# *SECTION 1*

# *DESIGN CONCEPTS AND STRUCTURAL SCHEMES FOR MULTI-STOREY STEEL BUILDINGS*

Extracted from ***A Resource Book for Structural Steel Design & Construction*** - a resource material jointly published by the Building and Construction Authority (BCA) & the Singapore Structural Steel Society (SSSS).

SECTION 1

## 1.1 MODERN TECHNIQUES IN STEEL FRAME CONSTRUCTION

### 1.1.1 Buildability of Steelwork Construction

One of the main considerations in planning a building project is to have the building ready and occupied as early as possible. In order to reduce the time over which the investment is tied up in construction and maximize the return of investment through the use of the building, the design needs to consider the buildability aspects of the construction.

Speed in construction is achieved through a number of factors, some of which are listed below:

- Simple building design to avoid complicated site works
- Design for minimum delay in construction
- Maximize use of pre-fabricated and precast elements to avoid delays on site
- Reduce the number of operations on the critical path
- Complete all the designs before starting work on site

Complicated geometry and building layout design should be avoided where possible. This is especially critical in crowded city sites where access and storage of materials may be a problem. Repetition of work means that the work can be done in a much faster process. The more repetition in elements, the quicker the site team goes through the process of familiarization.

In steel framed building the positioning of services may need careful consideration at the design stage to allocate service zones. Hence, conflict of interests between various professions can be avoided.

Steel offers the best framing material for pre-fabrication. With the use of metal decks, the concept of Fast Track Construction is introduced. The metal decking can be placed easily and used as the slab reinforcement. Through-deck stud welding for composite action reduces beams weights and/or depths. It also helps ensure that the floor slab can be used as a diaphragm to transfer lateral loads to the bracing frames or stiff cores. Lightweight fire protection can be applied at a later stage, taking it off the critical path.

### 1.1.2 Prefabrication and Ease of Construction

Steel members and plates can be shop-fabricated using computer-controlled machinery, which have less chance of mistakes. On site the assembly is mainly carried out by a bolting procedure. Lateral load resisting system should be located at the lifts, stair towers etc to provide stability throughout, rather than a rigid unbraced frame where temporary bracing may be required during erection.

Structural members delivered to site should be lifted directly and fixed into position, avoiding storage on site. Steel stairs, erected along with the frame, give immediate access for quicker and safer erection.

Metal decking may be lifted in bundles and no further craning is required as it is laid by hand and fixed by welded studs. This gives both a working and safety platform against accidents.

Secondary beams should be placed close enough to suit the deck, so that temporary propping can be avoided, and the deck could be concreted immediately.

### 1.1.3 Steel-concrete Composite Design

Considerable benefit is gained by composition of the slab with the steel beam with possible weight savings of up to 30%. An effective width of a slab is assumed to carry the compressive stresses leaving virtually the whole of the steel beam in tension creating a T - beam effect. Interaction between the slab and the beam is generated by 'through deck" stud welding on to the beam flange.

### 1.1.4 Deflection and Cambering

Where the floors are unpropped, the deflection due to wet construction requires consideration to avoid the problem of ponding.

Dead load deflections exceeding 15-20mm can be easily offset by cambering which is best achieved by cold rolling the beam. This is a specialized operation but not only is the camber permanent because of stress re-distribution due to controlled yielding. These will depend upon a number of factors and the advice of the specialist should be sought.

Because cambering can add 10-20% to the basic steel cost, this should be compared with the cost of a deeper and stiffer beam section provided that the increase in building height does not compromise additional cladding costs.

### 1.1.5 Fire Resistance

In the event of fire the metal deck unit would cease to function effectively due to loss of strength. However, additional strength can be provided by the added wire mesh for up to one hour fire rating.

For higher period of fire resistance or for exceptionally high imposed loads, heavier reinforcement in the form of bars placed within the deck troughs can be used. Up to 4 hours fire rating can be obtained using this method based upon fire engineering calculations with the deck units serving only as a permanent formwork.

For beams and columns, fire resistance may be provided by lightweight systems, which are quick to apply and economic. Normally cement based sprays are applied to beams, and boards around columns. For tall buildings, steel columns may be encased or circular steel columns infilled with high-strength concrete to enhance resistance against compression and fire.

## 1.2 CLASSIFICATION OF MULTI-STOREY FRAMES

It is useful to define various frame systems to simplify the modelling of multistorey frames. For more complicated three-dimensional structures involving the interaction of different structural systems, simple models are useful for preliminary design and for checking computer results. These models should capture the behaviour of individual subframes and their effects on the overall structures.

This section describes what a framed system represents, defines when a framed system can be considered to be braced by another system, what is meant by a bracing system, and the difference between sway and non-sway frames. Various structural schemes for multistorey building construction are also given.

### 1.2.1 Moment Frames

A moment frame derives its lateral stiffness mainly from the bending rigidity of frame members inter-connected by rigid joints. The joints shall be designed in such a manner that they have enough strength and stiffness and negligible deformation. The deformation must be small enough to have any significant influence on the distribution of internal forces and moments in the structure or on the overall frame deformation.

An unbraced rigid frame should be capable of resisting lateral loads without relying on additional bracing system for stability. The frame, by itself, has to resist the design forces, including gravity as well as lateral forces. At the same time, it should have adequate lateral stiffness against side sway when it is subjected to horizontal wind or earthquake loads. Even though the detailing of the rigid connections results in a less economic structure, rigid unbraced frame systems perform better in load reversal situation or in earthquakes. From the architectural and functional points of view, it can be advantageous not to have any triangulated bracing systems or solid wall systems in the building.

### 1.2.2 Simple Frames

A simple frame referred to a structural system in which the beams and columns are pinned connected and the system is not capable of resisting any lateral loads. The stability of the entire structure must be provided by attaching the simple frame to some forms of bracing systems. The lateral loads are resisted by the bracing systems while the gravity loads are resisted by both the simple frame and the bracing system.

In most cases, the lateral load response of the bracing system is sufficiently small such that second-order effects may be neglected for the design of the frames. Thus the simple frames that are attached to the bracing system may be classified as non-sway frames. Figure 1.1 shows the principal components - simply frame and bracing system - of such a structure.

There are several reasons of adopting pinned connections in the design of steel multistorey frames:

1. pin-jointed frames are easier to fabricate and erect. For steel structures, it is more convenient to join the webs of the members without connecting the flanges.
2. bolted connections are preferred than welded connections, which normally require weld inspection, weather protection and surface preparation.
3. it is easier to design and analyse a building structure that can be separated into system resisting vertical loads and system resisting horizontal loads. For example, if all the girders are simply supported between the columns, sizing of the girders and columns is a straightforward task.
4. it is more cost effective to reduce the horizontal drift by means of bracing systems added to the simple framing than to use unbraced frame systems with rigid connections.

### 1.2.3 Bracing Frames

Bracing systems provide lateral stability to the overall framework. It may be in the forms of triangulated frames, shear wall/cores, or rigid-jointed frames. It is common to find bracing systems represented as shown in Figure1.2.

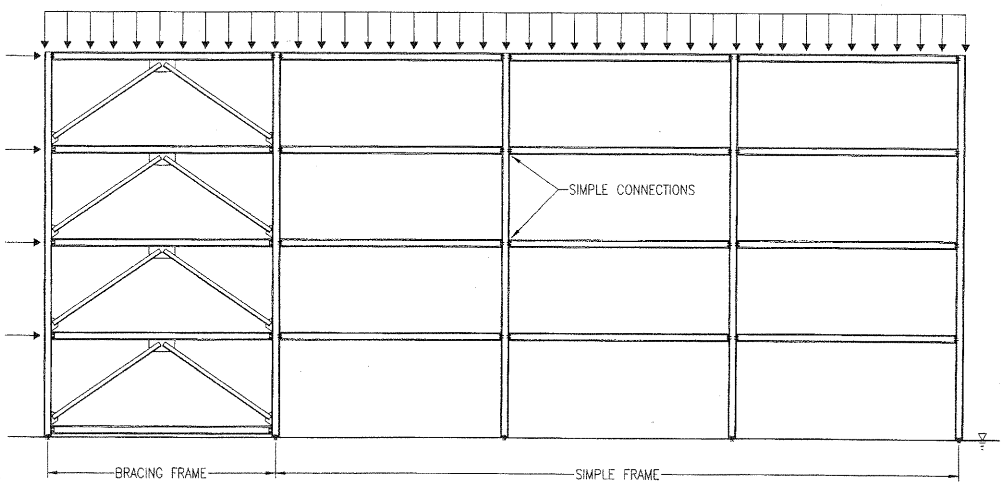
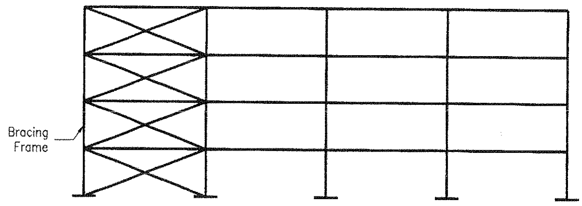


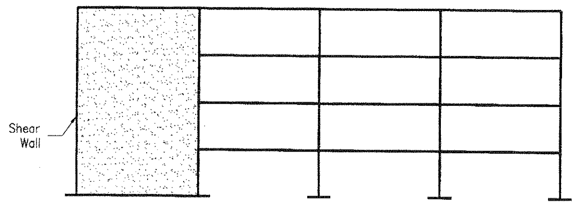

***Figure 1.1*** *Simple Braced Frame*

They are normally located in buildings to accommodate lift shafts and staircases.

In steel structures, it is common to have triangulated vertical truss to provide bracing (see Fig. 1.2a). Unlike concrete structures where all the joints are naturally continuous, the most direct way of making connections between steel members is to hinge one member to the other. For a very stiff structure, shear wall or core wall is often used (Figure 1.2b). The efficiency of a building to resist lateral forces depends on the location and the types of the bracing systems employed, and the presence or otherwise of shear walls and cores around lift shafts and stairwells.




(a)




(b)

***Figure 1.2*** *Commom bracing systems*
*(a) Vertical truss system*
*(b) Shear Wall*

### 1.2.4 Braced Versus Unbraced Frames

Building frame systems can be separated into vertical load-resistance and horizontal load-resistance systems. The main function of a bracing system is to resist lateral forces. In some cases the vertical load-resistance system also has some capability to resist horizontal forces. It is necessary, therefore, to identify the sources of resistance and to compare their behaviour with respect to the horizontal actions. However, this identification is not that obvious since the bracing is integrated within the structure. Some assumptions need to be made in order to define the two structures for the purpose of comparison.

Figures 1.3 and 1.4 represent the structures that are easy to define, within one system, two sub-assemblies identifying the bracing system and the system to be braced. For the structure shown in Figure 1.3, there is a clear separation of functions in which the gravity loads are resist by the hinged subassembly (Frame B) and the horizontal load loads are resisted by the braced assembly (Frame A). In contrast, for the structure in Fig. 1.4, since the second sub-assembly (Frame B) is able to resist horizontal actions as well as vertical actions, it is necessary to assume that practically all the horizontal actions are carried by the first sub-assembly (Frame A) in order to define this system as braced.

According to Eurocode 3 (EC3, 1992) a frame may be classified as braced if its sway resistance is supplied by a bracing system in which its response to lateral loads is sufficiently stiff for it to be acceptably accurate to assume all horizontal loads are resisted by the bracing system. The frame can be classified as braced if the bracing system reduces its horizontal displacement by at least 80 percent.

### 1.2.5 Sway Versus Non-Sway Frames

A frame can be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptable to neglect any additional internal forces or moments arising from horizontal displacements of the frame. In the design of multi-storey building frame, it is convenient to isolate the columns from the frame and treat the stability of columns and the stability of frames as independent problems. For a column in a braced frame it is assumed the columns are restricted at their ends from horizontal displacements and therefore are only subjected to end moments and axial loads as transferred from the frame. It is then assumed that the frame, possibly by means of a bracing system, satisfies global stability checks and that the global stability of the frame does not affect the column behaviour. This gives the commonly assumed *non-sway frame*. The design of columns in non-sway frame follows the conventional beam-column capacity check approach, and the column effective length may be evaluated based on the column end restraint conditions.

Another reason for defining "sway" and "non-sway frames" is the need to adopt conventional analysis in which all the internal forces are computed on the basis of the undeformed geometry of the structure. This assumption is valid if second-order effects are negligible. When there is an interaction between overall frame stability and column stability, it is not possible to isolate the column. The column and the frame have to act interactively in a "sway" mode. The design of sway frames has to consider the frame sub-assemblage or the structure as a whole.

British Code: BS5950: Part 1(1990) provides a procedure to distinguish between sway and non-sway frames as follows:

1) Apply a set of notional horizontal loads to the frame. These notional forces are to be taken as 0.5% of the factored dead plus imposed loads and are applied in isolation, i.e., without the simultaneous application of actual vertical or horizontal loading.
2) Perform a first-order linear elastic analysis and evaluate the individual relative sway deflection d for each storey.
3) If the actual frame is uncladed, the frame may be considered to be non-sway if the inter-storey deflection satisfies the following limit:

$$\delta \leqslant \frac{h}{4000} \quad \text{where h = storey height} \qquad (1.1)$$

for every storey.
4) If the actual frame is claded but the analysis is carried out on the bare frame, then in recognition of the fact that the cladding will substantially reduce

deflections, the condition is reflected and the frame may be considered to be non-sway if

$$\delta \leqslant \frac{h}{2000} \quad \text{where h = storey height} \quad (1.2)$$

for every storey.

5) All frames not complying with the criteria in Eqs. (1.1) or (1.2) are considered to be sway frames.

Eurocode 3 (1992) also provides some guidelines to distinguish between sway and non-sway frames. It states that a frame may be classified as non-sway for a given load case if $P_{cr} / P \geq 10$ for that load case, where $P_{cr}$ is the elastic critical buckling value for sway buckling and P is the design value of the total vertical load. When the system buckling load factor is ten times more than the design load factor, the frame is said to be stiff enough to resist lateral load, and it is unlikely to be sensitive to side sway deflections.

### 1.2.6 Classification of Multi-storey Buildings

The selection of appropriate structural systems for tall buildings must satisfy both the strength and stiffness requirements. The structural system must be adequate to resist lateral and gravity loads that cause horizontal shear deformation and overturning deformation. Other important issues that must be considered in planning the structural schemes and layout are the requirements for architectural details, building services, vertical transportation, and fire safety, among others. The efficiency of a structural system is measured in term of their ability to resist higher lateral load which increases with the height of the frame. A building can be considered as tall when the effect of lateral loads is reflected in the design. Lateral deflections of tall building should be limited to prevent damage to both structural and non-structural elements. The accelerations at the top of the building during frequent windstorms should be kept within acceptable limits to minimise discomfort to the occupants.

Figure 1.5 shows a chart which defines, in general, the limits to which a particular framing system can be used efficiently for multi-storey building projects. The various structural systems in Fig. 2.5 can be broadly classified into two main types: (1) medium-height buildings with shear-type deformation predominant and (2) high-rise cantilever structures such as framed tubes, diagonal tubes and braced trusses. This classification of system forms is based primarily on their relative effectiveness in resisting lateral loads.

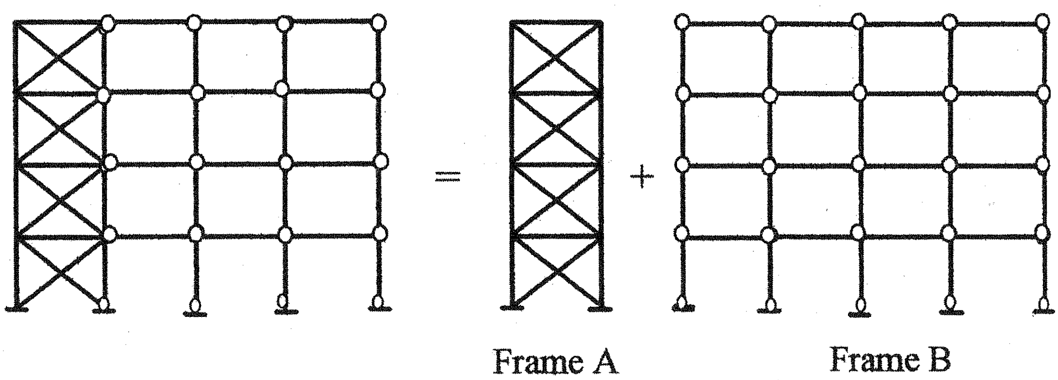



***Figure 1.3*** *Frames split into two subassemblies*

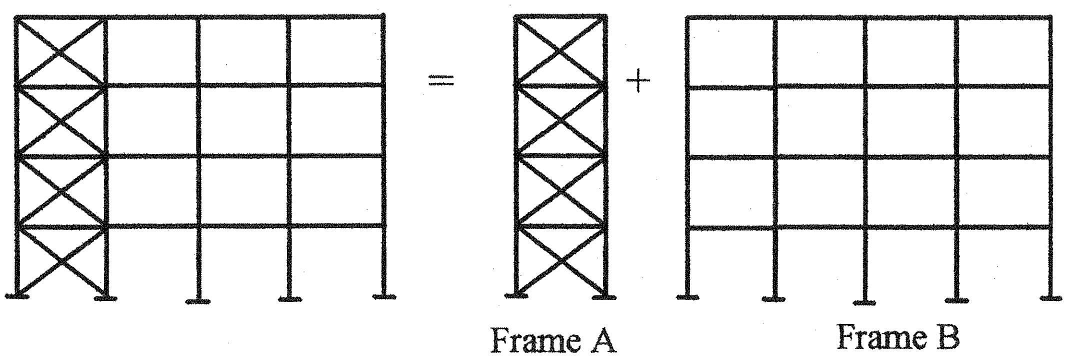



***Figure 1.4*** *Mixed frames split into two subassemblies*

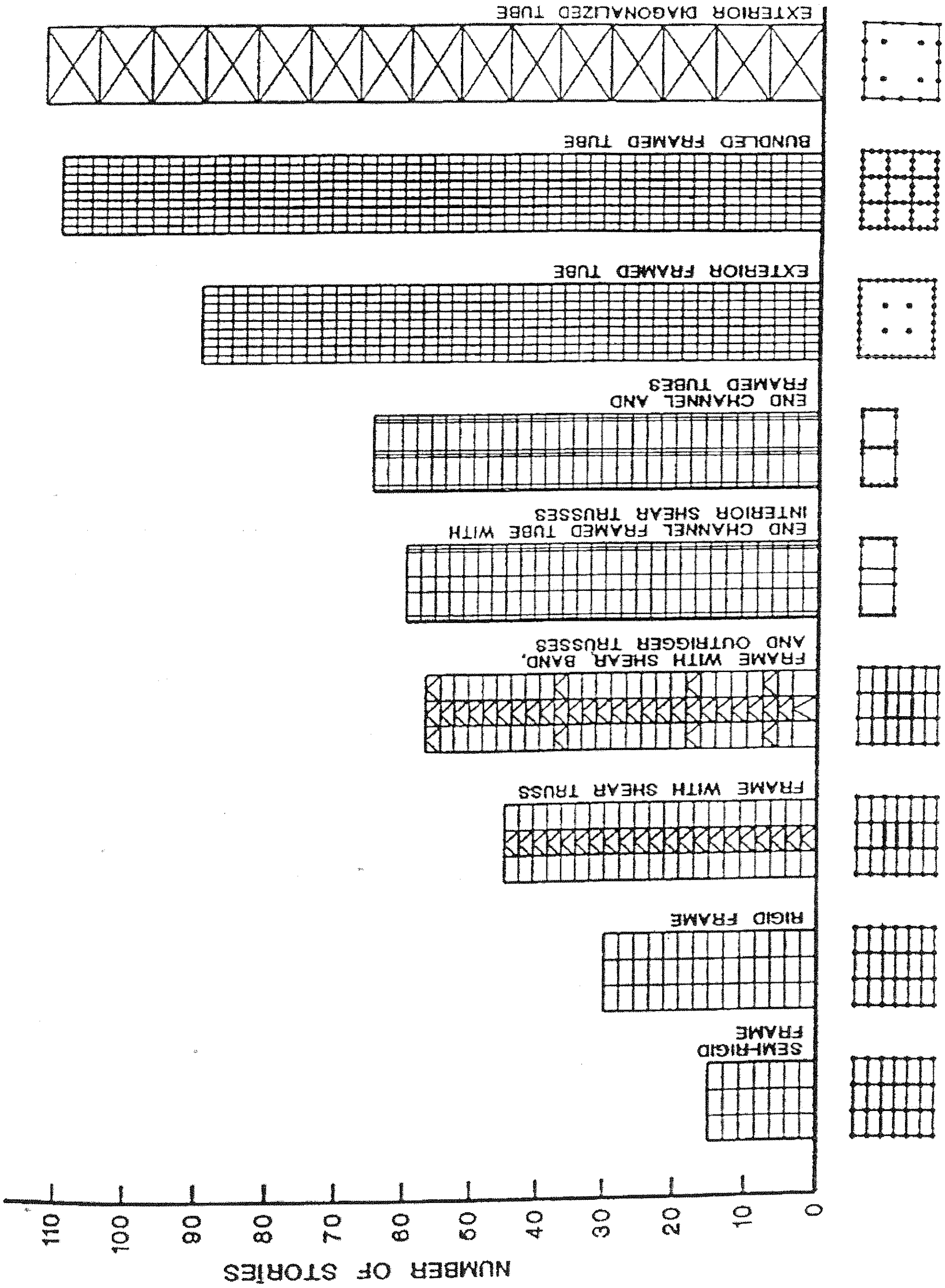



*Figure 1.5* *Categorization of Tall Building Systems.*

## 1.3 FLOOR SYSTEMS

### 1.3.1 Design Consideration

Floor systems in tall buildings generally do not differ substantially from those in low-rise buildings. However, the following aspects need to be considered in design:

1. Weight to be minimised.
2. Self-supporting during construction.
3. Mechanical services to be integrated in the floor zone.
4. Adequate fire resistance.
5. Buildability.
6. Long spanning capability.
7. Adequate floor diaphragm

Modern office buildings require large floor span in order to create greater space flexibility for the accommodation of greater variety of tenant floor plans. For building design, it is necessary to reduce the weight of the floors so as to reduce the size of columns and foundations and thus permit the use of larger space. Floors are required to resist vertical loads and they are usually supported by secondary beams. The spacing of the supporting beams must be compatible with the resistance of the floor slabs.

The floor systems can be made buildable using prefabricated or precast elements of steel and reinforced concrete in various combinations. Floor slabs can be precast concrete slab or composite slabs with metal decking. Typical precast slabs are 4 m to 7 m, thus avoiding the need of secondary beams. For composite slabs, metal deck spans ranging from 2 m to 7 m may be used depending on the depth and shape of the deck profile. However, the permissible spans for steel decking are influenced by the method of construction, in particular it depends on whether temporary propping is provided. Propping is best avoided as the speed of construction is otherwise diminished for the construction of tall buildings.

Floor beans systems must have adequate stiffness to avoid large deflections which could lead to damage of plaster and slab finishers. Where the deflection limit is too severe, pre-cambering with an appropriate initial deformation equal and opposite to that due to the permanent loads can be employed to offset part of the deflection.

Sometimes openings in the webs of beams are required to permit passage of horizontal services, such as pipes (for water and gas), cables (for electricity, tele- and electronic-communication), and ducts (air-conditioning), etc. Various long span flooring systems in Sections 3.4 offer solutions to integrate building service into the structural depth leading to potential saving in weight and cladding cost.

### 1.3.2 Composite Floor Systems

Composite floor systems typically involve structural steel beams, joists, girders, or trusses linked via shear connectors with a concrete floor slab to form an effective T-beam flexural member resisting primarily gravity loads. The versatility of the system results from the inherent strength of the concrete floor component in compression and the tensile strength of the steel member. The main advantages of combining the use of steel and concrete materials for building construction are:

- Steel and concrete may be arranged to produce ideal combination of strength, with concrete efficient in compression and steel in tension.
- Composite flooring is lighter in weight than pure concrete slab.
- The construction time is reduced since casting of additional floors may proceed without having to wait for the previously cast floors to gain strength. The steel decking system provides positive-moment reinforcement for the composite floor and requires only small amount of reinforcement to control cracking and for fire resistance.
- The construction of composite floor does not require highly skilled labour. The steel decking acts as a permanent formwork. Composite beams and slabs can accommodate raceways for electrification, communication, and air distribution system. The slab serves as a ceiling surface to provide easy attachment of a suspended ceiling.
- The composite floor system produces a rigid horizontal diaphragm, providing stability to the overall building system while distributing wind and seismic shears to the lateral load resisting systems.
- Concrete provides thermal protection to steel at elevated temperature. Composite slabs of two hours fire rating can be easily achieved for most building requirements.

The floor slab may be constructed by the following methods:

- a flat-soffit reinforced concrete slab (Fig. 1.6a),
- precast concrete planks with cast in-situ concrete topping (Fig. 1.6b),
- precast concrete slab with in-situ grouting at the joints (Fig. 1.6c), and
- a metal steel deck tops with concrete, either composite or non-composite (Fig. 1.6d).

The composite action of the metal deck results from side embossments incorporated into the steel sheet profile.

### 1.3.3 Composite Beams and Girders

Steel and concrete composite beams may be formed by shear connectors connecting the concrete floor to the top flange of the steel member. Concrete encasement will provide fire resistance to the steel member. Alternatively, direct sprayed-on cementitious and board type fireproofing materials may be used economically to replace the concrete insulation on the steel members. The most common arrangement found in composite floor systems is a rolled or built-up steel beam connected to a formed steel deck and concrete slab (Fig. 1.6d). The metal deck typically spans unsupported between steel members while also providing a working platform for concreting work.

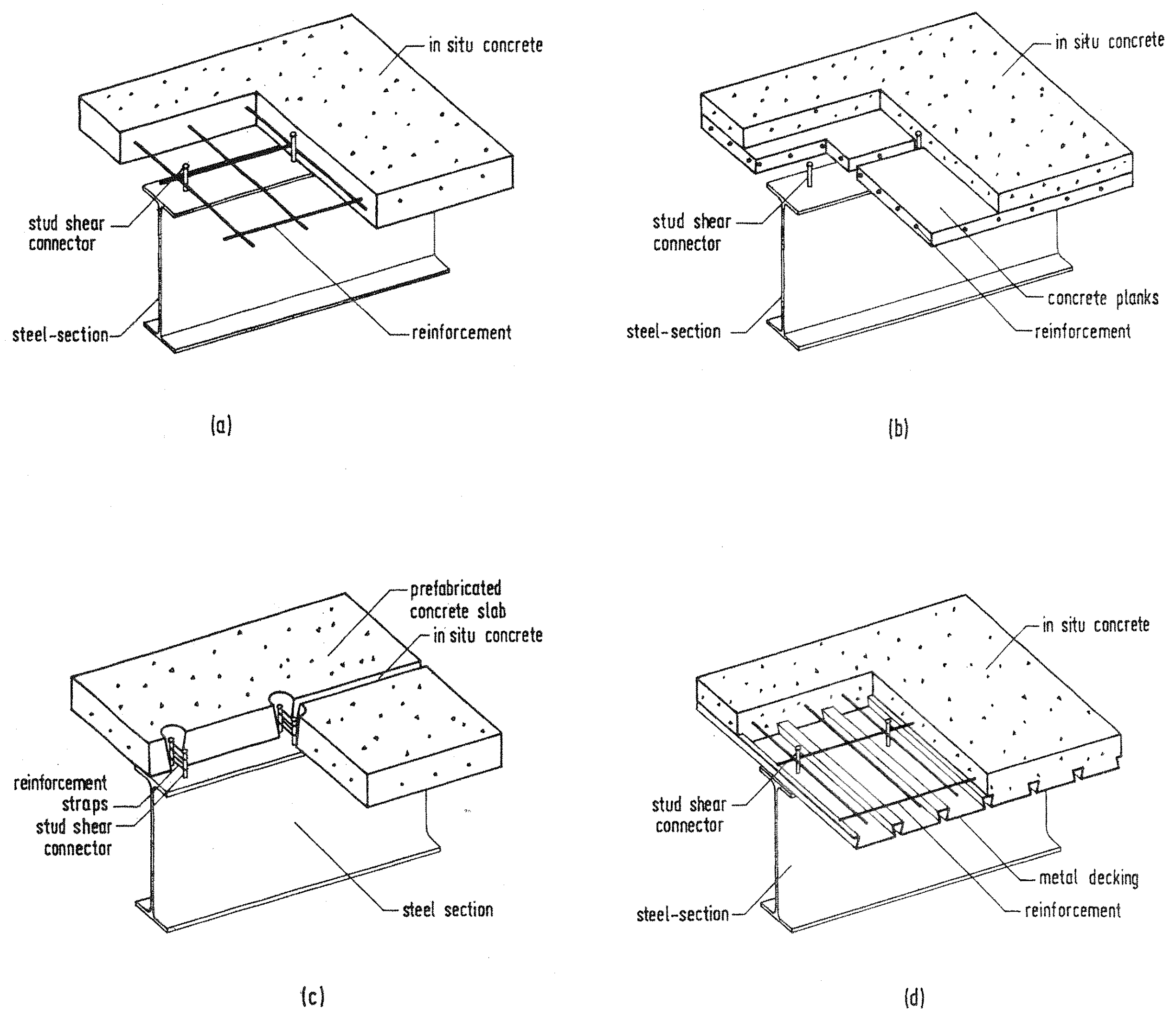
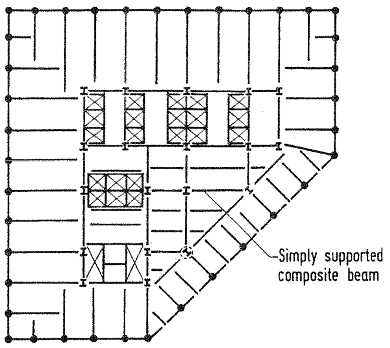
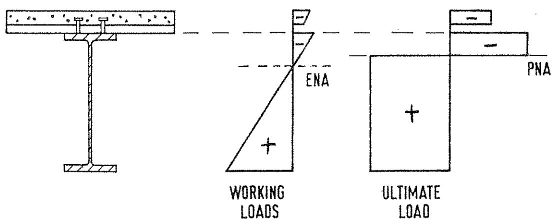



***Figure 1.6*** *Composite beams with*
*(a) flat-soffit reinforced concrete slab*
*(b) precast concrete planks and cast-in-situ concrete topping*
*(c) precasted concrete slab and in-situ concrete at the joints*
*(d) metal steel deck supporting concrete slab*

Figure 1.7a shows a typical building floor plan using composite steel beams. The stress distribution at working loads in a composite section is shown schematically in Fig. 1.7b. The neutral axis is normally located very near to the top flange of the steel section. Therefore, the top flange is lightly stressed. For construction point of view, a relatively wide and thick top flange must be provided for proper installation of shear stud and metal decking. However, the increased fabrication costs must be evaluated, which tend to offset the saving from material efficiency.




(a)




(b)

***Figure 1.7.*** *(a) Composite floor plan*
*(b) Stress distribution in a composite cross section*

A number of composite girder forms allow passage of mechanical ducts and related services through the depth of the girder (Fig. 1.8). Successful composite beam design requires the consideration of various serviceability issues such as long-term (creep) deflections and floor vibrations. Of particular concern is the occupant-induced floor vibrations. The relatively high flexural stiffness of most composite floor framing systems results in relatively low vibration amplitudes and therefore is effective in reducing perceptibility. Studies have shown that short to medium span (6 m to 12 m) composite floor beams perform quite well and are rarely found to transmit annoying vibrations to the occupants. Particular care is required for long span beam more than 12 m.

### 1.3.4 Long-Span Flooring Systems

Long spans impose a burden on the beam design in terms of larger required flexural stiffness for serviceability design. Besides satisfying serviceability and ultimate strength limit states, the proposed system must also accommodate the incorporation of mechanical services within normal floor zones. Several practical options for long-span construction are available and they are discussed in the following subsections.

#### 1.3.4.1 Beams With Web Openings

The One Raffles Link project (see Section 3) utilises the castellated beam with circular web openings for accommodating building services. Standard castellated beams can be fabricated from hot-rolled beams by cutting along a zigzag line through the web. The top and bottom half-beams are then displaced to form castellations (Fig. 1.9). Castellated composite beams can be used effectively for lightly serviced building. Although composite action does not increase the strength significantly, it increases the stiffness, and hence reduces deflection and the problem associated with vibration. Castellated beams have limited shear capacity and best used as long span secondary beams where loads are low or where concentrated loads can be avoided. Its use may be limited due to the increased fabrication cost and the fact that the standard castellated openings are not large enough to accommodate the large mechanical ductwork common in modern high-rise buildings.

Horizontal stiffeners may be required to strengthen the web opening, and they are welded above and below the opening. The height of the opening should not be more than 70% of the beam depth, and the length should not be more than twice the beam depth. The best location of the openings is in the low shear zone of the beams. This is because the webs do not contribute much to the moment resistance of the beam.

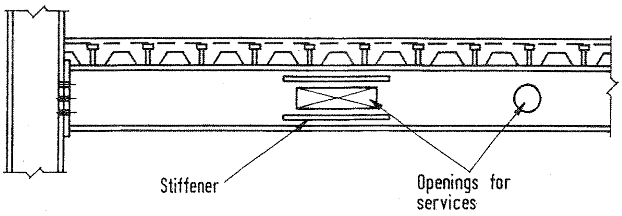



***Figure 1.8*** *Web opening with horizontal reinforcements*

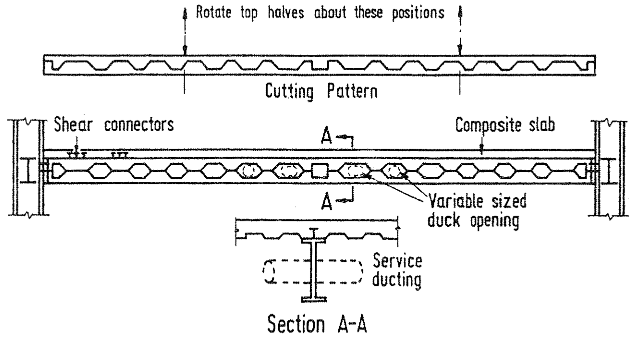



***Figure 1.9*** *Composite castellated beams*

#### 1.3.4.2 Fabricated Tapered Beams

The economic advantage of fabricated beams is that they can be designed to provide the required moment and shear resistance along the beam span in accordance with the loading pattern along the beam. Several forms of tapered beams are possible. A simply supported beam design with a maximum bending moment at the mid-span would require that they all effectively taper to a minimum at both ends (Fig. 1.10). Whereas a rigidly connected beam would have minimum depth towards the mid-span. To make best use of this system, services should be placed towards the smaller depth of the beam cross sections. The spaces created by the tapered web can be used for running services of modestly size (Fig. 1.10).

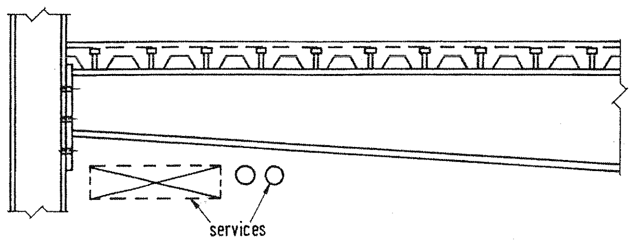



***Figure 1.10*** *Tapered composite beam*

A hybrid girder can be formed with the top flange made of lower-strength steel in comparison with the steel grade for the bottom flange. The web plate can be welded to the flanges by double-sided fillet welds. Web stiffeners may be required at the change of section when taper slope exceeds approximately six degrees. Stiffeners are also required to enhance the shear resistance of the web especially when the web slenderness ratio is too high. Tapered beam is found to be economical for spans up to 20m.

#### 1.3.4.3 Haunched Beams

Haunched beams are designed by forming a rigid moment connection between the beams and columns. The haunch connections offer restraints to beam and it helps to reduce mid-span moment and deflection. The beams are designed in a manner similar to continuous beams. Considerable economy can be gained in sizing the beams using continuous design which may lead to a reduction in beam depth up to 30% and deflection up to 50%.

The haunch may be designed to develop the required moment which is larger than the plastic moment resistance of the beam. In this case, critical section is shifted to the tip of the haunch. The depth of the haunch is selected based on the required moment at the beam-to-column connections. Length of haunch is typically 5%-7% the span length for non-sway frames or 7%-15% for sway frames. Service ducts can pass below the beams (Fig. 1.11).

Haunched composite beams are usually used in the case where the beams frame directly into the major axis of the columns. This means that the columns must be designed

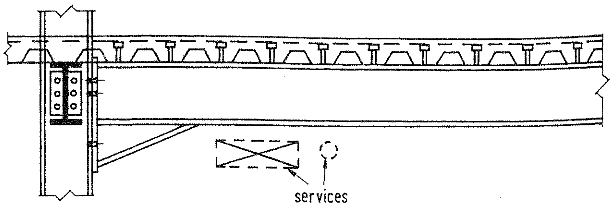



***Figure 1.11*** *Hunched composite beam*

to resist the moment transferred from the beam to the column. Thus a heavier column and a more complex connection would be required in comparison with structure designed based on the assumption that the connections are pinned. The rigid frame action derived from the haunched connections can resist lateral loads due to wind without the need of vertical bracing. Haunched beams offer higher strength and stiffness during the steel erection stage thus making this type of system particular attractive for long span construction. However, haunched connections behave differently under positive and negative moments, as the connection configuration is not symmetrical about the bending axis.

#### 1.3.4.4 Parallel Beam System

The system consists of two main beams with secondary beams run over the top of the main beams (see Fig. 1.12). The main beams are connected to either side of the column. They can be made continuous over two or more spans supporting on stubs attached to the columns. This will help in reducing the construction depth, and thus avoiding the usual beam-to-column connections. The secondary beams are designed to act compositely with the slab and may also be made to span continuously over the main beams. The need to cut the secondary beams at every junction is thus avoided. The parallel beam system is ideally suited for accommodating large service ducts in orthogonal directions (Fig. 1.12). Small saving in steel weight is expected from the continuous construction because the primary beams are non-composite. However, the main beam can be made composite with the slab by welding beam stubs to the top flange of the main beam and connected to the concrete slab through the use of shear studs (see the stud-girder system in Section 3.4.6). The

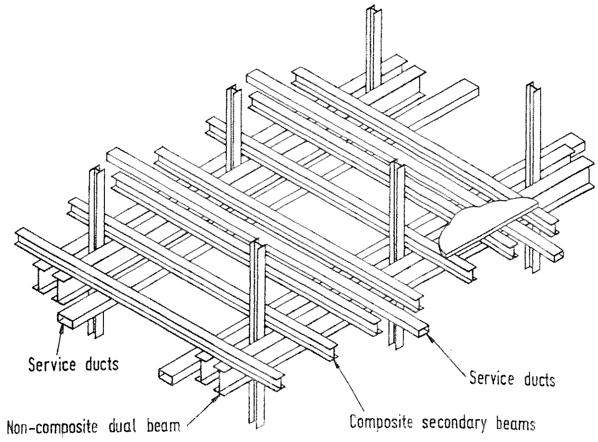



*Figure 1.12 Parallel composite beam system*

simplicity of connections and ease of fabrication make this long-span beam option particularly attractive.

### 1.3.4.5 Composite Trusses

Composite truss systems have been used in the OUB Centre and Suntec City projects (see Sections 3 and 4). The openings created in the truss braces can be used to accommodate large services. Although the cost of fabrication is higher in relation to the material cost, truss construction can be cost-effective for very long span when compared to other structural schemes. One disadvantage of the truss configuration is that fire protection is labour intensive and sprayed-protection systems cause a substantial mess to the services that pass through the web opening (see Fig. 1.13).

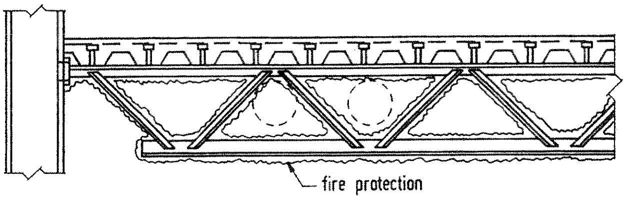



***Figure 1.13*** *Composite truss*

Several forms of truss arrangement are possible. The three most common web framing configurations in floor truss and joist designs are: (a) Warren Truss, (b) Modified Warren Truss and (c) Pratt Truss as shown in Fig. 1.14. The efficiency of various web members in resisting vertical shear forces may be affected by the choice of a web-framing configuration. For example, the selection of Pratt web over Warren web may effectively shorten compression diagonals resulting in more efficient use of these members.

The resistance of a composite truss is governed by (1) yielding of the bottom chord (2) crushing of the concrete slab, (3) failure of the shear connectors, (4) buckling of top chord during construction, (5) buckling of web members, and (6) instability occurring during and after construction. To avoid brittle failures, ductile yielding of

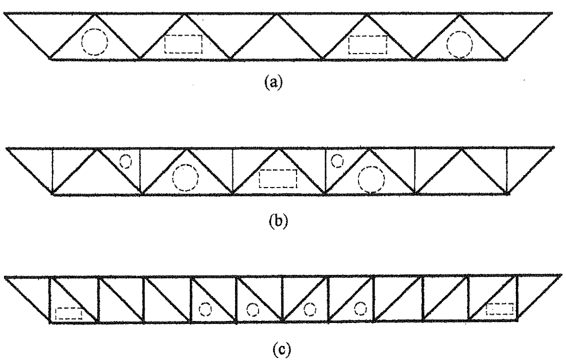



***Figure 1.14*** *Truss configuration:*
*(a) Warren truss,*
*(b) Modified Warren truss, and*
*(c) Pratt truss*

the bottom chord is the preferred failure mechanism. Thus the bottom chord should be designed to yield prior to crushing of concrete slab. The shear connectors should have sufficient capacity to transfer the horizontal shear between the top chord and the slab. During construction, adequate plan bracing should be provided to prevent top chord buckling. When consider composite action, the top steel chord is assumed not to participate in the moment resistance of the truss, since it is located very near to the neutral axis of the composite truss and, thus, contributed very little to the flexural capacity.

### 1.3.4.6 Stub Girder System

The stub girder system involves the use of short beam stubs which are welded to the top flange of a continuous, heavier bottom girder member, and connected to the concrete slab through the use of shear studs. Continuous transverse secondary beams and ducts can pass through the openings formed by the beam stub. The natural openings in the stub girder system allow the integration of structural and service zones in two directions (Fig. 1.15), permitting storey-height reduction when compared with some other structural framing systems.

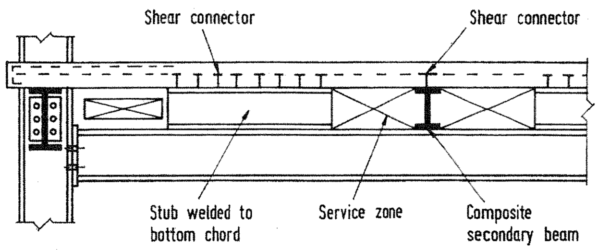



***Figure 1.15*** *Stub girder system*

Ideally, stub-girders span about 12 to 15 meters in contrast to the conventional floor beams which span about 6 to 9 meters. The system is therefore very versatile, particularly with respect to secondary framing spans with beam depths being adjusted to the required structural configuration and mechanical requirements. Overall girder depths vary only slightly, by varying the beam and stub depths. The major disadvantage of the stub girder system is that it requires temporary props at the construction stage, and these props have to be remained until the concrete has gained adequate strength for composite action. However, it is possible to introduce additional steel top chord, such as a T-section, which acts in compression to develop the required bending strength during construction. For span length greater than 15 meters, stub-girders become impractical, because the slab design becoming critical.

In the stub girder system, the floor beams are continuous over the main girders and splice at the locations near the points of inflection. The sagging moment regions of the floor beams are usually designed compositely with the deck-slab system, to produce savings in structural steel as well as to provide stiffness. The floor beams are bolted to the top flange of the steel bottom chord of the stub-girder, and two shear studs are usually specified on each floor beam, over the beam-girder connection, for anchorage to the deck-slab system. The stub-girder may be analysed as a vierendeel girder, with the deck-slab acting as a compression top-chord, the full-length steel girder as a tensile bottom-chord, and the steel stubs as vertical web members or shear panels.

#### 1.3.4.7 Prestressed Composite Beams

Prestressing of the steel girders is carried out such that the concrete slab remains uncracked under the working loads and the steel is utilised fully in terms of stress in the tension zone of the girder.

Prestressing of steel beam can be carried out using a precambering technique as depicted in Fig. 1.16. First a steel girder member is prebent (Fig. 1.16a), and is then subjected to preloading in the direction against the bending curvature until the required steel strength is reached (Fig. 1.16b). Secondly, the lower flange of the steel member, which is under tension, is encased in a reinforced concrete chord (Fig. 1.16c). The composite action between the steel beam and the concrete slab is developed by providing adequate shear connectors at the interface. When the concrete gains adequate strength, the steel girder is prestressed by stress-relieving the precompressed tension chord (Fig. 1.16d). Further composite action can be achieved by supplementing the girder with in-situ or prefabricated reinforcement concrete slabs, and this will produce a double composite girder (Fig. 1.16e).

The main advantage of this system is that the steel girders are encased in concrete on all sides, no corrosion and fire protection are required on the sections. The entire process of precambering and prestressing can be performed and automated in a factory. During construction, the lower concrete chord cast in the works can act as formwork. If

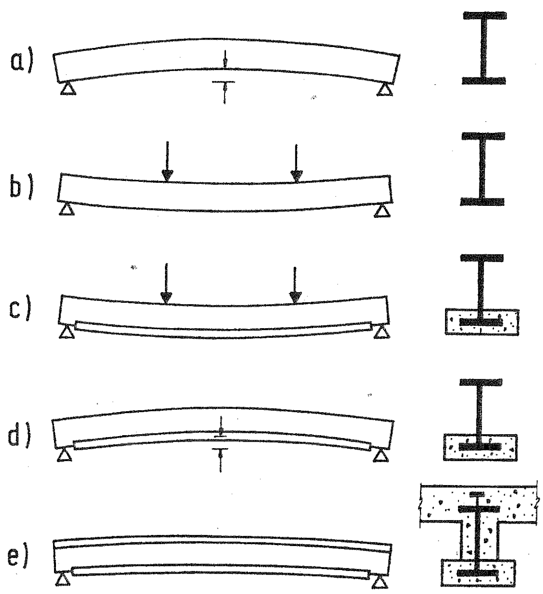



*Figure 1.16* *Process of prestressing using precambering technique*

the distance between two girders is large, precast planks can be supported by the lower concrete chord which is used as permanent formwork.

Prestressing can also be achieved by using tendons which can be attached to the bottom chord of a steel composite truss or the lower flange of a composite girder to enhance the load-carrying capacity and stiffness of long-span structures (Fig. 1.17). This technique has been found to be popular for bridge construction in Europe and USA, although less common for building construction.

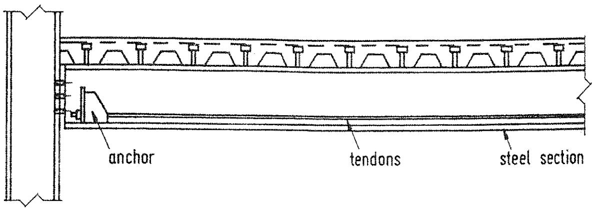



*Figure 1.17* *Prestressing of composite steel girders with tendons*

### 1.3.5 Comparison of Floor Spanning Systems

The conventional composite beams are the most common forms of floor construction for a large number of building projects. Typically they are highly efficient and economic with bay sizes in the range of 6 to 12 m. There is, however, much demand for larger column free areas where, with a traditional composite approach, the beams tend to become excessively deep, thus unnecessary increases the overall building height, with the consequent increases in cladding costs, etc. Spans exceeding 12 m are generally achieved by choosing an appropriate structural form which integrates the services within the floor structure, thereby reducing the overall floor zone depths. Although a long span solution may entail a small increase in structural costs, the advantages of greater flexibility and adaptability in service and the creation of column-free space often represent the most economic option over the design life of the building. Figure 1.18 compares the various structural options of typical range of span lengths used in practice.

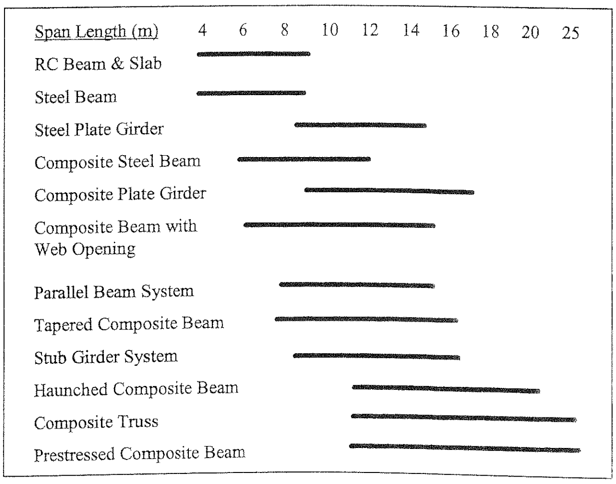



*Figure 1.18* *Comparison of flooring systems*

### 1.3.6 Floor Diaphragms

Typically, beams and columns rigidly connected for moment resistance are placed in orthogonal directions to resist lateral loads. Each plane frame would assume to resist a portion of the overall wind shear which is determined from the individual frame stiffness in proportion to the overall stiffness of all frames in that direction. This is based on the assumption that the lateral loads are distributed to the various frames by the floor diaphragm. In order to develop proper diaphragm action, the floor slab must be attached to all columns and beams that participate in lateral-force resistance. For building relying on bracing systems to resist all lateral load, the stability of a building depends on rigid floor diaphragm to transfer wind shears from their point of application to the bracing systems such as lattice frames, shear walls, or core walls.

The use of composite floor diaphragms in place of in-plane steel bracing has become an accepted practice. The connection between slab and beams are often through shear studs which are welded directly through metal deck to the beam flange. The connection between seams of adjacent deck panels is crucial and often through inter-locking of panels overlapping each other. The diaphragm stresses are generally low and can be resisted by floor slabs which have adequate thickness for most buildings.

Plan bracing is necessary when diaphragm action is not adequate. Figure 1.19a shows a triangulated plan bracing system which resists lateral load on one side and spans between the vertical walls. Fig. 1.19b illustrated the case where the floor slab has adequate thickness and it can act as diaphragm resisting lateral loads and transmitting the forces to the vertical walls. However, if there is an abrupt change in lateral stiffness or where the shear must be transferred from one frame to the other due to the termination of lateral bracing system at certain height, large diaphragm stresses may be encountered and they must be accounted for through proper detailing of slab reinforcement. Also, diaphragm stresses may be high where there are large openings in the floor, in particular at the corners of the openings.

The rigid diaphragm assumption is generally valid for most high-rise buildings (Fig. 1.20a); however, as the plan aspect ratio (b/a) of the diaphragm linking two lateral systems exceeds 3 in 1 (see the illustration in Fig. 1.20b), the diaphragm may become semi-rigid or flexible. For such cases, the wind shears must be allocated to the parallel shear frames according to the attributed area rather than relative stiffness of the frames.

From the analysis point of view, a diaphragm is analogous to a deep beam with the slab forming the web and the peripheral members serving as the flanges as shown in Fig. 1.20b. It is stressed principally in shear, but tension and compression forces must be accounted for in design.

A rigid diaphragm is useful to transmit torsional forces to the lateral-load resistance systems to maintain lateral stability. Figure 1.21a shows a building frame consisting of three shear walls resisting lateral forces acting in the direction of Wall A. The lateral load is assumed to act as a concentrated load with a magnitude F on each storey. Figure 1.21b and 1.21c show the building plan having dimensions of $L_1$ and $L_2$. The lateral load resisting system are represented in plan by the solid lines which represent Wall A, Wall B and Wall C. Since there is only one lateral resistance system (Wall A) in the direction of the applied load, the loading condition creates a torsion (Fe), and the diaphragm tends to rotate as shown by the dashed lines in Fig. 1.21b. The lateral load resistance systems in Wall B and Wall C will provide the resistance forces to stabilise the torsional force by generating a couple of shear resistance.

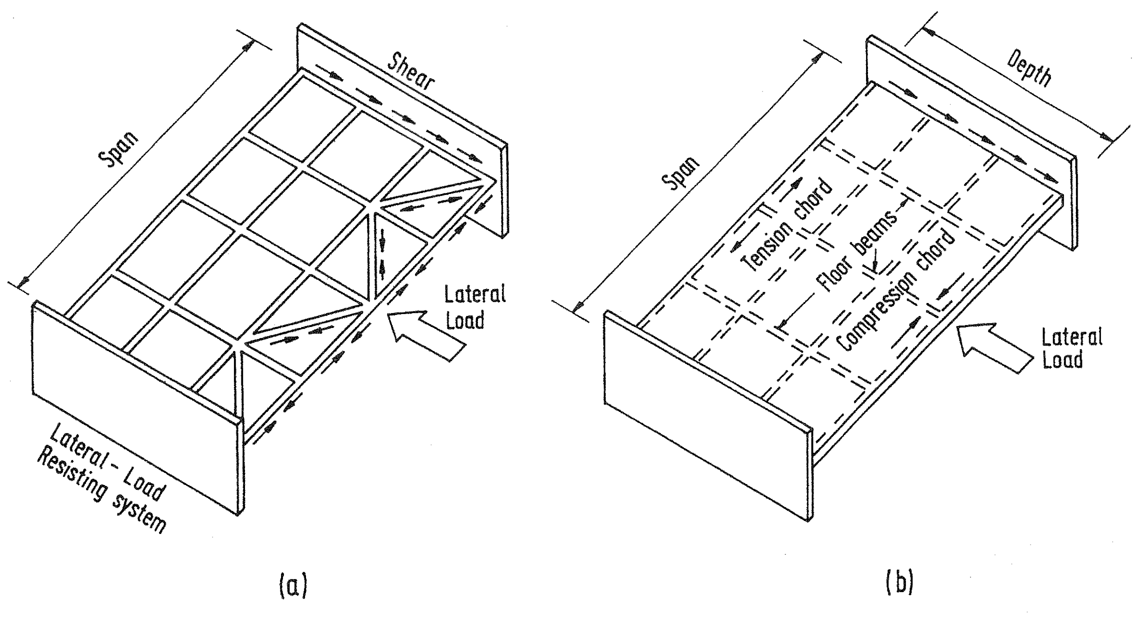



*Figure 1.19* (a) *Triangulated plan bracing system*
(b) *concrete floor diaphragm*

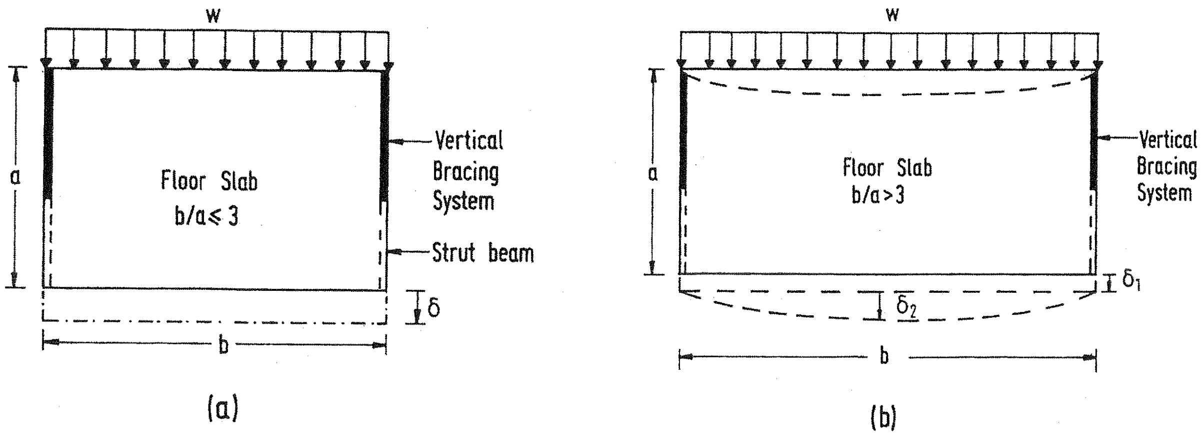



*Figure 1.20* Diaphragm rigidity
*(a) Aspect ratio ≤ 3*
*(b) Aspect ratio > 3*

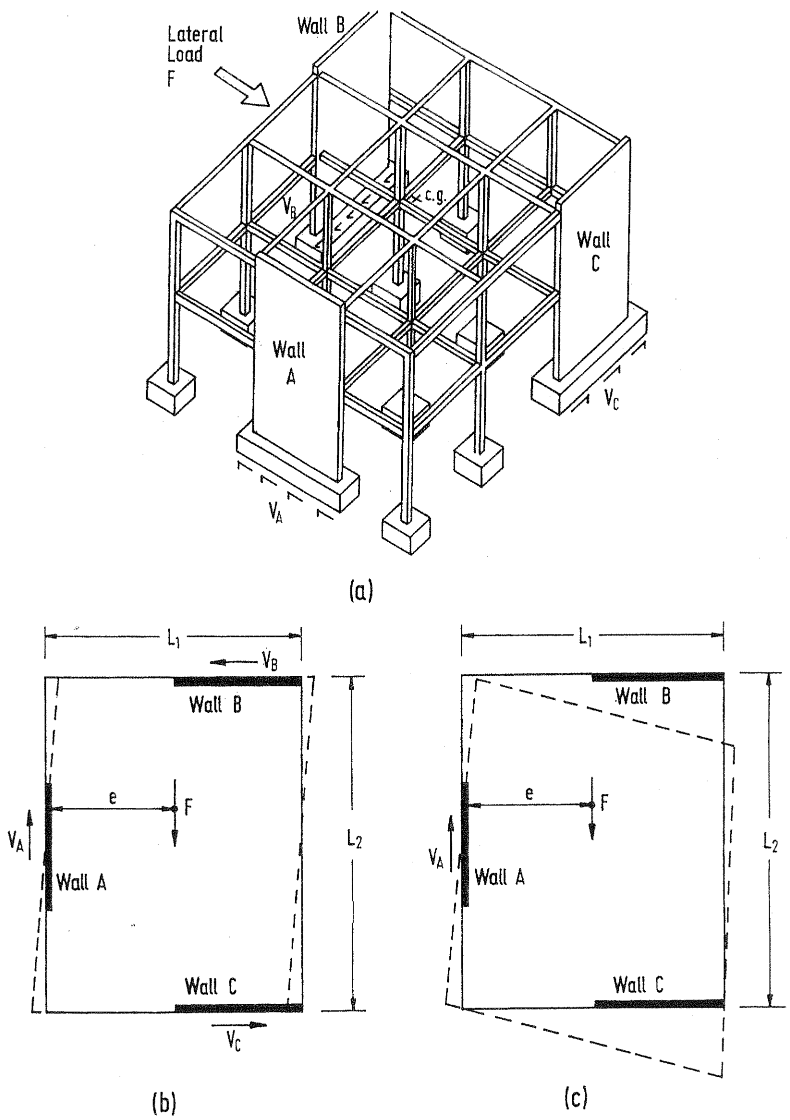



***Figure 1.21*** *(a) Lateral force resisting systems in a building*
*(a) Rigid diaphragm*
*(b) Flexible diaphragm*

Figure 1.21c illustrates the same condition except that a flexible diaphragm is used. The same torsional tendency exists, but the flexible diaphragm is unable to generate a resisting couple in Wall B and Wall C, and the structure will collapse as shown by the dashed lines. To maintain stability, a minimum of two vertical bracings in the direction of the applied force is required to resist any torional effects.

The adequacy of the floor to act as a diaphragm depends very much on its type. Precast concrete floor planks without any prestressing offer limited resistance to the racking effects of diaphragm action. In such cases, supplementary bracing systems in plan, such as those shown in Fig. 1.19a, are required for resistance of lateral forces. Where precast concrete floor units are employed, sufficient diaphragm action can be achieved by using a reinforced structural concrete topping, so that all individual floor planks are combined to form a single floor diaphragm. Composite concrete floors, incorporating permanent metal decking, provide excellent diaphragm action provided that the connections between the diaphragm and the peripheral members are adequate. When composite beams or girders are used, shear connectors will usually serve as boundary connectors and intermediate diaphragm-to-beam connectors. By fixing the metal decking to the floor beams, an adequate floor diaphragm can be achieved during the construction stage.

It is essential at the start of the design of structural steelworks, to consider the details of the flooring system to be used, since these have a significant effect on the design of the structure. Table 1.1 summarises the salient features of the various types of flooring systems in terms of their diaphragm actions.

**Table 1.1** Details of typical flooring systems and their relative merits

| Floor System | Typical span length (m) | Typical depth (mm) | Construction Time | Degree of lateral restraint to beans | Degree of diaphragm action | Usage |
|---|---|---|---|---|---|---|
| In-situ concrete | 3-6 | 150-250 | medium | very good | very good | all categories but not often used in multistorey buildings |
| Steel deck with in-situ concrete | 2.5-3.6 unshore<br>> 3.6 shore | 110-150 | fast | very good | very good | all categories especially in multistorey office buildings |
| Precast concrete | 3 - 6 | 110-200 | fast | fair-good | fair-good | all categories with cranage requirements |
| Pre-stressed concrete | 6 - 9 | 110-200 | medium | fair-good | fair-good | multistorey buildings & bridges |

## 1.4 DESIGN CONCEPTS AND STRUCTURAL SCHEMES

### 1.4.1 Introduction

Multistorey steel frames consist of column and beam interconnected to form a three-dimensional structure. A building frame can be stabilized either by some forms of bracing systems (braced frames) or by itself (unbraced frames). All building frames must be designed to resist lateral load to ensure overall stability. A common approach is to provide a gravity framing system with one or more lateral bracing systems attached to it. This type of framing system, which is generally referred to as simple braced frames, is found to be cost effective for multistorey buildings of moderate height (up to 20 storeys).

For gravity frames, the beams and columns are pinned connected and the frames are not capable of resisting any lateral loads. The stability of the entire structure is provided by attaching the gravity frames to some forms of bracing systems. The lateral loads are resisted mainly by the bracing systems while the gravity loads are resisted by both the gravity frame and the bracing system. For buildings of moderate height, the bracing system's response to lateral forces is sufficiently stiff such that second-order effects may be neglected for the design of such frames.

In moment resisting frames, the beams and columns are rigidly connected to provide moment resistance at joints, which may be used to resist lateral forces in the absence of any bracing systems. However, moment joints are rather costly to fabricate. In addition, it takes longer time to erect a moment frame than a gravity frame.

A cost effective framing system for multistorey buildings can be achieved by minimizing the number of moment joints, replacing field welding by field bolting, and combining various framing schemes with appropriate bracing systems to minimize frame drift. A multistorey structure is most economical and efficient is when it can transmit the applied loads to the foundation by the shortest and most direct routes. For ease of construction, the structural schemes should be simple enough, which implies repetition of member and joints, adoption of standard structural details, straightforward temporary works, and minimal requirements for inter-related erection procedure to achieve the intended behavior of the completed structure. Sizing of structural members should be based on the longest spans and largest attributed roof and/or floor areas. The same sections should be used for similar but less onerous cases.

Scheme drawings for multistorey building design should include the followings:

1. general arrangement of the structure including, column and beam layout, bracing frames, and floor systems,
2. critical and typical member sizes,
3. typical cladding and bracing details,
4. typical and unusual connection details, and
5. proposals for fire and corrosion protection.

This section offers advice on the general principles to be applied when preparing a structural scheme for multistorey steel and composite frames. The aim is to establish several structural schemes that are practicable, sensibly economic, and functional to the changes that are likely to be encountered as the overall design develops. The section begins by examining the design procedure and construction considerations that are specific to steel gravity frames, braced frames and moment resisting frames, and the design approaches to be adopted for sizing multi-storey building frames. The potential use of steel-concrete composite material for high-rise construction is then presented. Finally, the design issues related to braced and unbraced composite frames are discussed.

### 1.4.2 GRAVITY FRAMES

Gravity frames refer to structures that are designed to resist only gravity loads. The bases for designing gravity frames are as follows:

1) The beam and girder connections transfer only vertical shear reactions without developing bending moment that will adversely affect the members and the structure as a whole.
2) The beams may be designed as simply supported member.
3) Columns must be fully continuous. The columns are designed to carry axial loads only. Some codes of practice (e.g., BS5950, 1990) require the column to carry nominal moments due to the reaction force at the beam end, applied at an appropriate eccentricity.
4) Lateral forces are resisted entirely by bracing frames or by shear walls, lift or staircase closures, through floor diaphragm action.

#### 1.4.2.1 General Guides

The following points should be observed in the design of gravity frames:

1) provide lateral stability to gravity framing by arranging suitable braced bays or core walls deployed symmetrically in orthogonal directions, or wherever possible, to resist lateral forces.
2) adopt a simple arrangement of slabs, beams and columns so that loads can be transmitted to the foundations by the shortest and most direct load paths.
3) tie all the columns effectively in orthogonal directions at every storey. This may be achieved by the provision of beams or ties that are placed as close as practicable to the columns.
4) select a flooring scheme that provides adequate lateral restraint to the beams and adequate diaphragm action to transfer the lateral load to the bracing system.
5) for tall building construction, choose a profiled-steel-decking composite floor construction if uninterrupted floor space is required and/or height is at a premium. As a guide, limit the span of the floor slab to 2.5-3.6m; the span of the secondary beams to 6-12m; and the span of the primary beams to 5-7m. Otherwise, choose a precast or

an in-situ reinforced concrete floor, limiting their span to 5-6m, and the span of the beams to 6-8m approximately

### 1.4.2.2 Structural Layout

In building construction, greater economy can be achieved through a repetition of similarly fabricated components. A regular column grid is less expensive than a non-regular grid for a given floor area. In addition, greater economies can be achieved when the column grids in plan are rectangular in which the secondary beams should span in the longer direction and the primary beams in the shorter as shown in Figs. 1.22a&b. This arrangement reduces the number of beam-to-beam connections and the number of individual members per unit area of supported floor.

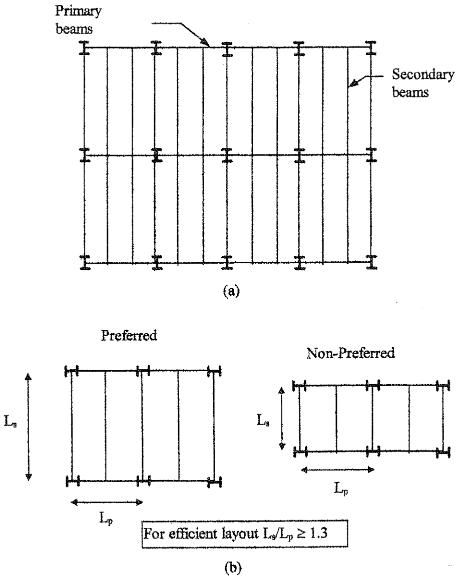



***Figure 1.22*** *(a) Rectanglular grid layout*
*(b) Preferred and non-preferred grid grid layout*

In gravity frames, the beams are assumed to be simply supported between columns. The effective beam span to depth ratio (L/D) is about 12 to 15 for steel beams and 18 to 22 for simply supported composite beams. The design of beam is often dependent on the applied load, the type of beam system employed and the restrictions on structural floor depth. The floor-to-floor height in a multistorey building is influenced by the restrictions on overall building height and the requirements for services above and/or below the floor slab. Naturally, flooring systems involving the use of structural steel members that act compositely with the concrete slab achieve the longest spans.

### 1.4.2.3 Analysis and Design

The analysis and design of a simple braced frame must recognize the following points:

1) The members intersecting at a joint are pin connected.
2) The columns are not subject to any direct moment transferred through the connection, but nominal moments due to eccentricity of the beam reaction forces should be considered. The design axial force in the column is predominately governed by floor loading and the tributary areas.
3) The structure is statically determinate. The internal forces and moments are therefore determined from a consideration of statics.
4) Gravity frames must be attached to a bracing system so as to provide lateral stability to the part of the structure resisting gravity load. The frame can be designed as a non-sway frame and the second-order moments associated with frame drift can be ignored.
5) The leaning column effects due to column side-sway must be considered in the design of the frames that are participated in side sway resistance.

Since the beams are designed as simply supported between their supports, the bending moments and shear forces are independent of beam size. Therefore, initial sizing of beams is a straight-forward task.

Most conventional types of floor slab construction will provide adequate lateral restraint to the compression flange of the beam. Consequently, the beams may be designed as laterally restrained beams without the moment resistance being reduced by lateral-torsional buckling.

Under the service loading, the total central deflection of the beam or the deflection of the beam due to unfactored live load (with proper precambering for dead load) should satisfy the deflection limits as given in Table 1.2.

In some occasions, it may be necessary to check the dynamic sensitivity of the beams. When assessing the deflection and dynamic sensitivity of secondary beams, the deflection of the supporting beams must also be included. Whether it is the strength, deflection or dynamic sensitivity which controls the design will depend on the span-to-depth ratio of the beam. Figure 1.18 gives typical span ranges for beams in office buildings for which the design would be optimized for strength and serviceability. For beams with their span lengths exceeding those shown in Fig. 1.18, serviceability limits due to deflection and vibration will most likely be the governing criteria for design.

The required axial forces in the columns can be derived from the cumulative reaction forces from those beams that frame into the columns. Live load reduction should be considered in the design of columns in a multistorey frame. If the frame is braced against side sway, the column node points are prevent from lateral translation. A conservative estimate of column effective length, KL, for buckling considerations is 1.0L, where L is the storey height. However, in cases where the columns above and below the storey under consideration are underutilized in terms of load resistance, the restraining effects offered by these members may result in an effective length of less than 1.0L for the column under consideration. Such a situation arises where the column is continuous through the restraint points and the columns above and/or below the restraint points are of different length.

**Table 1.2** Recommended deflection limits for steel building frames.

| | |
|---|---|
| **Beam deflections from unfactored imposed loads** | |
| Beams carrying plaster or brittle finish | span/360 |
| Other beams | span/240 |
| Edge beams supporting cladding | span/500 |
| **Beam deflection due to unfactored dead and imposed loads** | |
| Internal beams with ceilings | span/200 |
| Edge beams supporting cladding | span/350 |
| **Columns deflections from unfactored imposed and wind loads** | |
| Column in single storey frames | height/300 |
| Column in multistorey frames | height of storey/300 |
| Column supporting cladding which is sensitive to large movement | height of storey/500 |
| **Frame drift under 50 years wind load** | |
| Frame drift | Frame height/450 ~ Frame height/600 |

### 1.4.2.4 Simple Connections

Simple connections are designed to resist vertical shear at the beam end. Depending on the connection details adopted, it may also be necessary to consider an additional bending moment resulting from the eccentricity of the bolt line from the supporting face. Often the fabricator is told to design connections based on the beam end reaction for one-half uniformed distributed load (UDL). Unless the concentrated load is located very near to the beam end, UDL reactions are generally conservative. Because of the large reaction, the connection becomes very strong which may require large number of bolts. Thus it would be a good practice to design the connections for the actual forces used in the design of the beam. The engineer should give the design shear force for every beam to the steel fabricator so that a more realistic connection can be designed, instead of requiring all connections to develop the shear capacity of the beam. Figure 1.23 shows the typical connections that can be designed as simple connections. When the beam reaction is known, capacity tables developed for simple standard connections can be used for detailing such connections.

## 1.4.3 BRACING SYSTEMS

Bracing frames provide the lateral stability to the entire structure. It has to design to resist all possible kinds of lateral loading due to external forces, e.g. wind forces, earthquake forces and "leaning forces" from the gravity frames. The wind or the equivalent earthquake forces on the structure, whichever are greater, should be assessed and divided into the number of bracing bays resisting the lateral forces in each direction.

### 1.4.3.1 Structural Forms

Steel braced systems are often in a form of vertical truss which behaves like cantilever elements under lateral loads developing tension and compression in the column chords. Shear forces are resisted by the bracing members. The truss diagonalization may take various forms, as shown in Figure 1.24. The design of such structures must take into account the manner in which the frames are erected, the distribution of lateral forces and their side sway resistance.

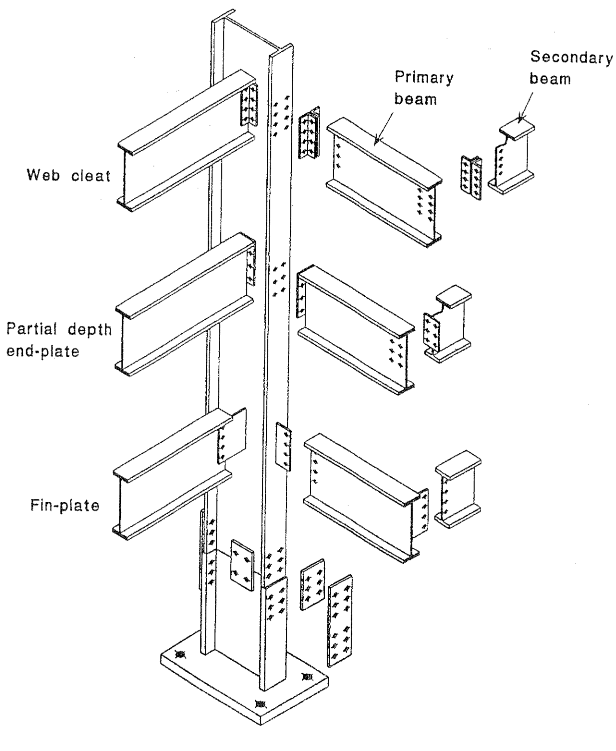



*Figure 1.23 Simple Beam-to-Column Connection*

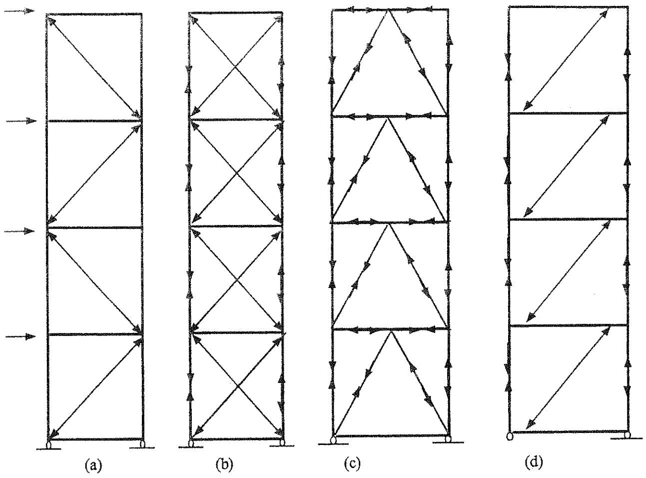



*Figure 1.24* *(a) Diagonal bracing*
*(b) Cross-bracing*
*(c) K-bracing*
*(d) Eccentric bracing*

In the single braced forms, where a single diagonal brace is used (Fig. 1.24a), it must be capable of resisting both tensile and compressive axial forces caused by the alternate wind load. Hollow sections may be used for the diagonal braces as they are stronger in compression. In the design of diagonal braces, gravity forces may tend to dominate the axial forces in the members and due consideration must be given in the design of such members. It is recommended that the slenderness ratio of the bracing member must not be greater than 200 to prevent the self-weight deflection of the brace limiting its compressive resistance.

In a cross-braced system (Fig. 1.24b), the brace members are usually designed to resist tension only. Consequently, light sections such as structural angles and channels, or tie rods can be used to provide a very stiff bracing. The advantage of the cross braced system is that the beams are not subjected to significant axial force, as the lateral forces are mostly taken up by the bracing members.

For K trusses, the diagonals do not participate extensively in carrying column load, and can thus be designed for wind axial forces without gravity axial force being considered as a major contribution. K-braced frame is more efficient in preventing side sway than cross-braced frame for equal steel areas of braced members used. This type of system is preferred for longer bay width because of the shorter length of the braces. K-braced frame is found to be more efficient if the apexes of all the braces are pointing in the upward direction (Fig. 1.24c).

For eccentrically braced frame, the center line of the brace is positioned eccentrically to the beam-column joint, as shown in Fig. 1.24d. The system relies, in part, on flexure of the short segment of the beam between the brace-beam joint and the beam-column joint. The forces in the braces are transmitted to the column through shear and bending of the short beam segment. This particular arrangement provides a more flexible overall response. Nevertheless, it is more effective against seismic loading because it allows for energy dissipation due to flexural and shear yielding of the short beam segment.

### 1.4.3.2 Drift Assessment

The storey drift $\Delta$ of a single storey diagonally braced frame, as shown in Fig. 1.25, can be approximated by the following equation:

$$\Delta = \Delta_s + \Delta_f = \frac{HL_d^3}{A_d EL^2} + \frac{Hh^3}{A_c EL^2} \qquad (1.3)$$

where
$\Delta$ = total inter-storey drift
$\Delta_s$ = storey drift due to shear component
$\Delta_f$ = storey drift due to flexural component
$A_c$ = area of the chord
$A_d$ = area of the diagonal brace
E = modulus of elasticity
H = horizontal force in the storey
h = storey height
L = length of braced bay
$L_d$ = length of the diagonal brace

The shear component $\Delta_s$ in Eq. (1.3) is caused mainly by the axial deformation of the diagonal brace. The deformation associated with girder compression has been neglected in the calculation of $\Delta_s$ because the axial stiffness of the girder is very much larger than the stiffness of the brace. The elongation of the diagonal braces gives rise to shear deformation of the frame, which is a function of the brace length, $L_d$ and the angle of the brace ($L_d$/L). A shorter brace length with a smaller brace angle will produce a lower storey drift.

The flexural component of the frame drift is due to tension and compression of the windward and leeward columns. The extension of the windward column and shortening of the leeward column cause flexural deformation of the

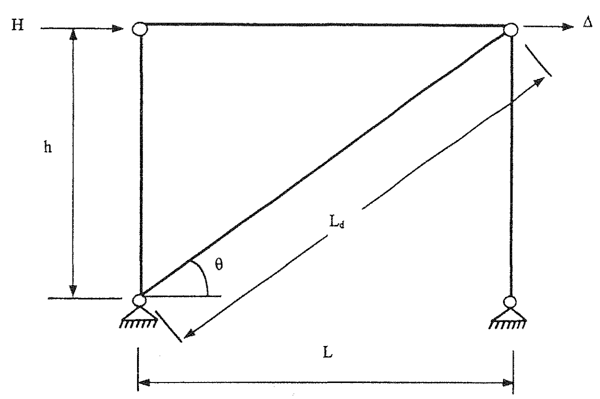



*Figure 1.25* *Lateral displacement of a diagonally braced frame*

frame, which is a function of the area of the column and the ratio of the height to bay length, (h/L). For slender bracing frame with large h/L ratio, the flexural component can contribute significantly to the overall storey drift.

Low-rise braced frame deflects predominately in shear mode while high-rise braced frames tend to deflect more in flexural mode.

### 1.4.3.3 Design Considerations

Frames with braces connecting columns may obstruct locations of access openings such as windows and doors, thus they should be placed where such access is not required, e.g., around elevators, service and stair wells. The location of the bracing systems within the structure will influence the efficiency with which the lateral forces can be resisted. The most appropriate position for the bracing systems is at the periphery of the building (Figure 1.26a) since this arrangement provides greater torsional resistance. Bracing frames should be situated where the center of lateral resistance is approximately equal to the center of shear resultant on plan. Where this is not possible, torsional forces will be induced, and they must be considered when calculating the load carried by each braced system.

When core braced systems are used, they are normally located at the center of the building (Fig. 1.26b). The torsional stability is then provided by the torsional rigidity of the core brace. For tall building frames, a minimum of three braced bents are required to provide transitional and torsional stability. These bents should be carefully arranged so that their planes of action do not meet at one point so as to form a center of rotation. The bracing arrangement shown in Fig. 1.26c should be avoided.

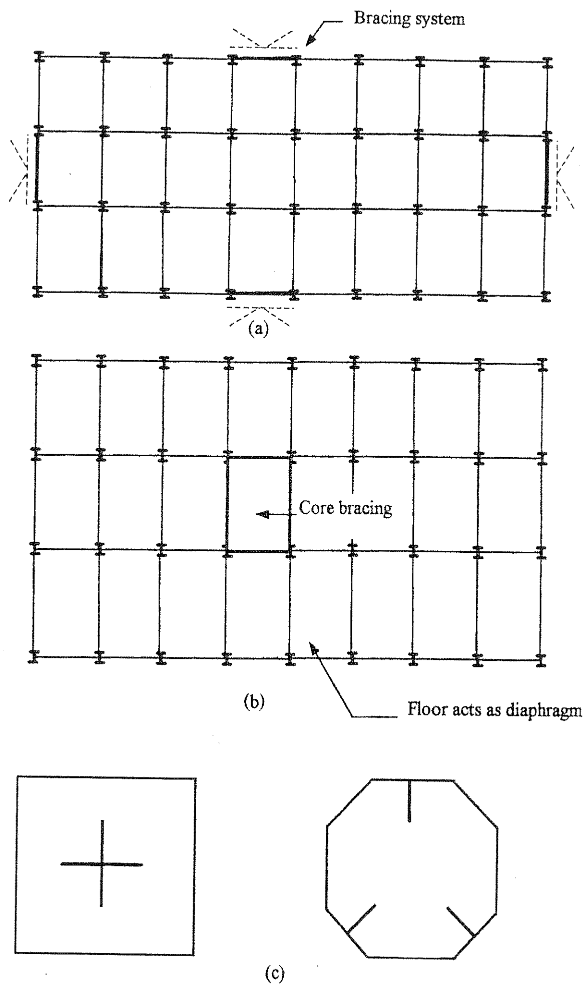



*Figure 1.26 Locations of braciang systems*

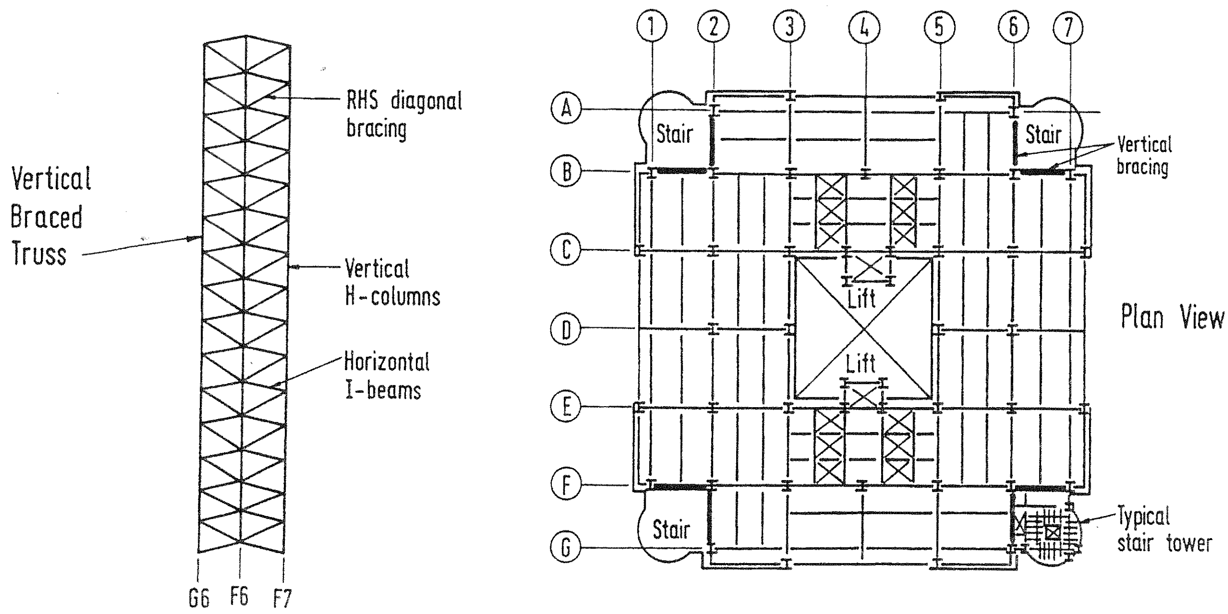



*Figure 1.27 Simple building frame with vertical braced trusses located at the corners*

The flexibility of different bracing systems must be taken into account in the analysis, since the stiffer braces will attract a larger share of the applied lateral load. For tall and slender frames, the bracing system itself can be a sway frame, and a second-order analysis is required to evaluate the required forces for ultimate strength and serviceability checks.

Lateral loads produce transverse shears, over turning moments and side sway. The stiffness and strength demands on the lateral system increase dramatically with height. The shear increases linearly, the overturning moment as a second power and sway as a fourth power of the height of the building. Therefore, apart from providing the strength to resist lateral shear and overturning moments, the dominant design consideration (especially for tall building) is to develop adequate lateral stiffness to control sway.

For serviceability verification, it requires that both the inter-storey drifts and the lateral deflections of the structure as a whole must be limited. The limits depend on the sensitivity of the structural elements to shear deformations. Recommended limits for typical multistorey frames are given in Table 1.2. When considering the ultimate limit state, the bracing system must be capable of transmitting the factored lateral loads safely down to the foundations. Braced bays should be effective throughout the full height of the building. If it is essential for bracing to be discontinuous at one level, provision must be made to transfer the forces to other braced bays. Where this is not possible, torsional forces may be induced, and they should be allowed for in design.

Figure 1.27 shows an example of a building which illustrates the locations at vertical braced trusses provided at the four corners to achieve lateral stability. Diaphragm action is provided by lightweight concrete slab which acts compositely with metal decking and floor beams. The floor beam-to-column connections are designed to resist shear force only as shown in the figure.

### 1.4.4 MOMENT-RESISTING FRAMES

In cases where bracing systems would disturb the functioning of the building, rigidly jointed moment resisting frames can be used to provide lateral stability to the building, as illustrated in Fig. 1.28a. The efficiency of developing of lateral stiffness is dependent on bay span, number of bays in the frame, number of frames and the available depth in the floors for the frame girders. For building with heights not more than three times the plan dimension, the moment frame system is an efficient form. Bay dimensions in the range of 6m to 9 m and structural height up to 20-30 storeys are commonly used. However, as the building height increases, deeper girders are required to control drift, thus the design becomes uneconomical.

When a rigid unbraced frame is subjected to lateral load, the horizontal shear in a storey is resisted predominantly by the bending of columns and beams. These deformations cause the frame to deform in a shear mode. The design of these frames is controlled therefore by the bending stiffness of individual members. The deeper the member, the more efficiently the bending stiffness can be developed. A small part of the frame side sway is caused by the overturning of the entire frame resulting in shortening and elongation of the columns at opposite sides of the frame. For unbraced rigid frames up to 20-30 storeys, the overturning moment contributes for about 10-20% of the total sway, whereas shear racking accounts for the remaining 80-90% (Fig. 1.28b). However, the storey drift due to overall bending tends to increase with height, while that due to shear racking tends to decrease.

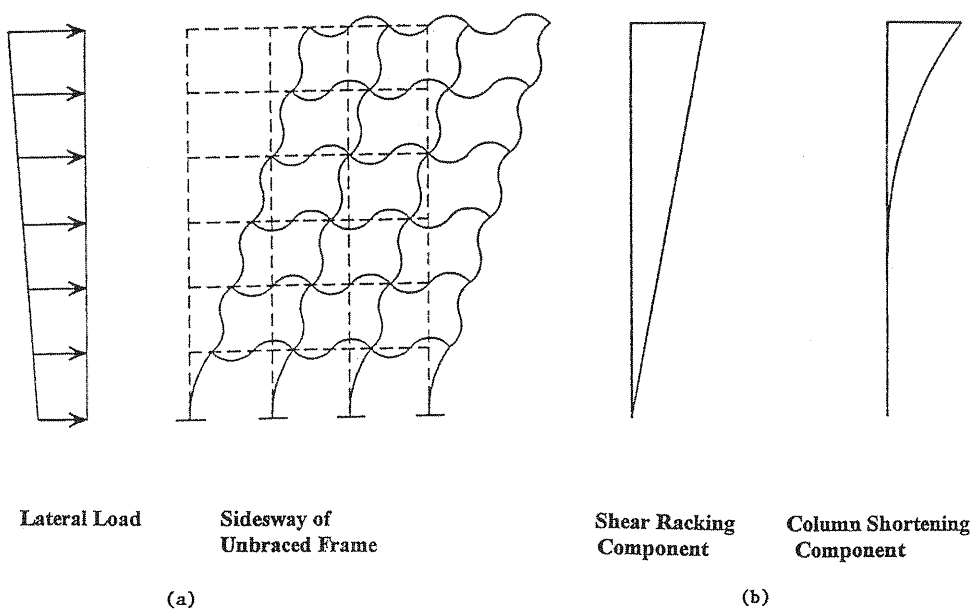



*Figure 1.28 Side away resistance of a rigid unbraced frame*

#### 1.4.4.1 Drift Assessment

Since shear racking accounts for most of the lateral sway, the design of such frames should direct towards minimizing the side sway due to shear. The shear displacement Δ in a typical storey in a multistorey frame, as shown in Fig. 1.29, can be approximated by the equation:

$$\Delta_i = \frac{V_i h_i^2}{12E}\left(\frac{1}{\Sigma(I_{ci}/h_i)} + \frac{1}{\Sigma(I_{gi}/L_i)}\right) \qquad (1.4)$$

where

$\Delta_i$ = is the shear deflection of the i-th storey
E = modulus of elasticity
$I_c$, $I_g$ = second moment of area for columns and girders respectively
$h_i$ = height of the ith storey
$L_i$ = length of girder in the i-th storey
$V_i$ = total horizontal shear force in the ith storey
$\Sigma(I_{ci}/h_i)$ = sum of the column stiffness in the i-th storey
$\Sigma(I_{gi}/L_i)$ = sum of the girder stiffness in the i-th storey

Examination of Eq. 1.4 shows that side-sway deflection caused by storey shear is influenced by the sum of column and beam stiffness in a storey. Since for multistorey construction, span lengths are generally larger than the storey height, the moment of inertia of the girders needs to be larger to match the column stiffness, as both of these members contribute equally to the storey drift. As the beam span increases, considerably deeper beam sections will be required to control frame drift.

Since the gravity forces in columns are cumulative, larger column sizes are needed in lower stories as the frame height increases. Similarly, storey shear forces are cumulative, and therefore, larger beam properties in lower stories are required to control lateral drift. Because of limitations in available depth, heavier beam members will need to be provided at lower floors. This is the major shortcoming of unbraced frames because considerable premium for steel weight is required to control lateral drift as building height increases.

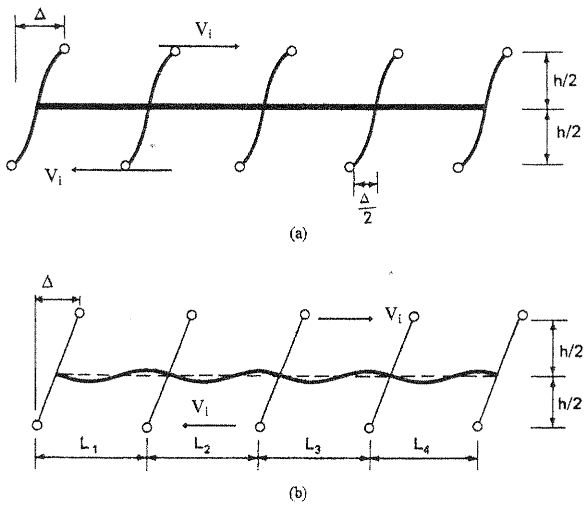



*Figure 1.29 Story drift due to (a) bending of columns (b) bending of girders*

Apart from the beam span, height-to-width ratios of the building play an important role in the design of such structure. Wider building frames allow a larger number of bays (i.e., larger values for storey summation terms $\Sigma(I_{ci}/h_i)$ and $\Sigma(I_{gi}/L_i)$ in Eq. 1.4) with consequent reduction in frame drift. Moment frames with closed spaced columns which are connected by deep beams are very effective in resisting side-sway. This kind of framing system is suitable for use in the exterior planes of the building.

#### 1.4.4.2 Moment Connections

Fully welded moment joints are expensive to fabricate. To minimize labor cost and to speed up site erection, field bolting instead of site welding should be used. Figure 1.30 shows several types of bolted or welded moment connections that are used in practice. Beam-to-column flange connections can be shop-fabricated by welding of a beam stub to an end plate or directly to a column. The beam can then be erected by field bolting the end plate to the column flanges or splicing beams (Fig.1.30c and 1.30d).

An additional parameter to be considered in the design of columns of an unbraced frame is the "panel zone" between the column and the transverse framing beams. When an unbraced frame is subjected to lateral load, additional shear forces are induced in the column web panel as shown in Fig. 1.31. The shear force is induced by the unbalanced moments from the adjoining beams causing the joint panel to deform in shear. The deformation is attributed to the large flexibility of the unstiffened column web. To prevent shear deformation so as to maintain the moment joint assumption as assumed in the global analysis, it may be

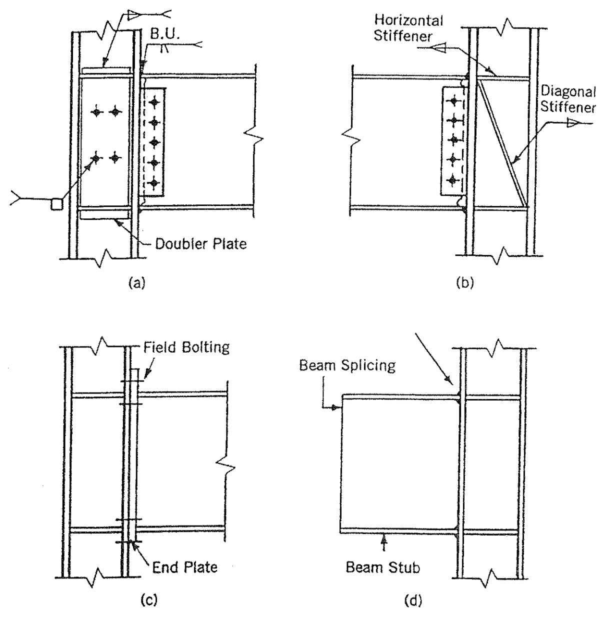



*Figure 1.30 (a) Bolted and welded connection with a doubler plate*
*(b) Bolted and welded connection with a diagonal stiffener*
*(c) Bolted end-plate connection*
*(d) Beam stub welded to a column*

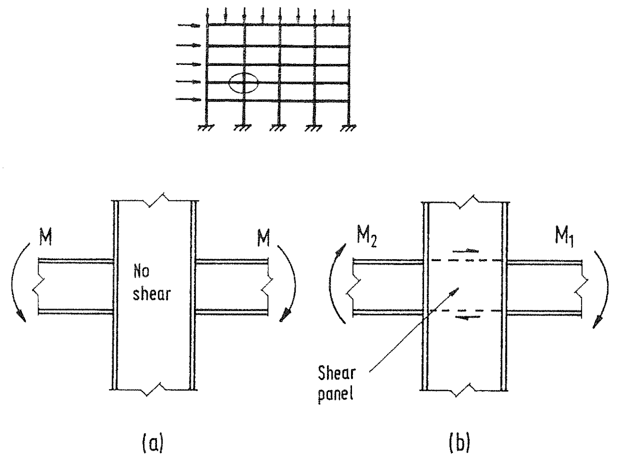



*Figure 1.31* *Forces acting on a panel joint*
*(a) balanced moment due to gravity load*
*(b) unbalanced moment due to lateral load*

necessary to stiffen the panel zone using either a doubler plate or a diagonal stiffener as shown in the joint details in Figs. 1.30a and 1.30b. Otherwise, a heavier column with larger web area is required to prevent excessive shear deformation, and this is often the preferred method as stiffeners and doublers can add significant costs to fabrication.

The engineer should not specify full-strength moment connections unless they are required for ductile frame design for high seismic loads. For wind loads and for conventional moment frames where beams and columns are sized for stiffness (drift control) instead of strength, full strength moment connections are not required. Even so, many designers will specify full strength moment connections, adding to the cost of fabrication. Designing for actual loads has the potential to reduce column weight or reduce the stiffener and doubler plate requirements.

If the panel zone is stiffened to prevent inelastic shear deformation, the conventional structural analysis based on the member center-line dimension will generally overestimate the frame displacement. If the beam-column joint sizes are relatively small compared to the member spans, the increase in frame stiffness using member center-line dimension will be offset by the increase in frame deflection due to panel-joint shear deformation. If the joint sizes are large, a more rigorous second-order analysis, which considers panel zone deformations, may be required for an accurate assessment of the frame response.

#### 1.4.4.3 Analysis And Design of Unbraced Frames

Multistorey moment frames are statically indeterminate, the required design forces can be determined using either: (1) elastic analysis or (2) plastic analysis. Whilst elastic methods of analysis can be used for all kind of steel sections, plastic analysis is only applicable for frames whose members are of plastic sections so as to enable the development of plastic hinges and to allow for inelastic redistribution of forces.

First-order elastic analysis can be used only in the following cases:
1) Where the frame is braced and not subjected to side sway.
2) Where an indirect allowance for second-order effects is made through the use of moment amplification factors and/or the column effective length. Eurocode 3 requires only second-order moment or effective length factor to be used in the beam-column capacity checks. However, column and frame imperfections need to be modeled explicitly in the analysis. In the American Standard (AISC LRFD), both factors need to be computed for checking the member strength and stability, and the analysis is based on structures without initial imperfections.

The first-order elastic analysis is a convenient approach. Most design offices possess computer software capable of performing this method of analysis on large and highly indeterminate structures. As an alternative, hand calculations can be performed on appropriate sub-frames within the structure (see Figure 1.32) comprising a significantly reduced number of members. However, when conducting the analysis of an isolated sub-frame it is important that:
1) The sub-frame is indeed representative of the structure as a whole.
2) The selected boundary conditions are appropriate.
3) Account is taken of the possible interaction effects between adjacent sub-frames.
4) Allow for second-order effects through the use of column effective length or moment amplification factors.

Plastic analysis generally requires more sophisticated computer programs, which enable second order effects to be taken into account. For building structures in which the required rotations are not calculated, all members containing plastic hinges must have plastic cross-sections.

A basic procedure for the design of an unbraced frame is as follows:
1) Obtain approximate member size based on gravity load analysis of sub-frames shown in Fig. 1.32. If side-sway deflection is likely to control (e.g., slender frames) use Eq. 1.4 to estimate the member sizes.
2) Determine wind moments from the analysis of the entire frame subjected to lateral load. A simple portal wind analysis may be used in lieu of the computer analysis.
3) Check member capacity for the combined effects of factored lateral load plus gravity loads.
4) Check beam deflection and frame drift.
5) Redesign the members and perform final analysis/ design check (a second-order elastic analysis is preferable at the final stage).

The need to repeat the analysis to correspond to changed section sizes is unavoidable for highly redundant frames. Iteration of Steps 1 to 5 gives results that will converge to an economical design satisfying the various design constraints imposed on the analysis.

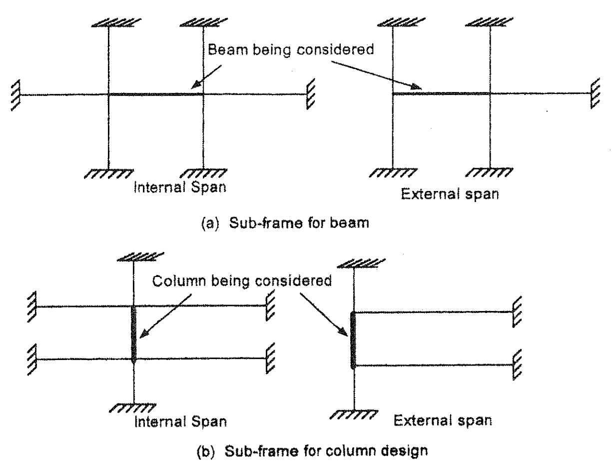



***Figure 1.32*** *Sub-frame analysis for gravity loads*

### 1.4.5 BUILDING FRAMING SYSTEMS

The following subsections discuss four classical systems that have been adopted for tall building constructions, namely (1) core braced system, (2) moment-truss system (3) outriggle and belt system, and (4) tube systems. Tall frames that utilize cantilever action will have higher efficiencies, but the overall structural efficiency depends on the height-to-width ratio. Interactive systems involving moment frame and vertical truss or core are effective up to 40 stories and represent most building forms for tall structures. Outrigger truss and belt truss help to further enhance the lateral stiffness by engaging the exterior frames with the core braces to develop cantilever actions. Exterior framed tube systems with closely spaced exterior columns connected by deep girders mobilize the three-dimensional action to resist lateral and torsional forces. Bundled tubes improve the efficiency of exterior frame tubes by providing internal stiffening to the exterior tube. Finally, by providing diagonal braces to the exterior framework, a superframe is formed and can be used for ultra-tall magastructures.

#### 1.4.5.1 Core Braced Systems

This type of structural system relies entirely on the internal core for lateral load resistance. The basic concept is to provide internal shear wall core to resist the lateral forces (Fig. 1.33). The surround steel framing is designed to carry gravity load only if simple framing is adopted. Otherwise, a rigid framing surrounding the core will enhance the overall lateral-force resistance of the structure. The steel beams can be simply connected to the core walls using a typical corbel detail, or by bearing in a wall pocket or by shear plate embedded in the core wall through studs. If rigid connection is required, the steel beams should be rigidly connected to steel columns embedded in the core wall. Rigid framing surrounding the cores are particularly useful in high seismic areas, and for very tall buildings that tend to attract stronger wind loads. They act as moment frames and provide resistance to some part of the lateral loads by engaging the core walls in the building.

The core generally provides all torsional and flexural rigidity and strength with no participation from the steel system. Conceptually, the core system should be treated as a cantilever wall system with punched openings for access. The floor-framing should be arranged in such a way that it distributes enough gravity loads to the core walls so that their design is controlled by compressive stresses even under wind loads. The geometric location of the core should be selected so as to minimize eccentricities for lateral load. The core walls need to have adequate torsional resistance for possible asymmetry of the core system where the center of the resultant shear load is acting at an eccentricity from the center of the lateral-force resistance.

A simple cantilever model should be adequate to analyze a core wall structure. However, if the structural form is a tube with openings for access, it may be necessary to perform a more accurate analysis to include the effect of openings. The walls can be analysed by a finite element analysis using thin-walled plate elements. An analysis of this type may also be required to evaluate torsional stresses when the vertical profile of the core-wall assembly is asymmetrical.

The concrete core walls can be constructed using slip-form techniques, where the core walls could be advanced several

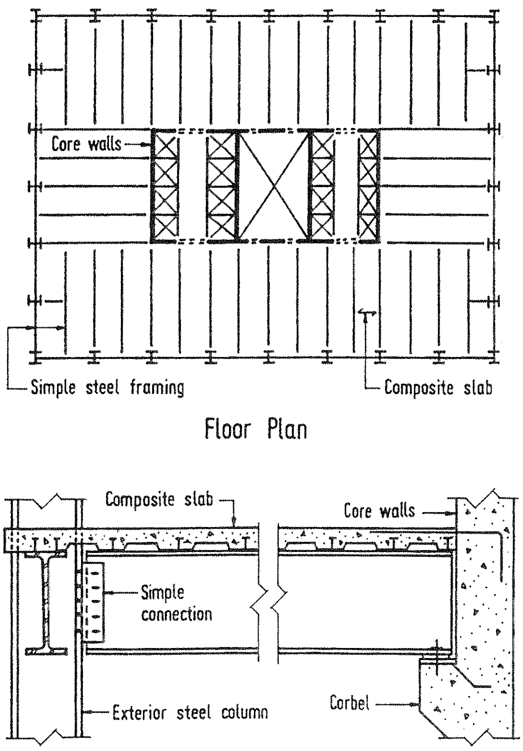



***Figure 1.33*** *Core-braced frame*
*(a) Internal core walls with simple exterior framing*
*(b) Beam-to-wall and beam to exterior column connections*

floors (typically 4 to 6 storeys) ahead of the exterior steel framing. Core wall system represents an efficient type of structural system up to certain height premium because of its cantilever action. However, when it is used alone, the massiveness of the wall structure increase with height, thereby inhabiting the free planning of interior spaces, especially in the core. The space occupied by the shear walls leads to loss of overall floor area efficiency, as compared to tube system which could otherwise be used.

In commercial buildings where floor space is a valuable, the large area taken up by a concrete column can be reduced by the use of an embedded steel column to resist the extreme loads encountered in tall building. Sometimes, particularly at the bottom open floors of a high rise structure where large open lobbies or atriums are utilized as part of the architectural design, a heavy embedded steel section as part of a composite column is necessary to resist high load and due to the long unbraced length. A heavy steel section in a composite column is often utilized where the column size is restricted architecturally and where reinforcing steel percentages would otherwise exceed the maximum code allowed values for the deign of reinforced concrete columns.

### 1.4.5.2 Frame-Truss Systems

Vertical shear trusses located around the inner core in one or both directions can be combined with perimeter moment-resisting frames in the facade of a building to form an efficient structure for lateral load resistance. An example of a building consisting of moment frames with shear trusses located at the center of the building is shown in Fig. 1.34a. For the vertical trusses arranged in the North-South direction, either the K- or X-form of bracing is acceptable since access to lift-shafts is not required. However, K trusses are often preferred because in the case of X or single brace form bracings the influence of gravity loads is rather significant. In the East-West direction, only the Knee bracing is effective in resisting lateral load.

In some cases internal bracing can be provided using concrete shear walls as shown in Fig. 1.34b. The internal core walls substitute the steel trusses in K, X or single brace form which may interfere with openings that provide access to, for example, elevators.

The interaction of shear frames and vertical trusses produces a combination of two deflection curves with the effect of more efficient stiffness. These moment frame-truss interacting systems are considered to be the most economical steel systems for buildings up to 40 stories. Figure 1.35 compares the sway characteristic of a twenty-storey steel frame subjected to same lateral forces, but with different structural schemes namely (1) unbraced moment frame, (2) simple-truss frame and (3) moment-truss frame. The simple-truss frame helps to control lateral drift at the lower stories, but the overall frame drift increases toward the top of the frame. The moment frame, on the other hand, shows an opposite characteristic for side sway in comparison with the simple braced frame. The combine of moment frame and truss frame provides overall improvement in reducing frame drift; the benefit becomes

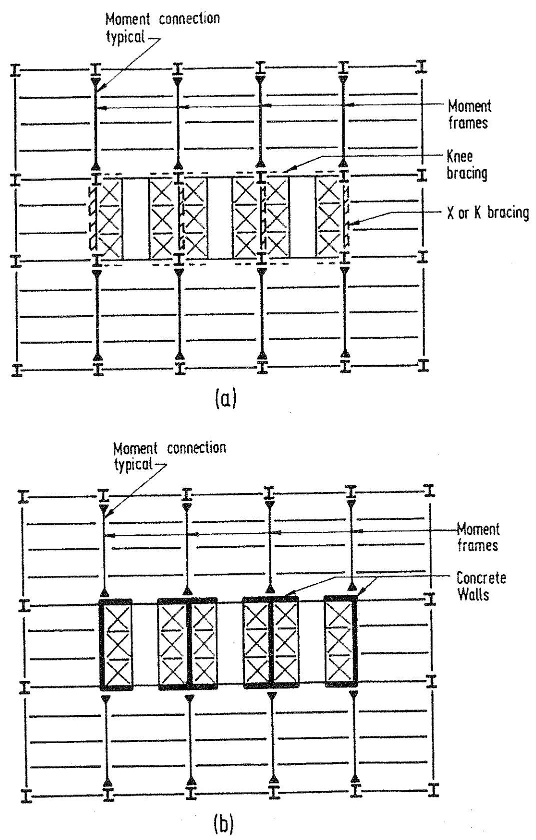



*Figure 1.34* *(a) Moment-frames with internal braced trusses*
*(b) Moment-frames with internal core walls*

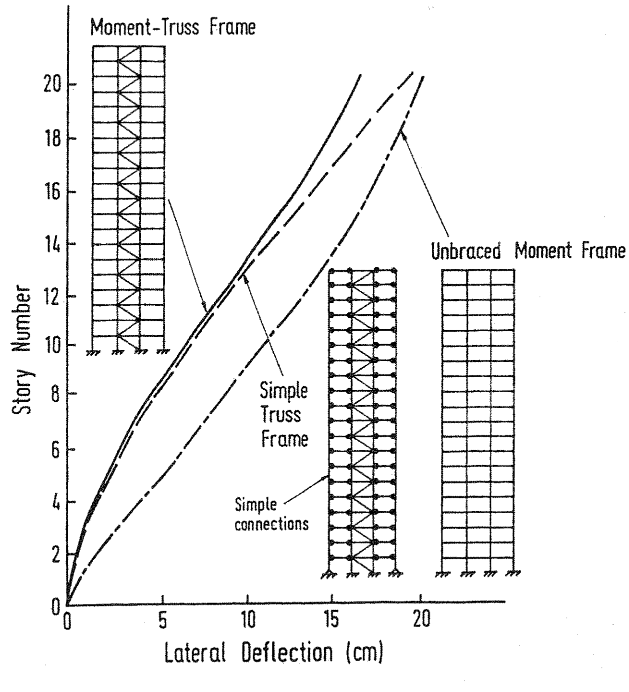



*Figure 1.35* *Sway characteristics of rigid braced frame, simple braced frame and rigid unbraced frame*

more pronounce towards the top of the frame. The braced truss is restrained by the moment frame at the upper part of the building while at the lower part, the moment frame is restrained by the truss frame. This is because the slope of frame sway displacement is relatively smaller than that of the truss at the top while the proportion is reversed at the bottom. The interacting forces between the truss frame and moment frame, as shown in Fig. 1.36, enhance the combined moment-truss frame stiffness to a level larger than the summation of individual moment frame and truss stiffnesses.

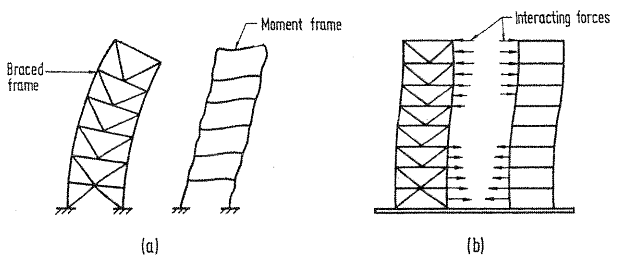



*Figure 1.36 Behavior of frames subjected to lateral load*
*(a) independant behavior*
*(b) interactive behavior*

### 1.4.5.3 Outrigger and Belt Truss Systems

Another significant improvement of lateral stiffness can be obtained if the vertical truss and the perimeter shear frame are connected on one or more levels by a system of outrigger and belt trusses. Figure 1.37 shows a typical example of such system. The outrigger truss leads the wind forces of the core truss to the exterior columns providing cantilever behavior of the total frame system. The belt truss in the facade improves the cantilever participation of the exterior frame and creates a three-dimensional frame behavior.

Figure 1.38 shows a schematic diagram that demonstrates the sway characteristic of the overall building under lateral load. Deflection is significantly reduced by the introduction of the outrigger-belt trusses. Two kinds of stiffening effects can be observed; one is related to the participation of the external columns together with the internal core to act in a cantilever mode; the other is related to the stiffening of the external facade frame by the belt truss to act as a three-dimensional tube. The overall stiffness can be increased up to 25% as compared to the shear truss and frame system without such outrigger-belt trusses.

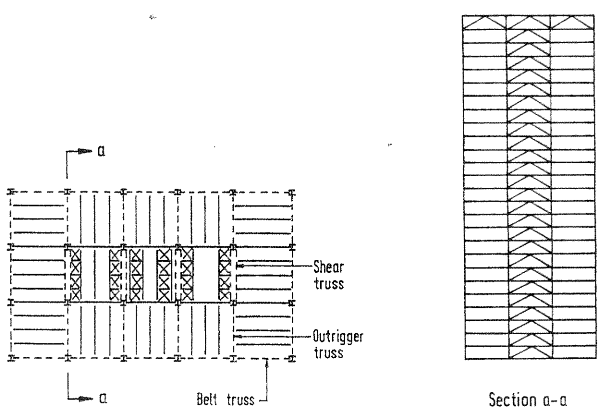



*Figure 1.37 Outrigger and belt-truss system*

The efficiency of this system is related to the number of trussed levels and the depth of the truss. In some cases the outrigger and belt trusses have a depth of two or more floors. They are located in services floors where there are no requirements for wide open spaces. These trusses are often pleasingly integrated into the architectural conception of the facade.

### 1.4.5.4 Frame Tube Systems

Figure 1.39 shows a typical frame tube system, which consists of a frame tube at the exterior of the building and gravity steel framing at the interior. The framed tube is constructed from wide columns placed at close centers connected by deep beams creating a punched wall appearance. The exterior frame tube structure resists all lateral loads of wind or earthquake whereas the gravity steel framing in the interior resist only its share of gravity loads. The behavior of the exterior frame tube is similar to a hollow perforated tube. The over-turning moment under the action of lateral load is resisted by compression and tension of the leedward and windward columns, which are called the flange columns. The shear is resisted by bending of the columns and beams at the two sides of the building parallel to the direction of the lateral load, which are called the web frames.

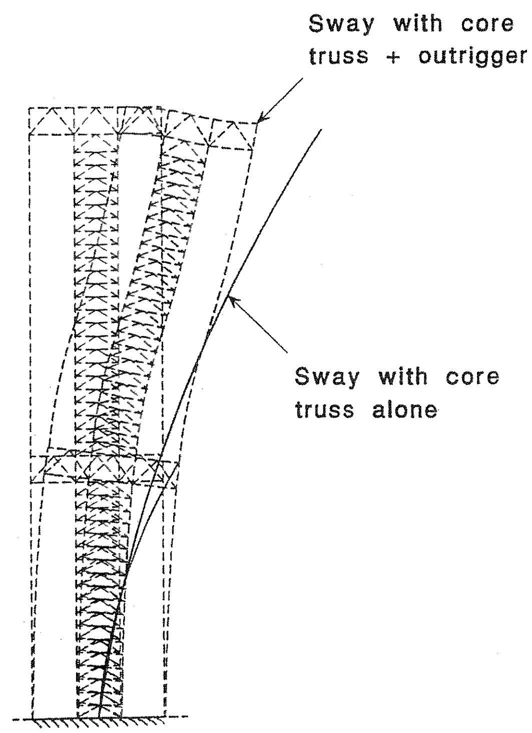



*Figure 1.38 Improvement of lateral stiffness using outrigger-belt truss system*

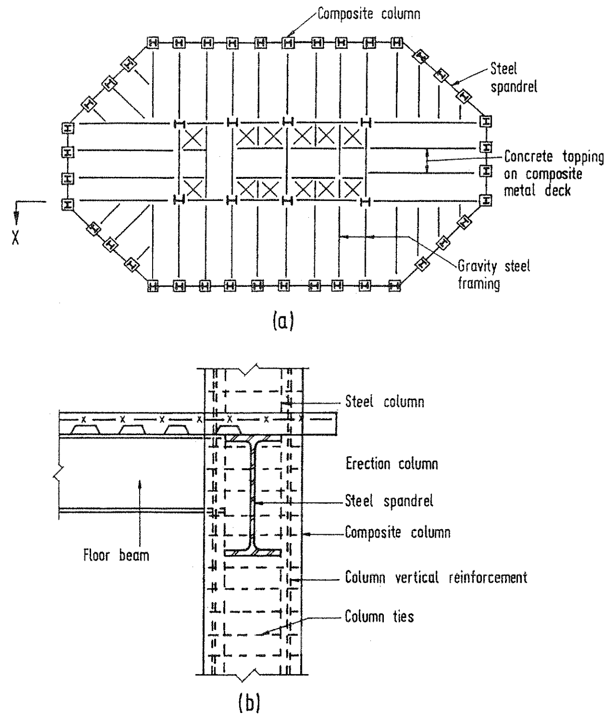



*Figure 1.39 Composite tubular system*

Deepening on the shear rigidity of the frame tube, there may exist a shear lag across the windward and leeward sides of the tube. As a result of this, not all the flange columns resist the same amount of axial force. An approximate approach is to assume an equivalent column model as shown in Fig. 1.40. In the calculation of the lateral deflection of the frame tube it is assumed that only the equivalent flange columns on the windward and leeward sides of the tube and the web frames would contribute to the moment of inertia of the tube.

The use of exterior framed tube has two distinct advantages: (1) It develops high rigidity and strength for torsional and lateral-load resistance, since the structural components are effectively placed at the exterior of the building forming a three-dimensional closed section. (2) Massiveness of frame tube system eliminates potential uplift difficulties and produces better dynamic behavior. (3) The use of gravity steel framing in the interior has the advantages of flexibility, and enables rapid construction. If composite floor with metal decking is used, electrical and mechanical services can be incorporated in the floor zone.

Composite columns are frequently used in the perimeter of the building where the closely spaced columns work in conjunction with the spandrel beam (either steel or concrete) to form a three-dimensional cantilever tube rather than an assembly of two-dimensional plane frames. The exterior frame tube significantly enhances the structural efficiency in resisting lateral loads and thus reduces the shear wall requirements. However, in cases where a higher magnitude of lateral stiffness is required (such as for vary tall buildings), internal wall cores and interior columns with floor framing can be added to transform the system into a tube-in-tube system. The concrete core may be strategically located to recapture elevator space and to provide transmission of mechanical ducts from shafts and mechanical rooms.

### 1.4.6 STEEL-CONCRETE COMPOSITE SYSTEMS

Steel-concrete composite construction has gained wide acceptance as an altérnative to pure steel and pure concrete construction. Composite building systems can be broadly categorized into two forms: one utilizes the core-braced system by means of interior shear walls, and the other utilizes exterior framing to form a tube for lateral load resistance. Combining these two structural forms will enable taller buildings to be constructed.

For composite frames resisting gravity load only, the beam-to-column connections behave as pinned before the placement of concrete. During construction, the beam is designed to resist concrete dead load and the construction load (to be treated as temporary live load). At the composite stage, the composite strength and stiffness of the beam should be utilized to resist the full design loads. For gravity frames consisting of bare steel columns and composite beams, there is now sufficient knowledge available for the designer to use composite action in the structural element as well as the semi-rigid composite joints to increase design choices, leading to more economical solutions.

Figures 1.41a&b show the typical beam-to-column connections, one using flushed end-plate bolted to the column flange and the other using bottom angle with double web cleats. Composite action in the joint is developed based on the tensile forces developed in the rebars acting with the balancing compression forces transmitted by the lower portion of the steel section that bear against the column flange to form a couple. Properly designed and detailed composite connections are capable of providing moment resistance up to the hogging resistance of the connecting members.

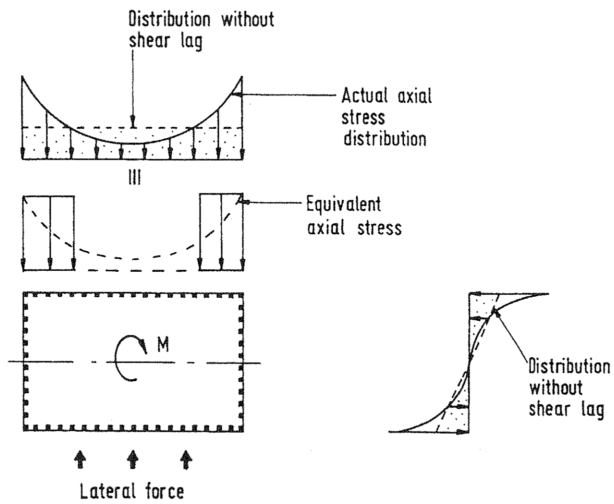



*Figure 1.40 Equivalent column model for frame tube*

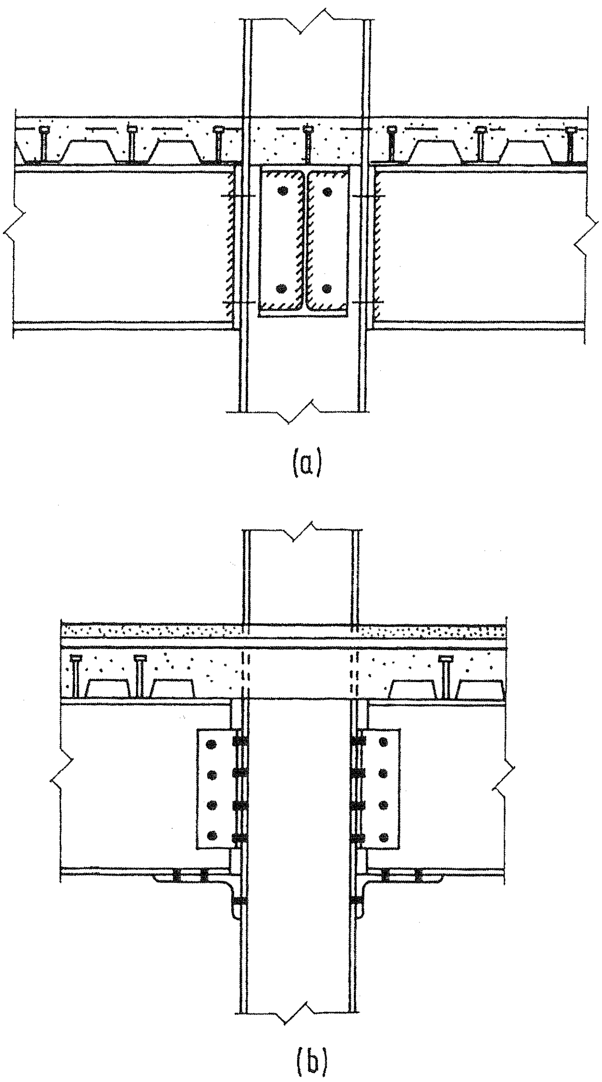



*Figure 1.41* *Composite beam-to-column connections with*
*(a) flushed end plate*
*(b) seat and double web angles*

In designing the connections, slab reinforcements placed within a horizontal distance of 6 times the slab depth are assumed to be effective in resisting the hogging moment. Reinforcement steels that fall outside this width should not be considered in calculating the resisting moment of the connection (see Fig. 1.42). The connections to edge columns should be carefully detailed to ensure adequate anchorage of re-bars. Otherwise they shall be designed and detailed as simply supported. In braced frame a moment connection to the exterior column will increase the moments in the column, resulting in an increase of column size. Although the moment connections restrain the column from buckling by reducing the effective length, this is generally not adequate to offset the strength required to resist this moment.

For an unbraced frame subjected to gravity and lateral loads, the beam typically bends in double curvature with negative moment at one end of the beam and positive moment on the other end. The concrete is assumed to be ineffective in tension, therefore only the steel beam stiffness on the negative moment region and the composite stiffness on the positive moment region can be utilized for frame action. The frame analysis can be performed with variable moment of inertia for the beams. Further research is still needed in order to provide tangible guidance for design.

If semi-rigid composite joints are used in unbraced frames, the flexibility of the connections will contribute to additional drift over that of a fully rigid frame. In general, semi-rigid connections do not require the column size to be increased significantly over an equivalent rigid frame. This is because the design of frames with semi-rigid composite joints takes advantage of the additional stiffness in the beams provided by the composite action. The increase in beam stiffness would partially offset the additional flexibility introduced by the semi-rigid connections.

Further research is required to assess the performance of various types of composite connections used in building construction. Issues related to accurate modeling of effective stiffness of composite members and joints in unbraced frames for the computation of second-order effects and drifts need to be addressed.

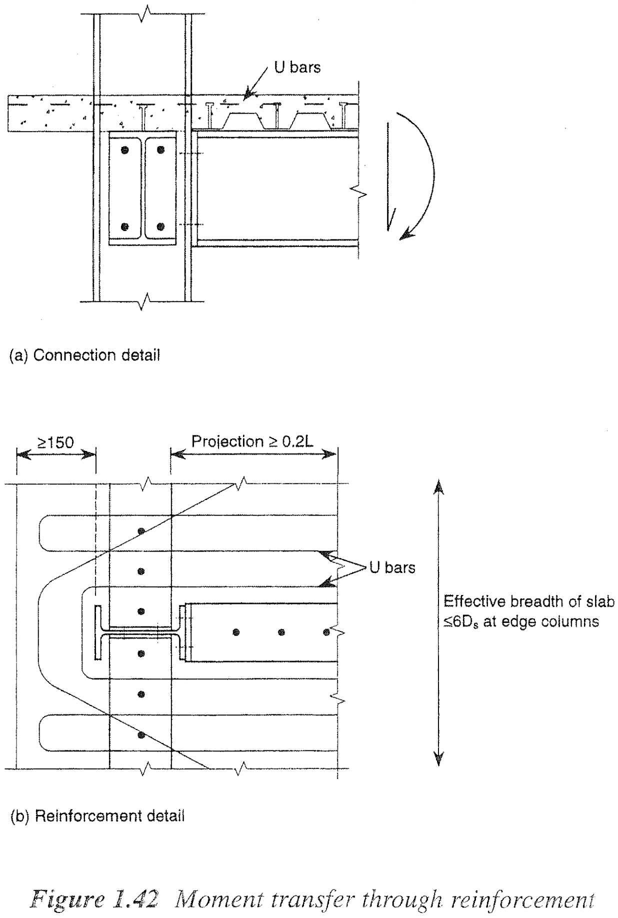



*Figure 1.42* *Moment transfer through reinforcement at perimeter columns*

## 1.5 WIND EFFECTS ON BUILDINGS

The design of multistorey buildings in non seismic areas is often governed by the need to limit the wind induced accelerations and drift to acceptable levels for human comfort and integrity of non structural components respectively. To check for serviceability of tall buildings, the peak resultant horizontal acceleration and displacement due to the combination of along wind, across wind and torsional loads are required. As an approximate estimation, the peak effects due to along wind, across wind and torsional responses may be determined individually and then combined vectorially. A reduction factor of 0.8 may be used on the combined value to account for the fact that in general the individual peaks do not occur simultaneously. If the calculated combined effect is less than any of the individual effects, then the latter should be considered for the design.

Acceleration limits and their effects on human comfort are given in Table 1.3. The factors affecting the human confort are:

- Period of building - tolerence to acceleration tends to increase with period.
- Perception threshold increases while sitting than standing.
- Perception threshold level decreases with prior knowledge that motion will occur.
- Human body is more sensitive to fore-and-back motion than to side-to-side motion.
- Perception threshold is higher while walking than standing.
- Women and children are more sensitive to movement.

**Table 1.3** Acceleration limits for different perception levels

| Perception | Acceleration limits |
|---|---|
| Imperceptible | < 0.005 g |
| Perceptible | 0.005 to 0.015 g |
| Annoying | 0.015 to 0.05 g |
| Very annoying | 0.05 to 0.15 g |
| Intolerable | > 0.15g |

- Visual cue - very sensitive to rotation of the building relative to fixed landmarks outside.
- Acoustic cue - Building make sounds while swaying due to rubbing of contact surfaces. These sounds, and sounds of the wind whistling focus the attention on building motion even before motion is perceived, and thus lower the perception threshold.
- The resultant translational acceleration due to the combination of longtitudinal, lateral and torsional motions causes human discomfort. In addition, angular (torsional) motion appears to be more noticeable.

The tolerable acceleration levels increase with period of building. The recommended design standard for peak acceleration for 10 year wind in commercial and residential buildings is as depicted in Figure 1.43. Lower acceleration levels are used for residential buildings for the following reasons:

1. Residential buildings are occupied for longer hours of the day and night and are therefore more likely to experience the design wind storm.

**Table 1.4** Serviceability problems at various deflection or drift indices

| Deformation as a Fraction of Span or Height | Visibility of Deformation | Typical Behaviour |
|---|---|---|
| < 1/500 | Not visible | Cracking of partition walls |
| $\frac{1}{300}$ to $\frac{1}{500}$ | Visible | General architectural damage<br>Cracking in reinforced walls<br>Cracking in secondary members<br>Damage to ceiling and flooring<br>Facade damage<br>Cladding leakage<br>Visual annoyance |
| 1/200 to 1/300 | Visible | Improper drainage |
| 1/100 to 1/200 | Visible | Damage to lightweight partitions, windows, finishes<br>Impaired operation of removable components such as doors, windows, sliding partitions |

2. People are less sensitive to motion when they are occupied with their work than when they relax at home.
3. People are more tolerant of their work environment than of their home environment.
4. Occupancy turnover rates are higher in commercial buildings than in residential buildings.
5. People can be easily evacuated from commercial buildings than residential buildings in the event of a peak storm.

The effects of excessive deflection on building components is described in Table 1.4. The allowable drift, defined as the resultant peak displacement at the top of the building divided by the height of the building is generally taken to be in the range 1/450 to 1/600.

Figure 1.44 depicts schematically the procedure of estimating the wind induced accelerations and displacements in a building. Wind tunnel studies may be required for tall building structures where analytical methods cannot be used to estimate certain types of wind induced responses. For examples, the aerodynamic shape of the building is uncommon or building is very flexible.

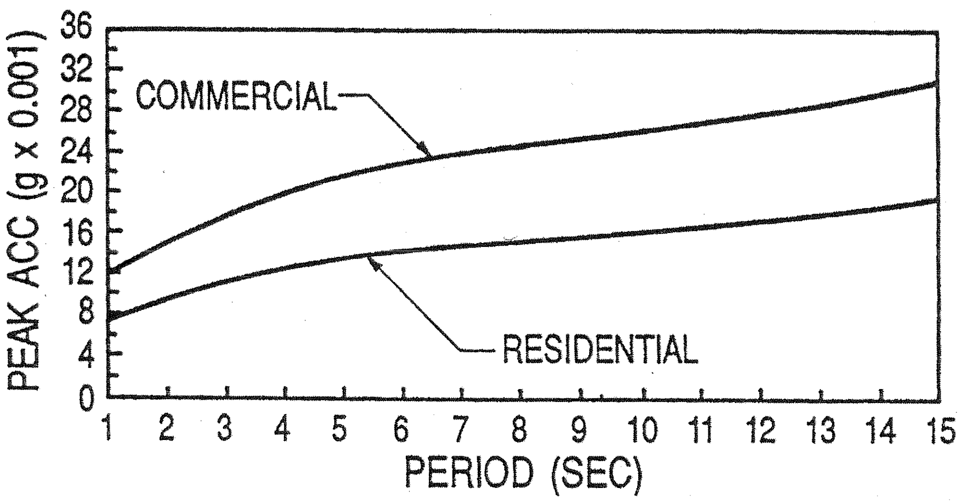



*Figure 1.43 Peak acceleration for 10 years return period*

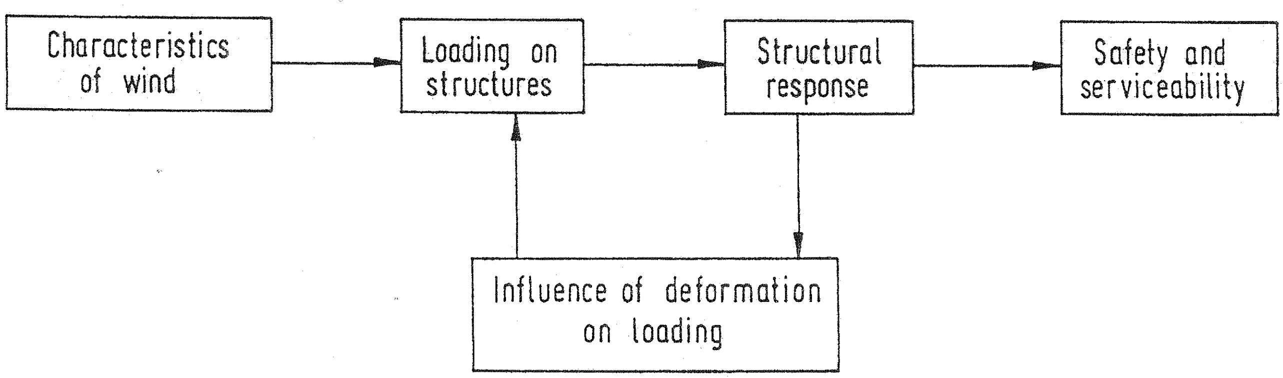



*Figure 1.44 Schematic diagram for wind resistant design*

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