorably impacts orientation of the angle of development of the tension field action, and makes possible the use of very slender webs. For these reasons, the use of beam and plate girder design provisions as discussed in Chapters 10 and 11 are not appropriate for the design of SPSW (Berman and Bruneau 2004).

The key principal for design in AISC 341 is that significant inelastic deformation capacity in SPSW is provided primarily through web plate yielding and as plastic-hinge formation in the ends of *horizontal boundary elements* (HBEs). *Vertical boundary elements* (VBEs) are not expected to yield in shear; VBEs are not expected to yield in flexure except at the column base. It has been found that yielding of the web in SPSW occurs by development of tension field action at an angle of about 45° (in the range 38 - 43°) from the vertical and buckling of the plate in the orthogonal direction. The sizing of vertical and horizontal boundary elements makes it possible to develop this tension field action across the entire webs.

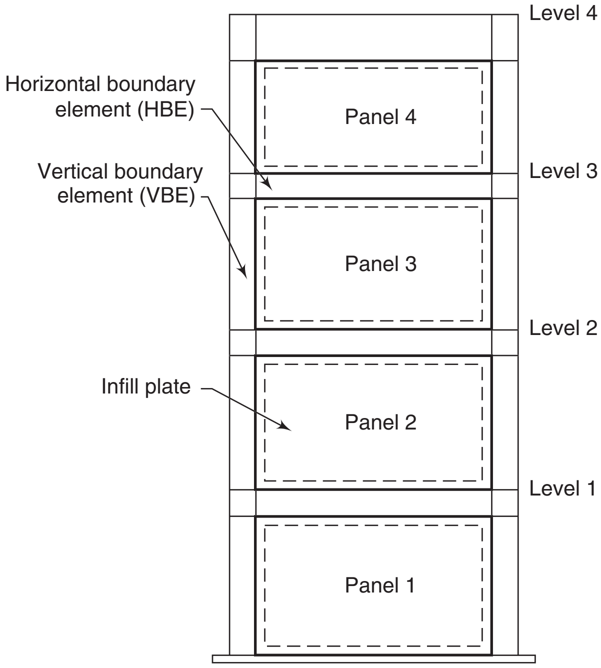


FIG. 17.42 Typical steel plate shear wall

For the purposes of analysis each web plate can be modeled as a series of inclined pin-ended struts or strips oriented at an angle α (see Figs 17.42 and 17.43). A minimum of 10 struts per panel has to be used to provide realistic results. Timler and Kulak (1983) derived the equation for the inclination angle α (in radians) of the tension field as the angle between the direction of the strip and the vertical direction as given in Eqn (17.15) (ANSI / AISC 341 – 10):

$$\tan^4\alpha = \frac{1+\left(\dfrac{t_w L}{2A_c}\right)}{1+t_w h\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)} \qquad (17.15)$$

where h is the distance between the centerlines of horizontal boundary elements (mm), A_b is the gross-sectional area of horizontal boundary elements (mm^2), A_c is the gross-sectional area of vertical boundary elements (mm^2), I_c is the moment of inertia of vertical boundary element about axis perpendicular to the plane of the web plate (mm^4), t_w is the thickness of the web, and L is the distance between center line of vertical boundary elements (mm).

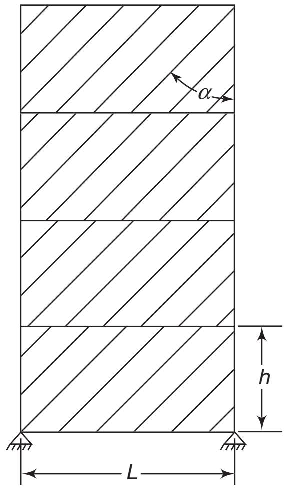


FIG. 17.43 Strip model representation of a SPSW

The design shear strength of the panel (with a system over strength factor of 1.2) is given by (ANSI / AISC 341 – 10)

$$V_n = 0.42 f_y t_w L_{cf} \sin 2\alpha / \gamma_{m0} \qquad (17.16)$$

where, t_w = thickness of the web (mm), L_{cf} is the clear distance between flanges of vertical boundary elements (mm), α is the angle of web yielding in degrees, as measured relative to the vertical. The angle of inclination, α, is permitted to be taken as 40°, or can be calculated as per Eqn (17.15), and γ_{m0} = partial safety factor = 1.1

The ratio of panel length to height, L/h should be limited to $0.8 < L/h < 2.5$. This limit has been specified based on past research, which has not investigated the seismic behaviour of SPSW having L/h greater than 2.0. Also the modeling with strips is reasonably accurate only when L/h is greater than 0.8. Past research has focused on walls with L/t_w ratio ranging from 300-800. Drift limits will indirectly constrain this ratio. Since thicker plates will delay the development of tension – field action, a minimum value of 180 may be adopted for L/t_w ratio.

ASCE 7-10 also specifies the values for variables used in design: Seismic response modification coefficient $R = 7$; Over strength factor $\Omega_o = 2$; Deflection amplification factor $C_d = 6$. For a dual system with SMF the modifications required are: $R = 8$, $\Omega_o = 2.5$, $C_d = 6.5$. (Note these values are related to ASCE 7-10 and not with IS 800:2007, which does not include provisions for the design of SPSW).

The boundary elements are very important to the proper performance of SPSW systems. As mentioned already, the top and bottom horizontal boundary elements will have substantial size (in order to anchor the forces due to tension–field action) and the intermediate horizontal boundary elements are relatively small in size. It is important that the columns (vertical boundary elements) are strong enough such that they will not fail before the plate develops its full tension field. ANSI / AISC 341 – 10 suggests that the columns should have moment of inertia about an axis taken perpendicular to the plane of the web, I_c, greater than 0.0031 $t_w h^4 / L$. Similarly, the horizontal boundary elements (HBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, greater than $0.0031L^4/h$ times the difference in web plate thicknesses above and below. In addition, HBEs shall be braced to satisfy the requirements for moderately ductile members.

For preliminary proportioning of horizontal and vertical boundary elements and webs, a SPSW may be approximated by a vertical truss with tension diagonals. Each web may be represented by single diagonal tension brace within the storey. For an assumed angle of inclination of the tension field, the web thickness, t_w may be taken as (ANSI / AISC 341 – 10)

$$t_w = \frac{2A\Omega_s \sin\theta}{L\sin 2\alpha} \tag{17.17}$$

where A is the area of the equivalent tension brace (mm^2), θ is the angle between the vertical and the longitudinal axis of the equivalent diagonal brace, L is the distance between centre lines of vertical boundary elements (mm), α is the assumed angle of inclination of the tension field measured from vertical (see Eqn17.15), and Ω_s is the system over strength factor, which may be taken as 1.2 for SPSW. The value for the area A, is initially estimated from an equivalent brace size to meet the structure's drift requirements.

The expected tensile strength of the web strips is defined as $R_yF_yA_s$, where A_s = area of a strip = $(L\cos\alpha + H\sin\alpha)/n$, (mm^2), L = width of panel (mm), H = height of panel (mm), n = number of strips per panel and n should be taken greater than or equal to 10.

As mentioned earlier, one of the critical factors limiting the implementation of SPSWs is the large column sizes required to resist the combined axial and flexural demands from overturning, frame action and web plate forces (Berman, 2014). Recent research by Tsai et al. (2014) has developed recommendations for design that allow the formation of the column plastic hinges at a height of 0.25 to 0.33 times the storey height above the base (instead of forming at the base as previously recommended), where the moment is typically maximum in the compression column. This reduces flexural demands significantly and does not impact performance of the system as long as the column does not form a plastic hinge at the top of the first storey. This will result in a 20% reduction in column weight with no impact on performance.

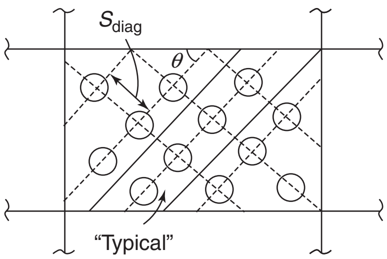
For other provisions like protected zones, demand critical welds, HBE-to-VBE Connections, Connections of Webs to Boundary Elements, column splices, etc. of SPSW reference should be made to AISC 341-10 and AISC Design Guide 20.

Perforated Webs

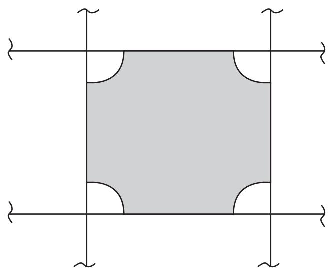
It has been found that the available hot-rolled steel plates are often thicker or stronger than those required as per design for the webs of SPSW in low to medium rise buildings. In such cases, using the minimum available thickness would result in large panel force over-strength, proportionally larger design demands on the surrounding VBE and HBE, and thus the economy of the SPSW may be compromised. Several solutions are available to alleviate this concern, viz.:

(a) Use of light-gauge cold-rolled steel,
(b) Use of low yield steel sheets, and
(c) Use of perforated SPSW(the holes may also be used to allow utilities to pass through).

Perforated webs should have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to vertical (see Fig. 17.44a). This concept has been analytically and experimentally proven to be effective and the system remains ductile up to the drift demands corresponding to severe earthquakes (Vian et al. 2009).



(a) SPSW with perforated web



(b) SPSW with reinforced corner cut-outs

FIG. 17.44 SPSW with openings

The spacing of holes, S_{diag}, should be at least 1.67D. The distance between the first holes and web connections to the HBEs and VBEs should be at least D, but shall not exceed $(D + 0.7S_{diag})$, where D is the diameter of the holes. The panel design shear strength in this case is calculated as

$$V_n = 0.42 f_y t_w L_{cf} \left(1 - \frac{0.7D}{S_{diag}}\right) / \gamma_{m0} \tag{17.18}$$

The stiffness of such regularly perforated infill plates should be calculated using an effective web-plate thickness, t_{eff}, given by:

$$t_{eff} = \frac{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\left(1 - \frac{N_r D \sin\alpha}{H_c}\right)} t_w$$

where, H_c is the clear column (and web-plate) height between beam flanges (mm), N_r is the number of horizontal rows of perforations, t_w = web-plate thickness (mm), and α is the angle of the shortest center-to-center lines in the opening array to vertical(degrees).

The effective expected tension stress to be used in place of the effective tension stress for analysis is $R_y f_y\,(1 - 0.7\,D/S_{diag})$.

AISC 341-10 also permits quarter-circular cut-outs at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cutouts (see Fig. 17.44b). AISC 341-10 may be consulted for the provisions of webs with cut-outs at the corners.

Coupled SPSWs

Even though *coupled SPSWs* may offer the flexibility of using SPSW systems around lift wells of tall buildings, till recently no guidance was available on design methods, steel coupling beam detailing, and general behavior. Borello and Fahnestock (2013) recently developed design concepts for coupled SPSW, and recommend target values for the degree of coupling (ratio of the overturning moment resisted by the individual walls to the total overturning moment). They also showed that significant steel weight savings can be achieved when two individual walls are coupled and by using nonlinear analysis, demonstrated that they have excellent seismic performance.

Self-Centering SPSW

Current research in earthquake engineering research is focused on minimizing residual drift and ensuring simple post-earthquake repair strategies. In this context, *self-centering steel plate shear walls* (SC-SPSW) has been developed. The schematic of this system is shown in Fig. 17.45. In this new system, the web plate is intended to yield under cyclic loading, whereas the boundary elements and PT connection elements remain undamaged. Thus, the post-tensioned beam-to-column connections provide recentering after earthquakes and web plate tension field action provides stiffness and energy dissipation. The damaged web plate can easily be replaced after any major earthquake. Large-scale subassemblage tests, shake table tests on systems with different connections and two-storey full-scale proof-of-concept tests were conducted by Bruneau and associates to study the effects of various design parameters on the system and connection response [Dowden et al. (2012), Clayton et al. (2012)]. These experimental results show that the SC-SPSW system has high ductility, high initial stiffness, recentering capabilities, an overall system response as anticipated, and more energy dissipation than expected.

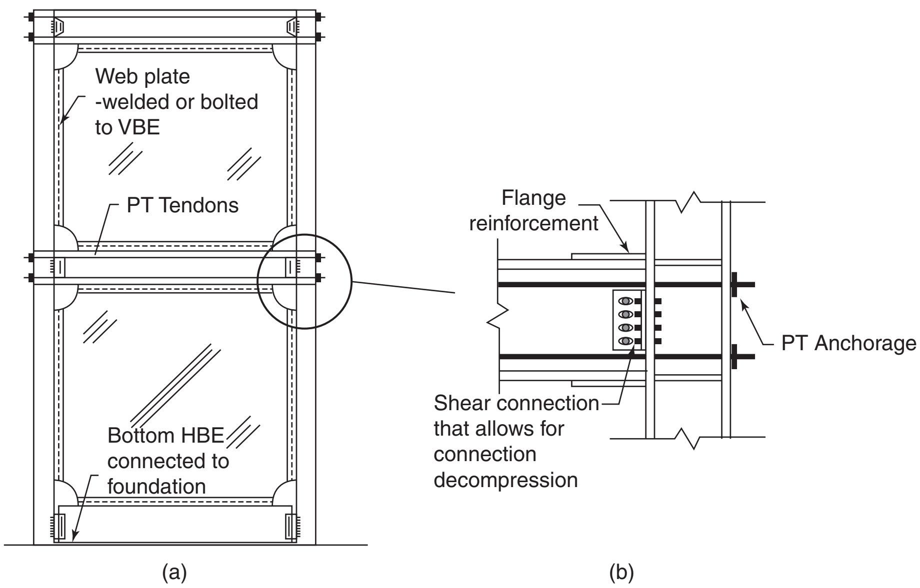


FIG. 17.45 Schematic diagram of self-centering SPSW

17.13 BUCKLING-RESTRAINED BRACES (BRB)

When a concentrically braced frame (CBF) system, as discussed in section 17.10.1, is subjected to earthquake loads, the braces will be subjected to alternate compression and tension. Hence traditionally, CBFs have been treated as high strength, low ductility systems, because, the steel braces show significant strength and ductility in tension, whereas deliver only a fraction of this strength and ductility in compression, due to buckling.

Theoretically, buckling in compression members can be eliminated by providing lateral bracings at close intervals, such that the un-braced length of the member approaches a small value. In the 1980s Prof. Akira Wada of the Tokyo Institute of technology, developed a system, called *Un-bonded Brace*^{TM} in collaboration with Nippon Steel Corporation; his inspiration came from the collarbone of the human body. In this system, which resembled a typical bone (bigger at the ends and a reduced section in the middle), Euler buckling of the central steel core is prevented by encasing it over its entire length in a steel tube.

The load-resisting component of a BRB is a steel core restrained against overall buckling by an outer casing filled with concrete, which is the restraining mechanism preventing buckling (see Fig. 17.46). Bonding of the steel core to the concrete is prevented during the manufacture to ensure that the BRB components remain separate to prevent composite action that would change the behavior. It has to be noted that that the steel rectangular hollow section (RHS) core is divided into five segments: the central restrained yielding segment with a reduced section within the RHS; restrained, non-yielding transition segments of larger area than the yielding segment at both ends within the RHS; and unrestrained,

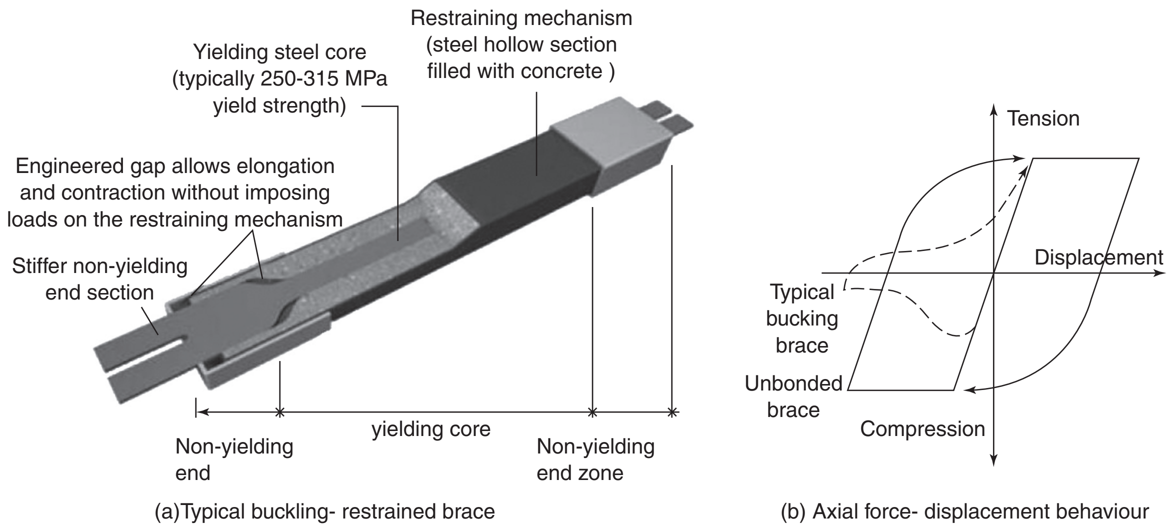


FIG. 17.46 Typical buckling-restrained brace and its behaviour as opposed to conventional brace

Source: Specifying Buckling-Restrained Brace Systems, Modern Steel Construction, November 2009. Reprinted with permission from AISC.

non-yielding connection segments at both ends that extend past the RHS and connect to the frame, typically by means of gusset plates. By confining the inelastic behavior to axial yielding of the steel core, great ductility can be achieved by the brace itself. The ductility of the steel material is realized over the majority of the brace length. Thus the hysteretic performance of these braces is similar to that of the steel core material (López and Sabelli 2004). Fig. 17.46(a) shows a typical WILDCAT^{TM} BRB manufactured by Star Seismic (www.starseismic.net). Fig. 17.46(b) shows the comparison of the behaviour of a BRB and a conventional brace. Most of the BRBs developed to date are proprietary, but all of them are based on the similar concept. Some of them are Unbonded Brace^{TM} manufactured by Nippon Steel Corporation, CoreBrace^{TM} by CoreBrace, WILDCAT^{TM} and Star Seismic™ Modular Systems by Star Seismic, and POWERCAT^{TM} by PKM steel.

Interestingly a similar concept of '*core loaded sleeved strut'* was originally proposed by Er. B.N. Sridhara of Bangalore. This sleeved strut was experimentally studied for compression load carrying capacity in IIT Madras by Kalyanaraman et al. 1994, Prasad 1992 and Sridhara 1990. Sridhara also obtained an US patent for his invention and Star Seismic^{TM} and CoreBrace^{TM} are now manufactured in USA using his patent. Interested reader may consult Uang and Nakashima (2003) for a more detailed summary on the background and historey of the buckling-restrained braced frames.

A *buckling-restrained braced frame* (BRBF) is a structural steel frame which resists a building's lateral forces by using the buckling-restrained brace (BRB). A BRBF is typically a special case of a concentrically braced frame. Advantages of a BRBF over other CBFs are that they exhibit higher ductility and energy dissipation. BRBs most commonly brace a bay of steel frame diagonally or in a chevron pattern. The symmetrical capacity in tension and compression allows BRB braces to be used in single-diagonal configurations without penalty.

According to AISC 341-10, BRBF are expected to provide significant inelastic deformation capacity primarily through brace yielding in tension and compression.

The first BRBF system was installed in the United States at UC-Davis in 2000. Till 2008, more than 150 structures and 20,000 BRBs have been used in the USA alone (López 2008). The 56-floor Los Angeles Convention Center, the Bennet Federal Building in Salt Lake City, Utah, and the 60-storey One Rincon Hill building in San Francisco are among those fitted with the 'sleeved column' braces. BRBs have been used in a variety of applications such as bridges, horizontal diaphragm elements, buildings, high rise outrigger frames, externally anchored braces, and in wind towers. Fig. 17.47 shows BRBFs at Kaiser Permanente Hospital, Vallejo, California**.**

FIG. 17.47 BRBF at Kaiser Permanente Hospital, Vallejo, CA. Reprinted with permission, STRUCTURE magazine, February 2008.

BRBF may be used in future, not only as primary lateral force resisting elements in new construction, but also as supplemental hysteretic dampers in *seismic retrofitting*, since the original motivation behind the initial development of the BRBF system was to use it for seismic retrofitting. Careful analysis and brace sizing can result in a considerable increase in damping without an intolerable decrease in building period.

BRBFs are typically designed using an equivalent-lateral-force method. As in the typical design procedure employed for other concentrically braced-frame types, a linear elastic analysis is done and the frame is subjected to a reduced seismic load in order to determine the required strength and to verify adequate stiffness of the frame (ASCE 7-10 specifies the following to be used in design: Seismic response modification coefficient $R = 8$; Over strength factor $\Omega_o = 2.5$; and deflection amplification factor $C_d = 5$). For a BRBF with braces proportioned according to this method, the difference between the elastic and inelastic deformation modes is much different than for a Special Concentrically Braced Frame (SCBF). Because of this, an inelastic dynamic analysis is not typically required, although inelastic analyses give a much better estimate of brace ductility demands than elastic analyses (López and Sabelli 2004).

For such an elastic analysis to be valid, the braces used in the analysis should correspond to tested brace behaviour, and similarly, brace tests should confirm to the strength and ductility assumed in the analysis. Accordingly, BRBF design is based on the results of successful tests. The BRBF design procedure requires the columns to have the strength to resist the vertical component of the expected yield strength of each brace in a frame. Once BRBs have been designed for adequate strength, the adjoining frame elements are designed to the adjusted BRB strengths corresponding to a storey drift of at least 2% of the storey height or two times the design storey drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading. This design philosophy allows the column to remain elastic during a seismic event, while the BRBs yield and absorb the seismic energy. Hence BRBF performs with a higher degree of ductility than conventional braced frames. The adjusted brace strength in compression is specified in AISC 341-10 as $\beta\omega R_y P_{ysc}$, where β is the compression strength adjustment factor, ω is the strain hardening adjustment factor, P_{ysc} is the axial yield strength of *steel core* (MPa), and R_y is the ratio of the expected yield stress to the specified minimum yield stress, f_y.

Design storey shear has to be shared between the braces and the braced frame columns in proportion to their relative rigidities. BRBF columns resist high tensile loads. As a result, complete joint penetration welds and thick plates are normally specified at the column base, with a vertical gusset stiffening the joint. The moment generated at the column base will be resisted by a concrete-compression anchor-rod-tension couple. The shear generated at the column base will be resisted by steel elements (angles or plates) parallel to the frame and welded to the top of the base plate allowing the anchor rods to resist tension only. Because of these procedures, simple truss-force models are not sufficient, and a model that includes flexural properties is required.

Unlike the massive gusset plates required on the special concentric braced frame, the connections for the BRBF system are of smaller size and require less welding/bolting. Use of the *uniform force method* (AISC Design Guide 29, 2014) provides more compact gusset plates and less expensive designs. Recent testing has demonstrated that gusset plate connections may be critical aspect of the design of BRBF. Thornton and Muir (2009) provide detailing of the gusseted joint with beam hinges, to take into account the potential distortional forces induced by large seismic drifts. BRBFs do not need zipper columns in chevron configurations and require lighter beams. Section F4 of AISC 341-10 contains provisions for BRBF design. A design example of seven-storey office building with BRBFs is provided by López and Sabelli (2004).

It has to be noted that BRBF's cannot be spliced. Welded or bolted splices of braces are not allowed in situations where the braces are likely to be subjected to inelastic demands because that would probably result in undesirable behavior leading to possible brittle fracture (Hussain et al. 2006).

Experimental results conducted by Vargas and Bruneau (2009) indicate that the objectives of the *structural fuse* concept can be successfully achieved in the case of buckling restrained braced frames (i.e., beams and columns performed elastically, while BRBs worked as metallic fuses and dissipated the seismically induced energy). In general, analytical models reasonably predicted maximum response values for the BRB frames. They also proposed an eccentric gusset-plate detail which was found to be effective in preventing performance problems, such as local buckling and out-of-plane buckling of the plates at the connection point.

Composite Moment Frames

Provisions for composite systems like composite ordinary/intermediate moment frames composite SMF, composite partially restrained moment frames, composite ordinary braced frames, composite special concentrically braced frames, composite eccentrically braced frames, composite ordinary shear walls (reinforced concrete walls with structural steel or composite sections serving as boundary elements) and special shear walls (coupled wall systems with steel or composite coupling beams) can also be used to resist earthquake.

17.14 DEVICES TO REDUCE EARTHQUAKE EFFECTS

In addition to the aforementioned guidelines for analysis and design, the structural engineer now has the option of using a variety of devices to ensure safety or serviceability of the structure under severe earthquakes. These devices either isolate the structure from ground vibration or absorb the energy provided by the earthquake to the building. They are similar to the shock absorbers provided in motor vehicles, which absorb the vibrations caused by the undulated road surfaces. But unlike shock absorbers, the vibration control needed for buildings is lateral, because the most destructive seismic motions act in the horizontal direction.

17.14.1 Base Isolation

The concept of *base isolation* is to introduce special bearings (called *base isolators*) between the ground and foundation of the structure, such that the building is isolated from the ground. (This concept is similar to the provision of neoprene bearings at the supports below the bridge decks). Buildings resting on such base isolators are called *base-isolated buildings*. Figure 17.48(a) shows the basic elements of a base isolation system; the supplemental *dampers* shown are optional and hence may or may not be utilized within an isolation system. These dampers absorb energy and thus increase the damping of the building.

By decoupling the structure from ground shaking, isolators reduce the level of response in the structure that would otherwise occur in a conventional, fixed-base building (see Fig. 17.48b). Conversely, base-isolated buildings may be designed with a reduced level of earthquake load to produce the same degree of seismic protection. Qualitatively, a conventional structure experiences deformations within each storey of the structure (i.e., inter-storey drifts) and amplified accelerations at upper floor levels. In contrast, base-isolated structures will experience deformation primarily at the base of the structure (i.e., within the isolation system) and the accelerations are relatively uniform over the height.

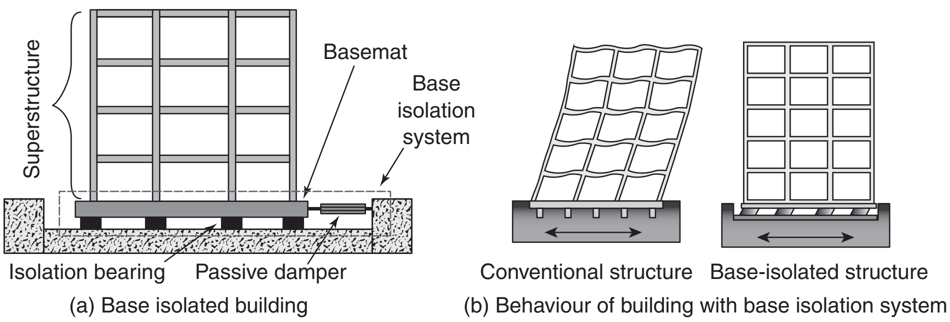


FIG. 17.48 Concept of base isolation

Figure 17.49(a) shows typical acceleration design response spectra for three different damping levels. The major effect of seismic isolation is to increase the natural period which reduces the acceleration and thus force demand on the structure. Thus the forces induced by ground shaking will be much smaller than those experienced by 'fixed-base buildings' directly resting on the ground. In terms of energy, an isolation system shifts the fundamental period of a structure away from the strongest components in the earthquake ground motion, thus reducing the amount of energy transferred into the structure. The energy that is transmitted to the structure is largely dissipated by efficient energy dissipation mechanisms within the isolation system.

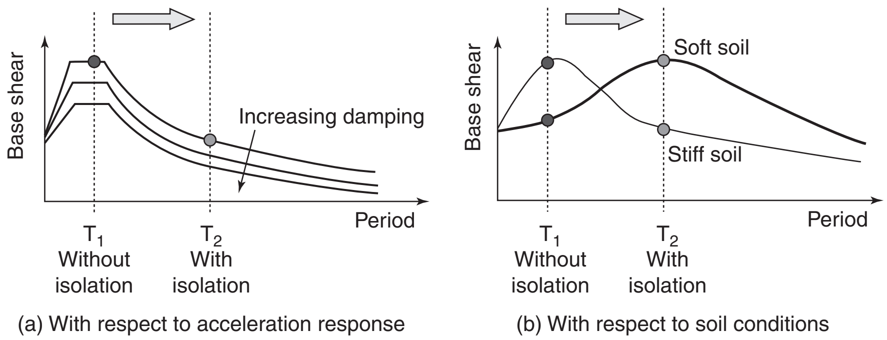


FIG. 17.49 Effect of seismic isolation

However, as shown in Fig. 17.48(b), softer soils tend to produce ground motion at higher periods which, in-turn, amplifies the response of structures having high periods. Hence, seismic isolation systems, should not be used in sites with soft soils, for example in Mexico City (The fundamental natural period of soft soil in Mexico City is found to be approximately 2*s*). Thus, base isolation systems are most effective on structures built on stiff soil and structures with low fundamental period (low-rise building); on the other hand they are least effective on structures built on soft soil and structures with high fundamental period (high-rise building).

Though the first structure which used the principle of base isolation is believed to be the *Tomb of Cyrus* in Pasargadae, a city in ancient Persia (now Iran) in the 6th century BC, the American architect, Frank Lloyd Wright, was the first person to implement the idea of base isolation for isolating Imperial Hotel structure in Tokyo. He provided closely spaced short length piles in the top 2.5 m layer of firm soil that covered a deep deposit of shaky mud. The building survived the devastating 8.3 magnitude 1923 Tokyo earthquake, while other buildings around it collapsed (But eventually the foundation sank irrecoverably into the silt, and the structure was demolished in 1968).

The present day modern base-isolation devices started with the pioneering work done by R. Ivan Skinner, W.H. Robinson, and G.H. McVerry at the Physics and Engineering Laboratory of the Department of Scientific and Industrial Research (PEL, DSIR) in New Zealand during 1977 (they used the World's first isolator developed by them in the William Clayton Building, New Zealand) and later by Prof. James M. Kelly at the University of California at Berkeley. The first base-isolated building in the United States is the Foothill Communities Law and Justice Center, about 97 km east of downtown Los Angeles. Completed in 1985, the building is four stories high with a full basement and sub-basement for the isolation system, which consists of 98 high-damping elastomeric bearings. The superstructure of the building has a structural steel frame stiffened by braced frames in some bays. As of 2015, 500 structures in USA have been seismically isolated (Walters, 2015). During January 2002, the 300-bed district hospital in Bhuj, was the first in India to be installed with 280 lead-rubber and sliding bearings (this hospital replaced the one that collapsed tragically in the Bhuj earthquake). The first large base-isolated building in Japan was completed in 1986. Since the Kobe earthquake, more than 2000 base-isolated buildings, many of them apartment blocks, have been constructed in Japan- total number of buildings with seismic isolation in Japan till April 2015 is 7800 (Walters 2015). Base-isolation is being adopted in several buildings all over the world. As per Walters (2015) more than 4000 buildings in China have been equipped with base isolators. The application of this technology to high-rise buildings is also becoming popular, but requires very large isolators. Isolators of up to 1,600 mm diameter and around 600 mm height are currently available, a size capable of sustaining over 20 MN axial load and 800 mm shear displacement (Nishi et al. 2009). In addition to buildings, seismic isolation has been used for the seismic protection of structures such as bridges, liquefied natural gas (LNG) tanks, and offshore platforms.

The efficacy of this technology was verified during the 1994 Northridge earthquake of California, USA, the 1995 Kobe earthquake of Japan, as well as the 2008 Sichuan earthquake of China. For example, a California hospital remained operational, unlike conventionally built structures in the area (Nishi et al. 2009).

Types of Base Isolation Systems

Many types of isolation system have been proposed and have been developed to varying stages, with some remaining only in concepts and others having installed in several projects. Fig. 17.50 shows the various types of base isolation systems; they may be broadly classified as (a) elastomeric bearings (lead-rubber bearing, high-damping natural rubber bearing, low-damping natural or synthetic rubber bearing, low damping natural rubber with lead core), (b) sliding bearings (flat sliding bearing, spherical sliding bearing, friction pendulum systems), (c) sliding/friction bearing, (d) rolling systems (using cylindrical rods or elliptical bearing), and (f) combined systems (electricide-de-France system and resilient-friction base isolators). Some of the frequently used isolator systems are shown in Fig. 17.51.

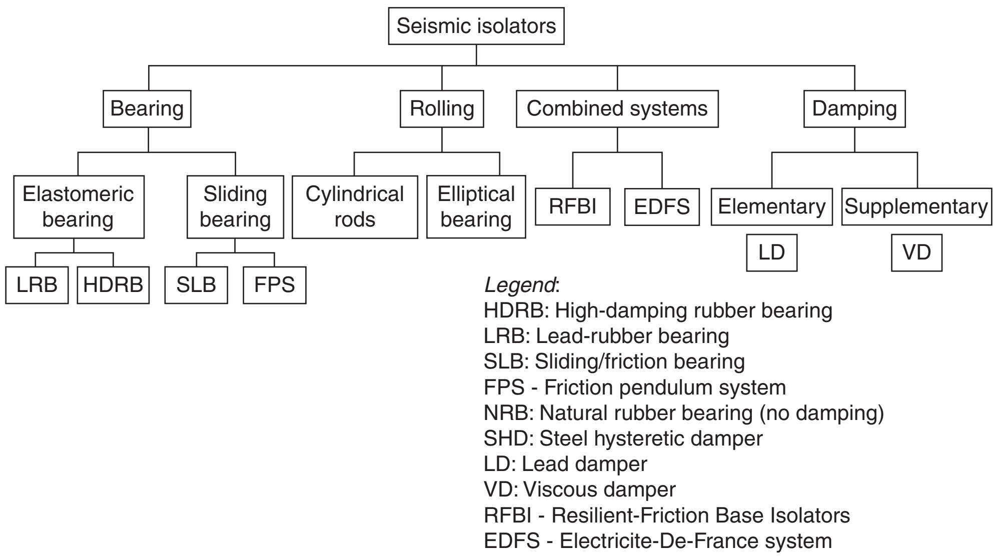


FIG. 17.50 Types of seismic isolators

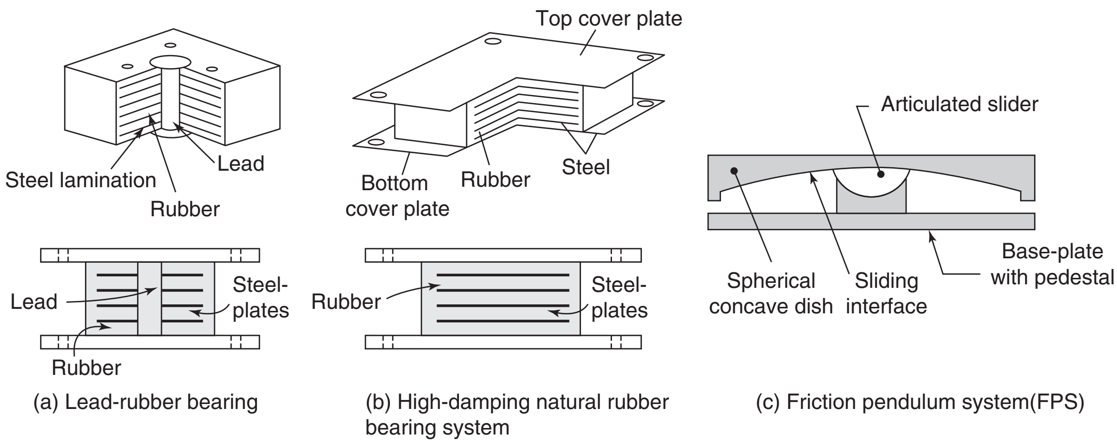


FIG. 17.51 Some frequently used base-isolators

Elastomeric bearing is the most common type of base isolation device, and consist of alternating rubber and thin steel plates layers (about 3 mm thick), firmly bonded to each other (Figs 17.51a and 17.51b). The bearings are constructed by placing un-vulcanized rubber sheets and steel shims in a mold, then subjecting the mold to elevated temperature and pressure to simultaneously vulcanize and bond the rubber. The steel plate reinforcement provides a high compressive stiffness to reduce vertical deflection under the heavy weight of the structure, making the isolator stable. The rubber layers provide the very low horizontal stiffness needed to give the structure a horizontal natural frequency (typically 0.5 Hz), lower than the peak frequencies of an earthquake (Nishi et al. 2009). This decouples the structure from ground shaking, reducing the transmission of earthquake forces into the structure and protecting both the structure and its contents (50–85% reduction has been achieved). In addition, rubber cover is provided on the top, bottom, and sides of the bearing to protect the steel plates. In some cases, a lead cylinder is installed in the center of the bearing to provide high initial stiffness and a mechanism for energy dissipation.

Lead-rubber bearings (LRB) were first introduced and used in New Zealand in the late 1970s. They differ from low-damping natural rubber bearings only by the addition of a lead-plug that is press-fit into a central hole in the bearing. The lead-plug deforms plastically under shear deformation, enhancing the energy dissipation capabilities compared to the low-damping natural rubber bearing (see Fig. 17.51a). After the lead yields, it dissipates energy as it is cycled. Fatigue of the lead is not a concern since lead recrystallizes at normal temperatures. During a large earthquake, a shear (horizontal) displacement of several hundred millimeters may be imposed on the isolators. The rubber layers provide the large shear deformation capacity needed. The service life of the isolators is anticipated to be at least several decades.

Sliding bearings typically utilize either spherical or flat sliding surfaces. The *friction pendulum system* (FPS) bearing utilizes an articulated slider that moves horizontally on a spherical dish-shaped surface and is used extensively in the United States. Usually, the sliding surface is oriented concave down to minimize the possibility of debris collecting on the sliding surface (see Fig. 17.51c). The articulated slider is faced with a Teflon coating. Under horizontal motion the spherical concave dish displaces horizontally relative to the articulated slider and base-plate. Friction between the PTFE type material and stainless steel surface provides frictional resistance and energy dissipation, whereas the radius of curvature of the spherical concave dish provides a restoring force. The most recently developed *triple friction pendulum* version of FPS, patented and manufactured by Earthquake Protection Systems, Inc., contains a compound articulated slider with multiple sliding surfaces to allow control of the sliding sequence and the resulting hysteresis curve (Walters 2015).

Mayes et al. (2012) compared the cost-benefit analysis of isolated and non-isolated buildings and concluded that considering the cost of earthquake insurance premiums, using base isolation without earthquake insurance can be a more cost-effective solution than a conventional fixed based structure with insurance, despite the first cost premium for base isolation.

It has to be noted that normal base-isolation systems provide only horizontal isolation and are rigid or semi-rigid in the vertical direction. A rare exception to this rule is the full isolation (horizontal and vertical) of a building in southern California isolated by large helical coil springs and viscous dampers (Kircher 2012). The implementation of the base isolation requires optimal design, which depends on the magnitude and frequency range of the earthquake that is being considered. Recent research reveals that the base isolation system may be vulnerable for buildings situated in the near-fault and far-fault earthquakes zones. Near-fault earthquakes with a large displacement and long-period pulse, such as the 1994 Northridge earthquake, may lead to over-stretching of isolator and resulting in malfunctioning of the system (Jangid and Kelly 2001). While far-field earthquakes (with its low-frequency components falling into the resonant region of the conventional base isolation system) may result in amplification of destructive responses to the protected structures.

No detailed provisions exist in the Indian codes for seismic isolators. More information on seismic base-isolators, analytical and numerical models, code provisions for seismic isolation, buckling and stability of isolators, design examples, computer applications, and recent trends may be found in Kelly (2012), Kircher, 2012, Naeim and Kelly (1999), Skinner et al. (1993), Walters (2015), and Zhou and Xian (2001).

17.14.2 Energy Absorbing Devices

While a base isolator may be effective at protecting buildings from seismic movements, it cannot necessarily dissipate the energy that it obtains during an earthquake. Designers realized that for controlling seismic damage in buildings and improving seismic performance, a *damping system* (seismic energy dissipating devices) must be used in conjunction with an isolator (see Fig. 17.48a). These devices can be used at the base of the structure or mounted on the superstructure at appropriate locations (usually on diagonal braces). Dampers deplete the energy that would normally keep a building oscillating side to side by applying a restoring force to bring the superstructure and foundation into alignment. They act like the hydraulic shock absorbers provided in automobiles, absorbing the vibration of sudden jerks and transmitting only a part of the vibration above the chassis of the vehicles. *Dampers* were first used in the 1960s to absorb the vibration caused by winds in tall buildings and only since the 1990s are they being used to protect buildings against the effects of earthquakes. When the device merely absorbs the energy during vibration without any energy input from outside, it is termed a *passive device*. On the other hand, if it opposes the vibration by means of an external energy source, it is called an *active device*. We could also have semi-active and hybrid dampers. Commonly used dampers are shown in Fig. 15.52. They include the following (Patil and Reddy 2012):

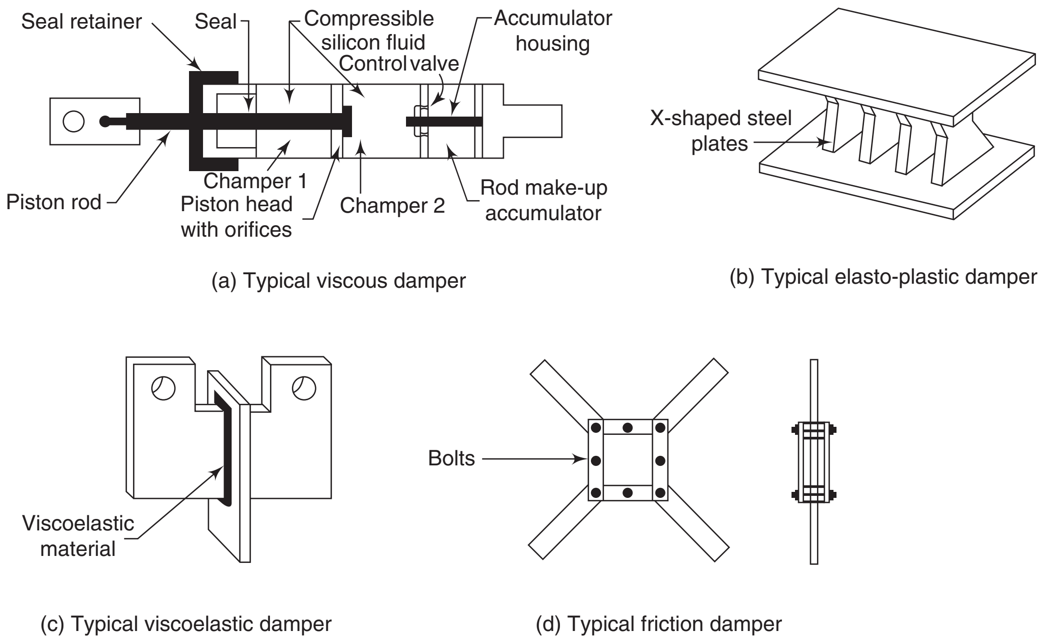


FIG. 17.52 Various types of dampers

1. *Viscous dampers* (VDs)—They consist of a piston-cylinder arrangement filled with a viscous silicon-based fluid. As the damper piston rod and piston head are stroked, fluid is forced to flow through orifices around the piston head, which absorbs the energy.
2. *Friction dampers* (FDs)—energy is absorbed by the friction between two layers, which are made to rub against each other.
3. *Elasto-plastic dampers* (EPDs) or *added damping and stiffness* (ADAS) *dampers* are made of number of small 'X' shaped plates, which yield at small deformation thereby dissipating high amount of energy. These metallic yielding devices is similar in principle to the buckling restrained brace (BRB), discussed in Section 17.13. It has to be noted that BRBs are regarded as being part of a bracing system, rather than as part of a damping system.
4. *Viscoelastic dampers* (VEDs)—containing viscoelastic material, sandwiched between two steel plates, which undergoes shear deformation, thus dissipating energy.
5. *Shape memory alloy dampers* (SMADs) made of nickel–titanium (Ni–Ti) alloy it has an interesting pseudo-elastic property by which the alloy—regains its initial shape when external load is removed. This property is useful in putting back the structure to its original shape. It can also sustain large amount of inelastic deformation (Song et al. 2006).
6. *Lead extrusion dampers* (LEDs) work on the principle of extrusion of lead. It absorbs vibration energy by plastic deformation of the lead, during which mechanical energy is converted into heat; lead gets heated up and on being extruded and recrystallizes immediately and recovers its original mechanical properties before next extrusion.

These dampers are attached to the main structural framing system via a bracing system. The bracing system may be diagonal bracing, chevron bracing or cross-bracing. Other types of dampers are as follows:

Tuned mass dampers (TMD) TMD was first suggested by Frahm in 1909, to attenuate undesirable vibrations in ships. They are extra masses attached to the structure by a spring-dashpot system and designed 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. TMDs have been successfully installed in many tall structures throughout the world, especially for dampening wind induced vibrations. These installations include the 535 m tall CN Tower in Canada, the 60 storey John Hancock Building in Boston, USA, the 305 m tall Center-Point Tower in Sydney, Australia, and the 101 storey (508 m tall) Taipei 101 Tower in Taipei [Tuan and Shang (2014)]. However, TMDs has been shown to have a varying effect on the vibrations of structures subjected to earthquake ground motions and in some cases TMDs can be detrimental to the structure.

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

Hydraulic activators They are active vibration control devices and have a sensor to sense the vibration and activate the activator to counter it. These devices require external energy source, and are expensive.

More information on energy absorbing devices, latest developments, and applications may be found in Soong and Dargush 1997, Kareem, et al.(1999), Spencer Jr and Nagarajaiah (2003), Symans et al, (2008), and Li et al. (2013).

EXAMPLES

Example 17.1: RBS connection design
Design an RBS connection between a IPE 600 M beam and a W460 × 280 × 235 column. The span of the beam is 9 m. The flange reduction will be an arc-cut shape. We will use the guidelines of AISC 358 and gravity loads will be neglected (see Fig. 17.52).

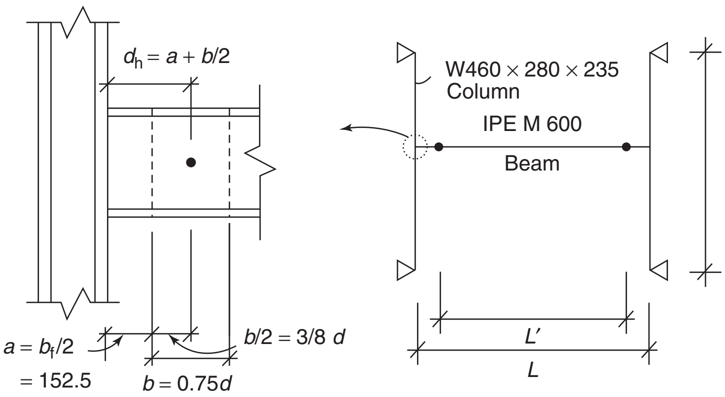


FIG. 17.52

Solution:
HE M 600, $b_{bf} = 305$ mm, $t_f = 40$ mm, $d = 620$ mm, $t_w = 21$ mm, $R = 27$ mm, $Z_{pz} = 7660 \times 10^3$ mm³, Since $t_f = 40$ mm, as per Table 1.9, $f_y = 240$ MPa, $f_u = 410$ MPa
W 530 × 310 × 248, $b_f = 315$ mm, $t_f = 34.5$ mm, $d = 571$ mm, $t_w = 19$ mm, $R = 14$ mm

Let us assume the dimensions of a, b, and c as follows:
$a = 0.5b_{bf} = 305/2 = 152.5$ mm
$b = 0.75d = 0.75(620) = 465$ mm
$c = 0.2b_{bf} = 0.2(305) = 61$ mm.

Step 1: *The plastic section modulus at the center of the reduced beam*

$$Z_{RBS} = Z_{pz} - 2ct_{bf}\left(d - t_{bf}\right) = 7660 \times 10^3 - 2 \times 61 \times 40(620 - 40) = 4829.6 \times 10^3$$

Step 2: *Compute probable maximum moment, M_p, at the center of the reduced beam section*

$$M_p = C_p R_y f_y Z_{RBS}$$

$$C_p = \frac{f_y + f_u}{2f_y} = \frac{240 + 410}{2 \times 240} = 1.35 > 1.2,$$

Hence take $C_p = 1.2$

From Table A3.1 of AISC 341-10, $R_y = 1.5$ for grade Fe 250 steel

Hence

$$M_p = C_p R_y f_y Z_{RBS} = 1.2 \times 1.5 \times 240 \times 4829.6 \times 10^3 / 10^6 = 2086.38\ \text{kNm}$$

Step 3: *Compute the shear force at the center of the reduced beam sections at each end of the beam*

Distance of plastic hinge from column face = $d_h = a + b/2 = 152.5 + 3/8 \times 620 = 385$ mm

Distance between plastic hinges = $L' = L - d_c - 2d_h = 9000 - 571 - 2\times 385 = 7659$ mm

$$V_p = \frac{2M_p}{L'} = \frac{2 \times 2086.38}{7.659} = 544.8\ \text{kN}$$

Step 4: *Compute probable maximum moment at the face of the column*

$$M_f = M_p + V_{RBS} d_h = 2086.38 + 544.8 \times 0.385 = 2296.13\ \text{kNm}$$

Step 5: *Compute M_{pe}, the plastic moment of the beam based on the expected yield stress:*

$$M_{pe} = R_y f_y Z_{pz} = 1.5 \times 240 \times 7660 \times 10^3 = 2872.5\ \text{kNm}$$

Step 6: *Check the flexural strength of the beam at the face of the column*

$$M_f = \phi_d M_{pe} = 1 \times 2872.5 = 2872.5\, kNm > 2296.13\ \text{kNm}$$

Hence OK.

Note: In addition checks in steps 8–11, as per section 17.9.3 should also be done.

SUMMARY

There appears to be an increasing earthquake activity throughout the world. The recent earthquakes have demonstrated that the damages and loss of lives will be extensive if the buildings are not designed and detailed properly. Designing for earthquake is different than designing for other loads, as earthquake codes allow us to consider only a fraction of the earthquake load in the design (as it will be expensive to design for the full earthquake loads, which are also less frequent and not predictable). The codes specify a factor called the response reduction factor (R) and also divide the maximum considered earthquake by a factor to arrive at the design basis earthquake. The response reduction factor thus, represents the combined effect of over strength, redundancy, and ductility. It has to be noted that the values of R may change as and when more research is conducted on the behaviour of different structural systems. The parameters influencing seismic damage are briefly discussed.

Past earthquakes have demonstrated that structures having simple and regular configuration, adequate lateral strength, adequate stiffness, and adequate ductility have performed well. Hence the rules to be followed while planning buildings in high seismic zones are provided and the plan (due to re-entrant corners, torsional, diaphragm discontinuity, out-of-plane offsets, and non-parallel systems) and vertical regularities (stiffness, mass, vertical geometry, in-plane discontinuity in vertical, lateral force resisting elements, weak storey), which should be avoided are clearly indicated. Other aspects of planning such as short column effect and careful design and detailing of cantilevers and floating columns are also discussed. Importance and necessity of considering vertical component of earthquake in structures close to the faults or with heavy mass concentrated at the top are also stressed.

The most common systems adopted to resist seismic forces include: moment-resisting frames, a combined system of moment frames and shear walls, braced frames with horizontal diaphragms and a combination of these systems. Moment-resisting frames are classified as special moment-resisting frame (SMRF), ordinary moment-resisting frame (OMRF) or ordinary moment frame (OMF), depending on the details adopted at and near their joints. The proportioning and detailing requirements of these moment-resisting frames as per IS 800 as well as the strong column-weak beam concept have been explained. Proportioning for drift requirements, considering panel zone shear deformations is also discussed.

Though the recent version of steel code (IS 800) contained provisions for seismic design and detailing, designers are not given guidance to choose proper beam-to-column connections, in SMF and IMFs. After 10 years of extensive research, initiated by Federal Emergency Management Agency, USA (after the 1994 Northridge earthquake in USA and the 1995 Kobe earthquake in Japan, which revealed serious drawbacks in the welded moment connections of SMFs), AISC has developed a few pre-qualified connections, which have shown to provide the required amount of ductility. A brief description of such connections is given to aid the designers. For more details about the design methods, the readers should consult AISC 358-10 and AISC 341-10.

Similarly, braced frames may be classified as ordinary concentrically braced frames (OCBF), special concentrically braced frames (SCBF), and eccentrically braced frames (EBF). These systems are explained briefly.

While designing these seismic-resistant systems, it is necessary to provide adequate redundancy in the structure to prevent progressive collapse. It is also necessary to select a system in which the damaged element during an earthquake could be replaced effectively and easily after an earthquake (similar to a fuse in an electrical system). Two such systems, namely the steel plate shear walls and the buckling-restrained braced frames are also described.

Innovative design concepts have been developed recently to better protect structures, against the destructive effects of earthquakes. Base isolation is a passive structural control system, where isolators are used to decouple the building from its foundations resting on shaking grounds, thus protecting the structural integrity of the building, Four types of base isolation – elastomeric, sliding, rolling and a combination of these are briefly described.

Another recent concept is the use of passive energy dissipation devices called dampers which absorb part of the input energy, thereby reducing energy dissipation demand on primary structural members and minimizing possible structural damage. These devises in the form of viscous fluid dampers, visco-elastic solid dampers, friction dampers and metallic dampers are also briefly discussed.

MULTIPLE-CHOICE QUESTIONS

17.1 Response reduction factor for OMRF is taken in IS 1893 as
(a) 3 (b) 4 (c) 5 (d) 3.5

17.2 Response reduction factor for SMRF is taken in IS 1893 as
(a) 3 (b) 4 (c) 5 (d) 3.5

17.3 When shear walls are also provided in addition to MRFs, MRFs should be designed to resist at least
(a) 30% of the total seismic load
(b) 20% of the total seismic load
(c) 25% of the total seismic load
(d) none of these

17.4 Which of the following influence seismic design?
(a) Duration of earthquake
(b) Distance of site from epicenter
(c) Amplitude of earthquake
(d) All of these

17.5 Normal buildings with up to 20 stories will have fundamental natural periods in the range of
(a) 0.5–20 s (b) 0.1–2.0 s
(c) 1–5 s (d) none of these

17.6 The fundamental period T of soil layer of thickness H, having an average shear wave velocity of Vs is
(a) $T = 4H/Vs$
(b) $T = 2H/Vs$
(c) $T = 8H/Vs$

17.7 To perform well in an earthquake, a building should posses
(a) regular configuration and adequate strength
(b) adequate stiffness and ductility
(c) regular configuration, adequate strength, stiffness, and ductility
(d) adequate strength and ductility

17.8 Which shape of plan is better in earthquake zone?
(a) L (b) O (c) H (d) Y

17.9 Torsional irregularity need not be considered, when the maximum storey drift is less than the average of drifts at the two ends of the building by
(a) 1.2 times
(b) 1.4 times
(c) 2.0 times

17.10 The building should be considered to have extreme torsional irregularity when the maximum storey drift is greater than the average of drifts at the two ends of the building by
(a) 1.2 times
(b) 1.4 times
(c) 2.0 times

17.11 Diaphragm discontinuity should be considered when open areas are greater than
(a) 40% of the gross diaphragm area
(b) 50% of the gross diaphragm area
(c) 60% of the gross diaphragm area

17.12 A storey is considered soft storey when it’s lateral stiffness is less than

(a) 50% of the stiffness in the storey above or 60% of the average stiffness of 3 stories above
(b) 60% of the stiffness in the storey above or 70% of the average stiffness of 3 stories above
(c) 70% of the stiffness in the storey above or 80% of the average stiffness of 3 stories above

17.13 Mass irregularity exists when the seismic weight of any floor is
(a) 150 % more than that of its adjacent floors
(b) 200 % more than that of its adjacent floors
(c) 250 % more than that of its adjacent floors

17.14 Vertical geometric irregularity exists when the horizontal dimension of the lateral force-resisting system in any storey is
(a) more than 130% of that in an adjacent storey
(b) more than 150% of that in an adjacent storey
(c) more than 175% of that in an adjacent storey

17.15 A storey is considered as a weak storey when the lateral strength of that storey is
(a) Less than 75% lateral strength of the storey above
(b) Less than 80% lateral strength of the storey above
(c) Less than 85% lateral strength of the storey above

17.16 Vertical cantilever projections should be designed for
(a) 3 times the horizontal seismic coefficient, A_h
(b) 4 times the horizontal seismic coefficient, A_h
(c) 5 times the horizontal seismic coefficient, A_h

17.17 Horizontal cantilever projections should be designed for
(a) 8/3 times the horizontal seismic coefficient, A_h
(b) 3 times the horizontal seismic coefficient, A_h
(c) 10/3 times the horizontal seismic coefficient, A_h

17.18 Two adjacent buildings with same floor elevation should be separated by a distance
(a) *R*/2 times the sum of the calculated storey displacements
(b) 2R/3 times the sum of the calculated storey displacements
(c) *R* times the sum of the calculated storey displacements

17.19 The vertical seismic coefficient is taken in IS 1893 as
(a) 1/2 times that of horizontal seismic coefficient
(b) 2/3 times that of horizontal seismic coefficient
(c) 1.25 times that of horizontal seismic coefficient

17.20 The common seismic force-resisting system consist of
(a) moment-resisting rigid frames
(b) shear walls
(b) braced frames
(d) all of these

17.21 As per IS 1893, the columns and beams of soft storey should be designed for
(a) 1.5 times the storey shears and moments
(b) 2.0 times the storey shears and moments
(c) 2.5 times the storey shears and moments

17.22 IS 1893 also suggests shear walls in such soft stories which should be designed for
(a) 1.5 times the lateral storey shear force
(b) 2.0 times the lateral storey shear force
(c) 2.5 times the lateral storey shear force

17.23 To have strong-column weak-beam structures, IS 800 suggests the summation strength of columns to beams ratio at a joint be greater than
(a) 1.1 (b) 1.2 (c) 1.5

17.24 Members in SMF should undergo plastic rotation of
(a) greater than 0.025 radians
(b) greater than 0.030 radians
(c) greater than 0.040 radians

17.25 Members in OMF should undergo plastic rotation of
(a) greater than 0.025 radians
(b) greater than 0.030 radians
(c) greater than 0.040 radians

17.26 Which of the following strategies are adopted to improve connection behaviour?
(a) Toughening system
(b) Strengthening system
(c) Weakening system
(d) All of these

17.27 In earthquake zones, concentrically braced frames may have
(a) X-bracing
(b) K-bracing
(c) tension-only bracing

17.28 Joints in OCBF should be designed to have joint rotation greater than
(a) 0.020 radians
(b) 0.025 radians
(c) 0.040 radians

17.30 In special concentrically braced frames, the slenderness of bracing members should not exceed
(a) 125 (b) 150 (c) 160 (d) 180

REVIEW QUESTIONS

17.1 What is the significance of response reduction factor?
17.2 What are the three factors that are considered while arriving at the *R* values?
17.3 State the parameters that influence seismic design.
17.4 Why a layer of soft soil above bed rock has to be considered carefully while designing structures for earthquakes?
17.5 What is meant by liquefaction?
17.6 What are the methods adopted to prevent liquefaction?
17.7 What are the four main attributes a building should posses in order to perform well during earthquakes?
17.8 What are the two types of irregularities that should be considered in seismic design?
17.9 What are the shapes of buildings that are to be avoided? If such shapes are inevitable what precautions should we have to take?
17.10 When torsional irregularities should be considered? When a building is considered to have extreme torsional irregularity? How their effects could be minimized?
17.11 What is diaphragm irregularity?

17.12 How can out-of-plane offset irregularity be avoided? What detailing should be adopted when they are present?
17.13 How do we deal with non-parallel systems?
17.14 When is a building considered to have soft storey?
17.15 What constitutes mass irregularity?
17.16 When is a building considered to have vertical geometric irregularity?
17.17 When is a building considered to have in-plane discontinuity in vertical lateral force resisting elements?
17.18 Which storey is considered as a week storey?
17.19 How to tackle a building with some irregularity?
17.20 What is short column effect?
17.21 How vertical and horizontal cantilever projections are suggested to be designed in IS 1893?
17.22 How can we avoid pounding as per IS 1893?
17.23 Why and when is it important to consider vertical component of earthquake?
17.24 List four systems that can be adopted to resist seismic lateral forces.
17.25 List 4 factors that contributed to the damage of welded joints in the Northridge earthquake.
17.26 What is strong-column, weak-beam concept? How is it considered in IS 800:2007?
17.27 Write short notes on (a) concentrically braced frames, (b) eccentrically braced frames, and (c) moment-resisting frames.
17.28 What is the minimum percentage of lateral load to be resisted by a moment-resistant frame in a dual system?
17.29 What are the provisions in IS 1893 (Part 1):2002 for the design of columns, beams and shear walls in a soft storey?
17.30 What are the provisions in IS 800:2007 for the design of members in SMF?
17.31 How storey drift is checked in a SMF as per IS 1893 (Part 1):2002? How is it checked differently in ASCE 7-10?
17.32 How do panel zone shear deformations affect the total shear mode of drift?
17.33 State some of the rules specified in IS 800:2007 for beam-column joints and connections of SMF.
17.34 How the column web or doubler plate thickness is determined empirically by using IS 800:2007?
17.35 State some of the advantages and disadvantages of SMF over braced frames.
17.36 How the design of an OMF differs from an SMF as per IS 800:2007?
17.37 How can the cost of SMF be minimized?
17.38 What are the three schemes by using which the connection behaviour of SMFs be improved? Explain these schemes, by taking examples.
17.39 State the design steps given in AISC 358-10 for the design of reduced beam sections.
17.40 What are pre-qualified seismic moment connections?
17.41 Name any five pre-qualified seismic moment connections, specified in AISC 358.
17.42 Write short notes on the following:
(a) Bolted unstiffened extended end plate (BUEEP) connections and bolted stiffened extended end plate (BSEEP) connections
(b) Bolted flange plate (BFP) moment connections
(c) Welded un-reinforced flange-welded web (WUF-W) connections
(d) Kaiser bolted bracket (KBB) moment connections
(e) SidePlate connections
(f) ConXtech's ConXL™ moment connections
(g) Braced frames
17.43 Discuss about the various types of bracings that can be used with Concentrically Braced Frames.
17.44 What are the advantages of using elliptical clearance over linear clearance in gusset plates of special concentrically braced frames (SCBFs)?
17.45 What are the problems faced by using SCBFs in recent earthquakes?
17.46 How do eccentrically braced frames (EBF) behave during an earthquake and what are their advantages over SCBFs?
17.47 What are dual systems? Explain the design features of these systems.
17.48 What is the minimum percentage of lateral load to be resisted by a moment-resistant frame in a dual system?
17.49 What are the three types of steel plate shear walls?
17.50 What are the advantages of SPSW over reinforced concrete shear walls? What are its disadvantages?
17.51 Why is the behaviour of SPSW different from that of a plate girder?
17.52 Explain the model that is used in AISC code for the analysis of SPSW.
17.53 State the method used for the preliminary proportioning of SPSW.
17.54 What necessitates the use of perforated webs in SPSWs?
17.55 Write short notes on
(a) Coupled SPSW
(b) Self-centering SPSW
17.56 What are buckling restrained braces? What are the advantages of BRBF over other concentrically braced frames?
17.57 Explain the concept of base isolation. How base-isolated buildings behave during an earthquake compared to fixed base buildings?
17.58 What are the three broad types of isolators?
17.59 Write short notes on (a) elastomeric bearing, (b) lead-rubber bearing, (c) sliding bearing, and (d) energy absorbing devices.
17.60 What are the different types of dampers used to control seismic damage?

REFERENCES

Adan, S.M., and Gibb, W., 'Experimental Evaluation of Kaiser-bolted Bracket Steel Moment-resisting Connections', *Engineering Journal, AISC*, Vol. 46, No.3, Third quarter 2009, pp. 181–195.

Ambrose, J. and Vergun, D., S*implified Building Design for Wind and Earthquake Forces,* Third edition, Wiley–Inter science, New York, NY, 1995, p. 368.

AISC Design Guide 12—*Modification of Existing Steel Welded Moment Frame Connections for Seismic Resistance* (Authors: Gross, J.L., Engelhardt, M.D., Uang, C.-M., Kasai, K., and Iwankiw, N.*R*.), American Institute of Steel Construction, Inc., Chicago, 1999.

AISC Design Guide 13—*Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, (Author: Carter, C.J.), American Institute of Steel Construction, Inc., Chicago, 2003.

AISC Design Guide 20- *Steel Plate Shear Walls*, (Authors: Sabelli *R*., and Bruneau, M.), American Institute of Steel Construction, Inc., Chicago, 2006.

AISC Design Guide 29: *Vertical Bracing Connections-Analysis and Design*, (Authors: W.A. Thornton and L.S. Muir), American Institute of Steel Construction, Inc., Chicago, 2014.

ANSI/AISC 358-10, *Pre-qualified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications including Supplement* Nos 1 and 2, American Institute of Steel Construction, Inc., Chicago, IL, 2010 and Draft AISC 358:2016.

ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, IL., 2010.

Arnold, C., 'Seismic Issues in Architectural Design', in FEMA 454 *Designing for Earthquakes- A Manual for Architects*, FEMA (Federal Emergency Management Agency, Washington, D.C.) and NEHRP (National Earthquake Hazards Reduction Program), Dec. 2006, p. 392.

Arnold, C. and Reitherman, R., *Building Configuration and Seismic Design*, John Wiley, 1982, p. 310.

Bachmann, H., *Seismic Conceptual Design of Buildings—Basic Principles for Engineers, Architects, Building Owners, and Authorities*, Swiss Federal Office for Water and Geology (FOWG), Biel, 2002, p. 81.

Becker, R., *Seismic Design of Special Concentrically Braced Frames*, Structural Steel Education Council, Morga, CA, 1995.

Becker, R. and Ishler, M., *Seismic Design Practice for Eccentrically Braced Frames*, Structural Steel education Council, Morga, CA, 1996.

Berman, J.W., 'Advances in Steel Plate Shear Walls, *STRUCTURE Magazine*, ASCE, Sep 2014, pp. 26–27.

Berman, J.W. and Bruneau, M., 'Steel Plate Shear Walls are not Plate Girders', *Engineering Journal*, AISC, Vol. 41, No. 3, 2004, pp. 95–106.

Bolt, B.A., *Earthquakes*, Fifth edition, W.H. Freeman (Macmillan Education), London, 2006, p. 320.

Borello, D. and Fahnestock, L., 'Seismic Design and Analysis of Steel Plate Shear Walls with Coupling', *Journal of Structural Engineering*, ASCE, Vol. 139, Special Issue: NEES 2: Advances in Earthquake Engineering, 2013, pp. 1263–1273.

Bozorgnia, Y. and Bertero, V.V., *Earthquake Engineering: From Engineering Seismology to Performance – Based Engineering*, CRC Press, Boca Raton, Florida, 2004, p. 1152.

Bozorgnia, Y., and Campbell, K.W., Chapter 5, Engineering Characterization of Ground Motion, In *Earthquake Engineering: From Engineering Seismology to Performance-based Engineering*, Bozorgnia and Bertero (eds), CRC Press, June 2004a, p. 74, http://www.ce.berkeley.edu/~mahin/CE227web/Bozorgni-CampbellCh_BerteroBozorgnia.pdf.

Bozorgnia, Y. and Campbell, K.W., 'The Vertical-to-horizontal Response Spectral Ratio and Tentative Procedures for Developing Simplified V/H and Vertical Design Spectra', *Journal of Earthquake Engineering*, Vol. 8, 2004b, pp. 175–207.

Bozorgnia, Y., Mahin, S.A., and Brady, A.G., 'Vertical Response of Twelve Structures Recorded During the Northridge Earthquake', *Earthquake Spectra*, Vol. 14, 1998, pp. 411–432.

Bruneau, M., Uang, C.M., and Whittaker, A., *Ductile Design of Steel Structures*, Second edition, McGraw Hill, New York, 2011, p. 921.

Caccese, V., Elgaaly, M., and Chen, R., Experimental Study of Thin Steel Plate Shear Walls under Cyclic Load, *Journal of Structural Engineering*, ASCE, Vol. 119, No. 2, Feb 1993, pp. 573-587, also Post-buckling of Steel Plate Shear Walls under Cyclic Loads, pp. 588–605.

Carter, C.J. and Grubb, K.A., 'Prequalified Moment Connections (revisited)', *Modern Steel Construction*, Vol. 50, No. 1, Jan 2010, pp. 54–57.

Cartwright, C., 'Wasatch Property Management Corporate Headquarters', *The Newsletter of the Structural Engineers Association of Utah*, Vol. XI, No. 1, Sep 2006, pp. 2–3.

Charney, F., Tola, A.T., and Atlayan, O., 'Structural Anaysis: Example 1, Twelve Storey Moment-resisting Steel Frame', 2009 *NEHRP Recommended Seismic Provisions*-FEMA P-752 CD, June 2013, www.fema.gov/media-library-data/1393888487069-c071850c3050c34da135376baa478a1b/P-752_Unit4.pdf, last accessed on 5th Aug 2015.

Chen, S.J., Yeh, C.H., and Chu, J.M., 'Ductile Steel Beam-to-Column Connections for Seismic Resistance', *Journal of Structural Engineering*, ASCE, Vol. 122, No. 11, Nov 1996, pp. 1292–1299.

Clayton, P. M., Berman, J. W., and Lowes, L. N., 'Seismic Design and Performance of Self-centering Steel Plate Shear Walls', *Journal of Structural Engineering*, ASCE, Vol. 138, No. 1, Jan 2012, pp. 22–30.

Cordova, P.P., and Hamburger, R.O., 'Steel Connections: Proprietary or Public Domain', *Modern Steel Construction*, AISC, Oct 2011, pp. 24–31.

Dana, M., Cussen, A., Chen Y.N., Davis, C., Greer, M., Houston, J., Littler, P., and Roufegarinejad, A., 'Effects of the Seismic Vertical Component on Structural Behavior - An Analytical Study of Current Code Practices and Potential Areas of Improvement', *Proceedings of the 10th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Anchorage, AK, 21–25 Jul 2014, p. 11, https://nees.org/resources/12050/download/10NCEE-001107.pdf, last accessed on 20 Jul 2015.

DeVall, R.H., 'Background Information for Some of the Proposed Earthquake Design Provisions for the 2005 Edition of the National Building Code of Canada', *Canadian Journal of Civil Engineering*, Vol. 30, 2003, pp. 279–286.

Dowden, D.M., Purba, *R*., and Bruneau, M., 'Behavior of Self-Centering Steel Plate Shear Walls and Design Consideration', *Journal of Structural Engineering*, ASCE, Vol. 138, No. 1, January 1, 2012, pp.11–21.

Driver, R.G. and Grondin, G.Y., 'Steel Plate Shear Walls, now Performing on the Main Stage', *Modern Steel Construction*, AISC, Vol. 41, No. 10, Sep 2001, pp. 48–58.

Driver, R.G., Kulak, G.L., Kennedy, D.J.L., and Elwi, A.E., Cyclic Test of a Four-storey Steel Plate Shear Wall, *Journal of Structural Engineering*, ASCE, Vol. 124, No. 2, Feb 1998, pp. 111–120.

Elnashai, A.S. and Mwafy, A.M., 'Overstrength and Force Reduction Factors of Multi-storey Reinforced-Concrete Buildings', *The*

Structural Design of Tall Buildings, Wiley Interscience, Vol. 11, 2002, pp. 329-351.

Engelhardt, M.D., Winneberger, T., Zekany, A.J., and Potyraj, T.J., 'Experimental Investigation of Dogbone Moment Connections', *Engineering Journal,* AISC, Vol. 35, No. 4, Fourth quarter 1998, pp. 128–139.

EN 1998-1:2004, *Eurocode 8—Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings*, European Committee for Standardization, Brussels, Dec 2003, p. 229.

FEMA 350, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, Federal Emergency Management Agency, Washington D.C., Jun 2000, p. 224.

FEMA-355C, *State-of-the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*, Federal Emergency Management Agency, Washington D.C., Sep 2000, p. 344.

FEMA 355D, *State-of-the art Report on Connection Performance*, prepared by the SAC Joint Venture for Federal Emergency Management Agency, Washington, D.C., 2000, http://www.sacsteel.org/, last accessed on 20 Jul 2015.

FEMA 451, *2003 NEHRP Recommended Provisions: Design Examples*, National Institute of Building Sciences, Washington, D.C., Aug 2006, p. 860.

FEMA 454, *Designing for Earthquakes—A Manual for Architects*, Federal Emergency Management Agency, Washington D.C., Dec 2006, p. 392.

FEMA P-750, *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, Building Seismic Safety Council, Washington D.C., 2009, p. 388.

Fisher, J.M., 'Expansion Joints: Where, When, and How', *Modern Steel Construction*, AISC, Vol. 45, No. 4, Apr 2005, pp. 29–34.

Gardiner, D.R., D.K. Bull, and A.J. Carr, 'Trends in Internal Forces in Concrete Floor Diaphragms of Multi-storey Structures during Seismic Shaking', *The 14th World Conference on Earthquake Engineering,* Beijing, China, 12–17 Oct 2008, http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0178.PDF, last accessed on 29 Jul 2015.

Gioncu, V. and F.M. Mazzolani, *Ductility of Seismic Resistant Steel Structures*, Spon Press, London & New York, 2002, p. 685.

Gross, J.L., Engelhardt, M.D., Uang, C.-M., Kasai, K., and Iwankiw, N.*R*., *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance,* STEEL Design Guide series 12, American Institute of Steel Construction, Inc., Chicago, IL, 1999.

Hamburger, R.O., Krawinkler, H., Malley, J.O., and Adan, S.M., *Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers,* NEHRP Seismic design technical brief no.2, National Institute of Standards and Technology, Gaithersburg, USA, June 2009, p. 33, www.nehrp.gov/pdf/nistgcr9-917-3.pdf, last accessed on 1 Aug 2015.

Haskett, T., B. Breukelman, J. Robinson, J. Kottelenberg., *Tuned Mass Dampers Under Excessive Structural Excitation*, Motioneering Inc., Ontario, Canada, 2004, p. 7, http://www.waterfordmgmt.com/school/Articles/tmd%20article.pdf, last accessed on 20 Jul 2015.

Humar, J.L. and Ragozar M.A., 'Concept of Over-strength in Seismic Design', In *Proceedings 11th World Congress on Earthquake Engineering*, International Association for Earthquake Engineering (IAEE), Acapulco, Mexico, Paper 639, 1996.

Hussain, S., Benschoten, P.V., Al Satari, M., and Lin, S., 'Buckling Restrained Braced Frame (BRBF) Structures: Analysis, Design and Approvals Issues', *Proceedings of the 75th SEAOC Annual Convention*, Long Beach, California, 13–16 Sep 2006, http://www.starseismic.net/wp-content/uploads/2013/08/brbfpaperfinal.pdf, last accessed on 17 Aug 2015.

International Network For The Design Of Earthquake Resilient Cities (INDERC): http://inderc.blogspot.com/2012/04/seismic-regulations-versus-modern.html

Iwankiw, N.R. and Carter, C.J., 'The Dogbone: A New Idea to Chew On', *Modern Steel Construction, AISC,* Vol. 36, No. 4, Apr 1996, pp. 18–23.

Ivankiw, N.R., 'Ultimate Strength Considerations for Seismic Design of the Reduced Beam Section (Internal Plastic Hinge)', *Engineering Journal*, AISC, Vol. 34, No. 1, 1997, pp. 3–16.

Jain, S.K., Explanatory Examples on Indian Seismic Code IS: 1893 (Part 1), www.iitk.ac.in/nicee/IITK-GSDMA/EQ21.pdf, last accessed on 30 Dec 2004.

Jain, S.K., 'A Proposed Draft of IS: 1893 Provisions on Seismic Design of Buildings – Part II Commentary and Examples', *Journal of Structural Engineering*, SERC, Vol. 22, No. 2, Jul 1995, pp. 73–90.

Jain, S.K. and Murty, C.V.R., *Proposed Draft Provisions and Commentary on Indian Code IS: 1893 (Part 1)*, www.iitk.ac.in/nicee/IITK-GSDMA/EQ05.pdf, last accessed on 10 Oct 2015.

Jangid, R.S. and Kelly, J. M., 'Base Isolation for Near-fault Motions', *Earthquake Engineering & Structural Dynamics,* Vol. 30, 2001, pp. 691–707.

Kalayanaraman, V., Sridhara, B.N., and Mahadevan, K., *Sleeved Column System, Proc. of the SSRC Task Group Meetings and Task Force Sessions*, Lehigh Univ., Bethlehem, Pa., 1994.

Kareem, A., Kijewski, T., and Tamura, Y., 'Mitigation of Motions of Tall Buildings with Specific Examples of Recent Applications', *Wind and Structures*, Vol. 2, No. 3, 1999, pp. 201–251.

Krawinkler, H., 1978, 'Shear in Beam-Column Joints in Seismic Design of Steel Frames', *Engineering Journal*, AISC, Vol. 5, No. 3, pp. 82–91.

Kelly, J.M., *Earthquake-resistant Design with Rubber*, Second edition, Springer-Verlag, London, 2012, p. 243.

Kircher, C.A., 'Chapter 12: Seismically Isolated Structures', in FEMA P-751 , *2009 NEHRP Recommended Seismic Provisions: Design Examples*, Federal Emergency Management Agency, Washington D.C., Sep 2012, p. 916.

Kulak, G.L., Kennedy, L., Driver, R.G., and Medhekar, M., 'Steel Plate Shear Walls – An Overview', *Engineering Journal*, AISC, Vol. 39, No. 1, 2001, pp. 50–62.

Lee, D., Cotton, S.C., Dexter, R.J., Hajjar, J.F., Ye, Y., and Ojard, S.D., *Column Stiffener Detailing and Panel Zone Behavior of Steel Moment Frame Connections*, Report, University of Minnesota, Jun 2002.

Lee, D., Cotton, S.C., Hajjar, J., Dexter, R.J. and Ye, Y. (2005a), 'Cyclic Behavior of Steel Moment-resisting Connections Reinforced by Alternative Column Stiffener Details I. Connection Performance and Continuity Plate Detailing,' *Engineering Journal*, AISC, Vol. 42, No. 4 (Fourth quarter), 2005, pp. 189–214, 'Part II. Panel Zone Behavior and Doubler Plate Detailing', Vol. 42, No. 4, 2005, pp. 215–238.

Li, J., Li, Y., Li, W., and Samali, B., 'Development of Adaptive Seismic Isolators for Ultimate Seismic Protection of Civil Structures', *Sensors and Smart Structures Technologies for Civil, Mechanical, and Aerospace Systems,* 2013, pp. 1–12.

López, W.A., 'On Designing with Buckling–Restrained Braced Frames', *STRUCTURE Magazine*, ASCE, Jul 2008, pp. 40–41.

López, W.A. and Sabelli, *R*., *Seismic Design of Buckling-restrained Restrained Braced Frames*, Steel Tips, Structural Steel Educational Council, Jul 2004, p. 66, http://www.steeltips.org/steeltips/tip_details.php?id=76, last accessed on 20 Jul 2015.

Mathews, M.S., Venugopal, M.S., and Subramanian, N., 'Planning and Design of Houses in Disaster Areas', *International Conference on Disaster Area Housing*, Turkey, Sep 1977.

Mayes, R.L., Brown, A.G., and Pietra, D., 'Using Seismic Isolation and Energy Dissipation to Create Earthquake-resilient Buildings', Paper Number 093, 2012 NZSEE Conference www.nzsee.org.nz/db/2012/Paper093.pdf, last assessed on 19 Aug 2015.

Moehle, J.P., J.D. Hooper, D.J. Kelly, T.R. Meyer, 'Seismic Design of Cast-in-place Concrete Diaphragms, Chords, and Collectors—A Guide for Practicing Engineers', *NEHRP Seismic Design Technical Brief No. 3*, National Institute of Standards and Technology, Gaithersburg, MD, Aug 2010, p. 29, http://www.nehrp.gov/pdf/nistgcr10-917-4.pdf, last accessed on 19 Sep 2015.

Murty, C.V.R., *IITK—BMTPC Earthquake Tips*, Dept of Civil Engineering, Indian Institute of Technology, Kanpur & Building Material and Technology Promotion Council, New Delhi, Sep 2005, p. 48, www.nicee.org/EQTips/IITK-BMTPC.htm, last accessed on 20 Jul 2015.

Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R., and Mehta, V.V., *Some Concepts in Earthquake Behaviour of Buildings*, National Information Center of Earthquake Engineering (NICEE), Indian Institute of Technology, Kanpur, 2013a, p. 262, http://www.iitk.ac.in/nicee/IITK-GSDMA/EBB_001_30May2013.pdf, last accessed on 20 Jul 2015.

Murty, C.V.R., Goswami, R., Vijayanarayanan, A.R., Pradeep Kumar, R., and Mehta, V.V., *Introduction to Earthquake Protection of Non-structural Elements in Buildings*, National Information Center of Earthquake Engineering (NICEE), Indian Institute of Technology, Kanpur, 2013b, p. 168, http://www.iitk.ac.in/nicee/IITK-GSDMA/NSE_002_31May2013.pdf, last accessed on 20 Jul 2015.

Naeim, F., and Kelly, J.M., *Design of Seismic Isolated Structures; From Theory to Practice*, John Wiley, New York, 1999, p. 304.

Nishi,T., Kelly, J.M., and Zhou, F.L., 'Rubber Structural Mounts Save Lives During Earthquakes', *ISO Focus*, Jun 2009, pp. 26–28.

Özmen, G., Girgin, K., and Durgun, Y., 'Torsional Irregularity in Multi-storey Structures', *International Journal on Advanced Structural Engineering*, Springer, Vol. 6, No. 4, Dec 2014, pp. 121–131.

Park R., 'Explicit Incorporation of Element and Structure Over-strength in the Design Process', *Proceedings 11th World Conference on Earthquake Engineering* (WCEE), Acapulco, Mexico, Paper 2130, 1996, http://www.iitk.ac.in/nicee/wcee/article/11_2130.pdf, last accessed on 20 Jul 2015.

Patil, S.J. and Reddy. G.R. 'State Of Art Review - Base Isolation Systems For Structures', *International Journal of Emerging Technology and Advanced Engineering*, Vol. 2, No. 7, Jul 2012, pp. 438–453.

Priestley, M.J.N., Calvi, G.M., and Kowalsky, M.J., *Displacement-based Seismic Design of Structures*, IUSS Press, Pavia, Italy, 2007, p. 721.

Roeder, C.W., and Lehman, D.E., 'Seismic Design and Behavior of Concentrically Braced Steel Frames', *Structure Magazine*, Feb 2008, pp. 37–39.

Sabelli, R., 'Recommended Provisions for Buckling-restrained Braced Frames', *AISC Engineering Journal*, Fourth quarter 2004, pp 155–175.

Sabelli, R., Roeder, C.W., and Hajjar, J.F., 'Seismic Design of Steel Special Concentrically Braced Frame Systems-A Guide for Practicing Engineers', *NEHRP Seismic Design Technical Brief*, No. 8, NIST GCR 13-917-24, National Institute of Standards and Technology, Gaithersburg, USA, Jul 2013, p. 36.

Sadashiva, V.K., MacRae, G.A., and Deam, B.L., 'Determination of Structural Irregularity Limits-Mass Irregularity Example', *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol. 42, No. 4, Dec. 2009, pp. 288–301.

Sato, A., Newell, J.D., Uang, C.-M., 'Cyclic Behavior and Seismic Design of Bolted Flange Plate Steel Moment Connections', *AISC Engineering Journal*, Vol. 45, Fourth quarter, 2008, pp. 221–233.

Saunders, M.C., 'Seismic Joints in Steel Frame Buildings', *Modern Steel Construction*, AISC, Vol. 45, No. 4, Apr 2005, pp. 57–62.

Seek, M.W. and Murray, T.M., *Cyclic Test of 8-Bolt Extended Stiffened Steel Moment End Plate Connection with Concrete Structural Slab*, Report submitted to the American Institute of Steel Construction, Virginia Polytechnic Institute and State University, Blacksburg, VA, 2005.

Seilie, I.F. and Hooper, J.D., 'Steel Plate Shear Walls - Practical Design and Construction, *Modern Steel Construction*, AISC, Vol. 45, No. 4, Apr 2005, pp. 37–42.

Sengupta, A.K. and A.D. Shetty, 'Analysis of Chord Forces in Floors with Large Openings or Re-entrant Corners in Reinforced Concrete Buildings', *ICI Journal*, Indian Concrete Institute, Jul–Sep, Vol. 12, No. 2, 2011, pp. 7–15.

Skinner, R.I., Robinson, W.H., and McVerry, G.H., *An Introduction to Seismic Isolation*, John Wiley & sons, New York, 1993, p. 376.

Song, G., Ma, N., and Li, H.-N., 'Applications of Shape Memory Alloys in Civil Structures', *Engineering Structures*, Vol. 28, 2006, pp. 1266–1274.

Soong, T.T. and Dargush, G.F., *Passive Energy Dissipation Systems in Structural Engineering,* John Wiley, New York, 1997, p. 368.

Spencer Jr, B.F. and Nagarajaiah, S., 'State-of-the-art of Structural Control', *Journal of Structural Engineering*, ASCE, Vol. 129, No. 7, Jul 2003, pp. 845–856.

Subramanian, N., 'How to Build Earthquake Proof Structures', *Civil Engineering and Construction Review*, Vol. 7, No. 4, Apr 1994, pp. 39–41.

Subramanian, N., *Design of Reinforced Concrete Structures*, Oxford University Press, New Delhi, 2013, p. 865.

Subramanian, N., 'Pre-qualified Seismic Moment Connections', *New Building Materials & Construction World (NBM & CW)*, Vol. 17, No. 2, Aug 2010, pp. 160–171.

Sumer, A., Fleischman, R.B., and Hoskisson, B.E., 'Development of a Cast Modular Connector for Seismic-resistant Steel Moment Frames, Part 1: Prototype development, Part 2: Experimental

Verification', *Engineering Journal, AISC*, Vol. 44, No.3, Third quarter, 2007, pp. 195–231.

Symans, M.D., Charney, F.A., Whittaker, A.S., Constantinou, M.C., Kircher, C.A., Johnson, M.W., and McNamara, R.J., 'Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments, *Journal of Structural Engineering*, ASCE, Vol. 134, No. 1, Jan 2008, pp. 3–21.

Tamboli, A.R., *Handbook of Structural Steel Connection Design and Details*, Second edition, McGraw Hill, New York, 2009, p. 656.

Taranath, B.S., *Steel, Concrete and Composite Design of Tall Buildings*, Second edition, McGraw Hill, New York, 1998, p. 998.

Thomas, P. and A.K. Sengupta, 'Analysis of chord forces in Reinforced Concrete Buildings Subjected to Seismic Loads', *ICI Journal*, Indian Concrete Institute, Vol. 9, No. 3, Oct–Dec 2008, pp. 21–27.

Thornton, W.A. and Muir, L.S., 'Design of Vertical Bracing Connections for High-seismic Drift', *Modern Steel Construction*, AISC, Vol. 49, No. 3, Mar 2009, pp. 61–65, http://msc.aisc.org/globalassets/modern-steel/archives/2009/03/2009v03_vertical_bracing.pdf, last accessed on 17 Aug 2015.

Timler, P.A. and Kulak, G.L., *Experimental Study of Steel Plate Shear Walls,* Structural Engineering Report No. 114, Department of Civil Engineering, The University of Alberta, Canada, Nov 1983.

Tsai, K.C., Li, C.H, and Lee, H.C., 'Seismic Design and Testing of the Bottom Vertical Boundary Elements in Steel Plate Shear Walls. Part 1: Design Methodology, Part 2: Experimental Studies', *Earthquake Engineering and Structural Dynamics*, Vol. 43, Nov 2014, pp. 2155–2177 and Dec 2014, pp. 2237–2259.

Tuan, A.Y. and G.Q. Shang, 'Vibration Control in a 101-Storey Building Using a Tuned Mass Damper', *Journal of Applied Science and Engineering*, Vol. 17, No. 2, 2014, pp. 141–56.

Uang, C.M., and Nakashima, M., 'Steel Buckling-restrained Frames', in *Earthquake Engineering: Recent Advances and Application*, Chapter 16, Y. Bozorgnia and V. V. Bertero, eds, CRC Press, 2003.

Uriz, P. and Mahin, S.A., *Towards Earthquake Resistant Design of Concentrically Braced Steel-frame Structures*, Report 2008/8, Pacific Earthquake Engineering Research Center(PEER),University of California, Berkeley, Nov 2008, p. 401.

Vargas, R. and Bruneau, M., 'Analytical Response and Design of Buildings with Metallic Structural Fuses- I', *Journal of Structural Engineering*, ASCE, Vol. 135, No. 4, Apr 2009, pp. 386–393, Part II - Experimental Response of Buildings Designed with Metallic Structural Fuses, pp. 394–403.

Vian D., Bruneau, M., and Purba, R., 'Special Perforated Steel Plate Shear Walls with Reduced Beam Section Anchor Beams II: Analysis and Design Recommendations', *Journal of Structural Engineering,* ASCE, Vol. 135, No. 3, Mar 2009, pp. 221–228.

Walters, M., 'Seismic Isolation-The Gold Standard of Seismic Protection', *STRUCTURE Magazine*, ASCE, Jul 2015, p. 11–14.

Williams, A., *Seismic Design of Building and Bridges*, Third edition, Engineering Press, California, 2004.

www.hpsdma.nic.in/Development%20of%20Probablistic%20Seismic%20Hazard%20Map%20of%20India.pdf

www.starseismic.net

www.steelcastconnections.com

Yang, T.S. and E. E. Popov, 'Comparison of MRF Connection Damage due to Northridge Quake with Theory', In *Behaviour of Steel Structures in Seismic Areas, STSSA 97*, (Eds Mazzolani, F. M. and H. Akiyama), Kyoto, 2–8 Aug 1997, 10/17, Salerno, pp. 897–904.

Yu, Q.S., Noel, S., and Uang, C.M., *Experimental and Analytical Studies on Seismic Rehabilitation of Pre-Northridge Steel Moment Connections*: *RBS and Haunch Approaches*, Structural Engg. Div., Report No. SSRP-97/08, University of California, San Diego, 1997.

Zhou, F. and Xian, Q., 'Recent Development on Seismic Isolation, Energy Dissipation, Passive and Semi-active Control of Structures in P.*R*. China', in *Earthquake Engineering Frontiers in the New Millennium*,. B.F. Spencer and Y.S. Hu, (Editors) A.A. Balkema, Lisse, Netherlands, 2001, pp. 279–285.