Chapter 2

High-Rise Office Buildings: Systems, Parameters and Variables

2.1 INTRODUCTION

Factors that affect decisions made in the design of high-rise buildings are primarily initiated by the interests of the different parties involved, as follows (Guise 1990):

1. Owner:
   - Market Feasibility
   - Financial feasibility
2. Architect:
   - Spatial requirements of building envelope and services
   - Quality and cost of internal environment
3. Structural Engineer:
   - Gravity and Lateral load systems
   - Foundations
4. Mechanical/ Electrical Engineer:
   - Hydraulics/ Piping
   - Electrical/ Lighting
   - Elevators/ lifts
   - HVAC
   - Energy consumed by service systems
5. Construction Engineer:
   - Labor/ Equipment
   - Time/ Climate

Globalization of building optimality is yet difficult to achieve because of the lack of agreement across the industry for standard global models. Often, the optimization interests of the parties involved in the design are in conflict. For example, an architect wants maximum flexibility of floor space usage and high comfort level while a structural engineer desires the most economical and safe structure. It is apparent that optimum floor flexibility may conflict with having the lightest structure as column and girder layouts that achieve a least-weight structure may have an adverse impact on floor space usage. As another example, by increasing the height of a building for constant required area, the building footprint and, hence, the land cost will decrease but the structural, vertical transportation and façade costs will increase. Moreover, even the type of structural system and material may change with height of a building, as illustrated in Figure 2.1 (Khan 1974). For constant floor area, a taller building means a smaller footprint, which then implies the use of mat foundations or piles in lieu of less expensive spread footings. Furthermore, for a fixed required floor area, the more slender a building is in one direction the greater is its surface area on the perimeter, which causes increased capital and operating costs for heating, ventilation and air conditioning (HVAC) systems. At the same time, a greater perimeter means more access to daylight which decreases the lighting expenses and the heat generated by the lighting system and increases the quality of the space and the comfort level of occupants. This results in decreased HVAC cost during summertime and increased HVAC costs during wintertime. Conversely, the increased absorbed energy from the sun causes more spending on HVAC systems during summertime and less spending during wintertime. It is also known that the occupants of a high-rise building are generally negligent in turning off lights, even if there is enough light from outside, and, therefore, to benefit from daylight it is necessary to install an automated system which dims the lights in the presence of enough daylight, which will itself increase the lighting capital costs.

Considering the interactions noted in the forgoing for but a few examples, one can see that the prediction of optimal conceptual design scenarios for a high-rise building is a very complicated task indeed. Fortunately, relatively recent advances in distributed computing paradigms have been shown to be well suited for the complex task of modelling the global conceptual design optimization problem.

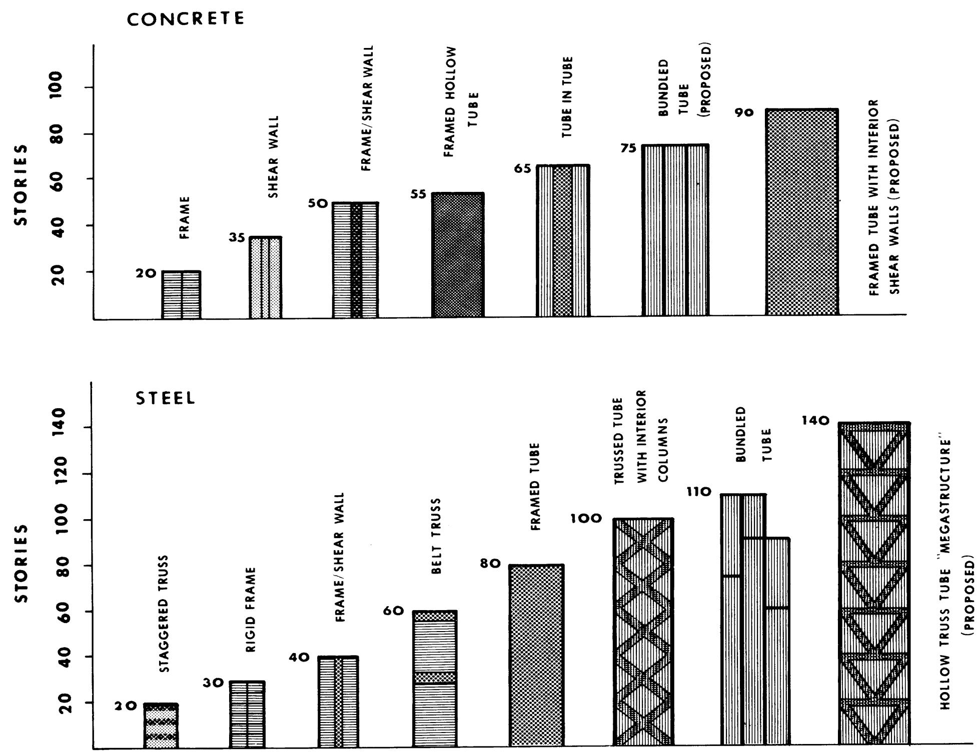

Figure 2.1: Different Structural Systems

2.2 SYSTEMS IN HIGH-RISE BUILDINGS

The first step towards optimizing a building is to identify its major systems. While an optimized high-rise building does not necessarily result from individually optimized systems, the identification of optimum individual major systems must be the first step prior to integrating these systems into the whole building. Structural, mechanical and electrical systems are the major systems for a high-rise building that are of primary concern to engineers.

2.2.1 Structural Systems for High-Rise Buildings

In general, the structural system of a building is a complex three-dimensional assemblage of various combinations of interconnected structural elements. The primary function of a structural system is to carry effectively and safety all the loads acting on the building, and eventually to transmit them to the foundations. A structural system is therefore expected to: carry dynamic and static vertical loads; carry horizontal loads due to wind and seismic effects; resist stresses caused by temperature and shrinkage effects; resist external or internal blast and impact loads; and resist vibration and fatigue effects. At the same time, the structural system is subject to the following requirements: it should conform with architectural requirements and those of the building's users and owner; it must interact with and facilitate service systems, such as heating, ventilating, air conditioning, horizontal and vertical transport, and other electrical and mechanical systems; it should facilitate simple and fast erection of the building; it must be resistant to fire; it must enable the building, foundations, and the ground to interact properly; and it should be economical.

A variety of factors has to be considered in the process of selecting the most suitable structural system for a high-rise building. The selection is a complicated process, and no simple clear-cut design procedures are available. The design team must use every available means, such as imagination, previous experience, and relevant literature to arrive at the best possible solution in each particular case.

There are several sub-systems common to all types of structural systems (steel, concrete, composite), namely:

1. Vertical load resisting systems: a)Floor systems; b)Columns
2. Horizontal load resisting systems
3. Structural joints
4. Energy dissipation systems (dampers)

In this study, only the first two subsystems will be investigated. The most frequently used structural systems for high-rise steel and concrete buildings are shown in Figure 2.1 (Khan, 1974). It can be observed that Figure 2.1 recommends different types of structural systems depending on the number of stories and the building material. In general, however, it is extremely difficult to apply accurately a classification system for structural systems of high-rise buildings.

As the height of a building increases, the design of its structural system becomes increasingly specialized and complex. A variety of factors, many of them difficult to identify at the schematic level, can have a major influence on the selection and design of a structural system; the immense vertical loads on the structure, the character of wind and earthquake forces applied to a building specific to the building site, the local foundation conditions and, on top of all, the relative cost of various construction systems within the region are all important factors that a structural engineer has to consider. For these reasons no serious attempt at the design of a high-rise structure should be made without the participation of a qualified structural engineer, even in the early phases of design. In general, for high-rise buildings designed for a similar purpose and of the same material and height, the efficiency of different structures can be compared roughly by their weight per unit floor area. In these terms, the weight of the floor framing is influenced mainly by the floor span and is virtually independent of the building height, while the weight of the columns, considering gravity load only, is approximately proportional to the height of the building, see Figure2.2 (Smith and Coull 1991). Buildings up to 10 stories designed for gravity loading can usually accommodate wind loading without any increase in design stresses for combined loading. For buildings of more than 10 stories, however, the additional material required for lateral load resistance increases nonlinearly with height so that for buildings of 50 stories and more the selection of an appropriate structural form may be critical for the economy and, indeed, the feasibility of the building (Smith and Coull 1991).

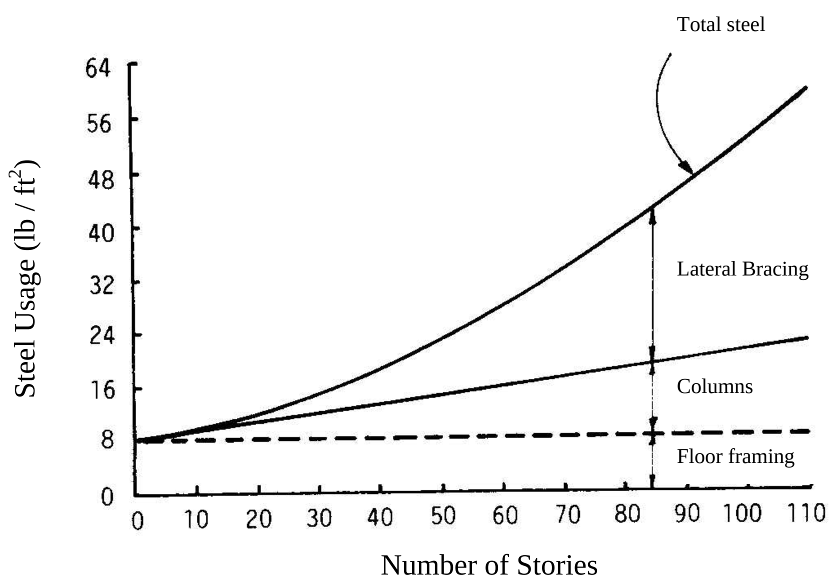

Figure 2.2: Use of Steel in Tall Buildings (Smith and Coull 1991)

Refer to Appendix 2.A for a description of the basic structural systems for tall buildings, and their relationships to the total design of the building, considered by this study.

2.2.2 Mechanical Systems

The most important mechanical systems in a tall building are: 1) Heating Ventilation and Air Conditioning (HVAC); and 2) vertical distribution services (elevators). Refer to Appendix 2.B where the mechanical systems considered by this study are discussed with the view to establish appropriate rules for their design within the context of high-rise office buildings.

2.2.3 Electrical Systems

The main components of the electrical system in an office buildings are: electrical outlets; lighting; and the electrical parts related to mechanical systems. Since the electrical parts related to mechanical systems are directly dealt with in the design of HVAC and elevator systems, this study is only concerned with electrical outlet and lighting systems. The electrical outlet system is dependent on the total area and function of the building and its cost for an office building is a function of total area and the unit cost for electrical outlets. The design of good lighting in buildings, daylight or artificial, is a matter of both quality and quantity. The architect in collaboration with the lighting engineer is concerned not only with providing enough light for the given tasks in each space but also with visual efficiency and comfort.

Lighting systems in a building can be categorized as: artificial lighting; and daylight. Identifying the best artificial lighting system is a straightforward task since it is an accepted fact that fluorescent lamps generate the best kind of lighting at a low cost for office buildings. Such lighting generally demands a level of illumination that consumes only about 20w/hr if recent lighting fixtures are employed (Reid, 1984). On the other hand, natural lighting or daylight is not available in all times, is less predictable and controllable than artificial lighting, varies with place, time and weather, and is not necessarily free because of the heat gain it causes through the windows. Daylight does have some significant advantages, however, such as decreasing internal energy consumption on sunny days and increasing the efficiency of the occupants of the building. The ratio of window area to the perimeter surface area of the building is an important factor in providing daylight of appropriate quantity and quality.

2.3 PARAMETERS AND VARIABLES FOR CONCEPTUAL DESIGN

The structural, mechanical and electrical systems discussed in this Chapter and related Appendices 2.A and 2.B give rise to the parameters and variables that govern and define the computer-based method for the conceptual design of high-rise office buildings developed in Chapter 3 and applied in Chapter 4.

2.3.1 Design Parameters

The basic design parameters considered by this study are defined by local location information, and are (e.g., see Table 4.1): land cost and property tax rates; office space lease rates; mortgage and inflation rates; electrical and gas energy unit costs; daylight factors; inside and outside temperatures and humidities; building geographical location and orientation; gravity and lateral loads; and cost location factors which relate US national average costs to the local cost of building components, see Table 2.1 (refer to Table 4.1 for representative number of these cost location parameters).

Table 2.1: Cost Location Factors

Cost Location Factor	**Description**
CCLF	Concrete Cost Location Factor (ratio of local concrete cost to US national average concrete cost)
C_LCLF	CLadding Cost Location Factor (ratio of local cladding cost to US national average cladding cost)
ECLF	Electrical Cost Location Factor (ratio of local electrical cost to US national average electrical cost)
E_LCLF	ELevators Cost Location Factor (ratio of local elevators cost to US national average elevators cost)
FCLF	Forming Cost Location Factor (ratio of local forming cost to US national average forming cost)
F_ICLF	FInishing Cost Location Factor (ratio of local finishing cost to US national average finishing cost)
MCLF	Mechanical Cost Location Factor (ratio of local mechanical cost to US national average mechanical cost)
RCLF	Reinforcement Cost Location Factor (ratio of local reinforcement cost to US national average reinforcement cost)
R_OCLF	ROofing Cost Location Factor (ratio of local roofing cost to US national average roofing)
SCLF	Steel Cost Location Factor (ratio of local steel cost to US national average steel cost)
WCLF	Windows Cost Location Factor (ratio of local windows cost to US national average windows cost)

Additional design-specific parameters considered by this study are defined by the building restriction limits, and are (e.g., see Table 4.1): a_{max}, b_{max} = maximum allowable footprint dimensions in the a and b directions for the building; H_{max} = maximum building height; A_{req} = minimum required area of lease/rental office space; h_{cle} = minimum permitted floor-to-ceiling clearance height; CPD_{min} = minimum permitted distance between building core and perimeter; $C_a \times C_b$ = core area as a fixed percentage of footprint area; $D_a / D_{b\ \mathrm{min}}$ = minimum aspect ratio allowed for the building; and $H/D_{a\ min}$ = maximum slenderness ratio allowed for the building.

2.3.2 Primary Design Variables

For given parameter values, the computer-based method for conceptual design developed by this study initially finds the values of a number of primary variables that define the architectural and structural systems for a high-rise office building. The primary variables (along with the ranges of possible alpha-numeric values they may be assigned) that are adopted by this study are listed in Table 2.2 in concise form, and are further elaborated upon in the following: *ST* = structural type (steel rigid frame, concrete rigid frame, steel frame and bracing, steel rigid frame and bracing, steel frame and concrete shear wall, steel rigid frame and concrete shear wall, concrete rigid frame and concrete shear wall, steel frame with bracing and outrigger trusses, steel framed tube, and concrete framed tube); *BT* = bracing type (K&K and K&X); *CFT* = concrete floor type (flat plate, flat slab, beam and slab, and waffle slab); *SFT* = steel floor type (steel joist and beam with steel deck and concrete slab, composite beam & cast-in-place slab, W-shape composite beam with steel deck and concrete slab, and composite beam with steel deck and concrete slab); S_a, S_b = the span distances between columns in the two orthogonal directions *a* and *b* of the building footprint (from 4.5m to 12m in increments of 0.5m); NS_a, NS_b = the number of column bays (from 3 to 10 in increments of 1); NTS_a, NTS_b = the number of tube column bays within the span distances S_a and S_b (from 2 to 5 in increments of 1); *DCDD* = direction of randomly chosen core dimension to be designed first (*a* or *b*); *CDF* = fraction of building dimension to be assigned to the *DCDD* core dimension (from 25%

to 80% in increments of 7.86%); *WIT* = window type (standard, insulated, standard heat absorbing and insulated heat absorbing); *WIR* = ratio of window area to maximum window area available on the surface of the building perimeter (from 25% to 100% in increments of 5%); and *WAT* = cladding type (pre-cast concrete, metal siding panel, stucco wall, glazing panel).

Table 2.2: Ranges of Primary Variable Values for the Conceptual Design of Office Buildings

Index	*ST*	*BT*	*CFT*	*SFT*	S_a,S_b (m)	NS_a NS_b	NTS_a, NTS_b (m)	*DCDD*	*CDF*	*WIT*	*WIR* %	*WAT*
0	([1]c) [3]Rigid frame	K&K	Flat plate	Steel joist & beam	4.5	3	2	*a*	0.250	Standard	25	Pre-cast concrete
1	(c)Rigid frame & shear wall	K&X	Flat slab	Com. beam & [5]CIP slab	5.0	4	3	*b*	0.329	Insulated	30	Metal siding panel
2	(c)Framed tube		Slab & beam	W & com. deck & slab	5.5	5	4		0.407	Standard [6]HA	35	Stucco wall
3	([2]s)Rigid Frame		Waffle slab	Com. beam, deck & slab	6.0	6	5		0.486	Insulated HA	40	Glazed panel
4	(s)[4]Frame & bracing				6.5	7			0.564		45	
5	(s)Rigid frame & bracing				7.0	8			0.643		50	
6	(s)Frame & (c)shear wall				7.5	9			0.721		55	
7	(s)Rigid frame & (c)shear wall				8.0	10			0.800		60	
8	(s)Frame, bracing & outriggers				8.5						65	
9	(s)Framed tube				9.0						70	
10					9.5						75	
11					10.0						80	
12					10.5						85	
13					11.0						90	
14					11.5						95	
15					12.0						100	

ST = Structural type (10 choices); *BT*= Bracing type (2 choices); *CFT* = Concrete floor type (4 choices); *SFT* = Steel floor type (4 choices); S_a, S_b = span distances between *columns along the building width a and length b (16 choices in each direction); NS_a, NS_b = number of column bays along the building width a and length b (8 choices in each direction); NTS_a, NTS_b = number of perimeter tube column spans within S_a and S_b; DCDD = direction of the core dimension to be designed first (2 choices); CDF = ratio of the core dimension to the overall length of the building in the same direction (8 choices); WIT= Window type (4 choices); WIR= Window ratio (16 choices); WAT = Wall cladding type (4 choices); [1]c = Concrete; [2] s = Steel; [3] Rigid frame = framework participates in carrying lateral loads; [4] Frame = framework does not participate in carrying lateral loads; [5] CIP = cast-in-place concrete; [6] HA = heat absorbing.*

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2.3.3 Secondary Design Variables

For given values of the design parameters and determined values of the primary design variables, the values of a number of secondary variables are calculated to complete the description of the conceptual design of an office building. These secondary variables, which are concisely listed in Table 2.3, are described in the following.

Table 2.3: Secondary Variables

Secondary Variables	Description
$ADBL_a$, $ADBL_b$	Average Distance Between column Lines in *a* & *b* directions
C_a , C_b	Core dimensions in *a* & *b* directions
CFA	Column-Free Area factor
D_a , D_b	Building Dimensions in *a* & *b* directions (m)
DF	Depth of Floor (m)
H	Height of the building (m)
HF	Height of Floor
NCL_a , NCL_b	Number of Column Lines between the perimeter and core of the building in *a* & *b* directions
NE	total Number of Elevators
NF	Number of Floors
NOPF	Number of Occupants Per Floor
NRF, *NMF*	Number of Rentable and Mechanical Floors
NRSC	Number of Risers in a Stair Case for one floor
NSC, *WSC*	Number and Width of Stair Cases
NSE, *NPE*,	Number of Service and Passenger Elevators
OILSC	Overall Inside Length of Stair Case
OIWSC	Overall Inside Width of Stair Case
TCS_a , TCS_b	Tube Column Spans in *a* & *b* directions
TNO	Total Number of Occupants

Knowing the values of the primary variables S_a, S_b, NS_a and NS_b the building width D_a and length D_b are found as,

$$D_a = NS_a \times S_a \tag{2.1a}$$

$$D_b = NS_b \times S_b \tag{2.1b}$$

Having D_a and D_b from Eq. (2.1), and knowing the required floor area A_{req} (see Appendix 2.A), it is assumed that 20% of the floor area is taken by the core, and that 4% of the total area of a building is needed for mechanical floors, such that the number of rentable floors NRF and mechanical floors NMF are found as,

$$NRF = (A_{req} \times 1.25 / (D_a \times D_b))_{Rounded\ up} \tag{2.2a}$$

$$NMF = (A_{req} \times 1.25 \times 0.04 / (D_a \times D_b))_{Rounded} \tag{2.2b}$$

where the minimum acceptable value of NMF is unity (1), and total number of floors NF is then found as,

$$NF = NRF + NMF \tag{2.2c}$$

To find the height of the building, this study assumes that the depth of false ceiling is one-half meter (0.5m) and, for known depth of floor DF and specified floor-to-ceiling clearance height h_{cle}, finds the height of each floor HF to be

$$HF = (h_{cle} + DF + 0.5) \tag{2.3a}$$

Then, having NF and HF from Eqs. (2.2c) and (2.3a), the total height H of the building is found as,

$$H = HF \times NF \tag{2.3b}$$

Core dimensions are chosen to satisfy the requirement that the core area be 20% of the total floor area at each story level. This is achieved by randomly choosing one dimension of the core to be a fraction CDF of the dimension D_a or D_b of the building footprint in that direction (see Appendix 2.A), and then calculating the other core dimension to meet the required core area. For example, if the randomly chosen core direction $DCDD = a$ (see Table 2.2), the dimensions C_a and C_b of the core area are found as follows, in the order shown,

$$C_a = D_a \times CDF \tag{2.4a}$$

$$C_b = (0.2 \times D_a \times D_b) / C_a \tag{2.4b}$$

For known number of tube column spans NTS_a and NTS_b, within S_a and S_b, the corresponding distances between the tube columns are found as,

$$TCS_a = S_a / NTS_a \tag{2.5a}$$

$$TCS_b = S_b / NTS_b \tag{2.5b}$$

The minimum number of service elevators NSE and passenger elevators NPE are found as, see Appendix 2.B (Allen and Iano 1995),

$$NSE = ((NRF \times ((D_a \times D_b) - (C_a \times C_b)) / 24600)_{\text{Rounded up}} \tag{2.6a}$$

$$NPE = ((NRF \times ((D_a \times D_b) - (C_a \times C_b)) / 3250)_{\text{Rounded up}} \tag{2.6b}$$

The total number of elevators NE for the building is,

$$NE = NSE + NPE \tag{2.6c}$$

The number of occupants per floor $NOPF$ and the total number of occupants TNO are found as, see Appendix 2.B (Allen and Iano 1995),

$$NOPF = ((D_a \times D_b - C_a \times C_b) / 9.3)_{Rounded\ up} \tag{2.7a}$$

$$TNO = NOPF \times NRF \tag{2.7b}$$

The number of stair cases NSC and their widths WSC are a function of the number of occupants per floor, and are found as (NBCC, 1990),

$$NSC = (NOPF / 500)_{Rounded\ up} + 1 \tag{2.8a}$$

$$WSC = (NOPF / NSC) \times 0.0092 \tag{2.8b}$$

Eq (2.8a) is accurate for buildings with footprints as large as 130m by 130m, which is in keeping with the upper bounds set on the primary variables NS_a, NS_b, S_a, and S_b in this study (see Table 2.2). The number of risers for each two-flight stair case $NRSC$ is a function of the height of floor HF and is found as (Figure 2.3),

$$NRSC = (5.42 \times HF + 0.25)_{Rounded\ up} \tag{2.8c}$$

Allowing for 0.15 m of space between ramps, and taking a landing area to be as wide as the stair itself, the overall inside length $OILSC$ and width $OIWSC$ of a stair case are found as (Allen and Iano 1995),

$$OILSC = (NRSC \times 0.280) / 2 + WSC \times 2 \tag{2.8d}$$

$$OIWSC = WSC \times 2 + 0.15 \tag{2.8e}$$

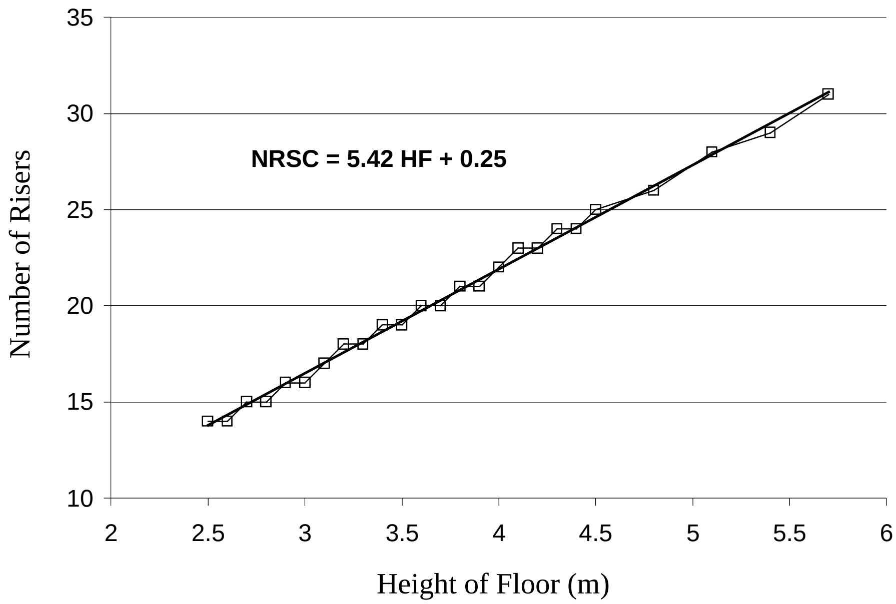


Figure 2.3: Relationship Between Height of Floor and Number of Stair Risers

To facilitate optimum usage of floor area, it is desirable to have columns spaced as far apart as possible. Specifically, longer floor spans are generally more beneficial than shorter spans since they provide greater flexibility for internal layout and unexpected future changes of floor use. In this study, a factor that corresponds to the amount of free-column area for the floor plan (Figure 2.4) is calculated to quantify the flexibility of floor space usage. To this end, the number of column lines between the building perimeter and the core in the a and b directions for the building, NCL_a and NCL_b, are first found as (Figure 2.4),

$$NCL_a = ((D_a - C_a)/(2 \times S_a) - 1)_{\text{Rounded up}} \tag{2.9a}$$

$$NCL_b = ((D_b - C_b)/(2 \times S_b) - 1)_{\text{Rounded up}} \tag{2.9b}$$

Then, the average distances between the column lines, $ADBL_a$ and $ADBL_b$, are found as,

$$ADBL_a = (D_a - C_a) / (2 \times (NCL_a + 1)) \tag{2.9c}$$

$$ADBL_b = (D_b - C_b) / (2 \times (NCL_b + 1)) \tag{2.9d}$$

Finally, the column-free area factor for the floor plan, *CFA*, is found as (Figure 2.4),

$$CFA = \frac{AreaA \times \sqrt{\dfrac{ADBL_a \times (D_b + C_b)}{2}} + AreaB \times \sqrt{\dfrac{ADBL_b \times (D_a + C_a)}{2}}}{AreaA + AreaB} \tag{2.9e}$$

For a fixed total floor area, Eq. (2.9e) yields larger values of the columns-free area factor *CFA* for buildings having larger footprints and widely spaced columns, and smaller values for buildings having smaller footprints and closely spaced columns. (As explained in Chapter 3, the *CFA* value is used to quantify the quality of space for a building).

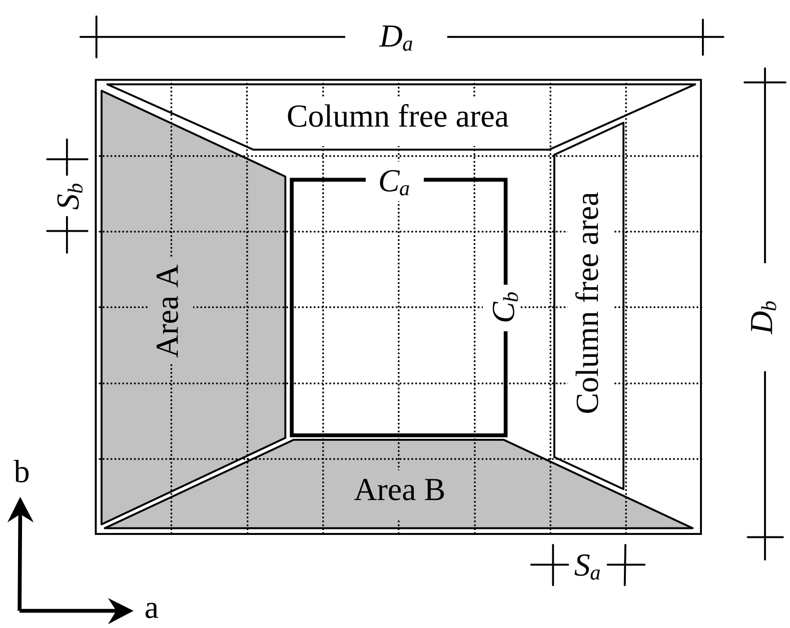


Figure 2.4: Schematic of a Typical Floor Plan

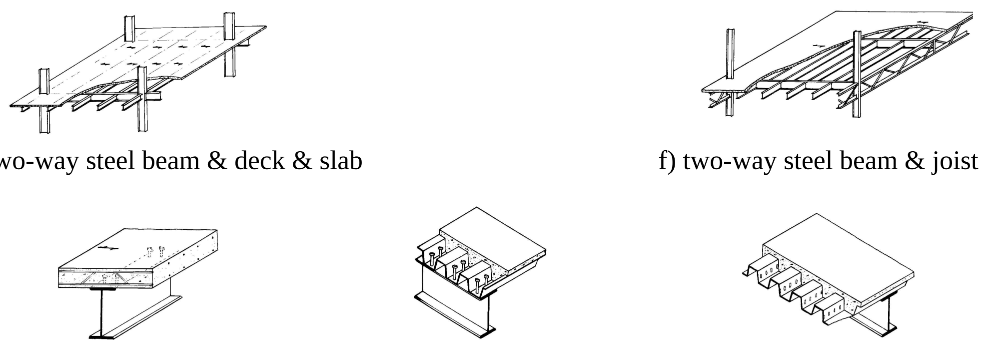
Appendix 2.A - Structural Systems

2.A.1 Vertical Load Resisting Systems

The vertical load resisting systems for high-rise buildings are essentially the same as those for low-rise structures, namely: 1) floors; 2) columns; and 3) load bearing walls. A suitable floor system is an important factor in the overall economy of the building. Some factors that effect choosing the floor system are architectural. For example, shorter floor spans are possible in residential buildings due to the permanent division of area into smaller spaces, while in modern office buildings longer span systems are preferred because their design philosophy leans toward more open and temporarily sub-divisible areas. Hence, in an office building, the structure's main vertical components are generally arranged as far apart as possible so as to leave large column-free areas available for office space planning. Other factors affecting the choice of a floor system are related to its intended structural performance, such as whether it is to participate in the lateral load resisting system. Floor systems can be categorized into three types (Cristiansen et al, 1980):

1. One-way systems: a) one-way slab, b) closely spaced joists
2. Two-way concrete systems: a) flat plate, b) flat slab with drop panel, c) slab and beam, and d) waffle slab
3. Two-way steel systems: a) beam and slab, and b) joists, girders and slab; in both of these systems the slab can be comprised of concrete with or with out steel deck and act as a non- or composite system.

Since one-way systems demand shorter spans and, as discussed earlier, it is desirable for office buildings to have large column-free spaces, this study only considers the two-way floor systems in concrete and steel shown in Figure 2.A.1.



a) concrete flat plate
b) concrete flat slab
c) concrete waffle slab
d) two-way concrete beam & slab
e) two-way steel beam & deck & slab
f) two-way steel beam & joist system
g) composite steel W-Section & concrete slab
h) composite steel W-Section & deck and concrete slab
i) steel W-Section & deck and concrete slab

Figure 2.A.1: Floor Systems

In taller buildings, columns and beams are the predominant load bearing systems due to their efficient use of space, versatility as structural systems, and ease of construction. Because of the large gravity loads associated with tall buildings, special care should be taken that major structural elements are not interrupted vertically. Whenever possible, the building’s cores, columns and loadbearing walls should not shift laterally from story to story but should be continuous from the roof down to the foundation of the building. Some structural configurations may occur, however, for which all loads do not have direct and continuous paths to the foundations.

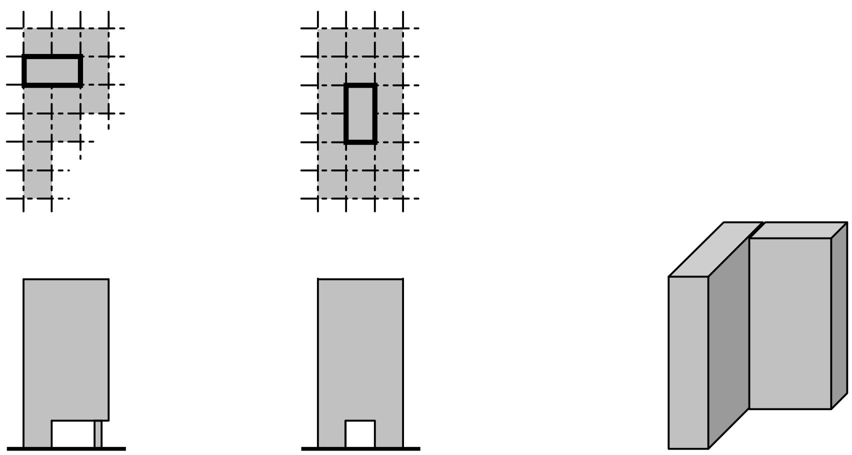
In some cases it is desirable to redistribute vertical loads out towards the perimeter of the building so to improve resistance to overturning. Special spaces in the lower levels of tall buildings, such as auditoriums, lobbies, atriums and mezzanines, often require longer span systems that must interrupt the paths of loadbearing elements from above. This sudden change in the arrangement or spacing of structural elements cause changes in the mass distribution along the height of a building. In extreme cases, a drastic change in the mass distribution requires reconsideration of the basic structural system for a building.

2.A.2 Horizontal Load Resisting Systems

Increasing the height of a building increases its sensitivity to both wind and earthquake forces. The taller the building, the more these forces will dominate the design of the entire structure, and the more attention should be given to the designing of them. Discussed in the following are guidelines important to the design of lateral load resisting systems for high-rise buildings.

Tall, narrow buildings are more difficult to stabilize against lateral forces than broader buildings. More effective bracing mechanisms may be required and bracing elements may assume more importance in the final design of such buildings. The most efficient structure is one in which the forces induced in the members due to lateral and gravity loadings do not greatly surpass those induced by gravity loading alone (Schueller 1977). In areas of great lateral loads (high seismic activity or hurricanes), tall buildings that are non-symmetrical or unbalanced in either weight distribution or the arrangement of bracing elements (Figure 2.A.2a) should be avoided in favour of symmetrical and balance buildings (Figure 2.A.2b).

Parts of a building that have independent mass can be expected to move differently under dynamic loads associated with earthquakes. The leg of an L-shaped building (Figure 2.A.2c), the stem of a T-shaped building, the wide base of a narrow tower, or any other form composed of discrete masses, may react in potentially destructive ways under such load conditions. All such masses should be designed as separate structures, with independent vertical and lateral load resisting systems, to minimize these effects.



a) unsymmetrical and unbalanced building

b) symmetrical and balanced building

c) unsymmetrical building designed as two structures

Figure 2.A.2: Symmetry in Buildings

Buildings with inherently unstable massing should be avoided. Discontinuities in the stiffness of a structure at different levels may lead to excessive deflections or other unfavourable responses to lateral loads. For instance, an open space in the long horizontal span direction at the base of a tall building may produce excessive flexibility at that level. If such a “soft story” cannot be avoided, the addition of special bracing elements at that level may be required.

Tall buildings may interact with winds in unpredictable ways. With buildings of irregular or unusual form, or building sites where adjacent structures or other features may produce unusual air movements, specialized studies of the building’s response to local wind pressures and fluctuations may be required.

The conventional arrangements of stabilizing elements used in low-rise buildings may be extended for use in buildings up to 20 to 25 stories in height (Allen and Iano 1995). The same considerations that apply to low-rise buildings apply to taller buildings as well. Stabilizing elements should be arranged so as to resist lateral forces along all major axes of the building. These elements should be arranged in a balanced manner either within the building or at the perimeter, and such elements must be integrated with the building plan of elevation.

Shear walls and braced frames are the stabilizing elements most commonly used in buildings of medium height, due to their structural efficiency. They may be used either separately or in combinations. The use of rigid frames as the sole means of stabilizing structures of medium height is possible, although this may be less than desirable because of the large size of the beams and columns that are generally required. For steel structures, the fabrication of welded joints required for rigid frame behaviour also becomes increasingly uneconomical as the number of connections increases. Rigid frames may also be used in combination with either shear walls or braced frames to enhance the total lateral resistance of a structure.

The proper arrangement of shear walls, diagonal braces, or rigid joints in a structure is crucial to their effectiveness in resisting lateral forces acting on the building. As illustrated in the schematic floor plans in Figure 2.A.3, these elements may be placed within the interior of the buildings or at the perimeter, and they may be combined in a variety of ways. However, they must be arranged so as to resist lateral forces acting from all directions. This is usually accomplished by aligning one set of stabilizing elements along each of the two perpendicular plan axes of a building. Stabilizing elements must also be arranged in as balanced a fashion as possible in relation to the mass of the building (Figures 2.A.3a, b, d and e). Unbalanced arrangements of these elements result in the displacement of the centre of stiffness of the building away from its centre of mass (Figure 2.A.3c and e). Such a condition causes torsional building movements under lateral loads that may be difficult or impossible to control.

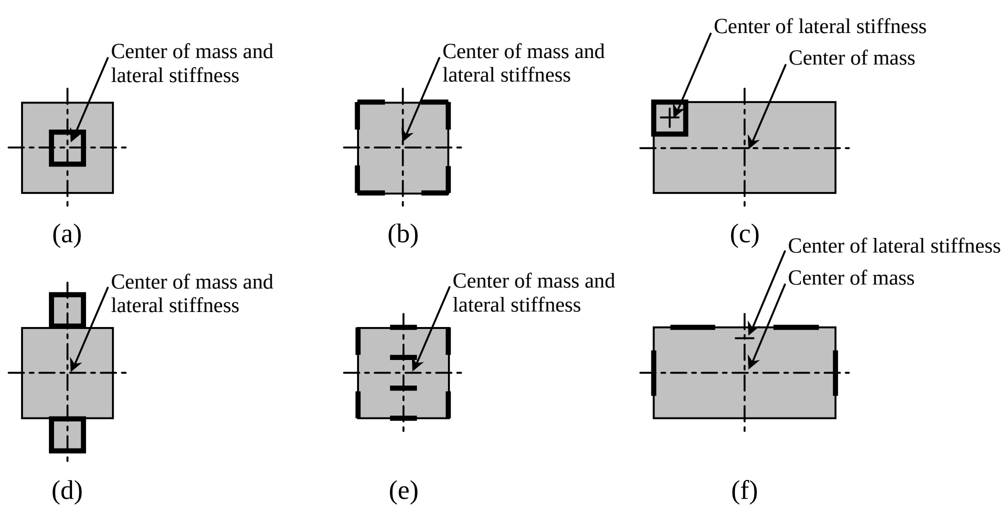


Figure 2.A.3: Arrangement of Stabilizing Elements in Buildings

All buildings must include structural elements designed specifically to resist lateral forces, such as those due to wind and earthquakes. The choice and location of these elements can influence building design in important ways even at the preliminary stage. The three stabilizing mechanisms used in buildings are the rigid frame, the braced frame, and the shear wall. Any one of these can be used to stabilize a building, or they may be used together in a variety of combinations.

The systems shown in elevation and plan view in Figure 2.A.4 are presented in left-to-right order of increasing resistance to lateral forces.

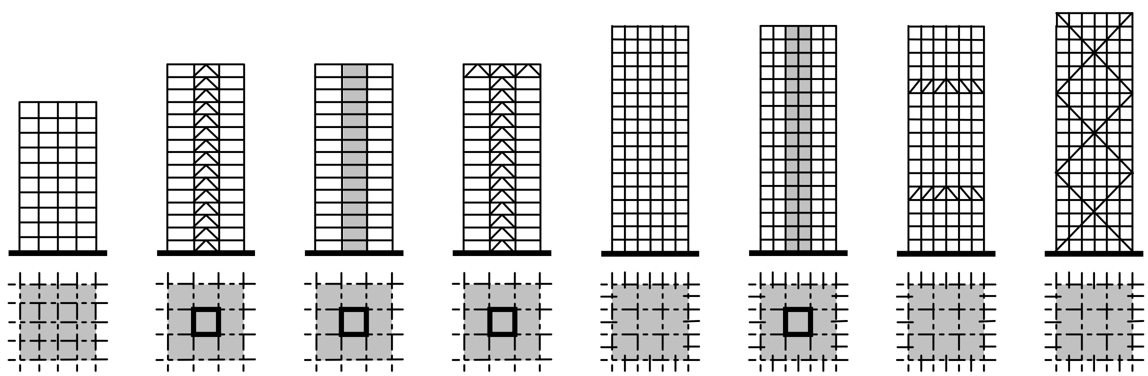


Figure 2.A.4: Schematic Representation of Different Structural Systems

The horizontal load resisting systems in Figure 2.A.4 can be categorized into the following groups (Cristiansen et al, 1980):

1. Moment resistant (rigid) frames (Figure 2.A.4a)
2. Braced frames (Figure 2.A.4b)
3. Shear wall systems (Figure 2.A.4c)
4. Combination systems: braced rigid frame (Figure 2.A.4b);
   rigid frame and shear wall (Figure 2.A.4c);
   braced frame and outrigger trusses (Figure 2.A.4d);
   tube (Figure 2.A.4e);
   tube-in-tube (Figure 2.A.4f);
   tube and belt trusses (Figure 2.A.4g);
   tube and external bracing (Figure 2.A.4h);
   bundled tube (not shown)

Moment Resistant (Rigid) Frames. Rigid frames depend on rigid connections between columns and beams (or slabs) to develop resistance to lateral forces. Rigid frame skeletons generally consist of a rectangular grid of horizontal beams and vertical columns connected together in the same plane by means of rigid joints. Though the least efficient of the three basic stabilizing mechanisms, rigid frames find use in buildings that require relatively modest lateral resistance (e.g., low, broad buildings), or in buildings where the presence of stabilizing walls or braces is undesirable. The frame may be in-plane with an interior wall of the building, or in-plane with the façade. The rigid frame is economical up to approximately 30 stories for steel buildings and up to 20 stories for concrete buildings (Schueller 1977).

Compared to shear wall or braced frame systems, the use of rigid frames may set greater restrictions on the arrangement and sizing of the structural frame. Column spacing often must be reduced, variations or irregularities in column placement may be limited, and the size of columns and depths of beams may need to be increased. The size of the columns and girders at any level of a rigid frame are directly influenced by the magnitude of the external shear at that level (Smith and Coull 1991) and, therefore, they increase in size toward the base of the structure. Consequently, the design of the floor framing system cannot be repetitive as it is in some braced frames. Also, in the lowest stories it is sometimes not even possible to accommodate the required dept of girder within the normal ceiling space. The rigid joints necessary in this system can be easily constructed in steel (at added cost compared to hinge-connections), or in sitecast concrete, where they are formed as a normal part of the construction process. Though possible, rigid joints are difficult to construct in precast concrete and are rarely used.

Rigid frames are often combined with either shear walls or bracing for improved results compared to either system acting alone.

Because of the type of connections between the structural elements, a rigid frame responds to lateral loads primarily through flexure of the beams and columns. This continuous character of the rigid frame is dependent on the resistance of the member connections against any rotational slippage. The load capacity of the frame relies very much on the strength of the individual beams and columns, and its capacity decreases as story height and columns spacing become larger. The lateral deflection of rigid frames is caused generally by two factors:

1. Deflection due to cantilever bending: This phenomenon is known as chord drift, where, in resisting the over-turning moment, the frame acts as a vertical cantilever beam that bends through axial deformation of its fibres. In this case, lengthening and shortening of the columns produce the lateral sway of the frame. This mode of lateral deflection accounts for about 20% of the total drift of structures (Schueller 1977).
2. Deflection due to bending of beams and columns: This phenomenon is known as frame racking, where shear forces cause bending moments to be introduced into columns and beams such that as they bend, the entire frame distorts. This mode of deformation accounts for about 80% of the total sway of the structure; 65% is due to beam flexure and 15% is due to column flexure (Schueller 1997). The curvature of the deflection corresponds to the external shear diagram; the slope of the deflection curve is maximum at the base of the structure, where the largest shear occurs.

Braced Frames. Braced frames are quite effective in resisting lateral forces. They may be constructed from steel or, occasionally, from concrete. The diagonal bracing elements that comprise these systems act similar to shear walls in transferring lateral forces between floors of a building. Diagonal bracing is inherently obstructive to the architectural plan and can pose problems in the organization of internal spaces and access as well as in locating window and door openings. For this reason, bracing is usually concentrated in vertical panels or bents that are located near the centre of the building to cause minimum obstruction while satisfying the structural requirements to resist shear and torque forces on the building. The most efficient, but also the most obstructive, types of bracing are those that form a fully triangulated vertical truss. These include single-diagonal, double-diagonal and K-braced types (Figures 2.A.5a, b, c, and d).

The full diagonal types of braced bent are usually located where passage is not required, such as between elevator, service and stair shafts, which entities are unlikely to be relocated in the lifetime of the building.

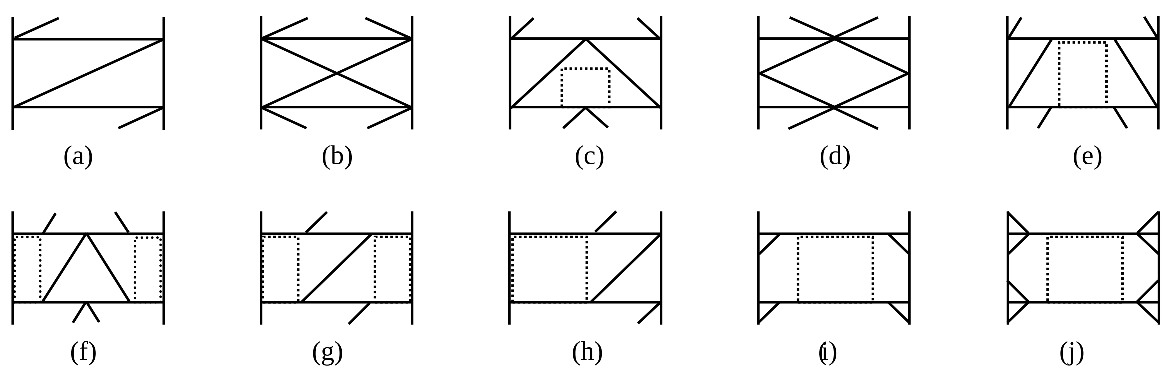


Figure 2.A.5: Different Bracing Types

Other types of braced bents that allow for window and door openings, but whose arrangement cause bending in the girders, are shown in Figures 2.A.5e, f, g, h, i, and j. Because lateral (wind, seismic) loading on a building is reversible, braces can be subjected to both tension and compression forces, but they are generally designed for the more severe case of compression loading. For this reason, bracing systems with shorter braces, e.g., the K-type, may be preferred to full-diagonal types. As an exception to designing braces for compression, the braces in the double-diagonal system are sometimes assumed to buckle in compression and each diagonal is designed to carry in tension the full shear in the panel.

A significant advantage of the fully triangulated bracing types, Figures 2.A.a, b, c, and d, is that the girder moments and shears are independent of the lateral loading on the structure. Consequently, the floor system can be designed for gravity loading alone and, as such, can be repetitive throughout the height of the structure with obvious economic benefit. Generally, the types of braced bent that respond to lateral loading by bending of the girders, or the girders and columns, are laterally less stiff and therefore less efficient, than the fully triangulated braced bent that develops axial forces alone in the members (Smith and Coull 1991).

Shear Walls. Shear walls are extremely effective in resisting lateral forces. They are easily constructed from concrete, masonry or wood and, sometimes in tall buildings, from steel. The superior resistance of shear walls to lateral forces often makes them a good choice in situations where the maximum resistance to lateral forces is required, such as across the narrow dimension of a tall, slender building. Shear walls are commonly integrated into the enclosure of vertical building cores or stair cases. They may or may not carry gravity loads. When shear walls are incorporated into the interior of a building their locations must be coordinated with the building's plan. Shear walls placed at the perimeter of a building can restrict the size, number or arrangement of openings, and this is generally not desirable for proper access and natural lighting for the building.

Shear wall systems can assume a number of geometrical configurations, which may be subdivided into open and closed systems. Open systems are made up of single linear shear wall elements, or a combination of such elements, that do not completely enclose a geometric space. Such shapes are L, X, V, Y, T and H (Schueller 1977). Conversely, closed systems enclose a geometrical space, common forms of which are square, triangular, rectangular and circular cores of buildings. Shear wall systems may be arranged symmetrically or asymmetrically so as to minimize the effect of eccentricity of lateral loads.

The shape and location of shear walls have significant effects on their structural behaviour under lateral loads. A core that is eccentrically located with respect to the building shape has to carry torsion as well as bending and direct shear. Moreover, torsion may even develop in buildings featuring symmetrical shear wall arrangements when the wind loads act on facades of different surfaces texture and roughness (Schueller 1977), or when the building's centre of mass and stiffness do not coincide.

Optimal torsional resistance is obtained with closed core sections. When evaluating core section resistance, however, the tosional rigidity must be reduced to account for door, window and other openings. For maximum performance, shear walls should have a minimum of perforations or openings. In fact, walls having large openings to accommodate mechanical and electrical systems might not be able to carry lateral loads.

Floors acting as horizontal diaphragms transmit lateral loads to the shear walls. If the floors have no major openings, they are generally assumed to be infinitely stiff and the distribution of lateral forces to the shear walls is strictly a function of the geometrical arrangement of the resisting wall systems.

If the resultant of the lateral forces acts through the centre of stiffness for a building, only translation reaction will be generated. The most obvious case in this regard is the symmetrical pure shear wall building (Figures 2.A.3a, b, d, and e). In a rigid frame shear wall building, the shear may be assumed to be resisted completely by the core as a first approximation (Schueller 1977). This is because core lateral stiffness is generally much greater than the lateral stiffness of the frame. If the shear wall arrangement is asymmetrical, the resultant of the lateral forces does not act through the stiffness centeriod of the building, and rotation of the shear walls will occur in addition to translation.

When the loads acting on an individual shear wall have been determined, the next stage of the design process is to determine the corresponding wall stresses. The distribution of stresses in a shear wall is dependent on the shape of the system. If the wall is rectangular in elevation and has a height-to-width ratio greater than five, a close estimate of the axial stresses is given by simple bending theory (Smith and Coull 1991). The same methodology can be extended to coupled shear walls, where the forces induced in the connecting beams can be approximated from the sum of shear flows for the coupled walls.

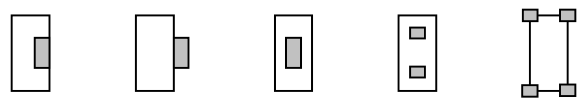
Core Structures. Cores typically take up approximately 20% to 25% of the total floor area of a high-rise building (Allen and Iano 1995). They should be formed as closed elements, approximately square or cylindrical, with openings in the core kept to a minimum.

Core structures are perhaps the systems that are most commonly used to laterally stabilize all but the tallest buildings (Schueller 1977). These structures integrate stabilizing elements into the vertical shafts that house the circulation and mechanical service systems for a tall building. One of the principal advantages of these structures is that interference with the surrounding usable space in the building is minimized. In concrete construction, core walls intended to enclose building service systems can be readily designed to also act as shear walls, in many cases with no increase in size. In steel construction, core structures are usually designed as braced frames.

In buildings with more than one core, the cores should be located symmetrically in the building plan so as to provide balanced resistance under lateral loads from any direction. A single core servicing an entire building should be located at the centre of the building, which typically provides the overall best solution to meet various architectural and structural criteria for office buildings, as indicated in Table 2.A.1 (Allen and Iano 1995).

Simple core structures can be used in buildings as high as 35 to 40 stories (Allen and Iano 1995). The lateral stability of simple core structures can be enhanced with the addition of bracing in the form of “hat” trusses which serve to also engage the perimeter columns of the building in the task of resisting lateral loads, thus significantly improving the overall performance of the building. Albeit, such trusses may influence the design of the building façade or the location of mechanical floors. Columns at the perimeter of the building may also increase in size with this system. These core-interactive structures are suitable for buildings up to approximately 55 stories in height (Allen and Iano 1995).

Table 2.A.1: Characteristics of Core Placements



1 = Best, 5 = Worst	Edge	Detached	Central	Two	Corners
Flexibility of typical rental area	2	1	3	4	2
Perimeter for rental area	4	3	1	1	5
Ground floor high-rent area	3	1	3	4	2
Typical distance of travel from core	4	5	2	1	3
Clarity of circulation	3	4	2	1	3
Daylight and view for core spaces	2	1	5	5	1
Service connection at roof	3	5	1	2	4
Service connection at ground	3	4	2	1	5
Suitability for lateral bracing	4	5	1	1	2
Total	28	29	20	20	27
Overall ranking	3rd	4th	1st	1st	2nd

Shear Core Structures. The linear shear wall system works quite well for apartment buildings in which functional and utilitarian needs are fixed. Commercial buildings, however, require maximum flexibility in layout, calling for large open spaces that can be subdivided by movable partitions. A common solution is to gather together vertical transportation and energy distribution systems, such as elevators, stairs, toilets and mechanical shafts, to form a core or cores depending on the size and function of the building. These cores are then also utilized as shear wall systems to provide the necessary lateral stability for the building.

Cores can be made of steel, concrete, or a combination of both. In a steel framed core, diagonal bracing is used to achieve the necessary lateral stiffness for taller buildings. The advantage of steel framed cores lies in the relatively rapid assemblage of the core using prefabricated members. The concrete core, in addition to carrying loads, completely encloses the space such that no further considerations need to be given to fireproofing. At the same time, the lake of ductility inherent in concrete as a material is a disadvantage when responding to earthquake loading.

Lateral-loads resisting shear core structures may be visualized somewhat as huge beams cantilevering out of the ground, for which bending and shear stresses are similar to those of a box Section beam. Since the core also carries gravity loads it has the advantage of being prestressed by the induced compressive stresses, and thus may not need to be designed for tensile stresses due to bending caused by lateral loads (this is especially true for heavy concrete cores). In addition, the capacity of the core material to resist shear stress is increased in the presence of compressive stresses.

The response of a core structure to lateral loading is dependent on its shape, degree of homogeneity and rigidity, and the direction of the load. At every floor level there are openings in the core, and the amount of continuity provided by the coupling beams determines the behavior of the core. The design must avoid having the core act like an open section that distorts (warps) in its upper portion with no restraint, especially under asymmetrical loading causing twisting.

Frame-Shear Wall Building Systems. Pure rigid frame systems are not practical in buildings higher than 30 stories (Schueller 1977). Thereafter, such systems generally also employ a shear wall of some type within the frame to resist lateral loads. The shear walls are either concrete or trussed-steel bracing . They may be closed interior cores, as around elevator shafts or stair wells, or parallel walls within the building, or they may be vertical façade trusses.

Frame-shear wall systems are classified with respect to their response to lateral loading, which may be one of the following two types: 1) Hinged frame-shear wall systems; 2) Fixed frame-shear wall systems. In the hinged frame-shear wall system the column-girder connections do not take any bending moment, such that the frame only carries gravity loads while the shear walls resist all the lateral loads. For such systems, however, it may not be possible at times to make the shear walls sufficiently strong to resist the lateral forces by themselves alone. In such cases, the fixed frame-shear wall system is used where both shear walls and the rigid frame act together to resist the lateral forces. Here, the lateral deflection of the combined shear wall and rigid frame is obtained by superimposing their individual modes of deformation, as shown in Figure 2.A.6.

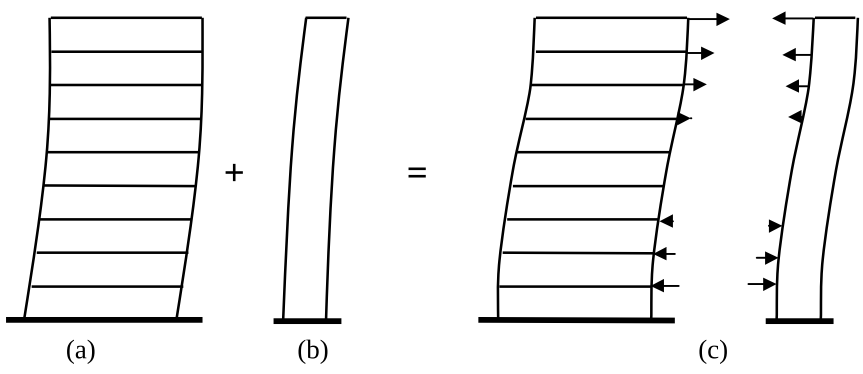


Figure 2.A.6: Frame-Shear Wall Interaction

The rigid frame shear mode deformation is indicated schematically in Figure 2.A.6a. Note that the slope of the deflection curve is greatest at the base of the structure where the maximum shear is acting. The shear wall system is assumed to act as a vertical cantilever beam in bending. The shear wall bending deformation mode is indicated in Figure 2.A.6b. Note that the slope of the deflection curve is greatest at the top of the building, indicating that the shear wall system contributes the least stiffness in this region. The combined frame and shear wall deformation is obtained by superimposing the two separate deflection modes, resulting in the flat S-curve shown in Figure 2.A.6c (Schueller 1977). Because of the different deflection characteristics of the shear wall and frame, the shear wall is pulled back by the frame in the upper portion of the building, and pushed forward near its base. As a consequence, the lateral shear force is carried mostly by the frame in the upper portion of the building and by the shear wall in the lower portion.

It is desirable in tall buildings to proportion the wall and frame components so as to optimize the overall desirable effect of wall-frame interaction. Such an optimization aims to not only achieve significant reductions in lateral deflections and wall moments, but also to cause an approximately uniform distribution of shear over the height of the frame. This then permits the repetitive design and construction of the floor system. To achieve such a well proportioned shear wall-frame structure, a common rule is to size the shear walls in the preliminary stage of design to carry their gravity loading together with two-thirds of the total horizontal loading, leaving the frame to carry one-third of the total lateral load on the building (Smith and Coull 1991).

Flat Slab Building Structures. Flat slab systems consist of solid or waffle-type concrete slabs supported directly on columns, thus eliminating the need for floor framing. This results in minimum story height, an obvious economic advantage. The systems are adaptable to an irregular support layout. Drop panels and/or column capitals are frequently used because of high shear concentrations around the columns. Slabs without drop panels are commonly called flat plates. Some disadvantages of flat slab systems are: a) undesirable large dead load; b) small depth-to-span ratios can cause the appearance of excessive deflection; and c) their relatively short span capability.

Usually for multi-story buildings, flat slab structures rely on the shear walls to provide the necessary lateral stiffness. Albeit, the monolithic character of such concrete structures requires the entire building to react to lateral loads as a unit, and it is not realistic to assume that lateral loads are resisted entirely by the more rigid core or shear wall and that the slabs and columns contribute no resistance at all. In fact, the flat slab itself, though relatively flexible, provides lateral stiffness to the structure because of its continuity with the shear walls. As well, a portion of the slab will act as a shallow beam continuous with the columns such that the behavior of the total structure is similar to that of a core-frame system (e.g., see Figure2.A.6)

Frame-Shear Wall Systems with Belt Trusses. The braced frame becomes inefficient above about 40 stories because excessive bracing is required beyond that point to provide adequate lateral stiffness to the structure. The efficiency of the building structure may be improved by about 30% through the use of horizontal belt trusses that tie the frame to the core (Schueller 1977). The trusses are fixed rigidly to the core and simply connected to the exterior columns. When the shear core tries to bend, the belt trusses act as lever arms that directly transfer axial stresses into the perimeter columns. The columns, in turn, act as struts to resist the lateral deflection of the core. That is, the core fully develops the horizontal shear and the belt trusses transfer the vertical shear from the core to the façade frame. Thus, the building is made to act as a unit that is very similar to a cantilever tube.

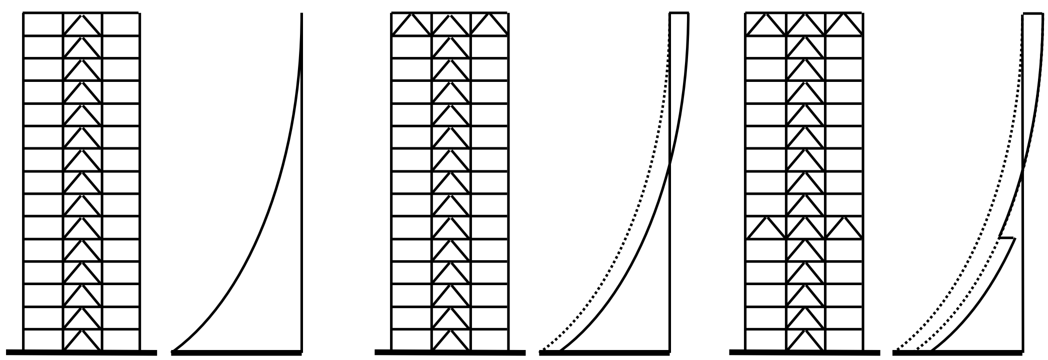
The building can have one or several belt truss; the more trusses used, the better the integration of core and façade columns. They should be placed at locations within the building where the diagonal bracing will not interfere with the building's function. The structural principle of employing belt trusses at the top and mid-height of a building seems to be economical in applications up to approximately 60 stories (Schueller 1977).

The stress diagram in Figure 2.A.7 illustrates the relative efficiency of hinging the belt trusses to the perimeter columns rather than fixing them rigidly. If the trusses were to be continuously connected to the columns, the entire system would act as a unit, thus utilizing only a small percentage of the moment-resisting capacity of the core, whose walls are relatively close to the neutral axis of the building. This is indicated by the continuous distribution of stresses shown for the rigid frame in Figure 2.A.7a. On the other hand, belted trusses that are cantilevered from the core and hinged to the perimeter columns better develop the moment resisting capacity of the core while still engaging the exterior columns as in the rigid system (Figure 2.A.7b). In fact, since the hinged shear connections induce no bending moments into the columns, the axial capacity of the columns is increased relative to that for the case of fixed shear connections.

(a) (b)

Figure 2.A.7: Stress Distribution in Frame-Shear Wall Systems with Belt Trusses

The response of a core frame building with belt trusses to lateral loading is shown in Figure 2.A.8. This Figure schematically shows the reduction of moment in the shear-core for a one-outrigger system (Figure 2.A.8b) and a two-outrigger system (Figure 2.A.8c) compared to that for a no-outrigger system (Figure 2.A.8a).



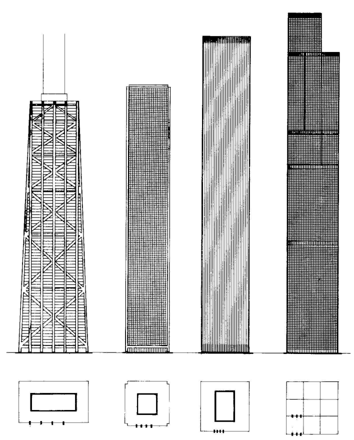
(a) (b) (c)

Figure 2.A.8: The Effect of Outriggers on Core Moment

When the frame is hinged to the core of the structure, the core behaves like a cantilever and its top is free to rotate. The frame itself hardly resists any rotation. If the frame is tied to the core by a belt truss, however, any rotation at the top of the system is restricted, since the perimeter columns tie the belt truss down. There is then no bending moment in the columns. The partial fixity provided at the top of the system by the belt truss is reflected in the moment diagram in Figure 2.A.8b. The system no longer acts as a pure cantilever because it is restrained at the top as well as at the bottom. The resulting deflection is a flat S-curve, with a zero moment at a point of inflection above the mid point of the building. The bending moment in the shear wall at the base of the building is less than that for the no-outrigger case in Figure 2.A.8 a. The strength and stiffness of the system is further increased by adding additional belt trusses at intermediate levels within the building. At each truss level the system is restrained from rotating. The fixity provided at these levels pulls the moment diagram back, as shown in Figures 2.A.8c, such that the bending moment at the base of the building is further reduced (along with building sway).

Smith and Coull (1991) studied the optimum location of outriggers by considering hypothetical structures whose outriggers were flexurally rigid. They found that a single outrigger in a one-outrigger system should be located at approximately half height of the building, that the outriggers in a two-outrigger system should be located roughly at one-third and two-thirds height, and that in a three-outrigger system they should be at approximately one-quarter, one-half, and three-quarters height, and so on. Generally for the optimum performance of an n-outrigger structure, the outriggers should be placed at the 1/(n+1), 2/(n+1), up to the n/(n+1) height locations. The Smith and Coull study found that the reduction in core base bending moment is approximately 58%, 70%, 77% and 81% for one-outrigger, two-outrigger, three-outrigger and four-outrigger structures, respectively. Unexpectedly, contrary to a traditional location for outriggers (Shueller 1977), they found that it is structurally inefficient to locate an outrigger at the top of a building. In an optimally arranged outrigger system, the moment carried by any one outrigger is approximately 58% of that carried by the outrigger below. However, if an additional outrigger is placed at the top of the building, it carries a moment that is roughly only 13% of that carried by the outrigger below, which clearly shows the inefficiency of this outrigger location.

Tubular Systems. A relatively recent development in tall building design is the concept of tubular behaviour introduced by Fazlur Khan (Schueller 1977). The tallest buildings currently being constructed are designed as tube structures. In fact, four of the world's tallest buildings are tubular systems: the Hancock Building, Sears Building and Standard Oil Building in Chicago, and the World Trade Center in New York, Figures 2.A.9 a, b, c and d, respectively.



(a) (b) (c) (d)

Figure 2.A.9: Four As-Built Tube Structures

In tubular systems, stabilizing elements are located at the perimeter of the structure, leaving the layout of the interior of the building virtually unrestricted by concerns for lateral stability. Tubular systems are so efficient that in most cases the amount of structural material used per square meter of floor space is comparable to that used in conventionally framed buildings of half the size (Schueller 1977)

Tubular design assumes that the façade structure responds to lateral loads as a closed hollow box beam cantilevering out of the ground. Since the exterior walls resist all or most of the wind load, costly interior diagonal bracing or shear walls are eliminated.

The use of rigid frame tubes may effect the size and spacing of framing elements at the perimeter of the building. Beams may need to be deeper and columns may need to be larger and more closely spaced than would otherwise be required. When constructed of steel, the welded joints required in tube systems may be more costly to construct, although construction techniques have been developed that allow for the off-site fabrication of these joints, thus minimizing this disadvantage. The walls of a tube system consist of closely spaced columns around the perimeter of the building that are tied together by deep spandrel beams. This façade structure looks like a perforated wall. The stiffness of the façade wall may be further increased by adding diagonal braces to cause truss-like action (Figure 2.A.9a). The rigidity of a tube is so high that it responds to lateral loading in a way similar to a cantilever beam. As we will see in the following, an exterior tube can resist all of the lateral loads on its own, or it can be further stiffened by adding interior bracing of some kind.

<u>Framed Tube.</u> The framed tube, the earliest application of the tubular concept, was first used in a 43-story apartment building in Chicago in 1961 (Schueller 1977). In this particular tube system, the exterior walls of the building consist of a closely spaced

rectangular grid of beams and columns rigidly connected together, which resist lateral loads through cantilever tube action without using interior bracing. Interior columns are assumed to carry gravity loads alone and do not contribute to the lateral stiffness of the building. Stiff floor systems act as rigid diaphragms that distribute lateral forces to the perimeter walls.

Other examples of hollow framed tube buildings are the 83-story Standard Oil Building in Chicago and the 110 story World Trade Center in New York (Figures 2.A.9c and d). Although these buildings have interior cores, they act as hollow tubes because the cores are not designed to resist lateral loads. Such a system possesses excellent lateral stiffness and torsional qualities while retaining flexible interior space layout possibilities. In some framed tube buildings, the façade grid is so closely spaced that it can serve as mullions for the glazing.

It would be ideal in the design of framed tube systems if the exterior walls were to act as a unit, responding to lateral loads in pure cantilever bending. If this were the case, all columns that make up the tube would be either in direct axial tension or compression. The linear stress distribution that would result is indicated by the broken lines in Figure 2.A.10.

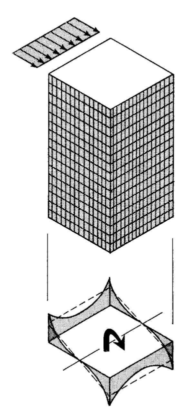


Figure 2.A.10: Stress Distribution for Façade Columns

However, the true behavior of the tube lies somewhere between that of a pure cantilever and a pure frame. Due to the flexibility of the spandrel beams, the sides of the tube parallel to the lateral force tend to act as independent multi-bay rigid frames. This flexibility results in racking of the frame due to shear (shear lag). Hence, bending takes place in the columns and beams. The effect of shear lag on the tube action results in a nonlinear pressure distribution over the column envelope, where the columns at the corners of the building are forced to take a higher share of the load than the columns in between; see solid-line stress distribution in Figure 2.A.10. Furthermore, the total deflection of the building no longer resembles a cantilever beam, as shear mode deformation becomes more significant. However, it has been suggested (Smith and Coull 1991) that for approximate analysis it is reasonable to assume that lateral forces cause shear in web panels parallel to the direction of the lateral load, and axial forces alone in flange columns perpendicular to the lateral load, Figure 2.A.11.

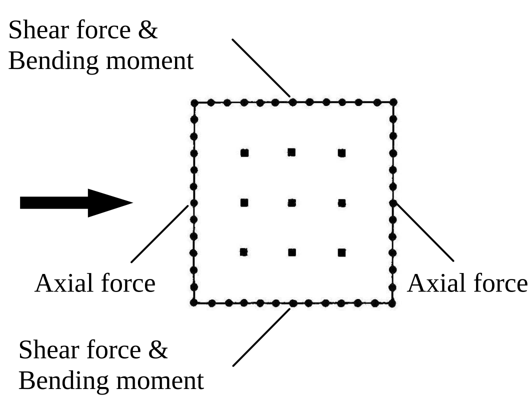


Figure 2.A.11: Forces Induced in the Columns and Spandrel Beams of a Tubular Structure

The shear problem severely affects the efficiency of tubular systems, and many developments in tubular design have attempted to overcome it. The framed tube principle seems to be economical for steel buildings up to 80 stories and for concrete buildings up to 60 stories (Schueller 1977). However, there is no obvious optimum height for this structural system as other sources report that simple tube structures perform very well up to 50 to 55 stories (Allen and Iano 1995).

<u>Braced Tube.</u> The performance of rigid frame tube structures may be enhanced with the addition of belt trusses located on the perimeter of the structure, Figure 2.A.4g. These trusses may be located at various levels on the structure, and they may influence the location of mechanical floors and overall façade design. The framed exterior tube may be stiffened in plane by adding diagonals, Figure 2.A.4h, or it may be stiffened from within the building by adding shear walls or interior cores, Figure 2.A.4f.

Braced frame tubes are very efficient lateral load resisting systems. When built in steel, these structures usually rely on easily constructed bolted connections. The diagonal braces that are an integral part of this system can have a significant impact on the appearance of the building façade, (e.g., see the Hancock Building shown if Figure 2.A.9a).

<u>Tube-in-Tube.</u> Variations on the tube structure are also possible. Tube-in-tube structures, in which perimeter tubes interact with interior rigid cores, may be designed for enhanced structural performance. In fact, the stiffness of a hollow tube system is very much improved by using the core not only for gravity loads but also to resist lateral loads as well. The floor systems tie together the exterior and interior tubes such that they respond to lateral forces as a unit. The response of a tube-in-tube system to wind is similar to that

of a frame and shear wall structure. However, the framed exterior tube is much stiffer than a simple rigid frame.

Figure 2.A.6, which was previously used to explain frame and shear wall structures can be viewed to clarify the interaction between the core and tube for tube-in-tube systems. The approach has been used in the 38-story Brunswick building in Chicago, and the 52-story One Shell Plaza building in Houston (Schueller 1977). Moreover, taking the tube-in-tube concept one step further, the designer of a 60-story office building in Tokyo used a triple tube. In this system, the exterior tube alone resists the wind loads, but all three tubes are connected by the floor systems and act as a unit in resisting earthquake loads, a significant design factor in Japan. Finally, bundled-tube structures have been developed that permit great variation in the massing of a structure so as to enhance the overall performance of the structure, (e.g., see the World Trade Center building shown in Figure 2.A.9d).

Except for the braced tube and tube-in-tube systems presented in the immediate forgoing, for which appropriate methods of approximate analysis are not readily available, it is noted all of the structural systems that have been discussed in this Section are accounted for in Chapters 3 and 4 concerning the implementation and application of the proposed computer based method for conceptual design of high-rise office buildings.

Appendix 2.B - Mechanical Systems

2.B.1 HVAC Systems

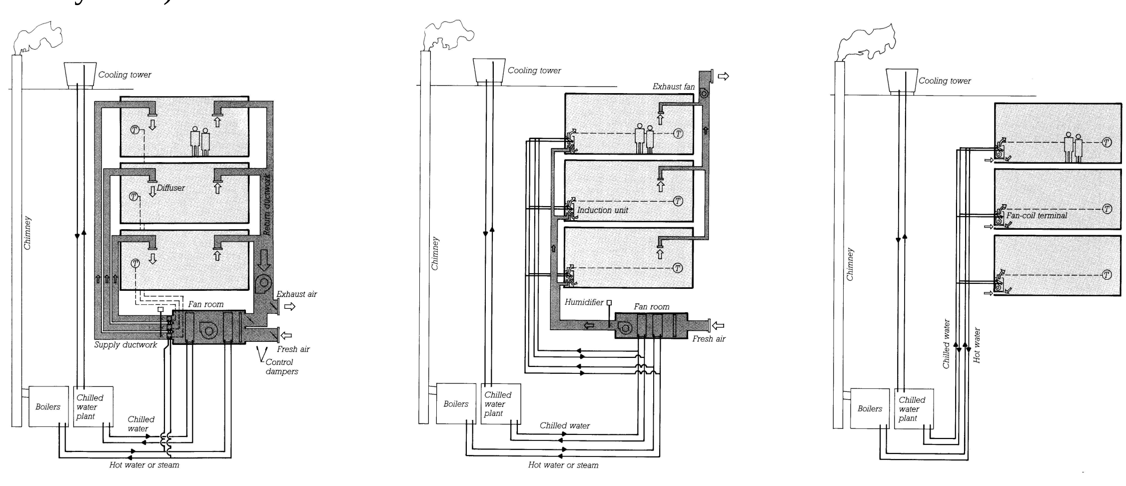
The HVAC system must fit the overall objectives of the building and, in this sense, must be thought of as an integral part of the building rather than as an appendage to be placed after the architectural design has been fixed. In most cases for tall buildings, the mechanical floors are strategically located over the height of the building so as to reduce the distance between the fan rooms and the boiler and chillers rooms. Generally, the designer must consider a variety of architectural, structural, occupancy, environmental, energy and cost issues for HVAC systems (Baum et at 1980):

In this study, while taking into account occupancy requirements, architectural and structural constraints, and the internal and external environment, the initial cost, annual operating cost and annual maintenance cost of HVAC systems are used to evaluate the overall optimality of a building. These three costs are functions of the loads applied on the HVAC system, of which there are two types:

1. Heating loads: the amount of energy to be provided to the building by boilers to arrive at a suitable temperature for the occupants during the cold season.
2. Cooling loads: the amount of energy to be taken from the building by chillers to arrive at a suitable temperature for the occupants during the hot season.

There are three different types of HVAC systems for large buildings(Allen and Iano, 1995), as described in the following.

All-Air Systems. In this system, air is conditioned (mixed with a percentage of outdoor air, filtered, heated or cooled, and humidified or dehumidified) at a central source. Supply and return fans circulate the conditioned air through ducts to the occupied spaces of the building. In each individual zone of the building a thermostat regulates the temperature by controlling the heating and cooling coils. In one type of multi-zone system, dampers blend hot and cold air in the fan to send air into the ducts at the temperature requested by the thermostat in each zone. In another type of system, shown in Figure 2.B.1a, reheat coils in the fan room regulate the temperature of the air supplied to each zone. This system offers a high degree of control of air quality and is comparatively simple and easy to maintain, its only drawback being that it requires a large amount of space for ductwork in the vicinity of the fan (however, this problem is not critical in tall building design since there is generally a core area existing to contain such systems).



(a) (b) (c)

Figure 2.B.1: Schematics of Different HVAC Systems

Air and Water Systems. In this system, fresh air is conditioned (heated or cooled, filtered, and humidified or dehumidified) at a central source and circulated in small high-velocity ducts to the occupied spaces of the building, Figure 2.B.1b. Each outlet is designed so that the air discharged from the duct (primary air) draws a much larger volume of room air through a filter. The mixture of primary air and room air passes over a coil that is either heated or cooled by secondary water pipes from the boiler room or the chilled water plant. The primary air (about 15% to 25% of the total airflow through the outlet) and the heated or cooled room air that has been induced into the outlet (75% to 85% of the total airflow) are mixed and discharged into the room. A local thermostat controls the water flow through the coil to regulate the temperature of the air in the space. Condensate that drips from the chilled water coil is caught in a pan and removed through a system of drainage piping. This system is very suitable for exterior spaces of buildings having a wide range of heating and cooling loads where close control of humidity is not required. As well, this system offers good local temperature control and the space required for its ductwork and fans are less than that for all-air systems. However, such systems are relatively complicated to design, install, maintain and manage. They tend to be noisy, inefficient in their use of energy and unable to closely control humidity. In fact, due to these disadvantages, this type of HVAC system is rarely designed or specified at the present time (Allen and Iano 1995).

All-Water Systems. In this system, hot and/or chilled water is pumped through pipes to fan-coil terminals, Figure 2.B.1c. At each terminal, a fan draws a mixture of room air and outdoor air through a filter and blows it across a coil of heated or chilled water and then back into the room. A thermostat controls the flow of hot and chilled water to the coils so as to control the room temperature. The same technique as is used in air-water systems conveys the condensate away from the occupied space. In most installations, the additional volume of air brought from the outdoors is used to pressurize the building to prevent infiltration of outside unconditioned air. The system can be used in buildings having many zones located on exterior walls, such as schools. It does not need a fan room or ductwork and allows control of temperature in different spaces individually. However, as for the air-water system, there is no control over the degree of humidity. As well, the system requires considerable maintenance, most of which must take place in the occupied spaces of the building.

Choosing an HVAC System. Each of the three HVAC systems described in the foregoing has its pro's and con's in terms of needed space and control over temperature and/or humidity in the various zones of a building. As mentioned, an all-air system needs much more space compared to the other two systems but it offers excellent control of interior air quality. Its central air-handling equipment can be designed for precise control of fresh air, filtration, humidification, dehumidification, heating, and cooling. When the outdoor air is cool, an all-air system can switch to an economizer cycle, in which it cools the building by circulating a maximum amount of outdoor air. Unlike the other systems, all-air systems concentrate maintenance activities in unoccupied areas of the building because there are no water pipes, condensate drains, valves, fans, or filters outside the mechanical equipment rooms.

Alternatively, air-and-water and all-water systems require less space and offer better individual control of temperature in the occupied spaces than some all-air systems. However, they are inherently more complicated and much of their maintenance work must carried out in occupied spaces of the building. For these reasons, an all-air system is generally the most suitable HVAC system for high-rise office buildings, and is the only system considered hereafter in this study.

Major Components in All-Air HVAC Systems. The designer must consider the following major components involved in the design of an all-air HVAC system (Allen and Iano 1995): boilers and chimneys; chillers; cooling tower; fan room; outdoor fresh air and exhaust louvres; and vertical and horizontal supply and return ducts, supply diffuser and return grills.

Horizontal ducting is usually concealed between a false ceiling and the ceiling. As shown in Figure 2.B.2, the wiring and ductwork share the above-ceiling space with lighting fixtures and sprinkler piping, which requires careful planning. Generally the lowest layer, about 200mm thick, is reserved for the sprinkler piping and lighting fixtures. Lighting fixture selection plays an important role in determining the thickness of this lower layer because some types of lighting fixtures require more space than others. The HVAC ducts, which are usually 200 to 250 mm deep, run above the lower layer and just below the beams and girders for the floor system above.

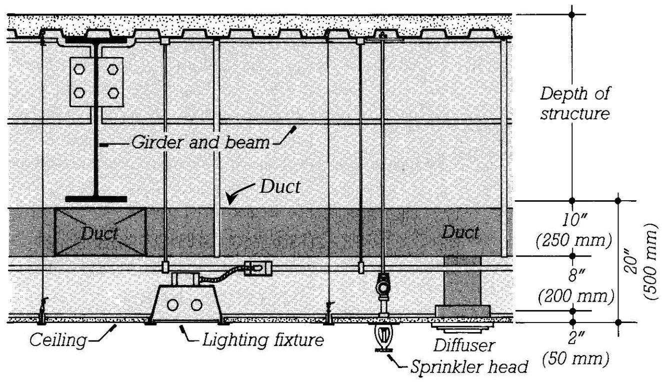


Figure 2.B.2: Schematic of Mechanical/Electrical Assembly in False Ceiling

Adding about 50 mm to account for the thickness of the suspended ceiling, it is generally the case that a minimum height of about 450 to 500 mm must be added to the thickness of the floor system in a typical building to allow for mechanical and electrical services. This causes the depth of the ceiling-floor assembly in the average tall office building to be about 1150 mm.

Other bigger components of the HVAC system cannot be concealed within the floors due to their size and demand their own special place in the building. The cooling tower is usually placed on top of the roof and a fan room is located on each floor within the core area. In fact, the fan room(s) may be located anywhere in the building, as shown in Figure 2.B.3.

The boilers and chillers for the HVAC system require special areas separate from the occupied spaces of the building due to their excessive noise. A boiler room for a large building normally contains at least two boilers, so that one may be in service while the other is being cleaned or repaired. The boiler room may be placed anywhere in a building, and common locations are in the basement, a mechanical room on grade, a mechanical floor, or on the roof. To reduce needed space, it is helpful to locate the boiler room next to the chilled water plant. The two facilities are often combined in the same space on a mechanical floor. The ceiling height in a chilled water plant varies from a minimum of 3.7 m for a building of a moderate size to a maximum of 4.9 m for a very large building, and the total space for the boiler room and chilled water plant is almost 4% of the total floor area for a large building (Allen and Iano 1995).

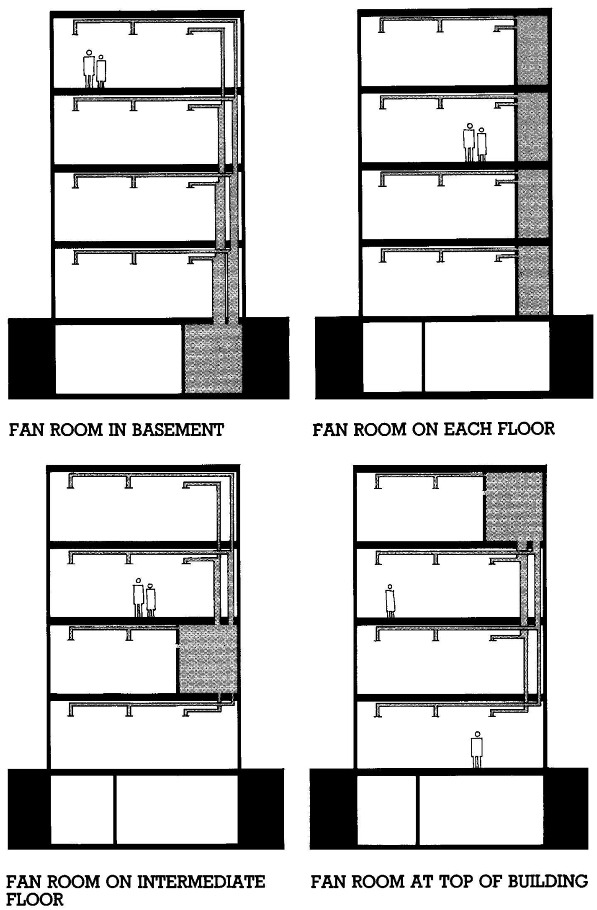


Figure 2.B.3: Different Locations for Fan Room(s)

The maximum vertical “reach” of a fan room is approximately 25 stories up and/or down; however, more typically, fan rooms are located so that none need to circulate air more than 11 to 13 stories in each direction (Allen and Iano 1995). Multiple fan rooms distributed throughout the building are often desirable because they allow the building to be zoned for better local control and tend to reduce the total volume of ductwork in the building. It is often advantageous to have a separate fan room for each floor of a building because such an arrangement saves floor space by eliminating most or all of the vertical runs of ductwork. The space on each floor occupied by the fan room is approximately 2.7% of the floor area. For an HVAC system having but a few fan rooms that serve the entire building, the total area needed for the fan rooms is 2.7% of the total floor area of the building (Allen and Iano 1995).

2.B.2 Elevator Systems

Because of its many complexities, an elevator system is usually designed by an elevator consultant or the engineering department of an elevator manufacturer. Discussed in the following are guidelines for the preliminary determination of the number of elevators needed and the allocation of corresponding space in the building. It is first noted that vertical elevator systems have a severe impact on the design of a building. Secondly, as elevators become extremely expensive as the height of a building increases, it is prudent to accurately estimate their cost a priori in order to arrive at an overall optimal conceptual design of the building. The following are the constraints and costs to be considered for the design of the vertical elevator system for a high-rise building: architectural and structural constraints (for placement within the core); initial cost; and annual operating cost (maintenance and energy).

Table 2.B.1 presents the minimum requirements for the arrangement of elevators in an office building (i.e., number of elevators and appropriate size of the cars). In very tall buildings, the number of shafts can be reduced somewhat by grouping together express and local elevators. Local elevators in high and low zones of the building can even run in the same shaft to save floor space. In some buildings, two-story lobbies served by two-story elevators can reduce the number of shafts by as much as one third. This study, however, will only consider regular use of elevators over the entire height of the building.

Table 2.B.1: Minimum Number of Elevator Shafts and Elevator Dimensions

Number of Elevators	**Capacity of Elevator (lb)**	**Inside Car Dimensions**	**Inside Shafts Dimensions**
1 per 3250 m^2 of area served, plus 1 service elevator for 24,600 m^2 of area served	3000	2032 x 1448 mm	2540 x 2261 mm

From Table 2.B.1, it is clear that the number of elevators in a tall building is not a function of the height of the building but, rather, of its total floor area. This is because the speed of elevators is increased as the height of building increases, rather than increasing the number of elevators, which demands less space on each floor. Walking distance from the elevator lobby to any location on a floor in an office building should not exceed 45m; the minimum width of an elevator lobby serving a single bank of elevators should be 2.45 m; while the minimum width for a lobby with banks of elevators on both sides is 3m (Allen and Iano 1995). For most buildings, including very tall ones, the most widely used elevator type is an electric traction elevator having its machine room at the top of the shaft.

Stair Cases. While stair cases are not part of the mechanical systems for a building, it is appropriate to mention the rules that govern their design here, just after discussing vertical elevator systems.

Stair width and exit discharges widths are based on the occupant load of the largest single floor. Occupant loads do not accumulate from one floor to the next, except at the floor of exit discharge for people who converge there from adjacent floors (Allen and Iano 1995). The minimum numbers of stair cases and exits required by NBCC (National Building Code of Canada 1990) are presented in Table 2.B.2. Based on NBCC guidelines, the occupancy load for an office building is estimated to be 9.3m ^{2} per person, and the minimum width of each stair should be at least 9.2 mm per number of persons assigned to that stair case and not less than 1.1 meter.

Table 2.B.2: Minimum Number of Stair Cases and Exits

Occupancy Load per Story	**Number of Stairs Cases and Exits**
500 Persons or fewer	2
501 to 1000 persons	3
More than 1000 persons	4