Richard Liew, J.Y.; Balendra, T. and Chen, W.F. “Multistory Frame Structures”
Structural Engineering Handbook
Ed. Chen Wai-Fah
Boca Raton: CRC Press LLC, 1999

Multistory Frame Structures

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12.1 Classification of Building Frames

For building frame design, it is useful to define various frame systems in order to simplify models of analysis. For example, in the case of a braced frame, it is not necessary to separate frame and bracing behavior because both can be analyzed with a single model. On the other hand, 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 be able to capture the behavior of individual subframes and their effects on the overall structures.

The remainder of this section attempts to describe what a framed system represents, define 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 tall building construction are also given.

12.1.1 Rigid Frames

A rigid 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 adequate 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.

A rigid unbraced frame should be capable of resisting lateral loads without relying on an additional bracing system for stability. The frame, by itself, has to resist all the design forces, including gravity as well as lateral forces. At the same time, it should have adequate lateral stiffness against sidesway 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 have the following benefits:

1. Rigid connections are more ductile and therefore the structure performs better in load reversal situations or in earthquakes.
2. 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.

12.1.2 Simple Frames (Pin-Connected Frames)

A simple frame refers to a structural system in which the beams and columns are pinned connected and the system is incapable of resisting any lateral loads. The stability of the entire structure must be provided for by attaching the simple frame to some form of bracing system. 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 12.1 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 multistory 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 over welded connections, which normally require weld inspection, weather protection, and surface preparation.
3. It is easier to design and analyze 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, the sizing of the simply supported girders and the 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.

Actual connections in structures do not always fall within the categories of pinned or rigid connections. Practical connections are semi-rigid in nature and therefore the pinned and rigid conditions are only idealizations. Modern design codes allow the design of semi-rigid frames using the concept of wind moment design (type 2 connections). In wind moment design, the connection is assumed to be capable of transmitting only part of the bending moments (those due to the wind only). Recent development in the analysis and design of semi-rigid frames can be obtained from Chen et al. [15]. Design guidance is given in Eurocode 3 [22].

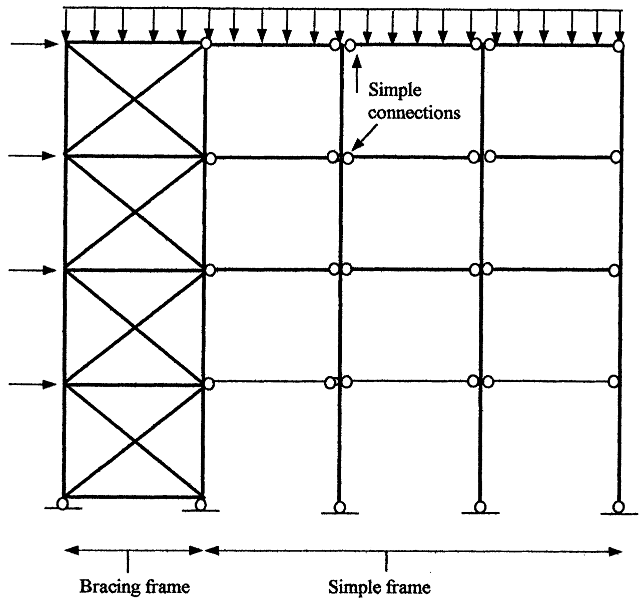


FIGURE 12.1: Simple braced frame.

12.1.3 Bracing Systems

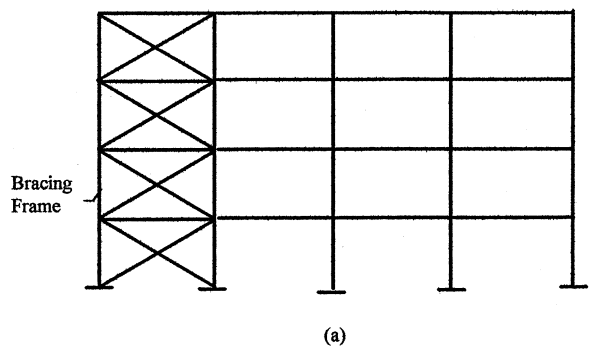
Bracing systems refer to structures that can provide lateral stability to the overall framework. It may be in the form of triangulated frames, shear wall/cores, or rigid-jointed frames. It is common to find bracing systems represented as shown in Figure 12.2. They are normally located in buildings to accommodate lift shafts and staircases.

In steel structures, it is common to represent a *bracing system* by a triangulated truss because, unlike concrete structures where all the joints are naturally continuous, the most immediate way of making connections between steel members is to hinge one member to the other. As a result, common steel building structures are designed to have bracing systems in order to provide sidesway resistance. Therefore, bracing can only be obtained by use of triangulated trusses (Figure 12.2a) or, exceptionally, by a very stiff structure such as shear wall or core wall (Figure 12.2b). 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 absence of shear walls and cores around lift shafts and stair wells.

12.1.4 Braced Frames vs. Unbraced Frames

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

Figures 12.3 and 12.4 represent the structures that are easy to define within one system: two subassemblies identifying the bracing system and the system to be braced. For the structure shown in Figure 12.3, there is a clear separation of functions in which the gravity loads are resisted by the hinged subassembly (Frame B) and the horizontal load loads are resisted by the braced assembly



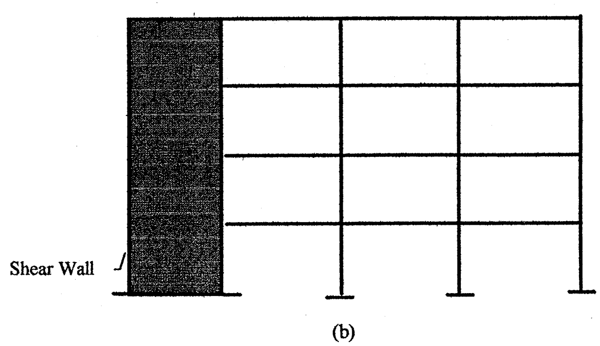


FIGURE 12.2: Common bracing systems: (a) vertical truss system and (b) shear wall.

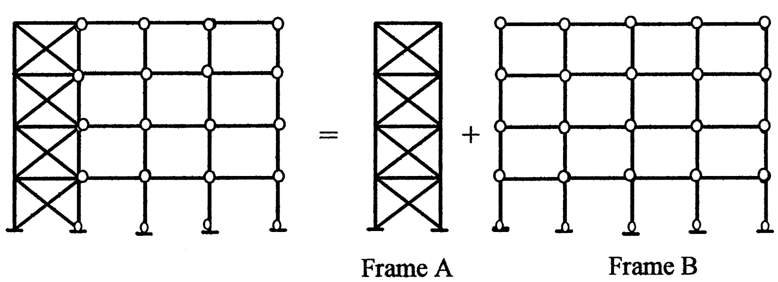


FIGURE 12.3: Pinned connected frames split into two subassemblies.

(Frame A). In contrast, for the structure in Figure 12.4, 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.

Eurocode 3 [22] gives a clear guidance in defining braced and unbraced frames. 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%.

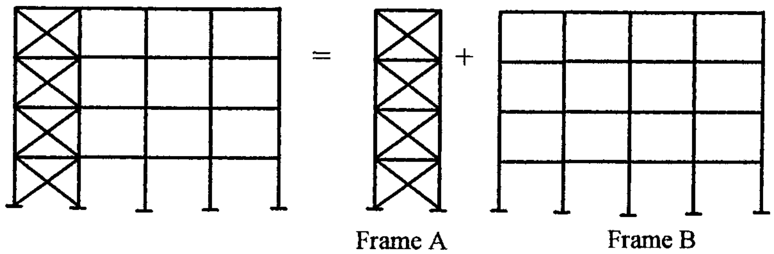


FIGURE 12.4: Mixed frames split into two subassemblies.

For the frame shown in Figure 12.3, the hinged frame (Frame B) has no lateral stiffness, and Frame A (truss frame) resists all lateral load. In this case, Frame B is considered to be braced by Frame A. For the frame shown in Figure 12.4, Frame B may be considered to be a braced frame if the following deflection criterion is satisfied:

$$\left(1 - \frac{\Delta_A}{\Delta_B}\right) \geq 0.8 \tag{12.1}$$

where

Δ_A = lateral deflection calculated from the truss frame (Frame A) alone

Δ_B = lateral deflection calculated from Frame B alone

Alternatively, the lateral stiffness of Frame A under the applied lateral load should be at least five times larger than that of Frame B:

$$K_A \geq 5K_B \tag{12.2}$$

where

K_A = lateral stiffness of Frame A

K_B = lateral stiffness of Frame B

12.1.5 Sway Frames vs. Non-Sway Frames

The identification of sway frames and non-sway frames in a building is useful for evaluating safety of structures against instability. In the design of multi-story 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 that 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 behavior. This gives the commonly assumed *non-sway frame*. The design of columns in non-sway frames follows the conventional beam-column capacity check approach, and the column effective length may be evaluated based on the column end restraint conditions. Interaction equations for various cross-section shapes have been developed through years of research spent in the field of beam-column design [12].

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 subassemblage or the structure as a whole. Moreover, the presence of “inelasticity” in the columns will render some doubts on the use of the familiar concept of “elastic effective length” [45, 46].

On the basis of the above considerations, a definition can be established for sway and non-sway frames as:

> *A frame can be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.*

British Code: BS5950:Part 1 [11] 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 vertical imposed loads and are applied in isolation, i.e., without the simultaneous application of actual vertical or horizontal loading.
2. Carry out a first-order linear elastic analysis and evaluate the individual relative sway deflection δ for each story.
3. If the actual frame is unclad, the frame may be considered to be non-sway if the inter-story deflection of every story satisfies the following limit:

   $$\delta < \frac{h}{4000}$$

   where h = story height.
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

   $$\delta < \frac{h}{2000}$$

   where h = story height.
5. All frames not complying with the criteria in (3) or (4) are considered to be sway frames.

Eurocode 3 [22] 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 the elastic buckling load ratio P_{cr}/P for that load case satisfies the criterion:

$$P_{cr}/P \geq 10$$

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 is 10 times the design load, the frame is said to be stiff enough to resist lateral load, and it is unlikely to be sensitive to sidesway deflections. AISC LRFD [3] does not give specific guidance on frame classification. However, for frames to be classified as non-sway in AISC LRFD format, the moment amplification factor, B_2, has to be small (a possible range is $B_2 < 1.10$) so that sway deflection would have negligible influence on the final value obtained from the beam-column capacity check.

12.1.6 Classification of Tall Building Frames

A tall building is defined uniquely as a building whose structure creates different conditions in its design, construction, and use than those for common buildings. From the structural engineer's view point, the selection of appropriate structural systems for tall buildings must satisfy two important criteria: strength and stiffness. 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 terms of its ability to resist higher lateral loads which increase with the height of the frame [30]. A building can be considered as tall when the effect of lateral loads is reflected in the design. Lateral deflections of tall buildings 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 minimize discomfort to the occupants (see Section 12.4).

Figure 12.5 shows a chart that defines, in general, the limits to which a particular system can be used efficiently for multi-story building projects. The various structural systems in Figure 12.5 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. At one end of the spectrum in Figure 12.5 is the moment resisting frames, which are efficient for buildings of 20 to 30 stories, and at the other end is the tubular systems with high cantilever efficiency. Other systems were placed with the idea that the application of any particular form is economical only over a limited range of building heights.

An attempt has been made to develop a rigorous methodology for the cataloging of tall buildings with respect to their structural systems [16]. The classification scheme involves four levels of framing division: (1) primary framing system, (2) bracing subsystem, (3) floor framing, and (4) configuration and load transfer. While any cataloging scheme must address the pre-eminent focus on lateral load resistance, the load-carrying function of the tall building subsystems is rarely independent. An efficient high-rise system must engage vertical gravity load resisting elements in the lateral load subsystem in order to reduce the overall structural premium for resisting lateral loads. Further readings on design concepts and structural schemes for steel multi-story buildings can be found in Liew [41], and the design calculations and procedures for building frame structures using the AISC LRFD procedure are given in Liew and Chen [44].

Some degree of independence can be distinguished between the floor framing systems and the lateral load resisting systems, but the integration of these subassemblies into the overall structural scheme is crucial. Section 12.2 provides some advice for selecting composite floor systems to achieve the required stiffness and strength, and also highlights the ways where building services can be accommodated within normal floor zones. Several practical options for long-span construction are discussed, and their advantages and limitations are compared and contrasted. Design considerations for floor diaphragms are discussed. Section 12.3 provides some advice on the general principles to be applied when preparing a structural scheme for multistory steel and composite frames. The design procedure and construction considerations that are specific to steel gravity frames, braced frames, moment resisting frames, and the design approaches to be adopted for sizing multistory building frames are given. The potential use of steel-concrete composite material for high-rise construction is presented. Section 12.4 deals with the issues related to wind-induced effects on multistory frames. Dynamic effects due to along wind, across wind, and torsional response are considered with examples.

12.2 Composite Floor Systems

12.2.1 Floor Structures in Multistory Buildings

Tall building floor structures generally do not differ substantially from those in low-rise buildings; however, there are certain aspects and properties that need to be considered in design:

1. Floor weight to be minimized.

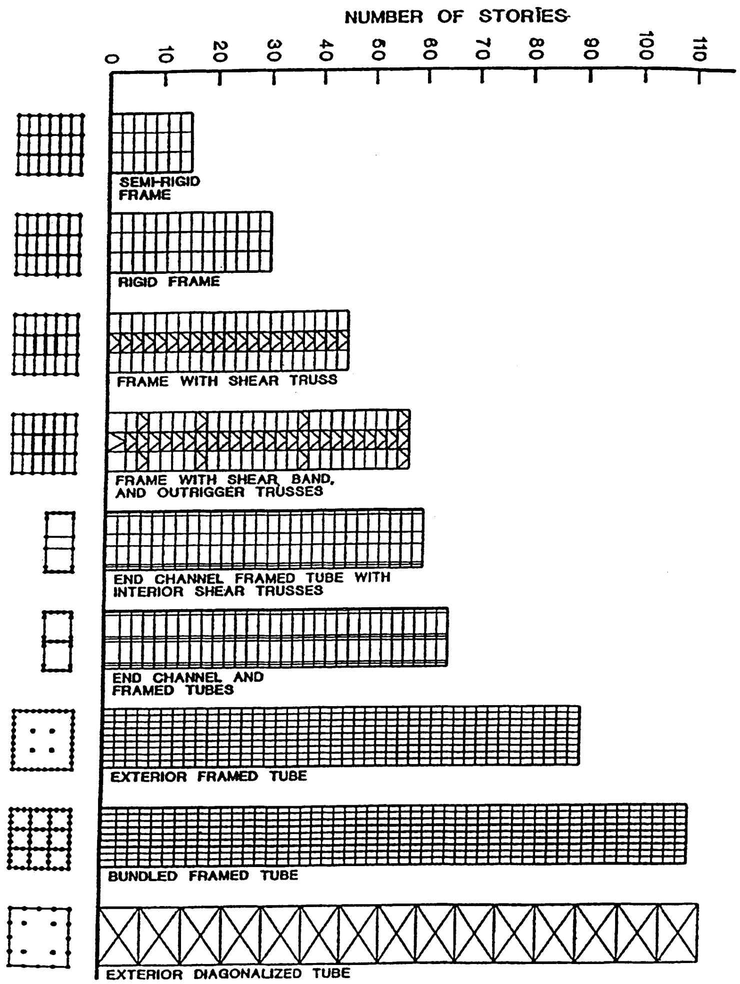


FIGURE 12.5: Various structural schemes.

2. Floor should be able to resist construction loads during the erection process.
3. Integration of mechanical services (such as ducts and pipes) in the floor zone.
4. Fire resistance of the floor system.
5. Buildability of structures.
6. Long spanning capability.

Modern office buildings require large floor spans in order to create greater space flexibility for the accommodation of a greater variety of tenant floor plans. For tall 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 by using prefabricated or precasted elements of steel and reinforced concrete in various combinations. Floor slabs can be precasted concrete slab, *in situ* concrete slab, or composite slabs with metal decking. Typical precast slabs are 4 to 7 m, thus avoiding the need of secondary beams. For composite slabs, metal deck spans ranging from 2 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 shoring is provided. Shoring is best avoided as the speed of construction is otherwise diminished for the construction of tall buildings.

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 and tele and electronic communication), ducts (air-conditioning), etc.

In addition to strength, floor spanning systems must provide adequate stiffness to avoid large deflections due to live load 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. In steel construction, steel members can be partially or fully encased in concrete for fire protection. For longer periods of fire resistance, additional reinforcement bars may be required.

12.2.2 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:

1. Steel and concrete may be arranged to produce ideal combinations of strength, with concrete efficient in compression and steel in tension.
2. Composite system is lighter in weight (about 20 to 40% lighter than concrete construction). This leads to savings in the foundation cost. Because of its light weight, site erection and installation are easier and thus labor cost can be minimized. Foundation cost can also be reduced.
3. The construction time is reduced because casting of additional floors may proceed without having to wait for the previously casted floors to gain strength. The steel decking system provides positive-moment reinforcement for the composite floor and requires only small amounts of reinforcement to control cracking and for fire resistance.
4. The construction of composite floors does not require highly skilled labor. The steel decking acts as a permanent formwork. Composite beams and slabs can accommodate raceways for electrification, communication, and an air distribution system. The slab serves as a ceiling surface to provide easy attachment of a suspended ceiling.
5. The composite slabs, when they are fixed in place, can act as an effective in-plane diaphragm that may provide effective lateral bracing to beams.
6. Concrete provides corrosion and thermal protection to steel at elevated temperatures. Composite slabs of 2-h fire rating can be achieved easily for most building requirements. Composite floor systems are advantageous because of the formation of the floor slab. The floor slab can be formed by the following methods:

   (a) a flat-soffit reinforced concrete slab (Figure 12.6a)

   (b) precast concrete planks with cast *in situ* concrete topping (Figure 12.6b)

   (c) precast concrete slab with *in situ* grouting at the joints (Figure 12.6c)

   (d) a metal steel deck, either composite or non-composite (Figure 12.6d)

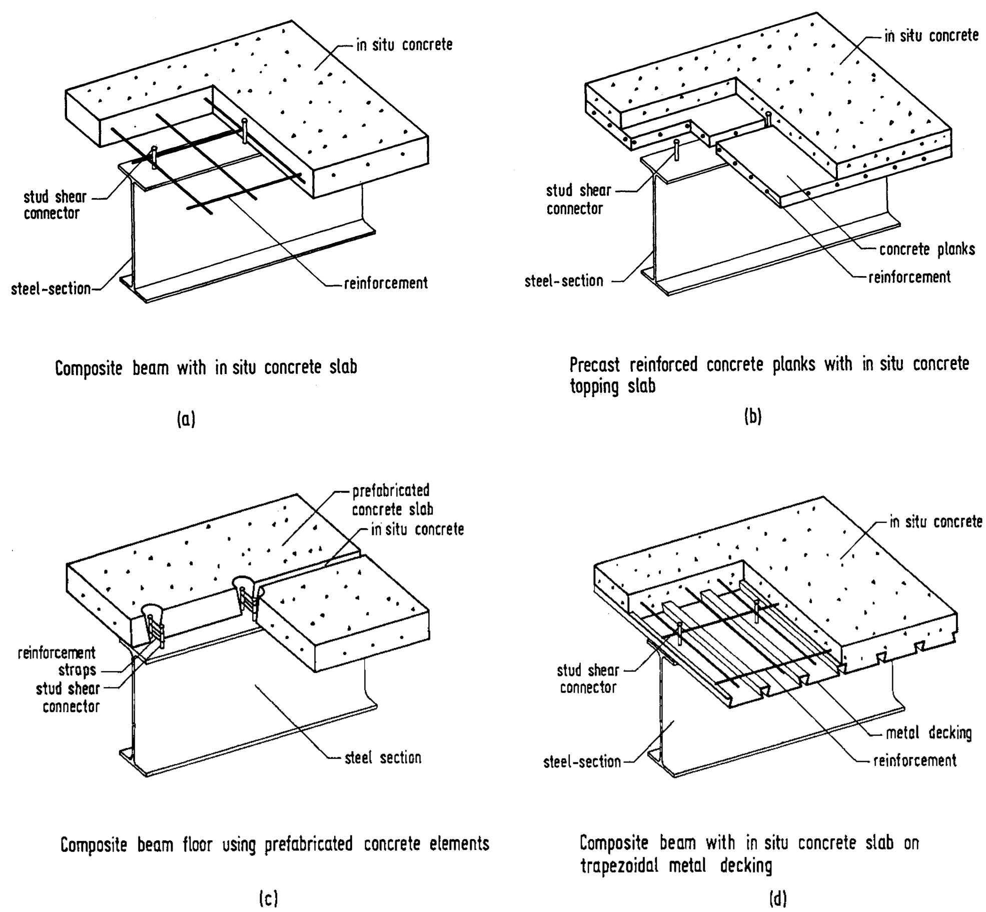


FIGURE 12.6: Composite beams with (a) flat-soffit reinforced concrete slab, (b) precast concrete planks and cast *in situ* concrete topping, (c) precast concrete slab and *in situ* concrete at the joints, and (d) metal steel deck supporting concrete slab.

The composite action of the beam or truss is due to shear studs welded directly through the metal deck, whereas the composite action of the metal deck results from side embossments incorporated into the steel sheet profile. The slab and beam arrangement typical in composite floor systems produces a rigid horizontal diaphragm, providing stability to the overall building system while distributing wind and seismic shears to the lateral load resisting systems.

12.2.3 Composite Beams and Girders

Steel and concrete composite beams may be formed by completely encasing a steel member in concrete with the composite action depending on the 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, instead of using concrete encasement, 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 (Figure 12.6d). The metal deck typically spans unsupported between steel members while also providing a working platform for concreting work. The metal decks may be oriented parallel or perpendicular to the composite beam span.

Figure 12.7a shows a typical building floor plan using composite steel beams. The stress distribution at working loads in a composite section is shown schematically in Figure 12.7b. The neutral axis is normally located very near to the top flange of the steel section. Therefore, the top flange is lightly stressed. Built-up beams or hybrid composite beams can be good choices in an attempt to use the structural steel material more efficiently (see Section 12.2.4). Also, composite beams of tapered

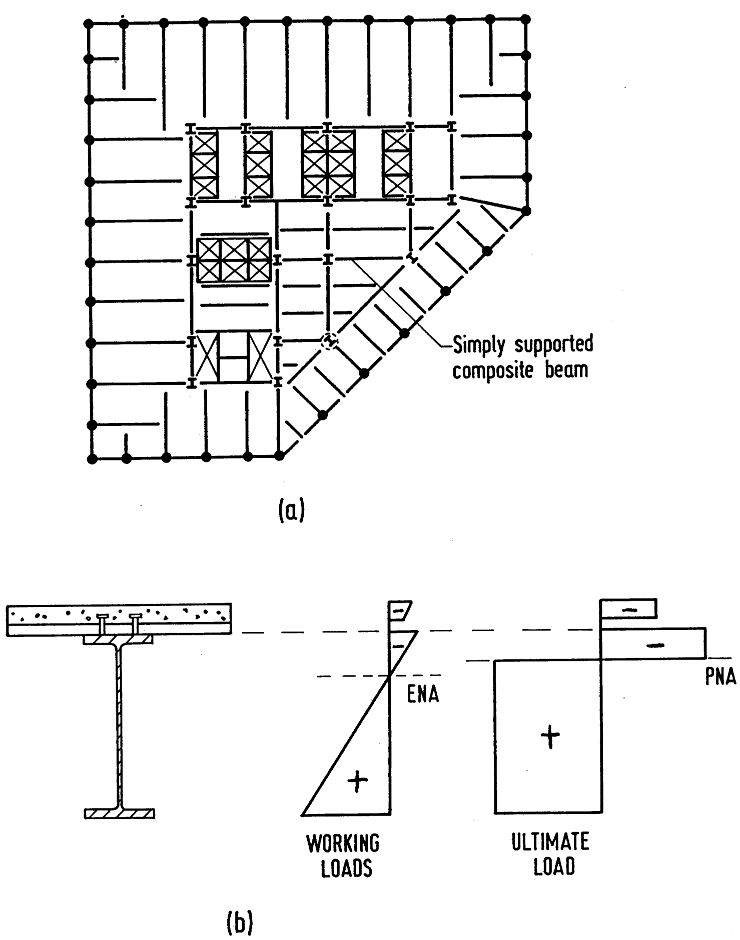


FIGURE 12.7: (a) Composite floor plan and (b) stress distribution in a composite cross-section.

flanges are possible. For a construction point of view, a relatively wide and thick top flange must be provided for proper installation of shear stud and metal decking. However, in all of these cases, the increased fabrication costs must be evaluated, which tend to offset the saving from material efficiency.

A prismatic composite steel beam has two fundamental disadvantages over other types of composite floor framing types.

1. The member must be designed for the maximum bending moment near midspan and thus is often under stressed near the supports.
2. Building-services ductwork and piping must pass beneath the beam, or the beam must be provided with web openings (normally reinforced with plates or angles leading to higher fabrication costs) to allow access for this equipment as shown in Figure 12.8.

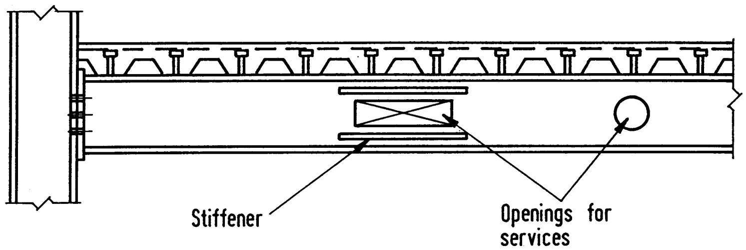


FIGURE 12.8: Web opening with horizontal reinforcements.

For this reason, a number of composite girder forms allowing the free passage of mechanical ducts and related services through the depth of the girder have been developed. 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 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 beams more than 12 m in range. Issues related to serviceability problems at various deflection or drift indices are discussed in Section 12.4.

12.2.4 Long-Span Flooring Systems

Long spans impose a burden on the beam design in terms of larger required flexural stiffness for limit-state designs. Besides satisfying both 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.

Beams With Web Openings

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 (Figure 12.9). Castellated composite beams can be used effectively for lightly serviced buildings. 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 are best used as long span secondary beams where loads are low or where concentrated loads can be avoided. Their 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.

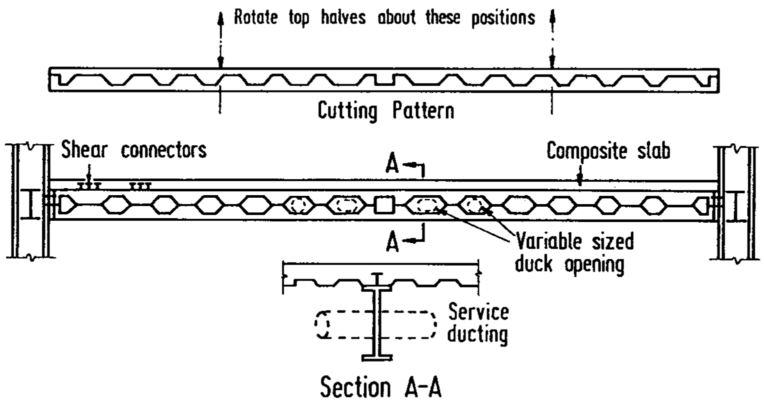


FIGURE 12.9: Composite castellated beams.

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, i.e., where the bending moment is high. This is because the webs do not contribute much to the moment resistance of the beam.

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 (Figure 12.10), whereas a rigidly connected beam would have a 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 modest size (Figure 12.10).

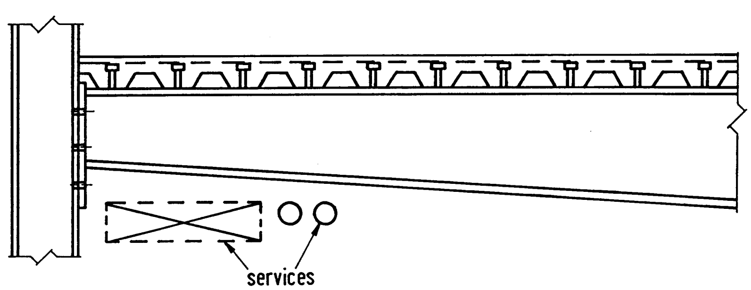


FIGURE 12.10: Tapered composite beams.

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 6°. 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 of 13 to 20 m. Further information on the design of fabricated beams with tapered webs can be found in Owens [51].

Haunched Beams

The span length of a composite beam can be increased by providing haunches or local stiffening of the beam-to-column connections as shown in Figure 12.11. Haunched beams are designed by

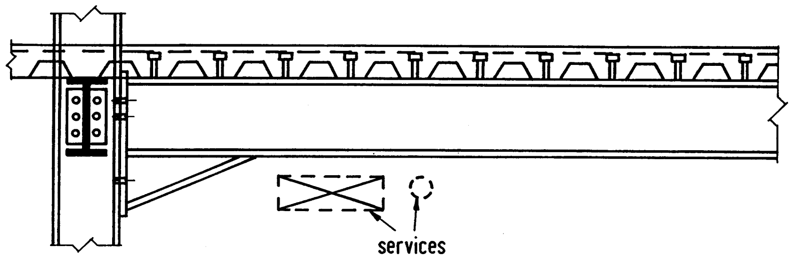


FIGURE 12.11: Haunched composite beam.

forming a rigid moment connection between the beams and columns. The haunch connections offer restraints to the 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, the 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. The length of haunch is typically 5 to 7% the span length for non-sway frames or 7 to 15% for sway frames. Service ducts can pass below the beams as in conventional construction (Figure 12.11).

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 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 a structure design 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 do offer higher strength and stiffness during the steel erection stage thus making this type of system particularly 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.

The rationale of using the haunched beam approach is explained as follows. In continuous beam design, the moment distribution of a continuous beam would show that the support moment is generally larger than the mid-span moment up to the ratio of 1.8 times. The effective cross-sections of typical steel-concrete composite beam under hogging and sagging moment can be determined according to the usual stress block method of design. It can be observed that the hogging moment capacity of the composite section at the support is smaller than the sagging moment capacity near the mid-span. Therefore, there is a mismatch between the required greater support resistance and the much larger available sagging moment capacity.

When elastic analysis is used in the design of continuous composite beams, the potential large sagging moment capacities available from composite action can never be realized. One way to overcome this problem is to increase the moment resistance at the support (and hence utilize the full potential of larger sagging moment) by providing haunches at the supports. An optimum design can be achieved by designing the haunched section to develop the required moment at the support and the composite section to develop the required sagging moment. If this can be achieved in practice, the design does not require inelastic force redistribution and hence elastic analysis is adequate. However, analysis of haunched composite beams is more complicated because the member is non-prismatic (i.e., cross-section property varies along the length). The analysis of such beams requires the evaluation of section properties such as the beam's stiffness (EI) at different cross-sections. The analysis/design process is more involved because it requires the evaluation of serviceability deflection and ultimate strength limit state of non-prismatic members. Some guides on haunched beam design can be found in Lawson and Rackham [36].

Parallel Beam System

The system consists of two main beams with secondary beams run over the top of the main beams (see Figure 12.12). The main beams are connected to either side of the column. They can

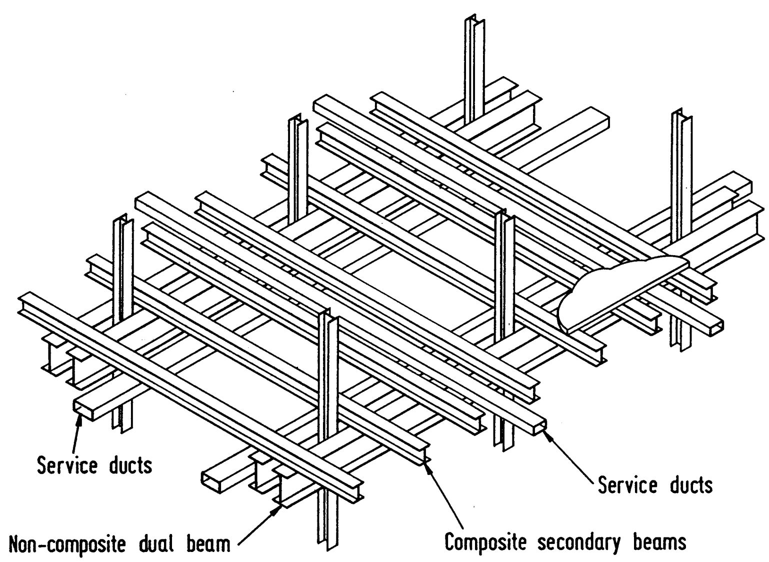


FIGURE 12.12: Parallel composite beam system.

be made continuous over two or more spans supported 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 (Figure 12.12). Small savings in steel weight are 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 connecting to the concrete slab through the use of shear studs (see the stud-girder system in Section 12.2.4).

The simplicity of connections and ease of fabrication make this long-span beam option particularly attractive. Competitive pricing can be obtained from the fabricator. Further details on the parallel beam approach can be found in Brett and Rushton [10].

Composite Trusses

Trusses are frequently used in multistory buildings for very long span supports. 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 spans when compared to other structural schemes. An additional disadvantage other than fabrication cost is that truss configuration creates difficulty for fire protection. Fire protection wrapping is labor intensive and sprayed-protection systems cause a substantial mess to the services that pass through the web opening (see Figure 12.13). From a structural point of view, the benefit of using a composite truss is due to the increase in stiffness rather than strength.

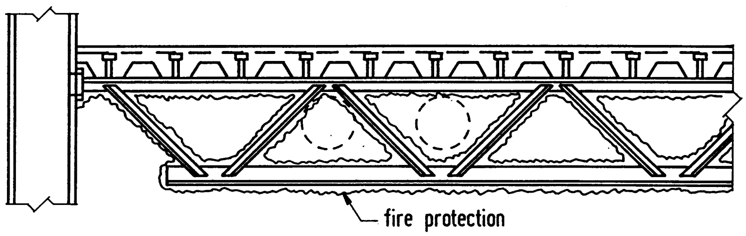


FIGURE 12.13: Composite truss.

Several forms of truss arrangement are possible. The three most common web framing configurations in floor truss and joist designs are: (1) Warren Truss, (2) Modified Warren Truss, and (3) Pratt Truss as shown in Figure 12.14. 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 a more efficient use of these members.

Experience has shown that both Pratt and Warren configurations of web framing are suitable for short span trusses with shallow depths. For truss with spans greater than 10 m, or effective depths larger than 700 mm, a modified Warren configuration is generally preferred. The Warren and modified Warren trusses are more popular for building construction since they offer larger web openings for services between bracing 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 the bottom chord is the preferred failure mechanism. Thus, the bottom chord should be designed to yield prior to crushing of the 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 composite action is considered, the top steel chord is assumed not to participate in the moment resistance of the truss because it is located very near to the neutral axis of the composite truss and, thus, contributes very little to the flexural capacity. However, the top chord has two functions: (1) it provides an attachment surface for the shear connectors, and (2) it resists the forces in the end panel without reliance on composite action unless shear connectors are placed over the

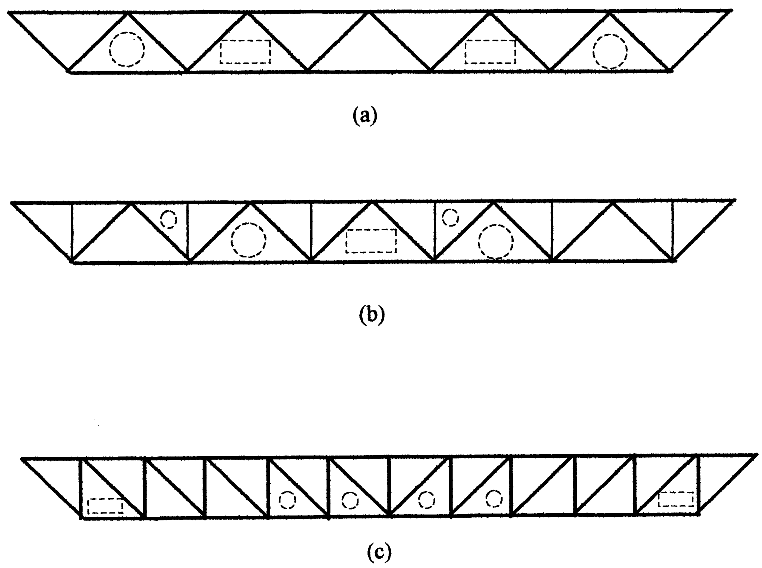


FIGURE 12.14: Truss configuration: (a) Warren truss, (b) Modified Warren truss, and (c) Pratt truss.

seat or along a top chord extension. Thus, the top chord must be designed to resist the compressive force equilibrating the horizontal force component of the first web member. In addition, the top chord also transfers the factored shear force to the support, and must be designed accordingly.

The bottom chord shall be continuous and may be designed as an axially loaded tension member. The bottom chord shall be proportioned to yield before the concrete slab, web members, or the shear connectors fail.

The shear capacity of the steel top and bottom chords and concrete slab can be ignored in the evaluation of the shear resistance of a composite truss. The web members should be designed to resist vertical shear. Further references on composite trusses can be found in ASCE Task Committee [7] and Neals and Johnson [50].

Stub Girder System

The stub girder system involves the use of short beam stubs that 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 (Figure 12.15), permitting story-height reduction when compared with some other structural framing systems.

Ideally, stub-girders span about 12 to 15 m (usually from the center core wall to the exterior columns in a conventional office building) with the secondary framing or floor beams spanning about 6 to 9 m. The system is 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 remain until the concrete has gained adequate strength for composite action. However, it is possible to introduce an additional steel top chord, such as a T-section, which acts in compression

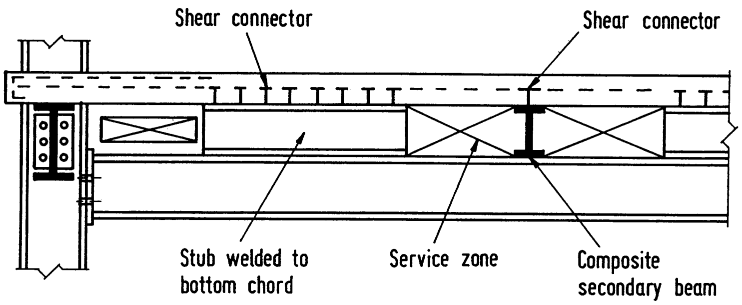


FIGURE 12.15: Stub girder system.

to develop the required bending strength during construction. For span length greater than 15 m, stub-girders become impractical because the slab design becomes 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 analyzed 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.

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 utilized fully in terms of stress in the tension zone of the girder.

Prestressing of a steel beam can be carried out using a precambering technique as depicted in Figure 12.16. First a steel girder member is prebent (Figure 12.16a), and is then subjected to preloading in the direction against the bending curvature until the required steel strength is reached (Figure 12.16b). Second, the lower flange of the steel member, which is under tension, is encased in a reinforced concrete chord (Figure 12.16c). 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 (Figure 12.16d). 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 (Figure 12.16e).

The major advantages of this system is that the steel girders are encased in concrete on all sides and 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 a formwork. If the distance between two girders is large, precast planks can be supported by the lower concrete chord as permanent formwork.

Prestressing can also be achieved by using tendons that 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 (Figure 12.17). This technique has been found to be popular for bridge construction in Europe and the U.S., although it is less common for building construction.

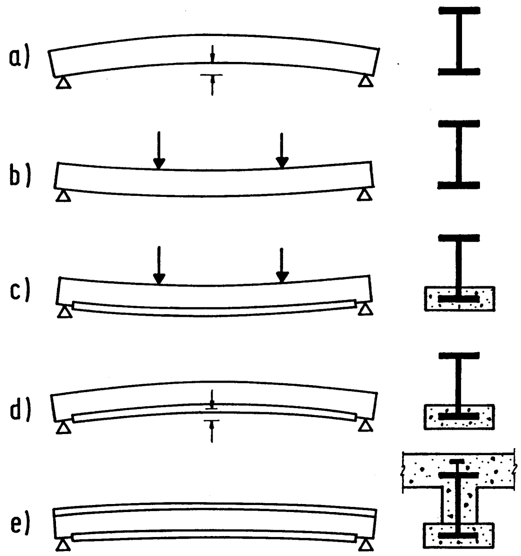


FIGURE 12.16: Process of prestressing using precambering technique.

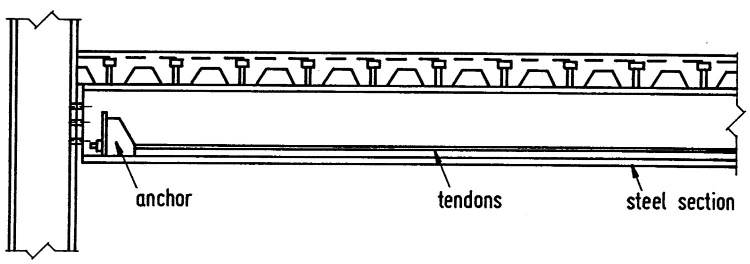


FIGURE 12.17: Prestressing of composite steel girders with tendons.

12.2.5 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 unnecessarily increasing 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 that 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 12.18 compares the various structural options of a typical range of span lengths used in practice.

12.2.6 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

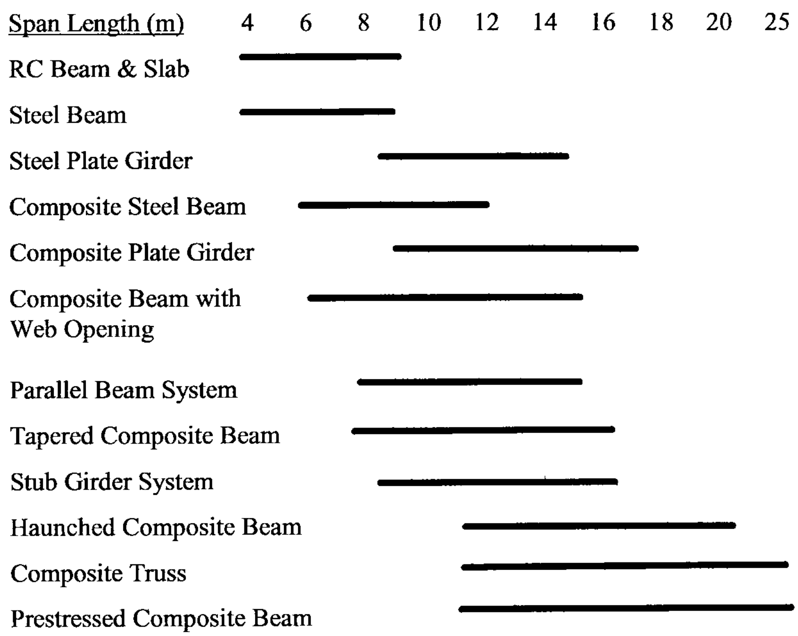


FIGURE 12.18: Comparison of composite floor systems.

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 which, for building structures, are normally assumed to have adequate in-plane stiffness. 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 a building relying on bracing systems to resist all lateral load, the stability of the building depends on a 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 is often through shear studs that are welded directly through the metal deck to the beam flange. The connection between seams of adjacent deck panels is crucial and often through interlocking of panels overlapping each other. The diaphragm stresses are generally low and can be resisted by floor slabs that have adequate thickness for most buildings. Plan bracing is necessary when diaphragm action is not adequate. Figure 12.19a shows a triangulated plan bracing system that resists lateral load on one side and spans between the vertical walls. Figure 12.19b illustrates 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 a lateral bracing system at a 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.

Diaphragms may be classified into three types, namely (1) flexible diaphragm, (2) semi-rigid diaphragm, and (3) rigid diaphragm. Common types of floor diaphragms that can be classified as rigid are (1) reinforced concrete slab, (2) composite slab with reinforced concrete slab supported by metal decking, and (3) precasted concrete slabs that are properly attached to one another. Floors

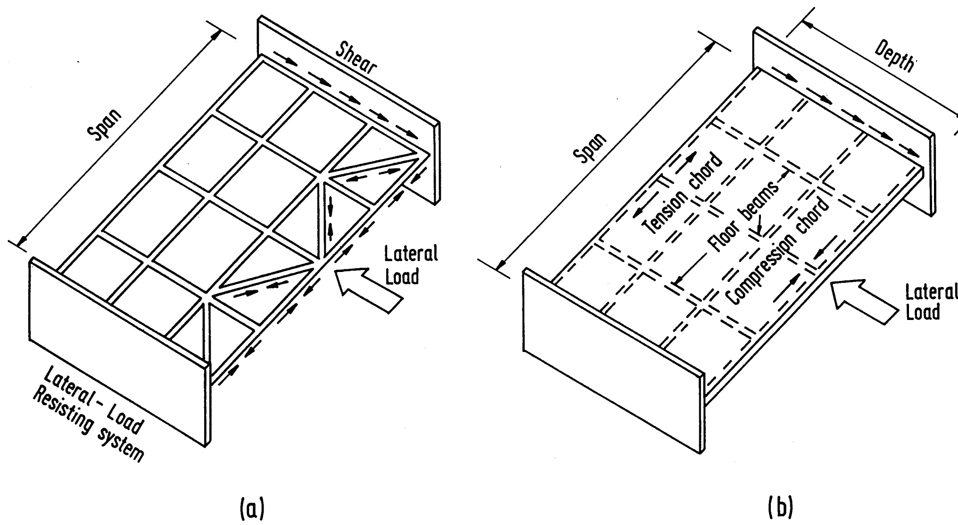


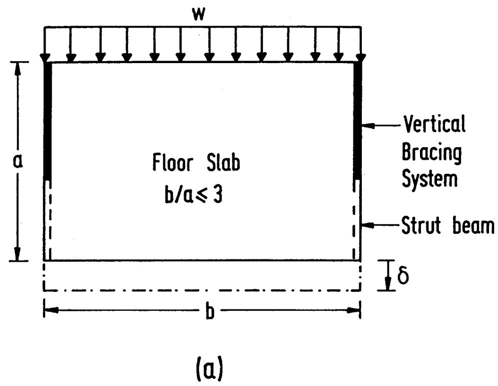
FIGURE 12.19: (a) Triangulated plan bracing system and (b) concrete floor diaphragm.

that are classified as semi-rigid or flexible are steel deck without concrete fill or deck that is partially filled with concrete. However, the rigidity of a floor system must be comparable to the stiffness of the lateral-load resistance system. A rigid diaphragm will distribute lateral forces to the lateral-load resisting elements in proportion to their relative rigidities. Therefore, a vertical bracing system with high lateral stiffness will resist a greater proportion of the lateral force than a system with lower lateral stiffness. A flexible diaphragm behaves more like a beam spanning between the lateral-load resistance elements. It distributes lateral forces to the lateral systems on a tributary load basis, and it cannot resist any torsional forces. Semi-rigid diaphragms deflect like a beam under load, but possess some stiffness to distribute the loads to the lateral-load resistance systems in proportion to their rigidities. The load distribution process is a function of the floor stiffness and the vertical bracing stiffness.

The rigid diaphragm assumption is generally valid for most high-rise buildings (Figure 12.20a); however, as the plan aspect ratio (b/a) of the diaphragm linking two lateral systems exceeds 3 in 1 (see the illustration in Figure 12.20b), 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 Figure 12.19b. 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 12.21a 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 story. Figure 12.21b and 12.21c show the building plan having dimensions of L_1 and L_2. The lateral load resisting systems are represented in the 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 (F_e), and the diaphragm tends to rotate as shown by the dashed lines in Figure 12.21a. The lateral load resistance systems in Wall B and Wall C will provide the resistance forces to stabilize the torsional force by



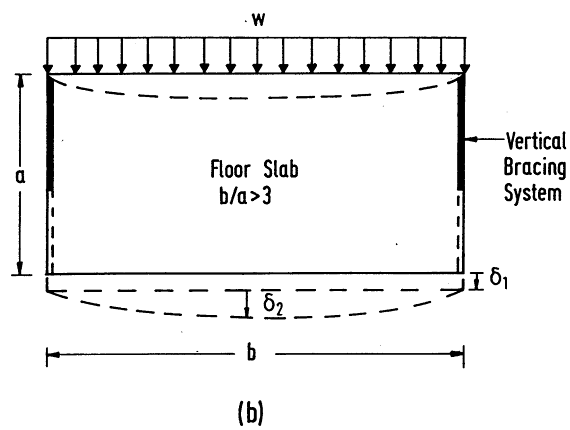


FIGURE 12.20: Diaphragm rigidity: (a) plan aspect ratio ≤ 3 and (b) plan aspect ratio > 3.

generating a couple of shear resistances as:

$$V_B = V_C = \frac{Fe}{L_2}$$

Figure 12.21c 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 eliminate the possibility of any torional effects.

The adequacy of the floor to act as a diaphragm depends very much on its type. Pre-cast concrete floor planks without any prestressing offer limited resistance to the racking effects of diaphragm action. In such cases, supplementary bracing systems in the plan, such as those shown in Figure 12.19a, 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

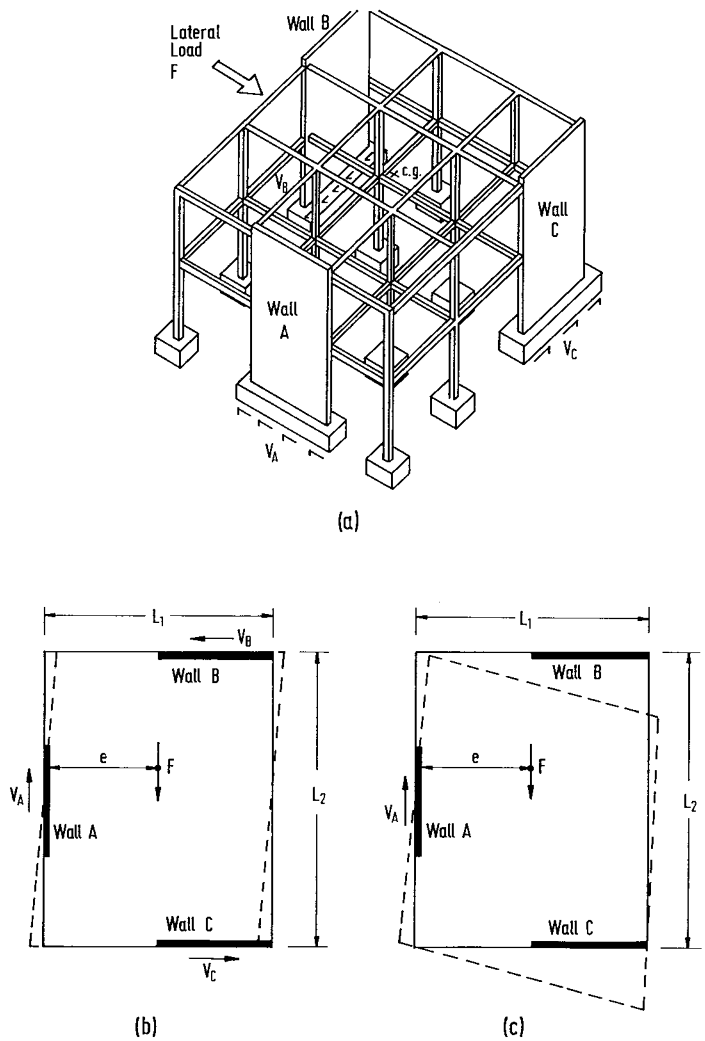


FIGURE 12.21: (a) Lateral force resisting systems in a building, (b) rigid diaphragm, and (c) flexible diaphragm.

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 because these have a significant effect on the design of the structure. Table 12.1 summarizes the salient features of the various types of flooring systems in terms of their diaphragm actions.

Floor diaphragms may also be designed to provide lateral restraint to columns of multi-story buildings. In such cases, the shear required to be resisted by the floor diaphragm can be computed from the second-order forces caused by the vertical load acting on the story deflection of the column at the floor level under consideration. The stability force for the column may be transmitted directly to the deck-slab through bearing and gradually transferred into the floor framing connections through shear studs.

If metal decking is used, the metal deck provides column stability during erection, prior to concrete slab placement. Column loads are much lower during construction; hence, this condition may not be too critical. Special precaution must be given to limit the number of stories of steel erected ahead of the concrete floor construction. Overall building stability becomes important, possibly requiring the steel deck diaphragm to be supplemented with a concrete cover slab at various height levels in the structure.

TABLE 12.1 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 beams	Degree of diaphragm action	Usage
In situ concrete	3–6	150–250	Medium	Very good	Very good	All categories but not often used in multistory buildings
Steel deck with *in situ* concrete	2.5–3.6 unshore > 3.6 shore	110–150	Fast	Very good	Very good	All categories especially in multistory office buildings
Pre-cast 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	Multistory buildings and bridges

12.3 Design Concepts and Structural Schemes

12.3.1 Introduction

Multistory steel frames consist of a column and a beam interconnected to form a three-dimensional structure. A building frame can be stabilized either by some form of bracing system (braced frames) or can be stabilized 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 system attached to it. This type of framing system, which is generally referred to as simple braced frames, is found to be cost-effective for multistory buildings of moderate height (up to 20 stories).

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 form of bracing system. 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 system. However, moment joints are rather costly to fabricate. In addition, it takes a longer time to erect a moment frame than a gravity frame.

A cost-effective framing system for multistory 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 multistory structure is most economical and efficient 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 procedures 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. Simple structural schemes are quick to design and easy to erect. It also provides a good "benchmark" for further refinement. Many building structures have to accommodate extensive services within the floor zone. It is important that the engineer chooses a structural scheme (see Section 12.2) which can accommodate the service requirements within the restricted floor zone to minimize overall cost.

Scheme drawings for multistory building designs should include the following:

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.
5. Proposals for fire and corrosion protection.

This section offers advice on the general principles to be applied when preparing a structural scheme for multistory 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 tall 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, and future directions for research are highlighted.

12.3.2 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 a simply supported member.
3. Columns must be fully continuous. The columns are designed to carry axial loads only. Some codes of practice (e.g., [11]) 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.

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 story. 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 to 3.6 m; the span of the secondary beams to 6 to 12 m; and the span of the primary beams to 5 to 7 m. Otherwise, choose a precast or an *in situ* reinforced concrete floor, limiting its span to 5 to 6 m, and the span of the beams to 6 to 8 m approximately.

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. Orthogonal arrangements of beams and columns, as opposed to skewed arrangements, provide maximum repetition of standard details. In addition, greater economies can be achieved when the column grids in the plan are rectangular in which the secondary beams should span in the longer direction and the primary beams in the shorter, as shown in Figures 12.22a and b. This arrangement reduces the number of beam-to-beam connections and the number of individual members per unit area of the supported floor [52].

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 composite beams. The design of the 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 multistory 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 (see Section 12.2.5).

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 subjected to any direct moment transferred through the connection (nominal moments due to eccentricity of the beam reaction forces may 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 sidesway must be considered in the design of the frames that are participating in sidesway 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 straightforward

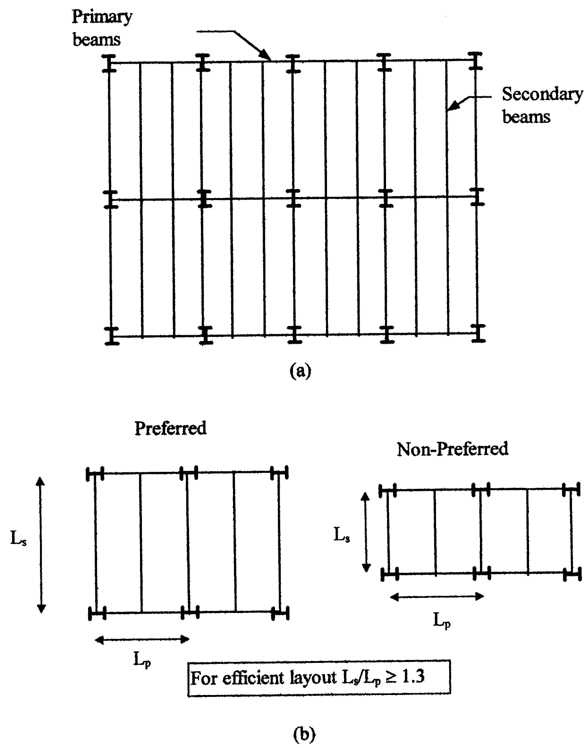


FIGURE 12.22: (a) Retangular grid layout and (b) preferred and non-preferred grid layout.

task. Beam or girder members supporting more than 40 m^2 of floor at one story should be designed for a reduced live load in accordance with ASCE [6].

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 12.2.

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 that controls the design will depend on the span-to-depth ratio of the beam. Figure 12.18 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 Figure 12.18, 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 multistory frame [6]. If the frame is braced against sidesway, the column node points are prevented from lateral translation. A conservative estimate of column effective length, KL, for buckling considerations is $1.0L$, where L is the story height. However, in cases where the columns above and below the story 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

TABLE 12.2 Recommended Deflection Limits for Steel Building Frames

Beam deflections from unfactored imposed loads	
Beams carrying plaster or brittle finish	Span/360 (with maximum of 1/4 to 1 in.)
Other beams	Span/240
Columns deflections from unfactored imposed and wind loads	
Column in single story frames	Height/300
Column in multistory frames	Height of story/300
For column supporting cladding which is sensitive to large movement	Height of story/500
Frame drift under 50 years wind load	
Frame drift	Frame height/450 $\sim$ frame height/600

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.

An example of such cases is the continuous column shown in Figure 12.23 in which Column AB is longer than Column BC and hence Column AB is restrained by Column BC at the restraint point B. A buckling analysis shows that the critical buckling load for the continuous column is $P_{cr} = 5.89EI/L^2$, which gives rise to an effective length factor of $K = 0.862$ for Column AB and $K = 1.294$ for Column BC. Column BC has a larger effective length factor because it provides restraint to Column AB, whereas Column AB has a smaller effective length factor because it is restrained by Column BC during buckling. Figure 12.24 summaries the reductions in effective length which may be considered for columns in a frame with different story heights having various values of a/L ratios [52].

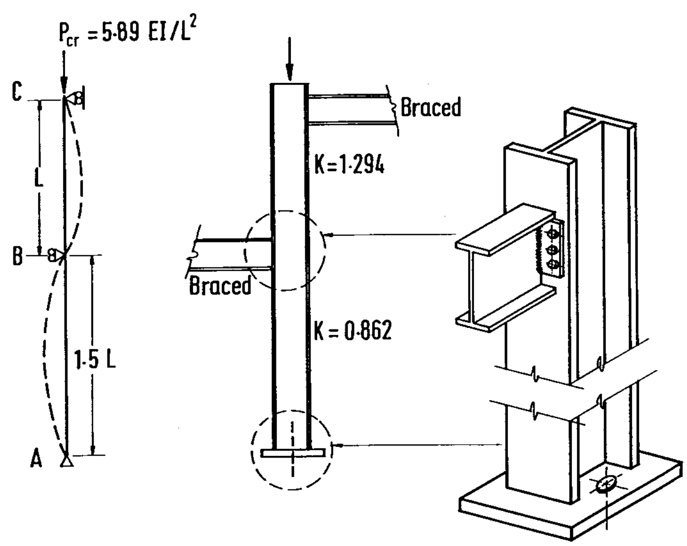


FIGURE 12.23: Buckling of a continuous column with intermediate restrain.

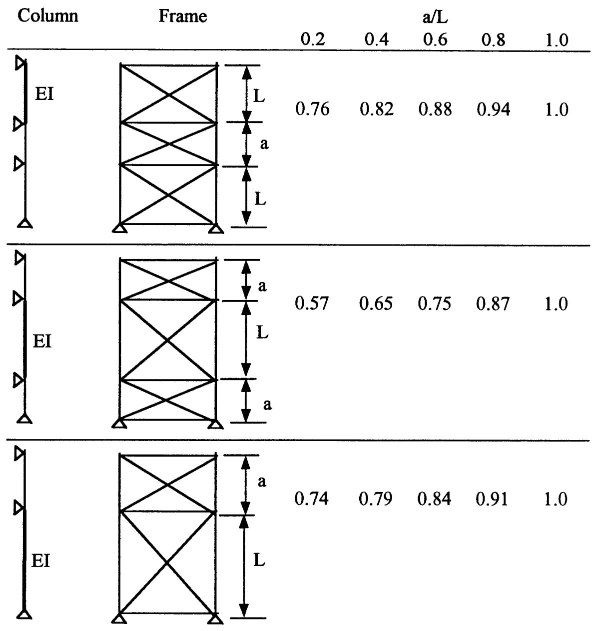


FIGURE 12.24: Effective length factors of continuous braced columns.

Simple Shear Connections

Simple shear connections should be designed and detailed to allow free rotation and to prevent excessive transfer of moment between the beams and columns. Such connections should comply with the classification requirement for a "nominally pinned connection" in terms of both strength and stiffness. A computer program for connection classification has been made available in a book by Chen et al. [15], and their design implications for semi-rigid frames are discussed in Liew et al. [47].

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 a 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 12.25 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 [2].

12.3.3 Bracing Systems

The main purpose of a bracing system is to provide the lateral stability to the entire structure. It has to be designed 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

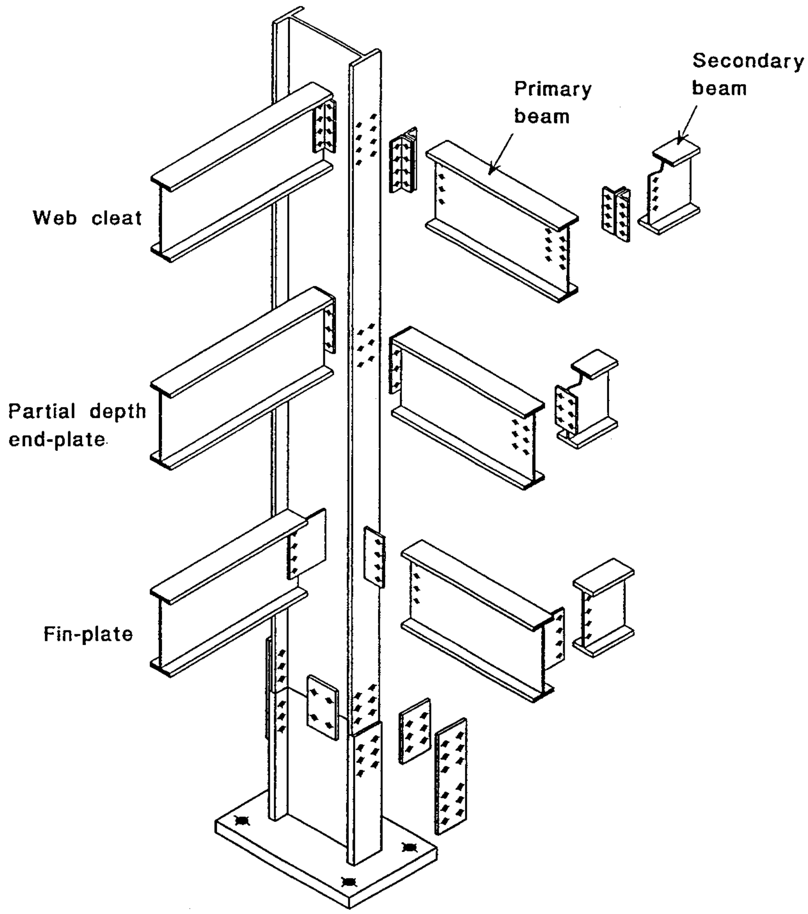


FIGURE 12.25: Typical beam-to-column connections to be considered as shear connections.

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.

Structural Forms

Steel braced systems are often in a form of a 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 12.26. The design of such structures must take into account the manner in which the frames are erected, the distribution of lateral forces, and their sidesway resistance.

In the single braced forms, where a single diagonal brace is used (Figure 12.26a), 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 (L/r) not be greater than 200 to prevent the self-weight deflection of the brace limiting its compressive resistance.

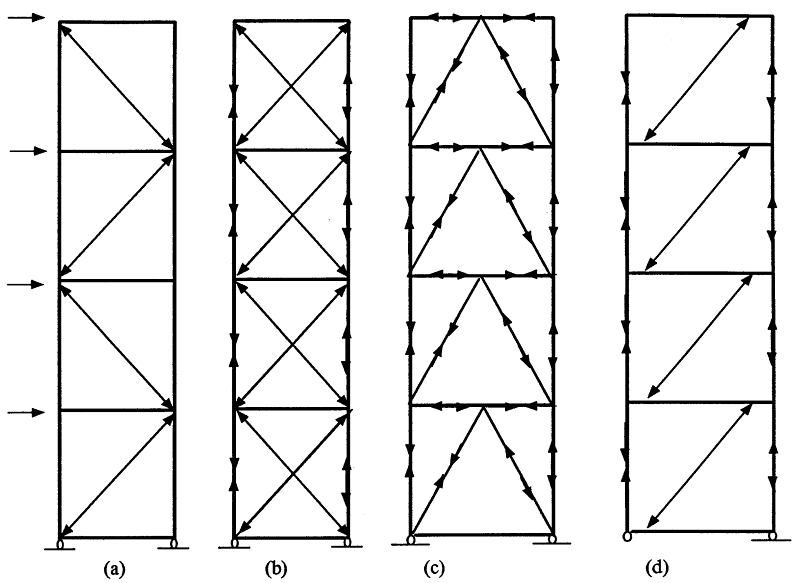


FIGURE 12.26: (a) Diagonal bracing, (b) cross-bracing, (c) K-bracing, and (d) eccentric bracing.

In a cross-braced system (Figure 12.26b), 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 overall structural response. 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.

The K trusses are common since 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. A K-braced frame is more efficient in preventing sidesway than a 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. A K-braced frame is found to be more efficient if the apexes of all the braces are pointing in the upward direction (Figure 12.26c).

For eccentrically braced frames, the center line of the brace is positioned eccentrically to the beam-column joint, as shown in Figure 12.26d. 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.

Drift Assessment

The story drift Δ of a single story diagonally braced frame, as shown in Figure 12.27, can be approximated by the following equation:

$$\begin{aligned} \Delta &= \Delta_s + \Delta_f \\ &= \frac{HL_d^3}{A_d E L^2} + \frac{Hh^3}{A_c E L^2} \end{aligned} \tag{12.3}$$

where

Δ	=	inter-story drift
Δ_s	=	story drift due to shear component
Δ_f	=	story 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 story
h	=	story height
L	=	length of braced bay
L_d	=	length of the diagonal brace

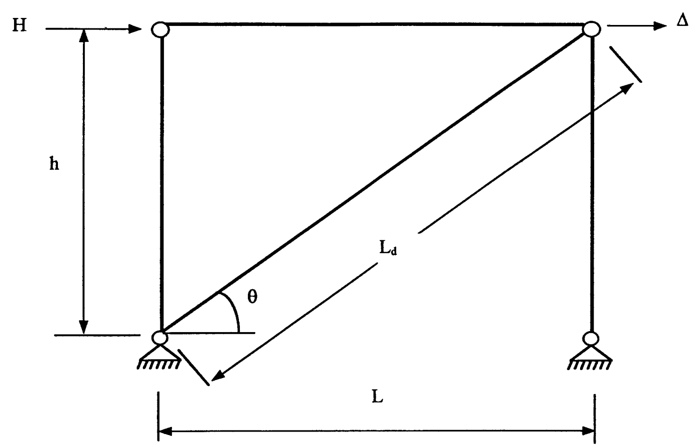


FIGURE 12.27: Lateral displacement of a diagonally braced frame.

The shear component Δ_s in Equation 12.3 is caused mainly by the straining of the diagonal brace. The deformation associated with girder compression has been neglected in the calculation of Δ_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 story 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 frame, which is a function of the area of the column and the ratio of the height-to-bay length (h/L). For a slender bracing frame with a large h/L ratio, the flexural component can contribute significantly to the overall story drift.

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

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 and 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 12.28a) because 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 the 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.

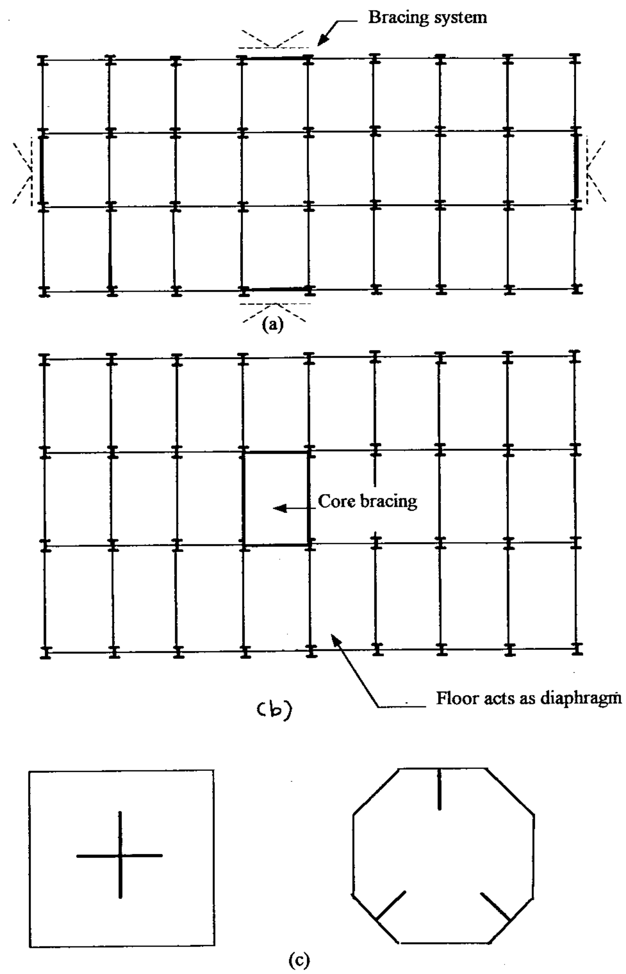


FIGURE 12.28: Locations of bracing systems: (a) exterior braced frames, (b) internal braced core, and (c) bracing arrangements to be avoided.

When core braced systems are used, they are normally located in the center of the building (Figure 12.28b). 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 Figure 12.28c should be avoided.

The flexibility of different bracing systems must be taken into account in the analysis because 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, overturning moments, and sidesway. 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 the 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-story 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 multistory frames are given in Table 12.2. 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 the design (see Section 12.2.6).

The design of the internal bracing members in a steel bracing system is similar to the design of lattice trusses. The horizontal member in a latticed bracing system serves also as a floor beam. This member will be subjected to bending due to gravity loads and axial compression due to wind. The columns must be designed for additional forces due to leaning column effects from adjacent gravity frames. The resistance of the members should therefore be checked as a beam-column based on the appropriate load combinations.

Figure 12.29 shows an example of a building that illustrates the locations of vertical braced trusses provided at the four corners to achieve lateral stability. Diaphragm action is provided by 130 mm lightweight aggregate 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.

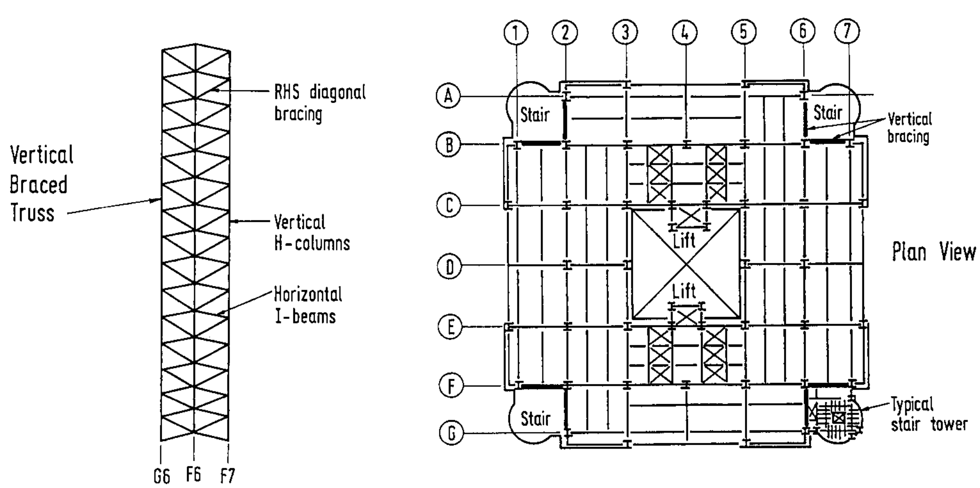


FIGURE 12.29: Simple building frame with vertical braced trusses located at the corners.

12.3.4 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 Figure 12.30a. The efficiency of development 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 6 to 9 m and structural height up to 20 to 30 stories are commonly used. However, as the building height increases, deeper girders are required to control drift; thus, the design becomes uneconomical.

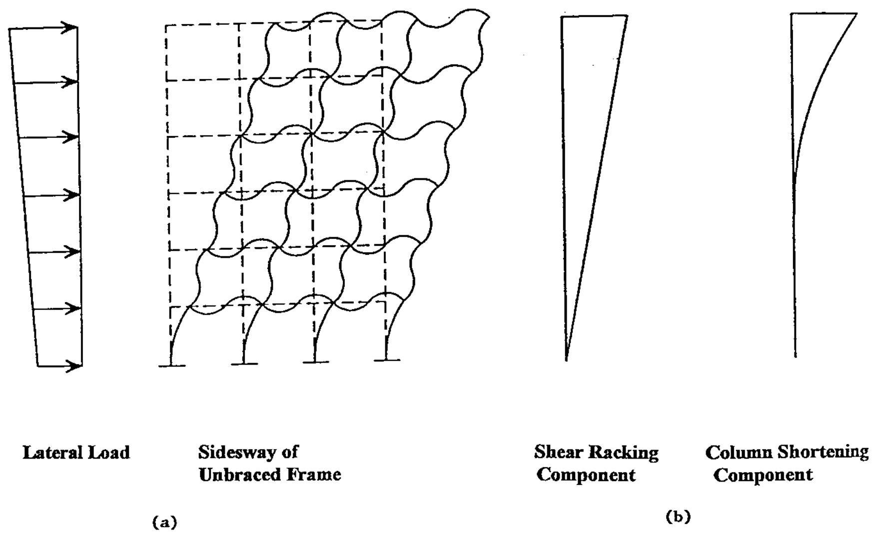


FIGURE 12.30: Sidesway resistance of a rigid unbraced frame.

When a rigid unbraced frame is subjected to lateral load, the horizontal shear in a story 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 sidesway 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 to 30 stories, the overturning moment contributes for about 10 to 20% of the total sway, whereas shear racking accounts for the remaining 80 to 90% (Figure 12.30b). However, the story drift due to overall bending tends to increase with height, while that due to shear racking tends to decrease.

Drift Assessment

Since shear racking accounts for most of the lateral sway, the design of such frames should be directed towards minimizing the sidesway due to shear. The shear displacement Δ in a typical story in a multistory frame, as shown in Figure 12.31, can be approximated by the equation:

$$\Delta_i = \frac{V_i h_i^2}{12E}\left(\frac{1}{\Sigma(I_{ci}/h_i)} + \frac{1}{\Sigma(I_{gi}/L_i)}\right) \tag{12.4}$$

where

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

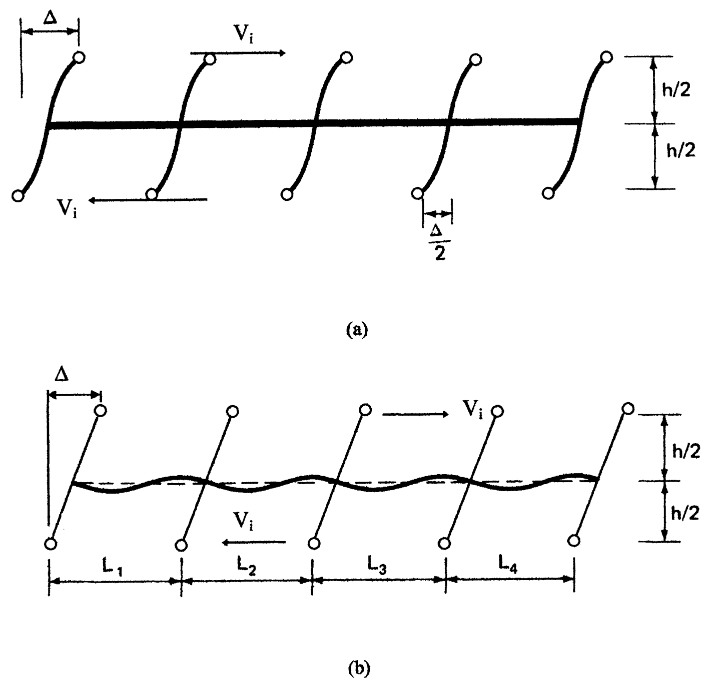


FIGURE 12.31: Story drift due to (a) bending of columns and (b) bending of girders.

Examination of Equation 12.4 shows that sidesway deflection caused by story shear is influenced by the sum of the column and beam stiffness in a story. Since for multistory construction, span lengths are generally larger than the story 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 story 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, story 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.

Apart from the beam span, height-to-width ratios of the building play an important role in the design of such structures. Wider building frames allow a larger number of bays (i.e., larger values for story summation terms $\Sigma(I_{ci}/h_i)$ and $\Sigma(I_{gi}/L_i)$ in Equation 12.4) with consequent reduction in frame drift. Moment frames with closed spaced columns that are connected by deep beams are very effective in resisting sidesway. This kind of framing system is suitable for use in the exterior planes of the building.

Moment Connections

Fully welded moment joints are expensive to fabricate. To minimize labor cost and to speed up site erection, field bolting instead of field welding should be used. Figure 12.32 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 (Figures 12.32c and d).

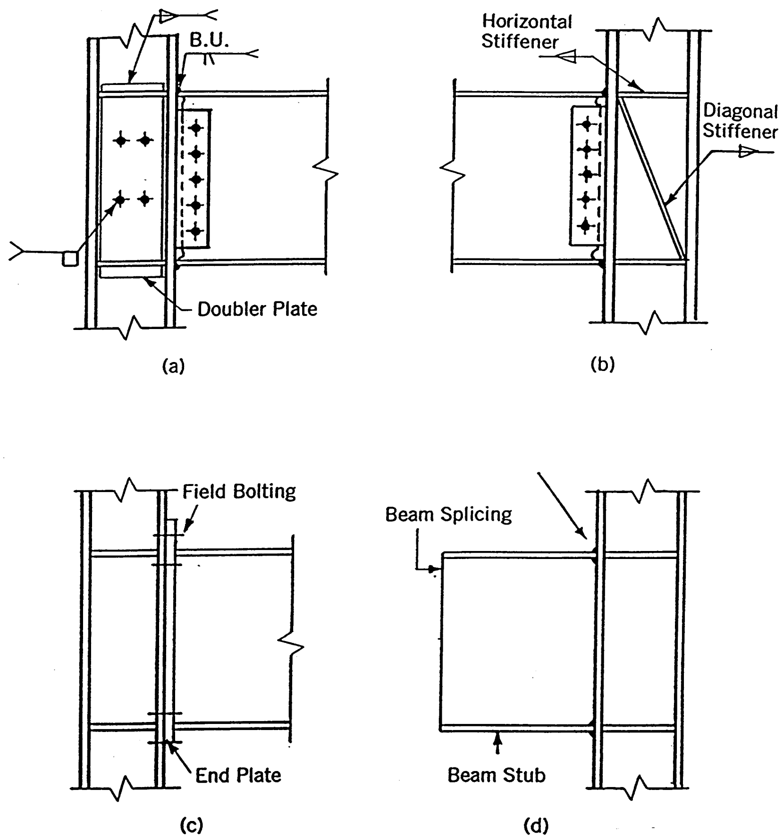


FIGURE 12.32: Rigid connections: (a) bolted and welded connection with doubler plate, (b) bolted and welded connection with diagonal stiffener, (c) bolted end-plate connection, and (d) beam-stub welded to column.

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

Figure 12.33. 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 necessary to stiffen the panel zone using either a doubler plate or a diagonal stiffener as shown in the joint details in Figures 12.32a and b. Otherwise, a heavier column with a 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.

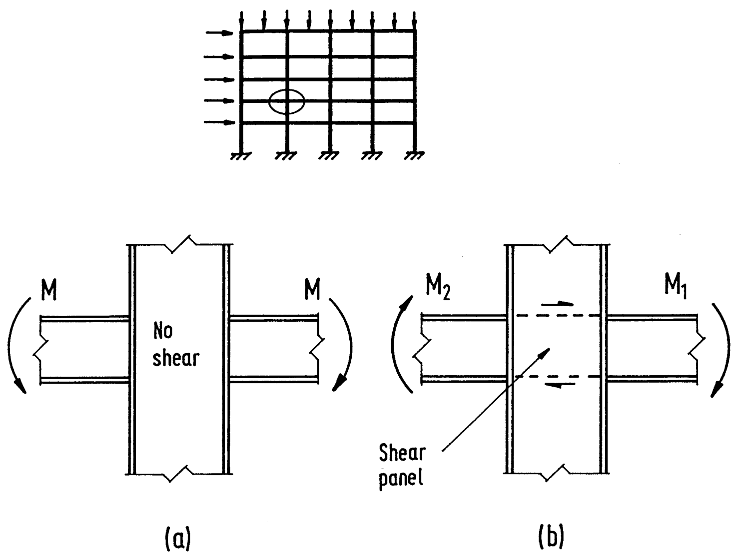


FIGURE 12.33: Forces acting on a panel joint: (a) balanced moment due to gravity load and (b) unbalanced moment due to lateral load.

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 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 [43].

Analysis and Design of Unbraced Frames

Multistory moment frames are statically indeterminate. The required design forces can be determined using either: (1) elastic analysis or (2) plastic analysis. While 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 sidesway.
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 AISC LRFD [3], 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 12.34) 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. allowance is made for second-order effects through the use of column effective length or moment amplification factors

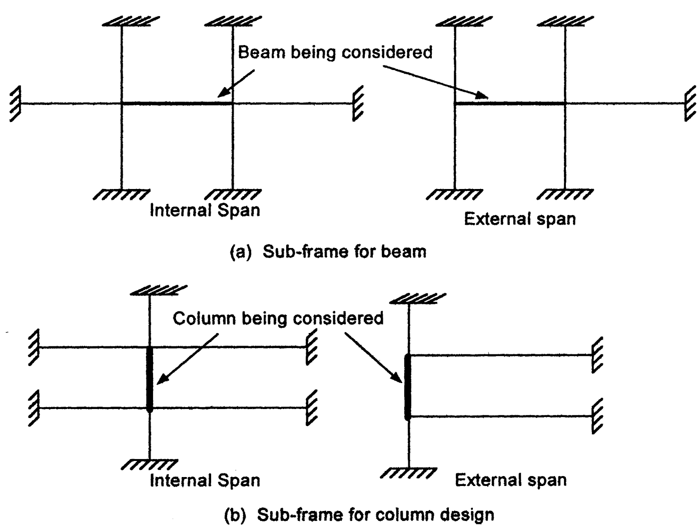


FIGURE 12.34: Sub-frame analysis for gravity loads.

Plastic analysis generally requires more sophisticated computer programs, which enable second-order effects to be taken into account. Computer software is now available through recent publications made available by Chen and Toma [14] and Chen et al. [15]. 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 Figure 12.34. If sidesway deflection is likely to control (e.g., slender frames) use Equation 12.4 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.

12.3.5 Tall Building Framing Systems

The following subsections discuss four classical systems that have been adopted for tall building constructions, namely (1) core braced, (2) moment-truss, (3) outriggle and belt, and (4) tube. 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 concept. Finally, by providing diagonal braces to the exterior framework, a superframe is formed and can be used for ultra-tall magastructures.

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 (Figure 12.35). The surrounding 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 is 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.

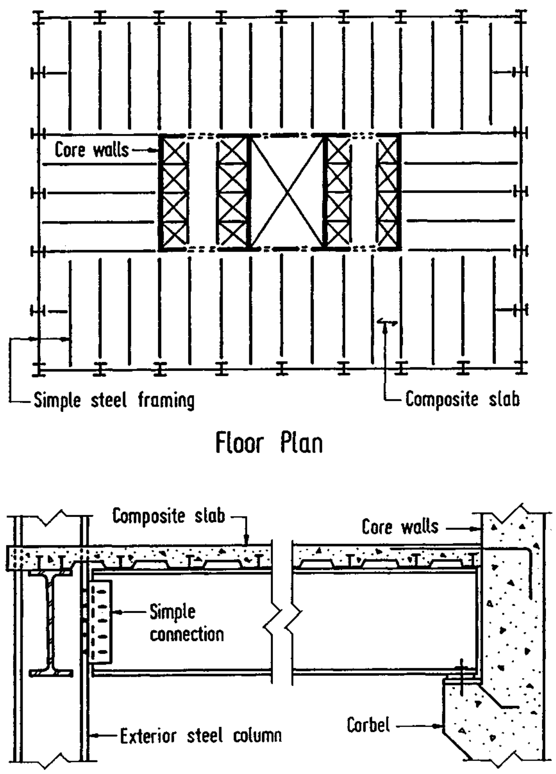


FIGURE 12.35: Core-brace frame. (a) Internal core walls with simple exterior framing and (b) beam-to-wall and beam-to-exterior column connections.

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 analyzed 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 floors (typically 4 to 6 story) ahead of the exterior steel framing. A core wall system represents an efficient type of structural system up to a certain height premium because of its cantilever action. However, when it is used alone, the massiveness of the wall structure increases 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 a tube system which could otherwise be used.

In commercial buildings where floor space is 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 buildings. 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 large 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.

Moment-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 Figure 12.36a. 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 Figure 12.36b. The internal core walls substitute the steel trusses in K, X, or a single brace form which may interfere with openings that provide access to, for example, elevators.

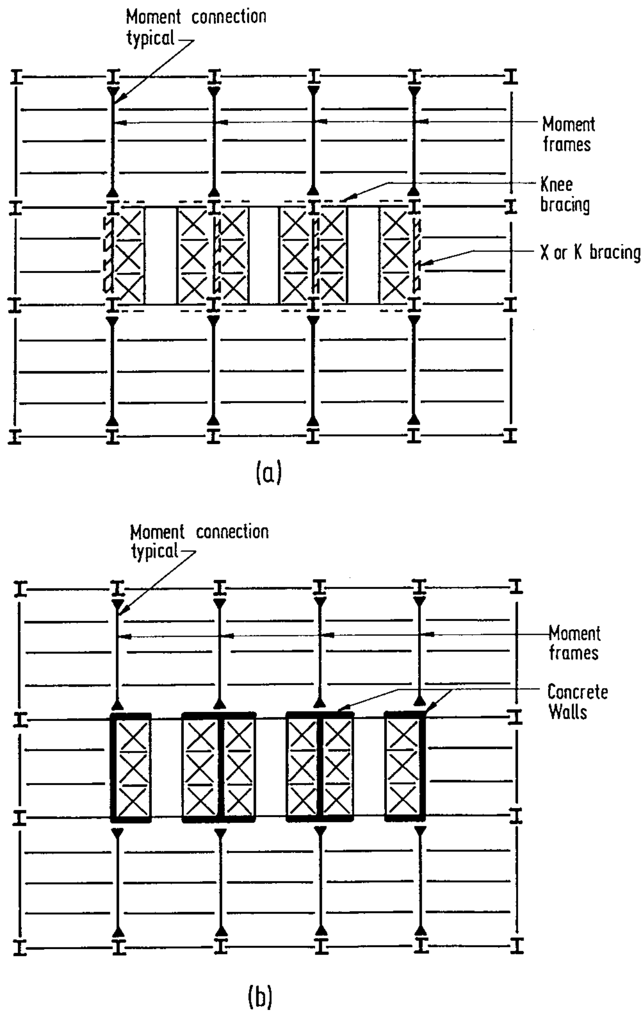


FIGURE 12.36: (a) Moment-frames with internal braced trusses. (b) Moment-frames with internal core walls.

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 12.37 compares the sway characteristic of a 20-story steel frame subjected to the 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,

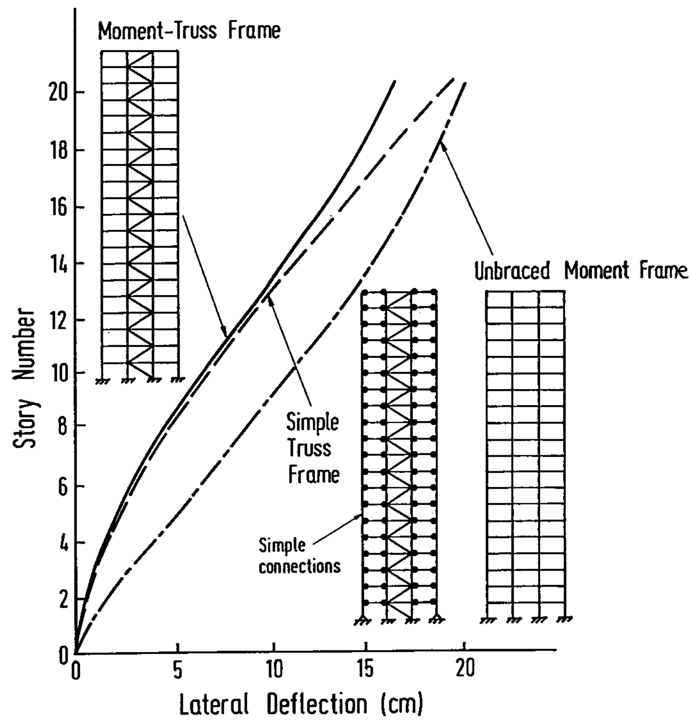


FIGURE 12.37: Sway characteristics of rigid braced frame, simple braced frame, and rigid unbraced frame.

but the overall frame drift increases toward the top of the frame. The moment frame, on the other hand, shows an opposite characteristic for sidesway in comparison with the simple braced frame. The combination of moment and truss frame provides overall improvement in reducing frame drift; the benefit becomes more pronounced 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 Figure 12.38, enhance the combined moment-truss frame stiffness to a level larger than the summation of individual moment frame and truss stiffnesses.

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.

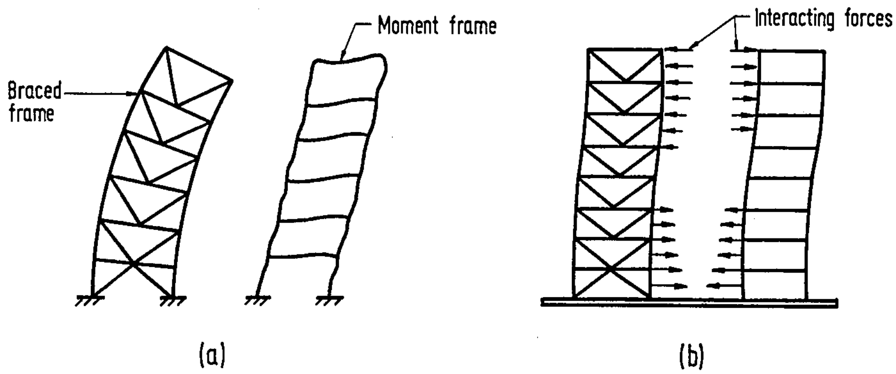


FIGURE 12.38: Behavior of frames subjected to lateral load: (a) independent behavior and (b) interactive behavior.

Figure 12.39 shows a typical example of such a 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.

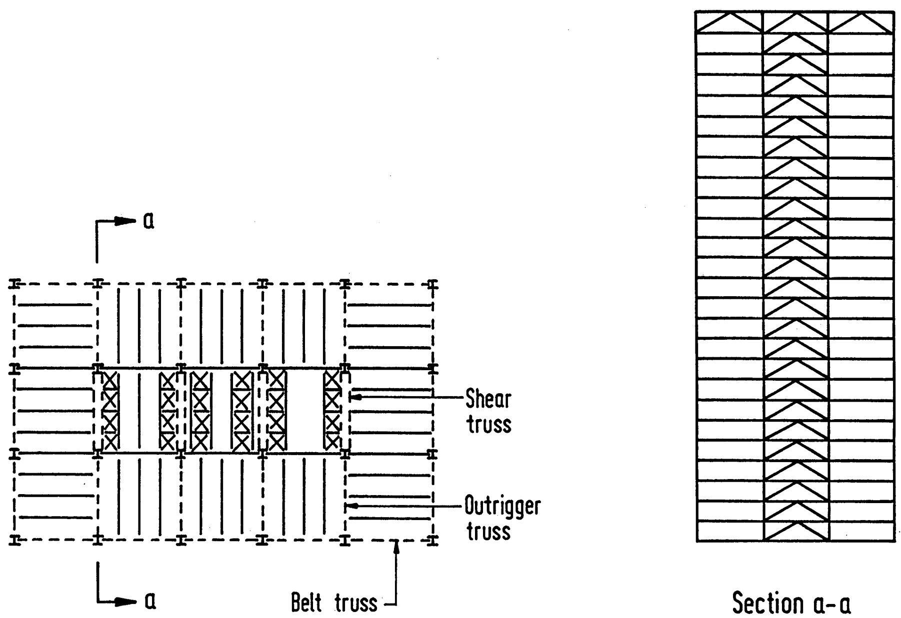


FIGURE 12.39: Outrigger and belt-truss system.

Figure 12.40 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.

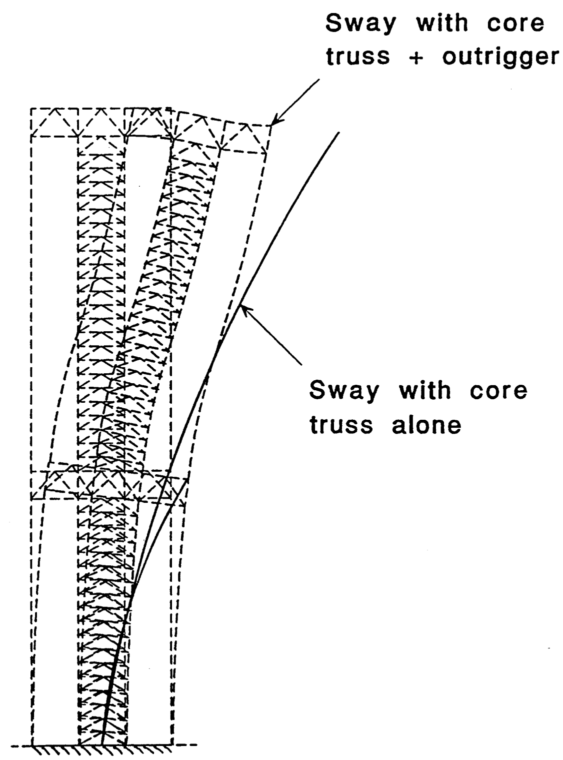


FIGURE 12.40: Improvement of lateral stiffness using outrigger-belt truss system.

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.

Frame Tube Systems

Figure 12.41 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 resists only its share of gravity loads. The behavior of the exterior frame tube is similar to a hollow perforated tube. The overturning 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.

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

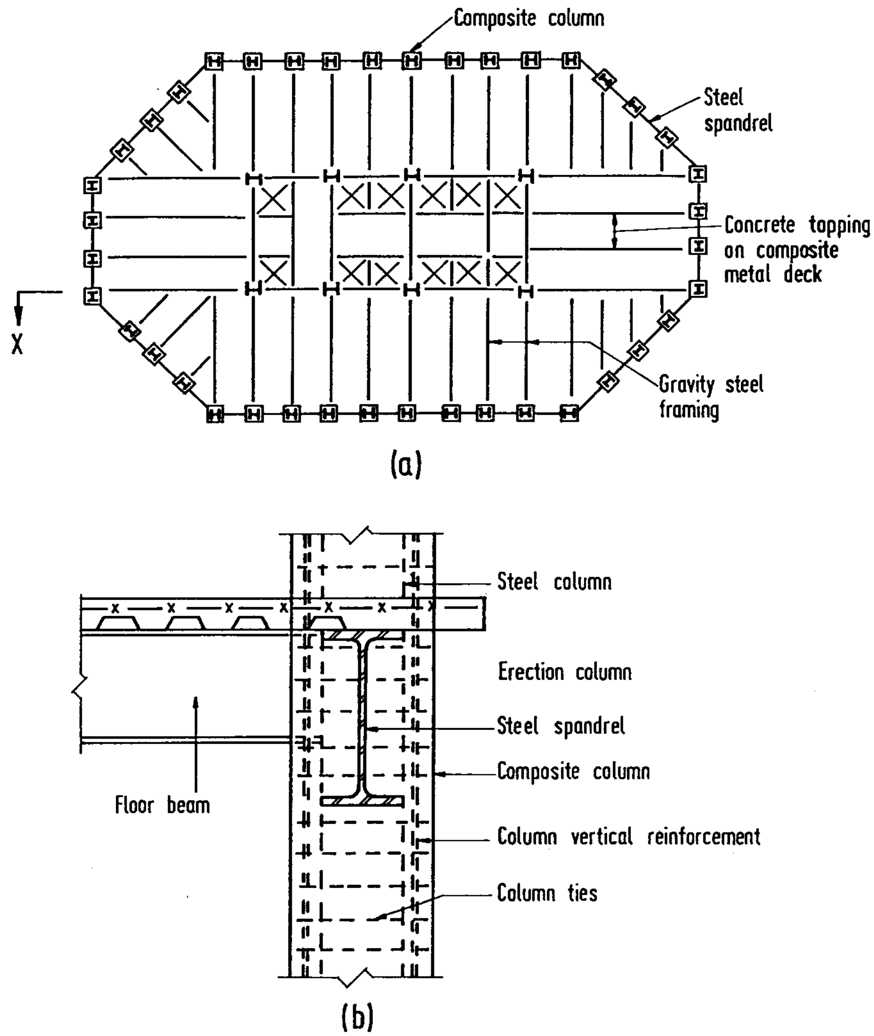


FIGURE 12.41: Composite tubular system.

of axial force. An approximate approach is to assume an equivalent column model as shown in Figure 12.42. 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 an exterior framed tube has three distinct advantages: (1) it develops high rigidity and strength for torsional and lateral-load resistance because the structural components are effectively placed at the exterior of the building forming a three-dimensional closed section; (2) massiveness of the frame tube system eliminates potential uplift difficulties and produces better dynamic behavior; and (3) the use of gravity steel framing in the interior has the advantages of flexibility and enables rapid construction. If a 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-

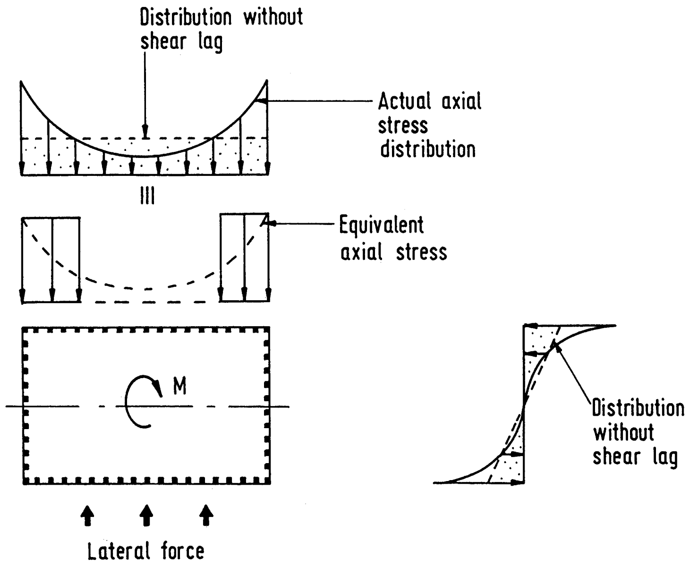


FIGURE 12.42: Equivalent column model for frame tube.

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 very 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.

12.3.6 Steel-Concrete Composite Systems

Steel-concrete composite construction has gained wide acceptance as an alternative 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.

Braced Composite Frames Subjected to Gravity Loads

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 [38, 39].

Figures 12.43a and b show the typical beam-to-column connections, one using a flushed end-plate bolted to the column flange and the other using a bottom angle with double web cleats. Composite action in the joint is acquired 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 bears 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.

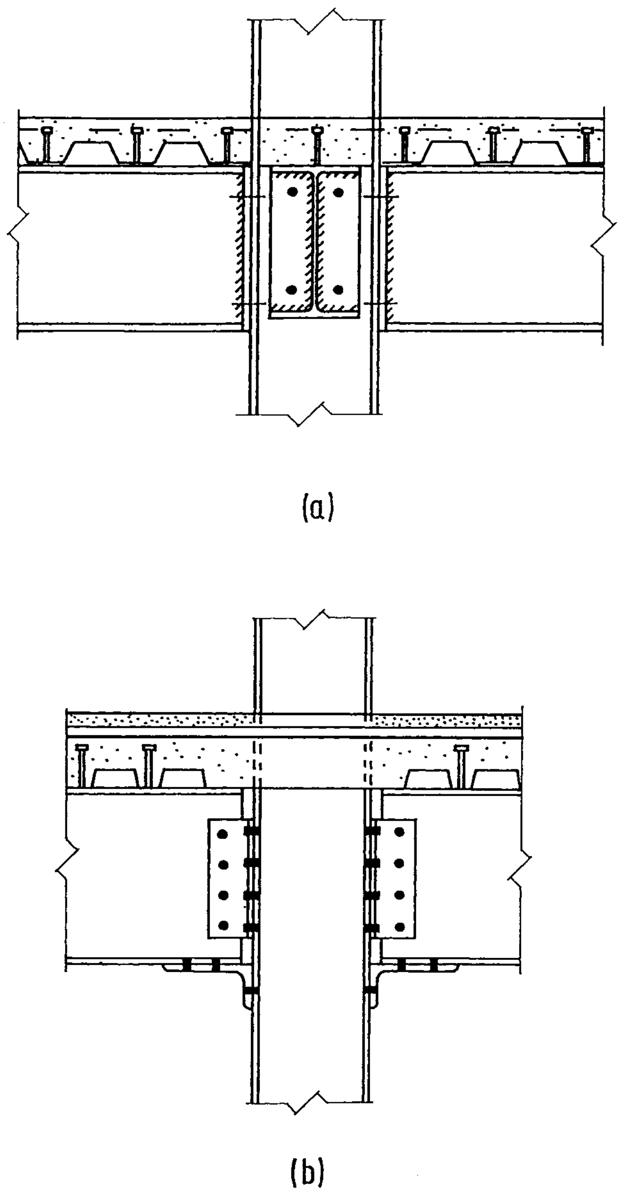


FIGURE 12.43: Composite beam-to-column connections with (a) flushed end plate and (b) seat and double web angles.

In designing the connections, slab reinforcements placed within 7 column flange widths 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. 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 a 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.

The moment of inertia of the composite beam I_{cp} may be estimated using a weighted average of moment of inertia in the positive moment region (I_p) and negative moment region (I_n). For interior spans, approximately 60% of the span is experiencing positive moment; it is suggested that [37]:

$$I_{cp} = 0.6 I_p + 0.4 I_n \tag{12.5}$$

where I_p is the lower bound moment of inertia for positive moment and I_n is the lower bound moment of inertia for negative moment. However, if the connections at both ends of the beam are designed and detailed for a simply supported beam, the beam will bend in single curvature under the action of gravity loads, and I_p should be used throughout.

Unbraced Composite Frames

If reinforcements are provided in the concrete encasement, composite design of members may be utilized for strength and stiffness assessment of the overall structure. The composite bending stiffness of the girder incorporating the slab may be utilized to reduce steel premium in controlling drift for high-rise frame design.

One approach is to use the composite beams as part of the frame. Since the slab element already exists, the composite flexural stiffness of the beams can be utilized with the steel beam alone resisting all negative moments. For an unbraced frame subjected to gravity and lateral loads, the beam typically bends in double curvature with a negative moment at one end of the beam and a positive moment at 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 a variable moment of inertia for the beams (see Figure 12.44). 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.

The story shear displacement Δ in an unbraced frame can be estimated using a modified expression from Equation 12.4 to account for the connection flexibility:

$$\Delta_i = \frac{V_i h_i^2}{12E}\left(\frac{1}{\Sigma(I_{ci}/h_i)} + \frac{1}{\Sigma(I_{cpi}/L_i)}\right) + \frac{V_i h_i^2}{\Sigma K_{\text{con}}} \tag{12.6}$$

where

Δ_i	=	the shear deflection of the i-th story
E	=	modulus of elasticity
I_c	=	moment of inertia for columns
I_g	=	moment of inertia of composite girder based on weighted average method
h_i	=	height of the i-th story
L_i	=	length of girder in the i-th story
V_i	=	total horizontal shear force in the i-th story

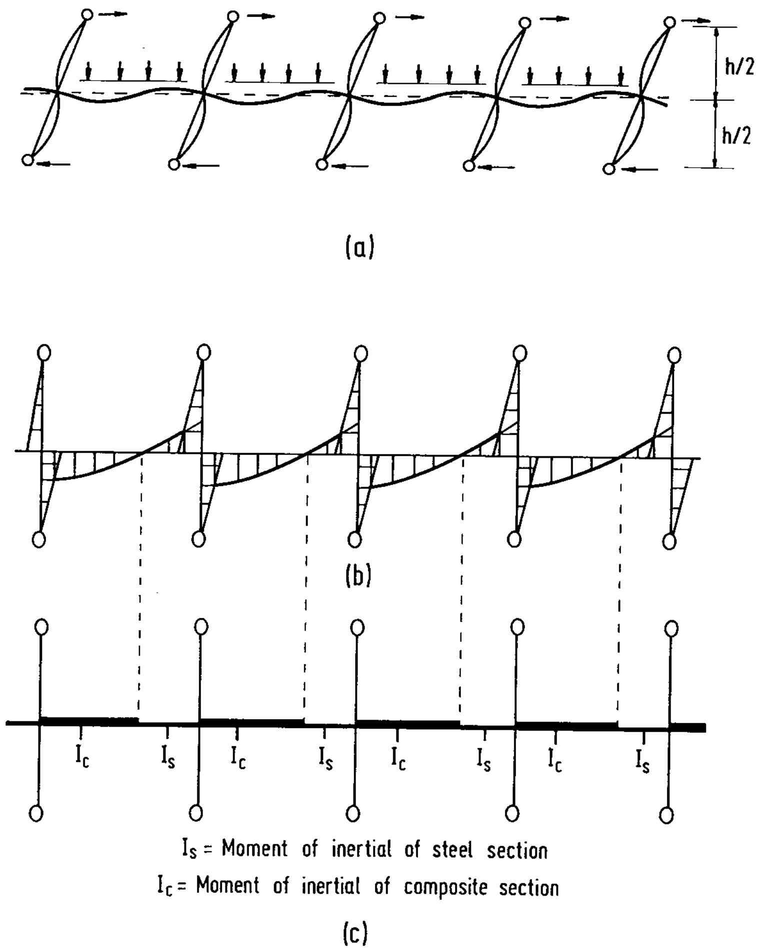


FIGURE 12.44: Composite unbraced frames: (a) story loads and idealization, (b) bending moment diagrams, and (c) composite beam stiffness.

$\Sigma(I_{ci}/h_i)$ = sum of the column stiffness in the i-th story
$\Sigma(I_{gi}/L_i)$ = sum of the girder stiffness in the i-th story
ΣK_{con} = sum of the connection rotational stiffness in the i-th story

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.

12.4 Wind Effects on Buildings

12.4.1 Introduction

With the development of lightweight high strength materials, the recent trend is to build tall and slender buildings. The design of such 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. Thus, 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 vectorally. A reduction factor of .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.

The effects of acceleration on human comfort is given in Table 12.3. The factors affecting the human response are:

1. Period of building—tolerence to acceleration tends to increase with period.
2. Women are more sensitive than men.
3. Children are more sensitive than adults.
4. Perception increases as you go from sitting on the floor, to sitting on a chair, to standing.
5. Perception threshold level decreases with prior knowledge that motion will occur.
6. Human body is more sensitive to fore-and-aft motion than to side-to-side motion.
7. Perception threshold is higher while walking than standing.
8. Visual cue—very sensitive to rotation of the building relative to fixed landmarks outside.
9. Acoustic cue—buildings 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.
10. 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.

TABLE 12.3 Acceleration Limits for Different Perception Levels

Perception	Acceleration limits
Imperceptible	$a < 0.005$ g
Perceptible	$0.005\text{ g} < a < 0.015$ g
Annoying	$0.015\text{ g} < a < 0.05$ g
Very annoying	$0.05\text{ g} < a < 0.15$ g
Intolerable	$a > 0.15$ g

TABLE 12.4 Serviceability Problems at Various Deflection or Drift Indices

Deformation as a fraction of span or height	Visibility of deformation	Typical behavior
1/500	Not visible	Cracking of partition walls
1/300	Visible	General architectural damage
		Cracking in reinforced walls
		Cracking in secondary members
		Damage to ceiling and flooring
		Facade damage
		Cladding leakage
		Visual annoyance
1/200 - 1/300	Visible	Improper drainage
1/100 - 1/200	Visible	Damage to lightweight partitions, windows, finishes
		Impaired operation of removable components such as doors, windows, sliding partitions

Since 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 12.45 [28]. Lower acceleration levels are used for residential buildings for the following

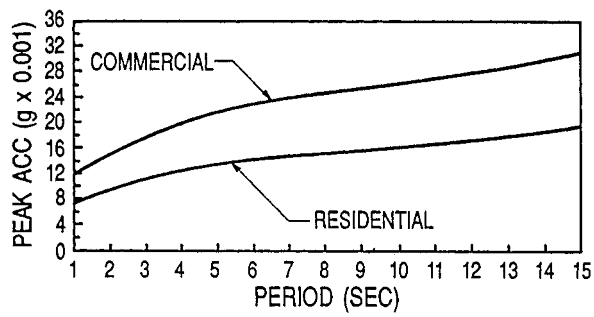


FIGURE 12.45: Design standard on peak acceleration for a 10-year return period.

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.
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 evacuated easily 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 12.4. Thus, the allowable drift, defined as the resultant peak displacement at the top of the building divided by the height of the building, is generatly taken to be in the range 1/450 to 1/600.

Figure 12.46 depicts schematically the procedure of estimating the wind-induced accelerations and displacements in a building. The steps involved in this design procedure are described below with numerical examples for situations where the motion of the building does not affect the loads acting on the building. Finally, the situations when a wind tunnel study is required are listed at the end of this section.

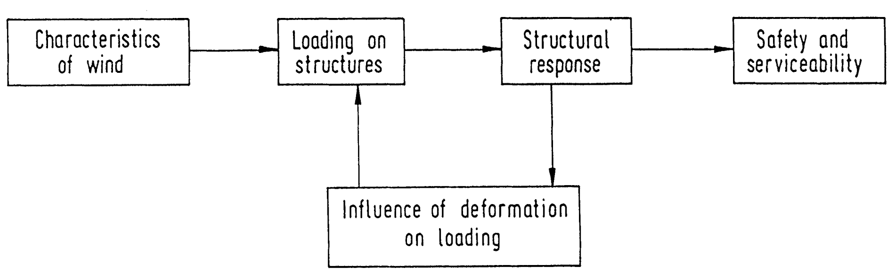


FIGURE 12.46: Schematic diagram for wind resistant design of structures.

12.4.2 Characteristics of Wind

Mean Wind Speed

The velocity of wind (wind speed) at great heights above the ground is constant and it is called the gradient wind speed. As shown in Figure 12.47, closer to the ground surface the wind speed is affected by frictional forces caused by the terrain and thus there is a boundary layer within which the wind speed varies from zero to the gradient wind speed. The thickness of the boundary layer (gradient height) depends on the ground roughness. For example, the gradient height is 457 m for large cities, 366 m for suburbs, 274 m for open terrain, and 213 m for open sea.

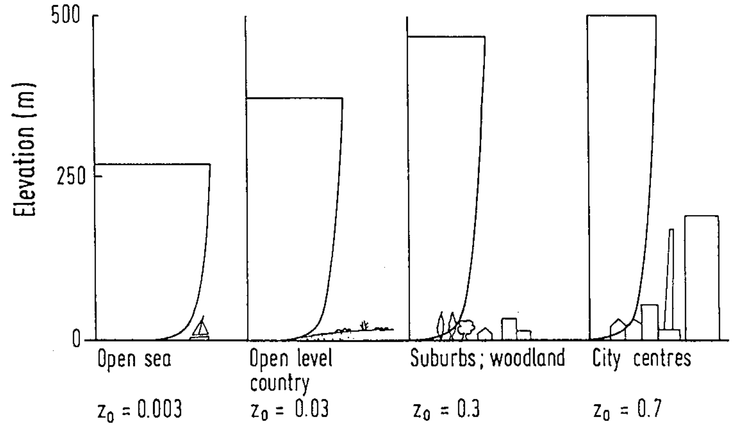


FIGURE 12.47: Mean wind profiles for different terrains.

The velocity of wind averaged over 1 h is called the hourly mean wind speed, $\bar{U}$. The mean wind velocity profile within the atmospheric boundary layer is described by a power law

$$\bar{U}(z) = \bar{U}(z_{\text{ref}}) \left(\frac{z}{z_{\text{ref}}}\right)^{\alpha} \tag{12.7}$$

in which $\bar{U}(z)$ is the mean wind speed at height z above the ground, z_{ref} is the reference height normally taken to be 10 m, and α is the power law exponent.

An alternative description of the mean wind velocity is by the logarithmic law, namely,

$$\bar{U}(z) = \frac{1}{K} u_* \ln\left(\frac{z-d}{z_o}\right) \tag{12.8}$$

in which u_* is the friction velocity, k is the von Karmon's constant equal to 0.4, z_o is the roughness length, and d is the height of zero-plane above the ground where the velocity is zero. Generally, zero plane is about 1 or 2 m below the average height of buildings and trees providing the roughness. Typical values of α, z_o, and d are given in Table 12.5 [4, 21].

The roughness affects both the thickness of the boundary layer and the power law exponent. The thickness of the boundary layer and the power law exponent increase with the roughness of the surface. Consequently the velocity at any height decreases as the surface roughness increases. However, the gradient velocity will be the same for all surfaces. Thus, if the velocity of wind for a particular terrain is known, using Equation 12.7 and Table 12.5, the velocity at some other terrain can be computed.

TABLE 12.5 Typical Values of Terrain Parameters z_o, α, and d

	z_o	α	d (m)
City centers	.7	.33	15 to 25
Suburban terrain	.3	.22	5 to 10
Open terrain	.03	.14	0
Open sea	.003	.10	0

Turbulence

The variation of wind velocity with time is shown in Figure 12.48. The eddies generated by the action of wind blowing over obstacles cause the turbulence. In general, the velocity of wind may be

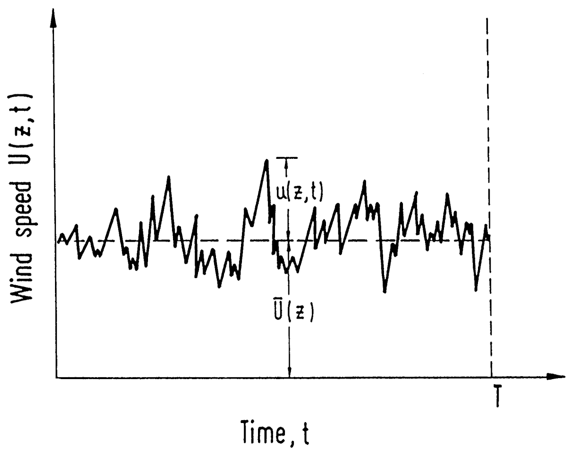


FIGURE 12.48: Variation of longitudinal component of turbulent wind with time.

represented in a vector form as

$$U(z,t) = \bar{U}(z)\underline{i} + u(z,t)\underline{i} + v(z,t)\underline{j} + w(z,t)\underline{k} \tag{12.9}$$

where u, v, and w are the fluctuating components of the gust in x, y, z (longitudinal, lateral, and vertical axes) as shown in Figure 12.49 and $\bar{U}(z)$ is the mean wind along the x axis. The fluctuating component along the mean wind direction, u, is the largest and is therefore the most important for the vertical structures such as tall buildings which are flexible in the along wind direction. The vertical component w is important for horizontal structures that are flexible vertically, such as long span bridges.

An overall measure of the intensity of turbulence is given by the root mean square value (r.m.s). Thus, for the longitudinal component of the turbulence

$$\sigma_u(z) = \left[\frac{1}{T_o}\int_o^{T_o}\{u(z,t)^2\}dt\right]^{1/2} \tag{12.10}$$

where T_o is the averaging period. For the statistical properties of the wind to be independent on the part of the record that is being used, T_o is taken to be 1 h. Thus, the fluctuating wind is a stationary random function over 1 h.

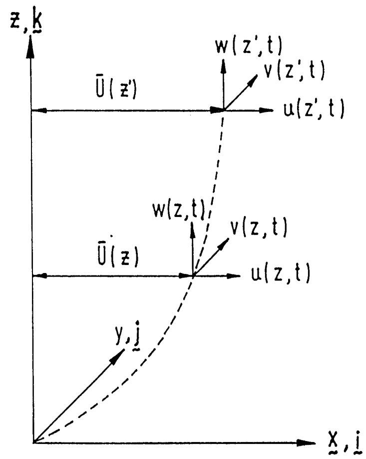


FIGURE 12.49: Velocity components of turbulent wind.

The value of $\sigma_u(z)$ divided by the mean velocity $\bar{U}(z)$ is called the turbulence intensity

$$I_u(z) = \frac{\sigma_u(z)}{\bar{U}(z)} \tag{12.11}$$

which increases with ground roughness and decreases with height.

The variance of longitudinal turbulence can be determined from

$$\sigma_u^2 = \beta u_*^2 \tag{12.12}$$

where u_* is the friction velocity determined from Equation 12.8 and β which is independent of the height is given in Table 12.6 for various roughness lengths.

TABLE 12.6 Values of β for Various Roughness Lengths

z_o (m)	.005	0.7	.30	1.0	2.5
β	6.5	6.0	5.25	4.85	4.0

Integral Scales of Turbulence

The fluctuation of wind velocity at a point is due to eddies transported by the mean wind $\bar{U}$. Each eddy may be considered to be causing a periodic fluctuation at that point with a frequency n. The average size of the turbulent eddies are measured by integral length scales. For eddies associated with longitudinal velocity fluctuation, u, the integral length scales are L_u^x, L_u^y, and L_u^z describing the size of the eddies in longitudinal, lateral, and vertical directions, respectively. If L_u^y and L_u^z are comparable to the dimension of the structure normal to the wind, then the eddies will envelope the structure and give rise to well-correlated pressures and thus the effect is significant. On the other hand, if L_u^y and L_u^z are small, then the eddies produce uncorrelated pressures at various parts of the structure and the overall effect of the longitudinal turbulence will be small. Thus, the dynamic loading on a structure depends on the size of eddies.

Spectrum of Turbulence

The frequency content of the turbulence is represented by the power spectrum, which indicates the power or kinetic energy per unit time associated with eddies of different frequencies. An expression for the power spectrum is [60]:

$$\frac{nS_u(z,n)}{u_*^2} = \frac{200f}{(1+50f)^{5/3}} \tag{12.13}$$

where $f = nz/\bar{U}(z)$ is the reduced frequency. A typical spectrum of wind turbulence is shown in Figure 12.50. The spectrum has a peak value at a very low frequency around .04 Hz. As the typical range for the fundamental frequency of a tall building is .1 to 1 Hz, the buildings are affected by high-frequncy small eddies characterizing the decending part of the power spectrum.

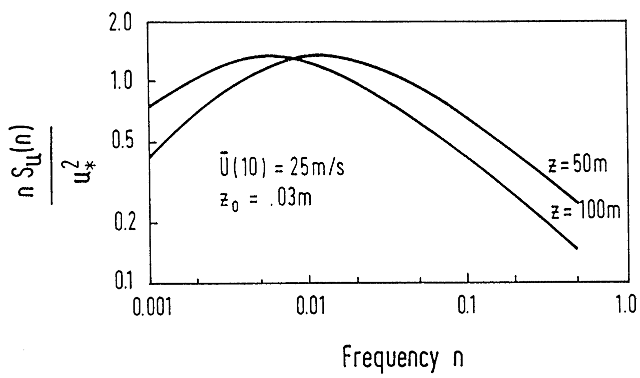


FIGURE 12.50: Power spectrum of longitudinal turbulence.

Cross-Spectrum of Turbulence

The cross-spectrum of two continuous records is a measure of the degree to which the two records are correlated. If the records are taken at two points, M_1 and M_2, separated by a distance, r, then the cross-spectrum of longitudinal turbulent component is defined as

$$S_{u_1u_2}(r,n) = S^c_{u_1u_2}(r,n) + iS^q_{u_1u_2}(r,n) \tag{12.14}$$

where the real and imaginary parts of the cross-spectrum are known as the co-spectrum and the quadrature spectrum, respectively. However, the latter is small enough to be neglected. Thus, the co-spectrum may be expressed non-dimensionally as the coherence and is given by

$$\gamma^2(r,n) = \frac{[S_{u_1u_2}(r,n)]^2}{S_{u_1}(n)S_{u_2}(n)} \tag{12.15}$$

where $S_{u_1}(n)$ and $S_{u_2}(n)$ are the longitudinal velocity spectra at M_1 and M_2, respectively.

The square root of the coherence is given by the following expression [19]:

$$\gamma(r,n) = e^{-\hat{f}} \tag{12.16}$$

where

$$\hat{f} = \frac{n[c_z^2(z_1-z_2)^2 + c_y^2(y_1-y_2)^2]^{1/2}}{1/2[\bar{U}(z_1)+\bar{U}(z_2)]} \tag{12.17}$$

in which y_1, z_1 and y_2, z_2 are the coordinates of points M_1 and M_2. The line joining M_1 and M_2 is assumed to be perpendicular to the direction of the mean wind. The suggested values of c_y and c_z for the engineering calculation are 16 and 10, respectively [62].

12.4.3 Wind Induced Dynamic Forces

Forces Due to Uniform Flow

A bluff body in a two-dimensional flow, as shown in Figure 12.51, is subjected to a nett force in the direction of flow (drag force), and a force perpendicular to the flow (lift force). Furthermore, when the resultant force is eccentric to the elastic center, the body will be subjected to torsional moment. For uniform flow these forces and moment per unit height of the object are determined

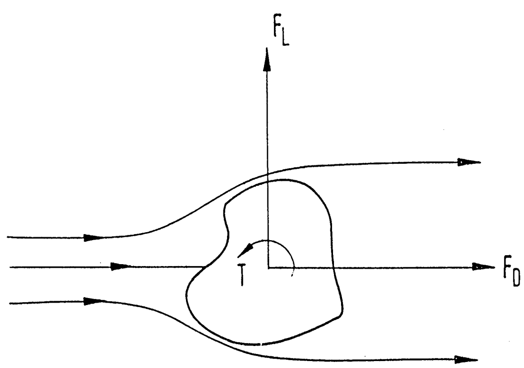


FIGURE 12.51: Drag and lift forces and torsional moment on a bluff body.

from

$$F_D = \frac{1}{2}\rho C_D B \bar{U}^2 \tag{12.18}$$

$$F_L = \frac{1}{2}\rho C_L B \bar{U}^2 \tag{12.19}$$

$$T = \frac{1}{2}\rho C_T B^2 \bar{U}^2 \tag{12.20}$$

where $\bar{U}$ is the mean velocity of the wind, ρ is the density of air, C_D and C_L are the drag and lift coefficients, C_T is the moment coefficient, and B is the characteristic length of the object such as the projected length normal to the flow.

The drag coefficient for a rectangular building in the plan is shown in Figure 12.52 for various depth-to-breadth ratios [5]. The shear layers originating from the separation points at the windward corners surround a region known as the wake. For elongated sections, the stream lines that separate at the windward corners reattach to the body to form a narrower wake. This is attributed to the reduction in the drag for larger aspect ratios. For cylindrical buildings in the plan, the drag coefficient is dependent on Reynolds number as indicated in Figure 12.53.

Unlike the drag force, the lift force and torsional moment do not have a mean value for a symmetric object with a symmetric flow around it, as the symmetrical distribution of mean forces acting in the across wind direction cannot cause a net force. If the direction of wind is not parallel to the axes of symmetries or if the object is asymmetrical, then there will be a mean lift force and torsional moment.

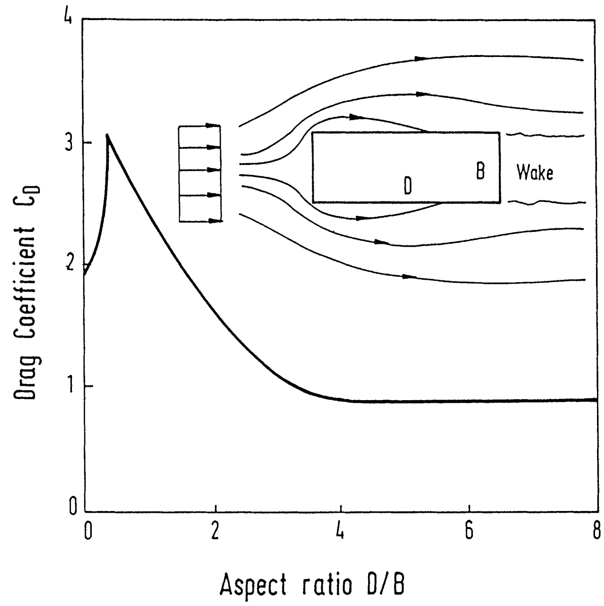


FIGURE 12.52: Drag coefficient for a retangular section with different aspect ratios.

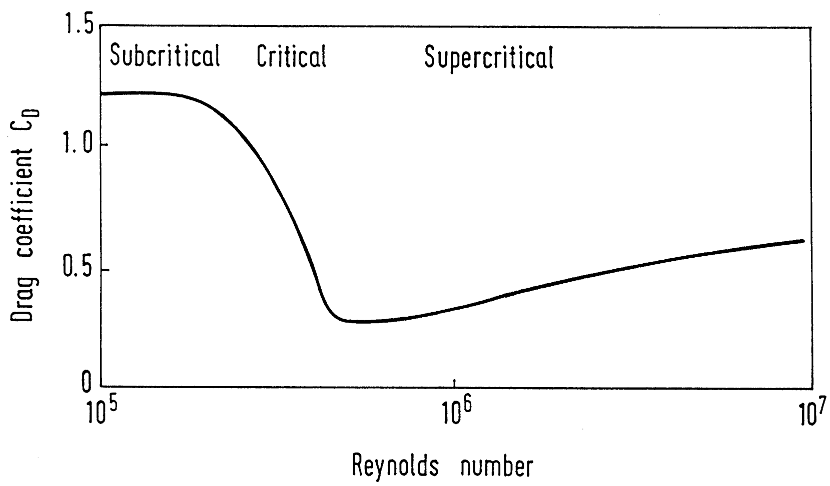


FIGURE 12.53: Effects of Reynolds number on drag coefficient of a circular cylinder.

However, due to vortex shedding, fluctuating lift force and torsional moment will be present in both symmetric and non-symmetric structures. Figure 12.54 shows the mechanism of vortex shedding. Near the separation zones, strong shear stresses impart rotational motions to the fluid particles. Thus, discrete vortices are produced in the separation layers. These vortices are shed alternatively from the sides of the object. The asymmetric pressure distribution created by the vortices around the cross-section leads to an alternating transverse force (lift force) on the object. The vortex shedding frequency in Hz, n_s, is related to a non-dimensional parameter called the Strouhal number, S, defined as

$$S = n_s B / \bar{U} \tag{12.21}$$

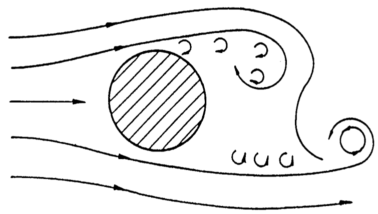


FIGURE 12.54: Vortex shedding in the wake of a bluff body.

where $\bar{U}$ is the mean wind speed and B is the width of the object normal to the wind. For objects with rounded profiles such as circular cylinders, the Strouhal number varies with the Reynolds number "Re" defined as

$$Re = \rho \bar{U} B / \mu \tag{12.22}$$

where ρ is the density of air and μ is the dynamic viscosity of the air. The vortex shedding becomes random in the transition region of $4 \times 10^5 < Re < 3 \times 10^6$ where the boundary layer at the surface of the cylinder changes from laminar to turbulent. Outside this transition range, the vortex shedding is regular producing a periodic lift force. For cross-sections with sharp corners, the Strouhal number is independent of the Reynolds number. The variation of the Strouhal number with length-to-breadth ratio of a rectangular cross-section is shown in Figure 12.55.

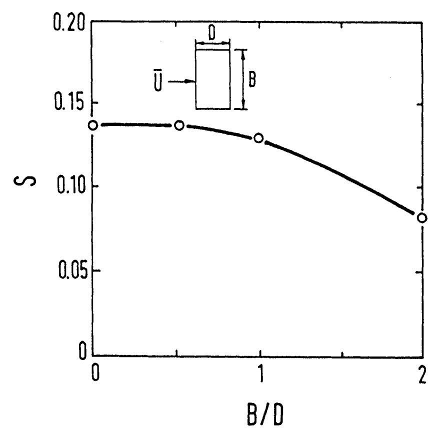


FIGURE 12.55: Strouhal number for a rectangular section.

Forces Due to Turbulent Flow

If the wind is turbulent, then the velocity of the wind in the along wind direction is described as follows:

$$U(t) = \bar{U} + u(t) \tag{12.23}$$

where $\bar{U}$ is the mean wind and $u(t)$ is the turbulent component in the along wind direction. The time dependent drag force per unit height is obtained from Equation 12.18 by replacing $\bar{U}$ by $U(t)$. As the ratio $u(t)/\bar{U}$ is small, the time dependent drag force can be expressed as

$$f_D(t) = \bar{f}_D + f'_D(t) \tag{12.24}$$

where $\bar{f}_D$ and f'_D are the mean and the fluctuating parts of the drag force per unit height which are given by

$$\bar{f}_D = \frac{1}{2}\rho\bar{U}^2 C_D B \tag{12.25}$$

$$f'_D = \rho\bar{U}uC_D B \tag{12.26}$$

The spectral density of the fluctuating part of the drag force is obtained from the Fourier transformation of the auto correlation function as

$$S_{fD}(n) = \rho^2\bar{U}^2 B^2 C_D^2 S_u(n) \tag{12.27}$$

where $S_u(n)$ is the spectral density of the turbulent velocity, which may be obtained from Equation 12.13.

In practice, the presence of the structure distorts the turbulent flow, particularly the small high-frequency eddies. A correction factor known as the aerodynamic admittance function $\chi(n)$ may be introduced [17] to account for these effects. The following emprical formula has been suggested for $\chi(n)$ [62]:

$$\chi(n) = \frac{1}{1 + \left[\frac{2n\sqrt{A}}{\bar{U}(z)}\right]^{4/3}} \tag{12.28}$$

where A is the frontal area of the structure. Now with the introduction of the aerodynamic admittance function, Equation 12.27 may be rewritten as

$$S_{fD}(n) = \rho^2\bar{U}^2 B^2 C_D^2 \chi^2(n) S_u(n) \tag{12.29}$$

It is evident from Equation 12.26 that the fluctuating drag force varies linearly with the turbulence. Thus, large integral length scale and high turbulent intensities will cause strong buffeting and consequently increase the along wind response of the structure. However, the regularity of vortex shedding is affected by the presence of turbulence in the along wind and, hence, the across wind motion and torsional motion due to vortex shedding decrease as the level of turbulence increases.

12.4.4 Response Due to Along Wind

Tall slender buildings, where the breadth of the structure is small compared to the height, can be idealized as a line-like structure as shown in Figure 12.56. Modeling the building as a continuous system, the governing equation of motion for along wind displacement $x(z,t)$ can be written as [29]:

$$m(z)\ddot{x}(z,t) + c(z)\dot{x}(z,t) + EI(z)x''''(z,t) - GA(z)x''(z,t) = f(z,t) \tag{12.30}$$

where m, c, El, and GA are, respectively, the mass, damping coefficient, flexural rigidity, and shear rigidity per unit height. Furthermore, $f(z,t)$ is the fluctuating wind load per unit height given in Equation 12.26.

Expressing the displacement in terms of the normal coordinates,

$$x(z,t) = \sum_{i=1}^{N}\phi_i(z)q_i(t) \tag{12.31}$$

where ϕ_i is the i-th vibration mode shape and q_i is the i-th normal coordinate. Using the orthogonality conditions of mode shapes, Equation 12.30 can be expressed as [9]

$$m_i^*\ddot{q}_i + c_i^*\dot{q}_i + k_i^*q_i = p_i^* \quad i = 1 \text{ to } N \tag{12.32}$$

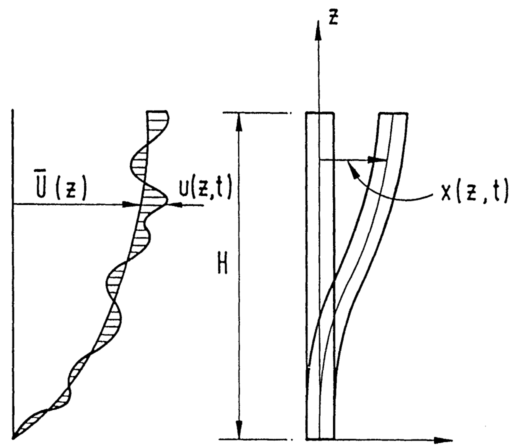


FIGURE 12.56: Typical deflection mode of a shear wall-frame building.

where m_i^*, c_i^*, k_i^*, and p_i^* are the generalized mass, damping, stiffness, and force in the i-th mode of vibration. The generalized mass and force are determined from

$$m_i^* = \int_o^H m(z)\phi_i^2(z)dz \tag{12.33}$$

$$\begin{aligned} p_i^* &= \int_o^H f(z,t)\phi_i(z)dz \\ &= \rho C_D B \int_o^H \bar{U}(z)u(z,t)\phi_i(z)dz \end{aligned} \tag{12.34}$$

Equation 12.32 consists of a set of uncoupled equations, each representing a single degree of freedom system. Using the random vibration theory [54], the power spectrum of the response in each normal coordinate is given by

$$S_{q_i}(n) = \mid H_i(n) \mid^2 S_{p_i^*}(n)\frac{1}{(k_i^*)^2} \tag{12.35}$$

where

$$\mid H_i(n) \mid = \frac{1}{\left(\left[1-\left(\frac{n}{n_i}\right)^2\right]^2 + 4\zeta_i^2\left(\frac{n}{n_i}\right)^2\right)^{1/2}} \tag{12.36}$$

and

$$k_i^* = 4\pi^2 n_i^2 m_i^* \tag{12.37}$$

in which n_i and ζ_i are the frequency and damping ratio in the i-th mode. The spectral density of the generalized force takes the form

$$\begin{aligned} S_{p_i^*}(n) = \; & \rho^2 C_D^2 B^2 \chi^2(n) \int_0^H \int_0^H \bar{U}(z_1)\bar{U}(z_2)S_{u_1u_2}(r,n) \\ & \phi_i(z_1)\phi_i(z_2)dz_1 \cdot dz_2 \end{aligned} \tag{12.38}$$

where $S_{u_1u_2}(r,n)$ is the cross-spectral density defined in Equation 12.14 with r being the distance between the coordinates z_1 and z_2. In Equation 12.38, the aerodynamic admittance has been incorporated to account for the distortion caused by the structure to the turbulent velocity.

In view of Equation 12.15, Equation 12.38 may be expressed as

$$\begin{aligned} S_{p_i^*}(n) &= \rho^2 C_D^2 B^2 \chi^2(n) \int_0^H \int_0^H \phi_i(z_1)\phi_i(z_2)\bar{U}(z_1)\bar{U}(z_2) \\ & \quad \sqrt{S_{u_1}(n)}\sqrt{S_{u_2}(n)}\gamma(r,n)dz_1dz_2 \end{aligned} \tag{12.39}$$

where $\gamma(r,n)$ is the square root of the coherence given in Equation 12.16, and $S_u(n)$ is the spectral density of the turbulent velocity.

The variance of the i-th normal coordinate is obtained from

$$\sigma_{q_i}^2 = \int_0^\infty S_{q_i}(n)dn = \frac{1}{(k_i^*)^2}\int_0^\infty |H_i \cdot (n)|^2 S_{p_i^*}(n)dn \tag{12.40}$$

The calculation of the above integral is very much simplified by observing the plot of the two components of the integrant shown in Figure 12.57. The mechanical admittance function is either 1.0 or 0 for most of the frequency range. However, over a relatively small range of frequencies around the natural frequency of the system, it attains very high values if the damping is small. As a result, the integrant takes the shape shown in Figure 12.57c. It has a sharp spike around the natural frequency

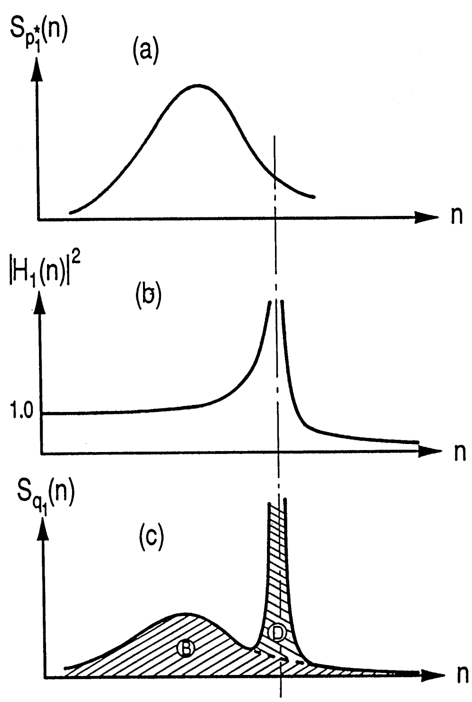


FIGURE 12.57: Schematic diagram for computation of response.

of the system. The broad hump is governed by the shape of the turbulent velocity spectrum which is modified slightly by the aerodynamic admittance function. The area under the broad hump is the broad band or non-resonant response, whereas the area in the vicinity of the natural frequency gives the narrow band or resonant response. Thus, Equation 12.40 can be rewritten as

$$\sigma_{q_i}^2 = \frac{1}{(k_i^*)^2}\left[\int_o^{n_i-\Delta n} S_{p_i^*}(n)dn + \frac{\pi n_i}{4\zeta_i} S_{p_i^*}(n_i)\right] = \sigma_{Bq_i} + \sigma_{Dq_i}^2 \tag{12.41}$$

in which σ_{Bq_i} and σ_{Dq_i} are the non-resonating and resonating root mean square response of the i-th normal coordinate. As the responses due to various modes of vibration are statistically uncorrelated, the response of the system is given by

$$\sigma_x^2(z) = \sum_{i=1}^{N} \phi_i^2(z)\sigma_{Bq_i}^2 + \sum_{i=1}^{N} \phi_i^2(z)\sigma_{Dq_i}^2 \tag{12.42}$$

which gives the variance and, hence, the root mean square displacement at various heights.

The total displacement is obtained by including the static deflection due to the mean drag load, which is determined conveniently as follows:

In view of Equation 12.25, the mean generalized force is given by

$$\begin{aligned} \bar{f}_i &= \int_o^H \frac{1}{2}\rho C_D \bar{U}^2(z) B\phi_i(z)dz \\ &= \frac{1}{2}\rho C_D B \int_o^H \bar{U}^2(z)\phi_i(z)dz \end{aligned} \tag{12.43}$$

Then the mean displacement is determined from

$$\bar{x}(z) = \sum_{i=1}^{N} \phi_i(z)\left[\frac{\bar{f}_i}{(2\pi n_i)^2 m_i^*}\right] \tag{12.44}$$

The root mean square acceleration is obtained from

$$\sigma_{\ddot{x}}(z) = \left[\sum_{i=1}^{N} (2\pi n_i)^4 \phi_i^2(z)\sigma_{Dq_i}^2\right]^{1/2} \tag{12.45}$$

The dynamic shear and bending moment at any height is obtained from the vibratory inertia forces in each mode and then by summing the modal contributions.

The probability of the response exceeding certain magnitude is determined using a peak factor on the root mean square response. Davenport [18] recommended the following expression for 50% probability of exceedence:

$$g_D = \sqrt{[2\ln(\nu T_0)]} + \frac{.577}{\sqrt{[2\ln(\nu T_0)]}} \tag{12.46}$$

where g_D is the peak factor, ν is the expected frequency at which the fluctuating response crosses the zero axis with a positive slope, and T_0 is the period (usually 3600 s) during which the peak response is assumed to occur.

For resonant response, ν is equal to the natural frequency and, thus, the peak factor for the resonant response g_D is obtained by setting $\nu = n$. For the non-resonating or broad band response the peak factor g_B has been evaluated to be 3.5 [20].

Using these peak factors, the most probable maximum value of the load effect, E, such as displacement, shear, bending moment, etc. are determined as follows:

$$E_{\max} = \bar{E} + \left[(g_B\sigma_{BE})^2 + (g_D\sigma_{DE})^2\right]^{1/2} \tag{12.47}$$

where σ_{BE} and σDE are the non-resonating and resonating components of the load effect and $\bar{E}$ is the load effect due to mean wind.

EXAMPLE 12.1:

A rectangular building of height $H = 194$ m is situated in a suburban terrain. The breadth B and width D of the building are 56 m and 32 m, respectively. The period of the building corresponding to the fundamental sway mode is 5.15 s. The values of the mode shape at various heights are given below:

H(m)	0	20	40	75	95	135	150	170	194
ϕ	0	.032	.096	.248	.365	.611	.746	.849	1.0

The generalized mass and damping ratio corresponding to this mode are 18×10^6 kg and 2%, respectively.

Assuming that the mean wind profile follows the power law with a power law coefficient $\alpha = .22$, determine the maximum drift for a 50-year wind storm of 21 m/s at 10 m height, blowing normal to the breadth of the building. Given that the friction velocity is 2.96 m/s, the drag coefficient C_D is 1.3 and density of air $\rho = 1.2$ kg/m^3.

Solution

The mean height of the building $\bar{H} = 97$ m

$$\bar{U}(97) = \bar{U}(10)\left(\frac{97}{10}\right)^{.22} = 21\left(\frac{97}{10}\right)^{.22} = 34.6 \text{ m/s}$$

At mid-height, the reduced frequency

$$f = \frac{n\bar{H}}{\bar{U}(\bar{H})} = \frac{97n}{34.6} = 2.8n$$

From Equation 12.13, the spectrum of turbulent wind is given by

$$S_u(\bar{H}, n) = \frac{2.96^2 \times 200 \times 2.8}{(1 + 50 \times 2.8n)^{5/3}} = \frac{4906}{(1 + 140n)^{5/3}}$$

Resonant displacement

$$\begin{aligned} n_1 &= \frac{1}{5.15} = .194 \text{ Hz}, \\ S_u(\bar{H}, n_1) &= \frac{4906}{(1 + 140 \times .194)^{5/3}} = 18.8 \text{ m}^2\text{/s} \end{aligned}$$

The admittance function, from Equation 12.28, becomes

$$\begin{aligned} \chi(n) &= \frac{1}{1 + \left(\frac{2n\sqrt{56 \times 194}}{34.6}\right)^{4/3}} = \frac{1}{1 + 10.96n^{4/3}} \\ \chi(n_1) &= .45 \end{aligned}$$

From Equation 12.39,

$$S_{p_i^*}(n) = \rho^2 C_D^2 B^2 \chi^2(n) S_u(\bar{H}, n) \frac{[\bar{U}(\bar{H})]^2}{\bar{H}^{2\alpha}}$$
$$\int_0^H \int_0^H \phi_1(z_1)\phi_1(z_2) z_1^\alpha z_2^\alpha \gamma(z_1, z_2, n) dz_1 dz_2$$

The square root of coherence γ is determined from Equation 12.16, considering only the vertical correlation. Thus,

$$\begin{aligned} S_{p_i^*}(n_1) &= 1.2^2 \times 1.3^2 \times 56^2 \times (.45)^2 \times 18.8 \times \frac{(34.6)^2}{97^{.44}} \times 16{,}900 \\ &= 7.85 \times 10^{10} \text{ N}^2/\text{ Hz} \end{aligned}$$

From Equations 12.41 and 12.42, the variance of resonant displacement at the top of the building is obtained as

$$\begin{aligned} \sigma_D^2 &= \phi_1^2(H)\sigma_{Dq_1}^2 = \frac{1}{(k_1^*)^2} \frac{\pi n_1}{4\zeta_1} S_{p_1^*}(n_1) \\ \sigma_D^2 &= \left(\frac{1}{26.8 \times 10^6}\right)^2 \left(\frac{\pi(.194)}{4(.02)}\right)(7.85 \times 10^{10})10^6 \text{ mm}^2 \\ \sigma_D &= 28.9 \text{ mm} \end{aligned}$$

Non-resonant displacement

The variance of non-resonant displacement at the top of the building is determined from Equations 12.41 and 12.42 as

$$\begin{aligned} \sigma_B^2 &= \phi_1^2(H)\sigma_{Bq_1}^2 = \frac{1}{(k_1^*)^2} \int_0^{n_1 - \Delta n} S_{p_1^*}(n) dn \\ &= \frac{565 \times 10^9}{(26.8 \times 10^6)^2} \times 10^6 \text{ mm} \\ \sigma_B &= 28 \text{ mm} \end{aligned}$$

Response to mean wind

From Equation 12.43, the mean generalized force

$$\begin{aligned} \bar{f}_1 &= \frac{1}{2}\rho C_D B \int_o^H \bar{U}^2(z)\phi_1(z) dz \\ &= \frac{1}{2}\rho C_D B[\bar{U}(\bar{H})]^2 \left(\frac{1}{\bar{H}}\right)^{2\alpha} \int_o^H z^{2\alpha}\phi_1(z) dz \\ &= \frac{1}{2} \times 1.2 \times 1.3 \times 56 \times (34.6)^2 \left(\frac{1}{97}\right)^{.44} \times 684 \\ &= 4.8 \times 10^6 \text{ N} \end{aligned}$$

The generalized stiffness

$$\begin{aligned} k_1^* &= \left(\frac{2\pi}{5.15}\right)^2 \times 18 \times 10^6 \\ &= 26.8 \times 10^6 \text{ N/m} \end{aligned}$$

Thus, the mean displacement

$$\bar{X} = \frac{4.8 \times 10^6}{26.8 \times 10^6} \times 10^3 = 179 \text{ mm}$$

The peak factor g_D for resonant response is determined from Equation 12.46 as 3.78. Using a peak factor of 3.5 for non-resonant response, the most probable maximum displacement is

$$\begin{aligned} X_{\max} &= \bar{X} + \sqrt{(g_B \sigma_B)^2 + (g_D \sigma_D)^2} \\ &= 179 + \sqrt{(3.5 \times 28)^2 + (3.78 \times 28.9)^2} \\ &= 326 \text{ mm} \end{aligned}$$

The most probable maximum drift would be

$$= \frac{326}{194{,}000} = \frac{1}{595}$$

The peak acceleration would be

$$\begin{aligned} &= .0289 \times 3.78 \times (2\pi)^2 \times (.194)^2 \\ &= 0.16 \text{ m/s}^2 \ \ (1.6\% \text{ g}) \end{aligned}$$

12.4.5 Response Due to Across Wind

For most modern tall buildings, the across wind response is more significant than the along wind response. Across wind vibration of a building is caused by the combination of forces from three sources: (1) buffeting by the turbulence in the across wind direction, (2) wake excitation due to vortex shedding, and (3) lock-in, a displacement dependent excitation.

The across wind force due to lateral turbulence in the approaching flow is generally small compared to the effects due to other mechanisms. Lock-in is the term used to describe large amplitude across wind motion that occurs when the vortex shedding frequency is close to the natural frequency. If the across wind response exceeds a certain critical value, the across wind response causes an increase in the excitation force, which in turn increases the response. The vortex shedding frequency tends to couple with the natural frequency of the structure for a range of wind velocities, and the large amplitude response will persist. Lock-in is likely to occur only in the case of structures with relatively low stiffness and low damping, operating near the critical wind velocity given by

$$\bar{U}_{\text{crit}} = \frac{n_o B}{S} \tag{12.48}$$

in which $\bar{U}_{\text{crit}}$ is the critical wind speed, B is the breadth of the structure normal to the wind stream, n_o (in Hz) is the fundamental natural frequency of the structure in the across-wind direction, and S is the Strouhal number.

Buildings should be designed so that lock-in effects do not occur during their anticipated life. If the root mean square displacement at the top of the structure is less than a certain critical value, then lock-in will not occur. For square tall buildings, the critical root mean square displacements σ_{yc} expressed as a ratio with respect to the breadth (σ_{yc}/B) are approximately .015, .025, and .045, respectively [55], for open terrain ($z_o = .07$ m), suburban terrain ($z_o = 1.0$ m), and city centers ($z_o = 2.5$ m). For circular sections with diameter D, the value of σ_{yc}/D is approximately .006 for suburban terrain.

Thus, for buildings, the most common cause for across wind motion is the wake excitation. Although the turbulence in the atmospheric boundary layer affects the regularity of vortex shedding, the shed vortices have a predominant period which could be determined from an appropriate Strouhal number. Because the vortex shedding is random, the fluctuating across wind force is effectively broad-band as shown in Figure 12.58. The band width and the energy concentration near the vortex shedding frequency depends on the geometry of the building and the characteristics of the approach flow.

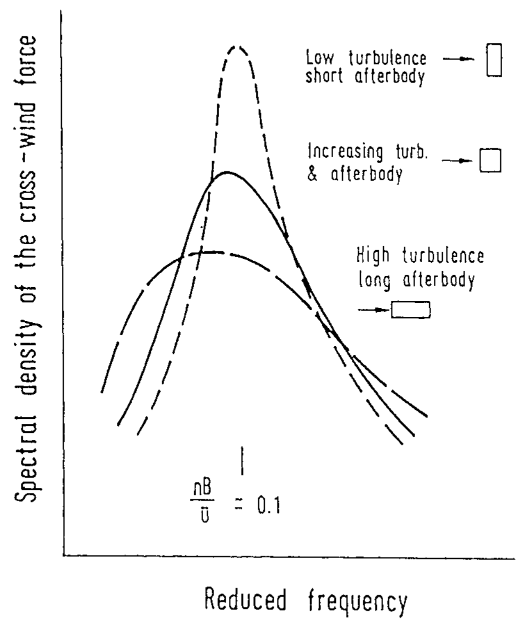


FIGURE 12.58: Effects of turbulence intensity and after body length on across wind force spectra.

The response due to this across wind random excitation can be determined using the random vibration theory. Idealizing the tall building as a line-like structure, the across wind displacement $y(z, t)$ may be expressed in terms of the normal coordinates $r_i(t)$ as

$$y(z, t) = \sum_{i=1}^{N} \psi_i(z) r_i(t) \tag{12.49}$$

where $\psi_i(z)$ is the i-th vibration mode in the across wind direction and N is the total number of modes considered to be significant. The governing equation of motion in terms of generalized mass m_i^*, generalized damping c_i^*, and generalized stiffness k_i^*, takes the form

$$m_i^* \ddot{r}_i + c_i^* \dot{r}_i + k_i^* r_i = f_i^*(t) \qquad i = 1 \text{ to } N \tag{12.50}$$

in which

$$\begin{aligned} m_i^* &= \int_o^H m(z) \psi_i^2(z) dz \\ k_i^* &= (2\pi n_i)^2 m_i^* \end{aligned}$$

$$\begin{aligned} c_i^* &= 2\zeta_i\sqrt{m_i^* k_i^*} \\ f_i^*(t) &= \int_o^H f(z,t)\psi_i(z)dz \end{aligned} \tag{12.51}$$

where H is the height of the building, $m(z)$ is the mass per unit length, n_i is the frequency of the i-th mode in the across wind direction, ζ_i is the damping ratio in the i-th mode, $f(z,t)$ is the across wind force per unit height, and $f_i^*(t)$ is the generalized across wind force in the i-th mode. The spectral density of each normal coordinate can be determined from

$$S_{r_i}(n) = \frac{|\ H_i(n)\ |^2}{(k_i^*)^2} S_{f_i^*}(n) \tag{12.52}$$

where $|H_i(n)|$ is the mechanical admittance function and $S_{f_i^*}$ is the power spectral density of the generalized across wind force.

The variance of the normal coordinate r_i is given by

$$\sigma_{r_i}^2 = \int_0^\infty S_{r_i}(n)dn \tag{12.53}$$

Hence, the variance of the across wind displacement is obtained from

$$\sigma_y^2(z) = \sum_{i=1}^N \psi_i^2(z)\sigma_{r_i}^2 \tag{12.54}$$

In Equation 12.53, if the contribution from the non-resonating component is neglected, then the root mean square response of the across wind displacement is given by

$$\sigma_y^2(z) = \left[\sum_{i=1}^N \frac{\psi_i^2(z)}{(2\pi n_i)^4 (m_i^*)^2}\left(\frac{\pi n_i}{4\zeta_i}\right) S_{f_i^*}(n_i)\right]^{1/2} \tag{12.55}$$

For convenient use of the above equation, the generalized force spectra obtained experimentally by Kwok and Melbourne [32] and Saunders and Melbourne [56] are presented in Figure 12.59 for various aspect ratios of square and rectangular buildings deflecting in a linear mode.

EXAMPLE 12.2:

Consider the building of Example 12.1. If the period of vibration in the across wind direction is 5.2 s, assuming a linear mode, determine the acceleration in the across wind direction for a 10-year wind storm of 14 m/s at 10 m height, given that the generalized mass corresponding to the linear mode is 17.5×10^6 kg and the damping in this mode of oscillation is 2%.

Solution

The building is rectangular with an aspect ratio of

$$H : B : D = 6 : 1.75 : 1$$

Since the building is in a suburban terrain, the generalized cross wind force can be determined from Figure 12.59e.

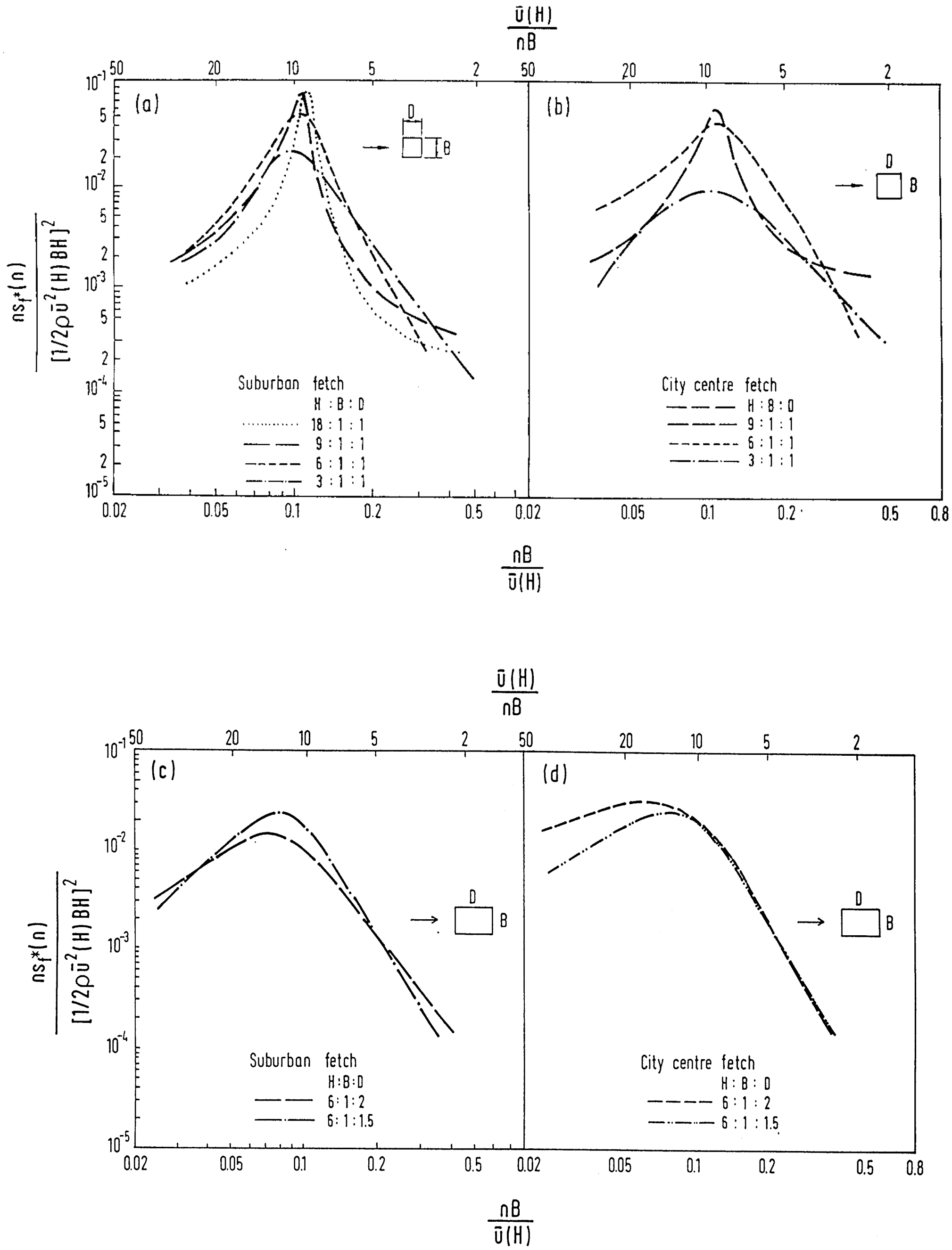


FIGURE 12.59: Generalized force spectra for a square and a rectangular building in suburban and city center fetch.

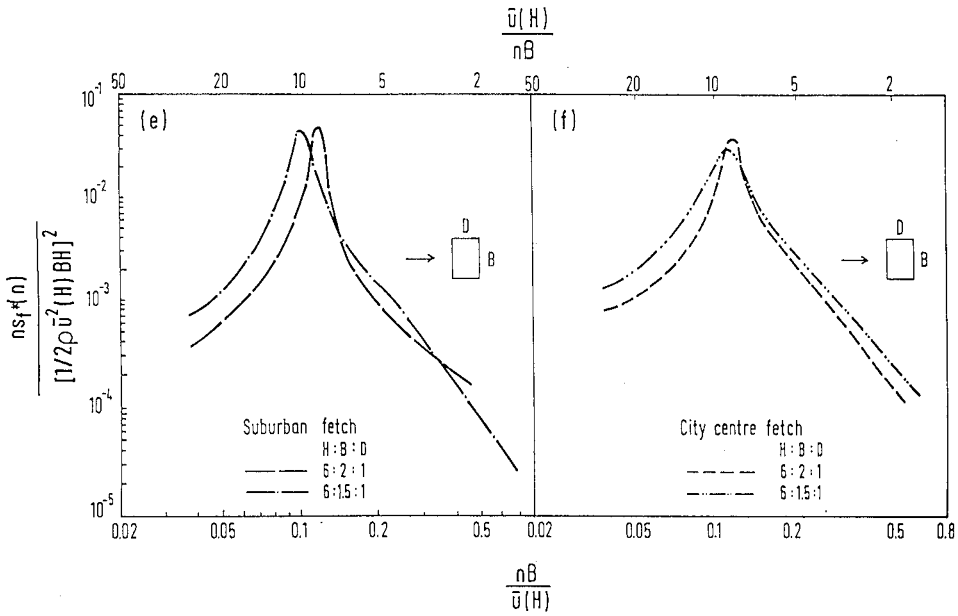


FIGURE 12.59: *(Continued)* Generalized force spectra for a square and a rectangular building in suburban and city center fetch.

The wind speed at the tip of the building

$$\bar{U}(H) = \bar{U}(10) \times \left(\frac{194}{10}\right)^{.22} = 26.9 \ \text{m/s}$$

The reduced frequency,

$$\frac{n_1 B}{\bar{U}(H)} = \frac{.192 \times 56}{26.9} = .40$$

then from Figure 12.59e.

$$\begin{aligned} S_{f_i^*}(n_1) &= \left(\frac{.00018}{.192}\right)\left(\frac{1}{2} \times 1.2 \times 26.9^2 \times 56 \times 194\right)^2 \\ &= 2.09 \times 10^{10} \ \text{N}^2/\text{Hz} \end{aligned}$$

From Equation 12.55,

$$\begin{aligned} \sigma_y &= \left[\left(\frac{\pi \times .192}{4 \times .02}\right)(2.09 \times 10^{10}) \frac{1}{(2\pi \times .192)^4 (17.5 \times 10^6)^2}\right]^{1/2} \\ &= .016 \ \text{m} \end{aligned}$$

Assuming a peak factor of 4, the peak acceleration in the cross wind direction

$$4 \times .016 (2\pi)^2 (.192)^2 = .093 \ \text{m/s}^2$$

12.4.6 Torsional Response

A building will be subjected to torsional motion when the instantaneous point of application of resultant aerodynamic load does not coincide with the center of mass and/or the elastic center. The major source for dynamic torque is the flow induced asymmetries in the lift force and the pressure fluctuation on the leeward side caused by the vortex shedding. Any eccentricities between the center of mass and center of stiffness present in asymmetrical buildings can amplify the torsional effects.

Balendra, Nathan, and Kang [8] have presented a time domain approach to estimate the coupled lateral-torsional motion of buildings due to along wind turbulence and across wind forces and torque due to wake excitation. The experimentally measured power spectra of across wind force and torsional moment [53] were used in this analysis. This method is useful at the final stages of design as specific details that are unique for a particular building can be easily incorporated in the analytical model. A useful method to assess the torsional effects at the preliminary design stage is given by the following empirical relation [58] which yields the peak base torque induced by wind speed $\bar{U}(H)$ at the top of the building as:

$$T_{\text{peak}} = \Psi(\bar{T} + g_T T_{\text{rms}}) \tag{12.56}$$

where Ψ is a reduction coefficient, g_T is the torsional peak factor equal to 3.8, and $\bar{T}$ and T_{rms} are the mean and root mean square base torques which are given by

$$\begin{aligned} T_{\text{rms}} &= .00167 \frac{1}{\sqrt{\zeta_T}} \rho L^4 H n_T^2 U_r^{2.68} && (12.57) \\ \bar{T} &= .038 \rho L^4 H n_T^2 U_r^2 && (12.58) \end{aligned}$$

in which

$$\begin{aligned} L &= \frac{\int \mid r \mid ds}{\sqrt{A}} && (12.59) \\ U_r &= \frac{\bar{U}(H)}{n_T L} && (12.60) \end{aligned}$$

where ρ is the density, H is the height of the building, n_T and ζ_T are the frequency and damping ratio in the fundamental torsional mode of vibration, $\mid r \mid$ is the distance between the elastic center and the normal to an element ds on the boundary of the building, and A is the cross-sectional area of the building. The expressions for $\bar{T}$ and T_{rms} are obtained for the most unfavorable directions for the mean and root mean square values of the base torque. In general, these directions do not coincide and furthermore will not be along the direction of the extreme winds expected to occur at the site. As such, a reduction coefficient Ψ $(.75 < \Psi \leq 1)$ is incorporated in Equation 12.56.

For a linear fundamental mode shape, the peak torsional induced horizontal acceleration at the top of the building at a distance "a" from the elastic center is given by [27]

$$a\ddot{\theta} = \frac{2 a g_T T_{\text{rms}}}{\rho_b B D H r_m^2} \tag{12.61}$$

where $\ddot{\theta}$ is the peak angular acceleration, ρ_b is the mass density of the building, B and D are the breadth and depth of the building, and r_m is the radius of gyration. For a rectangular building with uniform mass density,

$$r_m^2 = \frac{1}{12}(B^2 + D^2) \tag{12.62}$$

EXAMPLE 12.3:

If the torsional frequency of the building in Example 12.2 is .8 Hz, assuming a linear mode and 2% damping ratio, determine the peak acceleration at the corner of the building due to torsional motion for a 10-year wind storm of 14 m/s, given that the center of rigidity is at the geometric center of the building.

Solution

For a rectangular building

$$\int \mid r \mid ds = \frac{1}{2}(B^2 + D^2)$$

Thus, from Equations 12.59 and 12.60,

$$\begin{aligned} L &= \frac{1}{\sqrt{BD}}(B^2 + D^2)\frac{1}{2} = 49.1 \text{ m} \\ U_r &= \frac{U(H)}{n_T L}\,\frac{26.9}{.8 \times 49.1} = .685 \end{aligned}$$

From Equation 12.58

$$\begin{aligned} T_{\text{rms}} &= .00167\left(\frac{1}{\sqrt{(.02)}}\right)(1.2)(49.1)^4(194)(.8)^2(.685)^{2.68} \\ &= 3.71 \times 10^6 \text{ N.m} \end{aligned}$$

The average density of the building is determined as

$$\rho_b = \frac{3m_1^*}{AH} = \frac{3 \times 17.5 \times 10^6}{56 \times 32 \times 194} = 151 \text{ kg/m}^3$$

Thus, the peak torsional acceleration of the corner for which $a = 32.2$ m, is

$$a\ddot{\theta} = \frac{2 \times 32.2 \times 3.8 \times 3.71 \times 10^6}{151 \times 56 \times 32 \times 194 \times 346.7} = .05 \text{ m/s}^2$$

12.4.7 Response by Wind Tunnel Tests

There are many situations where analytical methods cannot be used to estimate certain types of wind loads and associated structural response. For example, the aerodynamic shape of the building is rather uncommon or the building is very flexible so that its motion affects the aerodynamic forces acting on the building. In such situations, a more accurate estimate of wind effects on buildings are obtained through aeroelastic model tests in a boundary-layer wind tunnel [9].

The aeroelastic model studies would provide the overall mean and dynamic loads, displacements, rotations, and accelerations. The aeroelastic model studies may be required under the following situations:

1. when the height-to-width ratio exceeds 5
2. when the structure is light with a density in the order of 1.5 kN/m^3
3. the fundamental period is long in the order of 5 to 10 s
4. when the natural frequency of the building in the cross wind direction is in the neighborhood of the shedding frequency
5. when the building is torsionally flexible
6. when the building is expected to execute strongly coupled lateral-torsional motion.

12.5 Defining Terms

Aeroelastic model: The model which simulates the dynamic properties of buildings to capture the motion dependent loads.

Along wind response: Response in the direction of wind.

Boundary layer: The layer within which the velocity varies because of ground roughness.

Bracing frames: Frames that provide lateral stability to the overall framework.

Composite beams: Steel beam acting compositely with part of the concrete slab through shear connectors.

Cross wind response: Response perpendicular to the direction of wind.

Drag force: Force in the direction of wind.

Frequency: Number of cycles per second.

Gradient height: Thickness of the boundary layer.

Gradient wind: Wind velocity above the boundary layer.

Generalized force: Force associated with a particular mode of vibration.

Generalized mass: Participating mass in a particular mode of vibration.

Integral length scale: A measure of average size of the eddies.

Lift force: Force perpendicular to the flow.

Lock-in: Situation where the vortex shedding frequency tends to couple with the frequency of the structure.

Long span systems: Structural systems that span a long distance. The design is likely to be governed by serviceability limit states.

Mode shapes: Free vibration deflection configurations in each frequency of the structure.

Non-resonating response: Response due to eddies whose frequencies are remote from the structural frequency.

Normal coordinates: Coordinates associated with modes of vibration.

Peak factor: Ratio between the peak and rms values.

Period: Duration of one complete cycle.

Power spectral density: Kinetic energy per unit time associated with eddies of different frequencies.

Resonant response: Response due to eddies whose frequencies are in the neighborhood of structural frequency.

Rigid frames: Frames resisting lateral load by bending of members which are rigidly connected.

Simple frames: Frames that have no lateral resistance and whose members are pinned connected.

Stiffness: Force required to produce unit displacement.

Sway frames: Frames in which the second-order effects due to gravity load acting on the deformed geometry can influence the force distribution in the structure.

Torsional response: Response causing twisting motion.

Turbulent intensity: Overall measure of intensity of turbulence.

Wake: Region surrounded by the shear layers originating from separation points.

Wake excitation: Excitation caused by the vortices in the wake.

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