Feroz Alam

Parallel Shear Walls (PSW) - An Innovative Concept on Megatall Buildings

@seismicisolation

Feroz Alam

Parallel Shear Walls (PSW) - An Innovative Concept on Megatall Buildings

Applied to One Kilometer Tall Concrete Skyscraper. Concept of Concrete Reduction from Shear Walls of Tall Buildings

Scholars' Press

Imprint
Any brand names and product names mentioned in this book are subject to trademark, brand or patent protection and are trademarks or registered trademarks of their respective holders. The use of brand names, product names, common names, trade names, product descriptions etc. even without a particular marking in this work is in no way to be construed to mean that such names may be regarded as unrestricted in respect of trademark and brand protection legislation and could thus be used by anyone.

Cover image: www.ingimage.com

Publisher:
Scholars' Press
is a trademark of
International Book Market Service Ltd., member of OmniScriptum Publishing Group
17 Meldrum Street, Beau Bassin 71504, Mauritius
Printed at: see last page
ISBN: 978-3-639-66041-8

@seismicisolation

Preface

There are numerous structural lateral systems used in high-rise building design such as shear frames, frames with shear core, framed tubes, tube and tube, super frames etc. Generally, the structural systems of tall buildings are considered to be two types. One is interior and the other one is exterior type. A system is categorized as an interior structure when the major parts of the lateral load resisting system are located within the interior of the building. Likewise, if the major parts of the lateral load resisting system are located at the building perimeter, the system is categorized as an exterior structure. This book will describe a new concept called "Parallel Shear Walls (PSW)" concept for Tall/Mega Tall Buildings, and this PSW concept's structural system can be applied up to One Kilometer Structure. This innovative concept is called "Parallel Shear Walls" because when wind will hit to a structure which is arranged based on this concept, several parallel grids containing shear walls will resist the wind forces. In PSW concept, several parallel shear walls have been arranged in both main directions of a building and connected with beams and R.C. floor slabs. The shear walls are continuous down to the base to which they are rigidly attached to form vertical cantilevers. Their high in plane stiffness and strength make them well suited for bracing buildings up to about 264 stories. In addition, all the vertical members are proportioned to resist gravity loads on equal stress basis to overcome the differential column shortening issues that are generally difficult to manage in super tall buildings. Fewer widely spaced gravity columns are arranged in the core area of the building to carry floor loads. Static and Dynamic analysis (Time History Analysis) has been carried out for 1003.2 meters tower. The drift for dynamic response is 1828 mm which is below the allowable limit of 2006 mm (If considered H/500, where H is the height of the structure[9]).

It is also found by research that, when this concept's structural arrangement is applied to around 830 meter tall structure with aspect ratio 9.8:1, no additional structural supporting system (like Outriggers, Perimeter Belts, Cross Bracing, Tuned Mass Dampers etc.) is required thus will save a large amount of money.

This shear walls arrangement is applicable for the tall buildings of any height to avoid additional supports to resist the lateral forces while taking advantage of the creative approach of this unique concept.

@seismicisolation

Table of Contents

@seismicisolation

@seismicisolation

LIST OF FIGURES

@seismicisolation

LIST OF TABLES

@seismicisolation

PART - 1

1. Introduction

In the last few decades there has been an enormous increase in the number of high-rise buildings worldwide. A skyscraper can also be called a high-rise, but the term skyscraper is often used for buildings higher than 50 m (164 ft). For buildings above a height of 300 m (984 ft), the term *Supertall* can be used, while skyscrapers reaching beyond 600 m (1,969 ft) are classified as *Megatall*.[15] A tall building is not defined by its height or number of stories.[7] The important criterion is whether or not the design is influenced by some aspect ''tallness''. It is a building in which tallness strongly influences planning, design, construction and use. It is a building whose height creates conditions different from those that exist in common buildings of a certain region and period.[8] There are physical, code prescribed and practical reasons why tall buildings tend to be safer than low-rise buildings.

Undoubtedly, the factor that governs the design of a tall and slender structure most of the times is not the fully stressed state, but the drift/acceleration of the building for wind loading. It is easy to understand that higher the building, the more import- ant is the lateral behavior. Thus, to understand the performance of high-rise buildings, the lateral resisting system of tall buildings becomes a key factor that needs to be investigated and understood.

[16]Structural systems for tall buildings have undergone a dramatic evolution throughout the previous decade and into the 1990s. Innovative structural systems involving mega frames, interior super diagonally braced frames, hybrid steel and high-strength concrete core and outrigger systems, artificially damped structures are among the compositions which represent a step in the development of structural systems for high-rise buildings. Structural systems for tall buildings have historically been grouped with respect to their ability to resist lateral loads effectively.

@seismicisolation

2. The Tall Building Structure

The structural system in a building supports the weight of the building and ensures the safety of the occupants, to maintain the integrity of the building, reinforces it against gravity and lateral loads. The building's structural system should provide:

- An appropriate architectural space:
- Strength to carry and resist all loads applied:
- Lateral stiffness to control drift due to wind and earthquake loads:
- Durability and energy absorbing or dissipating capability to withstand earth quakes:
- Appropriate dynamic characteristics or supplement with a damping system to limit motion to an acceptable level:
- Overall stability against overturning or against any load condition that renders the structures or some parts of it unstable: and
- A satisfactory solution to all serviceability requirements dictated by design criteria.[17]

The most efficient high-rise systems fully engage vertical gravity load resisting elements in the lateral load subsystem in order to reduce the overall structural premium for resisting lateral loads.[16]

In case of very tall buildings where the ratio of the vertical height to the base is very high or where the tall building is subjected to unusually high wind or seismic forces, tuned liquid or mechanical damping systems that control acceleration may be required.[17]

Modern building practices regarding supertall structures have led to the study of "vanity height". Vanity height, according to the CTBUH, is the distance between the highest floor and its architectural top (excluding antennae, flagpole or other functional extensions.[15]

[15]From onset of the design process, the structural design of the tower was formulated based on the objectives of integrating the structural and architectural design concept and included the following structural strategy:

1. Select and optimize the tower structural system for strength, stiffness, cost effectiveness, redundancy, and speed of construction.

@seismicisolation

2. Utilize the latest technological advances in structural materials that is available in the local market, and with due consideration to the availability of local skilled labor and construction method.
3. Manage and locate the gravity load resisting system so as to maximize its use in resisting the lateral loads while harmonizing with the architectural planning of luxury residential and hotel tower (original concept of the tower was mostly for residential use).
4. Incorporate the latest innovations in analysis, design, materials, and construction methods.
5. Limit the building Movement (drift, acceleration, torsional velocity, etc.) to within the international accepted design criteria and standards.
6. Control the relative displacement between the vertical members.
7. Control the dynamic response of the tower under wind loading by tuning the structural characteristics of the building to improve its dynamic behavior and to prevent lock-in vibration due to the vortex shedding. Favorable dynamic behavior of the tower was achieved by:

a. Varying the building shape along the height while continuing, without interruption, the building gravity and lateral load resisting system;
b. Reducing the floor plan along the height, thus effectively tapering the building profile.

Buildings can be divided into two categories: rigid and flexible. Buildings having a frequency of oscillation more than 1Hz (below one time per second) are rigid. When it is smaller than 1Hz (more than one time per second) they are called flexible.[17]

3. CTBUH height criteria

3.1. What is a Tall Building?

There is no absolute definition of what constitutes a "tall building." It is a building that exhibits some element of "tallness" in one or more of the following categories:

@seismicisolation

3.2. Tall Building Technologies

If a building contains technologies which may be attributed as being a product of "tall" (e.g., specific vertical transport technologies, structural wind bracing as a product of height, etc.), then this building can be classed as a tall building.

Although number of floors is a poor indicator of defining a tall building due to the changing floor to floor height between differing buildings and functions (e.g., office versus residential usage), a building of perhaps 14 or more stories or more than 50 meters (165 feet) in height could perhaps be used as a threshold for considering it a "tall building."

3.3. What are Supertall and Megatall Buildings?

The CTBUH defines "supertall" as a building over 300 meters (984 feet) in height, and a "megatall" as a building over 600 meters (1,968 feet) in height. As of June 2015 there were 91 supertall and 2 megatall buildings fully completed and occupied globally.

3.4. How is the Height of a Tall Building Measured?

The Council on Tall Buildings and Urban Habitat (CTBUH) recognizes tall building height in three categories:

3.5. Height to Architectural Top

Height is measured from the level of the lowest, significant, open-air, pedestrian entrance to the architectural top of the building, including spires, but not including antennae, signage, flag poles or other functional- technical equipment. This measurement is the most widely utilized and is employed to define the CTBUH rankings of the " World's Tallest Buildings".

3.6. Highest occupied floor

Height is measured from the level of the lowest, significant, open-air, pedestrian entrance to the finished floor level of the highest occupied floor within the building.

@seismicisolation

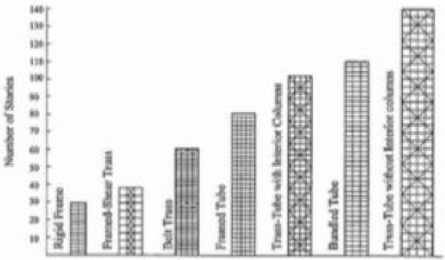
3.7. Height to Tip

Height is measured from the level of the lowest, significant, open-air, pedestrian entrance to the highest point of the building, irrespective of material or function of the highest element (i.e., including antennae, flagpoles, signage, and other functional-technical equipment).

4. Classification of Tall Building structural systems

[18]Steel was the material of choice in the beginning of the 20th century concrete was evolving to become a viable candidate because of its cheaper construction cost, better fire resistance and better mass dampening. In 1903 the first reinforced concrete high-rise building was built as the Ingalls Building in Cincinnati, Ohio, USA. Concrete was not often used as part of the structural system in high-rise buildings because of its weakness in tension along with non-developed calculations for reinforcement. Later developed structural systems like the outrigger and the buttress core has allowed for even higher buildings such as the Petronas Towers, Taipei101 and current tallest on earth; Burj Khalifa. In 1969, Fazlur Rahman Khan classified structural systems for tall buildings relating to their height with considerations for efficiency in the form of 'Height for Structural System diagrams, Fig. 1 for Steel Structures & Fig. 2 for Concrete Structures. He developed these schemes for both steel and concrete. This marked the beginning of a new era of skyscraper revolution in terms of multiple structural systems. Feasible structural systems, according to him, are rigid frames, frame shear trusses, belt trusses.

@seismicisolation



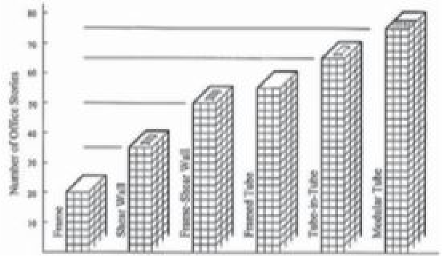


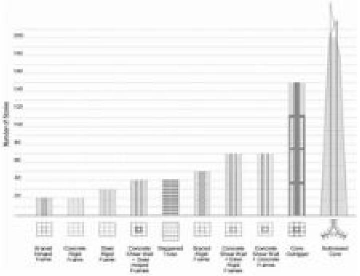
Figure 1. Framed tubes, Truss-Tube with interior columns, Bundled Tubes and Truss-Tube without interior columns for Steel Structures

Figure 2. Framed tubes, Truss-Tube with interior columns, Bundled Tubes and Truss-Tube without interior columns for Concrete Structures

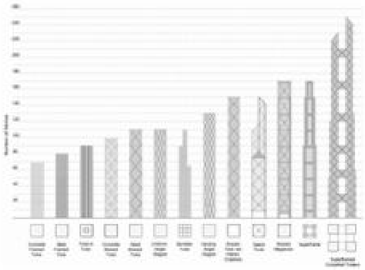
These structural systems can reach up to about 140 stories. It is imperative that each system has a wide range of height application depending upon other design and service criteria related to building shape, aspect ratio, architectural function, exterior load conditions, building stability, site constraints etc.

The systems charts shown in Figs. 1 & 2 were based on intensive research aided by computer simulations. It is the greatest single milestone in the development and evolution of tall building's structural design technology.[17]

Unlike Khan's charts where steel and concrete structures are separately presented, these new charts include both steel and concrete, and composite structures within the context of exterior and interior structures, see Fig. 3 (a) & (b).



(a). Interior Structures



(b). Exterior Structures

Figure 3. Classification of Tall Building Structural Systems

Framed tubes, Truss- Tube with Interior Columns, Bundle Tubes and Truss-Tube without Interior columns. These structural systems can reach up to about 140 stories. It is imperative that each system has a wide range of height applications depending upon other design and service criteria related to building shape, aspect ratio, architectural function, exterior load conditions, building stability, site constraints, etc.

5. Design Criteria

The design of tall buildings essentially involves a conceptual design, approximate analysis, preliminary design and optimization, to safely carry gravity and lateral loads. The design criteria are strength, serviceability, stability and human comfort. The strength is satisfied by limit stresses, while serviceability is satisfied by drift limits in the range of H/500 to H/1000. Stability is satisfied by sufficient factor of safety against buckling and P-Delta effects. The factor of safety is around 1.67 to 1.92. The human comfort aspects are satisfied by accelerations in the range of 10 to 25 milli-g, where g = acceleration due to gravity, about 981cms/sec^2. The aim of the structural engineer is to arrive at suitable structural schemes, to satisfy these criteria, and assess their structural weights in weight/unit area in square feet or square meters. This initiates structural drawings and specifications to enable construction engineers to proceed with fabrication and erection operations. The weight of steel in lbs/sqft or in kg/sqm is often a parameter the architects and construction managers are looking for from the structural engineer. This includes the weights of floor system, girders, braces and columns. The premium for wind, is optimized to yield drifts in the range of H/500, where H is the height of the building.[19]

6. Creep-Shrinkage-Temperature

[1][1] Creep and shrinkage of concrete have a pervasive cause of excessive deflections and damages in concrete structures. In a tall building, the time- dependent differential vertical shortening between adjacent vertical members due to creep and shrinkage of concrete may be sufficiently large to cause distress in nonstructural elements and to induce significantly structural actions in the horizontal elements (especially in the upper region of the building). These differential vertical shortenings arises as adjacent vertical members have different characteristics such as percentage of

[1] [1] Effect of relative humidity on creep-shrinkage behavior of composite tall buildings - Lat. Am. j. solids struct. vol.10 no.3 Rio de Janeiro May 2013 - Peeyush Chowdhary & Ravi K. Sharma

reinforcement, volume to surface ratio and stress level caused due to different gravity loadings or from non-uniform stresses caused by lateral force. These differential shortenings are of cumulative nature along the height and therefore, have importance with increasing height of the buildings.

[2]2 In recent years a large number of multi-story apartment and office buildings have been built in reinforced concrete. While in the low rise buildings the effects of temperature creep and shrinkage in the columns did not substantially control the stress or design of the structure, these otherwise secondary effects may become primary and must be considered in the analysis, design and detailing of the high-rise structure. The effects of temperature, creep and shrinkage in high-rise buildings are not only structural, but also architectural in that the exterior window wall details as well as the interior partition details must be designed to incorporate relative movements caused by these factors. A brief discussion of the philosophy for planning and design procedures of high-rise buildings subjected to these effects follows:

6.1. Temperature Effects

Exposed columns when subjected to seasonal temperature variations change their length relative to the interior columns which remain unchanged in a controlled environment. Furthermore, if the exterior columns have difference in size and are subjected to different average temperature due to the location of glass lines, there will be relative displacement between these adjacent columns when exposed to seasonal changes. The philosophy of design of structure with exposed columns involves one of the two basic concepts:

(a) To use an effective method of analysis and design and to develop details to accommodate large expected relative movements, or

(b) To plan a building for a controlled temperature movement.

6.2. Effects of Creep and Shrinkage

With increasing height of buildings, the importance of time dependent shortening of columns and shear walls becomes more critical due to the cumulative nature of such

2 [2] IABSE congress report – 1968 – Effects of Column temperature, creep and shrinkage in tall structures

@seismicisolation

shortening. It is known that columns with varying percentage of reinforcement and varying volume-to-surface ratio will have different creep and shrinkage strains. Increasing the percentage of reinforcement and the volume-to- surface ratio reduces strains due to creep and shrinkage. In very tall structures where a large heavy reinforced column may be adjacent to a lightly reinforcing shear wall a differential inelastic shortening causes moments in the horizontal members and also a load redistribution from the shear wall to the column which has less creep and shrinkage.

Although a large amount of research information is available on shrinkage and creep, it is not directly applicable to column of high-rise buildings but are applicable to flexural elements only. In the construction of a high-rise buildings, columns are loaded in as many increments as there are stories above the level under consideration. Such incremental loading over a long period of time makes a considerable difference in the magnitude of creep and consequently in the differential movement and load redistribution between adjacent columns.

7. Foundation settlement and soil structure interaction

[1]3 The gravity and lateral forces on the building will be transmitted to the earth through the foundation system, and, as the principles of foundation design are not affected by the quality of tallness of the superstructure, conventional approaches will suffice. The concern of the structural designer is then with the influence of any foundation deformation on the building's structural behavior and on the soil-structure interactive forces.

Because of its height, the load transmitted by the columns in a tall building can be very heavy. Where the underlying soil is rock or other strong stable subgrade, foundations may be carried down to the stiff load-bearing layers by use of piles, caissons, or deep basements. Problems are not generally encountered with such conditions since large variations in column loadings and spacings can be accommodated with negligible deferential settlement. In areas in which soil conditions are poor, loading on foundation elements must be limited to prevent shearing failures of excessive differential settlements. Relief may be obtained by excavating a weight of soil equal to a significant

[3] [1] Bryan Stafford Smith & Alex Coull – Tall Building Structure (1991)

@seismicisolation

portion of the gross building weight. Because of the high short-term transient moments and shears that arise from wind loads, particular attention must be given to the design of the foundation system for resisting moments and shears, especially if the precompression due to the dead weight of the building is not sufficient to overcome the highest tensile stresses caused by wind moments, leading to uplift on the foundation.

The major influences of the foundation deformations are twofold. First, if the bases of vertical elements yield, a stress redistribution will occur, and the extra loads imposed on other elements may then further increase the deformation there. The influence of the relative displacements on the forces in the horizontal elements must then be assessed. Second, if an overall rotational settlement θ of the entire foundation occurs, the ensuing lateral deflections will be magnified by the height H to give a top deflection of Hθ. As well as increasing the maximum drift, the movement will have a destabilising effect on the structure as a whole, by increasing any P-Delta effects that occur.

Soil-structure interaction involves both static and dynamic behavior. The former is generally treated by simplified models of sub grade behavior, and finite element methods of analysis are usual. When considering dynamic effects, both interactions between soil and structure, and any amplification caused by a coincidence of the natural frequencies of building and foundation, must be included. Severe permanent structural damage may be caused by earthquakes when large deformations occur due to the soil being compacted by the ground vibration, which under certain conditions may result in the development of excess hydrostatic pressures sufficient to produce liquefaction of the soil. These types of soil instability may be prevented or reduced in intensity by appropriate soil investigation and foundation design. On the other hand, the dynamic response of buildings to ground vibrations, which is also affected by soil conditions, cannot be avoided and must be considered in design.

[2][4]Spectacular failure of structures has been observed in every major seismic event. Gujarat earthquake of 26 January 2001 have demonstrated that the strength alone would not be sufficient for the safety of structures during the earthquake. In conventional design, buildings are generally considered to be fixed at their bases. In reality, flexibility of the supporting soil medium allows some movement of the

[4] [2] Jagadish ponraj Nadar, Hement S. Chore, P. A. Dode, "Soil Structure Interaction of Tall Buildings" International Journal of Computer Applications (0975 – 8887) International Conference on Quality Up-gradation in Engineering, Science and Technology (ICQUEST2015) - P. A. Dode

@seismicisolation

foundation. However, if the structure is very massive and stiff, such as high-rise buildings, and the foundation is relatively soft, the motion at the base of the structure may be significantly different than the free-field surface motion. A foundation is interface between superstructure with underlying soil or rock. In seismic environment, the loads imposed on a foundation from a structure under seismic excitation can greatly exceed the static vertical loads or even produce uplift; in addition, there will be horizontal forces and possibly moments at the foundation level. Structural engineering and geotechnical engineering are frequently isolated from one another as if they were two independent disciplines. In reality, however, a structure and the soil on which it is founded are one system, and their interplay must be considered in order to achieve reasonably accurate soil settlement prediction. Recent trend of construction is that numbers of buildings are constructed on island where founding base of building created by artificial created inland. As a result of this soft layers of soil the earthquake ground motion get modified & high rise buildings have relatively longer predominant time period. Due to this soil structure interaction, response of structure get significantly modified and detailed studies needs to be done while design such a buildings.

8. Load Effects

Serviceability and safety are directly related to the loads that are applied on tall buildings. Limit state is a condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).[17]

Load Combinations: Calculations assuring structural serviceability and safety for tall buildings should follow minimum load requirements issued by the American Society of Civil Engineers (ASCE-7-05, 2005) in the U.S. or other applicable codes in the U.S. and other countries. Loads and appropriate load combinations, which have been developed to be used together, are set forth for strength design and allowable stress design. [17]

Load combinations and load factors shown here as an example shall be used only in those cases in which they are specifically authorized by the applicable material design standard. Structures, components, and foundations shall be designed so that their design

@seismicisolation

strength equals or exceeds the effects of the factored loads in the following combinations (ASCE 7-05, 2005):

1.2 D

1.2 D + 1.6 (L+H) + 0.5 (Lr, S or R)

1.2 D + 1.6 (Lr, S or R) + (0.5 L or 0.8 W)

1.2 D + 1.6 W + 0.5 l + 0.5 (Lr, S or R)

0.9 D + 1.6 W + 1.6 H

0.9 D + 1.6 W +1.6 H

Where D = dead load; E = earthquake load; H = load due to lateral earth pressure; L = live load; Lr = roof live load; S = snow load; W = wind load. Loads listed below are considered for allowable stress design to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

D

D + W + L + (Lr, Lr or R)

0.6 D + W + H

0.6 D + 0.7 E + H

There are other load combinations that are not shown here they are not generally relevant to tall buildings. The lateral earth pressure is included here because tall buildings have often multiple levels of parking garage and retain soils from the outside and therefore the foundation below ground level is subjected to such pressure. [17]

Load combination for lateral deflection (Drift)

1.0D +0.5L+1.0W (ASCE 7-10)

9. Wind Loading

The wind loading on a skyscraper is considerable. In fact, the lateral wind load imposed on super-tall structures is generally the governing factor in the structural design.

@seismicisolation

Wind pressure increases with height, so for very tall buildings, the loads associated with wind are larger than dead or live loads.

Structural design of most tall buildings is controlled by wind effect. The movement of wind in the atmosphere is truly three-dimensional. However, the horizontal motion is the most dominant and is usually applied on buildings. Its flow is quite complex and turbulent in nature. It can fluctuate in a random manner. Therefore, statistical approaches are adopted in qualifying wind pressure.

Even though wind effects are truly dynamic in nature, they are evaluated as equivalent static effects. In other cases when the aerodynamic effects of wind are predominant, wind forces need to be calculated for such effects. For a tall slender building, it may be necessary to perform dynamic analysis of the structure to find its strength and deformation characteristics. [17]

The detailed procedure described in wind codes is sub-divided into Static Analysis and Dynamic Analysis methods. The static approach is based on a quasi-steady assumption, and assumes that the building is a fixed rigid body in the wind. The static method is not appropriate for tall structures of exceptional height, slenderness, or susceptibility to vibration in the wind. In practice, static analysis is normally appropriate for structures up to 50 meters in height. The subsequently described dynamic method is for exceptionally tall, slender, or vibration-prone buildings. The Codes not only provide some detailed design guidance with respect to dynamic response, but state specifically that a dynamic analysis must be undertaken to deter- mine overall forces on any structure with both a height (or length) to breadth ratio greater than five, and a first mode frequency less than 1 Hertz. [20]

[21]Wind force induces vibration of the building and applies bending, shear and torsion on the different structural components of the buildings. The vibration of a tall building when exposed to wind is studied in this section with respect to the following three issues:

1) Drift of the building under wind load;

2) Period of vibration of the building; and

3) Acceleration of vibration of the building.

@seismicisolation

10. Building Drift due to Wind Loading

[1]The provision of adequate stiffness, particularly lateral stiffness, is a major consideration in the design of a tall building for several important reasons. As far as the ultimate limit state is concerned, lateral deflections must be limited to prevent second order P-Delta effects due to gravity loading being of such a magnitude as to precipitate collapse. In terms of the serviceability limit state, deflections must first be maintained at a sufficiently low level to allow the proper functioning of non structure components such as elevators and doors; second, to avoid distress in the structure, to prevent excessive cracking and consequent loss of stiffness, and to avoid any redistribution of load to on-load-bearing partitions, infills, cladding, or glazing; and third, the structure must be sufficiently stiff to prevent dynamic motions becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken, or affect sensitive equipment. In fact, it is in the particular need for concern for the provision of lateral stiffness that the design of a high-rise building largely departs from that of a low-rise building.

One simple parameter that affords an estimate of the lateral stiffness of a building is the drift index, defined as the ratio of the maximum deflection at the top of the building to the total height. In addition, the corresponding value for a single story height, the interstory drift index, gives a measure of possible localized excessive deformation. The control of lateral deflections is of particular importance for modern buildings in which the traditional reserves of stiffness due to heavy internal partitions and outer cladding have largely disappeared. It must be stressed, however, that even if the drift index is kept within traditionally accepted limits, such as 1/500, it doesn't necessarily follow that the dynamic comfort criteria will also be satisfactory. Problems may arise, for example, if there is coupling between bending and torsional oscillations that leads to unacceptable complex motions or accelerations. In addition to static deflection calculations, the question of the dynamic response, involving the lateral acceleration, amplitude, and period of oscillation, may also have to be considered.

The establishment of a drift index limit is a major design decision, but, unfortunately, there are no unambiguous or widely accepted values, or even, in some of the National Codes concerned, any firm guidance. The designer is then faced with having to decide on an appropriate value. The figure adopted will reflect the building usage, the type of design criterion employed (for example, working or ultimate load conditions),

@seismicisolation

the form of construction, the materials employed, including any substantial infills or cladding, the wind loads considered, and, in particular, past experience of similar buildings that have performed satisfactorily.

Design drift index limits that have been used in different countries range from 0.001 to 0.005. To put this in perspective, a maximum horizontal top deflection of between 0.1 and 0.5 m (6 to 20 in.) would be allowed in a 33-story, 100-(330-ft.) high building, or, alternatively, a relative deflection of 3 to 15 mm (0.12 to 0.6 in.) over a story height of 3 m (10ft). Generally, lower values should be used for hotels or apartment buildings than for office buildings, since noise and movement tend to be more disturbing in the former. Consideration may be given to whether the sufficient effects of any internal partitions, infills, or cladding are included in the deflection calculations.

The consideration of this limit state requires an accurate estimate of the lateral deflections that occur, and involves an assessment of the stiffness of cracked members, the effects of shrinkage and creep and any redistribution of forces that may result, and of any rotational foundation movement. In the design process, the stiffness of joints, particularly in precast or prefabricated structures, must be given special attention to develop adequate lateral stiffness of the structure and to prevent any possible progressive failure. The possibility of torsional deformations must not be overlooked.

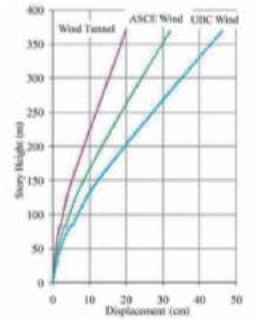
In practice, non-load-bearing infills, partitions, external wall panels, and window glazing should be designed with sufficient clearance or with flexible supports to accommodate the calculated movements.

Sound engineering judgment is required when deciding on the drift index limit to be imposed. However, for conventional structures, the preferred acceptable range is 0.0015 to 0.003 (that is, approximately 1/650 to 1/350), and sufficient stiffness must be provided to ensure that the top deflection does not exceed this value under extreme load conditions. As the height of the building increases, drift index coefficients should be decreased to the lower end of the range to keep the top story deflection to a suitably low level.

The drift criteria apply essentially to quasistatic conditions. When extreme force conditions are possible, or where problems involving vortex shedding or other unusual phenomena may occur, a more sophisticated approach involving a dynamic analysis may be required.

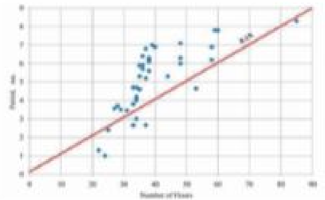
If, excessive, the drift of a structure can be reduced by changing the geometric configuration to alter the mode of lateral load resistance, increasing the bending stiffness of the horizontal members, adding additional stiffness by the inclusion of stiffer wall or core members, achieving stiffer connections, and even by sloping the exterior columns. In extreme circumstances, it may be necessary to add dampers, which may be of the passive or active type.[1]

As per different codes and standards an average drift should be limited to 1/500 of the building height to control cracking and distortion of the non-structural elements of the building. Drift can be controlled by either increasing the stiffness of the vertical and horizontal elements or by adopting stiffer elements such as walls or core walls. An accurate assessment of wind-induced load on the building is essential to attain a practical and economical design.[21] Fig. 4 shows the drift of one of the designed buildings under wind loads as calculated by either an extended UBC method or the ASCE method. It also shows the drift due to wind loads obtained from wind tunnel test, such as that shown in Fig. 5. It is clear that for economical reasons wind tunnel testing techniques should be used.



[21]**Figure 4. Drift calculated using different methods**

@seismicisolation



[21]Figure 5 shows the increase of the fundamental period of vibration of a designed buildings with their height as an example

11. Acceleration Limits

[21]Human perception of building vibration is related to both amplitude (i.e. drift) and frequency of the building vibration.[22] At drift ratios of 1/400 and 1/500 neither building conforms to acceptable standards for acceleration limits. The reason that drift ratios by themselves do not adequately control motion perception is because they only address stiffness and do not recognize the important contribution of mass and damping, which together with stiffness, are the predominant parameters affecting acceleration in tall buildings.[22] The tall building designers should pay attention in evaluating the acceleration of their designed towers especially at top floors to ensure the comfort of the human occupants and the reliable performance of equipment in these buildings. The effects of acceleration on human comfort is given in Table-1.

Degree of discomfort	Acceleration Limits
Imperceptible	< 5 milli-g
Perceptible	< 5 milli-g - 15 milli-g
Annoying	< 15 milli-g - 50 milli-g
Very Annoying	< 50 milli-g - 150 milli-g
Intolerable	< 150 milli-g

[22]Table 1. Acceleration Limits for Different Perception Levels

@seismicisolation

The acceptable acceleration values for designed towers are 10 – 30 milli-g as specified in NBCC 1990.[21]

For dynamically sensitive supertall buildings or those with irregular geometry, wind tunnel model testing provides a more precise simulation of wind forces, and is an invaluable tool in their structural design and optimization. Wind loads acting on tall buildings can be determined using the high frequency force balance technique, or in the case of flexible structures that are dynamically sensitive, the aero- elastic techniques, details of wind tunnel test can be found in the literature.[17]

12. Natural frequency

[1]Natural frequency, f_0, is the number of oscillations per second of a structure that may swing freely. An oscillating structure has a tendency to develop greater amplitude of a swing at the natural frequency than at other frequencies. At this frequency, even small periodic driving forces produce large amplitude swings, because the system stores vibrational energy, resonance is created. A structure has an unlimited number of natural frequencies, only a few are essential though.

Calculation of natural frequency of a building is very demanding. Knowledge of the fundamental natural frequency of a building is necessary to determine the design loading and response to either turbulent wind or earthquake actions.

Many studies reported in the literature have attempted to derive empirical formulas to allow an estimation of the fundamental natural frequency, or period, of a building, based only on simple geometric parameters.

The easiest method (Harris & Crede, 1976, Eq.1) is to consider the whole building as a cantilever, end fixed in the ground. The natural frequency is primarily dependent on the building's equivalent stiffness and mass. The variation between the floors cannot be done, which is also the main drawback of the method. Therefore the natural frequency can be calculated only for three types of modes, bending in two directions and torsion.

$$f_0 = \frac{1}{2\pi}\sqrt{\frac{3EI}{0.23mL^4}} \qquad \text{Eq. - 1}$$

@seismicisolation

E – elasticity modulus

I – moment of inertia

m – mass per unit length

L – length of the cantilever

[2]Two widely used formulas for the fundamental frequency no are

$$n_0 = \frac{\sqrt{D}}{0.091H}(Hz) \qquad \text{Eq. – 2}$$

where D is the base direction (in meter) in the direction of motion considered, and H is the height of the building (in meter).

It has been suggested[3] that this formula is particularly applicable to reinforced concrete shear wall buildings and braced frames.

$$n_0 = \frac{10}{N}(Hz) \qquad \text{Eq. - 3}$$

It has been suggested[3] that this formula should be used when the lateral force-resisting system consists of a moment-resisting space frame that resist the entire lateral forces, and the frame is not enclosed or adjoined by more rigid elements that would tend to prevent the frame from resisting lateral forces.

Another formula that has also been commonly used is,

$$n_0 = \frac{1}{C_T H^{3/4}} \qquad \text{Eq. - 4}$$

where C_T is equal to 0.035 or 0.025 for steel or concrete structures, respectively, and H is the building height (in feet). This formula is also most appropriate when moment-resisting frames are the sole lateral-load-resisting elements in the building [3].

The first two formulas were tested by Ellis[4] against the measured natural frequencies of 17 buildings, ranging in height from 7 to 44 stories. He found that errors greater than + 50% were not abnormal, but that the simpler formula [Eq. 2] generally gave more accurate results.

@seismicisolation

He tried different simple predictor formulas and compared the results with the actual measured frequencies of 163 buildings of rectangular plan-form. As a result of his study, the following formula was recommended:

$$n_0 = \frac{46}{H} \qquad \text{Eq. – 5}$$

where H is the building height (in meter).

It may be noted that he also suggested that the frequency of the first orthogonal translational mode can be estimated as 58/H, and the frequency of the first torsional mode as 72/H. Care must also be taken when using these formulas, particularly the torsional mode predictor.

Once a preliminary design has been achieved and the stiffness of the building is known, a more accurate estimate of the fundamental natural frequency may be determined from established procedures. A reasonably accurate approximate formula based on Rayleigh's method[3] is

$$n_0 = \frac{1}{2\pi}\left(\frac{g\sum F_i u_i}{\sum W_i u_i^2}\right)^{1/2} \qquad \text{Eq. - 6}$$

in which W_i is the weight of the ith floor, u_i is the calculated static horizontal deflection at the i^{th} level due to a set equivalent lateral loads F_i at the floor levels, and g is the gravitational acceleration. Any reasonable distribution of loads F_i may be selected, but it is convenient to use the statically equivalent forces due to wind or earthquake actions.

12.1. ASCE: Frequency Determination

[3]Rigid Building: A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

Flexible Building: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz.

To determine whether a building or structure is rigid or flexible, the fundamental natural frequency, n_1, shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis.

12.2. Limitations for Approximate Natural Frequency

As an alternative to performing an analysis to determine n_1, the approximate building natural frequency n_a, shall be permitted to be calculated in accordance with Eqs. 7, 8 & 9 for structural steel & concrete or masonry buildings meeting the following requirements:

1. The building height is less than or equal to 300 ft (91 m), and
2. The building height is less than 4 times its effective length, L_{eff}.

The effective length, L_{eff}, in the direction under consideration shall be determined from the following equation:

$$L_{eff} = A = \pi r^2 \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} \qquad \text{Eq. - 7}$$

The summations are over the height of the building where

h_i is the height above grade of level i

L_i is the building length at level i parallel to the wind direction.

Approximate Natural Frequency:

The approximate lower-bound natural frequency (n_a), in Hertz, of concrete or structural steel buildings meeting the conditions mentioned above, is permitted to be determined from one of the following equations:

For structural steel moment-resisting-frame buildings:

$$n_a = {}^{22}\!/_{h^{0.8}} \qquad \text{Eq. - 8}$$

For concrete moment-resisting frame buildings:

$$n_a = {}^{43.5}\!/_{h^{0.9}} \qquad \text{Eq. - 9}$$

@seismicisolation

For structural steel and concrete buildings with other lateral-force-resisting systems:

$n_a = {}^{75}/_{h}$ Eq. - 10

Observation from wind tunnel testing of buildings where frequency is calculated using analysis software reveals the following expression for frequency, appropriate for buildings less than about 400 ft in height, applicable to all buildings in steel or concrete:

$n_1 = 100/H$ (ft) average value Eq. - 11

$n_1 = 75/H$ (ft) lower bound value Eq. - 12

.

Based on full-scale measurements of buildings under the action of wind, the following expression has been proposed for wind applications (Zhou and Kareem 2001, Zhou, Kijewski, and Kareem 2002):

$f_{n1} = {}^{150}/_{H}$ (ft) Eq. - 13

This frequency expression is based on older buildings and overestimates the frequency common in U.S. construction for smaller buildings less than 400 ft in height, but becomes more accurate for tall buildings greater than 400 ft in height. The Australian and New Zealand Standard AS/NZS 1170.2, Eurocode ENV1991-2-4, Hong Kong Code of Practice on Wind Effects (2004), and others have adopted Eq. 13 for all building types and all heights.

Recent studies in Japan involving a suite of buildings under low-amplitude excitations have led to the following expressions for natural frequencies of buildings (Sataka et al. 2003):

$n_1 = {}^{220}/_{H}$ (ft) (concrete buildings) Eq. - 14

$n_1 = {}^{164}/_{H}$ (ft) (steel buildings) Eq. - 15

@seismicisolation

The expressions based on Japanese buildings result in higher frequency estimates than those obtained from the general expression given in Eqs. 11 through 13, particularly since the Japanese data set has limited observations for the more flexible buildings sensitive to wind effects and Japanese construction tends to be stiffer.

References:

(1) Evgenij Budajev, Christian Sandelin - The Stabilization of High-rise Buildings - An Evaluation of the Tubed Mega Frame Concept (December 2013)

(2) Bryan Stafford Smith, Alex Coull – Tall Building Structures, Analysis and Design (p 449) – 1991

(3) ASCE - American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures - 2010

(4) Ellis, B.R. "An Assessment of the Accuracy of Predicting the Fundamental Natural Frequencies of Buildings and the Implications Concerning the Dynamic Analysis of Structures." Proc. Inst. Civil Engineer. London 69(2), 1980, 763-776.

Table-2 shows the STAAD output result of natural frequency of 1000.8 meters Tall Tower.

STAAD SPACE

CALCULATED FREQUENCIES

MODE	FREQUENCY(CYCLES/SEC)	PERIOD(SEC)	ACCURACY
1	0.053	18.92037	7.550E-16
2	0.058	17.27685	2.099E-15
3	0.093	10.70647	8.062E-16
4	0.108	9.25973	6.028E-15
5	0.119	8.43127	9.996E-16
6	0.166	6.01472	6.104E-16
7	0.194	5.15969	3.743E-15
8	0.212	4.70682	3.661E-15
9	0.261	3.83515	1.158E-15
10	0.273	3.66675	6.250E-16
11	0.298	3.35098	4.800E-15
12	0.362	2.76213	7.209E-15
13	0.366	2.73142	4.031E-15
14	0.395	2.53425	9.681E-15
15	0.446	2.24313	6.822E-14
16	0.489	2.04409	5.136E-12
17	0.511	1.95575	5.094E-12
18	0.562	1.77829	1.789E-09
19	0.569	1.75847	1.483E-09
20	0.623	1.60482	9.189E-08

Table-2: Natural Frequency

@seismicisolation

13. Dynamic Analysis

A dynamic analysis is required only when the building is relatively flexible or, because of its shapes, structural arrangement, mass distribution, foundation condition, or use, is particularly is sensitive to wind or seismic accelerations. Sometimes it is tedious to do dynamic analysis of a tall building for wind loads. In static analysis, the peak dynamic forces and displacements may be determined by multiplying the values due to the mean wind loading by Gust Factor "G" from

Eq. 26.9-10 (ASCE 7-10).

[1]Skyscraper motions may be classified as static or dynamic. Static refers to the motions produced by slowly applied forces such as gravitational or the long period component of wind. Dynamic motions refer to those caused by time dependent dynamic forces, notably seismic accelerations, short period wind loads and machinery vibrations, the first two usually being of the greatest concern. Although the deformations causing from static forces may be of possible detriment to the integrity of the structure, unless they lead to the collapse to the building they are unlikely to provoke any reaction from the occupants.

Dynamic wind pressure produces sinusoidal or narrow-band random vibration motions of the building, which will generally oscillates in both along-wind and cross-wind directions, and possibly rotates about a vertical axis. The magnitudes of the three displacement components will depend on the velocity distribution and direction of the wind, and on the shape, mass, and stiffness properties of the structure. In certain cases, the effects of cross-wind motions of the structure may be greater than those due to the along-wind motions.

The ground shaking which occurs in an earthquake may be described as a series of virtually multi-directional random acceleration pulses. The ground movements will generally produce simultaneous translations along and rocking about the two orthogonal horizontal axes, as well as displacement along and torsion about the vertical axis of the structure.

When designing a tall building to resist seismic forces, the design loads may be determined from a dynamic analysis of the building's response to time-history base accelerations, based on an actual recorded local event, or an artificially generated time-

@seismicisolation

history. Such a time-consuming rigorous approach may be simplified by the use of earthquake response spectra, which, although requiring less computational effort, yield acceptably similar results for peak responses.

The seismic response of the building will depend on the dynamic properties of the structure, the ground motion at the foundation, and the mode of soil-structure interaction. The motion of a very stiff building will be almost identical to the ground motion, but that of a flexible structure will be quite different. The response will depend on the proximity of the natural frequencies of the structure to that of predominant ground-motion frequency, the damping inherent in the structure, the foundation behavior, the ductility of the structure, and the duration of the earthquake.

The nature and magnitude of both wind and earthquake loading on buildings are dynamic and transient in character, it has been shown that for design purposes they may be frequently replaces by equivalent static loads, which are chosen to represent their probable worst magnitudes. The equivalent static loads for wind effects will be based on a statistical knowledge of the likely occurrence and magnitude of wind velocities and pressures, and for earthquake on time-history of accelerations. For the majority of tall buildings, the quasistatic loadings are adequate for design purposes, have proved satisfactory in most situations. A dynamic analysis is required only when the building is relatively flexible or, because of its shapes, structural arrangement, mass distribution, foundation condition, or use, is particularly is sensitive to wind or seismic accelerations. Then consideration has to be given to both the stress levels that occur and accelerations that may affect the comfort of the occupants.

13.1. Dynamic Response to Wind Loading

A complete description of the wind loading process relies on a proper definition of the wind climate from meteorological records, together with an understanding of atmospheric boundary layers, turbulence properties and the variation of wind speed with height, the aerodynamic forces produced by the interaction of the building with the turbulence boundary layer, and the dynamic response of the structure to the wind forces.

@seismicisolation

13.2. Structure's sensitivity to Wind Forces

The principal structural characteristics that influence the decision to make a dynamic design analysis are the natural frequencies of the first few normal modes of vibration and the size of the building. When a building is small, the whole structure will be loaded by gusts so that the full range of frequencies from both boundary layer turbulence and building-generated turbulence will be encountered. On the other hand, when the building is relatively large or tall, the smaller gusts will not act simultaneously on all parts, and will tend to offset each other's effects, so that only the lower frequencies are significant.

When the structure is stiff, the first few natural frequencies will be relatively high, and there will be little energy in the spectrum of atmospheric turbulence available to excite resonance. The structure will thus tend to follow any fluctuating wind forces without appreciable amplification or attenuation. The dynamic deflections will not be significant, and the main design parameter to be considered is the maximum loading to which the structure will be subjected during its lifetime. Such a structure is termed "static", and it may be analyzed under the action of static equivalent wind forces.

When a structure is flexible, the first few natural frequencies will be relatively low, and the response will depend on the frequencies of the fluctuating wind forces. At frequencies below the first natural frequency, the structure will tend to follow closely the fluctuating force actions. The dynamic response will be attenuated at frequencies above the natural frequency, but will be amplified at frequencies at or near the natural frequency; consequently the dynamic deflections may be appreciably greater than the static values. The lateral deflection of the structure then becomes an important design parameter, and the structure is classified as ''dynamic.'' In such structures, the dynamic stresses must also be determined in the design process. Furthermore, the accelerations induced in dynamic structures may be important with regard to the comfort of the occupants of the building and must be considered.

When a structure is very flexible, its oscillations may interact with the aerodynamic forces to produce various kinds of instability, such as vortex- capture resonance, galloping oscillations, divergence, and flutter. In this exceptional case, the potential for disaster is so great that the design must be changed or the aerodynamic effects modified to ensure that this form of unstable behavior cannot occur.

@seismicisolation

It is thus important for the engineer to be able to determine in the early design stages if the structure is static or dynamic, particularly in view of the comfort criteria for the occupants. To rectify an unacceptable dynamic response after the structure has been built will, if at all possible, generally be difficult and very expensive.

Unfortunately, it is not yet possible, particularly in the early design stage, to assess accurately whether a dynamic analysis will be required, although several empirical guidelines are available in Design Codes. For example, the Australian Code [17.2] defines a dynamic building as one in which

1. the height exceeds five times the least plan dimension and
2. the natural frequency in the first mode of vibration is less than 1.0 Hz.

14. Mean Wind Speed

Wind velocity varies with height. The viscosity of air reduces its velocity adjacent to the earth's surface to almost zero, as shown in Fig. 6. A retarding effect occurs in the wind layers near the ground, and these inner layers in turn successively slow the outer layers. The slowing down is reduced at each layer as the height increases, and eventually becomes negligibly small. The height at which velocity ceases to increase is called the gradient height, and the corresponding velocity, the gradient velocity. This characteristic of variation of wind velocity with height is a well-understood phenomenon, as evidenced by higher design pressures specified at higher elevations in most building codes.

@seismicisolation

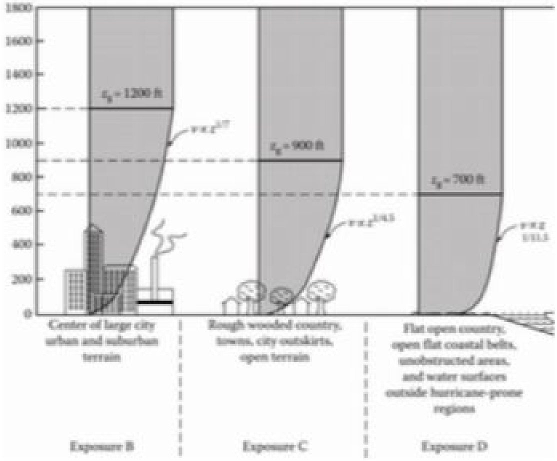


Figure 6. Wind velocity profiles as defined in the ASCE 7-05

At heights of approximately 1200 ft (366 m) above ground, the wind speed is virtually unaffected by surface friction, and its movement is solely dependent on prevailing seasonal and local wind effects. The height through which the wind speed is affected by topography is called the atmospheric boundary layer. The thickness of the boundary layer (gradient height) depends on the ground roughness.

15. Determination of Gust Effect Factor "G"

[2]The gust effect factor accounts for the loading effects in the along-wing direction due to wind turbulence-structure interaction. It also account for along-wind loading effects due to dynamic amplification for flexural buildings and structures. It does not include allowances for cross-wind loading effects, vortex sheddings, instability due to galloping or flutter, or dynamic torsional effects.

The formula for calculating G_f according to ASCE 7-10 is as follows:

(Equation numbers are followed to ASCE 7-10)

@seismicisolation

$$G_f = 0.925\left(\frac{1+1.7I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right) \quad \text{...... (Eq. 26.9-10)}$$

where g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\ \mathrm{In}(3600 n_1)} + \frac{0.577}{\sqrt{2\ \mathrm{In}(3600 n_1)}} \quad \text{...... (Eq. 26.9-11)}$$

and where R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad \text{...... (Eq. 26.9-12)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{\frac{5}{3}}} \quad \text{...... (Eq. 26.9-13)}$$

$$N_1 = \frac{n_1 L_z}{V_z} \quad \text{...... (Eq. 26.9-14)}$$

$$R_l = \frac{1}{-} - \frac{1}{2\ 2}(1 - e^{-2}\ ) \text{ for } \eta > 0 \quad \text{...... (Eq. 26.9-15a)}$$

$R_l = 1$ for $\eta = 0$ ...... (Eq. 26.9-15b)

Where the subscript l in Eqs. 26.9-15 shall be taken as h, B, and L, respectively,

where h = mean roof height.

B = Horizontal dimension of building measured normal to wind direction in feet.

L = Horizontal dimension of building measured parallel to wind direction in feet.

n_1 = the building natural frequency

$R_l = R_h$ setting η $4.6 n_1 h / V_z$

$R_l = R_B$ setting $\eta = 6.4 n_1 B / V_z$

$R_l = R_L$ setting $\eta = 15.4 n_1 L / V_z$

β is the damping ratio, percent of critical h, B, L are defined in Table 26.9-1 of AISC 7-10.

V_z is the mean hourly wind speed (ft/s) at height z determined from equation: $V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ ...... (Eq. 26.9-16)

where b and α are constants listed in Table 26.9-1 of V is the basic wind speed in miles per hour.

For Flexible, Tall and Mega Tall Buildings, Gust effect factor "G" is the main factor which can give dynamic responses of tall buildings when mean wind loading is multiplied by Gust Effect Factor "G".

Below is a work out example of Gust Effect Factor "G" for One kilometer Building.

15.1. Worked Example of Gust Effect Factor for One Kilometer Tall Building

Gust Effect Factor by ASCE 7-10

Mean roof height of the structure, h = 3280 ft.

Equavalent height of the structure (Section: 26.9.4)

$\bar{z} = 0.6h = 0.6 \times 3280 = 1968 ft.$

Consider exposer "B"

$\bar{\alpha} = \frac{1}{4}$ (3-sec. gust speed power law exponent – Table 26.9.1)

$\bar{b} = 0.45$ (mean hourly wind speed factor – 26.9.1)

c = 0.3 (Turbulence intensity factor – Eq. 26.9.7 from Table 26.9.1)

$\bar{\varepsilon} = \frac{1}{3}$ (Integral length scale power law exponent, Table 26.9.1)

B = 347.68 ft. (Horizontal dimension of building measured parallel to wind direction in ft.)

L = 347.68 ft. (Horizontal dimension of building measured parallel to wind direction in ft.)

l = 320 (Integral length scale factor from Table 26.9-1 in ft)

(Building Fundamental natural frequency n_0, in Hz) Eq. 13

$$n_0 = \frac{150}{h} = 0.04573$$

β = 0.02 (Damping ratio, percent critical for building)

(Basic wind speed for three second gust speed at 33 ft)

Consider a basic wind speed, $V1 = 140$ mph

(Intensity of turbulence at height $\overline{z}$ from Eq. 26.9-7)

$$I_{\overline{z}} = c\left(\frac{33}{\overline{z}}\right)^{\frac{1}{6}} \qquad I_{\overline{z}} = 0.152$$

(Integral length scale of turbulence in ft, Eq. 26.9-9)

$$L_{\overline{z}} = l\left(\frac{\overline{z}}{33}\right)^{\overline{\varepsilon}} \qquad L_{\overline{z}} = 1250.222$$

(Background response factor "Q" Eq. 26.9-8)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\overline{z}}}\right)^{0.63}}} \qquad Q = 0.669$$

(Mean hourly wind speed ft/s at height z_1, Eq. 26.9- 16)

$$V_{\overline{z}} = \overline{b}\left(\frac{\overline{z}}{33}\right)^{\overline{\alpha}} V1\left(\frac{88}{60}\right)$$

$$V_{\overline{z}} = 256.773$$

$N_1 = \frac{n_0 L_{\overline{z}}}{V_{\overline{z}}}$ $N_1 = 0.223$ Reduced frequency, Eq. 26.9-4

$R_n = \frac{7.47\, N_1}{(1+10.3\, N_1)^{\frac{5}{3}}}$ $R_n = 0.228$ (Eq. 26.9-13)

$\eta_h = \frac{4.6\, n_0 h}{Vz}$ $R_h = \left(\frac{1}{\eta_h}\right) - \left(\frac{1}{2\eta h2}\right)^{(1-e^{-2\eta h})}$ (Eq. 26.9-15a)

$\eta_h = 2.687$ $R_h = 0.303$

$\eta_B = \frac{4.6 n_0 B}{Vz}$ $R_B = \left(\frac{1}{\eta_B}\right) - \left(\frac{1}{2\eta B2}\right)^{(1-e^{-2\eta B})}$ (Eq. 26.9-15a)

$\eta_B = 0.285$ $R_B = 0.834$

$\eta_L = \frac{15.4 n_0 L}{Vz}$ $R_L = \left(\frac{1}{\eta_L}\right) - \left(\frac{1}{2\eta L2}\right)^{(1-e^{-2\eta L})}$ (Eq. 26.9-15a)

$\eta_L = 0.953604$ $R_L = 0.58$

$R = \sqrt{\left(\frac{1}{\beta}\right) R_n R_h R_B (0.53 + 0.47 R_L)}$ $R = 1.522$ (Eq. 26.9-12)

$g_Q = 3.4$ $g_v = 3.4$ (Section 26.9.5)

$g_R = \sqrt{2\ln(3600 n_0)} + \frac{0.577}{\sqrt{2\ln(3600 n_0)}}$ $g_R = 3.376$ (Eq. 26.9-11)

Gust Effect Factor

$$G_f = \frac{1+1.7\times I_{\bar{z}}\sqrt{g_Q{}^2Q^2+R^2g_R{}^2}}{1+1.7g_v I_{\bar{z}}}$$ $G_f = 1.305027$ (Eq. 26.9-10)

16. Wind force for Dynamic Response of One Kilometer Tall Buildings

The application of Gust Effect Factor "G" to get dynamic responses of a building, the wind load calculations yield a static design pressure, which is expected to produce the same peak effect (dynamic responses) as the actual turbulent wind, with due consideration for building properties such as height, width, natural frequency of vibration, and damping. This approach is primarily for determining the overall wind loading and response of tall slender structure.

Wind Load calculations according to ASCE 7-10

Let basic wind speed V_b = 140 miles/hours has been considered for this analysis.

V_b = 140 miles/hours = 62.5 m/s

Consider Exposer Category "B" – Chapter 26.

Kz = Velocity pressure co-efficient at height z, Chapter 27, Table 27.3-1.

$$K_Z = 2.01 \times \left(\frac{Z}{Z_g}\right)^{\frac{2}{\alpha}}$$

$\alpha = 7$, $z_g = 1200ft$ (α & z_g are tabulated in Table 26.9.1-Exposer "B")

Height considered, every after 100 meters for wind calculations.

As per Fig. 6 the gradient height for exposer "B" is 1200 ft. so z considered up to height 400m. Above this height wind pressure is uniform.

1) From 0 to 10m , z_10 = 10m,
2) From 10 to 100m, z_100 = 100m
3) From 100 to 200m, z_200 = 200m,
4) From 200 to 300m, z_300 = 300m,
5) From 300 to 400m, z_400 = 400m,

Values of velocity pressure co-efficients Kz at different heights

Apply the above values of z (heights) to Kz,

$Kz1 = 2.01\left(\frac{z\_10}{zg}\right)^{\frac{2}{\alpha}}$ Kz1 = 0.719

$Kz2 = 2.01\left(\frac{z\_100}{zg}\right)^{\frac{2}{\alpha}}$ Kz2 = 1.388

$Kz3 = 2.01\left(\frac{z\_200}{zg}\right)^{\frac{2}{\alpha}}$ Kz3 = 1.692

$Kz4 = 2.01\left(\frac{z\_300}{zg}\right)^{\frac{2}{\alpha}}$ Kz4 = 1.899

$Kz5 = 2.01\left(\frac{z\_400}{zg}\right)^{\frac{2}{\alpha}}$ Kz5 = 2.062

Topographic factor, $K_{zt} = 1$, Sec: 26.8.2

Wind directionality factor, $K_d = 0.85$, Table 26.6-1

Importance factor, I = 1.15 (Category III from Table 1-1 and applying to Table 6-1, ASCE 7-2

Velocity pressures (q) at different heights – ASCE 7-10, Eq. 27.3-1

$q = 0.613 \times Kz1 \times K_{zt} \times K_d \times (V_b)^2$

Values at different heights.

$q1 = 0.613 \times Kz1 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$

$q1 = 1.682 \times 10^3 \times \frac{newton}{m^2}$

$q2 = 0.613 \times Kz2 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$

$q2 = 3.248 \times 10^3 \times \frac{newton}{m^2}$

$q3 = 0.613 \times Kz3 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$

$q3 = 3.959 \times 10^3 \times \frac{newton}{m^2}$

$q4 = 0.613 \times Kz4 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$

$q4 = 4.446 \times 10^3 \times \frac{newton}{m^2}$

$q5 = 0.613 \times Kz5 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$

$q5 = 4.827 \times 10^3 \times \frac{newton}{m^2}$

@seismicisolation

Wind force on windward walls:

Gust Effect Factor, Eq. 29.9.10, G_1 = 1.305 (Value taken from the Equition of Sec: 15.1)

External pressure coefficient, C_{pw} = 0.8, Fig. 27.4-1, ASCE 7-10

Internal pressure coefficient, GC_{pi} = 0.18, Fig. 26.11-1, ASCE 7-10

Wind pressure on windward face,

$pw = q \times G_1 \times Cpw - q \times (GCpi)$, Eq. 27.4-1, ASCE 7-10

Pressures at different heights

$$pw1 = q1 \times G_1 \times Cpw - q1 \times (GCpi) \qquad pw1 = 2.1 \frac{KN}{m^2}$$

$$pw2 = q2 \times G_1 \times Cpw - q2 \times (GCpi) \qquad pw2 = 4.054 \frac{KN}{m^2}$$

$$pw3 = q3 \times G_1 \times Cpw - q3 \times (GCpi) \qquad pw3 = 4.942 \frac{KN}{m^2}$$

$$pw4 = q4 \times G_1 \times Cpw - q4 \times (GCpi) \qquad pw4 = 5.549 \frac{KN}{m^2}$$

$$pw5 = q5 \times G_1 \times Cpw - q5 \times (GCpi) \qquad pw5 = 6.024 \frac{KN}{m^2}$$

Wind force on Leeward walls:

External pressure coefficient, C_{pl} = 0.5, Fig. 27.4-1, ASCE 7-10

pc = wind pressure on Leeward face.

$$pc1 = q1 \times G_1 \times Cpl - q1 \times (GCpi) \qquad pc1 = 1.428 \frac{KN}{m^2}$$

$$pc2 = q2 \times G_1 \times Cpl - q2 \times (GCpi) \qquad pc2 = 2.757 \frac{KN}{m^2}$$

$$pc3 = q3 \times G_1 \times Cpl - q3 \times (GCpi) \qquad pc3 = 3.361 \frac{KN}{m2}$$

$$pc4 = q4 \times G_1 \times Cpl - q4 \times (GCpi) \qquad pc4 = 3.774 \frac{KN}{m^2}$$

$$pc5 = q5 \times G_1 \times Cpl - q5 \times (GCpi) \qquad pc5 = 4.097 \frac{KN}{m^2}$$

Tier-1: Windward Side, from 0 to 296.4 meters

"19" shown in the below calculations, are the number of node points in wind ward side (106 m) of **Tier-1** of the model where wind forces are applied and 3.8 meters are the storey heights.

Forces applied from 0 to 100 meters

$$F_{10\ to\ 100} = 106m \times 3.8m \times \frac{(pw1 + pw2)}{2} \qquad F_{10\ to\ 100} = 1.239 \times 10^3\ KN$$

$$n_{10\ to\ 100} = \frac{F_{10\ to\ 100}}{19} \qquad n_{10\ to\ 100} = 65.2[illegible]0\ KN$$

Forces applied from 100 to 200 meters

$F_{100\ to\ 200} = 106m \times 3.8m \times \frac{(pw2+pw3)}{2}$ $\quad F_{100\ to\ 200} = 1.812 \times 10^3\ KN$

$n_{100\ to\ 200} = \frac{F_{100\ to\ 200}}{19}$ $\quad n_{100\ to\ 200} = 95.356\ KN$

Forces applied from 200 to 296 meters

$F_{200\ to\ 296} = 106m \times 3.8m \times \frac{(pw3+pw4)}{2}$ $\quad F_{200\ to\ 296} = 2112.822\ KN$

$n_{200\ to\ 296} = \frac{F_{200\ to\ 296}}{19}$ $\quad n_{200\ to\ 296} = 111.201\ KN$

Tier-2: Windward Side, from 296 to 554.8 meters

"17" shown in the below calculations, are the number of node points in wind ward side (80 m) of **Tier-2** of the model where wind forces are applied and 3.8 meters are the storey heights.

Forces applied from 296 to 300 meters

$F_{296\ TO\ 300} = 80m \times 3.8m \times \frac{(pw3+pw4)}{2}$ $\quad F_{296\ to\ 300} = 1.595 \times 10^3 KN$

$n_{296\ to\ 300} = \frac{F_{296\ to\ 300}}{17}$ $\quad n_{296\ to\ 300} = 93.799\ KN$

Forces applied from 300 to 400 meters

$F_{300\ TO\ 400} = 80m \times 3.8m \times \frac{(pw4+pw5)}{2}$ $\quad F_{300\ to\ 400} = 1.759 \times 10^3 KN$

$n_{300\ to\ 400} = \frac{F_{300\ to\ 400}}{17}$ $\quad n_{300\ to\ 400} = 103.476\ KN$

Forces applied from 400 to 525 meters

$F_{400\ TO\ 525} = 80m \times 3.8m \times pw5$ $\quad F_{400\ to\ 525} = 1.831 \times 10^3\ KN$

$n_{400\ to\ 525} = \frac{F_{400\ to\ 525}}{17}$ $\quad n_{400\ to 525} = 107.727\ KN$

Forces applied from 525 to 555 meters

$F_{525\ TO\ 555} = 80m \times 3.8m \times pw5$ $\quad F_{525\ to\ 555} = 1.831 \times 10^3\ KN$

$n_{525\ to\ 555} = \frac{F_{525\ to\ 555}}{17}$ $\quad n_{525\ to 555} = 107.727\ KN$

Tier-3: Windward Side, from 554.8 to 775.2 meters

"17" shown in the below calculations, are the number of node points in wind ward side (54 m) of **Tier-3** of the model where wind forces are applied and 3.8 meters are the storey heights.

Forces applied from 555 to 600 meters

$F_{555\ TO\ 600} = 54m \times 3.8m \times pw5$ $\quad F_{555\ to\ 600} = 1.236 \times 10^3\ KN$

$n_{555\ to\ 600} = \frac{F_{525\ to\ 555}}{17}$ $\quad n_{555\ to600} = 72.715\ KN$

Forces applied from 600 to 775 meters

$F_{600\ TO\ 775} = 54m \times 3.8m \times pw5$ $\quad F_{600\ to\ 775} = 1.236 \times 10^3\ KN$

$n_{600\ to\ 775} = \frac{F_{525\ to\ 555}}{17}$ $\quad n_{600\ to\ 775} = 72.715\ KN$

Tier-4: Windward Side, from 775.2 to 957.6 meters

"15" shown in the below calculations, are the number of node points in wind ward side (34 m) of **Tier-4** of the model where wind forces are applied and 3.8 meters are the storey heights.

Forces applied from 775 to 957 meters

$FT_4 = 34m \times 3.8m \times pw5$ $\quad FT_4 = 778.324\ KN$

$nT_4 = \frac{FT_4}{15}$ $\quad nT_4 = 51.888\ KN$

Tier-5: Windward Side, from 957.6 to 1003.2 meters

"3" shown in the below calculations, are the number of node points in wind ward side (12 m) of **Tier-5** of the model where wind forces are applied and 3.8 meters are the storey heights.

Forces applied from 957 meter to 1003.2 meters

$FT_5 = 12m \times 3.8m \times pw5$ $\quad FT_5 = 274.703\ KN$

$nT_5 = \frac{FT_5}{3}$ $\quad nT_5 = 91.568\ KN$

Tier-1: Leeward Side, from 0.0 to 100 meters

Same procedure will be followed for Leeward side. Here I gave the calculation for a certain height (0.0 to 100 meters) as an example.

Forces applied from 0.0 meter to 100 meters

$FS_{0\ to\ 100} = 106m \times 3.8m \times \frac{(pc1+pc2)}{2}$ $\quad F_{0\ to\ 100} = 842.946KN$

$n_{0\ to\ 100} = \frac{FS_{0\ to\ 100}}{19}$ $\quad n_{0\ to\ 100} = 44.366\ KN$

17. Stiffness Modifiers

For the analysis and design of tall buildings, engineers sometimes become confused on selecting the modification factors for stiffnesses of the structural members. Especially for lateral deflection (drift) at the top of the building, is it required to decrease or increase the stiffness factors of the structural members? Below, described in details, where to consider the modification factors to be used for structural members according to Codes. Now-a-days, acceleration is considered one of the main factors for tall buildings for human perceptions. The author not found the modification factors of stiffness of the structural elements for acceleration. Author suggests that the modification factor for acceleration will be same as for drift of tall buildings, as there are relations between the acceleration and drift of a building.

17.1. What are stiffness modifiers?

Stiffness modifiers are the factors to increase or decrease some properties of the cross section for example area, inertia, torsional constant etc. Generally they are used to reduce stiffness of concrete sections to model for cracked behavior of concrete. They are only applied to concrete members because it cracks under loading.

17.2. Background

Design of sections is carried out based on the forces calculated from analysis of the structure. These forces depend upon stiffness of the members. Stiffness is the ability to attract moment, shear, axial force etc. St iffer an element, more fo rce it attracts and more reinforcement we design for. In a building some elements are more stiffs, and others are less stiff. So they attract different amounts of forces depending upon their stiffness. Applied loading on a building produces internal forces. These internal forces like flexure, shear, torsion and axial forces result in compression or tension in concrete fibers. Concrete is strong in compression but it is only good in tension as little as about 10% of its compressive strength.

At this limit, concrete cracks, reduces in area and stiffness. It is no longer available to resist tensile actions. As the stiffness reduces so does the moment attracting ability also reduces. Some of the moment which was present at this section (for example at beams) goes to other areas which are not yet cracked (for example columns). This reshuffling of the stiffness in the whole structure leads to redistribution of moments. So these uncracked regions (for example columns) have to be designed for more moment than what they actually received before moment redistribution. This phenomenon is called redistribution of moments.

Those regions which were uncracked and received extra moments from cracked regions will also crack once the concrete in that region reaches its tensile capacity limit. So this cycle of moment redistribution continues until all the members have been cracked. Steel reinforcement which sits idle before this stage now starts taking these redistributed moments.

17.3. Why we need to change stiffness, what is gross/cracked section analysis and are these modifiers for service or ultimate design?

Cracking will affect the stiffness of the structure which in result will affect the deflection and forces. No one knows how real the exact cracking would be, what would be its extent and how different the load distribution would be after that.

Since there is cracking that occurs in the section there is a variable moment of inertia. In regions of cracking there is a reduced moment of inertia and in areas without cracking the moment of inertia is much larger. Also, as the load increases, so does the cracking. The moment of inertia changes with load. This moment of inertia that changes with load makes our deflection equations non-linear with respect to loading and we cannot use superposition to determine load combinations.

A non-linear analysis shall be required to predict extent of cracking. For linear analysis it is necessary to have some reasonable assumptions about stiffness of the members.

Changing the relative stiffness of members of a structure so the intent is to have a good design by limiting its deflection, sway and cracking is a serviceability issue, that means we change stiffness, increase here and decrease there and change the analysis results to adjust our structure to control large deflections and cracks.

We may wish not to change the stiffness and just design on original stiffness of the members. That design may show large cracks and deflections and may impair its service use to the occupants but still would be within its strength design limits. Analysis can be good or bad, more cracked or less cracked depending on our selection of member stiffness but the results obtained will be used to design the members.

A non-linear analysis shall be required to predict extent of cracking. For linear analysis it is necessary to have some reasonable assumptions about stiffness of the members.

Changing the relative stiffness of members of a structure so the intent is to have a good design by limiting its deflection, sway and cracking is a serviceability issue, that means we change stiffness, increase here and decrease there and change the analysis results to adjust our structure to control large deflections and cracks.

We may wish not to change the stiffness and just design on original stiffness of the members. That design may show large cracks and deflections and may impair its service use to the occupants but still it would be within the strength design limits. Analysis can be good or bad, more cracked or less cracked depending on our selection of member stiffness but the results obtained will be used to design the members. Gross or cracked section means gross or cracked inertia of the section.

Inertia has the same meaning as that of stiffness because E is held constant in all this discussion. As the stiffness is a function of E and I. For RC members, we can assume a constant E, if deformations stay within the elastic region (the normal assumption for most materials). The effective moment of inertia, Ie, is a moment of inertia that, when used with deflection equations developed for prismatic members, will yield approximately the same result as a more rigorous analysis that considers variable moment of inertia. This effective moment inertia will always be between the two extremes of the gross moment of inertia and the cracked moment of inertia, both of which have been discussed above.

You need reduced stiffness for 3 purposes:-

1) Vertical Deflections

2) Horizontal Deflections

3) Slenderness

The ACI 318 Building Code Requirements for Structural Concrete uses Limit state design.

@seismicisolation

17.4. Limit State Design (LSD)

Limit state design (LSD) refers to a design method used in structural engineering. A limit state is a CONDITION of a structure beyond which it no longer fulfills the relevant DESIGN CRITERIA. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. There are two categories of limit states: strength and serviceability.

17.5. What values to use?

Let's see what chapter 8 of ACI 318-08 "Analysis and Design – General Considerations" says about stiffness:-

ACI 8.7.1 — Use of any set of reasonable assumptions shall be permitted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. The assumptions adopted shall be consistent throughout analysis.

Ideally, the member stiffnesses E_c I and GJ should reflect the degree of cracking and inelastic action that has occurred along each member before yielding. However, the complexities involved in selecting different stiffnesses for all members of a frame would make frame analyses inefficient in design offices. Simpler assumptions are required to define flexural and torsional stiffnesses.

ACI R8.8.1 — The selection of appropriate effective stiffness values depends on the intended performance of the structure. which means there are different assumptions of stiffness for different types of analysis. For example braced frames, sway frames, lateral analysis etc. We will explore them in detail.

17.5.1. Braced Frames

ACI R8.7.1 — For braced frames, relative values of stiffness are important. Two usual assumptions are to use gross E_cI values for all members or, to use half the gross E_cI of the beam stem for beams and the gross E_c I for the columns.

17.5.2. Unbraced Frames

ACI R8.7.1 — For frames that are free to sway, a realistic estimate of E_cI is desirable and should be used if second-order analyses are carried out. Guidance for the choice of Ec I for this case is given in R10.10.4.

17.5.3. Torsional Stiffness

ACI R8.7.1 — Two conditions determine whether it is necessary to consider torsional stiffness in the analysis of a given structure:

(1) the relative magnitude of the torsional and flexural stiffnesses, and

(2) whether torsion is required for equilibrium of the structure (equilibrium torsion) or is due to members twisting to maintain deformation compatibility (compatibility torsion). In the case of compatibility torsion, the torsional stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

17.5.4. Lateral Deflections

ACI 8.8.2 — The selection of appropriate effective stiffness for reinforced concrete frame members (for lateral deflections) has dual purposes: to provide realistic estimates of lateral deflection and to determine deflection-imposed actions on the gravity system of the structure. A detailed nonlinear analysis of the structure would adequately capture these two effects. A simple way to estimate an equivalent nonlinear lateral deflection (δem at the top story in IBC 2006) using linear analysis is to reduce the modeled stiffness of the concrete members in the structure. The type of lateral load analysis affects the selection of appropriate effective stiffness values.

Deflections from Factored lateral loads

17.5.5. Earthquake load in IBC 2006

ACI R8.8.1 — When analyzing a structure subjected to earthquake events at short recurrence intervals, some yielding without significant damage to the members may be a tolerable performance objective.

ACI R8.8.2 — The lateral deflection a structure sustains under factored lateral loads can be substantially different from that calculated using linear analysis in part because of the inelastic response of the members and the decrease in effective stiffness.

For earthquake loading, a level of nonlinear behavior is tolerable depending on the intended structural performance and earthquake recurrence interval.

The alternative options presented in 8.8.2 use values that approximate stiffness for reinforced concrete building systems loaded to near or beyond the yield level and have been shown to produce reasonable correlation with both experimental and detailed analytical results.

ACI R8.8.2 — The lateral deflection a structure sustains under factored lateral loads can be substantially different from that calculated using linear analysis in part because of the inelastic response of the members and the decrease in effective stiffness.

For earthquake loading, a level of nonlinear behavior is tolerable depending on the intended structural performance and earthquake recurrence interval.

The alternative options presented in 8.8.2 use values that approximate stiffness for reinforced concrete building systems loaded to near or beyond the yield level and have been shown to produce reasonable correlation with both experimental and detailed analytical results.

The effective stiffnesses in Option (a) were developed to represent lower- bound values for stability analysis of concrete building systems subjected to gravity and wind loads.

Option (a) is provided so that the model used to calculate slenderness effects may be used to calculate lateral deflections due to factored wind and earthquake loading.

ACI 8.8.2 — Lateral deflections of reinforced concrete building systems resulting from factored lateral loads shall be computed either by linear analysis with member stiffness defined by (a) or (b), or by a more detailed analysis considering the reduced stiffness of all members under the loading conditions:

(a) By section properties defined in 10.10.4.1(a) through (c); or

(b) 50 percent of stiffness values based on gross section properties. ACI 10.10.4.1 — It shall be permitted to use the following properties for the members in the structure:

(a) Modulus of elasticity.................. E_c from 8.5.1

(b) Moments of inertia, I

Compression members:

Columns 0.70Ig

Walls —Uncracked . ... 0.70 Ig

—Cracked...................................... 0.35Ig

Flexural members:

Beams 0.35Ig

Flat plates and flat slabs............................. 0.25Ig

(c) Area.. 1.0Ag

ACI R10.10.4.1 — The values of Ec,I, and A have been chosen from the results of frame tests and analyses and include an allowance for the variability of the computed deflections.

The modulus of elasticity of the concrete, Ec, is based on the specified concrete compressive strength while the sway deflections are a function of the average concrete strength, which is higher.

The moments of inertia are taken from Reference 10.35, (Mac Gregor, J. G., and Hage, S. E., "Stability Analysis and Design Concrete," Proceedings, ASCE, V. 103, No. ST 10, Oct. 1977.), which are multiplied by the stiffness reduction factor φK = 0.875.

For example, the moment of inertia for columns is 0.875(0.80Ig) = 0.70Ig. These two effects result in an overestimation of the second-order deflections on the order of 20 to 25 percent, corresponding to an implicit stiffness reduction of 0.80 to 0.85 on the stability calculation.

The moment of inertia of T-beams should be based on the effective flange width defined in 8.12. It is generally sufficiently accurate to take I_g of a T- beam as two times the I_g for the web, 2(b wh3/12).

If the factored moments and shears from an analysis based on the moment of inertia of a wall, taken equal to $0.70I_g$, indicate that the wall will crack in flexure, based on the modulus of rupture, the analysis should be repeated with $I = 0.35I_g$ in those stories where cracking is predicted using factored loads. Deflections from SERVICE lateral loads

17.5.6. Wind load in IBC 2006

ACI R8.8.2— For analyses with wind loading, where it is desirable to prevent nonlinear action in the structure, effective stiffness representative of pre-yield behavior may be appropriate.

ACI R8.8.1— For wind loading, it is desirable to maintain elastic behavior in members at service load conditions.

As with lateral stability analysis of concrete structures (R10.10.4), a factor of 1.4 times the stiffness used for analysis under factored lateral loads is adequate to model effective section properties for lateral deflection analysis under service loads.

So drift can be checked on full inertia properties (modifier=0.7x1.4=1.0 of vertical elements) for service wind loads. Alternatively, a more accurate level of stiffness based on the expected element performance can be determined.

ACI R10.10.4.1— Section 10.10 provides requirements for strength and assumes frame analyses will be carried out using factored loads. Analyses of deflections, vibrations, and building periods are needed at various service (unfactored) load levels to determine the serviceability of the structure and to estimate the wind forces in wind tunnel laboratories.

The moments of inertia of the structural members in the service load analyses should be representative of the degree of cracking at the various service load levels investigated.

Unless a more accurate estimate of the degree of cracking at service load level is available, it is satisfactory to use 1.0/0.70 = 1.43 times the moments of inertia given here for service load analyses.

Idea is to use the stiffness properties of cross section based on its expected behavior in particular situation, as mentioned in commentary.

Commentary gave the example of analysis under wind and earthquake situations. In case of wind structure is intended to remain elastic under service loads so use properties based on gross cross section or 1.4 times the reduced cross section properties.

In case of earthquake reduced cross section properties are required at service level to estimate drifts and related p-delta/second effects.

Bottom line: it is complex topic and one have to use assumptions. Even if one is using 0.35 and 0.7 factors to size the cross section of member, structure should still stand provided assumptions are uniform throughout the analysis, as concrete has this ability to distribute moments according to provided reinforcement.

18. Earthquake Loading

Earthquake effects on a tall building are dynamic in nature. For simple and regular structures up to a specified height limit, an equivalent static load approach is adopted by building codes. For dynamically sensitive buildings or buildings with irregular configuration, a dynamic analysis is required. Such analysis is done either by response spectrum analysis or by time history analysis method. Design criteria and formulas for calculating seismic loads are provided by various seismic standards and regulations (ASCE 7-05, 2005: ICBO, 2005: NEHRP, 2002: IBC, 2003).

Tall buildings respond to seismic events somewhat differently from low-rise buildings. The inertia forces induced by seismic motion depends upon the building's stiffness, mass and ground acceleration, the interaction of soil and foundation, the type of foundation, and dynamic properties of the structure. The inertia force is defined by Newton's third law, F = ma, where m is the building mass and a is the acceleration. Tall buildings are more flexible than low-rise buildings, and hence experience lower levels of acceleration than the low-rise buildings.[17]

[28]Since the 1960s, considerable research, as well as lessons learned from previous earthquakes, have led to improved understanding of the seismic behavior of structural walls. Among the first reported observations concerning the seismic behavior of structural walls were those after the Chilean earthquake of 1960, as reported in the Advance Engineering Bulletin No. 6 issued by the Portland Cement Association, where the efficiency of structural walls in controlling structural damage during severe earthquakes was noted. In the early 1970s, Fintel (1974) indicated that properly designed structural walls could be used effectively as the primary lateral-load resisting system for both wind and earthquake loading in multistory buildings. Structural wall systems have also been recognized as a favorable alter- native to ductile moment-resistant frames (Sittipunt et al., 2001). Today, rein-forced concrete structural walls are frequently used as the primary component of the lateral load-resisting system in buildings located in earthquake-prone regions because of their substantial contribution to building lateral

@seismicisolation

strength and stiffness. Reinforced concrete structural walls are deep and relatively thin, vertical cantilever members, also referred to as "shear walls". Structural walls are widely used in rein- forced concrete buildings located in earthquake-prone regions as the primary later- al-load resisting mechanism, because of their efficiency to provide lateral strength and stiffness, and control the lateral drift.

After the 1970s, the interest on the seismic behavior of isolated and coupled structural walls grew. Fintel (1991) documented the superiority of structural walls to resist lateral forces induced by seismic excitations and several experimental investigations focusing on structural walls were carried out in the U.S., Europe, New Zealand, and Japan. In particular, the remarkably good performance of structural wall dominant buildings during the 1985 Chilean earthquake inspired an increase in the amount of research on the seismic behavior and detailing requirements of reinforced concrete walls in the U.S. (Thomsen and Wallace, 1995).

[17]The main function of a building is to serve its users. Understanding the priority of user needs is especially critical for designers of tall buildings, which are highly vulnerable to wind induced motion which can cause excessive vibration. Service- ability and occupant comfort therefore, are of great concern. Even though the structure satisfactorily carries the lateral loads, it must meet the serviceability criteria of avoiding occupants' feeling of discomfort, such as headaches, dizziness, and nausea that may result from excessive movement of tall building.

[5]Earthquake-induced motions, as far as human response in concerned, are quite different from those that result from wind forces. Earthquakes occur much less frequently than wind storms, and only certain areas of the world. Their duration of vibration can be generally very short and transient in nature, and motions induced are generally much more violent. In addition to the horizontal accelerations, significant vertical accelerations may also be present. People who experience earthquakes of any severity are simply thankful to have survived the ordeal: hence the design criterion is one of safety rather than comfort.

@seismicisolation

19. Tuned Mass Damper

Damping is an important issue as the human comfort due to excessive acceleration beyond 25 milli-g, in the range of 35 to 50 milli-g, may have to be designed for. Tuned mass dampers and viscoelastic dampers are often used [10].

Tuned Mass Dampers (TMDs) transmit inertial force to the building's frame to reduce its motion around up to 50%.

In table-3, there are some examples of tall buildings which reduced their accelerations by introducing TDMs.

Table 3. Configurations of some TMDs in use

Host Structures	Descriptions	Results
Hancock Tower (244 m) in Boston, USA	Two TMDs were installed at opposite ends of 58th floor, each weighing 300 tons	Can reduce building's response 50% [4]
Citicorp Building (278 m) in NewYork, USA	A 40 tons concrete block with two spring damping mechanisms installed in 63rd floor	Reduces wind induced response 40% [4]
Sydney Tower (305 m), Australia	Doughnut-shaped water tanks & energy issipating shock absorbers	Response reduced 40-50% [4]
Sendai AERU (145.5) IN Sendai	TMD w/laminated rubber bearing + coil spring.	Response reduced ½ [4]
Petronas Twin Tower (452m) in Kuala Lumpur	12 Fluid Dampers	Prevent vortex shedding & reduce wind-induced excitation. [4]
Taipei 101 Skyscraper (509.2 m) in Taiwan	Installed world's largest & heaviest TDMs weighing 728 short-ton	To offset movements in the building caused by strong gusts. [6]
Burj-Al-Arab (321 m) in Dubai	Installed 11 TMDs	Reduced wind induced response. [14]

@seismicisolation

20. Vanity Height

Vanity/Spire height: In theory, we're in the midst of a "golden age" of skyscraper construction. But why, of the ten tallest building on Earth, nearly 30 percent of each structure totally unusable spire? In truth, this information is readily available to anyone with eyeballs. All supertalls (e.g, any building over 1,000 feet tall) have substantial spires and unoccupied upper floors, which serve to house hardware, observation decks, and often, mass damper that counter the sway of the building in the wind. But even taking into account the necessary infrastructure, the majority of spires are totally unnecessary. In fact, without the vanity height, 60 percent of the world's supertalls wouldn't actually be supertalls at all. The Burj-Khalifa would lose more than 700 feet. If an angry giant broke off the Burj's spire and planted it on the ground, it'd still be the 11th tallest building in Europe. The worst offender of all is the Burj Al Arab, of which 39 percent is vanity spire [12].

In table 4, there are some examples of vanity height of tall buildings

Table 4. Vanity Height of the Towers [13]

Towers	Total Heights	Vanity Heights	Percentage of vanity heights
Zifeng Tower – China	450	133	30
Bank of America Tower-New York	366	131	36
Burj Al-Arab- Dubai	321	124	39
Emirates Tower One-Dubai	355	133	32
New York Times- New York	319	99	31
Nakheel Tower- Dubai	1000	N/A	10

@seismicisolation

21. Story Drift

[9]Drift (lateral deflections) of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usages for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control of steel Building Structures 1993). These limits generally are sufficient to minimize damage to cladding and non structural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. West and Fisher (2003) contains recommendations for high limits that have successfully been used in law-rise buildings with various cladding types. It also contains recommendations for buildings containing cranes. An absolute limit on story drift may also need to be imposed in light of evidence that damage to non structural partitions, cladding, and glazing may occur if the story drift exceeds about 10 mm (3/8 in) unless special detailing practices are made to tolerate movement (Freeman 1977 and Cooney and King 1988).

22. High Rise Behavior

[5]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. Horizontal loading exerts at each level of a building a shear, a moment, and sometimes, a torque, which have maximum values at the base of the structure that increase rapidly with the building's height. The response of a structure to horizontal loading, in having to carry the external shear, moment, and torque, is more complex than its first-order response to gravity loading. 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 allocation of the external moment between the flexural and axial actions of the vertical components depends on the vertical shearing stiffness of the "web" system connecting the vertical components, that is, the girders, slabs, vertical diaphragm, and bracing. The stiffer the shear connection, the larger the proportion of the external moment that is carried by axial forces in the vertical members, and the stiffer and more efficiently the structure behaves.

The described flexural and axial actions of the vertical components and the shear action of the connecting members are interrelated (Fig. 25), 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 axial 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 building is resisted mainly by shear in the vertical components (In Parallel Shear-Wall [PSW] concept, only the shear walls), by the horizontal components 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 torsional 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. But in PSW concept, a structure's

@seismicisolation

resistance to bending is carried by coupled and individual shear walls. The structure's resistance to torsion will be carried by cross sectional areas of shear walls.

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. An additional mode of interaction between the vertical components, a horizontal force interaction can also play a significant role in stiffening the structure, and this also should be recognized when forming the model. Horizontal force interaction occurs when a horizontally 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. For this reason, in a tall wall-frame structure the wall tends to restrain the frame near the base while the frame restrains the wall near the top. Similarly, horizontal force interaction occurs when a structure consisting of dissimilar vertical components twists. 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. But in PSW concept, due to similar vertical components to displace about a Center of rotation and twist identically at each level, the connecting slabs/beams are not subjected to horizontal forces that redistribute the torque between the vertical components.

Having assessed a proposed structure's dominant modes of behavior, the formation of an appropriate model requires next knowledge of the available modeling elements and their methods of connection.

The concept of this structural system in mainly related to the influence of the composite action of the individual shear walls and the "frame" action resulting from the moment connection beam to resist the lateral loads. So the behaviors/functions of shear walls when connected with beams are discussed briefly in the paragraph below.

@seismicisolation

23. Replacement of Column by Shear Walls

A structure's stiffness can be increased by changing the shape of its vertical members such as columns to shear walls. These changed arrangements can resist lateral forces more than the column arrangements, which will also reduce horizontal deflection (drift) of the structure.

Below the calculation will show, how to increase the stiffness of a vertical member (column) by changing its shape (shear wall), by keeping the cross sectional area of the vertical member approximately same.

Let the column (Fig. 6) has width, cw = 1.5m, breath, cb = 1.5m

Cross sectional area, Ac = cw x cb = 2.25 m^2

Moment of inertia of the column in X or Y direction

$I_{XY} = \frac{(cw \times cb^3)}{12}$ $\quad I_{XY} = 0.422\ m^4$

Let us arrange two shear walls (Fig. 7), length & thickness are 2.813 and 0.4 meters respectively. One in X direction and the other is perpendicular to X direction, i.e. in Y direction.

Each shear wall has a length, sl = 2.813m, thickness st = 0.4m

Sum of two shear wall's cross sectional area

$As = 2 \times s1 \times st = 2.25m^2$

Which is same amount of sectional area of the above column's cross sectional area Ac.

Moment of inertia of a shear wall about the major axis,

$I_s = \frac{(st \times sl^3)}{12}$ $\quad I_s = 0.0.742\ m^4$

It is seen that the moment of inertia of one shear wall becomes 1.758 times (0.742/0.422 = 1.758) stiffer that the column in each direction whereas sum of the cross sectional area/volume of concrete of one column.

Table-5 shows 3 cases as example to show, how to gain strength by changing the shape of vertical members/columns.

Therefore it is observed from the above equations that the change of shape, from column to shear wall cases increment of lateral stiffness to a structure. Moment of inertia is inversely proportional to deflection, as the moment of inertia increases the deflection decreases. So by arranging shear walls with this concept, can reduce the drift at top of a building.

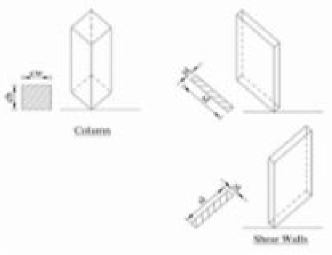


Figure 7. Column converted to shear walls

	Case-1		Case-2		Type-3	
	Column (1.0m x 1.0m)	Shear wall (2m x 0.5m)	Column (1.5m x 1.5m)	Shear wall (2m x 2.813m)	Column	Shear wall
Number of Members	2	2	1	2	2	2
Breath (m)	1	0.5	1.5	0.4	0.8	0.4
Width/Depth (m)	1	2	1.5	2.813	0.8	1.6
Area (m^2)	2	2	2.25	2.25	1.28	1.28
Inertia (m^4) X-Dir.	0.167	0.333	0.422	0.742	0.068	0.137
Inertia (m^4) Y-Dir.	0.167	0.333	0.422	0.742	0.068	0.137
Increase of Stiffness/Rigidity in %	200%		175%		200%	

Table 5. Comparison of Inertia forces after changing shapes

@seismicisolation

24. Behavior of Shear Walls

A shear wall structure is considered to be one whose resistance to horizontal loading is provided entirely by shear walls. They are usually continuous down to the base to which they are rigidly attached to form vertical cantilevers. 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.

The reinforced concrete shear wall is one of the most commonly used lateral load resisting in high-rise building. Shear wall systems are one of the most commonly used lateral-load resisting systems in high- rise buildings. Reinforced concrete shear walls have very high in-plane stiffness and strength, which can be used to simultaneously resist large horizontal loads and support gravity loads, making them quite advantageous in many structural engineering applications. There are lots of literatures available to design and analyze the shear wall. However, the decision about the location of shear wall in multi-storey building is not much discussed in any literatures. This PSW concept focused to determine the solution for shear wall location in multi-story buildings.

Today's tall buildings are becoming more and more slender, leading to the possibility of more sway in comparison with earlier high rise buildings. Improving the structural systems of tall buildings can control their dynamic response, with more appropriate structural forms such as shear walls and tube structures and improved material properties. The general design concept of the contemporary bearing wall building system depends upon the combined structural action of the floor and roof systems with the walls. The floor system carries vertical loads and, acting as a diaphragm, lateral loads to the walls for transfer to the foundation. Lateral forces of wind and earthquake are usually resisted by shear walls which are parallel to the direction of lateral load. These shear walls, by their shearing resistance and resistance to overturning, transfer the lateral loads to the foundation.

To reduce damage in the event of wind and an earthquake, it is desirable to have large lateral stiffness. Shear walls contribute significant lateral stiffness, strength, and overall ductility and energy dissipation capacity.

[24] Structural walls provide great stiffness as a lateral load resisting system. They reduce the seismic deformation demands and hence the damage to the remainder of the building (i.e. the gravity load carrying system) due to their high in-plane stiffness.

@seismicisolation

[24]One of the key points in the strategy of planning structural walls is to distribute the anticipated inelastic deformations in a reasonably uniform manner over the entire plan of the building rather than concentrating them in only a few walls.

[17]Shear walls increase the building's natural frequency and reduce wind induced acceleration to ensure occupant comfort. However, this leads to greater seismic forces on the structure. It has been noted though, after a few recent earthquakes, that the shear walls keep the buildings stable against total collapse and may be beneficial for residential buildings to ensure increased safety factor of occupants.

As an aid to understanding the behavior of shear wall structure, it is useful to categorize them as proportionate or nonproportionate system.

[5]A proportionate system is one in which the ratios of the flexural rigidities of the walls remain constant throughout their height as in Fig-11a. For examples, a set of walls whose lengths do not change throughout their height, but whose changing wall thickness 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.

A nonproportionate system is one in which the ratios of the walls' flexural rigidities are not constant up the height (Fig. 11b). At levels where the rigidities change, redistribution of the wall shears and moments occur, with corresponding horizontal interaction in the connecting member and possibility of very high local shears in the walls. Author's concept belongs to proportionate system.

25. Analysis of proportionate Wall Systems

The problem of analyzing a proportionate wall system is relatively uncomplicated because of its statical determinacy. It will be considered in two subcategories of structure- those that do not twist and those with twist.

25.1. Proportionate Nontwisting Structures

A structure that is symmetrical on plan about the axis of loading, as Fig. 8 will not twist. At any level i, the total external shear Qi, and the total external moment Mi, will be

@seismicisolation

distributed between the walls in the ratio of their flexural rigidities. The resulting shear and moment in a wall j at a level i can be expressed as

$$Q_{ji} = Q_i \frac{(EI)_{ji}}{\sum(EI)_i} \qquad (16)$$

$$M_{ji} = M_i \frac{(EI)_{ji}}{\sum(EI)_i} \qquad (17)$$

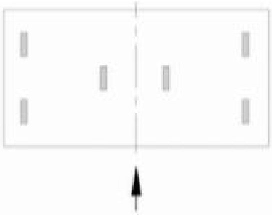


Figure 8. Symmetric shear wall structure

where $(EI)_{ji}$ is the flexural rigidity of wall j at level i and $\sum(EI)_i$ represents the summation of the flexural rigidities of all the walls at level i.

In such a proportionate non twisting structure, there is no redistribution of shear or moment at the change levels, and no interactive forces between the walls.

25.2. Proportionate Twisting Structures

A structure that is not symmetric on plan about the axis of loading will generally twist as well as translate. In a proportionate shear wall structure that twists under the action of horizontal loading (Fig. 9) the resulting horizontal displacement of any floor is a combination of a translation and a rotation of the floor about a centre of twist, which, in a proportionate structure, is located at the "centroid" of the flexural rigidities of the walls. Referring to the asymmetric cross-wall structure in Fig. 10, and assuming that the stiffness of a planar wall transverse to its plane is negligible, the X-location of the center of twist from an arbitrary origin is

$$x = \frac{\sum(EIx)_i}{\sum(EI)_i} \qquad (18)$$

in which $(EI)_i$ and $(EIx)_i$ are, respectively, the sum of the flexural rigidities and the sum of the first moments of the flexural rigidities about the origin, for all the walls parallel to the Y axis at level i.

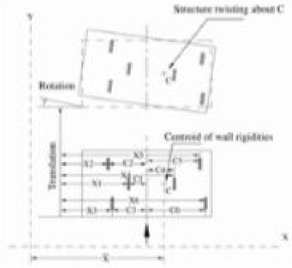


Figure 9. Displacement of asymmetric structure

In a proportionate structure, the center of twist and the shear center axis of the structure coincide. Consequently, the effect of horizontal loading on the structure is to produce at level i a resultant shear Q_i and its eccentricity e from the shear centre, that is Q_ie The resultant shear in any wall j at level i is a combination of its share of the external shear and the shear due to resisting its share of the external torque at that level, which may be expressed as

$$Q_{ji} = Q_i \frac{(EI)_{ji}}{\sum(EI)_i} + Q_i e \frac{(EIc)_{ji}}{\sum(EIc^2)_i} \qquad (19)$$

in which c_{ji} is the distance of wall j from the shear center. Noting that the moment in a wall can be obtained by integrating the shear $\left(M = \int_z^H Q dz\right)$, integrating Eq. 19 leads to an expression for the moment in wall j at level i,

$$M_{ji} = M_i \frac{(EI)_{ji}}{\sum(EI)_i} + M_i e \frac{(EIc)_{ji}}{\sum(EIc^2)_i} \qquad (20)$$

are associated with bending of the walls as the structure twists.

In Eqs. 19 & 20, c_{ji} is taken as positive when on the same side of the center of twist as the eccectricity e. Consequently, walls on the same side of the center of twist as the resultant loading will have their shears and moments increased by the twisting behaviour, while those on the opposite side will have their shears and moments reduced.

If a proportionate structure also includes walls perpendicular to the direction of external loading, that is, aligned in the X direction, as in Fig. 10, the Y location of the center of twist can be defined by

$$\bar{y} = \frac{\Sigma(EIy)_i}{\Sigma(EI)_i} \qquad (21)$$

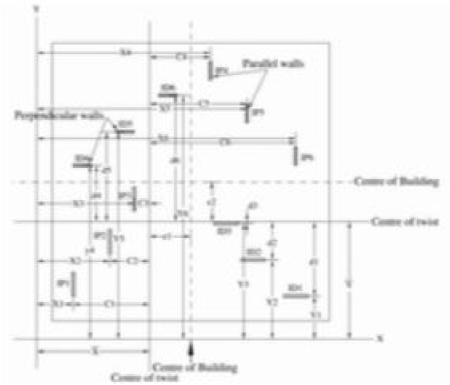


Figure 10. Proportionate Twisting Structure

in which the flexural rigidities refer to only the “perpendicular “ walls.

As the structure twists under the action of horizontal loading, the total set of orthogonally oriented walls will rotate about the axis of twist.

The effect of the “perpendicular” walls will be to stiffen the structure in torsion, to reduce the twist, and, in doing so, to influence the contributions to the “parallel” walls shears and moments that result from the structure’s twisting. The denominator of the second terms in Eqs. 19 and 20, for the shears and moments in the “parallel walls”, must then be modified to $\sum EIc^2 + \sum(EId^2)$, in which EIc^2 is the second moment of the “parallel” walls flexural rigidities about the centre of twist while EId^2 refers correspondingly to the “perpendicular” walls.

x6 = 21000mm

y6 = 20000mm

Shear wall lengths

sp1 = 4000mm

sp2 = 4000mm

sp3 = 4000mm

sp4 = 3000mm

sp5 = 3000mm

sp6 = 3000mm

Shear wall lengths

sd1 = 4000mm

sd2 = 4000mm

sd3 = 4000mm

sd4 = 3000mm

sd5 = 3000mm

sd6 = 3000mm

Consider thicknesses (st) of all shear wall, st = 500mm

Let the modulus of elasticity of concrete, $E_c = 2.5 \times 10^7 \frac{KN}{m^2}$

Moment of Inertia of the walls:

Parallel Walls

Perpendicular Walls

$IP1 = \frac{(st \times sp1^3)}{12}$ $IP1 = 2.667m^4$ – $ID1 = \frac{(st \times sd1^3)}{12}$ $ID1 = 2.667m^4$

$IP2 = \frac{(st \times sp2^3)}{12}$ $IP2 = 2.667m^4$ - $ID2 = \frac{(st \times sd2^3)}{12}$ $ID2 = 2.667m^4$

$IP3 = \frac{(st \times sp3^3)}{12}$ $IP3 = 2.667m^4$ - $ID3 = \frac{(st \times sd3^3)}{12}$ $ID3 = 2.667m^4$

$IP4 = \frac{(st \times sp4^3)}{12}$ $IP4 = 1.125m^4$ - $ID4 = \frac{(st \times sd4^3)}{12}$ $ID4 = 1.125m^4$

$IP5 = \frac{(st \times sp5^3)}{12}$ $IP5 = 1.125m^4$ - $ID5 = \frac{(st \times sd5^3)}{12}$ $ID5 = 1.125m^4$

$IP6 = \frac{(st \times sp6^3)}{12}$ $IP6 = 1.125m^4$ - $ID6 = \frac{(st \times sd6^3)}{12}$ $ID6 = 1.125m^4$

Perpendicular distance of the resultant force (middle of the building) from Y axis.

xd = 12000mm

Parallel Walls:

Sum of the flexural rigidities of all walls parallel to wind direction,

$$\sum EIp = E_c \times IP1 + E_c \times IP2 + E_c \times IP3 + E_c \times IP4 + E_c \times IP5 + E_c IP6$$

$$\sum EIp = 2.844 \times 10^8 KNm^2$$

@seismicisolation

Sum of the first moment of the flexural rigidities of all walls parallel to wind direction,

$$\sum EIpx = E_c \times IP1 \times x1 + E_c \times IP2 \times x2 + E_c \times IP3 \times x3 + E_c \times IP4 \times x4 + E_c \times IP5 \times x5 + E_c IP6 \times x6$$

$$\sum EIpx = 2.596 \times 10^9 KNm^3$$

Distance of the center of twist from axis 'y'

$X1 = \frac{\sum EIpx}{\sum EIp}$ X1 = 9.128m

Eccentricity   e1 = xd – X1, e1 = 2.872m

Distance of shear walls (IP1 to IP6) from the line (parallel to Y-axis) passing through centre of twist.

(c1, c2 & c3 are taken as +ve when on the same side of the center of twist as the eccentricity "e")

c1 = X1 - x1,   c2 = X1 - x2,   c3 = X1 – x3,   c4 = X1 – x4

c1 = 6.128m,   c2 = 3.128m,   c3 = 1.128m,   c4 = - 4.872m,

c5 = X1 – x5,   c6 = X1 – x6,

c5 = -7.872m,   c6 = - 11.872m

Consider shear and moment produced at the base due to wind forces,

Shear force, Q = 360 KN

Moment,   Mi = 360 KN x 50m = 18000 KN-m

Second moment of the parallel walls flexural rigidities about the center of twist,

$$\sum EIc^2 = E_c \times IP1 \times c1^2 + E_c \times IP2 \times c2^2 + E_c \times IP3 \times c3^2 + E_c \times IP4 \times c4^2 + E_c \times IP5 \times c5^2 + E_c \times IP6 \times c6^2$$

$$\sum EIc^2 = 9.615 \times 10^9 KNm^4$$

Shear forces & Moments on walls "IP1" & "IP2" (Fig. 10)

Shear at wall "IP1" $Q_{IP1} = \left[Q \frac{(E_c \times IP1)}{\sum EI_p} \right] + Q \times e1 \times \frac{(E_c \times IP1 \times c1)}{\sum EIc^2}$

$$Q_{IP1} = 128.324\ KN$$

Shear at wall "IP6" $Q_{IP6} = \left[Q \frac{(E_c \times IP6)}{\sum EI_p} \right] + Q \times e1 \times \frac{(E_c \times IP6 \times c6)}{\sum EIc^2}$

Q_{IP6} = [illegible] KN

Moment at wall "IP1" $M_{IP1} = \left[Mi \frac{(E_c \times IP1)}{\sum EI_p} \right] + Mi \times e1 \times \frac{(E_c \times IP1 \times c1)}{\sum EIc2}$

$$M_{IP1} = 6.416 \times 10^3 \times KN - m$$

Moment at wall "IP6" $M_{IP6} = \left[Mi \frac{(E_c \times IP6)}{\sum EI_p} \right] + Mi \times e1 \times \frac{(E_c \times IP6 \times c6)}{\sum EIc2}$

$$M_{IP6} = -14.845 \times KN - m$$

Perpendicular Walls:

Sum of the flexural rigidities of perpendicular walls,

$$\sum EId = E_c \times ID1 + E_c \times ID2 + E_c \times ID3 + E_c \times ID4 + E_c \times ID5 + E_c ID6$$

$$\sum EId = 2.844 \times 10^8 KNm^2$$

Sum of the first moment of the flexural rigidities of all walls perpendicular walls (Fig. 10).

$$\sum EIpy = E_c \times ID1 \times y1 + E_c \times ID2 \times y2 + E_c \times ID3 \times y3 + E_c \times ID4 \times y4 + E_c \times ID5 \times y5 + E_c ID6 \times y6$$

$$\sum EIpy = 2.734 \times 10^9 KNm^3$$

Perpendicular distance of centroid of building from X axis, yd = 11750mm

Distance of the center of twist from "X" axis

$Y1 = \frac{\sum EIpy}{\sum EId}$ Y1 = 9.615m

Eccentricity e2 = yd – Y1, e2 = 2.135m

Distance of shear walls (ID1 to ID6) from the line (parallel to X-axis) passing through centre of twist.

d1 = Y1 – y1 d1 = 6.115m

d2 = Y1 – y2 d2 = 3.115m

d3 = Y1 – y3 d3 = 0.115m

d4 = Y1 – y4 d4 = - 4.385m

d5 = Y1 – y5 d5 = 7.385m

d6 = Y1 – y6 d6 = - 10.385m

Second moment of the perpendicular walls' flexural rigidities about the center of twist,

$$\sum EId^2 = E_c \times ID1 \times d1^2 + E_c \times ID2 \times d2^2 + E_c \times ID3 \times d3^2 + E_c \times ID4 \times d4^2 + E_c \times ID5 \times d5^2 + E_c \times ID6 \times d6^2$$

Shear forces on walls ID1 & ID6

$Q_{\_ID1} = Q \times e2 \times \left[\frac{(E_c \times ID1 \times d1)}{\sum EIc^2 + \sum EId^2}\right]$ $\quad Q_{\_ID1} = 17.538\ KN$

$Q_{\_ID6} = Q \times e2 \times \left[\frac{(E_c \times ID6 \times d6)}{\sum EIc^2 + \sum EId^2}\right]$ $\quad Q_{\_ID6} = -12.564\ KN$

Moment on walls ID1 & ID6

$M_{\_ID1} = Mi \times e2 \times \left[\frac{(E_c \times ID1 \times d1)}{\sum EIc^2 + \sum EId^2}\right]$ $\quad M_{\_ID1} = 876.908\ KN$

$M_{\_ID6} = Mi \times e2 \times \left[\frac{(E_c \times ID6 \times d6)}{\sum EIc^2 + \sum EId^2}\right]$ $\quad M_{\_ID6} = -628.209\ KN$

In these calculations, the engineers can have an idea that, due to lateral forces on a structure, how the forces are distributed to the columns located at different places and in different directions. As an example, it is seen from Fig. 10 that shear wall "IP1" resists more lateral force than the shear wall of "ID1". Because the wall "IP1" is stiffer than "ID1" to the direction of forces and "IP1" nearer to the centre of twist than "ID1".

26. Coupled Walls (Shear walls connected with beams)

When two or more shear walls are interconnected by beams or slabs, the total stiffness of the system exceeds the sum of the individual wall stiffness.

[24]Coupled wall structures (Fig. 14) are outstanding lateral load resisting systems that not only reduce the deformation demands on the building, but also distribute the inelastic deformation both vertically and in plan, between the coupling beams and the wall piers. Different than cantilever walls, where the overturning moment is resisted entirely by flexural stresses, coupled walls resist the overturning moment by a combination of an axial force couple that develops in the wall piers as a result of shear demand in the coupling beams and flexural action in the wall piers.

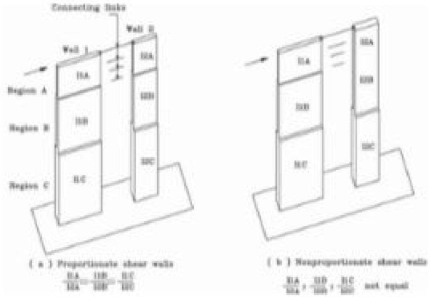


Figure 11. Proportionate & Non proportionate Shear Walls

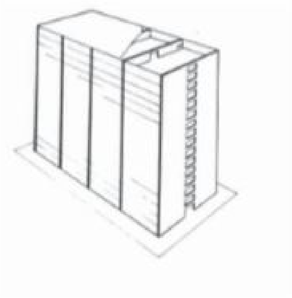
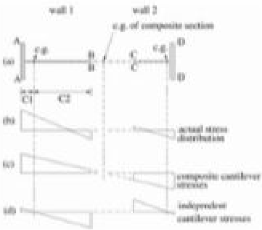


Figure 12. Coupled shear wall structure

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 the wall, with maximum tensile and compressive stresses at opposite edges (Fig. 13). 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 centroidal 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 (Fig. 13c). The practical situation of a pair of walls connected by flexible beams will lie between these two extreme cases, which may be regarded as bounds on the structural behavior of a couples shear system (Fig. 13b). The stiffer the connecting beams, the closer the structural behavior will approach that of a fully composite cantilever.



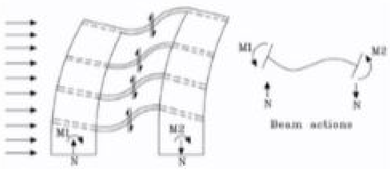
[5]**Figure 13. Superposition of stress distributions due to composite and individual cantilever actions to give true stress distribution in walls**

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 (Fig. 14). 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 and compressive in the leeward wall. The wind moment M at any level is then resisted by the sum of the bending moments M1 and M2 in the two walls at that level, and the moment of the axial forces N*l* (Fig. 14), where N is the axial forces in each wall at that level and l is the distance between their centroidal axes.

$M = M1+M2+Nl$

@seismicisolation

The last term N*l* represent the reverse moment caused by the bending of the connecting beams which oppose 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.



[5]Figure 14. Behavior of laterally loaded coupled shear walls

The action of the connecting beams 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. It is important to note that, to the author's structural arrangement, every shear wall is connected by beam in each grid. Several reverse moments N*l* which develops in beams, reducing the magnitude of the external moments of all the shear walls due to wind force. Therefore a large amount of external moment will be reduced due to the shear wall-beam connection system. [24]The degree of rigidity of the coupling beams governs the behavior of Couple wall (CW) systems.

The impact of the shear resistance of the connecting beams is to make the coupled wall system behave partly as a composite cantilever, bending about the centroidal axis of the wall group. The resulting stiffness of the coupled system is much greater than the summation of stiffnesses of the individual wall piers acting separately as uncoupled walls.

[24]The benefit of CWs becomes apparent when resisting much greater seismic later- al loads with the same wall layout required for wind loads. Coupled walls permit more compact wall layouts to resist a given lateral load; for example, CWs were used to great advantage to minimize the lateral load resisting system footprint in the tallest building in the world, the Burj Khalifa (Lee et al. 2008).

@seismicisolation

[24] In a coupled wall structure, the 'frame' action, that is: the axial forces in the walls resulting from the accumulated shear in the beams, is stiffer than the flexural response of the individual wall piers. As a result, the coupling beams exhibit greater ductility demands than do the wall piers. Figure-15 shows the idealized response of a coupled wall structure as the sum of the individual pier flexural responses and the 'frame' response resulting from the coupling action of the beams.

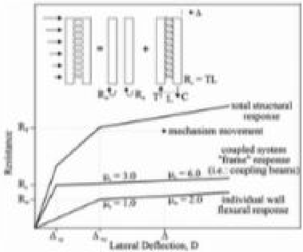


Figure 15. Idealized lateral response of CW structure- Harries 2001

Figure 16. Shear walls interconnected with horizontal flexible beams in a grid

Below is an experiment for the deflections of coupled and uncoupled shear walls due to lateral forces.

@seismicisolation

[27] A case study of displacement for Coupled & Uncoupled walls of "The Sail @ Marina Bay" tower of 245 meters high, 70 stories in Singapore has given below.

26.1. Displacement of Coupled & Uncoupled shear walls

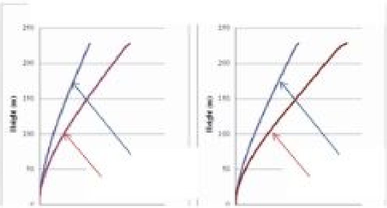


Figure 17. Lateral Displacements

26.2. Worked Example of Coupled Shear Walls

"PSW" concept is directly related to the mechanism of "Coupled Shear Wall Structure". Therefore to understand the "PSW" concept, it is required to know,

1) The forces/moments develop to the members (Beams & Shear Walls) due to lateral forces.

2) The stiffnesses, modulus of elasticities of the shear walls and the beams because "drift" of the structure depends on these factors.

In coupled shear walls, a number of shear walls are interconnected with the horizontal flexible beams. These beams are called coupling beams. When the walls deflect under the action of the lateral loads, the bending action induces shears in the connecting beams, which exerts bending moments, of opposite sense to the applied external moments on each walls. 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 M1 and M2 in two walls at that level, and the moment of the axial forces *Nl*, where N is the axial force in each wall at that level and *l* is the distance between their centroidal axes.

The analysis allows an assessment of how much of the applied moment is resisted by bending moments in the walls, and how much by axial forces (which produced by horizontal beam).

A typical system of plane coupled shear walls shown in Fig. 18 is considered. It is assumed that the system forms a coupled shear walls in a 50-story building, and is subjected to a uniformly distributed lateral load of intensity 16.5 KN/m.

It is assumed that,

Wind load on wall $w = 16.5 \times \frac{KN}{m}$ Story height, h = 3.5m

Poisson's Ratio v = 0.17

Young modusul of elasticity $E = 4.287 \times 10^7 \frac{KN}{m2}$

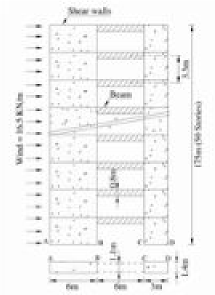


Figure 18. Coupled Shear Walls

Note: For simplicity, deflection due to shear is ignored because it contributes a very small percentage of total magnitude.

	Wall-1	Wall-2	Beam
Depth	h1 = 6m	h2 = 3m	h_d = 0.8m
Thickness/Width	b1 = 1.4m	b2 = 1.4m	b_b = 1.1m
Height/Span	H3 = 175m	H3 = 175m	bf = 6m

@seismicisolation

Wall properties:

Cross sectional area Wall-1, $A1 = h1 \times b1 = 8.4m^2$

Cross sectional area Wall-2, $A2 = h2 \times b2 = 4.2m^2$

Total area of W1 & W2, $Aa = A1+A2 = 12.6m^2$

Moment of inertia of Wall-1, $I_{w1} = \frac{(b1 \times h1^3)}{12}$ $I_{w1} = 25.2m^4$

Moment of inertia of Wall-2, $I_{w2} = \frac{(b2 \times h2^3)}{12}$ $I_{w2} = 3.15m^4$

Summation of moment of inertia of Wall-1 & Wall-2

$I = I_{w1} + I_{w2} = 28.35m^4$

Shear walls c/c distance, $sd = \left[\frac{(h1+h2)}{2}\right] + bl = 10.5m$

Beam properties:

Beam cross sectional area, $A3 = b_b \times h_d = 0.88m^2$

Consider beam entire cross section effective

Moment of inertia of beam, $I_b = \left[\frac{b_b(h_d)^3}{12}\right] = 0.047m^4$

Shear modulus, $G1 = \frac{E}{2X(1+\nu)} = 1.797 \times 10^{10} Pa$

Cross sectional shape factor of beam, $\lambda = 1.2$

[$\lambda = 1.2$, considered for the simplicity of the calculation. Because it has negligible effect on the drift. But the correction is required in the case of connecting beams with a span-to-depth ratio less than 5]

$r = \left[\frac{12 \times E \times I_b}{(bl)2 \times G1 \times A3}\right] \times \lambda = 0.0499$ Eq. (10.6)[5]

Effective second moment of area, $Ic = \frac{I_b}{1+r} = 0.045m^4$

Taking account of the wall-beam flexibility, effective length

le = true length (*bl*) +1/2 beam depth (h_d)

Beam effective length, $le = bl + \left(\frac{h_d}{2}\right) = 6.4m$

Determine the structural parameters k, α and kα H from Eq. (10.14)[5]

$k = \sqrt{\frac{Aa \times I}{A1 \times A2 \times sd^2}} = 1.045,$ sd = c/c of shear walls

$\alpha = \sqrt{\frac{12 \times Ic \times sd^2}{le^3 \times h \times I}} = 0.048 \times \frac{1}{m}$ $k \times \alpha \times H3 = 8.719$

Suppose that K_1 is the percentage of the total wind moment that is resisted by independent cantilever action, and that K_2 is the percentage resisted by composite cantilever action, Considering the reaction at base, therefore value of $z = 0.0m$

Composite cantilever factor

$$K_2 = \frac{200}{(k\alpha H3)\times\left(1-\frac{z}{H3}\right)^2}\times[1+(\sinh(k\alpha H3)-k\alpha H3)\sinh k\alpha[H3-z]]$$

$$-\cosh[k\alpha(H3-z)+(1/2)(k\alpha H3)^2\left(1-\frac{z}{H3}\right)^2]$$

$K_2 = 79.692$ Eq. 10.55[5]

Individual cantilever factor,

$K_1 = 100 - K_2 = 20.308$ Eq. 10.56[1]

Total moment at base,

$$Mt = \left(\frac{H3}{2}\right)\times w\times H3 = 2.52\times10^5 KN-m$$

Portion of base moment due effectively to individual cantilever action is,

$$Mi = \left(\frac{K_1}{100}\right)\times Mt = 5.131\times10^4 KN-m$$

Moment on wall 1, $M1 = \left(\frac{I_{w1}}{I}\right)\times Mi = 4.561\times10^4 KN-m$

Moment on wall 2, $M2 = \left(\frac{I_{w2}}{I}\right)\times Mi = 5.701\times10^3 KN-m$

Portion of base moment due effectively to composite cantilever action is

$$Mc = \left(\frac{K_2}{100}\right)\times Mt = 2.013\times10^5 KN-m$$

Axial force factor F1,

$$F1 = \left(\frac{1}{2}\right)\left(1-\frac{z}{H3}\right)^2+\left[\frac{1}{(k\alpha H3)^2}\right]\left[\frac{\cosh(k\alpha z)+k\alpha H3\times\sinh[k\alpha(H3-z)]}{\cosh(k\alpha H3)}\right]$$ Eq. 25/26[1]

F1 = 0.398

$$N1 = w\left(\frac{H3^2}{k^2\times sd}\right)F1 = 1.756\times10^4 KN$$ ..Eq. 10.26[5]

Resisting moment of the axial forces

$$N1\times sd = 1.844\times10^5 KN-m$$

From Eq. 10.26[5] it is observed that value of N1 will decrease when span of beam increases. At the same time resisting moment of the axial forces (N1 x sd) will decrease.

Effective composite second moment of area of cross section = Ig

$$I_g = I_{w1}+I_{w2}+\left(\frac{A1\times A2}{Aa}\right)\times(sd)^2 = 337.05m^4$$. Eq. 10.48[5]

Distance from point D (Fig. 18) to the center of gravity of the cross section,

@seismicisolation

$$x_1 = \frac{[1.5m \times A2 + (12m \times A1)]}{Aa} = 8.5m$$

Distance from point A to the center of gravity of the cross section

$x_2 = 15m - x_1 = 6.5m$

Using ordinary beam theory, the stresses at the salient points A, B, C and D are, on adding the stresses due to individual and composite cantilever stresses taking tensile stresses as negative.

$$\sigma_A = -\left(\frac{M1 \times 3m}{I_{w1}}\right) - \left(\frac{Mc \times 6.5m}{I_g}\right) \qquad \sigma_A = -9.313 \times 10^3 \frac{KN}{m^2}$$

$$\sigma_B = \left(\frac{M1 \times 3m}{I_{w1}}\right) - \left(\frac{Mc \times 0.5m}{I_g}\right) \qquad \sigma_B = 5.131 \times 10^3 \frac{KN}{m^2}$$

$$\sigma_C = -\left(\frac{M2 \times 1.5m}{I_{w2}}\right) + \left(\frac{Mc \times 5.5m}{I_g}\right) \qquad \sigma_C = 570.818 \frac{KN}{m^2}$$

$$\sigma_D = \left(\frac{2 \times 1.5m}{I_{w2}}\right) + \left(\frac{Mc \times 8.5m}{I_g}\right) \qquad \sigma_D = 7.793 \times 10^3 \frac{KN}{m^2}$$

If the walls are connected by pin-ended link beam, the external moment "Mt" will be resisted by individual moments in the two walls, the magnitudes of which will be proportional the walls' flexural rigidities, and the corresponding base stresses would be as follows:

$$\sigma_{A1} = \frac{Mt \times \left(\frac{I_{w1}}{I}\right) \times 3m}{I_{w1}} \qquad \sigma_{A1} = 2.674 \times 10^4 \frac{KN}{m^2} = \sigma_{B1}$$

$$\sigma_{C1} = \frac{Mt \times \left(\frac{I_{w2}}{I}\right) \times 1.5m}{I_{w2}} \qquad \sigma_{C1} = 1.337 \times 10^4 \frac{KN}{m^2} = \sigma_{D1}$$

$$F3 = 1 - \left(\frac{1}{k^2}\right) \times \left[\left(1 - \frac{4}{(k\alpha H3)^2}\right) + \left(\frac{8}{(k\alpha H3)^4 \times \cosh(k\alpha H3)}\right) \times (1 + k\alpha H3 \times \sinh(k\alpha H3) - \cosh(k\alpha H3))\right]$$

F3 = 0.12251 .... Eq. 10.34[5]

It is seen from the above equations that the depth of the beam is one of the factors that influence the drift of the structure, if depth increases the value of "F3" will decrease, when F3 decreases drift of the structure will reduce.

Maximum top deflection becomes,

$$y_{max} = \left(\frac{w \times (H3)^4}{8 \times E \times I}\right) \times F3 = 185.597mm$$

With no coupling beams, maximum top deflection would have been

$$y_{maxs} = \left(\frac{w \times (H3)^4}{8 \times E \times I}\right) = 1592mm$$

If these two shear walls are connected by pin-ended links that transmit only axial forces between them, and lateral force applied to the structure, moments created to the shear walls will be resisted by individual moments in the two walls and the maximum deflection at top will be 1592 mm. When the shear walls are interconnected by horizontal beam of the given sizes and length, the maximum deflection at top is 195 mm. When several shear walls are connected by horizontal beams in a grid, the composite action of the coupled shear walls will reduce a large amount of drift of the structure.

It is observed that when the two shear walls are connected with a beam than the deflection at top the shear walls are much less than the deflection of the shear walls without beam.

Author's innovative concept on structural arrangement took the advantage of this system to reduce the drift of buildings. All the shear walls in each directions connected by beams. Shear walls itself are very stiff to resist the lateral forces. And when these walls are connected by beams, become stiffer due to the composite action of shear walls and beam, thereby can reduce a large amount of deflection. This Parallel Shear Wall (PSW) concept arranges huge number of shear walls and beam connections.

These shear wall arrangement (instead of columns) with beams and the composite action between shear walls & beams "concept" will allow the structure to reach a "record breaking height" for habitable floor.

27. Structural Form

The determination of the structural form of a high-rise building would ideally involve only the selection and arrangement of the major structural elements to resist most efficiently the various combinations of gravity and horizontal loading. In reality, however, the choice of structural form is usually strongly influenced by other than structural considerations. The range of factors that has to be taken into account in deciding the structural form includes the internal planning, the material and method of construction, the external architectural treatment, the planned location and routine of the service systems, the nature and magnitude of the horizontal loading, and the height and proportions of the building. The taller and more slender a building, the more important the structural factors become, and the more necessary it is to choose an appropriate structural form.

Buildings of up to 10 stories designed for gravity loading can usually accommodate wind loading without any increase in member sizes, because of the typically allowed increase in permissible stresses in Design Codes for the combined loading. For buildings more than 100 stories, however, the additional material required for wind 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 viability, of the building.

In a residential building or hotel, accommodation is subdivided permanently and usually repetitively from floor to floor. Therefore, continuously vertical columns and walls can be distributed over the plan to form, or fit within, partitioning.

The principal objectives in choosing a building's structural form are to arrange to support the gravity, dead and live, loading, and to resist at all levels the external horizontal load shear, moment, and torque with adequate strength and stiffness. These requirements should be achieved, of course, as economically as possible.

With regard to horizontal loading, a high-rise building is essentially a vertical cantilever. This may comprise one or more individually acting vertical cantilevers, such as shear walls or cores, each bending about its own axis and acting in unison only through the horizontal in-plane rigidity of the floor slabs. Alternatively the cantilever may comprise a number of columns or walls that are mobilized to act compositely, to some degree, as the chords of a single massive cantilever, by vertically shear-resistant connections such as bracings or beams.

With the constraints of the selected structural form, advantage may be taken of locating the main vertical members on plan so that the dead load compressive stresses suppress the lateral load tensile stresses, thereby avoiding the possibility of net tension occurring in the vertical members and uplift on the foundation.

A major step forward in reinforced concrete high-rise structural form came with the introduction of shear walls for resisting horizontal loading. This was the first in a

@seismicisolation

series of significant developments in the structural forms of concrete high-rise buildings, freeing them from the previous 20 to 50 story height limitations of the rigid-frame and flat plate systems. The innovation and refinement of these new forms, together with the development of higher strength concretes, has allowed the height of concrete buildings to reach within striking distance of 100 stories.

The structural forms of the tall buildings, has concerned mainly the arrangement of the primary vertical components and their interconnections.

New developments of tall buildings of ever-growing heights have been continuously taking place worldwide. Consequently, many innovations in structural systems have emerged. This book presents main structural systems for tall buildings.

27.1. Shear Wall-Frame Interaction

This is the most popular system for resisting lateral loads. This system has been used for buildings as low as 10 stories to as high as 50 storey or even taller buildings. With the advent of haunch girders, the applicability of the system could be extended to buildings in the 70-80 storey range. The interaction of frame and shear walls has been understood for quite some time, the classical mode of the interaction between a prismatic shear wall and a moment frame is that the frame basically deflects in a so called shear mode while the shear wall predominantly responds by bending as a cantilever.

Compatibility of horizontal deflection introduces interaction between the two systems which tends to impose a reverse curvature in the deflection pattern of the system. The combines' structural action, therefore, depends on the relative rigidities of different elements used in the makeup of the lateral-load-resisting system. The distribution of total wind shear to the individual shear walls and frames as given by the simple interaction diagram is valid only if one of the following two conditions is satisfied.

1. Each shear wall and frame must have constant stiffness properties throughout height of the building.
2. If stiffness properties vary over the height, the relative stiffness of each wall and frame must remain unchanged throughout the height of the building.

@seismicisolation

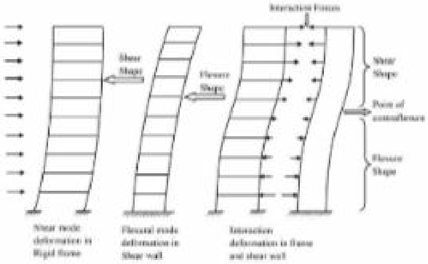


Figure 19. Shear Wall-Frame Interaction

27.2. Framed Tubed Structures

[1]For framed tube structures the lateral resistance is given by very stiff moment resisting frames that form a tube around the perimeter of the building. The frames consist of closely spaced columns, 2–4 meters between centers, connected by girders. The tube carries all the lateral loads and the self-weight is distributed between the outer tube and the interior columns or walls. For the lateral loading the perimeter frames aligned in the load direction acts as webs of the tube cantilever and those perpendicular to the load direction acts as flanges. The tube structure is suitable for both steel and reinforced concrete buildings and have been used in the range of 40–100 stories. Framed tube systems have been the most significant modern development in high-rise structural forms and is easily constructed and usable for great heights. For the aesthetics of the tube structure the enthusiasm is mixed, some like the logic of the clearly expressed structure while others criticise the grid-like façade as small windowed and repetitious. A disadvantage with the tube structure is the efficiency for the flange frames, for lateral loading, which tend to suff from shear lag with the result that the mid columns are less stressed than the corner columns and therefore not contributing as much as they could.

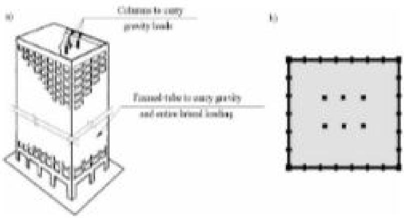


Figure 20. Framed Tube Str uctu re

27.3. Tube-in-Tube

[1] What differentiates the "tube in tube" concept from other structural systems is that an outer framed tube (hull), is working together with an internal tube (core), usually elevator shafts and stairs, to resist both the lateral and vertical loading, see Fig. 21 This provides increased lateral stiffness and can be seen as the shear and flexural components of a wall- frame structure.

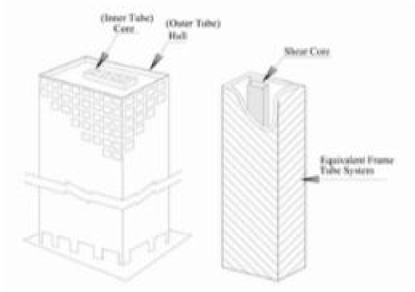


Figure 21. Tube in Tube

27.4. Bundled Tube

The bundled tube structure consists of four parallel rigid frames in each orthogonal direction, interconnected to form bundled tubes, see Fig. 22. The principle is the same as for the single tube structure where the frames are in the horizontal load direction acts as webs and the perpendicular frames acts as flanges. By introducing the internal webs the shear lag is drastically reduced and as a result the stresses in the columns are more evenly distributed and their contribution to the lateral stiffness is more significant. This allows for the columns to be spaced further apart and to be less striking.

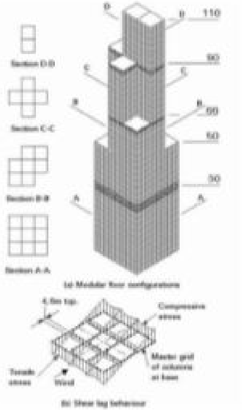


Figure 22. Bundled Tube

27.5. Outrigger system

[1] The outrigger system is an efficient structural form that consists of a central core with outriggers, connecting the core to the outer columns. The central core contains of either braced frames or shear walls. When the building is loaded laterally the vertical plane rotations are resisted by the outriggers through tension in the windward columns and compression in the leeward columns, see Fig. 23. This is augmenting the lateral stiffness of the building and reducing the lateral defections as well as the moments in the core. In addition, the outriggers join the columns and make the building behave almost as a composite cantilever. Even the perimeter columns, those not directly connected to the outriggers, can be used to increase the lateral resistance of the building by connecting all the perimeter columns with a horizontal girder around the building's facade. Multilevel outrigger systems can provide up to five times the moment resistance of a single outrigger

system. Outrigger systems have been used for buildings up to 70 stories but the concept should hold for even higher buildings.

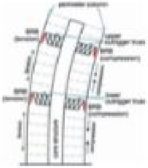


Figure 23. Outrigger System

27.6. Buttressed Core System

[2]The buttressed core is a different species. Permitting a dramatic increase in height, its design employs conventional materials and construction techniques and was not precipitated by a change in materials or construction technology. The essence of the system is a tripod-shaped structure in which a strong central core anchors three building wings. It is an inherently stable system in that each wing is buttressed by the other two. The central core provides the torsional resistance for the building, while the wings provide the shear resistance and increased moment of inertia (Fig. 24). The buttressed core represents a conceptual change in structural design whose evolutionary development began with Tower Palace III, designed by Chicago-based Skidmore, Owings & Merrill LLP (SOM).

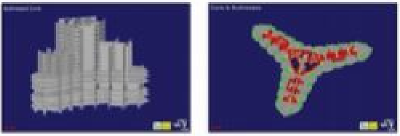


Figure 24. Buttressed Core

@seismicisolation

28. Parallel Shear Wall (PSW) Concept

28.1. Free body diagram of Coupled Shear Walls

This efficient structural form consists of several shear walls, which are interconnected with beams. The primary concept of "PSW" is based on the concept of "Coupled Shear Wall Structure". So, for PSW concept, it is required to understand the structural behavior of Coupled Shear Walls (Fig. 25). When the walls deflect under the action of lateral loads (26.2 Worked example), the connecting beam ends are forced to rotate and displace vertically, so that the beam bends in double curvature and thus resist the free bending of the walls. The bending action induces shears in the connecting beam, which exerts bending moments, of opposite sense to the applied external moments, on each wall. The shears also induces axial forces (N) in the two walls, tensile in the windward wall and compressive in the leeward wall. The term *Nl* represents the reverse moment caused by the bending of the connecting beam which opposes the free bending of the individual walls. The term *Nl* represents the reverse moment caused by the bending of the connecting beam which opposes the free bending of the individual walls. The action of the connecting beam 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.

As the beam increases in stiffness the induced axial forces in the walls increase, increasing the component of uniform tensile of compressive stresses in the walls, and reducing the wall bending moments, and hence the component of bending stress in each wall.

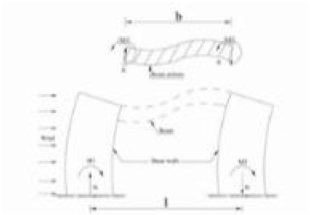


Figure 25. Behavior of laterally loaded coupled shear walls

28.2. Several shear walls forming a grid

For a given set of walls in a grid (Fig. 26), with fixed dimensions, the value of the stiffness parameter kαH (Eq. 10.14) will be a measure of the stiffness of the connecting beams, and it will increase if either effective second moment of area of the connecting beam Ic is increased or the clear span of beam "b" is decreased.

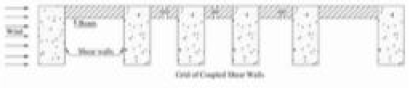


Figure 26. Grid of several shear walls connected with beams

The structural arrangement of "PSW" concept will be in such a way that at the base, there will be several parallel grids (Fig. 26), say 6 meters apart in each direction (X & Y). The number of grids (Fig. 27) in each direction will depend on the length, breath & height of the structure. Preferred grid spacing 6-8 meters. Each grid will consist of several shear walls which will be interconnected by beams.

"PSW" concept shows that each grid have several numbers of "beam (ends)" and "shear walls", which all are lateral load resisting elements as well as carrying the gravity loads. Each floor have multiple number of grids in both directions. When the number of lateral force resisting elements of a grid are multiplied by number of grids in each direction, it will be seen that a large amount of lateral resisting force will produce in a floor when applied lateral forces to the structure.

The effect of the shear-resistant connecting members (beams) is to cause the set of walls to behave as a composite cantilever, bending about the common centroidal axis of the walls. This results in a horizontal stiffness very much greater than if the walls acted as a set of separate uncoupled cantilevers.

In each grid, shear walls near the perimeter undergo maximum stresses (Fig. 28) due to wind force and the stresses linearly decrease towards the middle, i.e. all the shear walls in a grid taking part to resist the wind force. The grids in each directions, X or Y of a building will resist the wind forces. If the grids are equally spaced than the amount of applied wind forces on each middle grid will be same and the side grids will share half of the amount of middle grids.

@seismicisolation

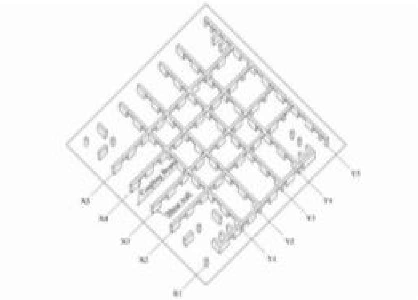


Figure 27. Arrangements of Shear Walls and Beams in Grids

So the axial deformation of all the walls in any raw will be nearly same. Therefore "shear lag effect'' will be minimized.

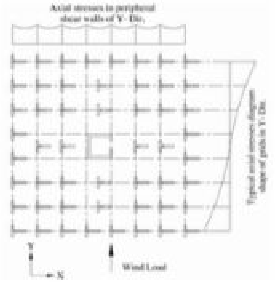


Figure 28. Shear lag in PSW structure.

28.3. Minimizing shear lag effect

For very tall buildings, the shear lag experienced by conventional framed Tubes may be greatly reduced by introducing the structural arrangement of PSW concept. When

@seismicisolation

the building is subjected to bending under the action of lateral forces, the high-in-plan rigidity of the floor slabs constrains all the grids parallel to the wind forces to deflect equally and the shears carried by each grid will be proportional to their stiffness. Consequently the presence of nearly equally spaced grids reduces substantially the non-uniformity of column forces caused by shear lag.

[25]When a frame tube building is subjected to lateral loading (Fig. 29), it is resisted by web side panels, which deform such that the columns A and B are in tension and columns C and D are in compression.

The principle interaction between the web and the flange would be through the vertical displacement of the corner columns. These displacements correspond to the vertical shear in the girder of the flange frame, which mobilizes the column forces in the flange column. For example if the column C is in compression it will try to compress the adjacent column since the two are connected by spandrel beam, the compressive deformation will not be identical since the spandrel beam will bend. Thus the axial deformation in adjacent columns will be less (Fig. 30), by an amount equal to the stiffness of the connecting beam.

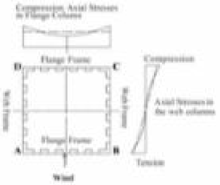


Figure 29. Axial stress distribution in columns of laterally loaded framed tube

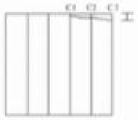


Figure 30. Deformation of the flange frame causing shear

Hypothetically, beams of infinite stiffness will develop a pure tubular action. Thus the deformations in successive adjacent columns in frame will be lesser than the previous one. Since the external applied moment is to be resisted by internal couple produced by the compressive and the tension stress produced on either side of neutral axis of structure, it follows that the stresses in the corner column will be greater than those in the interior column. This phenomenon is called the shear lag effect. Shear lag may lead to wrapping of floor slabs, local bulking on compression side & cracking on tension side.

29. PSW Concept's responses on Mega Tall Buildings

Tall buildings dominate the lay-out for the structural system. As with all structures, vertical elements need to accommodate the worst combination of gravity and lateral loads. Although the gravity loads will be large - particularly in very tall buildings - wind and seismic loads, acting on what is essentially a large vertical cantilever, dominate assessment of structural sizes for the preferred lateral framing system and will necessarily inform the architectural lay-out and special arrangement.

Still it is an imagination for the tall buildings that habitable floor will be at a height of 1000 meters. Any additional lateral supporting system can work efficiently up to a height (habitable floor) around 600 meters. Beyond this height acceleration is the main issue for human comfort in tall buildings. So it is required to invent a new structural system which can reach the habitable floor at a higher level. To do this,

1) Lateral forces should be controlled/adjusted.

2) Stiffness of tall buildings should be increased.

Below section describes how to overcome these two factors by using PSW concept.

29.1. Lateral forces should be controlled/adjusted:

In the "Tube" system wind forces resist by two side panels (Webs), in "Tube-in-Tube" system, resist two side panels and a inner core and in "Buttressed Core" system wind first resists by two opposite wings. So it is seen that lateral wind forces are mainly concentrating on two to three areas in tall buildings.

These concentration of forces make huge amount of stresses to the members therefore cannot be able to scale the system to achieve a much taller building. But if full width/face (as the grids of PSW concept, Fig. 27) of a building can involve to resist the wind forces than the stresses will be distributed to the members along the full width of building. This can be achieved by making parallel and nearly equal grid spacing. Each grid will contain several shear walls connected by beams which will resist the lateral forces. So the intensity of wind pressure will not concentrate on few places to increase the bending tendency of the structure.

29.2. Stiffness of tall buildings should be increased:

The drift of a building due to lateral loads depends on the stiffness of buildings. Here describes the technique to increase the stiffness, let a tall building resists the wind forces by inner or outer core (Tube system) or both (Tube-in-Tube system) and columns are placed to carry the gravity loads. Consider two columns and assume the sizes 1.5m x 1.5m. The cross sectional area of each column is 1.5m x1.5m = $2.25m^2$ and the total moment of inertia of these two columns in any major axis is $2x\{1.5mx(1.5m)^3/12\} = 0.844\ m^4$. To increase the stiffness of the building, reshape each column (shear wall type) to 0.6m x 3.75m (keeping the cross sectional area same, $2.25m^2$) and place one column perpendicular to other. The moment of inertia of each column to their major axis (neglecting the minor axis inertia force) is $0.6m \times (3.75m)^3/12 = 2.637\ m^4$. The stiffness increased in any major axis by an amount of $(2.637-0.844) = 1.793\ m^4$ which is more than two times of first arrangement. Therefore if all the gravitational columns are converted to shear wall types than a building will gain the stiffness, more than two times of the previous stiffness. Furthermore, if the reshaped columns/shear walls are connected by beams to get composite actions, will further reduce the drift. In PSW concept, maximum vertical members are replaced by shear walls to make the structure stiffer in both directions of the building.

29.3. PSW - Shear Wall locations

For simplicity, let us consider a skyscraper of height 700 meters and the base is 77m x 77m. Consider the shear wall spacing is 7 meters c/c. Therefore 12 number of shear walls are required in each row/column. As this building is assumed square than around 50% of the shear wall will be arranged in x-direction and 50% in y-direction as shown in (Fig. 31). To achieve the composite action, a combination of at least 2 or more shear walls will be arranged alternatively in x and y direction. If the building's lay-out i.e. ratio of breath and width is 2:1 than arrange the shear walls in such a way that around 67% of all the shear walls will be in the short direction.

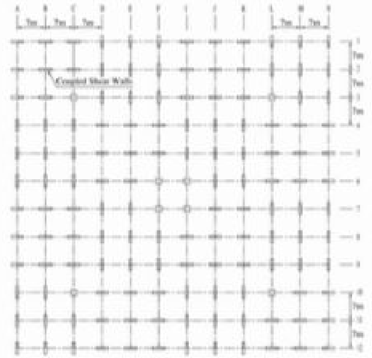


Figure 31. Typical arrangements of shear Walls

and the rest 33% will be in the long direction. As a thumb rule, direction and number of shear walls will depend on the breath and width of a building. Thicker shear walls will be placed on the outer perimeter and gradually decrease the thicknesses of the walls toward the center. Sometimes it will not be possible in a grid to provide all through the shear walls due to architectural problems. In that issue provide columns instead of shear walls which will work as to bear the gravity loads. That grid will work as a couple of 'coupled shear walls". But beams should be continued all through the grid to transfer the horizontal loads to the "coupled shear walls". As an example, in grid 5, there are two composite units, each consist of three shear walls. One is from grid "D to F" & the other one is from grid "L to N". Again in grid 6, there are two composite units, one is from "D to E" which consist of two shear walls & the other one is from "L to N" which consists of three shear walls.

To work each grid as a one unit, all the vertical members of a grid have to be connected by horizontal beams. Although the two coupled shear walls (two composite units) in the grid, say grid 5, are the strongest part to resist the lateral forces along the grid 5.

30. PSW - Primary Structural Arrangement

PSW concept is characterized by its symmetry. There are no transfers of vertical elements through the main body of the tower. It allows a uniform distribution of gravity forces through the structure. These characteristics allow for a more efficient structure.

There is no separation between the gravity system and the lateral system. The vertical structure is organized in such a way that the elements are all sized on sufficient lateral stiffness while at the same time providing strength consideration. This creates an extremely efficient structure where the materials perform double-duty (gravity and lateral support). This structure creates a uniform distribution of load reducing the differential shortening.

In PSW concept, the shear walls may serve both architecturally as partition and structurally to carry gravity and lateral loading. The arrangements of the shear wall are such a way that the lateral load tensile stresses are suppressed by the gravity load stresses. This allows them to be designed to have only the minimum reinforcement. Shear wall structure perform well in earthquakes, for which case ductility becomes an important consideration in their design.

30.1. PSW Concept applied to One Kilometer Tall Tower

The innovation of this new concept "parallel shear walls (PSW)", together with the development of higher strength concrete, has allowed the height of concrete buildings to reach within striking distance of 264 stories.

This is a theoretical study so the author's intension is to use the top of the structure as a habitable floor maximizing the total floor area to increase high value lease spaces.

The PSW concept applied to a 1003.2 meters tall structure, consists of five vertical portions (Fig. 32), which covers 264 stories, taller than the existing tallest building in the world. To gain an adequate footprint for stability, this tower extends to nearly 106m x 106m at base, resulting in the 9.4 : 1 aspect ratio (The ratio of the height of the building to its smaller width at the base), a ratio greater than the one hold by existing tallest building in the world, which is close to 9 : 1[11] .

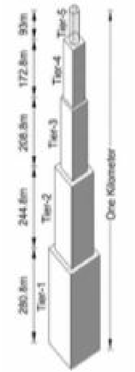


Figure 32. Tier heights

The building has 5 Tiers of different heights (Fig. 32). Structural arrangements of vertical members (Shear Walls & Columns) of different tiers are shown in Figs. 34 to 37. Beams are not shown for clearly understand the vertical arrangement of the structure. Columns will carry the vertical loads of the structure. A typical structural arr angement which consists of several parallel shear walls connected by beams in each direction of the building as shown in Fig. 33. This produces a completely interconnected structural system. The shear walls arrangement is, in such a way that they provide large amount of inertia forces and stiffness to the structure. When wind forces are applied to the structure (Fig. 39), the forces will be distributed almost equally to the 12 grids for this structural arrangement. Having multiple number of beams in each grid, a large amount of lateral forces due to wind will be resisted by these connected beams and it is seen from the analysis that for maximum floor, the beam depth required 0.8m to satisfy analysis and design criteria of this structure.

30.2 Building's Data

Building's Data	
Height-GroundFloor to Roof	1003.2m (3290 feet)
Number of Stories	264
Building uses (Assumed)	Hotel, Office and Residential.
Frame Materials	Concrete Structure
Typical Floor Live Load	3kn/m2 (60 psf).
Basic wind velocity considered (100 years returned period)	44 m/sec, (160km/hour). Multiply the Gust effect factor with the wind speed to get dynamic response.
Allowable Sway (Drift) [9]-(Commentary Appendix C-Sec: CC.1.2 ASCE 7-10)	H/500 (H = Height of the Structure)
Allowable Sway (Drift) at top	1003.2m/500 = 2006 mm (6'-7")
Sway (Drift) of the Structure at top for dynamic response	1828 mm (6' - 4")
Type of structure	Arrangements of Concrete Shear Walls and Beams for Tier-1, Tier-2, Tier-3 and Tier-4. Tier-5 is the frame structure.
Foundation Type	Future Assignment
Typical Floor height	3.8 m
Floor type	R.C.C. Slab
Shear Wall spacing	12m, 9m & 6m c/c
Core area	Column-Beam framing
Shear Wall thickness at ground floor	1.6m, 1.5m, 1.3m & 1.25m, gradually decreasing the thicknesses toward top.
Typical Beam sizes	Depth 0.8 m, Width 1.1m & 1.2 m
Column spacing	6m (20 feet) c/c
Column sizes at base	1.5m x 1.5m
Covered area at base by ShearWalls & Columns	13.45%
Concrete Strength	Shear Walls & Columns 80MPa, Beams & Slab 40MPa
Aspect Ratio (Height/Least length at Base)	9.4 : 1

Table 6. Building's Data

30.3. Shape & sizes of One Kilometer Tower

The structural model of 1003.2m tall tower (Fig. 32) has mainly composed of reinforced concrete shear walls, columns, beams and solid slabs.

The building has 5 Tiers of different heights. Table 7, describes the floor area and the different Tier levels.

@seismicisolation

One Kilometer Tower's Dimensions			
Tiers	Tier Heights (meters)	Elevations (meters)	Floor Areas (meters)
Tier-1	296.4m	0m to 296.4m	100m x 100m
Tier-2	258.4m	269.4m to 554.8m	80m x 80m
Tier-3	220.4m	554.8m to 775.2m	54m x 54m
Tier-4	182.4m	775.2m to 957.6m	34m x 34m
Tier-5	45.6m	957.6m to 1003.2m	12m x 12m

Table 7. One Kilometer Tall Tower's shape and sizes

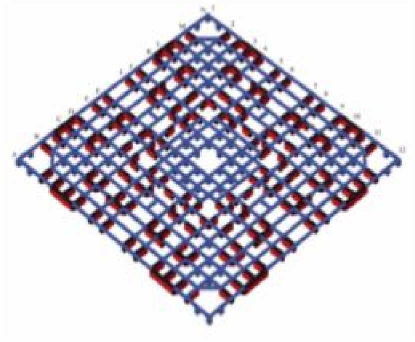


Figure 33. Typical Floor's Structural Arrangement of Tier-1

However, it is found that few beams requires 1.3m (depth) x 1.4m (width) near the Tier-1. Whereas in Tube system, wind forces concentrate on two side panel, therefore requires closed spaced columns to get Tube action.

Shear wall thicknesses:

Tier-1, at base 1.6m thick, gradually reducing towards top and at top 1.2m.

Tier-2, at base 1.5m thick, gradually reducing towards top and at top 1.1m.

Tier-3, at base 1.3m thick, gradually reducing towards top and at top 1.0m.

Tier-4, at base 1.25m thick, gradually reducing towards top and at top 0.8m.

5 typical floors of 5 Tiers are shown below. Vertical structural arrangements of each Tier is shown on their typical floor. All the shear walls and columns are connected by beams (Fig. 33). To show the vertical structural arrangement clearly, beams are not shown.

Each Tier has number of grids in each direction (A & 1). These grids are consist of several number of shear walls, connected by beams.

As an example, typical structural arrangement of a single floor of Tier-1 has been described below.

In Tier-1 (Fig. 34), there are 12 grids in building's main two directions to resist the wind forces. Grid-1 has two 'long' shear walls along the grid. Grid-2 has 6 number of shear walls along the grid. Between grids 2 & 3, there is a grid, which has only two shear walls, due to two number of shear walls at long distance apart, composite action of these two shear walls is not possible, cantilever action of the shear walls will dominate the deflection. So this grid is very weak compared to others, therefore it is not named as a grid. Grid-3 has eight shear walls. Grid-4 has nine shear walls. Grid 5 has six shear walls. Grid 6 has eight shear walls. Grid '7 to 12' is the mirror of Grids '1 to 6'. Each grid's stiffness/strength depends on number of shear wall, their c/c spacing, thickness and the connecting beam's depth and wind forces will be distributed among the grids according to their siffness/strength. For this structure, arrangement of grids are required for Tiers 1' to 4', and the Tier-5 has only columns which will work only support the gravitational loads, and can be placed randomly, only depends on the the vertical load's positions. Columns are shown in the Tiers will support the gravitational forces.

@seismicisolation

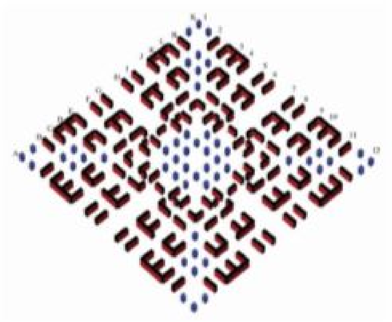


Figure 34. Vertical Structural Arrangement of typical floor of Tier-1

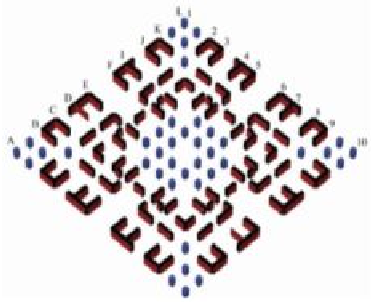


Figure 35. Vertical Structural Arrangement of typical floor of Tier-2

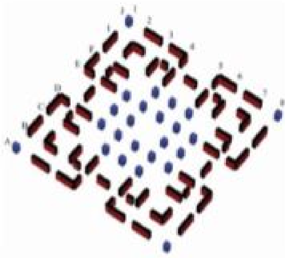


Figure 36. Vertical Structural Arrangement of typical floor of Tier-3

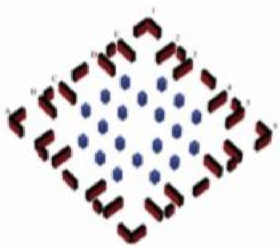


Figure 37. Vertical Structural Arrangement typical floor of Tier-4

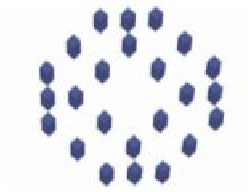


Figure 38. Vertical Structural Arrangement of typical floor of Tier-5

@seismicisolation

Each side of the building will resist the wind force by several parallel shear walls. The wind forces will be distributed to the structure almost uniformly to all grids due to proper shear wall placements (Fig. 39). Each Tier has its own core which starts from base. Several experiments show that Tiers with different heights (height of Tier-1 will be longest and gradually decrease towards top) give better results than the Tiers of uniform height. If "PSW" is applied to any mega tall building, the arrangement of several grids of parallel shear walls will make the different core for different Tiers. But the structural activities of the vertical elements will not like core system but the number of parallel grids to that Tier will work to resist the lateral forces.

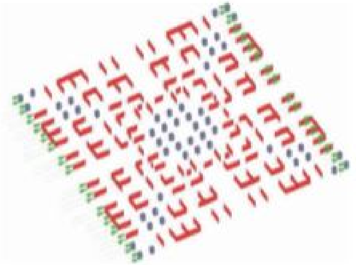


Figure 39. Wind force distribution

Various options have been studied through this parametric modeling method. The main goals were to maximize the lever arm of the shear walls, maintain the shear wall line in one single vertical plan, so as to minimize secondary forces. This shear wall arrangement can be said as an optimal balance and the most effective lateral load-resisting structure to stabilize this mega tall structure.

Base area of Tier-1 is maximum while base areas of other Tiers gradually decrease towards the top. The author has chosen and tuned the height increments towards bottom and gradually decreases towards top because the moments and the shears are high at the bottom. Tower design of any height with this "Parallel Shear Wall Concept", will have to choose and tune the different heights of the Tiers accordingly.

30.4. Grid actions for a group of shear walls

Consider the building which has several grids, consisting of coupled shear walls and is subjected to a horizontal loading, for example, wind or earthquake loading. Let it be assumed (Fig. 40) that it is possible to split up the building into ''stations'' by taking imaginary vertical cuts through the floor system along lines parallel to the direction of the applied loading such that each ''station'' then consists of one grid.

Each station has its own value of stiffness according to their number of shear walls, depth of connecting beams, few more factors and the width of the flooring system, which is effective in the interaction bending of the shear walls and floors at the station. Useful information exists in many literatures on this point.

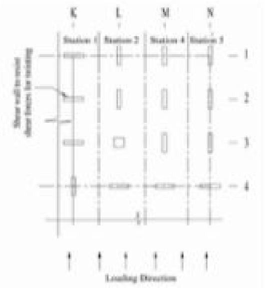


Figure 40. Stations of a structure of PSW

If the individual stations were uncoupled from one another, each would deflect in accordance with its own stiffness value and the external loading applied to it, and leaving out twisting effects. However, given sufficiently stiff flooring, a system of horizontal shears is set up in the floors along the "cut" lines, the effect of which is to compel all stations to deflect equally. The shear forces produced by the twisting tendency of the slab will further resisted by the shear walls which are perpendicular to these grids.

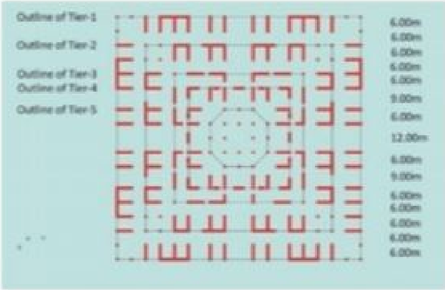


Figure 41. Typical shear wall spacings

Fig. 41 shows the placements of shear walls and columns at base of one kilometer tower. Small circles indicate the column's & bold lines indicate the shear walls. Fig. 41 also shows the spacing of shear walls.

Fig. 42 describes the five separate zones of shear walls & columns. Vertical structural arrangement of the 5 Tiers started from the base. Each zone extends up to their Tier number's height from the base. As an example, "vertical members of Tier-1 start from base and end at the height of Tier-1, and zones 2, 3, 4 & 5 extend up to their Tier numbers.

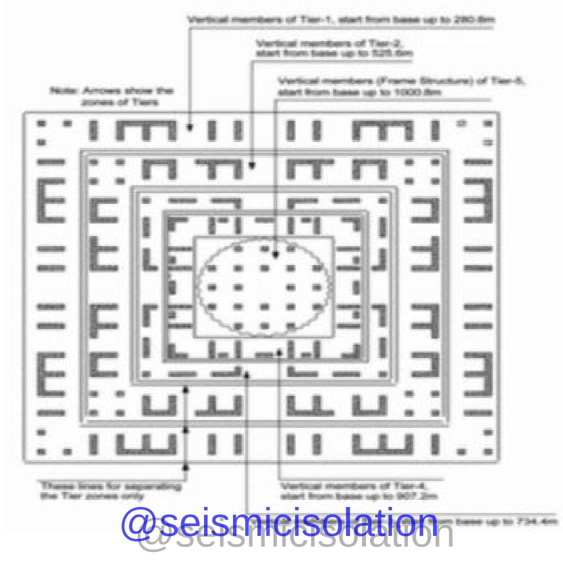


Figure 42. Shear Wall zones according to Tier numbers

31. Percentage of covered area by vertical members at base

Area covered by shear walls at ground floor = 1440.24 m2. Area covered

By columns at ground floor = 72 m2.

Gross area at ground floor = 106m x 106m = 11236 m2. Area covered by

Vertical elements (shear walls and columns) at ground floor,

[(1440.2+72)/11236] x 100 => 13.45%

It can be said that the percentage of covered area by vertical elements is very less for such a tall structure of one kilometer height.

Note: Percentage (usable area) of gross floor area with respect to vertical Elements (shear walls and columns) is one of the main efficiencies of structural arrangements.

The key structural solutions of this PSW concept are, shear wall arrangement & the composite actions between beams and shear walls. All the vertical elements will resist the lateral forces as well as gravity loads. For the One Kilometer Tall Tower, Tier-5 is the only frame structure composed of columns and beams

32. Result Analysis

Drift Limits in common usage for building design are in the order of 1/600 to 1/400 of the building height (ASCE). Generally, for tall buildings allowable drift is considered as H/500, which becomes 2001.6 mm for this 'One Kilometer Structure'.

Drift due to wind for dynamic response = 1828 mm.

33. Story Drifts

Drift (lateral deflection) of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control of

Steel Building Structures 1988 and Griff's 1993). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. West and Fisher (2003) contains recommendations for higher drift limits that have successfully been used in low-rise buildings with various cladding types. It also contains recommendations for buildings containing cranes. An absolute limit on story drift may also need to be imposed in light of evidence that damage to nonstructural partitions, cladding, and glazing may occur if the story drift exceeds about 10 mm (3/8 in) unless special detailing practices are made to tolerate movement (Freeman 1977 and Cooney and King 1988).

Result analysis of "Story Drift Limits" of One Kilometer Tall Tower with PSW concept are above 400 for maximum, except few stories which are from 208 to 216 stories.

The maximum drift value in 9.545 mm for 211th storey which is also allowable according to ASCE limit of 10 mm. Below tables show the storey drifts for first, middle and last few stories according to the analysis by STAAD software.

@seismicisolation

STORY	HEIGHT	LOAD	AVG. DISP(CM)		DRIFT(CM)		RATIO	STATUS
	(METE)		X	Z	X	Z		
BASE=	0.00						ALLOW. DRIFT = L / 360	
1	0.00	3	0.0000	0.0000	0.0000	0.0000	L /999999	PASS
2	3.60	3	0.0236	0.0000	0.0236	0.0000	L / 15236	PASS
3	7.20	3	0.0851	0.0001	0.0614	0.0001	L / 5860	PASS
4	10.80	3	0.1787	0.0002	0.0936	0.0001	L / 3846	PASS
5	14.40	3	0.3011	0.0003	0.1225	0.0001	L / 2939	PASS
6	18.00	3	0.4498	0.0003	0.1487	0.0001	L / 2421	PASS
7	21.60	3	0.6225	0.0004	0.1727	0.0001	L / 2084	PASS
8	25.20	3	0.8173	0.0005	0.1948	0.0001	L / 1848	PASS
9	28.80	3	1.0324	0.0007	0.2151	0.0001	L / 1674	PASS
10	32.40	3	1.2661	0.0008	0.2338	0.0001	L / 1540	PASS
11	36.00	3	1.5172	0.0009	0.2511	0.0002	L / 1434	PASS
12	39.60	3	1.7843	0.0011	0.2671	0.0002	L / 1347	PASS
13	43.20	3	2.0663	0.0013	0.2820	0.0002	L / 1276	PASS
14	46.80	3	2.3621	0.0015	0.2958	0.0002	L / 1217	PASS
15	50.40	3	2.6706	0.0017	0.3086	0.0002	L / 1166	PASS
16	54.00	3	2.9911	0.0019	0.3205	0.0002	L / 1123	PASS
17	57.60	3	3.3226	0.0021	0.3315	0.0002	L / 1086	PASS

Table 8a. Storey Drifts for 1 to 17 Stories

@seismicisolation

STORY	HEIGHT	LOAD	AVG. DISP(CM)		DRIFT(CM)		RATIO	STATUS
	(METE)		X	Z	X	Z		
BASE=	0.00						ALLOW. DRIFT = L / 360	
138	493.20	3	67.2762	0.1569	0.6079	0.0005	L / 592	PASS
139	496.80	3	67.8847	0.1563	0.6085	0.0006	L / 591	PASS
140	500.40	3	68.4940	0.1556	0.6093	0.0007	L / 591	PASS
141	504.00	3	69.1045	0.1549	0.6106	0.0007	L / 589	PASS
142	507.60	3	69.7168	0.1540	0.6123	0.0008	L / 588	PASS
143	511.20	3	70.3317	0.1531	0.6149	0.0010	L / 585	PASS
144	514.80	3	70.9506	0.1519	0.6189	0.0011	L / 581	PASS
145	518.40	3	71.5759	0.1505	0.6254	0.0014	L / 575	PASS
146	522.00	3	72.2121	0.1488	0.6361	0.0017	L / 566	PASS
147	525.60	3	72.8645	0.1465	0.6524	0.0023	L / 552	PASS
148	529.20	3	73.5946	0.1424	0.7301	0.0041	L / 493	PASS
149	532.80	3	74.3335	0.1382	0.7389	0.0043	L / 487	PASS
150	536.40	3	75.0954	0.1335	0.7619	0.0046	L / 472	PASS
151	540.00	3	75.8763	0.1285	0.7809	0.0050	L / 461	PASS
152	543.60	3	76.6731	0.1232	0.7968	0.0053	L / 452	PASS
153	547.20	3	77.4833	0.1176	0.8102	0.0056	L / 444	PASS
154	550.80	3	78.3046	0.1117	0.8213	0.0059	L / 438	PASS
155	554.40	3	79.1352	0.1054	0.8305	0.0062	L / 433	PASS

Table 8b. Storey Drifts for 138 to 155 Stories

248	889.20	3	157.5750	-0.7766	0.7101	0.0142 L /	507	PASS
249	892.80	3	158.2723	-0.7909	0.6973	0.0142 L /	516	PASS
250	896.40	3	158.9700	-0.8052	0.6977	0.0144 L /	516	PASS
251	900.00	3	159.6591	-0.8196	0.6891	0.0144 L /	522	PASS
252	903.60	3	160.3406	-0.8339	0.6815	0.0144 L /	528	PASS
253	907.20	3	161.0100	-0.8488	0.6694	0.0149 L /	538	PASS
254	910.80	3	161.6062	-0.8682	0.5961	0.0194 L /	604	PASS
255	914.40	3	162.2913	-0.8875	0.6852	0.0194 L /	525	PASS
256	918.00	3	162.9966	-0.9072	0.7053	0.0196 L /	510	PASS
257	921.60	3	163.7081	-0.9269	0.7115	0.0197 L /	506	PASS
258	925.20	3	164.4193	-0.9466	0.7111	0.0197 L /	506	PASS
259	928.80	3	165.1270	-0.9663	0.7077	0.0198 L /	509	PASS
260	932.40	3	165.8297	-0.9861	0.7028	0.0198 L /	512	PASS
261	936.00	3	166.5267	-1.0059	0.6969	0.0198 L /	516	PASS
262	939.60	3	167.2173	-1.0256	0.6907	0.0197 L /	521	PASS
263	943.20	3	167.9015	-1.0453	0.6841	0.0197 L /	526	PASS
264	946.80	3	168.5788	-1.0651	0.6773	0.0197 L /	531	PASS
265	950.40	3	169.2492	-1.0848	0.6704	0.0197 L /	537	PASS
266	954.00	3	169.9124	-1.1045	0.6632	0.0197 L /	543	PASS
267	957.60	3	170.5683	-1.1241	0.6559	0.0197 L /	549	PASS
268	961.20	3	171.2168	-1.1438	0.6485	0.0197 L /	555	PASS
269	964.80	3	171.8579	-1.1635	0.6411	0.0197 L /	561	PASS
270	968.40	3	172.4914	-1.1831	0.6335	0.0196 L /	568	PASS
271	972.00	3	173.1172	-1.2027	0.6258	0.0196 L /	575	PASS
272	975.60	3	173.7354	-1.2224	0.6182	0.0196 L /	582	PASS
273	979.20	3	174.3458	-1.2420	0.6104	0.0196 L /	590	PASS
274	982.80	3	174.9485	-1.2615	0.6027	0.0196 L /	597	PASS
275	986.40	3	175.5435	-1.2811	0.5950	0.0196 L /	605	PASS

BASE= 0.00 **ALLOW. DRIFT = L / 360**

276	990.00	3	176.1310	-1.3006	0.5874	0.0196 L /	613	PASS
277	993.60	3	176.7111	-1.3202	0.5801	0.0195 L /	620	PASS
278	997.20	3	177.2845	-1.3397	0.5735	0.0195 L /	628	PASS
279	1000.80	3	177.8527	-1.3592	0.5682	0.0195 L /	633	PASS

104419. FINISH

@seismicisolation

Table 8c. Storey Drifts for 248 to 279 Stories

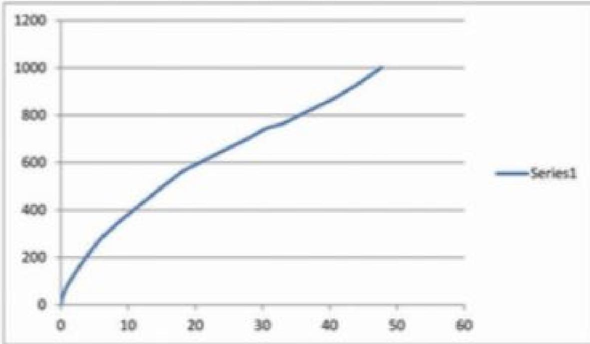


Figure 43. Acceleration to height relation of One Kilometer Tall Tower due to wind in milli-g (Horizontal values) and Height in meter (Vertical values)

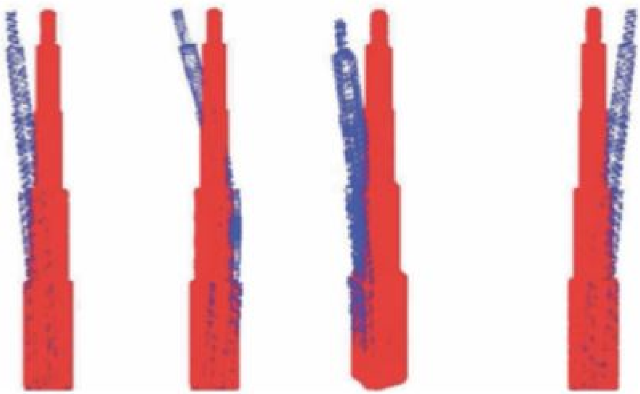


Figure 44. Four different mode shapes due to dynamic responses

Horizontal displacements are shown in large scale.

For Flexible, Tall and Mega Tall Buildings, Gust effect factor "G" is the main factor which can give dynamic responses of tall buildings when mean wind loading is multiplied by Gust Effect factor "G".

Below is a workout example of Gust Effect factor "G" for 380 meter tall Building.

34. Gust Effect factor for 100-Storey Building -"ASCE 7-10"

Mean roof height of the structure, h = 1246 ft.

Equivalent height of the structure (Section: 26.9.4)

$\overline{z} = 0.6h = 0.6 \times 1246 = 747.84ft.$

Consider exposer "B"

$\overline{\alpha} = \frac{1}{4}$ (mean hourly wind speed power law exponent - Table 26.9-1)

$\overline{b} = 0.45$ (mean hourly wind speed factor - Table 26.9-1)

c = 0.3 (Turbulence intensity factor in Eq. 26.9-7 - Table 26.9-1)

$\overline{\varepsilon} = \frac{1}{3}$ (Integral length scale factor law exponent - Table 26.9-1)

B = 144.3 ft. (Horizontal dimension of building measured parallel to wind direction in ft.)

(Average dimension of the 2 Tiers has been taken for simplicity of the calculation.)

L = 144.3 ft. (Horizontal dimension of building measured normal to wind direction in ft.)

l = 320 (Integral length scale factor from Table 26.9-1 in ft)

(Building Fundamental natural frequency no, Hz) Eq. 11

$$n_0 = \frac{100}{h} = 0.08023$$

β = 0.02 (Damping ratio, percent critical for building)

Basic wind speed considered

(Basic wind speed obtained, three second gust speen at 33 feet in mph)

V1 = 99 mph

(Intensity of turbulence at height z from Eq. 26.9-7)

$$I_z = c\left(\frac{33}{z}\right)^{\frac{1}{6}} \qquad Iz = 0.178$$

(Integral length scale of turbulence, in feet Eq. 26.9-9)

$$L_z = l\left(\frac{z}{33}\right)^{\varepsilon} \qquad L_z = 905.555$$

(Background response factor "Q" Eq. 26.9-8)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} \qquad Q = 0.74$$

(Mean hourly wind speed ft/s at height z_1, Eq. 26.9-16)

$$Vz = \overline{b}\left(\frac{z}{33}\right)^{\alpha} V1\left(\frac{88}{60}\right)$$

@seismicisolation

$V_z = 142.562\ ft/s$

$N_1 = \frac{n_0 L_z}{V_z}$ $\quad$ $N_1 = 0.51$ (Reduced frequency, Eq. 26.9-14)

$R_n = \frac{7.47\ N_1}{(1+10.3\ N_1)^{\frac{5}{3}}}$ $\quad$ $R_n = 0.18$ (Eq. 26.9-13)

$\eta_h = \frac{4.6\ n_0 h}{Vz}$ $\quad$ $Rh = \frac{1}{\eta_h} - \left(\frac{1}{2\times\eta_h{}^2}\right)^{(1-e^{-2\times\eta h})}$ (Section 26.9-15a)

$\eta_h = 3.227$ $\quad$ $R_h = 0.262$

$\eta_B = \frac{4.6 n_0 B}{Vz}$ $\quad$ $R_B = \frac{1}{\eta_B} - \left(\frac{1}{2\times\eta_B{}^2}\right)^{(1-e^{-2\times\eta_B})}$ (Eq. 26.9-15a)

$\eta_B = 0.374$ $\quad$ $R_B = 0.791$

$\eta_L = \frac{15.4 n_0 L}{Vz}$ $\quad$ $R_L = \frac{1}{\eta_L} - \left(\frac{1}{2\times\eta_L{}^2}\right)^{(1-e^{-2\times\eta_L})}$ (Eq. 26.9-15a)

$\eta_L = 1.25079$ $\quad$ $R_L = 0.506$

$R = \sqrt{\left(\frac{1}{\beta}\right) R_n R_h R_B (0.53 + 0.47 R_L)}$ $\quad$ $R = 1.195$ (Eq. 26.9-12)

$g_Q = 3.4$ $\quad$ $g_v = 3.4$ (Section 26.9.5)

$g_R = \sqrt{2ln(3600n_0)} + \frac{0.577}{\sqrt{2ln(3600n_0)}}$ $\quad$ $g_R = 3.538$ (Eq. 26.9-11)

Gust Effect Factor

$G_f = \frac{1+1.7\times I_z\sqrt{g_Q{}^2 Q^2 + R^2 g_R{}^2}}{1+1.7 g_v I_z}$ $\quad$ $G_f = 1.22707$ (Eq. 26.9-10)

35. Wind forces on 380 Meter Tall Building

The wind load calculations yields a static design pressure, which is expected to produce the same peak effect (Dynamic Response) as the actual turbulent wind, with due consideration for building properties such as height, width, natural frequency of vibration, and damping. This approach is primarily for determining the overall wind loading and response of tall slender structure.

Wind Loads, according to ASCE 7-10

Basic wind speed V_b = 99 mile/hour (160 KM/Hour) has been considered for this analysis.

@seismicisolation

V_b = 99 miles/hours = 44 m/s

Considered Exposer Category "B" -----ASCE 7-10, Chapter 26

Kz = Velocity pressure exposure co-efficient at height z.

Section 27.3.1, Table 27.3-1, ASCE 7-10

$$K_z = 2.01 \times \left(\frac{Z}{Z_g}\right)^{\frac{2}{\alpha}}$$

$\alpha = 7$, $z_g = 1200ft$ (α & z_g are tabulated in Table 26.9.1-Exposer "B")

Height considered, every after 50 meters for calculating wind forces to those heights.

1) From 0m to 10m, z_10 = 10m
2) From 10 to 70m, z_70 = 70m
3) From 70 to 120m, z_120 = 120m
4) From 120 to 170m, z_170 = 170m
5) From 170 to 220m, z_220 = 220m
6) From 220 to 300m, z_300 = 300m
7) From 300 to 380m, z_380 = 380m

Values of velocity pressure co-efficient "K_z" at different heights. Apply the above values (heights) to K_Z.

$$Kz1 = 2.01\left(\frac{z\_10}{zg}\right)^{\frac{2}{\alpha}} \quad Kz1 = 0.719$$

$$Kz2 = 2.01\left(\frac{z\_70}{zg}\right)^{\frac{2}{\alpha}} \quad Kz2 = 1.253$$

$$Kz3 = 2.01\left(\frac{z\_120}{zg}\right)^{\frac{2}{\alpha}} \quad Kz3 = 1.462$$

$$Kz4 = 2.01\left(\frac{z\_170}{zg}\right)^{\frac{2}{\alpha}} \quad Kz4 = 1.615$$

$$Kz5 = 2.01\left(\frac{z\_220}{zg}\right)^{\frac{2}{\alpha}} \quad Kz5 = 1.738$$

$$Kz6 = 2.01\left(\frac{z\_300}{zg}\right)^{\frac{2}{\alpha}} \quad Kz5 = 1.899$$

$$Kz7 = 2.01\left(\frac{z\_380}{zg}\right)^{\frac{2}{\alpha}} \quad Kz5 = 2.032$$

Topographic factor, K_{zt} = 1, Sec. 26.8.2

Wind directionality factor, K_d = 0.85, Table 26.6-1

Importance factor, I = 1.15 (Category III from Table 1-1 and applying to Table 6-1, ASCE 7-2

Velocity pressures (q) at different heights – ASCE 7-10, Eq. 27.3-1

$$q = 0.613 \times Kz1 \times K_{zt} \times K_d \times (V_b)^2$$

Values at different heights.

$$q1 = 0.613 \times Kz1 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q1 = 833.799 \times \frac{newton}{m^2}$$

$$q2 = 0.613 \times Kz2 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q2 = 1.454 \times 10^3 \times \frac{newton}{m^2}$$

$$q3 = 0.613 \times Kz3 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m2}$$

$$q3 = 1.696 \times 10^3 \times \frac{newton}{m^2}$$

$$q4 = 0.613 \times Kz4 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q4 = 1.873 \times 10^3 \times \frac{newton}{m^2}$$

$$q5 = 0.613 \times Kz5 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q5 = 2.017 \times 10^3 \times \frac{newton}{m^2}$$

$$q6 = 0.613 \times Kz6 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q6 = 2.203 \times 10^3 \times \frac{newton}{m^2}$$

$$q7 = 0.613 \times Kz7 \times K_{zt} \times K_d \times (V_b)^2 \times I \times \frac{newton}{m^2}$$

$$q7 = 2.357 \times 10^3 \times \frac{newton}{m^2}$$

Wind forces on Windward walls:

Gust Effect Factor, Eq. 29.9.10, G_f = 1.227 (Value taken from Sec: 34)

External pressure coefficient, C_{pw} = 0.8, Fig. 27.4-1, ASCE 7-10

Internal pressure coefficient, GC_{pi} = 0.18, Fig 26.11-1, ASCE 7-10

Wind pressure on windward face

$pw = q \times G_1 \times Cpw - q \times (GCpi)$, Eq. 27.4-1, ASCE 7-10

Pressures at different heights

$pw1 = q1 \times G_1 \times Cpw - q1 \times (GCpi)$ $\quad pw1 = 0.988 \frac{KN}{m^2}$

$pw2 = q2 \times G_1 \times Cpw - q2 \times (GCpi)$ $\quad pw2 = 1.722 \frac{KN}{m^2}$

$pw3 = q3 \times G_1 \times Cpw - q3 \times (GCpi)$ $\quad pw3 = 2.009 \frac{KN}{m^2}$

$pw4 = q4 \times G_1 \times Cpw - q4 \times (GCpi)$ $\quad pw4 = 2.219 \frac{KN}{m^2}$

$pw5 = q5 \times G_1 \times Cpw - q5 \times (GCpi)$ $\quad pw5 = 2.389 \frac{KN}{m^2}$

$pw6 = q6 \times G_1 \times Cpw - q6 \times (GCpi)$ $\quad pw6 = 2.61 \frac{KN}{m^2}$

$pw7 = q5 \times G_1 \times Cpw - q7 \times (GCpi)$ $\quad pw7 = 2.792 \frac{KN}{m^2}$

Wind forces on Leeward walls:

External pressure coefficient, $C_{pl} = 0.5$, Fig. 27.4-1, ASCE 7-10

pc = wind pressure on Leeward face.

$pc1 = q1 \times G_1 \times Cpl - q1 \times (GCpi)$ $\quad pc1 = 0.675 \frac{KN}{m^2}$

$pc2 = q2 \times G_1 \times Cpl - q2 \times (GCpi)$ $\quad pc2 = 1.176 \frac{KN}{m^2}$

$pc3 = q3 \times G_1 \times Cpl - q3 \times (GCpi)$ $\quad pc3 = 1.372 \frac{KN}{m^2}$

$pc4 = q4 \times G_1 \times Cpl - q4 \times (GCpi)$ $\quad pc4 = 1.516 \frac{KN}{m^2}$

$pc5 = q5 \times G_1 \times Cpl - q5 \times (GCpi)$ $\quad pc5 = 1.632 \frac{KN}{m^2}$

$pc6 = q6 \times G_1 \times Cpl - q6 \times (GCpi)$ $\quad pc6 = 1.783 \frac{KN}{m^2}$

$pc7 = q7 \times G_1 \times Cpl - q7 \times (GCpi)$ $\quad pc7 = 1.907 \frac{KN}{m^2}$

<u>Tier - 1: Windward: From 0m to 220.4m</u>

In **Tier-1**, 17 are the number of node points on each floor of the model where the wind forces are applied 3.8m is the floor height & 54m is the length of the side where forces are applied.

Forces applied from 0 meter to 70 meters

$F_{w0\ to\ 70} = 54m \times 3.8m \times \frac{[illegible]}{2}$

$F_{w0\ to\ 70} = 278.011\ KN$

$n_{w0\ to\ 70} = \frac{F_{0\ to\ 70}}{17}$ $n_{w0\ to\ 70} = 16.354\ KN$

Forces applied from 70 meter to 120 meters

$F_{w70\ to\ 120} = 54m \times 3.8m \times \frac{(pw2+pw3)}{2}$

$F_{w70\ to\ 120} = 382.779\ KN$

$n_{w70\ to\ 120} = \frac{F_{w70\ to\ 120}}{17}$ $n_{w70\ to\ 120} = 22.516\ KN$

Forces applied from 120 meter to 170 meters

$F_{w120\ to\ 170} = 54m \times 3.8m \times \frac{(pw3+pw4)}{2}$

$F_{w120\ to\ 170} = 433.76\ KN$

$n_{w120\ to\ 170} = \frac{F_{w120\ to\ 170}}{17}$ $n_{w120\ to\ 170} = 25.515\ KN$

Forces applied from 170 meter to 220 meters

$F_{w170\ to\ 220} = 54m \times 3.8m \times \frac{(pw4+pw5)}{2}$

$F_{w170\ to\ 220} = 472.729\ KN$

$n_{w170\ to\ 220} = \frac{F_{w170\ to\ 220}}{17}$ $n_{w170\ to\ 220} = 27.808\ KN$

Tier - 2: Windward: From 220.4m to 380m

In **Tier-2**, 15 are the number of node points on each floor of the model where the wind forces are applied 3.8m is the floor height & 34m is the length of the side where forces are applied.

Forces applied from 220.4 meter to 300 meters

$F_{w220.4\ to\ 300} = 34m \times 3.8m \times \frac{(pw5+pw6)}{2}$

$F_{w220.4\ to\ 300} = 322.9\ KN$

$n_{w220.4\ to\ 300} = \frac{F_{w220.4\ to\ 300}}{15}$

$n_{w220.4\ to\ 300} = 21.527\ KN$

Forces applied from 300 meter to 380 meters

$F_{w300\ to\ 380} = 34m \times 3.8m \times \frac{(pw6+pw7)}{2}$

$F_{w300\ to\ 380} = 348.978\ KN$

$n_{w300\ to\ 380} = \frac{F_{w300\ to\ 380}}{15}$ $n_{w300\ to\ 380} = 23.265KN$

@seismicisolation

Tier - 1: Leeward: From 0m to 220.4m

In Tier-1, 17 are the number of node points on each floor of the model where the wind forces are applied 3.8m is the floor height& 54m is the length of the side where forces are applied.

Forces applied from 0 meter to 70 meters

$F_{l0\ to\ 70} = 54m \times 3.8m \times \frac{(pc1 + pc2)}{2}$ $\quad F_{l0\ to\ 70} = 189.912\ KN$

$n_{l0\ to\ 70} = \frac{F_{l0\ to\ 70}}{17}$ $\quad n_{l0\ to\ 70} = 11.171\ KN$

Forces applied from 70 meter to 120 meters

$F_{l70\ to\ 120} = 54m \times 3.8m \times \frac{(pc2 + pc3)}{2}$

$F_{l70\ to\ 120} = 261.48\ KN$

$n_{l70\ to\ 120} = \frac{F_{l70\ to\ 70}}{17}$ $\quad n_{l70\ to\ 120} = 15.381\ KN$

Forces applied from 120 meter to 170 meters

$F_{l120\ to\ 170} = 54m \times 3.8m \times \frac{(pw3 + pw4)}{2}$

$F_{l120\ to\ 170} = 296.305\ KN$

$n_{l120\ to\ 170} = \frac{F_{l120\ to\ 170}}{17}$ $\quad n_{l120\ to\ 170} = 17.34\ KN$

Forces applied from 170 meter to 220 meters

$F_{l170\ to\ 220} = 54m \times 3.8m \times \frac{(pw4 + pw5)}{2}$ $\quad F_{l170\ to\ 220} = 322.926\ KN$

$n_{l170\ to\ 220} = \frac{F_{l170\ to\ 220}}{17}$ $\quad n_{l170\ to\ 220} = 18.996\ KN$

Tier - 2: Leeward: From 220.4m to 380m

In **Tier-2**, 15 are the number of node points on each floor of the model where the wind forces are applied 3.8m is the floor height & 34m is the length of the side where forces are applied.

Forces applied from 220.4 meter to 300 meters

$F_{l220.4\ to\ 300} = 34m \times 3.8m \times \frac{(pw5 + pw6)}{2}$

$F_{l220.4\ to\ 300} = 220.576\ KN$

$n_{l220.4\ to\ 300} = \frac{F_{l220.4\ to\ 300}}{15}$ $\quad n_{l220.4\ to\ 300} = 14.705\ KN$

Forces applied from 300 meter to 380 meters

$F_{l300\ to\ 380} = 34m \times 3.8m \times \frac{(pw6 + pw7)}{2}$

$F_{l300\ to\ 380} = 238.39\ KN$

$n_{l300\ to\ 380} = \frac{F_{300\ to\ 380}}{15}$ $n_{l300\ to\ 380} = 15.893KN$

36. Result analysis of Mega Tall Buildings of different heights by PSW concept

36.1. 1000.8 meters tall tower modeled in three different ways by introducing outriggers

This tower is modeled in three different types by adding additional supporting system “outriggers” to decrease the acceleration. Structural arrangements are same as shown in Figs. 34 to 38. Outriggers are rigid horizontal structures designed to improve building overturning stiffness and strength by connecting the building core or spine to distant columns. For tall buildings under wind cases, stiffness is always the most important to control the drift. In Model-1, 3 concrete wall outriggers are installed in three different heights. (Fig. 45). In Model-2, 6 concrete wall outriggers are installed in six different heights (Fig. 46). In Model-3 (Fig. 45), 3 steel beam outriggers in three different heights. The analysis results of these modeled shown in the Table 9.

Building types	Drift	Acceleration
Model-1	1740 mm	46.11 milli-g
Model-2	1639 mm	44.82 milli-g
Model-3	1808 mm	46.89 milli-g
Original Tower	1930 mm	47.67 milli-g

'Table 9. Acceleration and drift due to wind load

@seismicisolation

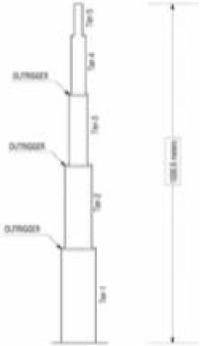


Figure 45. Model-1 & Model-3

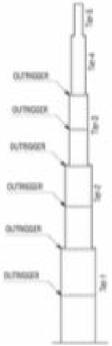


Figure 46. Model-2

It is seen from the result analysis of these four models that the deflection may be reduced by the provision of extra stiffness but it has only a slight influence on the reduction of acceleration.

Also comparing model-1 with model-2, concrete outriggers are more efficient than steel outriggers. Ali and Moore (2007) had a compressive review on the development of structural systems for tall buildings. Among the systems, adopted by Ali and Moore (2007), outrigger structure is the category with efficient height limit up to 150 stories. An obvious control on acceleration is, by an increase in damping for this One Kilometer Tall Tower.

36.2. A tower of 734.4 meters tall with different sizes and shapes analyzed using the PSW concept

A 734.4 meters tall tower is modeled in three different shapes. Model-4, in Fig. 47 holds three Tiers with Aspect Ratio 7.2: 1, sizes are shown in the Table-10. Structural arrangement has shown in the Fig. 50. Vertical lines and surface areas in Figs. 50 & 51 are the columns and shear walls respectively. Beams are not shown to visualize the clear pictures of vertical elements (Columns & Shear wall(s)). As the building is symmetric in

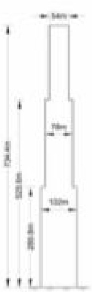
@seismicisolation

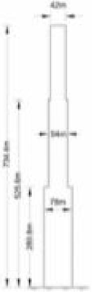
both directions, a corner of a typical floor near base of Tier-1 shows the spacing of shear walls and columns. Model-5 as shown in Fig. 48 holds three Tiers with aspect ratio 9.415 : 1 & structural arrangements as shown in Figs. 52 & 53. Model-6, as shown in Fig. 49 holds four Tiers with the same aspect ratio as of model-5 and structural arrangement also same as Model 5. There is only one modification that the third Tier has been divided to two Tiers, and few core columns are extended up to the forth Tier.

Three different models.	Model-4	Model-5	Model-6	Structure Type
Tiers	Tier-1 (Floor - 102m x 102m) & Height 280.8m	Tier-1 (Floor - 78m x 78m) & Height 280.8m	Tier-1 (Floor - 78m x 78m) & Height 280.8m	Shear Walls & Beams
	Tier-2 (Floor - 78m x 78m) & Height 244.8m	Tier-2 (Floor - 54m x 54m) & Height 244.8m	Tier-2 (Floor - 54m x 54m) & Height 244.8m	Shear Walls & Beams
	Tier-3 (Floor - 54m x 54m) & Height 208.8m	Tier-3 (Floor - 42m x 42m) & Height 208.8m	Tier-3 (Floor - 42m x 42m) & Height 104.4m	Shear Walls & Beams
			Tier-4 (Floor - 24m x 24m) & Height 104.4m	Frame Structure (Columns & Beams)
Aspect Ratio	7.2 : 1	9.415 : 1	9.415 : 1	
Drift/Sway: Allowable limit (H/500) = 1468 mm	726mm (Acceptable)	996mm (Acceptable)	992mm (Acceptable)	

Acceleration: allowable limit 30 milli-g	18.783 milli-g (Acceptable)	19.586 milli-g (Acceptable)	24.068 milli-g (Acceptable)	

'Table 10. Three different models of same heights





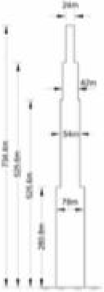


Figure 47. Model-4   **Figure 48. Model-5**   **Figure 49. Model-6**

@seismicisolation

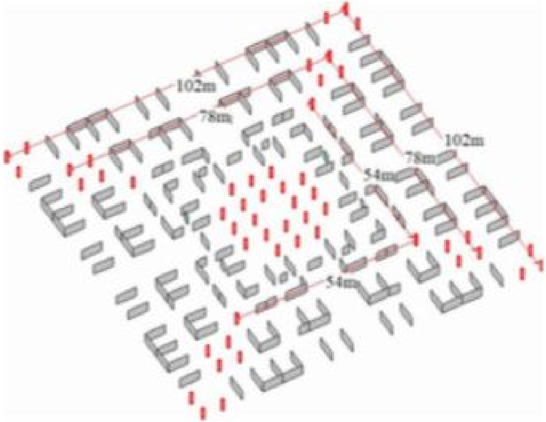


Figure 50. Vertical structural arrangement at base of Model-4

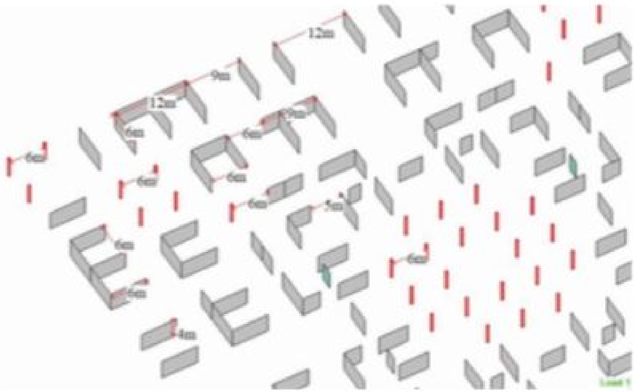


Figure 51. Enlarged corner part of Fig. 50

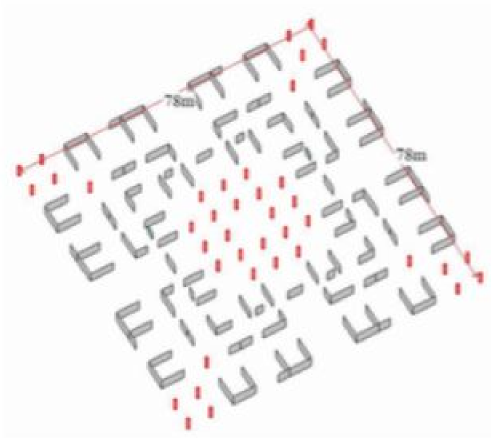


Figure 52. Vertical structural arrangement at base of models 5 & 6

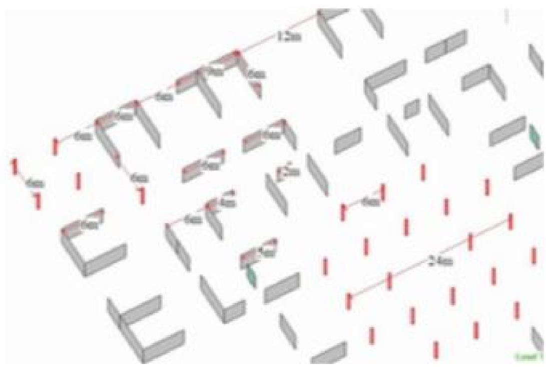


Figure 53. Enlarged corner part of Fig. 52

36.3. Impact on Building's Sway for different Tier heights

The One Kilometer Tall Tower of Fig. 32, analyzed in two different heights, keeping the total building height (1003.2 meters) same, thicknesses of the shear walls, slabs, sizes of beams, columns and floor dimensions of each Tier same for both the analysis.

Drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or storey height (ASCE Task Committee on Drift Control of Steel

Building Structure 1988 and Griffis 1993). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. An absolute limit on storey drift may also need to be imposed in light of evidence that damage to nonstructural partitions, cladding, and glazing may occur if the storey drift exceeds about (1/360) 10 mm (3/8 in.) unless special detailing practices are made to tolerate movement (Freeman 1977 and Coony and King 1988). ----- CC.1.2 - Commentary Appendix C - Serviceability Considerations - ASCE/SEI 7-10.

36.3.1. Type-1 (Equal heights)

This Building consisted of 5 Tiers. Height of 226.8 meters remained same for first 4 Tiers (Tier 1, Tier 2, Tier 3 & Tier 4) from bottom and the Tier 5 at top was 93.6 meters high.

Result analysis result shows that the Sway at top is 226.8 centimeters (7 feet-5 inch). The Drift index is 1003.2m/226.8cm = H/442.

36.3.2. Type-2 (Different Tier heights)

Keeping the top Tier (Tier 5) height (93.6 meters) same at Type-1 and heights of other 4 Tiers has been changed to the following values, Tier 1 - 280.8 meters, Tier 2 - 244.8 meters, Tier 3 - 208.8 meters & Tier 4 - 172.8 meters. Result analysis shows that the Sway at top is 1930 centimeters (6 feet-4 inches). The Drift index is 1003.2m/1.93m = H/519.

Therefore by changing the height of the Tiers, it is seen that Sway can be reduced by considerable amount.

Tier heights and number of stories for Type-1 & Type-2				
	Type-1 (Equal heights)		Type-2 (Different heights)	
	Height (m)	Stories	Height (m)	Stories
Tier-1	226.8	63	280.8	78
Tier-2	226.8	63	244.8	68
Tier-3	226.8	63	208.8	58
Tier-4	226.8	63	172.8	48
Tier-5	93.6	26	93.6	26

Table 11. Three different models of same heights

@seismicisolation

After changing the Tier heights, it is seen that sway reduced to (226.8 cm – 193 cm) = 33.8 cm.

Therefore by adjusting the heights of the Tiers, it is seen that sway can be reduced by considerable amount. As moments are maximum near the base so height increment should start from bottom Tier.

37. SW concept vs Cantilever beam

There is a close relation between "PSW concept" and a "Cantilever Beam" for lateral forces. Let's assume that a beam cantilevered from the earth. When lateral forces are applied to the beam, the beam will bend. The bending stress is zero at the beam's neutral axis, which is coincident with the centroid of the beam's cross section. The bending stress increases linearly away from the neutral axis until the maximum values at the extreme fibers at the two opposite sides of the beam along the force.

Now consider a mega tall building with a shear wall arrangement as shown in Fig. 33. When wind force is applied to any face of the structure, the bending stress is zero at the building's neutral axis, which is coincident with the centroid of the building's cross section. The bending stress increase to the shear walls linearly away from the neutral axis until the maximum values at the extreme shear walls at the two opposite sides of the building along the force.

38. Advantages of PSW Concept

1) The main concept of this system is to place the parallel shear walls in two main directions of the building. When wind force is applied to the structure, each grid will function individually to resist the wind force and the magnitude of forces to each grid will be of their average span length (Fig. 27). Due to this phenomenon, when this structural system is subjected to lateral loads such as wind load, the axial stresses in the shear walls is nearly linear. Therefore Shear Lag effect is minimized.

2) Shear walls can be moved on both sides from mid (if required for architectural demands) by keeping the area of centroid of the vertical sections at the same position. This has a negligible effect on the sway.

3) The shear walls are placed almost uniformly over the base, so the gravity loads are distributed almost uniformly to all the vertical elements (shear walls). Therefore reduce the differential settlement.

4) Plenty of natural sunlight will pass through the building's parimeter due to parallel shear wall arrangements.

5) No additional lateral load resisting system is required, like outriggers, belts or cross bracings, except tuned mass dampers (TMD)

6) Parallel shear walls from both the direction forming a perpendicular arrangement. The effect of the perpendicular walls will be to stiffen the structure in torsion, to reduce the twist, and, in doing so, to influence the contributions to the parallel wall shear and moment that result from the structure's twisting.

39. Conclusion

Engineering field professionals are trying to build buildings taller than the existing tallest ones. Generally these high-rise buildings require additional lateral systems to control the drift. But the use of the 'Parallel Shear Walls' concept in Skyscraper design is a relatively new idea which does not take any help of additional lateral systems except TMD (If the building's height is above 850 meters). This structural arrangement can be applied to any tall building of any height to get a perfect and optimized structure.

According to the study of the previous structural concepts for designing tall buildings and the Figs. 1 & 2, it is understood that by the application of the previous structural concepts, building can be built up to 140 stories. [18]Further the Buttress Core System has been able to reach 163 stories. Therefore it is required to think for new improved structural systems which can lead us to greater heights and more stable buildings. Author's structural concept "PSW" is able to reach 231 stories which will be habitable without any additional lateral supporting system. When Tuned Mass Damper is introduced, this structural system can make usable floor at 264 stories. The research work carried out on three models of heights 1008 meters, [illegible] meters & 734 meters with this

PSW concept. It is observed that the structures of heights 830 meters & 734 meters have less for drift and acceleration than the allowable limits as per International Codes and Standards. No bracings, outriggers or dampers are required for such mega tall structures. Only damping system is required beyond the height of 830 meters.

That is, PSW "Innovative Structural Arrangement" is a simple method to go for tall and mega tall structures.

@seismicisolation

PART - 2

1. Concept to deduct concrete from shear walls

When designing shear walls, stiffness properties of the walls play a valuable role in the shear stiffness of the wall. There are three basic types of shear walls and they are tall and slender (usually having an aspect ratio greater than 3 to 1), short and wide (usually having an aspect ratio less than 1.5 to 1), and walls that are in between (usually having an aspect ratio between 1.5 and 3 to 1). Tall, slender shear walls fail almost purely in flexure. Short, wide shear walls fail almost purely in shear. Shear walls with an aspect ratio between 1.5 and 3 to 1 fail in a combination of flexure and shear. They will fail first in flexure then in shear. When designing a shear wall, with any aspect ratio, the theoretical deflection under the theoretical load must be taken into account.[26]

Generally the shear walls with an aspect ratio greater than 3 to 1, so the flexural deflection equation would be used.

Two factors mainly govern the structural design of Tall/Mega Tall Buildings. These are Drift & the Acceleration. The drifts of tall buildings are mainly controlled by shear walls or bracings. Below, given a calculation and analysis where it is seen that, volume of concrete can be deducted from the shear walls which will not affect/increase the drift value of the building

When a shear wall is subjected to lateral forces due to wind, it deflects and the maximum tension develops in the windward side and maximum compression develop in the leeward side. At the middle the stress is zero due to lateral load. Participation of the middle portion of shear walls to resist the lateral loads, is less than the area of two sides of the shear wall.

Therefore there is a scope that a large amount of concrete can be deducted from the middle portion by making a hole and extend the length of shear wall which is negligible comparing to total length of the wall, and this changes will not affect the structural integrity. Below, the calculation will describe the concept of concrete deduction.

The experiments on shear walls are carried out by (1) Hand calculations /analysis & (2) Computer Analysis through STAAD/Pro software for comparison. Result analysis of STAAD/Pro also included here.

1.1. Hand Calculations

Consider a shear wall of height 3.5 meters (each story), the thickness and length are 0.6m & 5.5m respectively (Fig. 54). If it is applied to a 50 storied building and the story height is 3.5m than the height of the wall will be 175m.

Let us consider:

Lateral force on the shear wall, $w = 16.5\frac{KN}{m}$

Total height of wall, H3 = 175m

Poisson's ratio, $\nu = 0.17$ Storey height, $h = 3.5m$

Young modulus of elasticity, $E = 4.287 \times 10^7 \frac{KN}{m^2}$

Shear wall's cross section is 0.6m x 5.5m (Without making hole)

Moment of Inertia, $I_s = \frac{0.6m \times (5.5m)^3}{12} = 8.319m^4$

Total volume of concrete, $V_t = 5.5m \times 175m \times 0.6m = 577.5m^3$

Lateral deflection at top, $y_{max} = \frac{w \times (H3)^4}{8 \times E \times I_S} = 5.424m$

Note: Deflection due to shear is ignored because it represents a very small percentage of the entire deflection.

Now extend the length of the shear walls on both sides by 25mm, therefore the length will become 5.55m. Now make a hole 1.5m x 1.25m in each story and as shown in the Fig. 55. It can be imagined that there are two shear walls (wall1 & wall2) of equal lengths 2.025m connected with a beam whose length, depth and width are 1.5m, 2.25m & 0.6m respectively.

	Wall-1	Wall-2	Beam
Height/Depth	h1 = 2.025m	h2 = 2.025m	h_b = 2.25m
Thickness/Width	b1 = 0.6m	b2 = 0.6m	b_b = 0.6m
Length	H3 = 175m	H3 = 175m	b_l = 1.5m

Cross sectional area of Wall-1   A1 = h1 x b1 = 1.215m^2

Cross sectional area of Wall-2   A2 = h2 x b2 = 1.215m^2

Summation of cross sectional area of Wall-1 & Wall-2,

$A_t = A1 + A2 = 2.43m^2$

Cross sectional area of Beam, $A3 = b_b \times h_b = 1.35m^2$

Distance between the centroids of two walls,

$d_s = \frac{(h1+h2)}{2} + bl_1 = 3.525m$

Wall properties:

Moment of inertia of Wall-1, $I1_a = \frac{b1\times(h1)^3}{12} = 0.415m^4$

Moment of inertia of Wall-2, $I2_a = \frac{b2\times(h2)^3}{12} = 0.415m^4$

Summation of moment of inertia of Wall-1 & Wall-2,

$I = I1_a + I2_a = 0.83m^4$

For connecting beam, considering entire cross section effective,

Moment of inertia of the connecting Beam, $I_b = \frac{b_b\times(h_b)^3}{12} = 0.57m^4$

Clear span of beam, $bl_1 = 1.5m$

Beam cross section, $A3 = b_b \times h_b = 1.35m^2$

$$r = \left[\frac{12 \times E \times I_b}{(bl_1)^2 \times G \times A3}\right]$$

$r = 6.318$

Cross sectional shape factor for shear, $\lambda = 1.2$

[$\lambda = 1.2$, considered for the simplicity of the calculation. Because it has negligible effect on the drift. But the correction is required in the case of connecting beams with a span-to-depth ratio less than 5]

Where shear modulus, $G = \frac{E}{2\times(1+\nu)}$

Effective second moment of area, $Ic = \frac{I_b}{1+r} = 0.078m^4$

Taking account of the wall-beam flexibility, effective length = true length + (1/2) beam depth

Effective length, $le = bl_1 + \left(\frac{h_b}{2}\right) = 2.625m$

Determine structural parameters k, α & kα H

$$k = \sqrt{1 + \frac{A_t \times I}{A1 \times A2 \times (d_s)^2}} = 1.05357$$

From Fig. 10.8[5], for kα H =86.285 & k=1.05357, the value of the maximum deflection factor F3

@seismicisolation

$\alpha = \sqrt{\frac{12 \times Ic \times (d_s)^2}{(le)^3 \times h \times I}} = 0.469837$, $(\alpha kH3) = 86.626$

Using the Eq. 10.43[5] of "Tall Building Structure – Bryan & Stafford Smith-Alex Coull" the value of "F3" is written below.

$$F3 = 1 - \left(\frac{1}{k^2}\right) \times \left[1 - \frac{4}{(k\alpha H3)^2} + \frac{8}{(k\alpha H3)^4 \times \cosh k\alpha H3} \times (1 + k\alpha H3 \times \sinh k\alpha H3 - \cosh k\alpha H3)\right]$$

F3 = 0.099572

Second moment of area Ig of the composite cross section

$$I_g = I1_a + I2_a + \left(\frac{A1 \times A2}{A_t}\right) \times (d_s)^2 = 8.379 m^4$$

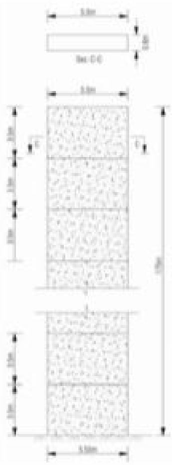


Figure 54. Solid shear wall

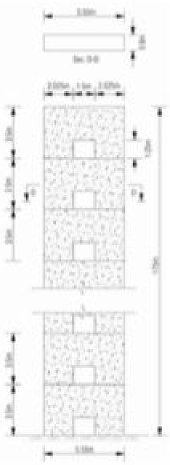


Figure 55. Shear wall with holes

@seismicisolation

After extending the length of shear wall by 50mm, and making hole of 1.5m x 1.5m as seen in the Fig. 55,

Maximim deflection at top,

$$y_{max(hole)} = \frac{w\times(H3)^4}{8\times E\times I_S} \times F3 = 5.411m$$

Lateral deflection at top of the primary structure (without making hole) is

$y_{(max)}$ = 5.424m which is greater than the value the wall with holes,

$y_{max\,(hole)}$ = 5411m

It is seen that by making a hole of size 1.5 x 1.25 from the shear wall at the middle bottom and extend both sides by 0.025m, the drift is less than that of primary structure's drift.

Volume of the shear wall with holes:

Width increased 0.025m both sides & made 50 nos of holes of size 1.5m x 1.25m

Total concrete volume V_P after making 50 holes & increased both sides by 25mm,

$$V_P = (5.5m + 2\times 25mm)\times 175m \times 0.6m - 50 \times 1.5m \times 1.25m \times 0.6m = 526.5m^3$$

Volume of concrete before making holes and extension on both sides.

$$V_L = 5.5m \times 175m \times 0.6m = 577.5m^3$$

Total concrete volume of the primary structure

Concrete saved in percentage, $sp = \left|\frac{(V_L\_V_P)}{V_L}\right| \times 100 = 8.831\%$

1.2. Comparison between Hand Calculation and STAAD Out-put result analysis

1.2 The same shear walls, F igs. 54 & 55 are analyzed through ST AAD/P ro Software for comparison with hand calculations

STAAD/PRO results analysis show that the shear wall of Fig. 54 have a deflection of 544.87 cm (X TRANS) whereas the shear wall of Fig. 55 have a deflection of 537.78 cm. Below are the results analysis by STAAD/PRO.

```
2526. ELEMENT PROPERTY
2527. 1 TO 3850 THICKNESS 0.6
2528. DEFINE MATERIAL START
2529. ISOTROPIC CONCRETE
2530. E 4.287E+007
2531. POISSON 0.17
2532. DENSITY 23.5615
2533. ALPHA 5.5E-006
2534. DAMP 0.05
2535. END DEFINE MATERIAL
2536. CONSTANTS
2537. MATERIAL CONCRETE ALL
2538. SUPPORTS
2539. 1 352 703 1054 1405 1756 2107 2458 2809 3160 3511 3862 FIXED
2540. LOAD 1 SELF WT + LATERAL FORCES
2541. SELFWEIGHT Y -1
2542. JOINT LOAD
2543. 2 TO 281 FX 8.25
2544. 282 TO 351 FX 8.25
2545. PERFORM ANALYSIS


          P R O B L E M   S T A T I S T I C S
          -----------------------------------

   NUMBER OF JOINTS        4212  NUMBER OF MEMBERS        0
   NUMBER OF PLATES        3850  NUMBER OF SOLIDS         0
   NUMBER OF SURFACES         0  NUMBER OF SUPPORTS      12

        SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

  ORIGINAL/FINAL BAND-WIDTH=   352/    20/    120 DOF
  TOTAL PRIMARY LOAD CASES =     1, TOTAL DEGREES OF FREEDOM =   25200
  SIZE OF STIFFNESS MATRIX =     3024 DOUBLE  KILO-WORDS
  REQRD/AVAIL. DISK SPACE  =     64.8/1687996.0 MB
```



```
2546. PRINT JOINT DISPLACEMENTS LIST 4212
```

```
JOINT DISPLACEMENT (CM   RADIANS)    STRUCTURE TYPE = SPACE
------------------

JOINT  LOAD   X-TRANS   Y-TRANS   Z-TRANS   X-ROTAN   Y-ROTAN   Z-ROTAN

 4212    1   544.8723  -12.2529   0.0000    0.0000    0.0000   -0.0344

************** END OF LATEST ANALYSIS RESULT **************

2547. FINISH

2448. ELEMENT PROPERTY
2449. 1 TO 4 8 TO 14 18 TO 25 27 TO 35 37 TO 45 47 TO 74 78 TO 84 88 TO 95 -
2450. 97 TO 105 107 TO 115 117 TO 144 148 TO 154 158 TO 165 167 TO 175 177 TO 185 -
2451. 187 TO 214 218 TO 224 228 TO 235 237 TO 245 247 TO 255 257 TO 284 -
2452. 288 TO 294 298 TO 305 307 TO 315 317 TO 325 327 TO 354 358 TO 364 -
2453. 368 TO 375 377 TO 385 387 TO 395 397 TO 424 428 TO 434 438 TO 445 -
2454. 447 TO 455 457 TO 465 467 TO 494 498 TO 504 508 TO 515 517 TO 525 -
2455. 527 TO 535 537 TO 564 568 TO 574 578 TO 585 587 TO 595 597 TO 605 -
2456. 607 TO 634 638 TO 644 648 TO 655 657 TO 665 667 TO 675 677 TO 3639 -
2457. 3640 THICKNESS 0.6
2458. DEFINE MATERIAL START
2459. ISOTROPIC CONCRETE
2460. E 4.287E+007
2461. POISSON 0.17
2462. DENSITY 23.5615
2463. ALPHA 5.5E-006
2464. DAMP 0.05
2465. END DEFINE MATERIAL
2466. CONSTANTS
2467. MATERIAL CONCRETE ALL
2468. SUPPORTS
2469. 1 352 703 1054 1405 2356 2707 3058 3409 3760 FIXED
2470. LOAD 1 SELF WT + LATERAL FORCE
2471. SELFWEIGHT Y -1
2472. JOINT LOAD
2473. 2 TO 281 FX 8.25
2474. 282 TO 351 FX 8.25
2475. PERFORM ANALYSIS
```


@seismicisolation

```
                P R O B L E M   S T A T I S T I C S
                -----------------------------------

        NUMBER OF JOINTS        4110  NUMBER OF MEMBERS        0
        NUMBER OF PLATES        3550  NUMBER OF SOLIDS         0
        NUMBER OF SURFACES         0  NUMBER OF SUPPORTS      10

              SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

        ORIGINAL/FINAL BAND-WIDTH=  352/    20/    126 DOF
        TOTAL PRIMARY LOAD CASES =     1, TOTAL DEGREES OF FREEDOM =   24600
        SIZE OF STIFFNESS MATRIX =     3100 DOUBLE  KILO-WORDS
        REQRD/AVAIL. DISK SPACE  =     64.0/1687948.0 MB


   2476. PRINT JOINT DISPLACEMENTS LIST 4110

    JOINT DISPLACEMENT (CM   RADIANS)     STRUCTURE TYPE = SPACE
    ------------------

  JOINT  LOAD   X-TRANS   Y-TRANS   Z-TRANS   X-ROTAN   Y-ROTAN   Z-ROTAN

    4110    1   537.7870  -12.3348    0.0000    0.0000    0.0000   -0.0379

    ************** END OF LATEST ANALYSIS RESULT **************

   2477. FINISH
```

2. Concrete reduction by changing cross-sectional shape of shear walls

One of the main factors which influences the deflection of a structure is, moment of inertia of the members. Deflection reduces when increase the moment of inertia of a member/structure. Let change the thickness of a shear wall by keeping the moment of inertia same or larger, than obviously it will not increase the deflection. Calculations given below, will describe the concept.

2.1. Hand Calculation

Below is a example (Figs. 56 & 57) how to change the thicknesses and keep the inertia forces same or larger,

@seismicisolation

Consider a shear wall,

Length L_1 = 6m, Height Hw = 50m, Thickness t = 0.5m

Lateral force on the shear wall, $w = 12\frac{KN}{m}$

Young modulus of elasticity, $E = 4.206 \times 10^{7}\frac{KN}{m^2}$

Cross sectional area of the wall, $A_1 = L_1 \times t = 3m^2$

Concrete volume $V_1 = L_1 \times Hw \times t = 150m^3$

Moment of inertia, $I_1 = \frac{t \times L_1^{\,3}}{12} = 9m^4$

Deflection due to Bending & Shear for uniform thickness:

$$y1 = \left[\frac{w \times (Hw)^4}{8 \times E \times I_1}\right] + \left[\frac{1.5 \times w \times (Hw)^2}{E \times t \times L_1}\right] = 25.1228mm$$

Deflection due to shear contributes a very small percentage of the total deflection. The deflection due to shear increases linearly as the shear wall's height increases, whereas the deflection due to bending increases very rapidly as a third power of the height of the shear wall.

Moment of inertia of a member increase in a large amount when increase the thickness of the outer portion and a less amount of moment of inertia decreases when decrease the thickness f the middle portion which in near the centroid of the section.

Now consider that the shear wall's thickness (0.5m) has been changed to three parts as shown in Fig. 57. Middle portion of length 3m with a thickness of 0.37m and each side of length 1.5m with the thickness of 0.55.

According to the Fig. 57, length & thickness of both ends are 1.5m (L_2) & 0.55m (t_2) respectively. The length and thickness of middle portion are 3m (L_3) & 0.37m (t_3) respectively.

Total cross sectional area, $A_2 = L_2 \times t_2 + L_3 \times t_3 + L_2 \times t_2 = 2.76m^2$

Concrete volume, $V_2 = A_2 \times Hw = 138m^3$

Moment of inertia from the mid point,

$$I_2 = \frac{t_3 \times (L_3)^3}{12} + 2 \times \left[\frac{t_2 \times (L_2)^3}{12} + t_2 \times \quad 2 \times \left(\frac{L_2}{2} + \frac{L_3}{2}\right)^2\right] = 9.495m^4$$

@seismicisolation

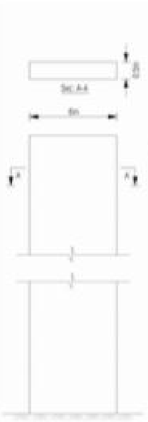


Figure 56. Shear wall of uniform thickness

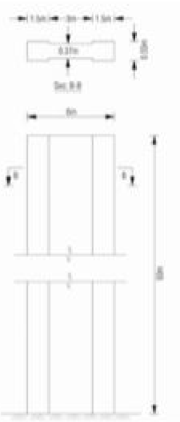


Figure 57. Shear wall with varying thicknesses

Deflection due to Bending & Shear at top,

$$y2 = \left[\frac{w \times (Hw)^4}{8 \times E \times I_2}\right] + \left[\frac{1.5 \times w \times (Hw)^2}{E \times t_3 \times (2 \times L_2 + L_3)}\right] = 23.957mm$$

It is seen that by reducing the cross sectional area and at the same time increasing the moment of inertia by thickness adjustment, the deflection could be same or less than the deflection of the original section (uniform thickness).

Percentage of concrete volume reduced

$$sp = \left[\frac{(V_1 - V_2)}{V_1}\right] \times 100 = 8\%$$

Therefore cross sectional area can be reduced by 8%.

Wind force governs the design of tall buildings, therefore if "inertia forces" of the vertical members can be increased, than the deflection limit can be controlled.

2.2. Comparison between Hand Calculation and STAAD Out-put result analysis

2.2 The same shear walls, Figs. 56 & 57 are analyzed through STAAD/Pro Software for comparison with hand calculations

STAAD/PRO results analysis show that the shear wall of Figure-56 have a deflection of 2.5438 cm (X TRANS) whereas the shear wall Figure 57 have a deflection of 2.4202 cm. Below are the results analysis of STAAD/PRO.

```
ELEMENT PROPERTY
1 TO 1200 THICKNESS 0.5
DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 4.204E+007
POISSON 0.17
DENSITY 24
ALPHA 5E-006
DAMP 0.05
TYPE CONCRETE
STRENGTH FCU 80000
END DEFINE MATERIAL
CONSTANTS
MATERIAL CONCRETE ALL
SUPPORTS
1 102 203 304 405 506 607 708 809 910 1011 1112 1213 FIXED
LOAD 1 LOADTYPE NONE  TITLE LOAD CASE 1
SELFWEIGHT Y -1
LOAD 2 LOADTYPE NONE  TITLE LOAD CASE 2
JOINT LOAD
2 TO 101 FX 6
*LOAD 2 LOADTYPE NONE  TITLE LOAD CASE 2 WIND LOAD
*JOINT LOAD
*2 TO 23 FX 6
LOAD COMB 3 COMBINATION LOAD CASE 3
1 1.0 2 1.0
PERFORM ANALYSIS
```

```
P R O B L E M   S T A T I S T I C S
-----------------------------------

NUMBER OF JOINTS        1313  NUMBER OF MEMBERS       0
NUMBER OF PLATES        1200  NUMBER OF SOLIDS        0
NUMBER OF SURFACES         0  NUMBER OF SUPPORTS     13

     SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH=   102/    22/    132 DOF
TOTAL PRIMARY LOAD CASES =     2, TOTAL DEGREES OF FREEDOM =    7800
SIZE OF STIFFNESS MATRIX =     1030 DOUBLE  KILO-WORDS
REQRD/AVAIL. DISK SPACE  =     29.5/1687900.8 MB


778. LOAD LIST 3
779. PRINT JOINT DISPLACEMENTS LIST 1313

 JOINT DISPLACEMENT (CM   RADIANS)    STRUCTURE TYPE = SPACE
 ------------------

JOINT  LOAD   X-TRANS   Y-TRANS   Z-TRANS   X-ROTAN   Y-ROTAN   Z-ROTAN

  1313    3     2.5438   -0.2724    0.0000    0.0000    0.0000   -0.0006

  ************** END OF LATEST ANALYSIS RESULT **************

  780. FINISH
751. ELEMENT PROPERTY
752. 1 TO 300 901 TO 1200 THICKNESS 0.55
753. 301 TO 900 THICKNESS 0.37
754. DEFINE MATERIAL START
755. ISOTROPIC CONCRETE
756. E 4.204E+007
757. POISSON 0.17
758. DENSITY 24
759. ALPHA 5E-006
760. DAMP 0.05
761. TYPE CONCRETE
762. STRENGTH FCU 80000
763. END DEFINE MATERIAL
764. CONSTANTS
765. MATERIAL CONCRETE ALL
766. SUPPORTS
```

```
767. 1 102 203 304 405 506 607 708 809 910 1011 1112 1213 FIXED
768. LOAD 1 DEAD LOAD
769. SELFWEIGHT Y -1
770. LOAD 2 LATERAL LOAD
771. JOINT LOAD
772. 2 TO 101 FX 4
773. LOAD COMB 3 1.0XDEAD LOAD+1.0XLATERAL LOAD
774. 1 1.0 2 1.0
775. PERFORM ANALYSIS

        P R O B L E M   S T A T I S T I C S
        -----------------------------------

  NUMBER OF JOINTS        1313  NUMBER OF MEMBERS        0
  NUMBER OF PLATES        1200  NUMBER OF SOLIDS         0
  NUMBER OF SURFACES         0  NUMBER OF SUPPORTS      13

       SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

  ORIGINAL/FINAL BAND-WIDTH=   102/    22/    132 DOF
  TOTAL PRIMARY LOAD CASES =     2, TOTAL DEGREES OF FREEDOM =    7800
  SIZE OF STIFFNESS MATRIX =     1030 DOUBLE  KILO-WORDS
  REQRD/AVAIL. DISK SPACE  =     29.5/1087883.9 MB


776. LOAD LIST 3
777. PRINT JOINT DISPLACEMENTS LIST 1313

  JOINT DISPLACEMENT (CM   RADIANS)    STRUCTURE TYPE = SPACE
  ------------------

JOINT  LOAD   X-TRANS   Y-TRANS   Z-TRANS   X-ROTAN   Y-ROTAN   Z-ROTAN

  1313    3    2.4204   -0.2619    0.0000    0.0000    0.0000   -0.0005

  ************** END OF LATEST ANALYSIS RESULT **************

  778. FINISH
```

3. History of Skyscrapers[27]

A skyscraper is a continuously habitable high-rise building that has over 40 floors[1] and is taller than approximately 150 m (492 ft).[2] Historically, the term first referred to buildings with 10 to 20 floors in the 1880s. The definition shifted with advancing construction technology during the 20th century.[1] Skyscrapers may host offices, residential spaces, and retail spaces. For buildings above a height of 300 m (984 ft), the term supertall skyscrapers can be used, while skyscrapers reaching beyond 600 m (1,969 ft) are classified as megatall skyscrapers.[3]

@seismicisolation

One common feature of skyscrapers is having a steel framework that supports curtain walls. These curtain walls either bear on the framework below or are suspended from the framework above, rather than resting on load-bearing walls of conventional construction. Some early skyscrapers have a steel frame that enables the construction of load-bearing walls taller than of those made of reinforced concrete.

Modern skyscrapers' walls are not load-bearing, and most skyscrapers are characterised by large surface areas of windows made possible by steel frames and curtain walls. However, skyscrapers can have curtain walls that mimic conventional walls with a small surface area of windows. Modern skyscrapers often have a tubular structure, and are designed to act like a hollow cylinder to resist wind, seismic, and other lateral loads. To appear more slender, allow less wind exposure and transmit more daylight to the ground, many skyscrapers have a design with setbacks, which in some cases is also structurally required.

As of 2019, only nine cities have more than 100 skyscrapers that are 150 m (492 ft) or taller: Hong Kong (355), New York City (284), Shenzhen (235), Dubai (199), Shanghai (163), Tokyo (155), Chongqing (127), Chicago (126), and Guangzhou (115).[4] Leading cities by the number of skyscrapers with a height of more than 100 meters, having more than 1000 such buildings, are: Hong Kong, Shanghai, Seoul, Incheon, Chongqing, Shenzhen, Guangzhou, Wuhan, Shenyang, Beijing, New York, Sao Paulo.[4]

The Home Insurance Building in Chicago, completed in 1885, was the first steel-frame skyscraper; it was demolished in 1931.

Definition

The term skyscraper was first applied to buildings of steel framed construction of at least 10 stories in the late 19th century, a result of public amazement at the tall buildings being built in major American cities like Chicago, New York City, Philadelphia, Detroit, and St. Louis.[5] The first steel-frame skyscraper was the Home Insurance Building (originally 10 stories with a height of 42 m or 138 ft) in Chicago, Illinois in 1885. Some point to Philadelphia's 10-story Jayne Building (1849–50) as a proto- skyscraper, or to New York's seven-floor Equitable Life Building (New York City), built in 1870, for its innovative use of a kind of skeletal frame, but such designation depends largely on what factors are chosen. Even the scholars making the argument find it to be purely academic.[6][7]

The structural definition of the word skyscraper was refined later by architectural historians, based on engineering developments of the 1880s that had enabled construction of tall multi-story buildings. This definition was based on the steel skeleton—as opposed to constructions of load-bearing masonry, which passed their practical limit in 1891 with Chicago's Monadnock Building.

What is the chief characteristic of the tall office building? It is lofty. It must be tall. The force and power of altitude must be in it, the glory and pride of exaltation must be in it. It must be every inch a proud and soaring thing, rising in sheer exaltation that from bottom to top it is a unit without a single dissenting line.

— Louis Sullivan's The Tall Office Building Artistically Considered (1896)

The Council on Tall Buildings and Urban Habitat defines skyscrapers as those buildings which reach or exceed 150 m (490 ft) in height.[8] Others in the United States and Europe also draw the lower limit of a skyscraper at 150 m (490 ft).[9][2]

The Emporis Standards Committee defines a high-rise building as "a multi-story structure between 35–100 meters tall, or a building of unknown height from 12–39 floors"[10] and a skyscraper as "a multi-story building whose architectural height is at least 100 m or 330 ft."[11] Some structural engineers define a highrise as any vertical construction for which wind is a more significant load factor than earthquake or weight.

Note that this criterion fits not only high-rises but some other tall structures, such as towers.

The word skyscraper often carries a connotation of pride and achievement. The skyscraper, in name and social function, is a modern expression of the age-old symbol of the world center or axis mundi: a pillar that connects earth to heaven and the four compass directions to one another.[12]

The tallest building in ancient times was the 146 m (479 ft) Great Pyramid of Giza in ancient Egypt, built in the 26th century BC. It was not surpassed in height for thousands of years, the 160 m (520 ft) Lincoln Cathedral having exceeded it in 1311–1549, before its central spire collapsed.[13] The latter in turn was not surpassed until the 555-foot (169 m) Washington Monument in 1884. However, being uninhabited, none of these structures actually comply with the modern definition of a skyscraper.

High-rise apartments flourished in classical antiquity. Ancient Roman insulae in imperial cities reached 10 and more stories.[14] Beginning with Augustus (r. 30 BC-14 AD), several emperors attempted to establish limits of 20–25 m for multi-story buildings, but met with only limited success.[15][16] Lower floors were typically occupied by shops or wealthy families, the upper rented to the lower classes.[14] Surviving Oxyrhynchus Papyri indicate that seven-story buildings existed in provincial towns such as in 3rd century AD Hermopolis in Roman Egypt.[17]

The skylines of many important medieval cities had large numbers of high-rise urban towers, built by the wealthy for defense and status. The residential Towers of 12th century Bologna numbered between 80 and 100 at a time, the tallest of which is the 97.2 m (319 ft) high Asinelli Tower. A Florentine law of 1251 decreed that all urban buildings be immediately reduced to less than 26 m.[18] Even medium-sized towns of the era are known to have proliferations of towers, such as the 72 up to 51 m height in San Gimignano.[18]

The medieval Egyptian city of Fustat housed many high-rise residential buildings, which Al-Muqaddasi in the 10th century described as resembling minarets. Nasir Khusraw in the early 11th century described some of them rising up to 14 stories, with roof gardens on the top floor complete with ox-drawn water wheels for irrigating them.[19] Cairo in the 16th century had high-rise apartment buildings where the two lower floors were for commercial and storage purposes and the multiple stories above them were rented out to tenants.[20] An early example of a city consisting entirely of high-rise housing is the 16th-century city of Shibam in Yemen. Shibam was made up of over 500 tower houses,[21] each one rising 5 to 11 stories high,[22] with each floor being an apartment occupied by a single family. The city was built in this way in order to protect it from Bedouin attacks.[21] Shibam still has the tallest mud brick buildings in the world, with many of them over 30 m (98 ft) high.[23]

An early modern example of high-rise housing was in 17th-century Edinburgh, Scotland, where a defensive city wall defined the boundaries of the city. Due to the restricted land area available for development, the houses increased in height instead. Buildings of 11 stories were common, and there are records of buildings as high as 14 stories. Many of the stone- built structures can still be seen today in the old town of Edinburgh. The oldest iron framed building in the world, although only partially iron framed, is The Flaxmill (also locally known as the "Maltings"), in Shrewsbury, England. Built in 1797, it is seen as the "grandfather of skyscrapers", since its fireproof combination of cast iron columns and cast iron beams developed into the modern steel frame that made modern skyscrapers possible. In 2013 funding was confirmed to convert the derelict building into offices.[24]

Early skyscrapers

In 1857, Elisha Otis introduced the safety elevator, allowing convenient and safe passenger movement to upper floors. Another crucial development was the use of a steel frame instead of stone or brick, otherwise the walls on the lower floors on a tall building would be too thick to be practical. An early development in this area was Oriel Chambers in Liverpool, England. It was only five floors high.[25][26][27] Further developments led to the world's first skyscraper, the ten-story Home Insurance Building in Chicago, built in 1884–1885.[28] While its height is not considered very impressive today, it was at that time. The building of tall buildings in the 1880s gave the skyscraper its first architectural movement the Chicago School, which developed what has been called the Commercial Style.[29]

Oriel Chambers in Liverpool is the world's red first glass curtain walled building. Louis, Missouri The stone mullions are decorative.

The Wainwright Building, a 10-storey brick office building in St. built in 1891

The architect, Major William Le Baron Jenney, created a load-bearing structural frame. In this building, a steel frame supported the entire weight of the walls, instead of load-bearing walls carrying the weight of the building. This development led to the "Chicago skeleton" form of construction. In addition to the steel frame, the Home Insurance Building also utilized fireproofing, elevators, and electrical wiring, key elements in most skyscrapers today.[30]

Burnham and Root's 45 m (148 ft) Rand McNally Building in Chicago,1889, was the first all-steel framed skyscraper,[31] while Louis Sullivan's 41 m (135 ft) Wainwright Building in St. Louis, Missouri, 1891, was the first steel-framed building with soaring vertical bands to emphasize the height of the building and is therefore considered to be the first early skyscraper.

In 1889, the Mole Antonelliana in Italy was 167 m (549 ft) tall.

Most early skyscrapers emerged in the land-strapped areas of Chicago and New York City toward the end of the 19th century. A land boom in Melbourne, Australia between 1888 and 1891 spurred the creation of a significant number of early skyscrapers, though none of these were steel reinforced and few remain today. Height limits and fire restrictions were later introduced. London builders soon found building heights limited due to a complaint from Queen Victoria, rules that continued to exist with few exceptions.

Concerns about aesthetics and fire safety had likewise hampered the development of skyscrapers across continental Europe for the first half of the twentieth century. Some

@seismicisolation

notable exceptions are the 43 m (141 ft) tall 1898 Witte Huis (White House) in Rotterdam; the Royal Liver Building in Liverpool, completed in 1911 and 90 m (300 ft) high;[32] the 57 m (187 ft) tall 1924 Marx House in Düsseldorf, Germany; the 61 m (200 ft) Kungstornen (Kings' Towers) in Stockholm, Sweden, which were built 1924–25,[33] the 89 m (292 ft) Edificio Telefónica in Madrid, Spain, built in 1929; the 87.5 m (287 ft) Boerentoren in Antwerp, Belgium, built in 1932; the 66 m (217 ft) Prudential Building in Warsaw, Poland, built in 1934; and the 108 m (354 ft) Torre Piacentini in Genoa, Italy, built in 1940. After an early competition between Chicago and New York City for the world's tallest building, New York took the lead by 1895 with the completion of the 103 m (338 ft) tall American Surety Building, leaving New York with the title of the world's tallest building for many years.

Modern skyscrapers

The Flatiron Building was completed in 1902 in New York City.

Completed in 1931, the Empire State Building in New York City was the Tallest building in the world for nearly 40 years.

Modern skyscrapers are built with steel or reinforced concrete frameworks and curtain walls of glass or polished stone. They use mechanical equipment such as water pumps and elevators. Since the 1960s, according to the CTBUH, the skyscraper has been reoriented away from a symbol for North American corporate power to instead communicate a city or nation's place in the world.[34]

Skyscraper construction entered a three-decades-long era of stagnation in 1930 due to the Great Depression and then World War II. Shortly after the war ended, the Soviet Union began construction on a series of skyscrapers in Moscow. Seven, dubbed the "Seven Sisters," were built between 1947 and 1953; and one, the Main building of Moscow State University, was the tallest building in Europe for nearly four decades (1953–1990). Other skyscrapers in the style of Socialist Classicism were erected in East Germany (Frankfurter Tor), Poland (PKiN), Ukraine (Hotel Ukrayina), Latvia (Academy of Sciences) and other Eastern Bloc countries. Western European countries also began to permit taller skyscrapers during the years immediately following World War II. Early examples include Edificio España (Spain) Torre Breda (Italy).

From the 1930s onward, skyscrapers began to appear in various cities in East and Southeast Asia as well as in Latin America. Finally, they also began to be constructed in cities of Africa, the Middle East, South Asia and Oceania (mainly Australia) from the late 1950s on.

Skyscraper projects after World War II typically rejected the classical designs of the early skyscrapers, instead embracing the uniform international style; many older skyscrapers were redesigned to suit contemporary tastes or even demolished—such as New York's Singer Building, once the world's tallest skyscraper.

German architect Ludwig Mies van der Rohe became one of the world's most renowned architects in the second half of the 20th century. He conceived of the glass façade skyscraper[35] and, along with Norwegian Fred Severud,[36] he designed the Seagram Building in 1958, a skyscraper that is often regarded as the pinnacle of the modernist high-rise architecture.[37]

Skyscraper construction surged throughout the 1960s. The impetus behind the upswing was a series of transformative innovations[38] which made it possible for people to live and work in "cities in the sky".[39]

@seismicisolation

Opened in 1973, the world Trade Center in New York the Empire State Building as tallest in the world from 1970 to 1973

Completed in 1973, the Sears Tower in Chicago City dethroned World Trade Center, and was the tallest in the world from 1974 to 1998

In the early 1960s structural engineer Fazlur Rahman Khan, considered the "father of tubular designs" for high-rises,[40] discovered that the dominating rigid steel frame structure was not the only system apt for tall buildings, marking a new era of skyscraper construction in terms of multiple structural systems.[41] His central innovation in skyscraper design and construction was the concept of the "tube" structural system, including the "framed tube", "trussed tube", and "bundled tube".[42] His "tube concept", using all the exterior wall perimeter structure of a building to simulate a thin-walled tube, revolutionized tall building design.[43] These systems allow greater economic efficiency,[44] and also allow skyscrapers to take on various shapes, no longer needing to be rectangular and box- shaped.[45] The first building to employ the tube structure was the Chestnut De-Witt apartment building,[38] this building is considered to be a major development in modern architecture.[38] These new designs opened an economic door for contractors, engineers, architects, and investors, providing vast amounts of real estate space on minimal plots of land.[39] Over the next fifteen years, many towers were built by Fazlur Rahman Khan and the "Second Chicago School",[46] including the hundred story

John Hancock Center and the massive 442 m (1,450 ft) Willis Tower.[47] Other pioneers of this field include Hal Iyengar and William LeMessurier, and Minoru Yamasaki, the architect of the World Trade Center.

Many buildings designed in the 70s lacked a particular style and recalled ornamentation from earlier buildings designed before the 50s. These design plans ignored the environment and loaded structures with decorative elements and extravagant finishes.[48] This approach to design was opposed by Fazlur Khan and he considered the designs to be whimsical rather than rational. Moreover, he considered the work to be a waste of precious natural resources.[49] Khan's work promoted structures integrated with architecture and the least use of material resulting in the least carbon emission impact on the environment.[50] The next era of skyscrapers will focus on the environment including performance of structures, types of material, construction practices, absolute minimal use of materials/natural resources, embodied energy within the structures, and more importantly, a holistically integrated building systems approach.[48]

Modern building practices regarding supertall structures have led to the study of "vanity height".[51][52] Vanity height, according to the CTBUH, is the distance between the highest floor and its architectural top (excluding antennae, flagpole or other functional extensions). Vanity height first appeared in New York City skyscrapers as early as the 1920s and 1930s but supertall buildings have relied on such uninhabitable extensions for on average 30% of their height, raising potential definitional and sustainability issues.[53][54][55] The current era of skyscrapers focuses on sustainability, its built and natural environments, including the performance of structures, types of materials, construction practices, absolute minimal use of materials and natural resources, energy within the structure, and a holistically integrated building systems approach. LEED is a current green building standard.[56]

Architecturally, with the movements of Postmodernism, New Urbanism and New Classical Architecture, that established since the 1980s, a more classical approach came back to global skyscraper design, that remains popular today.[57] Examples are the Wells Fargo Center, NBC Tower, Parkview quare, 30 Park Place, the Messeturm, the iconic Petronas Towers and Jin Mao Tower.

Other contemporary styles and movements in skyscraper design include organic, sustainable, neo-futurist, structuralist, high-tech, deconstructivist, blob, digital, streamline, novelty, critical regionalist, vernacular, Neo Art Deco and neo-historist, also known as revivalist.

3 September is the global commemorative day for skyscrapers, called "Skyscraper Day".[58]

New York City developers competed among themselves, with successively taller buildings claiming the title of "world's tallest" in the 1920s and early 1930s, culminating with the completion of the 318.9 m (1,046 ft) Chrysler Building in 1930 and the 443.2 m (1,454 ft) Empire State Building in 1931, the world's tallest building for forty years. The first completed 417 m (1,368 ft) tall World Trade Center tower became the world's tallest building in 1972. However, it was overtaken by the Sears Tower (now Willis Tower) in Chicago within two years. The 442 m (1,450 ft) tall Sears Tower stood as the world's tallest building for 24 years, from 1974 until 1998, until it was edged out by 452 m (1,483 ft) Petronas Twin Towers in Kuala Lumpur, which held the title for six years.

Design and construction

The design and construction of skyscrapers involves creating safe, habitable spaces in very tall buildings. The buildings must support their weight, resist wind and earthquakes, and protect occupants from fire. Yet they must also be conveniently accessible, even on the upper floors, and provide utilities and a comfortable climate for the occupants. The problems posed in skyscraper design are considered among the most complex encountered given the balances required between economics, engineering, and construction management.

One common feature of skyscrapers is a steel framework from which curtain walls are suspended, rather than load-bearing walls of conventional construction. Most skyscrapers have a steel frame that enables them to be built taller than typical load-bearing walls of reinforced concrete. Skyscrapers usually have a particularly small surface area of what are conventionally thought of as walls. Because the walls are not load-bearing most skyscrapers are characterized by surface areas of windows made possible by the concept of steel frame and curtain wall. However, skyscrapers can also have curtain walls that mimic conventional walls and have a small surface area of windows.

The concept of a skyscraper is a product of the industrialized age, made possible by cheap fossil fuel derived energy and industrially refined raw materials such as steel and concrete. The construction of skyscrapers was enabled by steel frame construction that surpassed brick and mortar construction starting at the end of the 19th century and finally surpassing it in the 20th century together with reinforced concrete construction as the price of steel decreased and labour costs increased.

The steel frames become inefficient and uneconomic for supertall buildings as usable floor space is reduced for progressively larger supporting columns.[59] Since about 1960, tubular designs have been used for high rises. This reduces the usage of material (more efficient in economic terms – Willis Tower uses a third less steel than the Empire State Building) yet allows greater height. It allows fewer interior columns, and so creates more usable floor space. It further enables buildings to take on various shapes.

Elevators are characteristic to skyscrapers. In 1852 Elisha Otis introduced the safety elevator, allowing convenient and safe passenger movement to upper floors. Another crucial development was the use of a steel frame instead of stone or brick, otherwise the walls on the lower floors on a tall building would be too thick to be practical. Today major manufacturers of elevators include Otis, ThyssenKrupp, Schindler, and KONE.

Advances in construction techniques have allowed skyscrapers to narrow in width, while increasing in height. Some of these new techniques include mass dampers to reduce vibrations and swaying, and gaps to allow air to pass through, reducing wind shear.[60]

Basic design considerations

Good structural design is important in most building design, but particularly for skyscrapers since even a small chance of catastrophic failure is unacceptable given the high price. This presents a paradox to civil engineers: the only way to assure a lack of failure is to test for all modes of failure, in both the laboratory and the real world. But the only way to know of all modes of failure is to learn from previous failures. Thus, no engineer can be absolutely sure that a given structure will resist all loadings that could cause failure, but can only have large enough margins of safety such that a failure is acceptably unlikely. When buildings do fail, engineers question whether the failure was due to some lack of foresight or due to some unknowable factor.

Loading and vibration

The load a skyscraper experiences is largely from the force of the building material itself. In most building designs, the weight of the structure is much larger than the weight of the material that it will support beyond its own weight. In technical terms, the dead load, the load of the structure, is larger than the live load, the weight of things in the structure (people, furniture, vehicles, etc.). As such, the amount of structural material required within the lower levels of a skyscraper will be much larger than the material required within higher levels. This is not always visually apparent. The Empire State Building's setbacks are actually a result of the building code at the time (1916 Zoning Resolution), and were not structurally required. On the other hand, John Hancock Center's shape is uniquely the result of how it supports loads. Vertical supports can come in several types, among which the most common for skyscrapers can be categorized as steel frames, concrete cores, tube within tube design, and shear walls.

The wind loading on a skyscraper is also considerable. In fact, the lateral wind load imposed on supertall structures is generally the governing factor in the structural design. Wind pressure increases with height, so for very tall buildings, the loads associated with wind are larger than dead or live loads.

Other vertical and horizontal loading factors come from varied, unpredictable sources, such as earthquakes.

History of the tallest skyscrapers

At the beginning of the 20th century, New York City was a center for the Beaux-Arts architectural movement, attracting the talents of such great architects as Stanford White and Carrere and Hastings. As better construction and engineering technology became available as the century progressed, New York City and Chicago became the focal point of the competition for the tallest building in the world. Each city's striking skyline has been composed of numerous and varied skyscrapers, many of which are icons of 20th-century architecture:

The Home Insurance Building in Chicago, which was built in 1884, is considered the world's first skyscraper due to its steel skeleton.[1] Subsequent buildings such as the Singer Building and the Metropolitan Life Tower were higher still.

The Flatiron Building, designed by Daniel Hudson Burnham and standing 285 ft (87 m) high, was one of the tallest buildings in New York City upon its completion in

1902, made possible by its steel skeleton. It was one of the first buildings designed with a steel framework, and to achieve this height with other construction methods of that time would have been very difficult.

The Tower Building, designed by Bradford Gilbert and built in 1889, is considered by some to be New York City's first skyscraper, and may have been the first building in New York City to use a skeletal steel frame.[2] The Woolworth Building, a neo-Gothic "Cathedral of Commerce" overlooking City Hall, was designed by Cass Gilbert. At 792 feet (241 m), it became the world's tallest building upon its completion in 1913, an honor it retained until 1930, when it was overtaken by 40 Wall Street.

Later in 1930 the Chrysler Building took the lead as the tallest building in the world, scraping the sky at 1,046 feet (319 m).[3] Designed by William Van Alen, an Art Deco style masterpiece with an exterior crafted of brick,[4] the Chrysler Building continues to be a favorite of New Yorkers to this day.[5]

The Empire State Building, the first building to have more than 100 floors (it has 102), was completed the following year. It was designed by Shreve, Lamb and Harmon in the contemporary Art Deco style. The tower takes its name from the nickname of New York State. Upon its completion in 1931 at 1,250 feet (381 m), it took the top spot as tallest building, and towered above all other buildings until 1972. The antenna mast added in 1951 brought pinnacle height to 1,472 feet (449 m), lowered in 1984 to 1,454 feet (443 m).[6]

The World Trade Center officially reached full height in 1972, was completed in 1973, and consisted of two tall towers and several smaller buildings. For a short time, the first of the two towers was the world's tallest building. Upon completion, the towers stood for 28 years, until the September 11 attacks destroyed the buildings in 2001. Various governmental entities, financial firms, and law firms called the towers home.

The Willis Tower (formerly Sears Tower) was completed in 1974, one year after the World Trade Center, and surpassed it as the world's tallest building. It was the first building to employ the "bundled tube" structural system, designed by Fazlur Khan. The building was not surpassed in height until the Petronas Towers were constructed in 1998, but remained the tallest in some categories until Burj Khalifa surpassed it in all categories in 2010. It is currently the second tallest building in the United States, after One World Trade Center, which was built to replace the destroyed towers.

@seismicisolation

Momentum in setting records passed from the United States to other nations with the opening of the Petronas Twin Towers in Kuala Lumpur, Malaysia, in 1998. The record for the world's tallest building has remained in Asia since the opening of Taipei 101 in Taipei, Taiwan, in 2004. A number of architectural records, including those of the world's tallest building and tallest free-standing structure, moved to the Middle East with the opening of the Burj Khalifa in Dubai, United Arab Emirates.

This geographical transition is accompanied by a change in approach to skyscraper design. For much of the twentieth century large buildings took the form of simple geometrical shapes. This reflected the "international style" or modernist philosophy shaped by Bauhaus architects early in the century. The last of these, the Willis Tower and World Trade Center towers in New York, erected in the 1970s, reflect the philosophy. Tastes shifted in the decade which followed, and new skyscrapers began to exhibit postmodernist influences. This approach to design avails itself of historical elements, often adapted and re-interpreted, in creating technologically modern structures. The Petronas Twin Towers recall Asian pagoda architecture and Islamic geometric principles. Taipei 101 likewise reflects the pagoda tradition as it incorporates ancient motifs such as the ruyi symbol. The Burj Khalifa draws inspiration from traditional Islamic art. Architects in recent years have sought to create structures that would not appear equally at home if set in any part of the world, but that reflect the culture thriving in the spot where they stand.

@seismicisolation

The following list measures some Tall Buildings built from 1870 to 2010

Built	Buildin g	City	Country	Official	Floors	Pinnacle	Current
1870	Equitable Life	New York City	United States	43 m	8		Destroyed by fire in
1889	Auditorium	Chicago		82m	17	106m	Standing
1890	New York World Building	New York City		94m	20	106m	Demolished in 1955
1894	Philadelphia	Philadelphia		155.8m	9	167m	Standing
1908	Singer	New York City		187m	47		Demolished
1909	Met Life			213m	50		Standing
1913	Woolworth			241m	57		Standing
1930	40 Wall			282m	70	283m	Standing
1930	Chrysler			319m	77	319m	Standing
1931	Empire State Building			381m	102	443m	Standing
1972	World Trade			417m	110	526.8m	Destroyed in 2001 in
1974	Willis Tower (formerly Sears	Chicago		442m	110	527.3m	Standing
1996	Petronas Towers	Kuala Lumpur	Malaysia	452m	88	452m	Standing
2004	Taipei 101	Taipei	Taiwan	509m	101	509.2m	Standing
2010	Burj Khalifa	Dubai	United Arab	828m	163	829.8m	Standing

Trump Tower

New York, New York, USA

Year of completion: 1982

Height from street to roof:

202m (664 ft) Number of stories: 58

Frame material: Concrete

Number of levels below ground: 3

Maximum lateral deflection: H/600

Type of structure: Concrete shear core linked by concrete outrigger walls to concrete perimeter frames.

Columns

Size at ground floor: 813mm x 813mm

Spacing: 12.2 to 7.3 m

Core

Shear walls, 457 mm thick at ground floor in 49-MPa (7000 psi) concrete.

Concrete strength: 49 MPa (7000 psi)

@seismicisolation

Melbourne Cent ral

Melbourne, Austr alia

Year of completion: 1991

Height from street to roof: 211m (692 ft) Number of stories: 54

Frame material: Concrete core, steel floor beams

Number of levels below ground: 3

Frame material: Concrete

Maximum lateral deflection: 100 mm

Type of structure: Concrete core, concrete perimeter tube in tube

Columns

Size at ground floor: 1.0m x 1.2m

Spacing: 6m

Core: Shear walls: 65 MPa (10000 psi) Thickness at ground floor: 600 and 200 mm

Concrete strength: 60 MPa (8500 psi)

ACT Tower

Hamamatsu City, Japan

Year of completion: 1994

Height from street to roof: 211.9m (695 ft)

Number of stories: 47

Frame material: Steel

Number of levels below ground: 2

Maximum lateral deflection: H/200

Type of structure: Braced frames

Beam span: 17.5m and 10m

Beam depth: 850mm and 700mm Columns

Size at ground floor: 750mm x 600mm Spacing: 3.2 and 6.4m

Core: X- and K- braced frames

@seismicisolation

Bourke Place

Melbourne, Australia

Year of completion: 1991

Height from street to roof: 223m (732 ft) Number of stories: 54

Number of levels below ground: 3

Frame material: Concrete

Maximum lateral deflection: 200 mm Type of structure: Concrete core, concrete perimeter tube in tube

Concrete strength: 60 MPa (8500 psi)

Columns

Size at ground floor: 1.1m x 1.1m

Spacing: 8.1m

Core

Shear walls: 65 MPa (10000 psi)

Thickness at ground floor: 400 and 200 mm

@seismicisolation

City Spire

New York, USA

Year of completion: 1987

Height from street to roof: 248m (814 ft)

Number of stories: 75

Frame material: Concrete

Number of levels below ground: 2

Maximum lateral deflection: H/500

Type of structure: shear walls with outriggers at transfer levels and interior diagonals in office levels

Columns: Size and spacing vary

Spacing: 6m

Core: Concrete walls with varying thickness.

Concrete strength: 56 MPa (8000 psi)

Two Prudential Plaza

Chicago, Illinois, USA

Year of completion: 1990

Height from street to roof: 278m (912 ft)

Number of stories: 64

Frame material: Concrete to level 59, steel above

Number of levels below ground: 5

Frame material: Concrete

Maximum lateral deflection: 488 & 419 mm, 50 year return

Type of structure: Shear core with outrigger beams and perimeter frame

Typical floor

Story height: 3.96m Beam span: 12.2m Beam depth: 610mm Beam spacing: 6.1m Slab: One-way 150-mm slabs, typically 28-MPa (4000 psi) concrete

Columns

Size at ground floor: 890mm x 1140mm at 6.1m centers Spacing: 6m

Core

Shear walls: 840, 610 & 460mm thick at ground floor

Thickness at ground floor: 600 and 200 mm

Material: Concrete, 84 MPa (12000 psi)

@seismicisolation

Overseas Union Bank Center

Singapore

Year of completion: 1986

Height from street to roof: 280m (919 ft) Number of stories: 63

Frame material: Steel with concrete walls to stairs and core

Number of levels below ground: 4

Typical floor live load: 2.5 kPa (50 psf) Maximum lateral deflection: 488mm (17.5 in)

Type of structure: Hybrid system of steel frames with concrete walls to increase rigidity

Typical floor

Story height: 4m

Beam span: 20.3m (66 ft 7 in) Beam depth: 950 mm (37.5 in) Beam spacing: 4.32m (14 ft 2 in)

Material: Steel, grade 50 and 43

Slab: 150-mm (6-in) concrete on metal deck

Columns:

Size at ground floor: 800mm x 800mm (31.5 by 31.5 in)

Spacing: varies

Material: Steel, grade 55 and 50

Core: Hybrid steel frame with concrete wall zones

Thickness at ground floor: 600 mm (24 in)

Material: Steel, grade 55 and 50; concrete, 45 MPa (64 psi)

Columbia Seafirst Center

Seattle, Washington, USA

Year of completion: 1985

Height from street to roof: 288m (947ft)

Number of stories: 76

Frame material: Structural steel with composite steel-concrete columns

Number of levels below ground: 6

Typical floor live load: 2.5 kPa (50 psf)

Maximum lateral deflection: 483mm (19 in), 100-yr return

Type of structure: Braced steel core incorporating viscoelastic damper; triangular core is linked by diagonal steel members at its corners to 3 large steel and high-strength concrete columns

Typical floor

Story height: 3.5m

Slab: 50-mm (2-in) concrete on 50-mm (2- in) steel deck

Columns: 3 major columns, 2.44 by 3.66 m (8 by 12 ft) at ground floor

Material: Concrete, 66 MPa (9500 psi) Core: Braced-steel rigid frame with arches up to 11 stories tall transferring load to composite columns

One Liberty Place

Philadelphia, Pennsylvania, USA

Year of completion: 1988

Height from street to roof: 288m (945 ft) Number of stories: 61

Frame material: Structural steel-braced core with super diagonal outriggers

Number of levels below ground: 1

Maximum lateral deflection: H/450

Type of structure: Braced steel core linked by steel girders to exterior columns

Beam span, depth & spacing are 13.4m, 530mm & 3.05m respectively

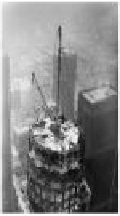
Columns

Size at ground floor: W350 by 384 & W350 by 257 built up to 2788 kg/m

Spacing: 6.1, 13.4, 21.3m

Core: Linked braced frame with outriggers.

@seismicisolation



First Interstate World Center

Los Angeles, California, USA

Year of completion: 1990

Height from street to roof: 310.3m (1018 ft)

Number of stories: 75

Frame material: Structural steel

Number of levels below ground: 2

Typical floor live load: 2.5 kPa (50 psf) Maximum lateral deflection: 584mm

Type of structure: Perimeter ductile tube with chevron braced core

Earthquake loading: C = 0.03, K = 0.8

Foundation conditions: Shale

Footing type: Spread footing

Story height: Office 4.04m (13 ft 3 in) Beam span: 16.76m

Beam depth: 610mm (24 in) Beam spacing: 4m (13 ft)

Slab: 133- or 159-mm (5.25- or 6.25- in) lightweight concrete on metal deck

Columns: Size at ground floor: 1067- by 610-mm (42- by 24 in) WF section, grade 350 MPa (50 ksi)

Spacing: 6.1 to 7.6m (20 to 25 ft) Core: Braced steel; column size at ground floor 1230 mm (48 in) square, 6308 sq m ([illegible])

Central Pla za

Hong K ong

Year of completion: 1992

Height from street to roof: 314m (1030 ft) Number of stories: 78

Frame material: Reinforced concrete

Number of levels below ground: 3

Typical floor live load: 3 kPa (63 psf) Maximum lateral deflection: 400mm, 50-yr return period wind

Type of structure: Perimeter tube and core

Typical floor

Story height: 3.6m

Beam span: 12m

Beam depth: 700-mm reinforced concrete Slab: 160-mm (5.5-in) reinforced concrete

Columns:

Size at ground floor: 2-m (6.5 ft) diameter

Spacing: 8.6-m (28-ft)

Material: Concrete, cube strength 60 N/mm2 (8500 psi)

Core: Shear walls 1.3m (4 ft 3 in) thick at base

Material: Concrete, cube strength 60 to 40 N/mm2 (8500 to 5800 psi)

@seismicisolation

Amoco Building

Chicago, Illinois, USA

Year of completion: 1973

Height from street to roof: 342m (1123 ft)

Number of stories: 82

Frame material: Structural steel

Number of levels below ground: 5

Typical floor live load: 4 kPa (80 psf) Frame material: Concrete

Maximum lateral deflection: H/400

Type of structure: Perimeter framed tube

Typical floor

Story height: 3.86

Truss span: 13.7m Truss depth: 965 mm

Beam spacing: 3.05m

Slab: 140-mm (5.5-in) lightweight concrete slab; 35 MPa (5000 psi) on 38-mm (1.5-in) steel deck

Columns: Folded plate, size not available

Spacing: 3.05m (10 ft) center to center

Material: Steel, grade 250 MPa (36 ksi)

Core: Structural steel frames carrying gravity loads only

@seismicisolation

John Hancock Center

Chicago, Illinois, USA

Year of completion: 1969

Height from street to roof: 344m (1128 ft) Number of stories: 100

Frame material: Structural steel

Number of levels below ground: 2

Typical floor live load: 2.5 kPa (50 psf) Maximum lateral deflection: H/500, 100 - yr return period

Type of structure: Diagonally braced perimeter framed tube

Earthquake loading: Not applicable

Foundation conditions: 41m (135 ft) of clay over dolomitic limestone

Footing type: 2.4-m (8-ft) diameter caissons on rock

Typical floor

Story height: Office 3.81m (12 ft 6 in); apartments 2.8m (9 ft 3 in)

Beam span: Office 15.2m (50 ft); apartments 13.7m (45 ft)

Beam depth: Office and apartments 610mm (24 ft)

Beam spacing: Office 3.05m (10ft)

Material: Steel, grade 250 MPa (36 ksi) Slab: 127-mm (5-in) concrete on metal deck

Columns:

Size at ground floor: 965mm x 965mm (38 by 38 in)

Spacing: 12.2m (40 ft)

Material: Steel, grade 250 MPa (36 ksi) Core: Not applicable

@seismicisolation

Sears Tower

Chicago, Illinois, USA

Year of completion: 1974

Height from street to roof: 443m (1454ft)

Number of stories: 110

Frame material: Structural steel

Number of levels below ground: 3

Typical floor live load: 2.5 kPa (50 psf)

Maximum lateral deflection: H/550, 100-yr return period

Type of structure: Bundled framed tube

Earthquake loading: Not applicable Foundation conditions: 18-m (20 ft) deep steel-lined concrete caissons

Footing type: Raft

Typical floor

Story height: Office 3.92m (12 ft 10.5 in) Truss span: 22.9m

Truss depth: 1016mm (40 in) Truss spacing: Office 4.6m (15 ft) Material: Steel, grade 250 MPa (36 ksi)

Slab: 63-mm (2.5-in) lightweight concrete on 76mm (3 in) metal deck

Columns:

Size at ground floor: 990mm x 910mm (39 by 24 in) Spacing: 4.6m (15 ft)

Material: Steel, grade 350 MPa (50 ksi)

Core: Not applicable

@seismicisolation

Taipei 101, Taiwan

Height

Architectural: 509.2 m (1671 ft)

Tip: 509.2 m (1671 ft)

Roof: 449.2 m (1474 ft)

Top Floor: 439.2 m (1441 ft)

Observatory: 449.2 m (1474 ft)

Technical Details

Floor Count: 101

Floor Area: 412500 m^2 (4440100 sq.ft)

Lifts/Elevators: 61

Architect: C.Y. Lee & Wang

CITIC Tower

Location: Beijing, China

Completed: 2018

Height: 527.7 m (1,731 ft)

Floor count: 109 (+8 below ground)

Floor area: 427,000 square metres (4,600,000 sq ft)

CITIC Tower is a supertall skyscraper in the Central Business District of Beijing. It is popularly known as China Zun

China Zun Tower is a mixed-use building, featuring 60 floors of office space, 20 floors of luxury apartments and 20 floors of hotel with 300 rooms, there will be a roof top garden on the top floor at 524 m high.

Guangzhou CTF Finance Centre

Height

Architectural: 530 m (1739 ft)

Tip: 530 m (1739 ft)

Top Floor: 496 m (1626 ft)

Technical Details

Floor Count: 111 (+5 below ground)

Floor Area: 507941 m^2 (5464633 sq ft)

Lifts/Elevators: 95

Tianjin CTF Finance Center

Height

Architectural: 530 m (1739 ft)

Tip: 530 m (1740 ft)

Top Floor: 439 m (1441 ft)

Technical Details

Materials: Composite

Floor Count: 97

Floor Area: 252144 m^2 (2714055 sq ft)

Lifts/Elevators: 81

@seismicisolation

	**One World Trade Center** Height Architectural: 541.3 m (1,776 ft) Tip: 546.2 m (1,792 ft) Roof: 417 m (1,368 ft) Top Floor: 386.5 m (1268 ft) Observatory: 386.5 m (1268.5m) Technical Details Floor Count: 94 (+5 below ground) Floor Area: 325279 m^2 (3501274 sq ft.) Lifts/Elevators: 73
	**Lotte World Tower** Height Tip: 555.7 m (1,823 ft) Top floor: 497.6 m (1,633 ft) Technical Details Floor count: 123 above ground, (6 below ground) Floor Area: 304081 m^2 (3237100 sq ft)

@seismicisolation

	**Ping An International Finance Center, China** Height Antenna spire: 599.1 m (1,965 ft) Roof: 555.1 m (1,821 ft) Top floor: 555.1 m (1,821 ft) Observatory: 562,2 m (1844 ft) Technical Details Floor count: 115, plus 5 underground floors Floor area: 385,918 m^2 (4,153,990 sq ft) Lifts/elevators: 80
	**Shanhai Tower, China** Height Architectural: 632 m (2,073 ft) Tip: 632 m (2,073 ft) Top Floor: 587.4 m (1927 ft) (Level 127) Observatory: 561.25 m (1841 ft) (Level 121) Technical Details Floor Count: 128 above ground, 5 below ground Floor Area: 380000 m^2 (4090300 sq ft) above ground, 170 m^2 below grade

@seismicisolation

Burj Khalifa

Height

Architectural: 828 m (2,717 ft)

Tip: 829.8 m (2,722 ft)

Top floor: 584.5 m (1,918 ft)

Observatory: 555.7 m (1,823 ft)

Technical Details

Structural system: Reinforced concrete, Steel, and Aluminum

Floor count: 154 + 9 maintenance

Floor area: 309,473 m^2 (3,331,100 sq ft)

Lifts/elevators: 57

@seismicisolation

Acknowledgements

I wish to express my sincere appreciation and thanks to my senior colleague Professor Mr. M. Ali, Ph.D., Fellow ASCE, Fellow CTBUH, Professor Emeritus of Structure, University of Illinois at Urbana-Champaign. The research work wouldn't have achieved without the support and advices from him. Without his encouragements, this modest contribution to structural engineering would not have been possible. I'm immensely grateful for the guidance and encouragement from him.

My sincere gratitude to my senior friend Engr. Anwar Hossain Akon for offering valuable suggestions and his encouragements. I am indebted to him for his confidence in me and will always remain true to his friendship.

I thank my individuals who extended their help and encouragement: Masud Khundoker (Ex-Ambassador in Qatar), Dr. Saiul Bari (Buet), Engr. Mahbub Haque (M.D. - BCL), Dr. Ainun Ninun Nishat (Professor Brac University), Dr. M. Shamim Z. Bosunia, Ex Professor of BUET, Ex- President of IEB, Dhaka. Engr. Sahidullah, Engr. Samsul Islam (KSA), Liton Kazi (IT Specialist), Engr. Said Chowdhury, Engr. Abu Saleh and Engr. Rezaul Islam (LGED).

A special note of thanks to Engr. Sivasubramanian Narayanan (G.M. - QDC in Qatar) for his encouragement. Also hidden thanks to QDC Library for the two books (stolen) which inspired me. My sincere appreciation and thanks to my ex-boss Ali Salem Hussain Alyafei (Head of Service Section, - ASHGHAL, Qatar) for his silent support in office during my research works in office hours.

I am also deeply grateful to the following individuals for their encouragements on this research: Engr. Mostak Hossain (Save the Children), Late Haider Jun Choudhory, Engr. Kausar Ahmed, Shariful Hasan (Prothom Alo), Mr. Asif (The Daily Samakal), Morshed Ahmed, childhood and unconditional friends Zahir & especially Masum.

My sincere appreciation and thanks to my wife, Sharmin Haque. Her patience and unyielding support, unconditional love, and congenial manner, always present over the many years of my book-writing career, have often left me astounded. Everlasting thanks to my wife and son, Faizan Tazwar Alam for their encouragements and love throughout the writing of this book that stole valuable time from my family life.

My sincere gratitude to my sister Zobeda Khanam (Chuby), and brother Ferdous Alam (Mona).

@seismicisolation

References

[1] Mir M. Ali (2001). ART OF SKYSCRAPER –THE GENIUS OF FAZLUR KHAN

[2] Joshua C. Feblowitz. Confusing The Wind: The Burj Khalifa Mother Nature, and the Modern Skyscraper

[3] Rizk A. S.-CTBUH Technical Paper, Structural Design of Reinforced Concrete Tall Buildings, Published CTBUH Journal issue I, 2010.

[4] Kareem, Kijewski & Tamura – Mitigation of Motion of Tall Buildings with Specific Examples of Recent Application.

[5] Bryan Stafford Smith & Alex Coull – Tall Building Structures (1991).

[6] Taipei mass damper – Wikipedia, the free encyclopedia.

[7] FRANCIS D. K. – Building Structures Illustrated- Patterns, System, and Design (Second Edition)

[8] Why Tall Buildings Often Considered safer than Low-Rise Buildings During Earthquakes? – CTBUH Journal 2014 Issue III.

[9] ASCE 7-10, Commentary Appendix C, Sec: CC.1.2.

[10] P. Jayachandran, Ph. D, M. ASCE – Design of Tall Buildings–Preliminary Design and Optimization.

[11] Robert Sinn 2012 – Taller: How Future Skyscraper Will Beat the Burj Khalifa.

[12] Kelsey Campbell-Dollaghan – Spire Shame: Why Today's Tallest Buildings Are Mostly Just Spire.

[13] CTBUH Journal, 2013 Issue III – Tall Buildings in Numbers. Vanity Height: the Empty Space in Todays Tallest.

[14] Tuned mass damper – Wikipedia, the free encyclopedia.

[15] Skyscraper– Wikipedia, the free encyclopedia. 1001

[16] STRUCTURAL SYSTEMS FOR TALL BUILDINGS-CTBUH, pages 1-4

[17] Lynn S. Beedle, Mir M. ALI & Paul J. Armstrong - The skyscraper and the City, book-2, page 487, 488)

[18] Christian Sandelin & Evgenij – The Stabilization of High-rise Buildings – An Evaluation of the Tubed Mega Frame Concept (1004)

[19] Design of Tall Buildings – Preliminary Design and Optimization – P. Jayachandran – 2009.

[20] P. Mendis, T. Ngo, N. Haritos, A. Hira, B. Samali & J. Cheung - EJSE Special Issue: Loading on Structures (2007) – Wind Loading on Tall Buildings.

[21] Structural Design of Reinforced Concrete Tall Buildings – CTBUH Journal, 2010 Issue. –(1002)

[22] Serviceability Limit States Under Wind Load – Lawrence G. Griffis – First Quarter / 1993. (1011)

[23] Richard Liew, J. Y., Balendra, T. and Chen, W.F. "Multistory Frame Structures" – Structural Engineering Handbook – Boca Raton: CRC Press LLC, 1999

[24] Abdelatee Abdalla Eljadei – Performance Based Design of Coupled Wall Structures – University of Pittsburgh 2012 -–(1005)

[25] Amit Urs – Stability Analysis of Frame Tube Tall Buildings, Oct, 2002

[26] Kyle Weiland – Lateral Load Behavior and Modeling of Low-Rise RC Wall For Performance-Based Design

[27] https://en.wikipedia.org/wiki/Skyscraper

@seismicisolation