CHAPTER 2

Literature Review of Steel Tall Buildings

2.1 Introduction

The growth in modern tall building construction began in 1880s, and has been largely for commercial and residential purposes. The rapid growths of the urban population and the consequent pressure on limited space have considerably influenced city residential development. In some cities local topographical restriction make tall buildings the only feasible solution for housing need. (3)

The multistory buildings were a feature since ancient Rome made of wood. After the great fire of Nero, they used a new brick and concrete materials in buildings. With the rapidly increasing number of masonry building toward the end of nineteenth century. The limit of this form become apparent in 1891 in 16-storey. In the middle of 19^{th} century two major technical innovation have occurred, the development of higher strength, wrought iron and subsequently steel, and the introduction of the elevator. The new materials allowed the development of lightweight skeletal structures to be permitted buildings of greater high and with larger interior open spaces. The golden age of skyscraper construction culminated in 1931 in its crowning glory, The Empire State building, whose 102-storey braced steel frame. At the mean time reinforced concrete construction does not appear to have been used for multistory buildings till the end of world war two. (5)

2.2 Structural Forms of Tall Buildings

Structural systems for tall buildings have undergone a dramatic evolution throughout the previous two decades. Developments in structural form have historically been realized as a response to emerging architectural trends in high-rise-building design. It’s only natural that the ordinary observer recognizes the tall buildings primarily with respect to its exterior architectural enclosure. The overall special form as well as detailing of the cladding urban environment. The aim of this chapter was to have a look under the outer covering of the building to reveal the structural skeleton of tall buildings. (3)

2.2.1 Braced Frame Structure

Bracing is generally regarded as an exclusive steel system because the diagonals are inevitably subjected to tension for one or the other directions of lateral loading. The advantage of braced frame structures is being able to produce the lateral very stiff structure for a minimum of additional material, however, makes it an economical structural form for any height of building. An additional advantage of fully triangulated bracing is that the girder usually participate only minimally in the lateral load bracing action. A major disadvantage of diagonal bracing is that it’s obstructs the internal planning and the local in of windows and doors. The lateral resistance in braced framed structure is provided by diagonal members that, together with girders, form the web of the vertical truss, with the columns acting as the chords as shown in **Figure (2.1)**. (5)

2.2.2 Rigid Frame Structures

Rigid frame structures consist of columns and girders joined by moment resistant connections. The rigid frame drives its unique strength to resist lateral loads from the moment interaction between beams and columns. The moment restrained at the ends leads to reduce positive bending moment for beams and reduce effective length for columns under gravity loads as shown in **Figure (2.2)**. Rigid frame construction is ideally suited for reinforced concrete buildings because of the inherent rigidity of reinforced concrete joints. The rigid frame form is also used for steel frame buildings, but moment resistant connection in steel tend to be costly and complex to construct. While rigid frames that serve alone to resist later loading have economic height of about 25-storeys. The height can be increased by using a combination of shear-walls or braced bents. (3)

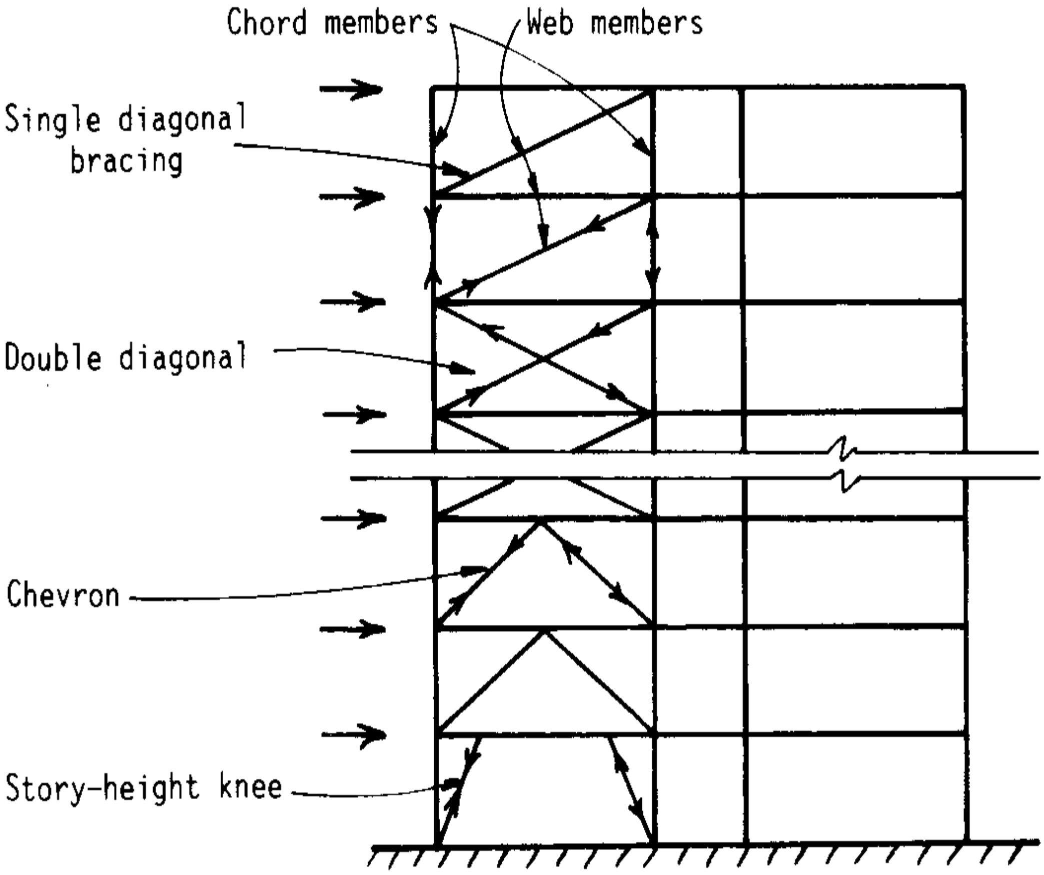

Figure (2.1): Typical types of bracing in tall building frame. (3)

2.2.3 Infilled-Frame Structures

Infilled-frames are the most usual form of construction for tall buildings of up to 30-storeys in height. Column and girder framing of reinforced concrete or sometimes steel, is infilled by panels of brick-wall or cast-insitu concrete. When the infilled frame is subjected to lateral loading, the infilled behaves effectively as a strut along its compression diagonal to brace the frame as shown in **Figure (2.3)**. (3)

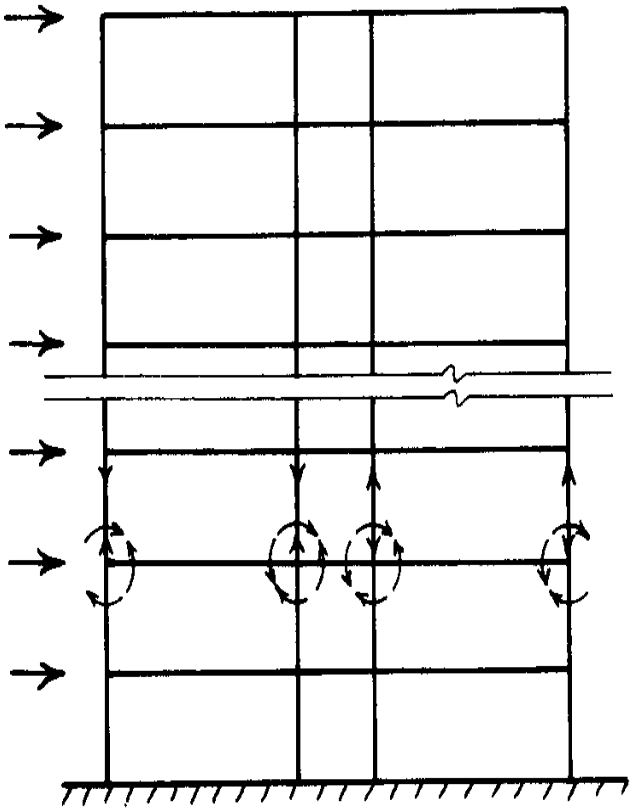

Figure (2.2): Typical example of rigid frame building. (3)

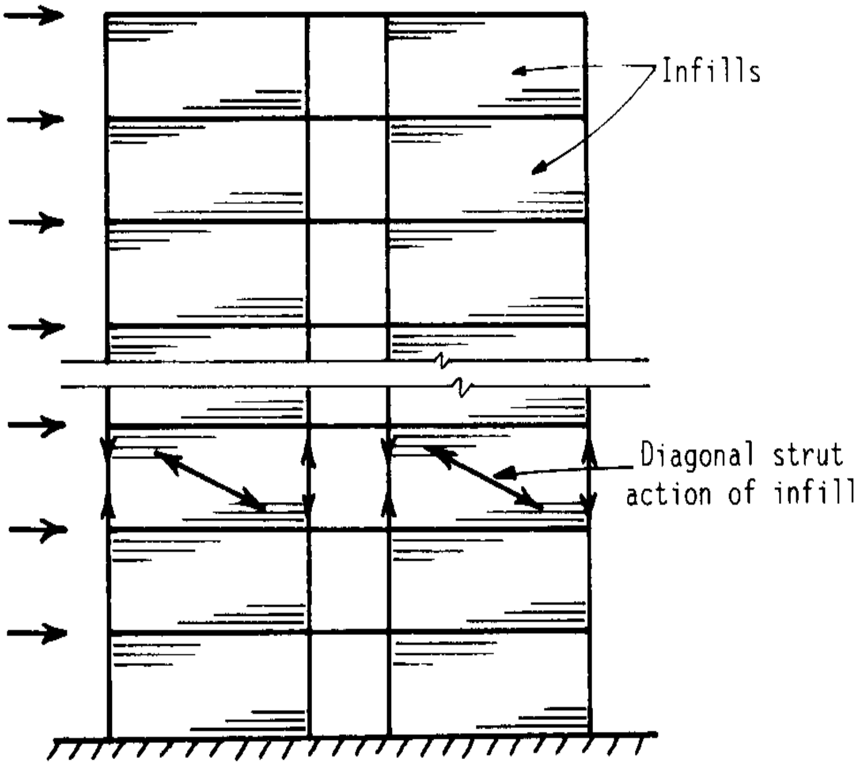

Figure (2.3): Typical example of infilled-frame structure. (3)

2.2.4 Flat Slab Structures

The flat slab structure consists of uniform slabs of about 15cm thickness, connected rigidly to supporting columns as shown in **Figure (2.4)**. Under lateral loading the behavior of a flat slab structure is similar to that of a rigid frame, that is, its lateral resistant depends on the flexural stiffness of the components and their connections, with the slab responding to the girders of the rigid frame. For lateral resistant on the flat slab buildings are economical up to about 25-storeys. [3]

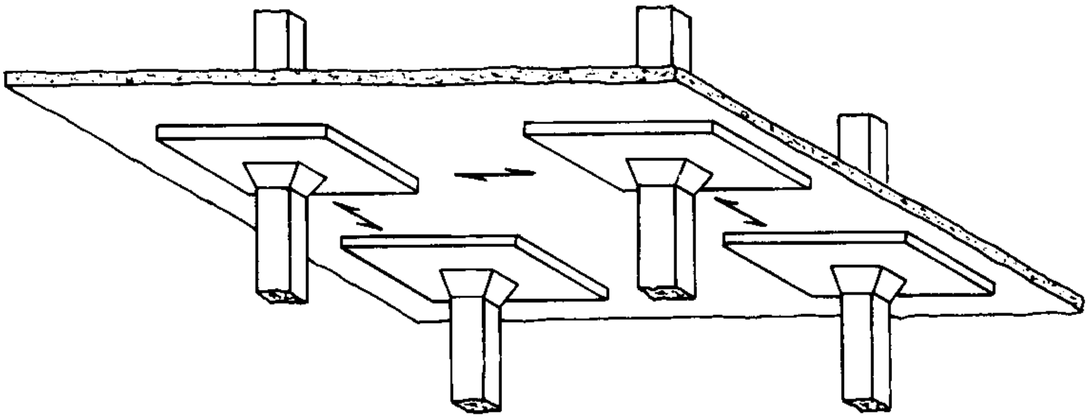

Figure (2.4): Typical example of flat slab with drop panel. [3]

2.2.5 Shear-wall Structures

I. General Shear-wall Structures

Shear-wall structures system consist of concrete or masonry .Vertical walls, may serve both as partition and structurally carry gravity and lateral loads. They act as vertical cantilevers in the form of separate planer walls and as non-planer assemblies of connected walls around elevator, stair and service shafts as shown in **Figure (2.5)**. Because they are much stiffer horizontally than rigid frames, shear-wall structures can be economical up to about 35-storeys. [3]

II. Coupled Shear-wall Structures

Coupled wall structures consist of two or more shear walls in the same plan or almost in the same plan connected at the floor levels by beams or stiff slabs as shown **Figure (2.6)**. The effect of the shear-resistant connecting members are to cause the set of walls to behave in their plan partly as a composite cantilever bending about the common centroid axis of the walls. This results in a horizontal stiffness very much greater than of the walls acted as a set of separate uncoupled cantilevers. [3]

2.2.6 Wall Frame Structure

When shear-walls are combined with rigid frames, the frame and walls, which tend to deflect in a flexural configuration in a shear mode, sequently, are constrained to adopt a common deflected shape by the horizontal rigidity of the girder and slabs .The interacting wall-frame combination is appropriate for buildings in the 40-60-storeys range as shown in **Figure (2.7)**, well beyond that of rigid frames or shear-walls alone. [3]

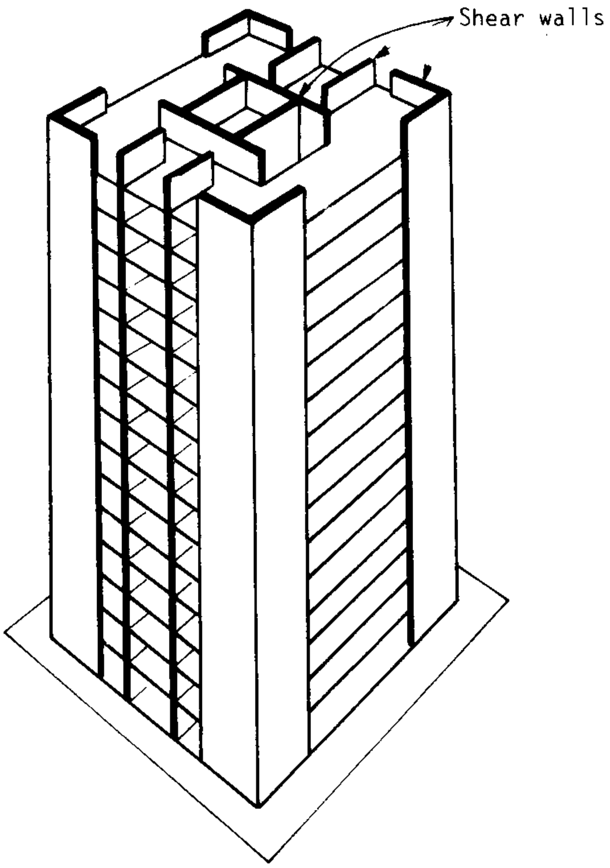

Figure (2.5): Typical example of shear-wall frame structure. [3]

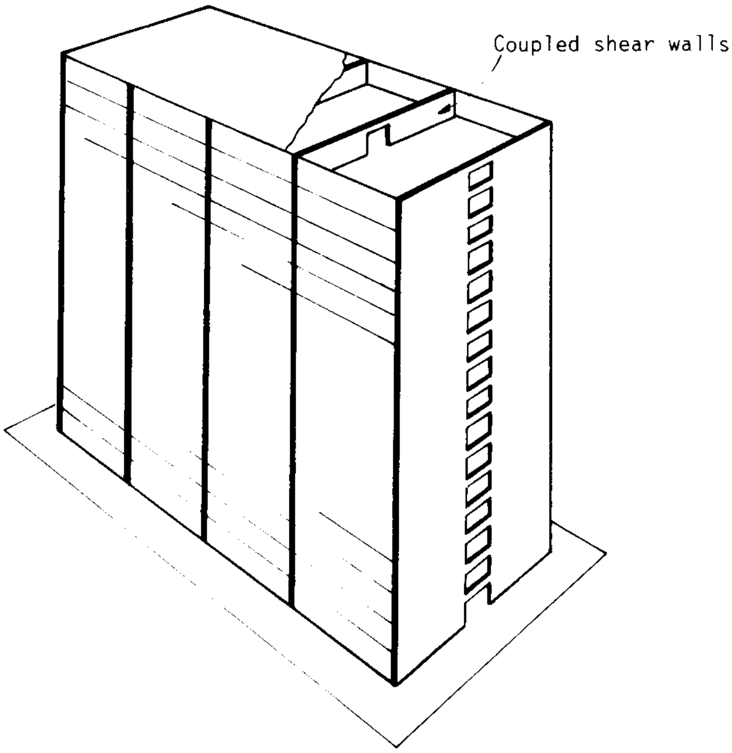

Figure (2.6): Typical example of coupled shear-wall frame structure. (3)

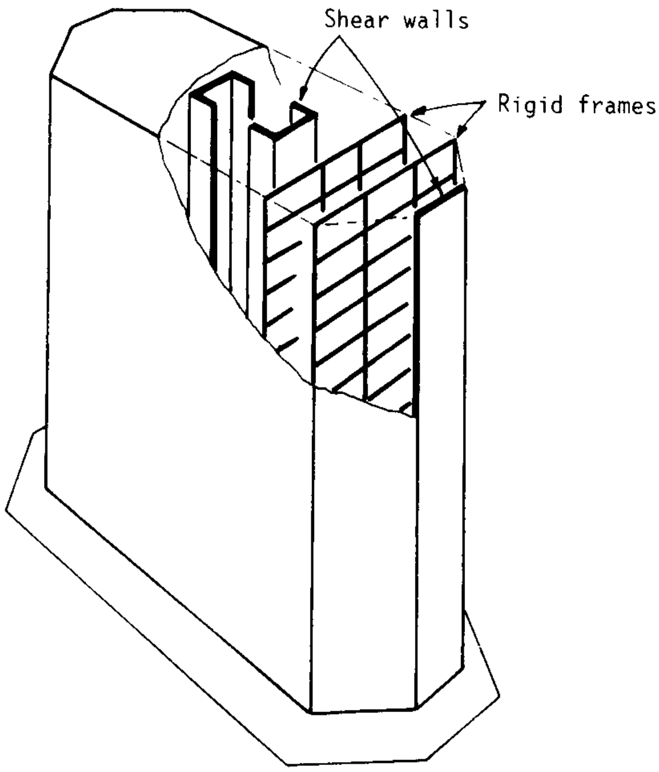

Figure (2.7): Typical example of wall-frame structure. (3)

2.2.7 Tube System Structures

The lateral resistant of framed tube structures is provided by a very stiff moment resisting frames that formed by a tube around the parameter of the building. The frames consist of closely spaced columns 2-4m between centers, joined by deep spandrel girders as shown in **Figure (2.8)**. Although the tube carries all the lateral loading, the gravity loading is shared between the tube and interior columns or walls. The tube form is suitable for both steel and reinforced concrete construction and has been used for buildings ranging from 40 to more than 100-storeys. The main advantage is to offer a relatively efficient, easy constructed structure and appropriate for up to the greatest height. [3]

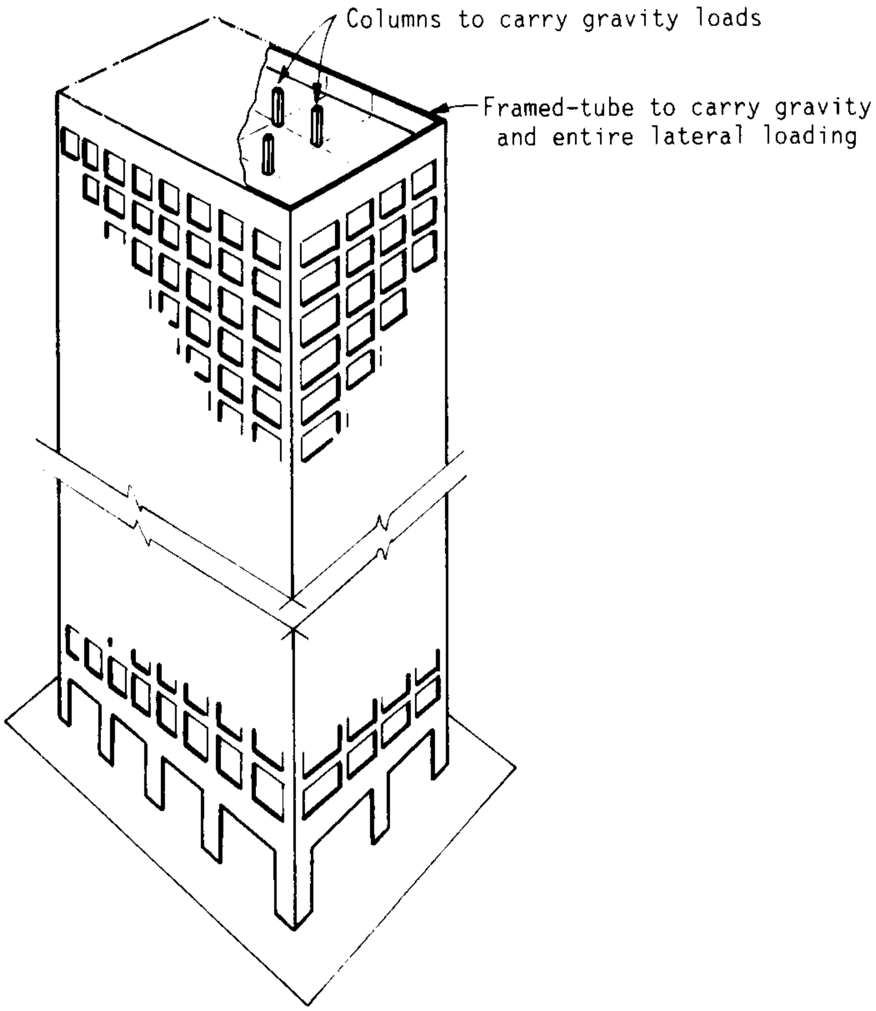

Figure (2.8): Typical example of tube-frame structure. [3]

A. Tube-in-Tube Structures

This variation of the framed tube consists of an outer framed tube, the "hull" together with an internal elevator and service core as shown in **Figure (2.9)**. The hull and core act jointly in resisting both gravity and lateral loading. In steel structures the core may consist of braced frames, whereas in a concrete structure it would consist of an assembly shear-walls. [3]

B. Bundled-tube Structures

This structural form used in one of the tallest buildings in the world, Sears's tower in Chicago. The Sears tower consists of four parallel rigid steel frames in each orthogonal direction, interconnected to form nine bundled tube as shown in **Figure (2.10)**. [5]

C. Braced-tube Structures

To improve the efficiency of the framed tube and increase its potential for use to even greater heights as well as allowing greater spacing between the columns is to add diagonal bracing to the face of the tube. In reinforced concrete structure the bracing is formed by a diagonal pattern of concrete window-size panels, poured integrally with the frame, whereas in the steel tube the bracing traverses the faces of the rigid frames as shown in **Figure (2.11)**. As a result, the structure behave under lateral loading more like a braced frame, with greatly dimensioned bending in the members of the frames. Consequently, the spacing of the columns can be larger and the depth of the spandrels less, thereby allowing larger size windows then in the conventional tube structure. [5]

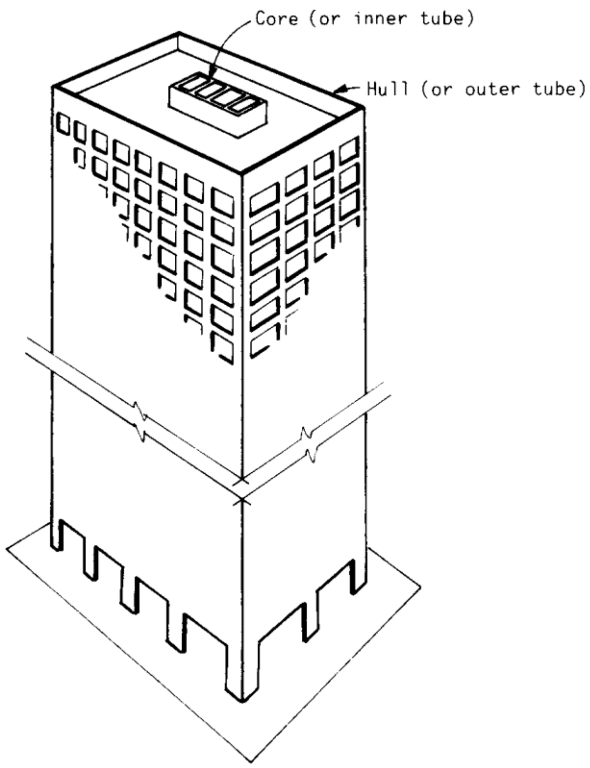

Figure (2.9): Typical example of tube-in-tube frame structure. (3)

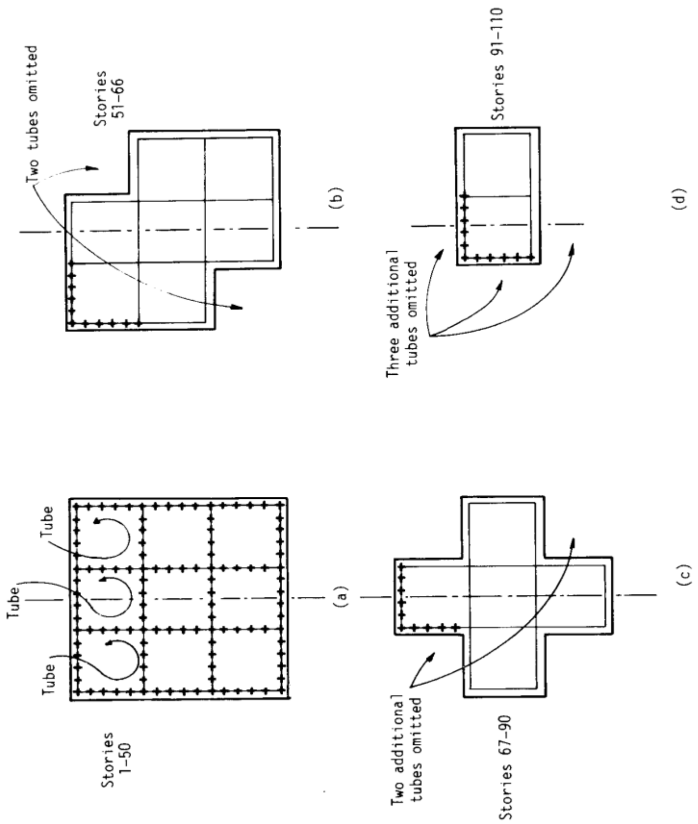

Figure (2.10): Typical example of bundled-tube frame structure. (3)

2.2.8 Outrigger-braced Structures

This efficient structural form consists of a central core, compressing either braced frames or shear walls, with horizontal cantilever “outrigger” trusses or girders connecting the core to the outer columns as shown in **Figure (2.12)**. The effective structural depth of the building is greatly increased, thus augmenting the lateral stiffness of the building and reducing the lateral deflections and moments in the core. In effect, the outriggers join the columns to the core to make structure behave as partly composite cantilever. In general, outrigger-braced structures have been used for buildings from 40-70-storeys high, but the system should be effective and efficient for much greater highs. (5)

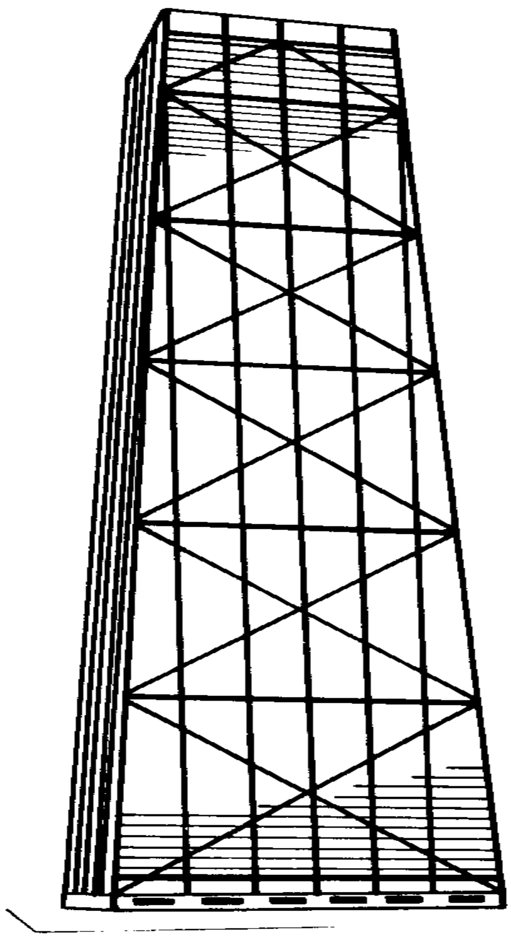

Figure (2.11): Typical example of steel braced tube frame structure. (3)

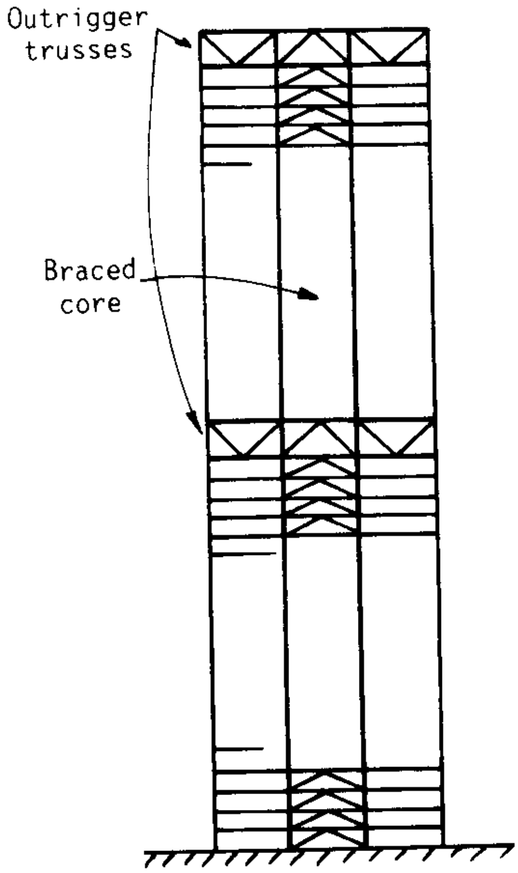

Figure (2.12): Typical example of outrigger frame structure. [3]

2.2.9 Core Structures

In these structures a single core serves to carry the entire gravity and horizontal loading as shown in **Figure (2.13)**. In some, the slabs are supported at each level by cantilever from the core. In others, the slabs are supported between the core and parameter columns, which terminated either on major cantilever at intervals down the height, or on a single massive cantilever a few stories above the ground. [3]

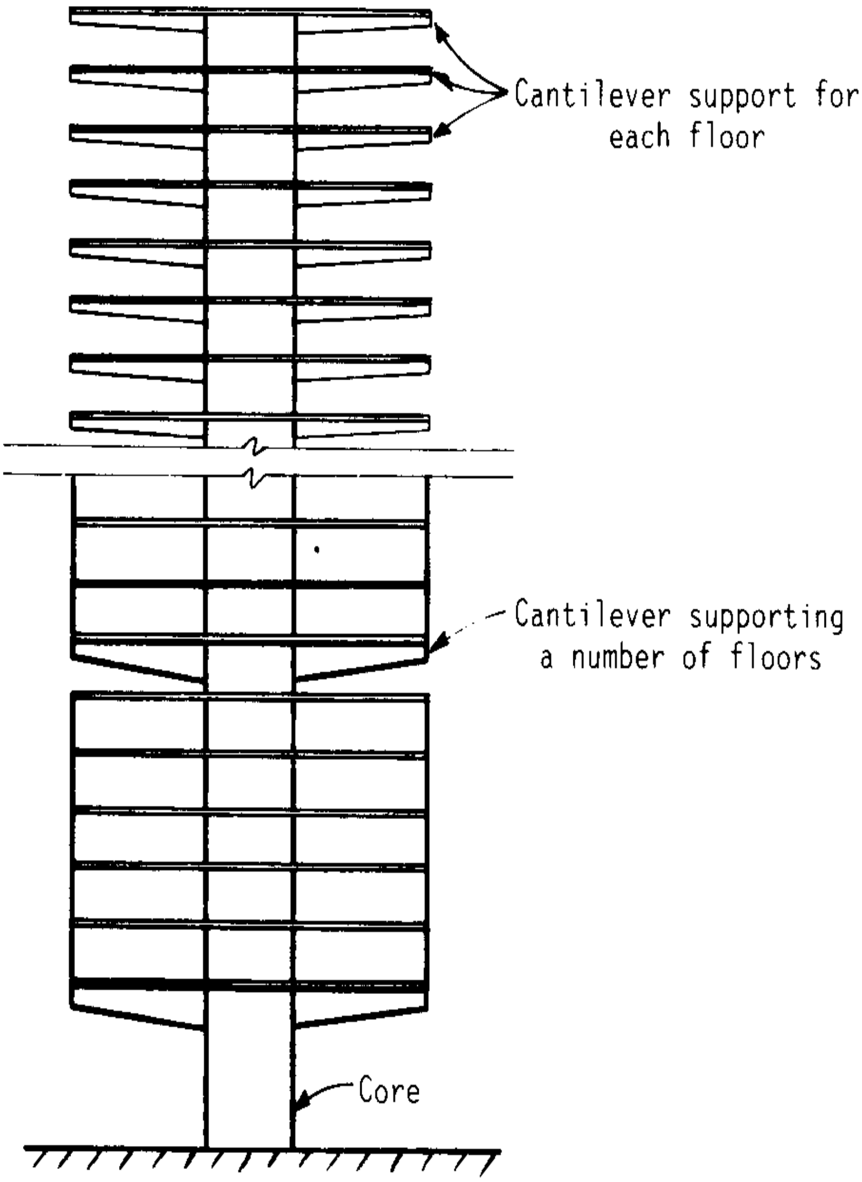

Figure (2.13): Typical example of core frame structure. [3]

2.2.10 Suspended Structures

The suspended structure consists of a central core, or cores, with horizontal cantilever at roof level, to which vertical hangers of steel cables, rod or plate are attached. The floor slabs are suspended from hangers as shown in **Figure (2.14)**. The advantages of this structural form are primarily architectural in that, the ground storey can be entirely free of major vertical members, except for presence of the central core. Also, the hangers, they are intension and consequently can be high strength steel, have a minimum sized section and are therefore less obtrusive. [3]

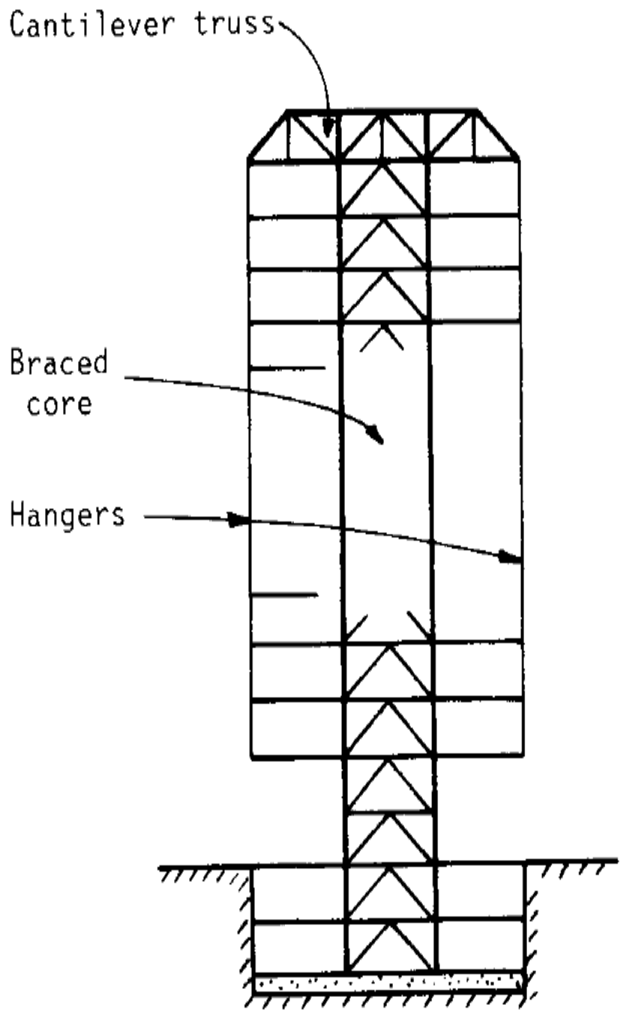

Figure (2.14): Typical example of suspended frame structure. [3]

2.2.11 Spaced Structures

The spaced structure consists essentially of a three-dimensional triangulated frame whose members serve dually resisting both gravity and horizontal loadings. One of the examples of spaced structures is shown in **Figure (2.15)**. The behavior of space structure in resisting both gravity and lateral load is relatively complex. One of the solution is to have an inner braced core, which serve to collect the lateral loading, and the inner region gravity loading, from the slabs over a number of multistory regions. [3]

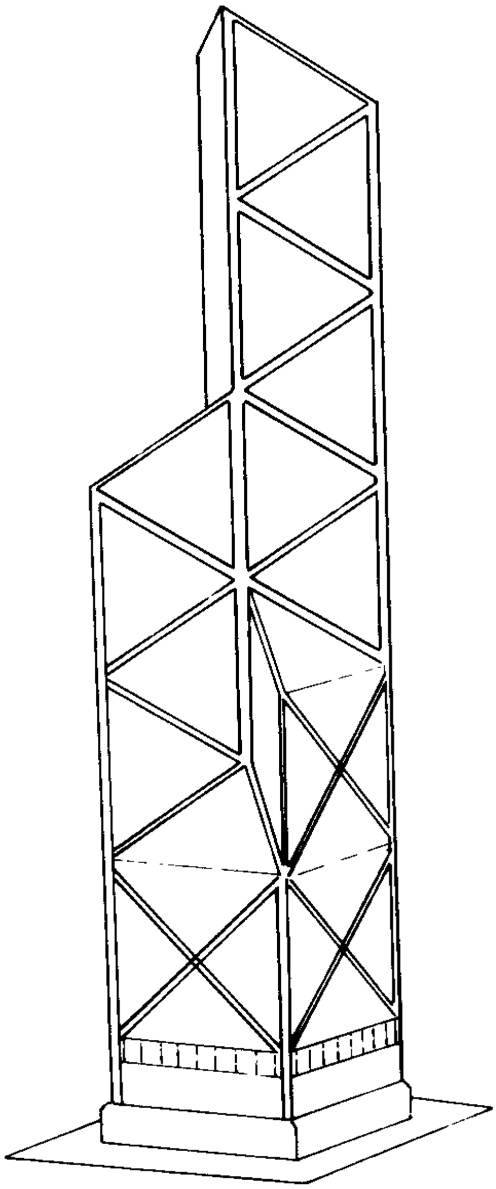

Figure (2.15): Typical example of spaced frame structure. (3)

2.3 Braced Frames

Bracing is a highly efficient and economical method of resisting horizontal forces in a frame structure. A braced bent consists of the usual columns and girders, whose primary purpose is to support the gravity loading, and diagonal bracing members that are connected so that the total set of members forms a vertical cantilever truss to resist the horizontal loading. The braces and girders act as the web members of the truss, while the columns act as the chords. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear. (5)

2.3.1 Types of Bracing System

Diagonal bracing is inherently obstructive to the architectural plan and can pose problems in the organization of internal space and traffic as well as in locating window and door openings. In many locations the type of bracing has to be selected primarily on the basis of allowing the necessary openings through the bay, often at the expense of efficiency in resisting the lateral forces. The most efficient, but also the most obstructive, types of bracing are those that form a fully triangulated vertical truss. These include the single-diagonal, double diagonal, and K-braced types as shown in **a, b, c, d and e** in **Figure (2.16)**. The full-diagonal types of braced bent are usually located where passage is not required, such as beside and between elevator, service, and stair shafts, which are unlikely to be relocated in the lifetime of the building. Other types of braced bent that allow window and door openings, but whose arrangements cause bending in the girder, are shown in **f, g, h, i, k and l** in **Figure (2.16)**. Some other types, which introduce bending in both the columns and the girders, are shown in **m, n and p** in **Figure (2.16)**. Generally, the types of braced bent that respond to lateral loading by bending of the girders, or of the girders and columns, are laterally less stiff and, therefore, less efficient, weight for weight, than the fully triangulated trusses, which respond with axial member forces only. (3)

2.4 Types of Loading on Tall Buildings

The loading on tall buildings differ from loading on low-rise buildings in its accumulation into much larger structural forces, in the increased significations of wind loading, and in the essentiality importance of dynamic effects. The gravity loadings over the large numbers of stories produced column loads of an order higher than those in low-rise buildings. Wind loading on tall buildings have larger effect than low-rise buildings, and greater moment intensity on the base. It is difficult to estimate all various loading types of tall buildings because of the variety of methods in the different codes of practice.

2.4.1 Gravity Loads

1. Dead Load

As in a low-rise buildings, dead loading is calculated from the members and estimated material densities. In general, dead loads consist of the weight of all materials of construction incorporated into the building including, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items, and fixed service equipment including the weight of cranes. (1)

2. Live Load

Live load is specified as the intensity of a uniformly distributed floor loads, according to the occupancy or use of the space. In certain situations such as in parking areas, offices and plant rooms the floors should be considered for the alternative worst possibility of specified concentrated load. The magnitude of live loading specified in the codes are estimates based on many statistical process on large number of buildings to get combination of experience and results of typical field services. Load capacity experiments have shown that even the code values, which are accepted as conservative, may in some critical circumstances underestimate the maximum possible values of live loadings. **Table (2.1)** shows the minimum distribution of live loads L_0 and the minimum concentrated live loads.

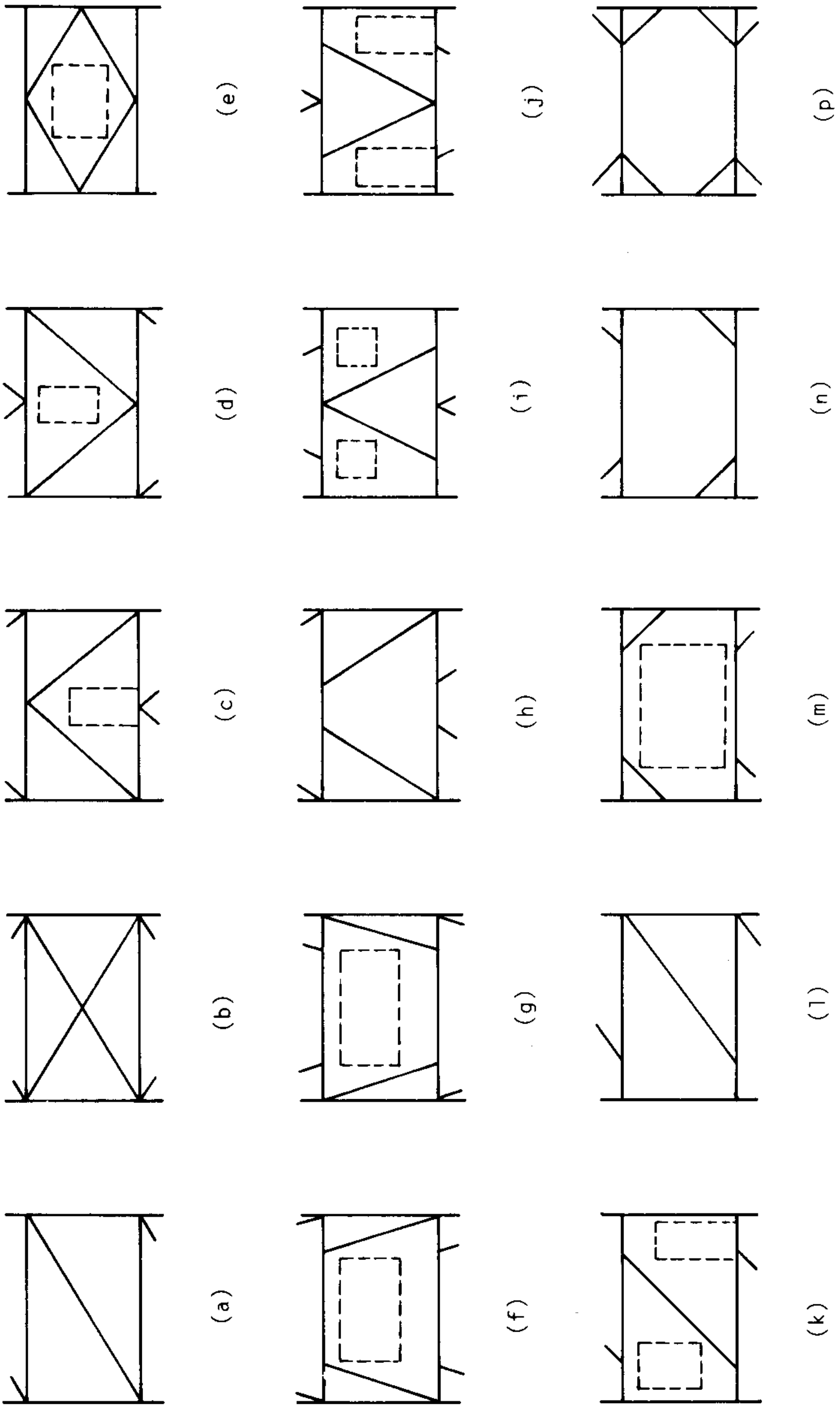

Figure (2.16): Typical examples of bracing systems. (3)

The philosophy of live load reduction is that although, at some time in the life of a structure, it is probable that a small area may be subjected to the full intensity of live load, it is improbable that the whole of a large area or a collection of areas, and the members supporting them, will be subjected simultaneously to the full live load. Consequently, it is reasonable to design the girders and columns supporting a large tributary area for significantly less than the full live loading. There are different methods of live load reduction, but generally allow different elements to reduce proportion of the fully live load with an increase the amount of supported area. A method that used is stated in ASCE 10-7 as follow:

$$L = L_0(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}) \qquad \text{Equation (4.7-1)}$$

Where:

$L\equiv$ Reduced design live load per m^2 of area supported by the member.

$L_0\equiv$ Unreduced design live load per m^2 of area supported by the member **(Table 2.1)**.

$K_{LL}\equiv$ Live load element factor **(Table 2.2)**.

$A_T\equiv$ Tributary area in m^2.

L shall not be less than $0.5L_0$ for members supporting one floor, and L shall not be less than $0.4L_0$ for members supporting two or more floors. (1)

2.4.2 Wind Load

The wind is air in motion. Obstacles in the path of wind, such as buildings and other topographic features deflect or stop wind, converting the wind's kinetic energy into potential energy of pressure, thereby creating wind load. The major factor that causes differ in design of high-rise buildings is the lateral effect due to lateral loads such as wind or seismic loading. There is not obvious height to classify any building a high–rise building. But, it depend essentially on the slenderness ratio of the building, that make the effect of lateral loading get clear to observe and measure. In order to calculate the wind loads pressure for design building, ASCE 7-10 states to the following steps: [1]

Step 1: Determine Risk Category of Building

Buildings and other structures shall be classified, based on the risk to human life, health, and welfare associated with their damage or failure by nature of their occupancy or use as shown in **Table (A.1)** at **Appendix (A)**. [1]

Step 2: Determine the Basic Wind Speed V for the Applicable Risk Category

The basic wind speed, V, used in the determination of design wind loads on buildings and other structures shall be determined as follows:

- For Risk Category II buildings and structures – use **Figure (A.1)** in **Appendix (A)**. [1]
- For Risk Category III and IV buildings and structures. [1]
- For Risk Category I buildings and structures. [1]

Step 3: Determine Wind Load Parameters

1- Wind Directionality Factor K_d:

The wind directionality factor, $K_{d,}$ shall be determined from **Table (A.2)** as shown in **Appendix (A)**. [1]

2- Exposer Category

For each wind direction considered, the upwind exposer shall be based on ground surface roughness, and constructed facilities. [1]

Exposure B for buildings with a mean roof height of less than or equal to 9.1 m, Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 457 m. For buildings with a mean roof height greater than 9.1 m, Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 792 m or 20 times the height of the building, whichever is greater. [1]

Table (2.1): Minimum uniformly distributed live loads L_0 and concentrated load. (1)

Occupancy or Use	Uniform psf (kN/m^2)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m^2)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Table (2.2): Live load element factor K_{LL}. (1)

Element	K_{LL}[a]
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified, including:	1
Edge beams with cantilever slabs	
Cantilever beams	
One-way slabs	
Two-way slabs	
Members without provisions for continuous shear transfer normal to their span	

[a]In lieu of the preceding values, K_{LL} is permitted to be calculated.

Exposure C shall apply for all cases where Exposures B or D do not apply (1)

Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 183 m or 20 times the building height, whichever is greater, from an Exposure D condition as defined in the previous sentence. (1)

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used. (1)

3- Topographic Factor K_{zt}

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography, located in any exposure category, and shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

1. The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100H) or 3.22 km, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined. (1)
2. The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3.22 km radius in any quadrant by a factor of two or more. The structure is located as shown in **Figure (A.2)** in **Appendix (A)** in the upper one-half of a hill or ridge or near the crest of an escarpment. (1)
3. $H/L_h \geq 0.2$. (1)
4. *H* is greater than or equal to 4.5 m for Exposure C and D and 18 m for Exposure B. (1)

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor *Kzt*:

$$\boldsymbol{K_{zt} = (1 + K_1 K_2 K_3)^2} \qquad \text{Equation (26.8-1).}$$

Where K_1, K_2, and K_3 are given in **Figure (A.2)**.

If site conditions and locations of structures do not meet all the conditions then $K_{zt} = 1.0$. (1)

4- Gust Effect Factor *G*

The gust-effect factor for a rigid building or other structure is permitted to be taken as 0.85. (1)

5- Internal Pressure Coefficient *GC*

Internal pressure coefficients, GC_{Pi}, shall be determined from **Appendix (A)** in **Table (A.4)** based on building enclosure classifications determined from the shape of building state. (1)

Step 4: Determine Velocity Pressure Exposure Coefficient K_z

Based on the exposure category determined a velocity pressure exposure coefficient K_z, as applicable, shall be determined from **Table (A.5)** as shown in **Appendix (A)**. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z, between those shown in **Table (A.5)** are permitted provided that they are determined by a rational analysis method defined in the recognized literature. [(1)]

Step 5: Determine External Pressure Coefficient C_p

External pressure coefficient from **Figures (A.3)** as shown in **Appendix (A)**. [(1)]

Step 7: Calculate Wind Pressure *p* on Each Building Surface

Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGCp - qi(GCPi) \ \ (N/m^2) \qquad \text{Equation (27.4-1)}$$

Where:

$q \equiv q_z$ for windward walls evaluated at height z above the ground

$q \equiv q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i \equiv q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings. [(1)]

$q_i \equiv q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering shall be treated as an opening in accordance with Section 26.10.3. For positive internal pressure evaluation, qi may conservatively be evaluated at height h ($q_i = q_h$). [(1)]

2.5 Loads Combinations

The load combinations and load factors given below shall be used only in those cases in which they are specifically authorized by the applicable material design standard. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations: (1)

I. $1.4D$.

II. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$.

III. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$.

IV. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$.

V. $1.2D + 1.0E + L + 0.2S$.

VI. $0.9D + 1.0W$.

VII. $0.9D + 1.0E$.

Where:

$D \equiv$ dead load.

$E \equiv$ earthquake load.

$L \equiv$ live load.

$L_r \equiv$ roof live load.

$R \equiv$ rain load.

$S \equiv$ snow load.

$W \equiv$ wind load.

2.6 Storey Drift

Drift (lateral deflection) in a building is a serviceability issue primarily from the effects of wind loads. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural parts in the building. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the total building drift, defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level [5]

$$\Delta/H.$$

For each floor, the applicable parameter is *interstory drift,* defined as the lateral deflection of a floor relative to that of the floor immediately below, divided by the distance between floors [5]

$$(\delta - \delta n{-}1)/h.$$

Typical drift limits most widely used in wind design are *H* (or *h*)/400 to *H* (or *h*)/500 where *H* is the height of the building and *h* is the height of the floor. An absolute limit on interstory drift is sometimes imposed in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the interstory drift exceeds about 10 mm, unless special details are provided to accommodate larger movements. It should be noted that many cladding components can accept deformations that are significantly larger. It is important to recognize that frame racking or shear distortion is the real cause of damage to building elements such as cladding and partitions, and that drift control limits by themselves in wind-sensitive buildings do not provide comfort of the occupants under wind load. [5]

ASCE 7-10 state that, the design storey drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the storey under consideration as shown in **Figure (A.4)** in **Appendix (A)**. [1]

Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the storey based on the vertical projection of the center of mass at the top of the storey. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces due to wind load as specified in Section 12.8 of ASCE 7-10 without reduction for allowable stress design. The deflection at Level *x* (δ_x) mm used to compute the design storey drift Δ, shall be determined in accordance with the following equation: (1)

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad \text{Equation (12.8-15)}$$

Where:

C_d ≡ The deflection amplification factor in **Table (A.6)** in **Appendix (A)**.

δ_{xe} ≡ The deflection at the location required by this section determined by an elastic analysis.

I_e≡ The importance factor determined in accordance with Section 11.5.1 of ASCE 7-10.

Also, it's important to be mentioned that δ_x shall not exceeded the allowance of the storey drift (Δ_a) that obtained from **Table (A.6)** in **Appendix (A)**. (1)

2.7 Structural Steel Specifications

According to AISC the structural steel was commonly used in four types of H-shaped or I-shaped as follow:

- W-shapes, which have essentially parallel inner and outer flange surfaces. (6)
- M-shapes, which are H-shaped members that are not classified in ASTM A6 as W-, S-, or HP-shapes. M-shapes may have a sloped inside flange face or other cross-section features that do not meet the criteria for W-, S-, or HP-shapes. (6)
- S-shapes (also known as American standard beams), which have a slope of approximately $16\frac{2}{3}$ percent (2 on 12) on the inner flange surfaces. (6)
- HP-shapes (also known as bearing piles), which are similar to W-shapes, except their webs and flanges are of equal thickness and the depth and flange width are nominally equal for a given designation. (6)

These shapes are designated by the mark W, M, S or HP, nominal depth (in.) and nominal weight (lb. /ft.). For example, a W24x55 is-a W-shape that is nominally 24 in. deep and weighs 55 lb. /ft. The dimensional and property information that given in **Appendix (A)** for the W- shapes that covered in ASTM A6 because it's most common type of structural steel used. **Table (2.3)** shows the mechanical properties of these structural steel shapes. Note that, each type of structural steel has a unique standard stated according to ASTM specifications. Also, ASTM A992 is the most commonly referenced specification for W- shapes. [(6)]

2.8 The Structural Analysis Computer Software (ETABS)

Extended **T**hree-dimensional **A**nalysis of **B**uilding **S**ystem (ETABS) is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. The innovative and revolutionary new Etabs is the ultimate integrated software package for the structural analysis and design of buildings. Incorporating 40 years of continuous research and development, this latest Etabs offers unmatched 3D object based modeling and visualization tools, blazingly fast linear and nonlinear analytical power, sophisticated and comprehensive design capabilities for a wide-range of materials, and insightful graphic displays, reports, and schematic drawings that allow users to quickly and easily decipher and understand analysis and design results. Etabs is a completely integrated system. Embedded beneath the simple, intuitive user interface are very powerful numerical methods, design procedures and international design codes, all working from a single comprehensive database. This integration means that you create only one model of the floor systems and the vertical and lateral framing systems to analyze and design the entire building. Etabs provides an unequaled suite of tools for structural engineers designing buildings, whether they are working on one-storey industrial structures or the tallest commercial high-rises. Once modeling is complete, Etabs automatically generates and assigns code-based loading conditions for gravity, seismic, wind, and thermal forces. Users may specify an unlimited number of load cases and combinations. Analysis capabilities then offer advanced nonlinear methods for characterization of static-pushover and dynamic response. Dynamic considerations may include modal, response-spectrum, or time-history analysis. P-delta effect account for geometric nonlinearity. Given enveloping specification, design features will automatically size elements and systems, design reinforcing schemes, and otherwise optimize the structure according to desired performance measures.

Table (2.3): Steel grades according to American standards. [7]

Standards	Grades	Yield Strength Re	Tensile Strength Rm	Ratio Re / Rm	Minimum elongation		Notch impact test	
					Min. 200 mm (8 in)	Min. 50mm (2 in)	ASTM A673, standard position longitudinal flange	
		MPa (ksi)	MPa (ksi)				Temperature °C (°F)	Energy average J (ft-lbf)
A36-04b	A36	≥250 [36]	400-550 [58-80]	-	20	21	-	-
A572-04	Grade42 Grade50 Grade55 Grade60 Grade65	≥290 [42] ≥345 [50] ≥380 [55] ≥415[60] ≥450[65]	≥415 [60] ≥450 [65] ≥485 [70] ≥520 [75] ≥50 [80]	-	20 18 17 16 I5	24 21 20 18 17	-	-
A588-04	Grade B Grade C	≥345[50] ≥345[50]	≥485 [70] ≥485[70]	-	18 18	21 21	-	-
A709-04a	Grade36 Grade50 Grade50S	≥250[36] ≥345[50] 345-450 [50-65]	400-550 [58-80] ≥450[65] ≥450[65]	≤0.85	20 18 18	21 21 21	-	-
A913-04	Grade50 Grade65	≥345[50] ≥450[65]	≥450 [65] ≥550 [80]	≤0.85	18 I5	21 17	21 [70] 21 [70]	≥54[40] ≥54[40]
A992-04a	A992	345-450 [50-65]	≥450 [65]	≤0.85	18	21		