CHAPTER TWO

LITERATURE REVIEW

2.1Introduction:

Structural concrete can be used for almost all buildings, whether single story or multistory. The concrete building may contain some or all of the following main structural elements [13]:

- Slabs are horizontal plate elements in building floors and roofs, they may carry vertical loads as well as horizontal loads.
- Beams are long horizontal or inclined members, their main function is to support loads from slab.
- Column are critical members that support axial loads from beams or slabs
- Frames are structural member that consist of a combination of columns and Beams or slabs.
- Footings are members that support columns and spread their loads directly to the soil.
- Walls are vertical plate elements resisting vertical loads as well as horizontal loads.

2.2 Loading:

Loading on high-rise buildings differs from loading on low-rise buildings in its accumulation into much larger structural forces, in the increased significance of wind loading, and in the greater importance of dynamic effects. The collection of gravity loading over a large number of stories in high-rise buildings can produce column loads of an order higher than those in low-rise buildings. Wind loading on high-rise buildings act not only over a very large building surface, but also with greater intensity at the greater heights and with a large moment arm about the base than on a low-rise building. Although wind loading on a low rise-building usually has an insignificant influence on the design of the structure, wind on a high-rise building can have a dominant influence on its structural arrangement and design. In an extreme case of a very slender or flexible structure, the motion of the building due to wind may have to be considered in assessing the loading applied by the wind. In earthquake regions, any inertial loads from the shaking of the ground may well exceed the loading due to wind and, therefore, be dominant in influencing the building's structural form, design, and cost. As an inertial problem, the building's dynamic response plays a large part in influencing, and in estimating, the effective loading on the structure [1]. Perhaps the most important and the most difficult task faced by the structural designer is the accurate estimation of the loads that may be applied to a structure during it`s life. No loads that may reasonably be expected to occur may be overlooked. After loads are estimated, the next problem is to decide the worst possible combinations of these loads that might occur at one time [2].

2.2.1Gravity Load:

Although the tributary areas, and therefore the gravity loading, supported by the beams and slabs in a high-rise building do not differ from those in a low-rise building, the accumulation in the former of many stories of loading by the columns and walls can be very much greater. As in a low-rise building, dead loading is calculated from the designed member sizes and estimated material densities. This is prone to minor inaccuracies such as differences between the real and the designed sizes, and between the actual and the assumed densities. Live loading is specified as the intensity of a uniformly distributed floor load, 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 loads [1].

2.2.2Impact Gravity loading:

Impact loading occurs as a gravity live load in the case of an elevator being accelerated upward or brought to a rest on its way down. An increase of 100% of the static elevator load has usually been used to give a satisfactory performance of the supporting structure [1].

2.2.3Construction loads:

Construction loads are often claimed to be the most basic loads that a building has to withstand. Certainly, many more failures occur in building under construction than in those that are complete, but it is rare for special provision to be made for construction loads in high-rise building design. Typically, the construction load that has to be supported is the weight of the floor forms and a newly placed slab, which, in total, may equal twice the floor dead load. This load is supported by props that transfer it to the previously constructed floors below. Now, with the possibility of as little as 3-day cycle ,or oven 2-day cycle, story construction, and especially with concrete pumping, which requires a more liquid mix, the problem is more severe; this is because the newly released slab, rather than contributing to supporting the construction loads, is still in need of support itself [1].

2.2.4Wind loading:

Wind is a term used to describe horizontal motion of air, motion in a vertical direction is called a current. Winds are produced by differences in atmospheric pressure that are primarily attributable to differences in temperature [3] The lateral loading due to wind or earthquake is the major factor that causes the design of high-rise buildings to differ from those of low to medium-rise buildings. For buildings of up to about 10 stories and of typical proportions, the design is rarely affected by wind loads. Above this height, however, the increase in size of the structural member, and the possible rearrangement of the structure to account for wind loading, incurs a cost premium that increases progressively with height. With innovations in architectural treatment, increases in the strengths of materials, and advances in methods of analysis, high-rise building structures have become more efficient and lighter and, consequently, more prone to deflect and even to sway under wind loading. This served as a spur to research, which has produced significant advances in understanding the nature of wind loading and in developing methods for its estimation. These developments have been mainly in experimental and theoretical techniques for determining the increase in wind loading due to gusting and the dynamic interaction of structures with gust forces [1].

2.2.4.1 Simple Static Approach:

This method is representative of modern static methods of estimating wind loading in that it accounts for the effects of gusting and for local extreme pressures over the faces of building. It also accounts for local differences in exposure between the open countryside and a city center, as well as allowing for vital facilities such as hospitals, and fire and police stations, whose safety must be ensured for use after an extreme windstorm [1].

2.2.4.2 Dynamic Method:

If the building is exceptionally slender or high-rise, the effective wind loading on the building may be increased by dynamic interaction between the motion of the building and the gusting of the wind. If it is possible to allow for it in the budget of the building, the best method of assessing such dynamic effects is by wind tunnel tests in which the relevant properties of the building and the surrounding countryside are modeled. For buildings that are not so extreme as to demand a wind tunnel test, but for which the simple design procedure is inadequate, alternative dynamic methods of estimating the wind loading by calculation have been developed [1]

2.2.4.3 Wind Tunnel Experimental Method:

Wind tunnel tests to determine loading may be quasi steady for determining the static pressure distribution or force on a building. The pressure or force coefficients so developed are then used in calculating the full-scale loading through one of the described simple methods. This approach is satisfactory for buildings whose motion is negligible and therefore has little effect of the wind loading. If the building slenderness or flexibility is such that its response to excitation by the energy of the gusts may significantly influence the effective wind loading, the wind tunnel test should be a fully dynamic one. In this case, the elastic structural properties and the mass distribution of the building as well as the relevant characteristics of the wind should be modeled. Building models for wind tunnel tests are constructed to scales which vary from 1/100 to 1/1000, depending on the size of the building and the size of the wind tunnel, with a scale of 1/400 being common. High-rise buildings typically exhibit a combination of shear and bending behavior that has a fundamental sway mode comprising a flexurally shaped lower region and a relatively linear upper region. This can be represented approximately in wind tunnel tests by a rigid model with a flexurally sprung base. It is not necessary in such a model to represent the distribution of mass in building, but only its moment of inertia about the base. More complex models are used when additional modes of oscillation are expected including, possibly, torsion. These models consist of lumped masses, springs, and flexible rods, designed to simulate the stiffnesses and mass properties of the prototype. Wind pressure measurements are made by flush surface pressure taps on the faces of the models, and pressure transducers are used to obtain the mean, root mean square (RMS), and peak pressures. The wind characteristics that have to be generated in the wind tunnel are the vertical profile of the horizontal velocity, the turbulence intensity, and the power spectral density of the longitudinal component. Special (boundary layer) Wind tunnels have been designed to generate these characteristics. Some use long working sections in which the boundary layer develops naturally over a rough floor; other short ones include grids, fences, or spires at the test section entrance together with a rough floor, while some activate the boundary layer by jets or driven flaps. The working sections of the tunnel are up to a maximum of about 6 ft^2 and they operate at atmospheric pressure [1].

2.2.5Earthquake Loading:

Earthquake loading consists of the building mass that result from the shaking of its foundation by a seismic disturbance. Earthquake resistant design concentrates particularly on the translational inertia forces, whose effects on a building are more significant than the vertical or rotational shaking components. Other severe earthquake forces may exist, such as those due to land sliding, subsidence, active faulting below the foundation, or liquefaction of the local sub-grade as a result of vibration. These disturbances, however, which are local effects, can be so massive as to defy any economic earthquake-resistant design, and their possibility may suggest instead the selection of an alternative site. Where earthquakes occur, their intensity is related inversely to their frequency of occurrence; severe earthquakes are rare, moderate ones occur more often, and minor ones are relatively frequent. Although it might be possible to design a building to resist the most severe earthquake without significant damage, the unlikely need for such strength in the lifetime of the building would not justify the high additional cost [1].

2.2.6 Design Load Combinations:

The design load combinations are used for determining the various combinations of the load cases for which the structure needs to be designed or checked. The load combination factors are applied to the forces and moments obtained from the analysis cases. ForACI318-05 code, if a structure is subjected to dead load (DL), live load (LL), wind load (WL), the following load combinations may need to be defined(ACI318-05 9.2.1), [10].

1.4D (2-1)

1.2DL + 1.6LL (2-2)

0.9DL$\pm$ 1.6WL (2-3)

1.2DL +1.0LL$\pm$ 1.6W (2-4)

2.2.7Lateral Loads Resisting Systems:

It can be said that there are many types of lateral systems as there are engineers. However, most of the systems can be grouped into three basic types: (1) shear wall system, (2) frame system, and (3) combination of the two, the shear wall-frame system (dual system) [3].

2.2.7.1Floor Systems:

2.2.7.1.1 Two-way Flat Plate System:

A uniformly thick two-way reinforced slap is supported by columns or individual short walls. It can be span up to (8m) in the ordinary reinforced form and up to (11m) when post tensioned. Because of its simplicity and its uniform thickness if allows considerable freedom in the location of the supporting columns and walls, with the possibility of using the clear soffit as a ceiling .It results in minimum story height [1]. {Fig. (2.1(a))}

2.2.7.1.2 Two-way Flat Slab System:

The flat slab differs from the flat plate in having capitals and /or drop panels at the tops of the columns the capitals increase the shear capacity while the drop panels increase both the shear and negative moment capacities at the supports .where the maximum values occur. The flat slap plate for behavior Loading and reinforcement. It is most suitably used in square or near-to-square arrangements [1]. {Fig. (2.1(b))}

2.2.7.1.3 Waffle Flat Slab:

A slab is supported by a square grid of closely spaced joists with filler panels over the columns .The flat slabs and joists are poured integrally over square domed forms .which are omitted around the columns to create the filler panels. The forms which are of size sup to (76m)square and up to (50cm) in deep. Provide geometrically interesting soffit. which is often left with out further finish as the ceiling [1]. {Fig(2.1(c))}

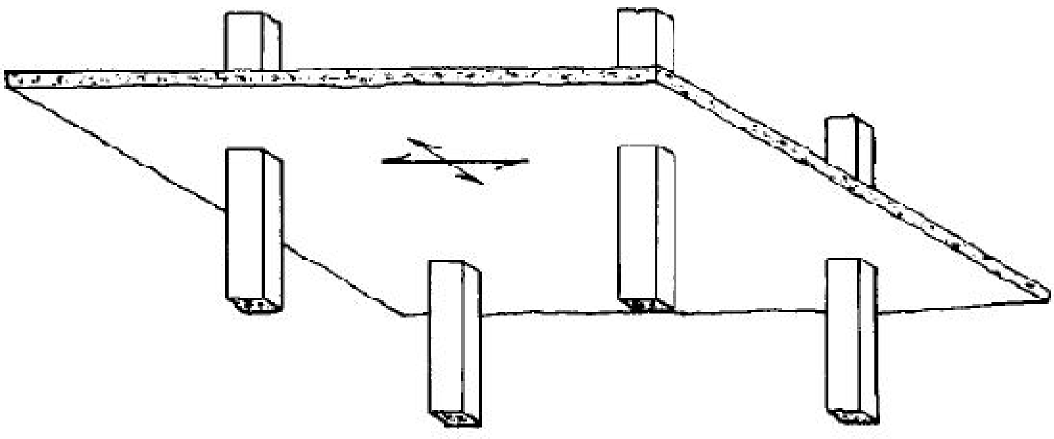


Fig. (2-1a): Flat Plate

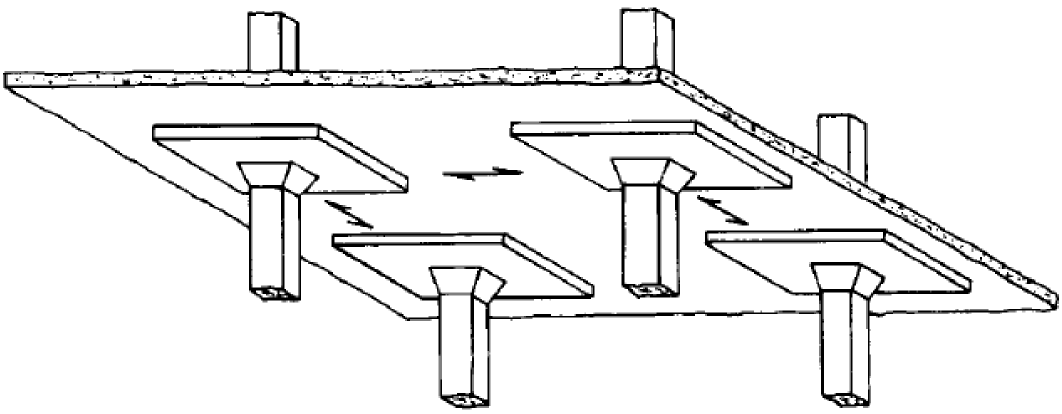


Fig. (2-1b): Flat Slab

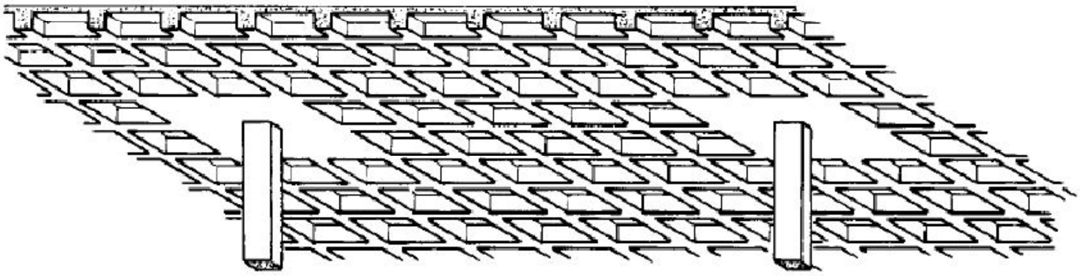


Fig. (2-1c): Waffle Slab

2.2.7.2Shear Wall Structures:

A shear wall structure is considered to be one whose resistance to horizontal loading is provided entirely by shear walls. The walls may be part of a service core of a stairwell, or they may serve as partitions between accommodations. They are usually continuous down to the base to which they are rigidly attached to form vertical cantilevers. Their high in-plane stiffness and strength makes them well suited for bracing buildings of up to about 35 stories, while simultaneously carrying gravity loading. It is usual to locate the walls on plan so that they attract an amount of gravity dead loading sufficient to suppress the maximum tensile bending stresses in the wall caused by lateral loading. In this situation, only minimum wall reinforcement is required. The term “shear wall” is in some ways a misnomer because the walls deform predominantly in flexure. Shear walls may be planar, but are often L, T, I or U shaped section to better suit the planning and to increase their flexural stiffness [1]. {Fig. (2.2)}

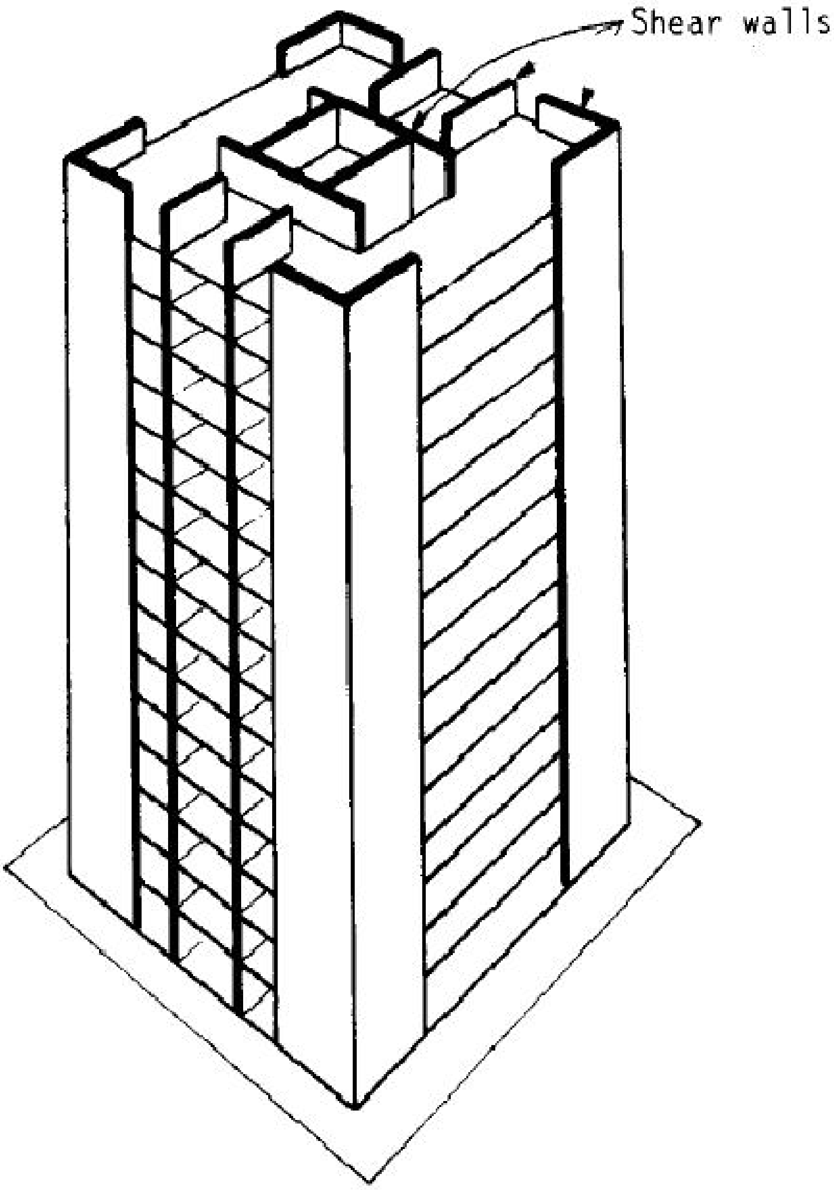


Fig. (2-2): Shear Walls

2.2.7.2.1 Behavior of Shear Wall Structures:

A proportionate system is one in which the ratios of the flexural rigidities of the walls remain constant throughout their height. For example, a set of walls whose lengths do not change throughout their height, but whose changing wall thicknesses are the same at any level, is proportionate. Proportionate systems of walls do not incur any redistribution of shears or moments at the change levels.

The static determinacy of proportionate systems allows their analysis to be made by considerations of equilibrium [1].

A non-proportionate system is one in which the ratios of the walls flexural rigidities are not constant up the height. At levels where the rigidities change, redistributions of the wall shears and moment occur. With corresponding horizontal interactions in the connecting members and the possibility of very high local shears in the walls. Non-proportionate structures are statically indeterminate and therefore much more difficult to visualize in behavior, and to analyze [1].

2.2.7.3Rigid-Frame Structures:

A rigid-frame high-rise structure typically comprises parallel or orthogonally arranged bents consisting of columns and girders with moment resistant joints. Resistance to horizontal loading is provided by the bending resistance of the columns, girders, and joints [1].

The advantages of a rigid frame are the simplicity and convenience of its rectangular form. Its unobstructed arrangement, clear of bracing members and structural walls, allows freedom internally for the layout and externally for the fenestration. Rigid frames are considered economical for buildings of up to about 25 stories, above which their drift resistance is costly to control.

As highly redundant structures, rigid frames are designed initially on the basis of approximate analysis, after which more rigorous analyses and checks can be made. The procedure may typically include the following stages [1]:

- Estimation of gravity load forces in girders and columns by approximate method.
- preliminary estimate of member sizes based on gravity load forces with arbitrary increase in sizes to allow for horizontal loading.
- Approximate allocation of horizontal loading to bents and preliminary analysis of member forces in bents.

- check on drift and adjustment of member sizes if necessary.
- check on strength of members for worst combination of gravity and horizontal loading.
- computer analysis of total structure for more accurate check on member strengths and drift.
- Detailed design of members and connections.

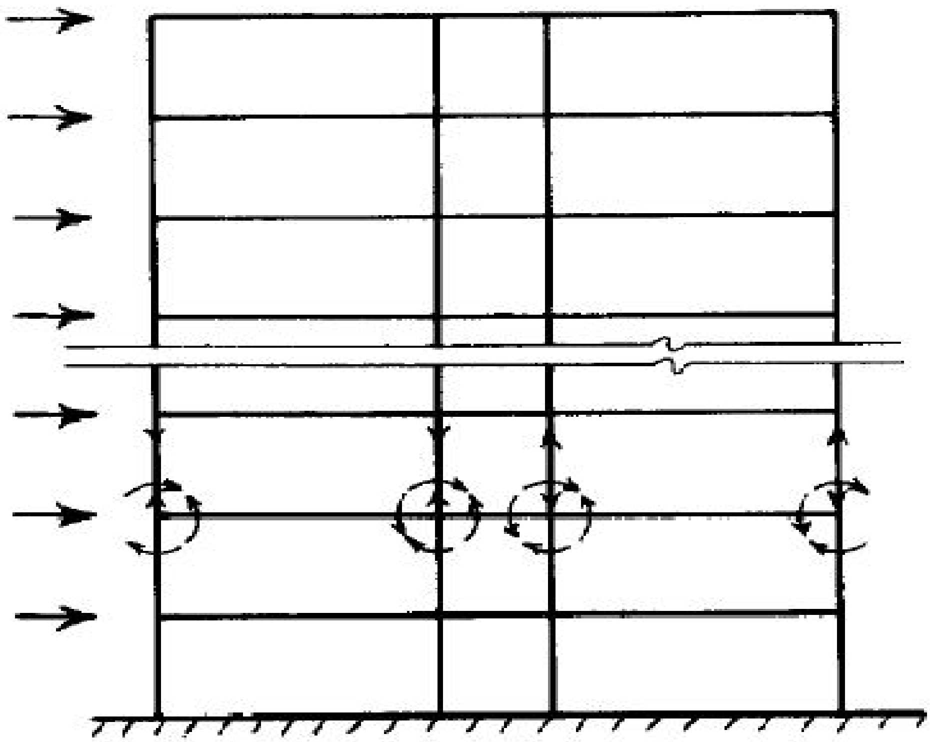


Fig.(2-3): Rigid Frame Structure

2.2.7.3.1Behavior of Rigid Frame:

The horizontal stiffness of a rigid frame is governed mainly by the bending resistance of the girders, the columns, and their connections, and, in a tall frame, by the axial rigidity of the column. The accumulated horizontal shear above any story of a rigid frame is resisted by shear in the columns of that story {Fig. (2.4)}, the shear causes the story-height columns to bend in double curvature with points of contra flexure at approximately mid-story height levels. The overall moment of the external horizontal load is resisted in each story level by the couple resulting from the axial tensile and compressive forces in the columns on opposite sides of the structure [1]. {Fig.(2.4)}

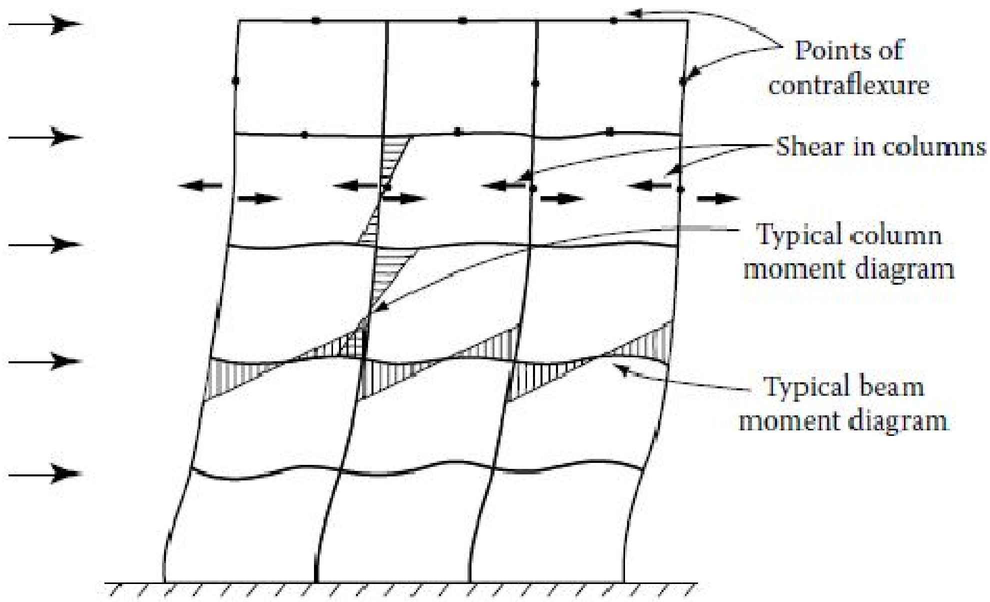


Fig. (2-4): Deformations Caused by External Shear Forces and Moment.

2.2.7.4Braced Frame:

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. Historically, bracing has been used to stabilize laterally the majority of the world tallest building structures, from the earliest examples at the end of the nineteenth century to the present time. The Status of Liberty, constructed in NEW YORK in 1883, was one of the first major braced structure. In the following three decades large numbers of braced steel-frame tall buildings were erected in Chicago and New York [1].

2.2.7.4.1 Types of Bracing:

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. For this reason it is usually concentrated in vertical panels or bents that are located to cause a minimum of obstruction while satisfying the structural requirements of resisting the shear and torque on the building. 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. The full-diagonal types of braced bent are usually located where passage is not required [1]. {Fig. (2.5)}

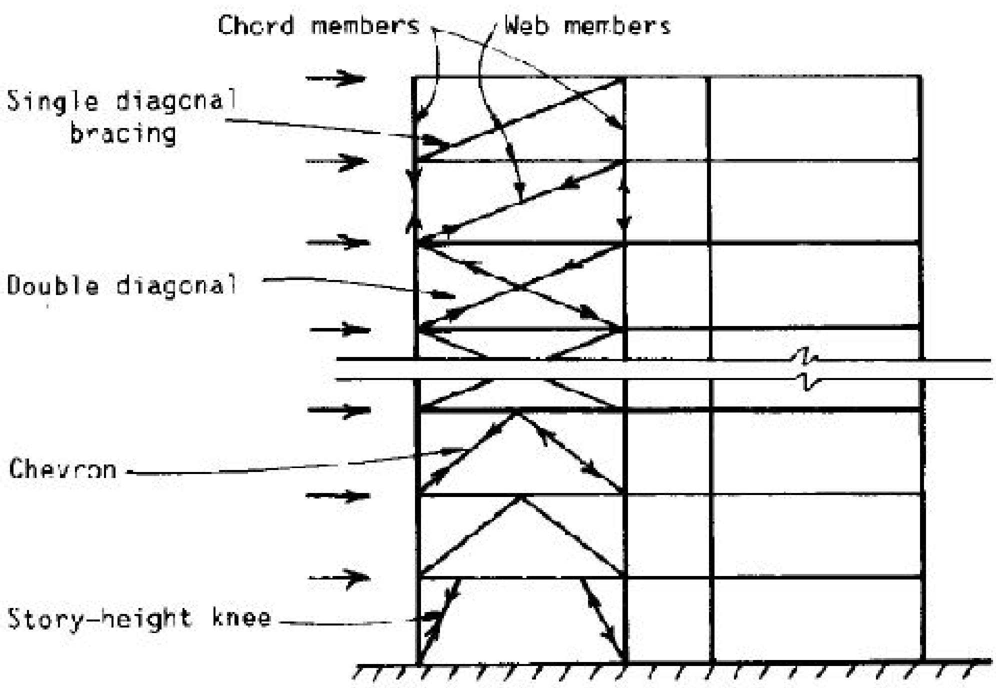


Fig. (2-5): Braced Frame -Showing Different Types of Bracing.

2.2.7.4.2 Behavior of Bracing:

Because of lateral loading on a building is reversible, braces will be subjected in turn to both tension and compression: consequently, they are usually designed for he more stringent case of compression. For this reason, bracing system with shorter braces, for example the K-types, may be preferred to the full diagonal types. Eccentric bracing systems may be used to design a ductile structure for an earthquake-resistant steel-framed building. The bracing acts in its usual elastic manner when controlling drift against wind or minor earthquakes [1].

2.2.7.5 Coupled Shear Wall Structures:

In the design of slab residential blocks consisting of walls and floor slabs only, self-contained apartment units are generally arranged on opposite sides of a central corridor along the length of the building. This arrangement naturally results in parallel assemblies of division walls running perpendicular to the face of the building, with intersecting longitudinal walls along the corridor and facade enclosing space, and providing fire and acoustic insulation between dwellings. The cores walls are employed as load bearing walls, since their disposition favors an efficient distribution of both gravity and lateral loads to the structure elements. If the floor slabs are rigidly connected to the walls, they serve in effect as connecting beams to produce a shear interaction between the two in-plane cross walls. Such structures, which consist of walls that are connected by bending-resistant connections greatly increase the stiffness and efficiency of the wall system [1].

2.2.7.5.1 Behavior of Coupled Shear Wall Structures:

If a pair of in-plane shear walls is connected by pin-ended links that transmit only axial forces between them, any applied moment will be resisted by individual moments in the two walls, the magnitudes of which will be proportional to the walls flexural rigidities. The bending stresses are then distributed linearly across each wall, with maximum tensile and compressive stresses on opposite edges. If, on the other hand, the walls are connected by rigid beams to form a dowelled vertical cantilever, the applied moment will be resisted by the two walls acting as a single composite unit, bending about the centered axis of the two walls. The bending stresses will then be distributed linearly across the composite unit, with maximum tensile and compressive stresses occurring at the opposite extreme edges. When the walls deflect under the action of the lateral loads, the connecting beam ends are forced to rotate and displace vertically, so that the beams bend in double curvature and thus resist the free bending of the walls. The bending action induces shears in the connecting beams, which exert bending moments, of opposite sense to the applied external moments, on each wall. The shears also induce axial forces in the two walls, tensile in the windward wall and compressive in the leeward wall. The wind moment M at any level is then resisted by the sum of the bending moments M_1, and M_2 in the two walls at that level, and the moment of the axial forces NI, where N is the axial force in each wall at that level and I is the distance between their centered axes [1]. {Fig. (2.6)}

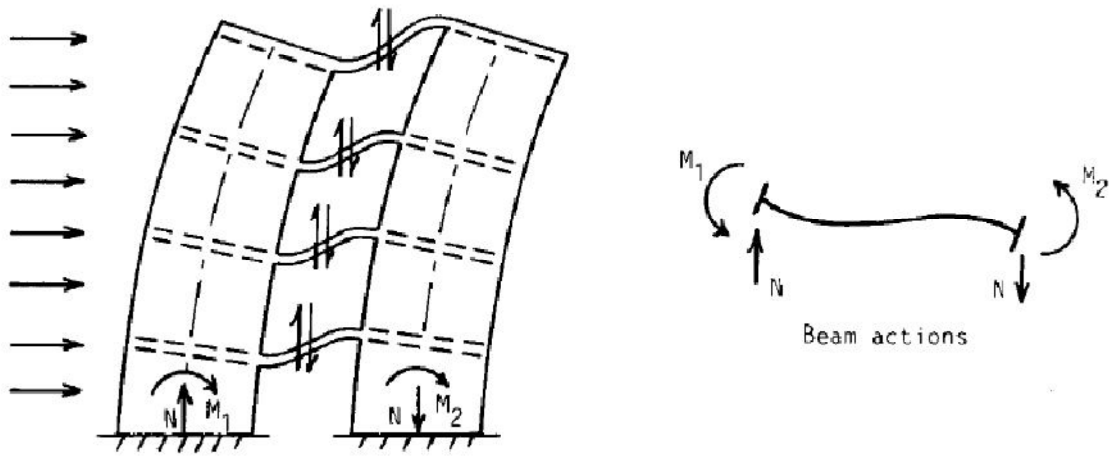


Fig. (2-6): Behavior of Laterally Loaded Coupled Shear Walls.

$$\mathbf{M = M_1 + M_2 + NI}$$

The last term NI represents the reverse moment caused by the bending of the connecting beams which opposes the free bending of the individual walls. This term is zero in the case of linked walls, and reaches a maximum when the connecting beams are infinitely rigid. The action of the connecting beams is then to reduce the magnitudes of the moments in the two walls by causing a proportion of the applied moment to be carried by axial forces. Because of the relatively large lever arm involved, a relatively small axial stress can give rise to a disproportionally larger resistance moment. The maximum tensile stress in the concrete may then be greatly reduced. This makes it easier to suppress the wind tensile stresses by gravity load compressive stresses [1].

2.2.7.6 Wall-Frame Structure:

A structure whose resistance horizontal loading is provided by a combination of shear walls and rigid frames or, in the case of a steel structure, by braced bents and rigid frames, may be categorized as a wall-frame. The shear walls or braced bents are often parts of the elevator and service cores while the frames are arranged in plan, in conjunction with the walls, to support the floor systems [3].

Now, wall-frame structure that do not twist and, therefore, that can be analyzed as equivalent planar models. These are mainly plan-symmetric structure subjected to symmetric loading. Structure that are asymmetric about the axis of loading inevitably twist. Although the benefits from horizontal interaction between the walls and frames apply also to twisting structures, their consideration in a general way is extremely complex because the amount of interaction is highly dependent on the relative plan location of the bents.

Two examples of symmetric wall-frame arrangements are shown in plan and symmetric structure is shown in fig. (2.7). The horizontal resistance is provided by walls and frames in parallel bents, which are constrained to deflect identically by the in-plane rigidity of the floor slabs and, therefore, interact horizontally through shearing actions in the slabs. each of the parallel bents consists of a wall and a frame in the same plane. In this case, the wall and frame in a planar bent interact horizontally through axial forces in the connecting beams or slabs[3].

The potential advantages of a wall-frame structure depend on the amount of horizontal interaction, which is governed by the relative stiffnesses of the walls and frames, and the height of the structure. In typically proportioned structures, the stiffer the frames, the greater the interaction. It used to be common practice in the design of high-rise structures to assume that the shear walls or cores resisted all the lateral loading, and to design the frames for gravity loading only. Although this assumption would have incurred little error for buildings of less than 20 stories with flexible frames, it is possible that in many cases where the frames were stiff and the buildings taller, opportunities were missed to design more rational and economical structure [3].

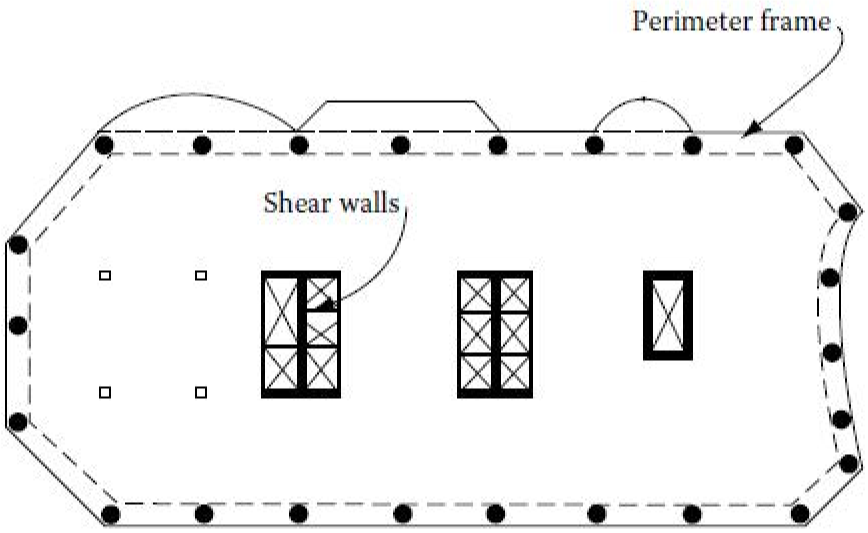


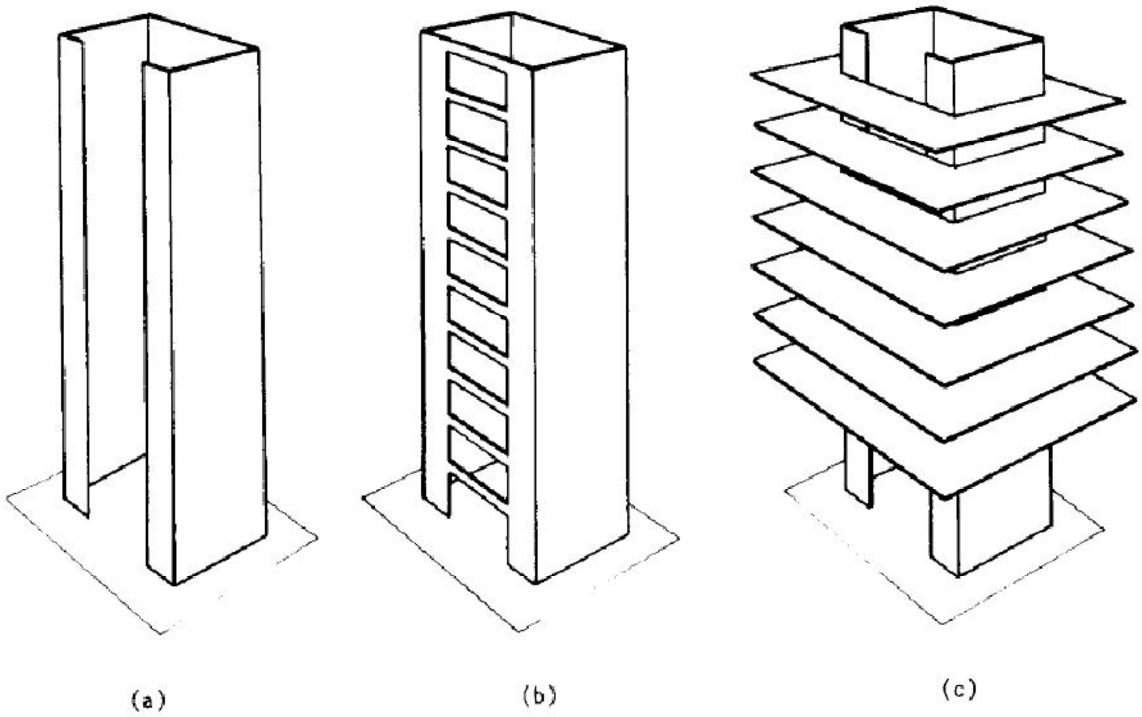
Fig. (2-7): Shear Wall Frame Structure

2.2.7.6.1 Behavior of Symmetric Wall Frames:

Considering the separate horizontal stiffnesses at the tops of a typical 10-stories elevator core and a typical rigid frame of the same height, the core might be 10 or more times as stiff as the frame. If the same core and frame were extended to height of 20 stories, the core would then be only approximately three times as stiff as the frame. At 50 stories the core would have reduced to being only half as stiff as the frame. This change in the relative top stiffness with the total height occur because the top flexibility of the core, which behaves as a flexural cantilever, proportional to the cube of the height, whereas the flexibility of the frame, which behaves as a shear cantilever, is directly proportional to its height. When the wall and frame are connected together by pin-ended links and subjected to horizontal loading, the deflected shape of the composite structure has a flexural profile in the lower part and a shear profile the upper part. The deflection curve and the wall moment cure indicate the reversal in curvature with a point of inflexion, about which the wall moment is opposite in sense to that in a free cantilever, and the shear as approximately uniform over the height of the frame except near the base where it reduces to a negligible amount. At the top, then the external shear is zero, the frame is subjected to a significant positive shear, which is balanced by an equal negative shear at the top of the wall, with a corresponding concentrated interaction force distribution between the frame and the wall. Special consideration may have to be given in the design to transferring this interaction force through the top connection slab or beam [1].

2.2.7.7Core Structures:

Elevator Cores are primary components for resisting both horizontal and vertical loading in tall building structures. Reinforced concrete cores usually comprise an assembly of connected shear walls forming a box section with opening that may be partially closed by beams or floor slabs [1].The moments of inertia of a reinforced concrete core are invariably large, So that it is often adequate in itself to carry the whole of the lateral loading. The horizontal load bending deflection and stresses of a core with a fully connected section are calculated conventionally, as for a vertical cantilever, on the basis of the core’s moments of inertia about its principal axes [6]. (figs. (2.8 a, b, c)).



Fig(2-8)Core Structure (a) Open Section (b) Core Partially closed by Beam(c) Core Partially Closed by Floor Slabs.

2.2.7.8 Framed-Tube Structures:

In structural engineering tube system, a building is designed to act like a hollow cylinder, cantilevered perpendicular to the ground in order to resist lateral loads (wind, seismic). The lateral resistance of framed-tube structures is provided by very stiff moment-resisting frames that form a (tube) around the perimeter of the building. The frames consist of closely spaced columns, (4-6)m between centers, joined by deep spandrel girders, (0.9-1.5)m. Although the tube carries all the lateral loading, the gravity loading is shared between the tube and interior columns or walls. When lateral loading acts, the perimeter frames aligned in the direction of loading act as the (web) of the massive tube cantilever, and those normal to the direction of the loading act as the (flanges). The exterior framing is designed sufficiently strong to resist all lateral loads on the building, thereby allowing the interior of the building to be simply framed for gravity loads. Interior columns are comparatively few and located at the core. The distance between the exterior and the core frames is spanned with beams or trusses and intentionally left column-free. This maximizes the effectiveness of the perimeter tube by transferring some of the gravity loads within the structure to it and increases its ability to resist overturning due to lateral loads. The tube form was developed originally for buildings of rectangular plan, and probably its most efficient use is in that shape. The tube is suitable for both steel and reinforced concrete construction and has been used for buildings ranging from 40 to more than 100 stories. It can be used for office, apartment and mixed-use buildings. Most buildings in excess of 40 stories constructed since the 1960s are of this structural type [3]. (Fig. (2.9)).

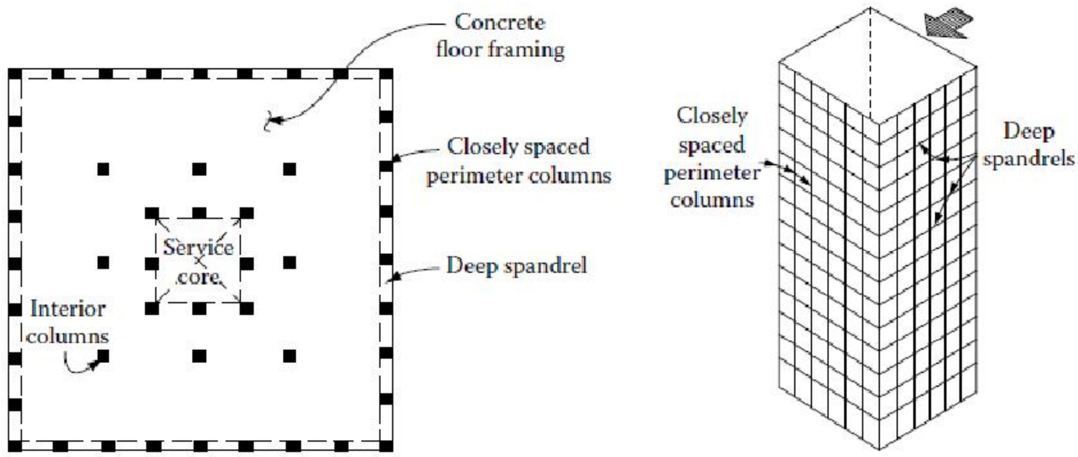


Fig. (2-9): Framed Tube structure.

2.2.7.8.1 Behavior of Framed Tube:

To understand the behavior of a framed tube, consider building shown in Figure (2.9) in which the entire lateral resistance is provided by closely spaced exterior columns and deep spandrel beams.

The floor system, typically considered rigid in its own plane, distributes the lateral load to various elements according to their stiffness. The lateral load-resisting system thus comprises four orthogonally oriented, rigidly jointed frame panels forming a tube in plan, as shown in Figure (2.9).

The “strong” bending direction of the columns is typically aligned along the face of the building, in contrast to a typical transverse rigid frame where it is aligned perpendicular to the face. The frames parallel to the lateral load act as webs of the perforated tube, while the frames normal to the load act as the flanges. When subjected to bending, the columns on opposite sides of the neutral axis of the tube are subjected to tensile and compressive forces. In addition, the frames parallel to the direction of the lateral load are subjected to the in-plane bending and the shearing forces associated with an independent rigid frame action. The discrete columns and spandrels distributed around the building periphery may be considered, in a conceptual sense, equivalent to a hollow tube cantilevering from the ground [3], as shown in Figure (2.10).

Although the structure has a tube-like form, its behavior is much more complex than that of a solid tube. Unlike a solid tube, it is subjected to the effects of shear lag, which has a tendency to modify the axial distribution in the columns. The influence of shear lag, is to increase the axial stresses in the corner columns while simultaneously reducing the same in the inner columns of the flange and the web panels [1].

The fundamental behavior explained with reference to the tube in Figure (2.10). Although in simplistic terms, the tube is similar to a hollow cantilever, in reality its response to lateral loads is in a combined bending and shear mode. The bending mode is due to axial shortening and elongation tube of the columns, whereas the shear mode is due to bending of individual columns and spandrels. The underlying principle for an efficient design is to eliminate or minimize shear deformation [3].

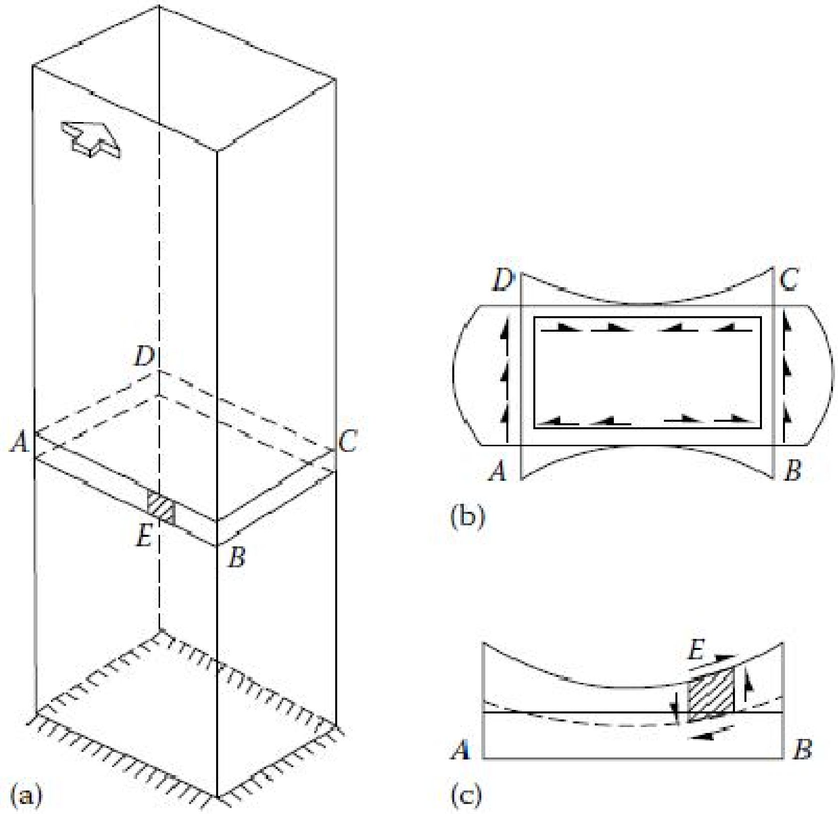


Fig. (2-10): Shear Lag Effects in a Hollow Tube Structure

2.2.7.9 Flat Slab-Frame With Shear Walls:

Frame action provided by a flat slab-beam and column interaction is generally insufficient to provide the required strength and stiffness for buildings taller about 10 stories. A system consisting of shear walls and flat slab-frames may provide an appropriate lateral bracing system. Coupling of walls and columns solely by slabs is a relatively weak source of energy dissipation. When sufficiently large rotations occur in the walls during an earthquake, shear transmission from the slab into wall occurs mainly around the inner edges of the wall. Because of cracking of the slab and shear distortions around the columns, the system's hysteretic response is poor [1]. (Fig. (2.11)).

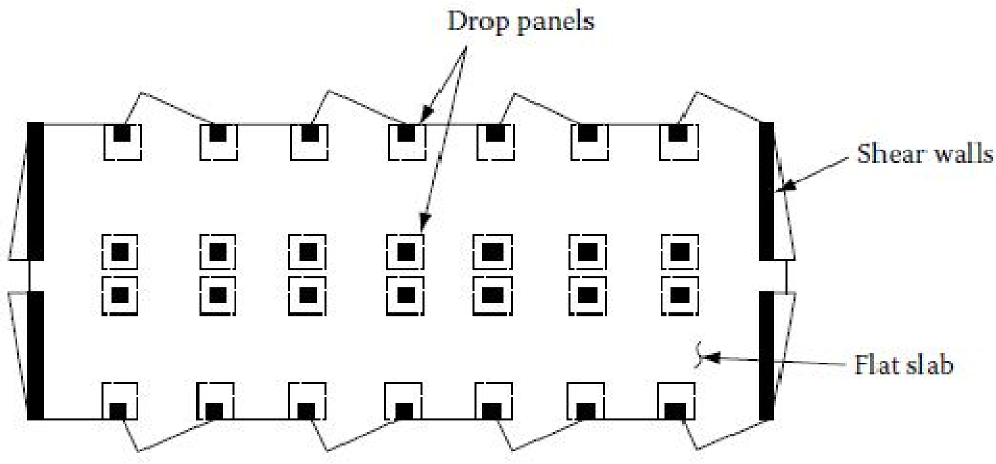


Fig. (2-11): Flat Slab Frame with Shear Walls

2.3 Modeling for Analysis:

A building's response to loading is governed by the components that are stressed as the building deflects. Ideally, for ease and accuracy of the structural analysis, the participating components would include only the main structural elements: slabs, beams, girders, columns, walls, and cores. In reality, however, other, nonstructural, elements are stressed and contribute to the building's behavior; these include, for example, the staircases, partitions, and cladding. To identify the main structural elements, it is necessary to recognize the dominant modes of action of the proposed building structure and to assess the extent of the various members' contribution to them. Then, by neglecting consideration of the nonstructural components, and the less essential structural components, the problem of analyzing a high-rise building structure can be reduced to a more viable size. For extremely large or complex building structures, it may be essential to reduce even further the size of the analysis problem by representing some of the structure's assemblies by simpler analogous components [1].

2.3.1 Approaches to Analysis:

The modeling of a high-rise building structure for analysis is dependent to some extent on the approach to analysis, which is in turn related to the type and size of structure and the stage of design for which the analysis is made. The usual approach is to conduct approximate rapid analyses in the preliminary stages of design, and more detailed and accurate analyses for the final design stages. A hybrid approach is also possible in which a simplified model of the total structure is analyzed first, after which the results are used to allow by part detailed analyses of the structure [1].

2.3.1.1 Preliminary Analyses:

The purpose of preliminary analyses, that is analyses for the early stages of design, may be to compare the performance of alternative proposals for the structure, or to determine the deflections and major member forces in a chosen structure so as to allow it to be properly proportioned. The formation of the model and the procedure for a preliminary analysis should be rapid and should produce results that are dependable approximations. The model and its analysis should therefore represent fairly well, if not absolutely accurately, the principal modes of action and interaction of the major structural elements. The simplifications adopted in making a preliminary analysis are often in the formation of the structural model. Sometimes the approximation is large, as, for example, when numerous hinges are inserted at assumed points of contra flexure in the beams and columns of a rigid frame to convert it from a highly statically indeterminate into a statically determinate system, thus allowing a simple solution using the equilibrium equations. Or the approximation may be to assume a simple cantilever to represent a complex bent, or that a bent is uniform throughout its height and that its beams are (smeared) to allow a continuum solution. These are just a few of the gross approximations that may be made in a structural model to allow a relatively simple preliminary analysis to be achieved [1].

2.3.1.2 Intermediate and Final Analysis:

The requirement of intermediate and final analyses is that they should give, as accurately as possible, results for deflections and member forces. The model should, therefore, be as detailed as the analysis program and computer capacity will allow for its analysis. All the major modes of action and interaction, and as many as possible of the lesser modes, should be incorporated. Except where a structure is symmetrical in plan and loading, the effects of the structure's twisting should be included.

The most complete approach to satisfying the above requirements would be a three-dimensional stiffness matrix analysis of a fully detailed finite element model of the structure. The columns, beams, and bracing members would be represented by beam elements, while shear wall and core components would be represented by assemblies of membrane elements. In contrast to the reductions above, however, certain final analyses may require separate, more detailed analyses of particular parts, using the forces or applied displacements from the main analysis, for example, in deep beams at transition levels of the structure, or around irregularities or holes in shear walls [1].

2.3.2Assumptions:

An attempt to analyze a high-rise building and account accurately for all aspects of behavior of all the components and materials, even if their sizes and properties were known, would be virtually impossible. Simplifying assumptions are necessary to reduce the problem to a viable size. Although a wide variety of assumptions is available, some are more valid than others. The ones adopted in forming a particular model will depend on the arrangement of the structure, its anticipated mode of behavior, and the type of analysis. The most common assumptions are as follows [1].

2.3.2.1 Materials:

The material of the structure and the structural components are linearly elastic. This assumption allows the superposition of actions and deflections and, hence, the use of linear methods of analysis. The development of linear methods and their solution by computer have made it possible to analyze large complex statically indeterminate structures [1].

2.3.2.2 Participating Components:

Only the primary structural components are considered and nonstructural components are assumed to be negligible and conservative. For example, the effects of heavy cladding may be not negligible and may significantly stiffen a structure. Similarly, masonry in fills may significantly change the behavior and increase the forces unconservatively in a surrounding frame [1].

2.3.2.3 Floor Slabs:

Floor slabs are assumed to be rigid in plane. This assumption causes the horizontal plane displacements of all vertical elements at a floor level to be definable in terms of the horizontal plane rigid-body rotation and translations of the floor. Thus the number of unknown displacements to be determined in the analysis is greatly reduced. Although valid for practical purposes in most building structures, this assumption may not be applicable in certain cases in which the slab plan is very long and narrow, or it has a necked region, or it consists of precast units without a topping [1].

2.3.2.4 Negligible Stiffness:

Component stiffness of relatively small magnitude are assumed negligible. These often include, for example, the transverse bending stiffness of slab, the minor-axis stiffness of shear walls, and the torsion stiffness of columns, beams, and walls. The use of this assumption should be dependent on the role of the component in the structure's behavior. For example, the contribution of a slab's bending resistance to the lateral load resistance of a column-and-beam rigid-frame structure is negligible, whereas its contribution to the lateral resistance of a flat plate structure is vital and must not be neglected [1].

2.3.2.5 Negligible Deformation:

Deformations that are relatively small, and of little influence, are neglected. These include the shear and axial deformations of beams, the previously discussed in-plane bending and shear deformations of floor slabs, and, in low-to medium-rise structures, the axial deformations of columns [1].

2.3.2.6 Cracking:

The effects of cracking in reinforced concrete members due to flexural tensile stresses are assumed represent able by a reduced moment of inertia. The gross inertias of beams are usually reduced to 50% of their uncracked values, while the gross inertias of columns are reduced to 80% [1].

2.3.2.7 Magnified Moments:

The factored axial forces, P_u, the factored moments, *M1* and *M2*, at the ends of the column and, where required, the relative lateral story deflections, Δ_0, shall be computed using an elastic first-order frame analysis with the section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member and effects of duration of loads [10].

2-4 High-Rise Behavior:

A reasonably accurate assessment of a proposed high-rise structure's behavior is necessary to form a properly representative model for analysis. A high-rise structure is essentially a vertical cantilever that is subjected to axial loading by gravity and to transverse loading by wind or earthquake. Gravity live loading acts on the slabs, which transfer it horizontally to the vertical walls and columns through which it passes to the foundation. The magnitude of axial loading in the vertical components is estimated from the slab tributary areas, and its calculation is not usually considered to be a difficult problem. The recognition of the structure's behavior under horizontal loading and the formation of the corresponding model are usually the dominant problems of analysis. The principal criterion of a satisfactory model is that under horizontal loading it should deflect similarly to the prototype structure. The resistance of the structure to the external moment is provided by flexure of the vertical components, and by their axial action acting as the chords of a vertical truss. The described flexural and axial actions of the vertical components and shear action of the connecting members are interrelated, and their relative contributions define the fundamental characteristics of the structure. It is necessary in forming a model to assess the nature and degree of the vertical shear stiffness between the vertical components so that the resulting flexural and axially generated resisting moments will be apportioned properly. The horizontal shear at any level in a high-rise structure is resisted by shear in the vertical members and by the horizontal component of the force in any diagonal bracing at that level. If the model has been properly formed with respect to its moment resistance, the external shear will automatically be properly apportioned between the components. Torsion on a building is resisted mainly by shear in the vertical components, by the horizontal component of axial force in any diagonal bracing members, and by the shear and warping torque resistance of elevator, stair, and service shafts. If the individual bents, and vertical components with assigned torque constants, are correctly simulated and located in the model, and their horizontal shear connections are correctly modeled, their contribution to the torsion resistance of the structure will be correctly represented also. A structure's resistance to bending and torsion can be significantly influenced also by the vertical shearing action between connected orthogonal bents or walls. It is important therefore that this is properly included in the model by ensuring the vertical connections between orthogonal components. The preceding discussion of a high-rise structure's behavior has emphasized the importance of the role of the vertical shear interaction between the main vertical components in developing the structure's lateral load resistance. Horizontal force interaction occurs when a horizontal deflected system of vertical components with dissimilar lateral deflection characteristics, for example, a wall and a frame, is connected horizontally. In constraining the different vertical components to deflect similarly, the connecting links or slabs are subjected to horizontal interactive forces that redistribute the horizontal loading between the vertical components. In constraining the different vertical components to displace about a center of rotation and to twist identically at each level, the connecting slabs are subjected to horizontal forces that redistribute the torque between the vertical components and increase the torque resistance of the structure [1].

2.5 Approximate Analysis by Cantilever Method:

The cantilever method is based on the concept that a tall rigid frame subjected to horizontal loading deflects as a flexural cantilever. The validity of this concept increases for taller, more slender frames, and for frames with higher girder stiffness. This method is suitable for the analysis of structures of up to 35 stories high with height to width ratios of up to 5:1.

The assumptions for the cantilever method are as follows [1]:

1. horizontal loading on the frame causes double curvature bending of all the columns and girders with points of contra-flexure at the mid-heights of columns and mid-span of girders.

2. The axial stress in a column is proportional to its distance from the centre of the column areas.

2.6 Approximate Analysis by Moments Distribution:

This method is used for estimating girder moment in a continues sub frames spans, it is the more accurate than the formulas (Three Moment Equations and Slope – Deflection) the following is assumed for the analysis:

1- A counterclockwise restraining moment on the end of the girder is positive and a clockwise moment is negative.

2- The end of the columns at the floors above and below the considered girder are fixed.

3- In the absence of known member sizes, distribution factor at each joint equal (1/ number of members in target joint) [1].

2.7 Approximate Analysis for Drift:

When the initial sizes of the frame members have been selected, an approximate check on the horizontal drift of the structure can be made. The drift of non-slender rigid frame is mainly caused by racking. The racking may be considered as comprising two components:

first is due to rotation of the joints, as allowed by the double bending of the girders, The second is caused by double bending of the columns [1].

* Components of Drift:

It is assumed for the drift analysis that points of contra-flexure occur in the frame at the mid-story level of the columns and at the mid-span of the girders. Hence, the component of the drift are, [1]:

1- Story Drift due to Girder Flexure.
2- Story Drift due to Column Flexure.
3- Story Drift due to Overall Bending.