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The Stabilization of High-rise Buildings

An Evaluation of the Tubed Mega Frame Concept

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Abstract

The Stabilization of High-rise Buildings. An Evaluation of the Tubed Mega Frame Concept

Christian Sandelin & Evgenij Budajev

Building tall has always been an expression of dreams, power and technical advancement. With the greatly increasing urbanization in recent years building tall has become a more viable option for office and residential housing. The Tubed mega frame concept tries to evolve the stabilizing systems of high-rise buildings with its mega frame around the buildings perimeter, created together with a new elevator system; the Articulated Funiculator. This thesis examines the effectiveness of the Tubed mega frame compared to other structural systems.

Information and background has been taken from different types of literature, analysis programs and verbally from supervisors; Fritz King and Peter Severin.

Using Finite Element Method (FEM-) programs studies on previously used structural systems along with the Tubed mega frame has been made, trying to draw conclusions about its advantages and drawbacks. The examinations have been done using SAP2000 and ETABS, both developed by CSI.

The tubed mega frame shows to require a large amount of concrete compare to other systems at lower heights, because of its geometry. As the height increases it does show an increase in effectiveness and by the time it reaches 480 meters it is using less materials and still achieving greater stiffness than other systems. Since the geometry of the Tubed mega frame is so flexible a conclusion is also made that the stiffness can be increased by sacrificing façade area or creating longer outriggers.

Key words: Articulated funiculator, Tubed mega frame, high-rise

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PREFACE

This thesis has been done at Tyréns, Stockholm. Tyréns is one of Sweden's leading consulting companies for the built environment.

Huge thanks to:

Tyréns; for the opportunity to write our thesis, utilizing Tyrén's many resources.

Our supervisors Peter Severin and Fritz King (Tyréns); for guidance and the enduring of all our questions, however big or small they might have been. Never stop being superheroes.

Per Isaksson our subject examiner and our examiner Patrice Godonou; for valuable input and suggestions.

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NOTATIONS

$\mathrm{A} = \text{Cross section area}$
$E = Youngs\ modulus\ of\ elasticity$
$I = Moment\ of\ inertia$
$EI = Stiffness, equivalent\ stiffness$
$M = Moment$
$M_0 = Initial\ moment\ (1st\ order)$
$M_{Rd} = Moment\ capacity$
$M_{tot} = Total\ bending\ moment$
$\Delta M = Moment\ from\ 2nd\ order\ effects$
$N, P = Axial\ force$
$N_{Cr,B} = Critical\ buckling\ load$
$N_{Rd} = Axial\ force\ capacity$
$\varepsilon = strain$
$\varepsilon_{cu} = concrete\ strain$
$\varepsilon_s = steel\ strain$
$\sigma = stress$
$\sigma_s = Steel\ stress$
$f_y = Steel\ strength$
$f_{ck} = Concrete\ compressive\ strength$
$W = Bending\ resistance$
$\varphi_{M0}, \varphi_{M1} = Axial\ capacity\ reduction\ coefficient$
$T = Time\ period$
$f = Frequency$
$f_0 = Natural\ frequency, Eigenfrequency$
$h, H = Height$
$b = Width$
$t = Thickness$
$V = Shear\ force$

1 INTRODUCTION

1.1 Background

With today's increased need for housing in big cities, there is a growing need for high-rise buildings. Tall, slender buildings are wanted for big cities where space is limited. The development of these buildings has not kept pace with the development of many other systems in society.

Tyréns has developed a new idea for improving the existing models of skyscrapers in the form of a new elevator system, known as "Articulated Funiculator", with a completely new structural system called the “Tubed mega frame”. What makes the Tubed mega frame unique is that all the load is carried down through the giant pillars of the building's outer edge instead of as in many of today's high-rise buildings where the a big part of the load is carried down through a central core. However, this system is only in an initial idea state and therefore needs more investigation before it is fully applicable.

1.2 Aim

Examining how the effectiveness of the Tubed mega frame stands against existing models such as the "Outrigger system", and the "Tube in a tube".

1.3 Limitations

A way to measure stabilizing efficiency is to look at a buildings bending resistance against wind loads. Looking at a buildings natural frequency, deflection and drift can be a way to analyze its stiffness. There are other ways to determine how effective a structural system is but these three are the ones used in this report.

1.4 Case

The goal is to produce a report about the efficacy of the Tubed mega frame which can be used in marketing and/or further research of the concept. The report should also work as a basic guide on high-rise systems and design.

1.5 Methodology

The report will start with an extensive literature study, first describing the historically most widely used systems for stabilization of tall buildings, wind dynamics and structures natural frequencies. These will be based on literature studies and knowledge given by our supervisors; Peter Severin and Fritz King.

Then the report will move on to calculations in the form of modeling and analysis, using finite element method programs like Frame Analysis (Strusoft), SAP2000 and ETABS (CSI). A further study and comparison will be made on used structural systems before moving on to the Tubed mega frame concept.

A number of rough models with the concept of Articulated Funiculator and the Tubed mega frame are already produced by Tyréns. One of them is selected to be remodeled to less number of stories to save time.

Finally, the report ends with results and a comparison between the Tubed mega frame, Outrigger and Tube in a tube system.

2 TALL BUILDINGS

There is no clear definition of what a tall building is, but according to the council of tall buildings and urban habitat it should have one of the following elements to be considered a tall building;

- *Height relative to context*: when a building is distinctly taller than an urban norm
- *Proportion:* a building that is slender enough to give an appearance of a tall building
- *Tall building technologies:* the building contains technologies that are a product of the buildings height, such as specific vertical transportation technologies and structural wind bracing.

From a structural engineers perspective a building would be considered tall when lateral loads, i.e. wind or earthquakes, play a significant part in the buildings structural design. (Coull & Smith, 1991)

2.1 The evolution of modern high-rise buildings

After the Great Chicago Fire in 1871 that left a big part of downtown Chicago empty, higher buildings started to emerge. There was a big demand for office space and land was expensive, investors expected maximum usage. With limited space around buildings the only way to go was vertical.

A lot of inventions helped make the high-rise buildings functional, such as the telephone and the elevator. Earlier it had been difficult to rent out space above the fifth floor because of the tiresome walk up and down staircases. When Elisha Otis invented a self-braking elevator vertical transportation was possible. However, this transportation was very slow until improved by Werner von Siemens in 1880 with his electrically powered elevator. The telephone, invented by Alexander Graham Bell, made it possible for people to communicate without talking face to face, in turn making office jobs more stationary.

The biggest contribution for the structure itself came perhaps from Gustave Eiffel who had demonstrated iron as a useful building material. Soon after, the iron skeleton concept was born. As seen in one of the first high-rise buildings, the Monadnock in Chicago from 1891, the use of masonry in such a building is not very effective in terms of the floor space it inhabits with the masonry being six feet thick. The change actually came earlier when constructing the Home Insurance Building five years earlier, also in Chicago. The Home insurance building used an iron skeleton in collaboration with masonry walls to create the world's first skyscraper. The Home Insurance building was never the tallest building in the world but considered the first skyscraper because of its iron skeleton. The iron frame was something that people didn't think was possible and the building had to shut down for a while during its construction. In this building the iron was used to handle the gravitational loads but today it's well-used for horizontal loads as well. Intentional or not, the masonry was the construction material handling horizontal loads in the Home Insurance Building.

It was perhaps the Monadnock that was the first tall building recognizing the effects of wind loads with its iron portal frame between the east and west side of the house. Iron, then steel, and its development led to new heights in construction possibilities. New York soon took over as the capital for high-rise buildings in the early 20th century with buildings like the Singer tower, Woolworth building and later Empire State Building.

Even though 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. It was not until the second half of the 20th century that it was being used as the primary part of a buildings structural system. This was because of earlier high-rise structures used the steel frame for stability until Fazlur Rahman Khan invented the tubular design.

Khans tubular design didn’t only allow for concrete as being a profitable option, it also made for less material needed even in steel high-rise buildings. With the new design buildings like the John Hancock Center, Willis- (formerly Sears) Tower and World Trade Center showed up in the US, these new buildings set the record for tallest buildings on earth at the time.

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.

One of the biggest issues in high-rise buildings is the elevators. The elevator itself hasn't changed that much since it became electrical, the speed and size may have increased some but we still see that an increase in floors also means more elevator shafts and more time spent in elevators. Solution for this has been more logistical than technical, making elevators move to a specific span of floors instead of up and down the whole building. Another thing Fazlur Khan invented for this matter was the sky lobby, specific floors that would have express elevators traveling directly there from the bottom of the building. People would then change to a regular elevator that took them to their wanted level. Today, vertical transportation in high-rise buildings is still one of the biggest issues in design; you want to have a lot of usable floor space while still maintaining acceptable travel time.

As the structural systems and materials in high-rise buildings have evolved so has the usage. From being almost always used as office buildings in the nineteen-hundreds to more and more residential usage in the 21st century.

Since the beginning, high-rise buildings have been the architectural expression of dreams, power and economic wealth. The race for taller, slender and more efficient high-rises continues however, with the need for even better elevators, construction materials and structural systems to reach new heights.

2.2 **Acting loads**

A building is exposed to a large number of different loads. They can be static or dynamic, come from outside or inside of the building. Simple categorization of them may be based on its direction; vertically or horizontally. Vertical loads, also known as gravity loads, generally consist of self-weight, live load and snow loads. Horizontal, or lateral loads, may occur in the form of wind load, tilt and seismic responses. Generally, the size of all these loads increases somewhat linearly with number of stories. The growth of the wind load on the other hand evolves differently and its effect intensifies rapidly with an increase in height. It is also the one which in most cases will be essential in the design of tall buildings - wind load as the main load.

A noticeable effect of the horizontal towards the vertical loads is illustrated in Figure 2.1. The need for material used in stabilizing the acting horizontal load increases dramatically as the number of floors reaches above 40-70. The amount of material for stabilization of the gravity loads, however, is proportional to the number of floors.

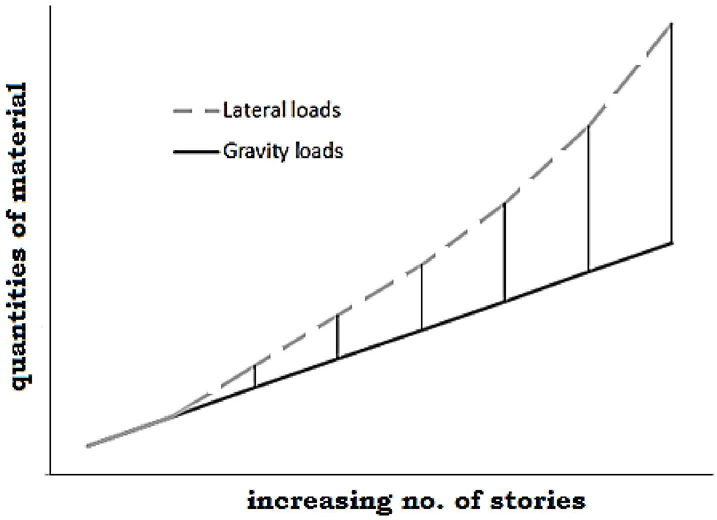


Figure 2.1 Cost of height diagram. The material used to resist gravity loads increases proportionally with the number of stories while the materials used for stabilization has an exponential increase (Khan, 2004)

While action of lateral loads is orthogonal to the building, which effect negatively on the building's stability as lateral displacement, overturning and twisting, gravity loads appear in the building's own direction and in that way to some extent, have a positive effect on the stability.

In all cases, regardless of the direction of the loads the building's main job is to transfer these loads to the ground. On the way down, different scenarios in the form of instability or breakage can occur in parts. Where exactly these instabilities occur depends a lot on the selected stabilization system.

Possible scenarios of instability due to static lateral loads are lifting and sliding. Figure 2.2 is displaying what may happen when a building has sufficient or non-sufficient resistance towards these scenarios. I.e. Insufficient shear resistance means that there will be horizontal movement of the floors, known as sliding, shown in Figure 2.2d. Lifting of the stories, shown in Figure 2.2b, happens because of insufficient bending resistance. The same fate may affect the entire building if modular or shear resistance is weak in the foundation, the building's connection with the ground. The building will then be deformed even though it has sufficient resistance against breakage, Figure 2.2a and c. The size of the deformation will depend on the buildings stiffness.

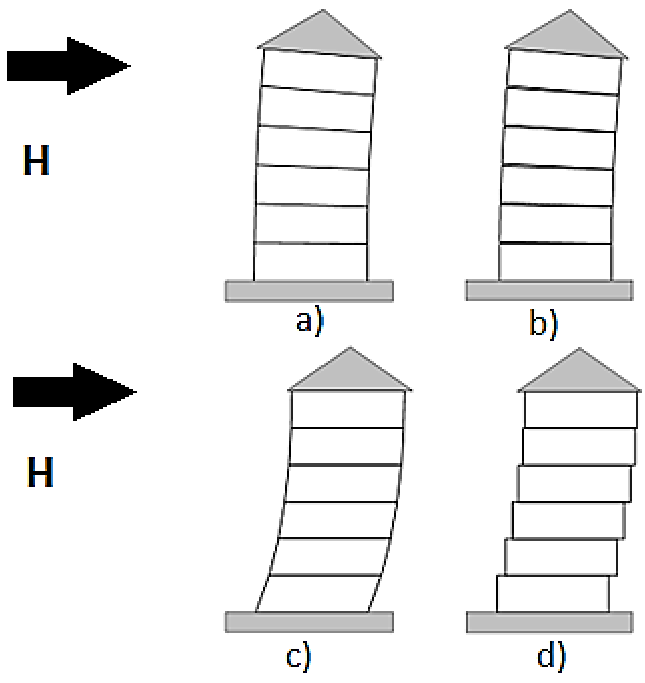


Figure 2.2 Scenarios due to lateral loads.
a) and b) is showing sufficient and non-sufficient bending resistance.
c) and d) is showing sufficient and non-sufficient shear resistance

2.2.1 *Wind*

The interaction between wind and a structure creates many different flow situations because of the winds complexity. Wind is composed of eddies that gives wind its gustiness, its turbulent character. The gustiness decreases with height but the wind speed over a longer time period increases. Due to that wind behavior is varies in time, i.e. dynamic, it will result in the magnitude of the static wind load on the building will vary.

Pressure that is created from acting wind is dependent on the geometry of the building and the nearby structures as well as the winds characteristics. Wind pressure is highly fluctuating with unevenly distribution over a structures surface. Fluctuating pressure can result in fatigue damages.

A structure that is affected by wind deforms. Since wind is dynamic the building will sway as the force from wind shifts. How the structure sways is dependent on its natural frequency, see Chapter 2.3, which in turn is dependent on its mass and stiffness. If the wind hits the structure at the same frequency as its natural frequency its sway will increase drastically, possibly leading to collapse of the structure.

In addition to the effects that occur in the wind direction the wind can affect a structure in its perpendicular direction. This can particularly happen to high and slender buildings. The reason for the effect is that wind at high speed spread first on one side of the structure and then on the other, instead of spread to both simultaneously, forming forces in the winds transverse direction as eddies and vertices. The phenomenon when a wind creates oscillations both in the winds along direction as well as in its transverse direction is called vortex shedding, Figure 2.3.

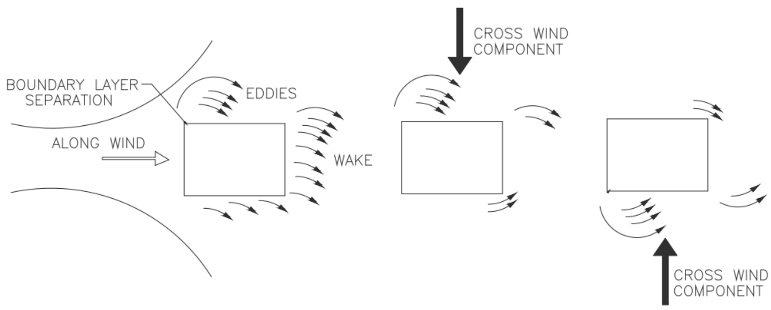


Figure 2.3 Vortex shedding

Another negative effect of the building´s dynamic response, when it is swinging, is that the horizontal acceleration at the top of high-rise buildings can reach very high levels. As a human is more sensitive to horizontal acceleration than vertical, preventing the acceleration in high-rise buildings may be of greater importance than decreasing the deformations. This also correlates to the mass and stiffness of the structure as well as its natural frequency.

Designing for wind load is done either by using coefficients in the wind loading code or by doing a wind tunnel test. Wind tunnel tests are often used when a structure has an uncommon shape or very flexible. Every tall building today undergoes a tunnel test during its design. Testing structures can get a more realistic load and perhaps reduce cost in design. (Hussain, 2010)

2.2.2 *Tilt*

Tilt, as mentioned earlier, is one of the lateral loads acting on a building. Its origin comes from the fact that in a column-system structure the columns have certain geometric imperfections. Under construction they can't be positioned totally vertical as in a perfect analysis model, therefore the vertical loads, V, acting on columns result an additional moment which negatively effects the stability of the building. Influence of the columns tilt can be replaced by an equivalent lateral load, H_{tilt}, shown in equation (2.2.1) and Figure 2.4. The equivalent lateral loads caused of the tilt can be calculated by each story and then be used in analysis of the building's overall stability and applied in the design of each column.

$$H_{tilt} = (V_1 + V_2) \cdot \theta_i \qquad (2.2.1)$$

$\theta_i -$ *unitended incination*

For concrete columns the initial tilt, θ_0, is calculated by (2.2.2):

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \qquad (2.2.2)$$

$\theta_0 = 0{,}005$

$\alpha_h = \frac{2}{\sqrt{l}} \qquad 2/3 \leq \alpha_h \leq 1$

$\alpha_m = \sqrt{0{,}5\left(1 + {}^{1}/_{n}\right)}$

$n -$ *the amount of columns*

For steel columns the initial tilt is calculated by (2.2.3):

$$\theta_i = \theta_0 + \frac{\alpha_\delta}{\sqrt{n}} \qquad (2.2.3)$$

$\theta_0 = 0{,}003$

$\alpha_\delta = 0{,}012$

$n -$ *the amount of columns*

(Engström, 2007), (Johansson, 2009)

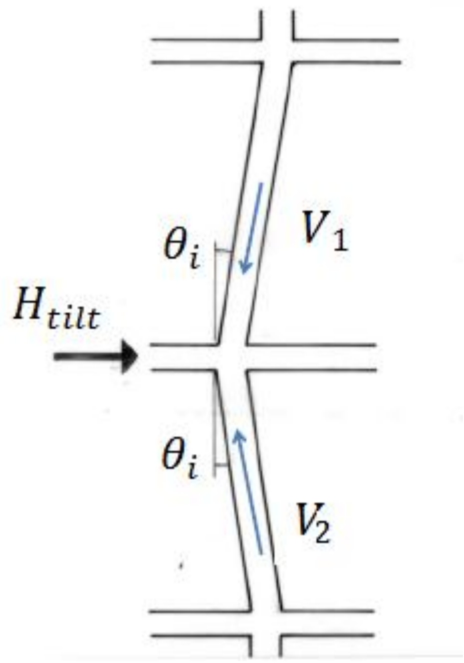


Figure 2.4 Equivalent lateral force because of tilt

2.2.3 *Seismic forces*

Plate movement in the earth's crust causes earthquakes that occur as vibrations. These vibrations move as waves with force components in every direction, the ones that are most dangerous for buildings are the horizontal components. Vertical components from earthquakes are often small enough to be taken care of by a structures vertical load resisting elements. Horizontal waves can be either P-waves, also known as compression waves, or S-waves, shear waves. Since these shake along different axis's ground motions can occur along any horizontal direction.

Inertial forces try to prevent a building from moving when hit by seismic forces. Ground movement causes the structure to move at the base, creating a lateral force in the form of shear. The inertial forces respond with a force being equal to Newton's second law as mass time's acceleration. The shear force is distributed from top to bottom of the structure with its maximum at the structures highest point. Equivalent base shear can be calculated by equation (2.2.4) using the coefficient C_s which is based on soil profile, ground motions, fundamental time period, stiffness and the structures distribution of mass. This is done for regular and low structures or structures with a low seismic risk. When it comes to tall buildings a more complex analysis is required.

$$V = C_s \cdot W \qquad (2.2.4)$$

$V = Equivalent\ shear$
$C_s = seismic\ design\ coefficient$
$W = total\ dead\ load\ of\ structure$

The natural frequency, or fundamental time period, is relevant to check and compare to the time period of the seismic event. Soft soil makes for greater shaking and a longer time period while hard soil tends to have less shaking and shorter period. If the structures period and the period from the earthquake match each other resonance is created. (Ching, Onouye, & Zuberbuhler, 2009)

2.3 **Natural frequency**

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. There are a number of approximate methods. Two of them are somewhat easier to understand.

The easiest method (2.3.1) 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.

The variation of the mass and stiffness between floors can, however, be considered in the second method (2.3.2). The stiffness is baked into the stories deformations, as these are strongly linked. The size of a deformation depends on the magnitude of a load, therefore the size and variety of loads must be known. A major advantage with this method is that the natural frequency can be obtained for all possible modes.
(Harris & Crede, 1976), (Coull & Smith, 1991)

$$f_o = \frac{1}{2\pi}\sqrt{\frac{3EI}{0{,}23mL^4}} \qquad (2.3.1)$$

$$f_o = \frac{1}{2\pi}\sqrt{\frac{g\sum_{i=1}^{n} F_i w_i}{\sum_{i=1}^{n} W_i w_i^2}} \qquad (2.3.2)$$

E – *the elasticity modulus*

I – *the moment of inertia*

m – *the mass per unit length*

L – *length of the cantilever*

g – *the gravity acceleration*

F_i – *the transverse load acting on the slab floor i*

w_i – *the static horizontal deflection of the building on the floor i*

W_i – *the vertical load on the floor i*

2.4 **Stabilization**

The stabilization of a high-rise building can be divided into different subsystems:

- Floor systems
- Vertical load resisting systems
- Dampening systems
- Lateral load resisting systems

2.4.1 *Floor systems*

The floor systems primary task is to resist the gravitational loads on them but they should also provide fire resistance, sound dampening, housing for ventilation and more. They may also help providing stability in resisting lateral loads by distributing lateral loads to vertical resisting elements and by connecting different systems together. Floor-types that are used in high-rise buildings are concrete floor systems and steel floor systems. Concrete floor systems consist of a reinforced concrete plate resting on supports. Prefabricated plates are most common today because of its effectiveness in production. The steel floor systems uses concrete and steel together either by having a concrete floor resting upon steel beams or by having metal decking with concrete above. (Jayachandran, 2009)

2.4.2 *Vertical load resisting systems*

To resist the vertical load a building uses columns, bearing walls, beams, hangers and cables. In high-rise buildings these are made up from structural steel, reinforced concrete and composite materials. When building tall the vertical load usually is not the biggest concern, it can even help in terms of dampening and prevent overturning.

2.4.3 *Dampening systems*

Damping is a measure of the rate at which the energy of the motion is dissipated. Higher damping means the motion is better reduced; making the building feel more stable to its occupants. To prevent the motion in high-rise buildings a dampening system, or damper, may be required. These can be divided into active, semi-active and passive. While active systems has great respond to lateral loads passive systems are often preferred because of their cost effectiveness and their ability to work during seismic events. Active systems and semi active systems require a power source and should not be used when power supply may be irregular. High-rise buildings that use reinforced concrete often have enough mass, not needing a dampening system. Some buildings, like the Taipei 101 which has a steel stability system, do require a damper however, Figure 2.5. One of the most used dampers for very tall structures is the tuned mass damper, an active system consisting of huge steel or concrete bodies. (Ching, Onouye, & Zuberbuhler, 2009)

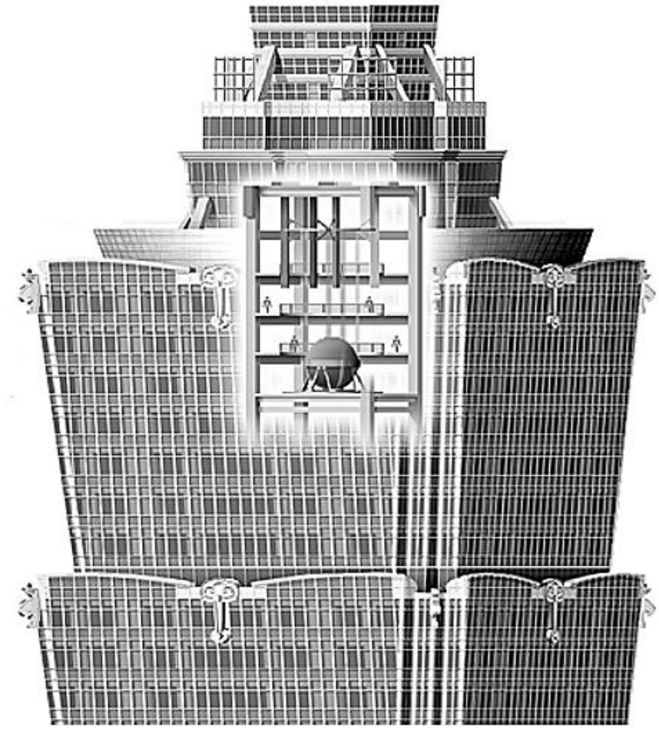


Figure 2.5 Taipei 101's largest tuned mass damper
(Wikipedia, 2013)

2.5 **Lateral load resisting systems**

Lateral load resisting systems are structural elements which resist seismic, wind and eccentric gravity loads. There are a lot of different systems but they can be broken down to three fundamental ones which all other systems are a combination of. They are:

- Shear walls
- Moment resisting frames
- Braced frames

2.5.1 *Shear walls*

Shear walls, Figure 2.6, are often defined as vertical elements of the horizontal load resisting system. Walls of steel or timber frames with a board fastened to it are included in the definition. Masonry shear walls are also used, with solid walls and grouted cavity masonry with reinforcements encased. However, considering high-rise building shear walls are more associated with reinforced concrete walls.

Shear walls generally start at foundation and are continuous throughout the building height. The walls provide large strength and stiffness to buildings in the direction of their orientation, mostly due to its large cross-section area that provide great moment of inertia, which significantly reduces lateral sway of the building. The reason to make big elements of reinforced concrete instead of other materials, e.g. steel which would give even more stiffness, is its much cheaper cost.

The location of the shear walls is different depending on the buildings intent to be either an office or a residential building. The need for shear walls in residential buildings occurs mostly between apartments because of higher demands for sound and fire resistance resulting in a web of smaller bearing walls. In office buildings there is a huge requirement for flexible floor plans and for a large amount of elevators and stairwells which makes it more sufficient to concentrate shear walls around them. A box-type structure of shear walls rigidly connected has greater stiffness by acting as a tube, seen in Chapter 2.5.5. (Council of tall buildings and urban habitat, 1994)

To stabilize a building against lateral loads from different directions a simple rule for the amount and placement of shear walls is that there should be at least three of them and they may not intersect at the same point or be only parallel to each other. (Severin & Sekic, 2012)

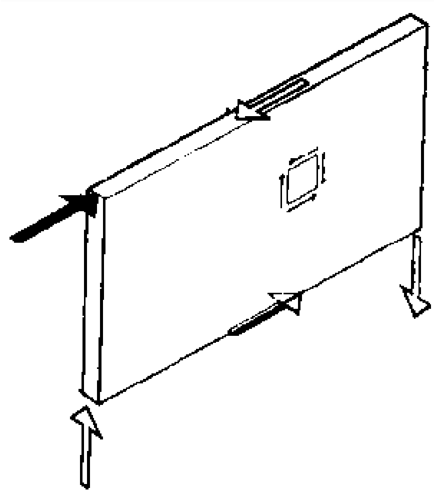


Figure 2.6 Responding forces of a shear wall with opening (Ching, Onouye, & Zuberbuhler, 2009)

2.5.2 *Moment resisting frame*

Moment resisting frame, also called moment frame or rigid frame, is made by rigid connections between horizontal and vertical members. Steel, reinforced concrete and steel-concrete composite rigid frames are used. In earlier high-rise buildings, while concrete were under development, steel frames were predominated. The combination of steel and concrete has evolved in recent times which offer the ability to quickly build the framework of steel and then incasing concrete into the frame to increase its stiffness and weight. The higher weight and stiffness improves the damping and the axial strength.

The lateral deformation of rigid frame depends mostly on shear sway but also on column shortening see Chapter 4.3. Its resisting of lateral loads includes primarily by the flexural stiffness and strength of members and joints. Number of stories, story height and column spacing has proportional influence on the frame's strength and stiffness. Larger bending moments appears in the lower levels with its maximum in the connections, shown in Figure 2.7. As building stories increase so does the bending moment both in beams and columns. Columns usually get bigger from top to bottom with respect to increasing gravity load. They can therefore withstand the increased moment while the beams are subjected to the same gravitational loads but needs to be resized to manage the increased moment. (Council of tall buildings and urban habitat, 1994)

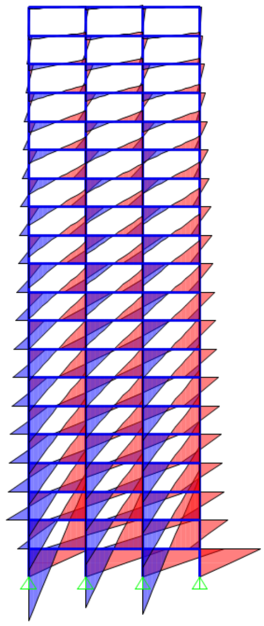


Figure 2.7
Bending moments for
a rigid frame

Why the bending moment increases in columns with increasing number of stories is easy to understand while moment increasing in beams is a little bit harder. By applying only one wind-load on the top of the building it is simpler to understand the principle of the developed force and the transmission through the buildings elements. As the force is applied at a distance from the ground there will be a moment that by the equation of equilibrium results as vertical reaction forces in the supports, as tension and compression. There will also be horizontal reaction forces that prevent the lateral movement of the building.

Vertical and horizontal reaction forces gives axial and shear loads at respective column at the supports. Without any beams in between these forces would transmit to the top unchanged. In the nodes the axial and shear forces would shift to shear and axial forces in the beam. As shear forces at the beam-ends act in different directions as the rotation of the beam would appear, there will be a moment to prevent it. The moment will be highest at the connections and has a contra flexure point in the middle of the beam.

The floor-beams at each story will act as the beam on the top. The axial forces in columns will be reduced proportionally between the stories caused of shorter lever arm, see Figure 2.8. As the reducing is the same between the stories all beams will be exposed for the same shear forces and therefore develop the same moment distribution and size. By adding one wind load at the time on each floor, from the top down, the moment in the beam below the applied floor will increase progressively.

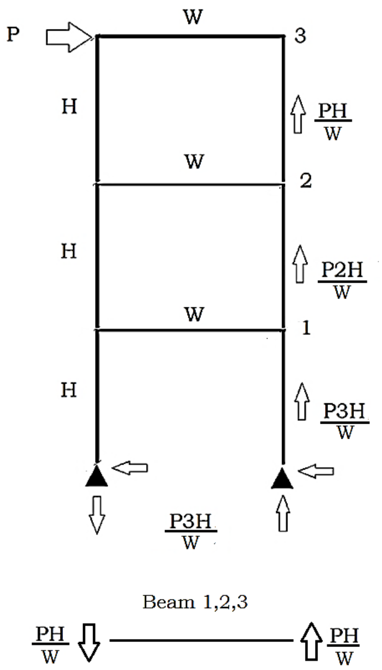


Figure 2.8
Distribution of axial forces in columns and shear forces in beams

Shear forces in the beam between stories 3 and 2;

$$\frac{P \cdot 2H}{W} - \frac{P \cdot H}{W} = \frac{P \cdot H}{W}$$

Shear forces in the beam between stories 2 and 1;

$$\frac{P \cdot 3H}{W} - \frac{P \cdot 2H}{W} = \frac{P \cdot H}{W}$$

When the number of stories exceeds of about 30, huge dimensions of frame beams and complex connections are required that the usage of a moment resisting frame gets uneconomical.

The prime advantage of this system is the flexibility regarding windows and doors which is very good. That is one of the reasons why the moment frame is not abandoned, instead quite the opposite, often used combined with other systems in high-rise buildings.

2.5.3 *Braced frame*

Bracing is another way to take care of horizontal loads. The simplest method is to place a diagonal brace, nodes are designed as leads. The transfer of horizontal loads down to one of the supports takes place in the braces direction in the form of either axial tension or compression depending on the direction of the horizontal load. This means that the axial stiffness of the frame members is what is resisting lateral loads. When subjected to a horizontal load, in an X-brace, one of the diagonals will be subjected to compression while the other is in tension, shown in Figure 2.9.

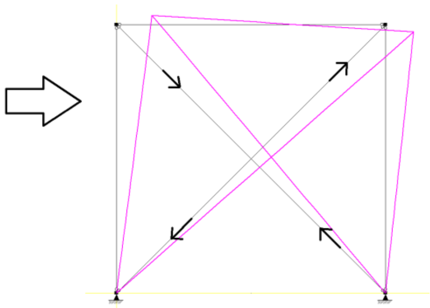


Figure 2.9 Forces in the diagonal braces when subjected to lateral load

There are many different types of bracing. While the most common and one of the most effective is the X-bracing, this takes a lot of space in the structure which makes little room left for openings. V,K, diagonal- and knee bracing are other types that is often used, these provide better room for openings but are less effective against horizontal loads. There are also eccentrically braced systems that provides different shapes and openings, they have good ductility for resisting seismic forces but provide less stiffness than the concentric braced frame. A few regular shapes are shown in Figure 2.10.

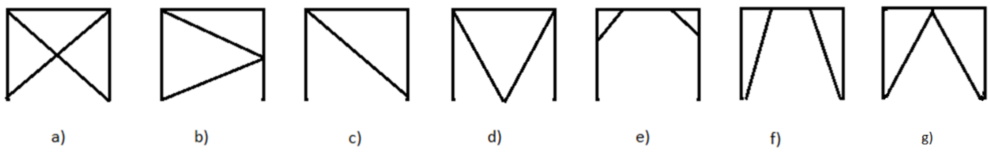


Figure 2.10.a) X-brace b) K-brace c) diagonal brace d) V-brace e) knee-brace f) eccentrically-braced g) Chevron brace

Braced frames, or vertical trusses, are often used in the core of high-rise buildings where it can be enclosed to form cells that are effective in resisting torsional forces, see Chapter 2.5.9. When subjected to a lateral load the truss shows a cantilever parabolic sway. In itself the braced frame is not economical if the buildings size is above 20 to 30 floors but combined with another system, such as the moment frame, it can become more effective.

While using a completely braced frame system in a high-rise building, as shown in Figure 2.11, the stability is very good. This system has a few major drawbacks however. The weight of the building when completely braced becomes massive, with a lot of different pieces to fit together. Another drawback is the limitations in terms of space for windows and doors, the bracing means the openings have to fit in accordingly which also means that the ability to form an architectural expression would be less.

Other type of ways using the bracing can be to just have one part of the building with a vertical truss and the outer connections as leads, as seen in Figure 2.11. This provides the same sway as before, but it will deflect more and earlier.

If combining the frame with only one vertical truss in a part of the building with braced arms reaching the outsides of the structure, see Figure 2.11, the new system would show of a different sway. Up until the first pair of arms the frame would sway like usual, but when reaching the arms the bracing is then making the buildings deflection less compared to the floor below. Above the arms the building is starting to sway as a cantilever again until reaching the next pair of arms, see figure 2.6. This system is what would later form the belt truss and the core + outrigger system when looking at it in three dimensions, see Chapter 2.5.11.

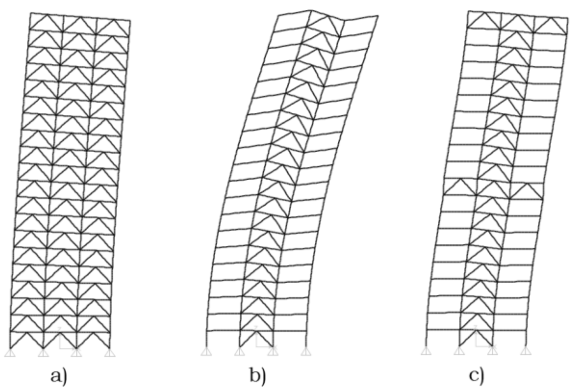


Figure 2.11
Deflection-shapes when exposed to a horizontal load,

a) Completely braced frame
b) Partially braced frame
c) Partially braced frame with outriggers

2.5.4 *Braced rigid frame*

If the braced frame, or shear walls, and a rigid frame are combined, it produces a greater amount of lateral stiffness. This is because of the way the two systems react to the horizontal loads. With the moment frames shear deformation and the bracing's bending deformation the combined deformation is more efficient, as shown in Figure 2.12. Instead of continuing to bend at the top the rigid frame keeps the shear wall or braced frame in place, while at the bottom the bracing, or wall, is restraining the shear deformation of the moment frame. This results in a deflection with an "S" shape.

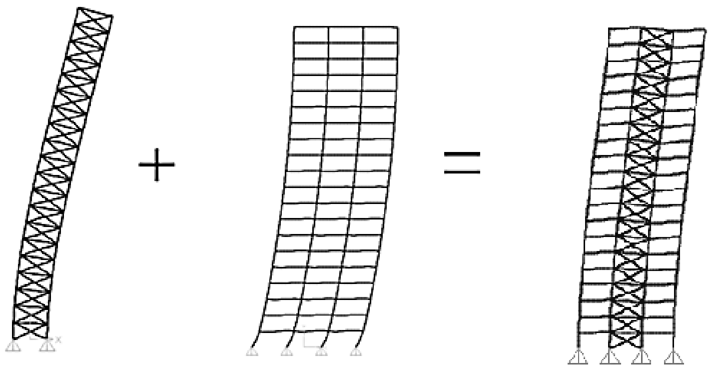


Figure 2.12 A braced frame combined with a rigid frame will decrease a buildings deflection

Instead of experiencing a maximum bending moment at the bottom of the building this now happens in the middle of the structure. This moment is also much smaller than the moment of the rigid frame system alone. A braced rigid frame structural system is most efficient when between 40-60 stories.

2.5.5 *Tubular design*

In the 1960's Fazlur Khan discovered that the steel frame systems was not the only way to stabilize high-rise buildings. By looking at a building as a vertically standing hollow box, cantilevering out of the ground, he discovered the tube design. As a theoretical idea he pictured the walls around the building being solid, later adding openings for windows for a workable application. This analysis showed that even with openings this structural form would provide a lot in terms of lateral resistance. This is because when the walls are connected as a box it will fully utilize the outer perimeter walls in every direction; see Figure 2.13 and following page.

Elementary beam theory indicates that the elements farthest away from the central axis will be the most utilized in supporting the structures bending loads and obtaining greater stiffness. Along with providing lateral stiffness the perimeter is often designed to take a larger part of the vertical load than before. With more vertical load in the perimeter the buildings ability to resist overturning increases. Khan's discovery of the tube offered a few new variations such as the framed tube, trussed tube and the bundled tube. (Khan, 2004)

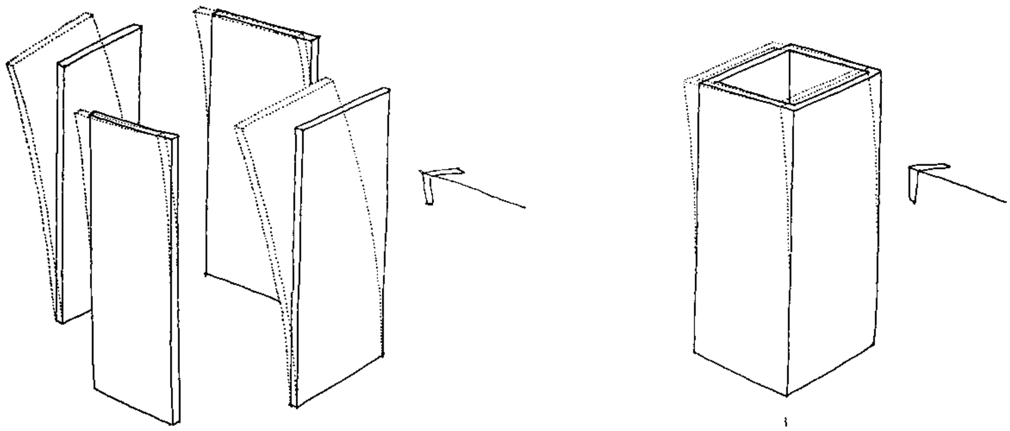
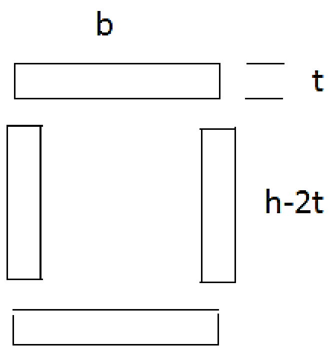


Figure 2.13 When subjected to wind loads, unconnected walls will bend around their weak axis offering little resistance. If connected to a tube, the walls will participate together in resisting the load. The effectiveness will increase significantly (Khan, 2004)

Moment of inertia if walls are connected

By not adding the walls together the moment of inertia will simply be each individual element added together as in Figure 2.14 and equation (2.5.5.1)



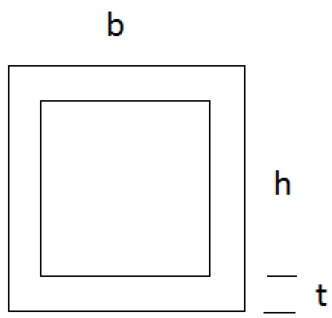
$$I = 2 \cdot \frac{tb^3}{12} + 2 \cdot \frac{(h-2t) \cdot t^3}{12} \quad (2.5.5.1)$$

For example: $h = b = 10, t = 0,25$

$$2 \cdot \frac{0,25 \cdot 10^3}{12} + 2 \cdot \frac{(10 - 2 \cdot 0,25) \cdot 0,25^3}{12} = 42\ m^4$$

Figure 2.14 When walls are disconnected I is calculated for each element alone and then added together

If added together the walls will create a bigger moment of inertia. This can be shown by calculating it as a solid and subtract the empty space, see Figure 2.15 and equation (2.5.5.2)



$$I = \frac{hb^3}{12} - \frac{(h-2t) \cdot (b-2t)^3}{12} \quad (2.5.5.2)$$

For example: $h = b = 10, t = 0,25$

$$\frac{10 \cdot 10^3}{12} - \frac{(10 - 2 \cdot 0,25) \cdot (10 - 2 \cdot 0,25)^3}{12} = 155\ m\text{^}4$$

Figure 2.15 With the walls connected a hollow box is created

The second equation will always have a bigger moment of inertia and the difference will increase as the width, t, decreases. In the example given the moment of inertia increases almost three times when the walls are connected.

2.5.6 *Framed tube*

Perhaps the first real application of the tube system came as a logical extension of the moment resisting frame with closer placed columns and spandrel-beams. Because the framed tube is partially a solid tube and partially a rigid frame it reacts similar to that of the combined system of a shear wall and rigid frame. Since it partly reacts the same way as a rigid frame it is partly resisting the lateral loads in forms of shear at each floor by bending and shearing in the beams. Story shear is however not optimally distributed among the columns in the frame. This phenomenon of incomplete transfer is known as shear lag and even though the frame doesn't carry over the bigger part of the shear, the shear lag causes approximately 70 percent of the buildings total deflection. With shear lag the axial stresses in the corner of the building will increase while the stress in the more centered columns will be less, according to Figure 2.16.

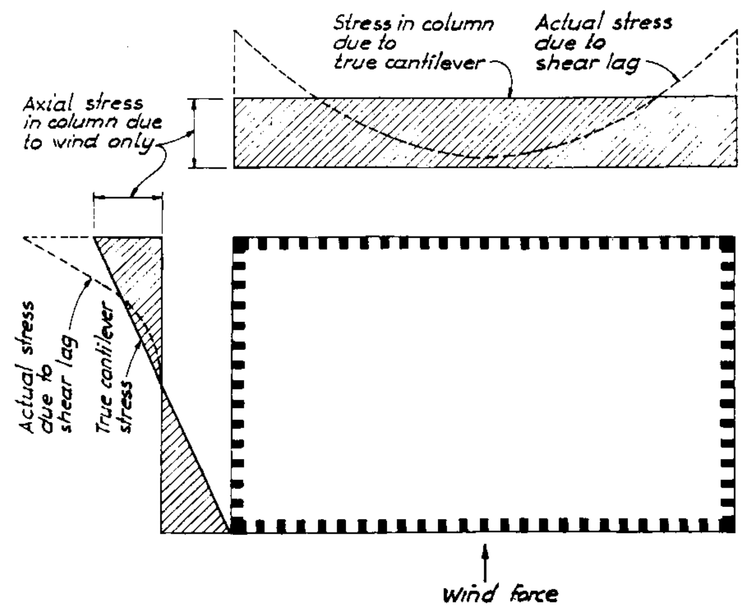


Figure 2.16 Stress diagram of a framed tube showing the ideal stress for a tube (solid) and that of the tube affected by shear lag (hollow) (Khan, 2004)

2.5.7 *Trussed tube*

With the shear lag and the way that the framed tube was consisted of a tightly spaced column-beam system it became apparent it was not applicable beyond a certain height. Building higher would require a smaller and smaller distance between the column and beam spacing. By having a minimum number of big diagonals on the building's façade who intersected with each other at the same place on the corner column a new system were founded. This made the building act as a tube since the diagonals formed a box around the building. Compared to the framed tube this system made it possible for greater spacing between columns, which made space for larger windows. The trussed tube also offered better redistribution of gravitational loads, making the load in the exterior columns more equalized, se Figure 2.17. Because of the systems bracing around the whole building the problem made from shear lag is greatly reduced.
(Khan, 2004)

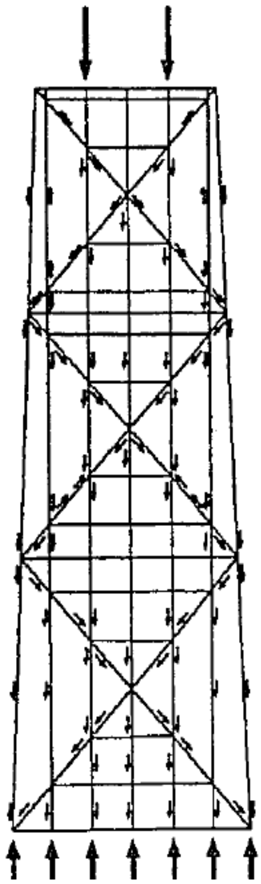


Figure 2.17
Distribution of gravity load in a trussed tube
(Khan, 2004)

Diagrid structures

A further extension of the trussed tube can be seen in diagrid structures. Structural members are placed in a diagonal grid, consisting of almost no vertical columns. The diagonal members carry almost all gravity loads and all lateral loads through triangulation, offering even more uniformed load distribution than that of the trussed tube. This system also has the ability to transfer loads throughout other paths in case of a localized structural failure.

2.5.8 *Bundled tube*

As a further extension of the framed tube the bundled tube concept was a tube that consisted of many tubes tied together. The tubes worked as one whole tube but the shear forces are more evenly distributed among the inner columns preventing the shear lag that had been the problem of the framed tube, shown in Figure 2.18.

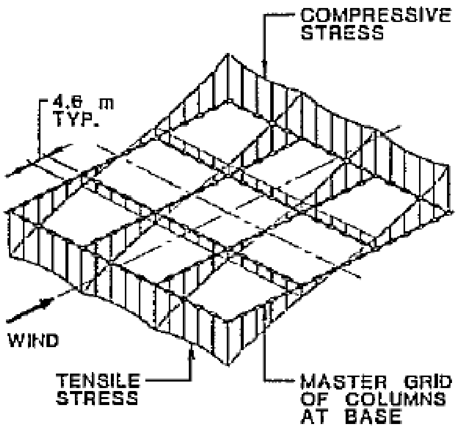


Figure 2.18 Behavior of the bundled tube, in this case the Willis tower, under stress of a lateral load. The shear lag is significantly less than that of a framed tube (Khan, 2004)

2.5.9 *The Core*

The central part where the elevators, shafts, utilities and more are located is often referred to as the core. It should also provide the building with lateral stability while carry gravitational loads. Usually consisting of reinforced concrete, but also braced steel frames, these structural elements are used in almost every high-rise building in some way. Most cores are placed in the central part of the building, but this is not always the case. While providing more flexibility for the use of floor space, moving the core off center can make the distance from the far sides too remote in terms of convenience and emergency exiting. An off-center core will also experience an increase in torsional forces. Even though the core is most often used together with another structural system it can be used by itself as a core structure. The floors are then cantilevered off of the core and produce a column free interior. This system is not very efficient however, with very limited width and height.

Another system that is very core-dependent is the suspended structure. It works by having a horizontal truss that redistributes the vertical loads from the perimeter of the floors below to the core, seen in Figure 2.19. This makes it possible to have a column free space below the hanging floors. Since all the vertical and horizontal loads are carried by the core it is not an effective system for very high buildings.

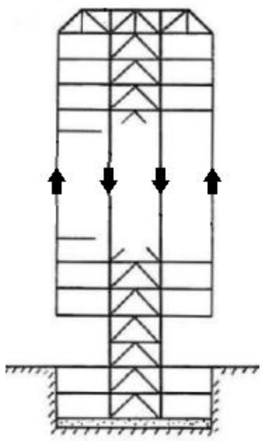


Figure 2.19 Vertical load distribution in a suspended structure (Coull & Smith, 1991)

2.5.10 *Tube in a tube*

Since the core with shear walls put together as a box will make it act as a tube, the core together with another tube system acting at the perimeter will have two tubes, creating the Tube in a tube system, see Figure 2.20. Only connected through the floors the two tubes will act together in countering the deflection caused from lateral loads.

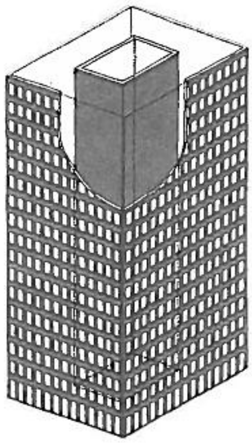


Figure 2.20 A core of shear walls working together with a framed tube creating the tube in a tube structural system (Khan, 2004)

2.5.11 *Core with Outrigger system*

Many high-rise buildings consist of two major structural elements, support columns at the facade and a central core. With almost column-free floor space between them two, this concept is very efficient in terms of open plan function but reduce the building's overall lateral resistance as the cores and columns are essentially disconnected. (Council of tall buildings and urban habitat, 1994)

The outrigger system is a development due to the desire to make them work as one by linking them together at one or more levels with rigid arms – outriggers, shown in Figure 2.21. It may be formed by any combination of steel, concrete or composite construction and reduce the structures internal overturning moment by up to 40 % compared to that of a free cantilever.

Figure 2.21 Core and outrigger system with belt truss (Tyréns)

There are two types of outrigger systems. One called conventional, or direct outrigger system, and the other for "virtual", or belt truss system. In the conventional outrigger system perimeter columns and core are directly connected to each other with outriggers, i.e. walls or trusses. In the virtual outrigger system, the coupling is achieved indirectly, through floor diaphragms and belt trusses. Belt trusses ties the columns together through a belt that loops around the building. The outrigger connection takes place at certain levels, often designed as mechanical floors for house elevator motors and other installations.

In the conventional outrigger system, when the central core tries to tilt, because of lateral loads, outriggers involve the outer columns resulting in tension and compression forces in them on each side of the core. These reaction forces are acting in opposition to the core's rotation. By that action it will reduce the cores internal overturning moment. One thing to keep in mind is that outriggers do not reduce the shear forces; instead they can increase and even change their direction.

In the virtual outrigger system, at the belt truss levels, the forces caused of core-tilting, make the floor diaphragms to move in different directions at different levels. As belt trusses are connected both to floors and columns the floor movements will be transferred to columns via the belt trusses. This, like in the direct outrigger system, results in tension and compression couple forces in columns that will push back, again through belt trusses, the floor diaphragms which will stabilize the core.

Belt trusses, a vital component in virtual outrigger system, can even be used in a direct outrigger system providing a secondary benefit, higher torsional stiffness. It is accomplished due to that the belt trusses are making outer columns act as fibers of a façade tube.

The advantage of the outrigger system can also be its drawback, because of differential vertical shortening of the core and the outer columns since stresses and, if they are made of concrete, reinforcing ratios and many other factors are different those between, see Chapter 4.3. Forces incurred of this can become almost as large as the design forces from wind. This problem is smaller if a virtual outrigger system is used as the core and the columns are not directly connected. Foundation dishing, caused by higher settlement under the core since more loads are concentrated there, can give the same result as the deferential vertical shortening.

Outrigger system is less suitable if;

- the stiffness of the core is already high, by a low aspect ratio (height/width)
- there is lack of symmetry
- torsional forces and deformations are of primary importance (without belt truss)
- columns size are strictly limited
- the mechanical floor design is already limited

(Choi, Ho, & Others, 2012)

2.5.12 *Buttressed core*

The central core stabilization principal often means that great gravity loads at the central part of a building´s foundation and in many cases soil bearing capacity can be critical for such constructions. Buttressed core system is a solution to spread gravity loads out from the center and also use that to give improved lateral stabilization with the ability to construct higher buildings.

The design of a building with buttressed core is a structure where the core is stabilized with outgoing wings, illustrated in Figure 2.22. The central core, providing torsional resistance, is attached with building wings, providing shear resistance and prohibiting overturning moment by an increased moment of inertia. Placement of smaller cores around stairs at ends of the wings can give an additional moment of inertia. The wing’s wall could be formed as an elongated box instead of one continuous piece given better torsional resistance. A virtual or direct outrigger, described in 2.5.11, can be used to engage the perimeter columns, stabilizing each wing. If smaller shear walls are placed orthogonally and connected to the wings the need for columns can be abandoned. (Baker & Pawlikowski, 2012)

The current highest building in the world, Burj Khalifa, is designed using this system. Three buttresses are connected to the core for stabilization.

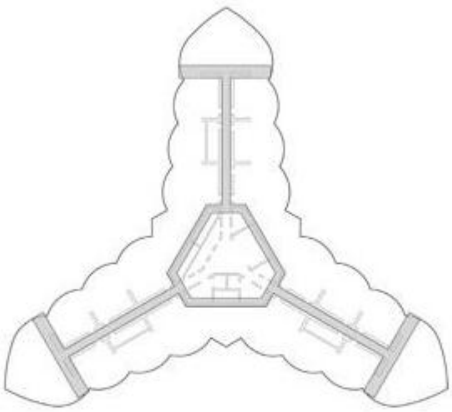


Figure 2.22 The buttress core of Burj Khalifa. The core is connected to three buttresses (Baker & Pawlikowski, 2012)

2.6 Structural systems and appropriate height

Figure 2.23 is showing different systems and their reasonable building height, expressed as stories. The buttressed core is not part of the graph but has been able to reach 163 floors.

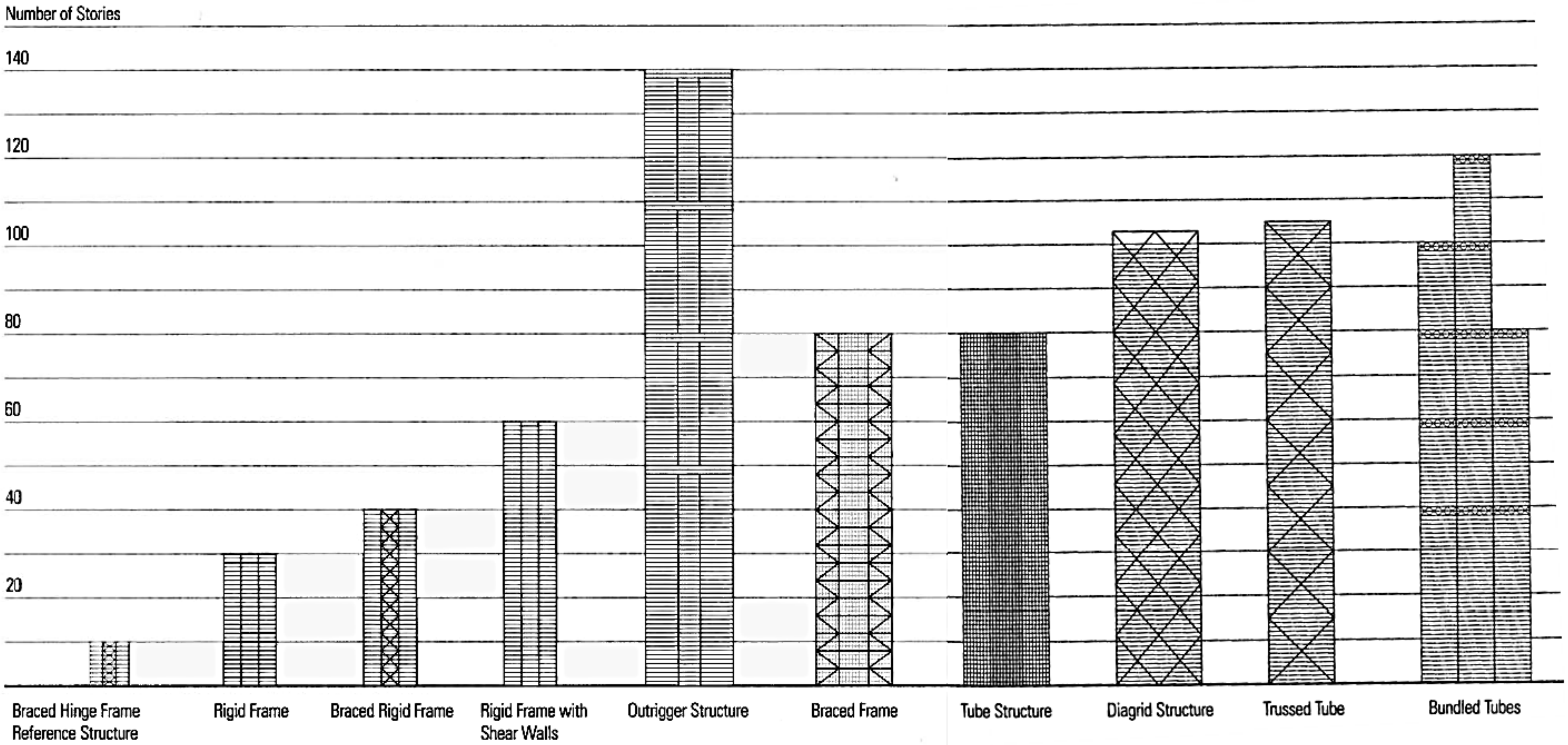


Figure 2.23 High-rise structural systems and their reasonably attainable number of stories
(Ching, Onouye, & Zuberbuhler, 2009)

3 TUBED MEGA FRAME

Tyréns, King and Severin with others, is in development of a new structural system concept for tall buildings. The concept gets rid of the tradition to have a central core and instead putting the transportation in giant mega columns at the perimeter, see Figure 3.1.This is possible due to a new innovation in vertical transportation; the Articulated Funiculator, also being developed by Tyréns. With the absence of a central core this structural concept gives new possibilities in terms of architecture and slenderness.

Figure 3.1 Tubed mega frame concept (Tyréns)

3.1 **Articulated funiculator concept**

Essentially, the articulated funiculator is vertical trains capable of switching between vertical and horizontal alignment. Having a vertical alignment when going up and down the structure and horizontal when on and off loading at stations. The trains follow a continuous loop throughout the whole building, following the tracks that snake from one side to the other, stopping at every station, shown in Figure 3.2. All the funiculator trains follow the same tracks and use the same cables, shafts and motors, which add efficiencies. Stations are separated by a fixed height, i.e. 250 meters, with conventional elevators making people able to go to specific floors, similar to that of a sky lobby. When a train is going down bound the energy from its motion when braking is to be saved and used for up bound transportation, making the articulated funiculator sustainable.

The trains and train cars are to be designed so that the passengers are to be standing upright during transportation. By having a carriage frame inside the train car that pitches when switching between vertical and horizontal alignment the passengers remain standing. Current design shows that the articulated funiculator elevators needs space of 3,5 meters by 3,5 meters.

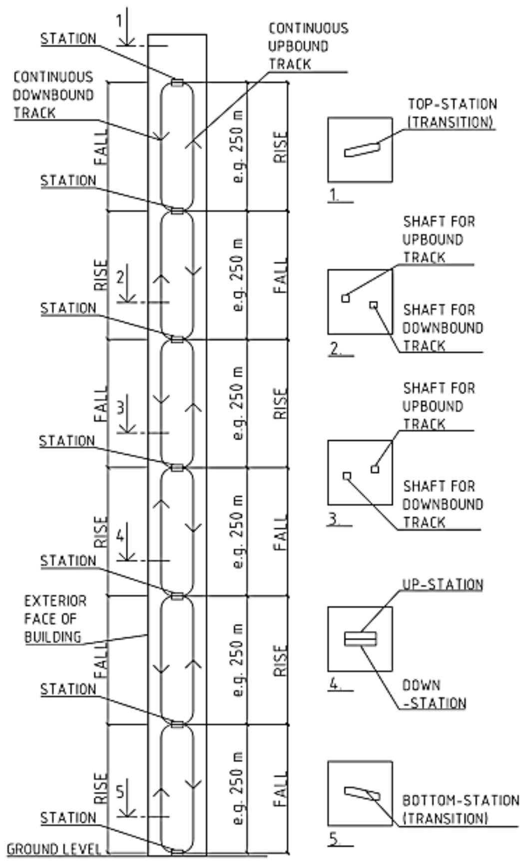


Figure 3.2 Way of travel for one funiculator train (Tyréns)

3.2 **Structural design**

The mega frame structure utilizes the vertical corridors housing the articulated funiculator as *reinforced concrete* mega columns. These are designed according to the tubular concept mentioned in Chapter 2.5.5 making them achieve good stiffness. To improve the stiffness even more the mega columns are to be placed at the perimeter to achieve maximum length of the lever arm. Similar to the outrigger system the legs are connected by horizontal tubes at certain floors; directly above and below the funiculator stations and at the top. The floor loads are carried through columns down to diagonals at outrigger levels which transfer the load to the mega columns, seen in Figure 3.3.

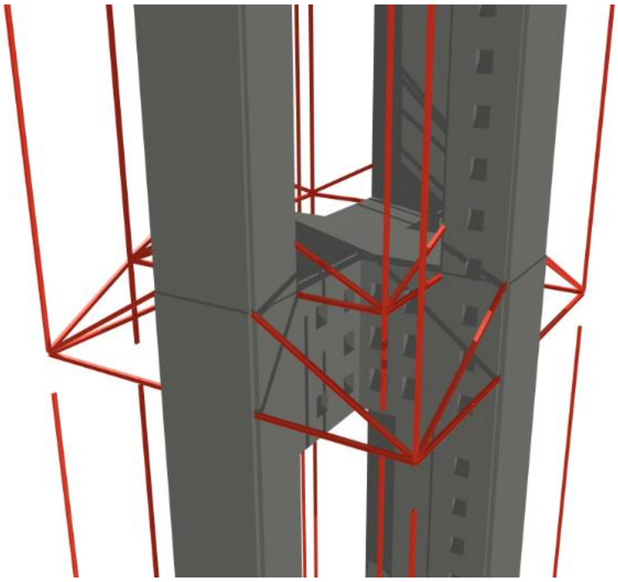


Figure 3.3 Floors around station, showing the framework connecting to mega columns via diagonals and the horizontal tubing connecting the legs (Tyréns)

3.3 Structural performance

Initial testing in ETABS indicates good response to the first five modes of natural frequency. A quick comparison between an 800 meter prototype and the 660 meter Ping An building can be seen in Table 3.1. The table shows slenderness ratio, floor utilization ratio and time periods for three modes of frequency. These are approximate tests with calculated assumptions made for floor utilization and natural frequencies. Ping An has an outrigger system with a central core with 8 super columns at the perimeter whereas the 800 meter prototype has 8 tubed super columns, outrigger walls and no central core. Floor plans for the 800 meter prototype and Ping An can be seen in Figure 3.4 and Figure 3.5.

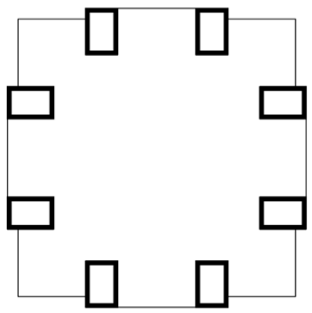


Figure 3.4 Floor plan for the 800 meter prototype building (Tyréns)

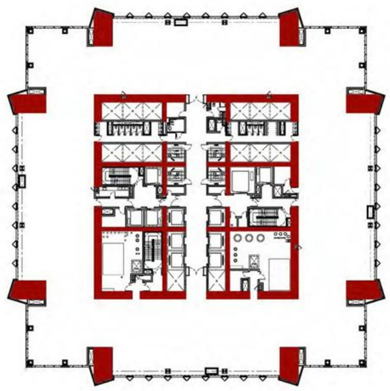


Figure 3.5 Floor plan for the Ping An (Tyréns)

Table 3.1 Quick comparison between the 800 m prototype and Ping An (Tyréns)

Building	Height	Slenderness ratio	Approx. Floor utilization ratio	T, mode 1	T, mode 2	T, mode 3 (torsion)
Prototype	800	1:14	0.89	8.4	8.4	2.8
Ping An	660	1:12	0.70	8.4	8.4	3.4

3.4 **A Variety of shapes**

The tube mega frame offers a variety of shapes and can be formed to fit many different requirements. Figure 3.6 displays a few of the already modeled prototypes. These are compared to each other in Chapter 5.3.

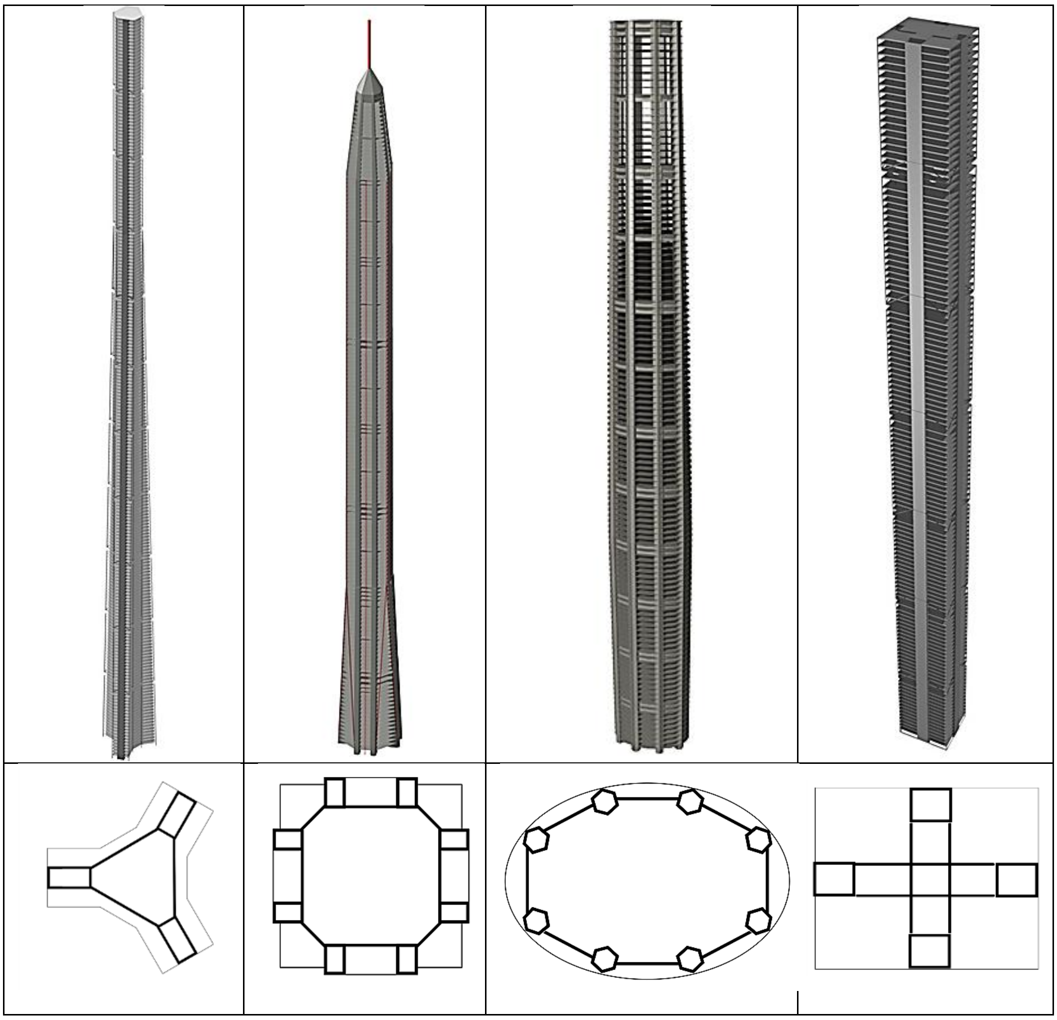


Figure 3.6 Different tube mega frame prototypes with their respective outrigger floor plans (Tyréns)

4 DESIGN OF HIGH-RISE COMPONENTS

Designing the primary building elements; beams, columns, walls, doesn't differ much from a low-rise building. Design for buckling, tilt and P-delta effects is done in the same way. However, since the loads are much greater their effects must be considered on a larger scale than for a low-rise. Column shortening that is often negligible in low-rise buildings must be taken into account when designing tall buildings.

During construction the structural behavior of the building can change from the ideal finished stage. Therefore a construction sequence analysis should be made, this is especially important for tall buildings.

Computer programs are today frequently used for design of structures and its components. While some programs focus more on modeling and analyze others focus more on design of elements. Often more than one program is required to get a thorough result. ETABS and SAP2000 are finite element programs commonly used for analyze and design of structures. ETABS is more adapted into building design whereas SAP2000 is used for structural analysis in general. A drawback when using SAP2000 is the missing ability to design shear walls. ETABS allows for concrete shear walls to be designed which is essential when doing a more thorough analysis on a system like the tube in a tube or outrigger that utilizes a concrete core. The ETABS version 9.7.4, which is used for the models in this report, doesn't have the ability to design walls according to euro code. Since these programs uses complex calculations when designing and analyzing it is necessary to check results with hand calculations.

4.1 **Buckling**

The buckling instability is a phenomenon of elongate columns which are loaded with compression forces in their longitudinal direction. The phenomenon means that at a certain critical load, the so-called buckling load, a columns shape is substantially changed in terms of deflection which results in that the column fail. Quite often the buckling of a column leads to sudden and dramatic failure and occurs before the normal stress reaches the strength of the columns material. As long as the acting load is less than the buckling load there is a slight deformation of the column, elastic shortening that can be calculated with the modulus of the elasticity.

Determination of the buckling load is originated in the calculation of the theoretical buckling load, also called critical load, which is done according to classical theory – Euler's buckling cases. It depends on length, stiffness, and design of the end connections of the column. The classical theory assumes no account of factors that are very critical in reality for determination of the real ultimate limit state. These factors are strength, straightness and residual stresses whose effects in many cases do not make it possible to reach the critical load.

A combination of the critical load and these factors nevertheless provides the real capacity. (Stålbyggnadsinstitutet, 2008). The combination of them is done differently depending on what material is used. For the steel columns a reduction factor is used for determination of the capacity but for concrete columns the buckling load is used to magnify the moment load instead of reducing the load bearing capacity.

4.2 **P-delta**

P-delta is second order effects. Second order effects can arise in every structure where elements are subject to axial load. When a model is loaded, it deflects. The deflection may give rise of an additional moment – a second order moment. It is of notable importance to consider this as additional moment may incur additional deflections which in turn again can incur additional moment, a third order, and so on until the loads can eventually exceed the capacity. Therefore in the design of members the total moment, summary of moments caused of the first order, M_e, and second order, M_g, should be included or proportionately "decreased" capacity should be used. Figure 4.1 and Figure 4.2 shows the P-delta effects on beams and columns.

The magnitude of P-delta effect is related to the:

- magnitude of axial load N (P label is often used therefore the name is P-delta)
- stiffness and slenderness of members

There are two different P-delta effects. The first one is P-δ, even called P - "small delta", and the other is P-Δ, also called P-"big-delta". "A P-δ effect is associated with local deformation relative to the element chord between end nodes". "A P- Δ effect is associated with displacements relative to member ends".

P-delta effect in a structure may be managed by increasing its strength or its lateral stiffness (e.g. lateral bracing) or by a combination of these.
(CSI Computer & Structures. INC., 2013)

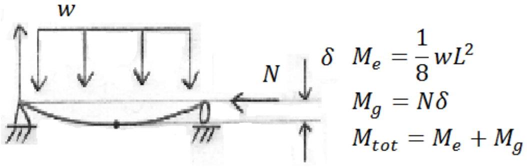


Figure 4.2 P-delta effects on beams (CSI Computer & Structures. INC., 2013)

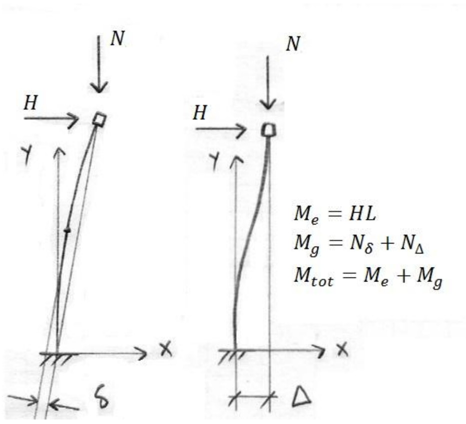


Figure 4.1 P-delta effects on columns (CSI Computer & Structures. INC., 2013)

4.3 **Column shortening**

When a vertical load is applied to a column it shortens. Shortening takes place in all structures but when reaching great heights its effect has significant importance. As the columns shortening are added together the overall shortening of a high-rise building becomes big enough to have real consequences. Floor slabs starts to tilt because of differential column shortening which in turn affects the cladding, partitions, mechanical equipment and more, a possible result is shown in Figure 4.3.

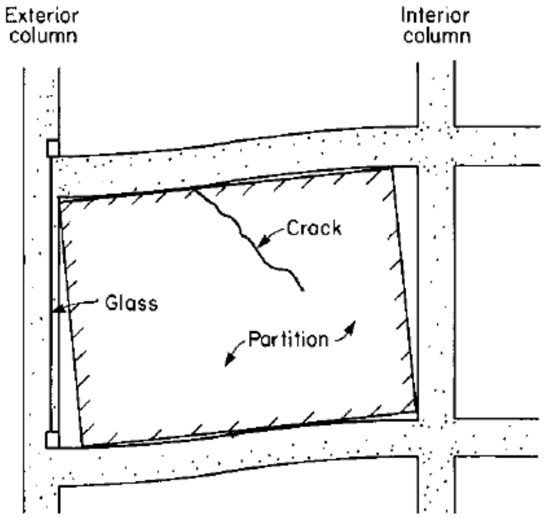


Figure 4.3 Effects of differential column shortening (Fintel, Ghosh, & Iyengar, 1987)

Depending on the material used for columns the shortening varies. Steel columns have a tendency to be more affected by column shortening than reinforced concrete. Reinforced concrete does however have length changes from creep and shrinkage making the two materials almost equal in total length change. (Fintel, Ghosh, & Iyengar, 1987)

By designing the vertical structural members connection to deform without stressing the affected elements (cladding, partitions etc.) column shortening can be contained. The problem of differential shortening between adjacent vertical elements still remains however, and must be taken into consideration.

To compensating for column shortening a few different methods can be applied. If the stresses are made equal between the vertical elements the length change will be more equal, especially if the material is the same. Steel columns can be compensated by making them longer in fabrication. Concrete columns can be adjusted by the formwork. Another counteraction can be made by tilting the floor slabs the opposite way than that because of column shortening during construction.

Shortening has to be taken into account when the building is being erected since it will vary as more stories are added. This can be calculated by using construction sequence, see Chapter 4.4. To counteract the shortening during construction shims are used. As the number of stories change the shims are either added or removed from columns successively to make them behave in a favorable manner.

4.4 **Construction sequence**

In conventional design the strength, stability and deflection are based on the structure when it's completely erected. This can be compared to constructing a structure in space without any gravitational loads and then adding all loads instantaneously when the structure is finished. In some cases this will create a false image of stress distribution because when the building is being erected components may have a different behavior than when the building is completed, making the building fail during construction. I.e. columns may be in tension which in reality experiences compression, illustrated in Figure 4.4. When designing a structure, especially high-rise structures, an analysis of the building during the construction process should therefore be made. Construction sequence allows each story to be added progressively which can change the designed load to better fit the real structure.

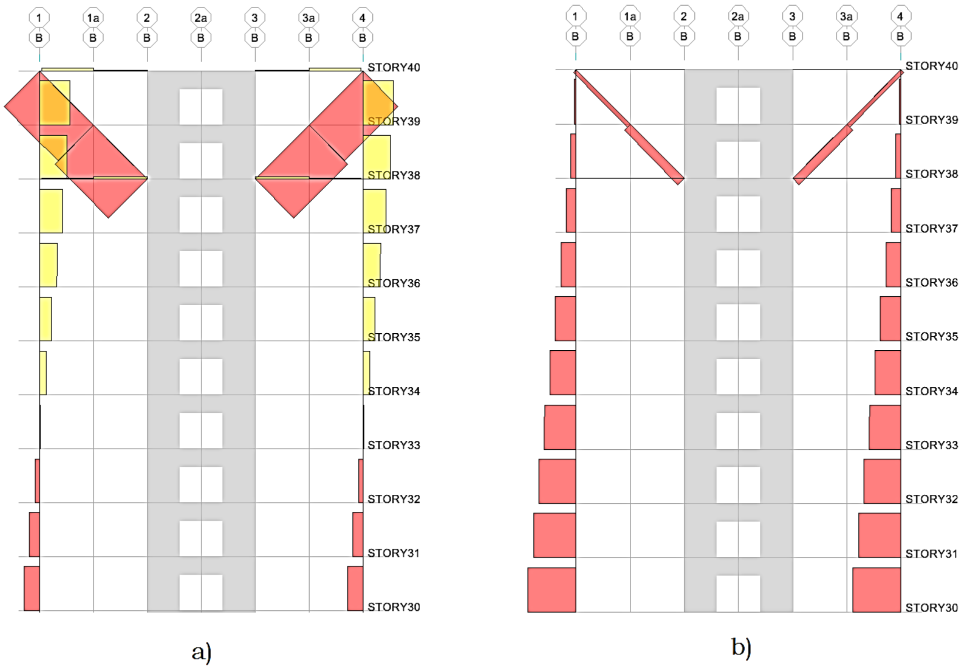


Figure 4.4 Axial forces in an outrigger system.
Figure a) is analysed with no consideration to construction sequence and has tension (yellow) in the upper columns.
Figure b) is analysed with construction sequence showing compression (red)

4.5 Design of rectangular concrete columns and walls in compression and bending

The moment capacity for a given cross-section of a column depends on the acting axial forces, at the same time as the moment. The moment capacity is therefore not a property of a cross-section instead it is due to acting load case. A strength interaction diagram defining the failure load and failure moment for a given column can be made Figure 4.5. The diagram will provide a graphic solution to determinate if a column will fail or not for all possible combinations of moment and axial load. This is useful since the column can be easily tested for different load cases.

A strength interaction diagram can be easily constructed for one direction for a given cross-section by calculating three major points, using method - simplified pressure block and reached ultimate strain of the concrete.

1. Point for the maximum axial capacity, when the moment is zero.
2. Point for the maximum moment capacity
3. Point for the moment capacity when axial forces are zero. (Calculating this point can be demanding. Neglecting reinforcement in compression zone can simplify the calculation.)

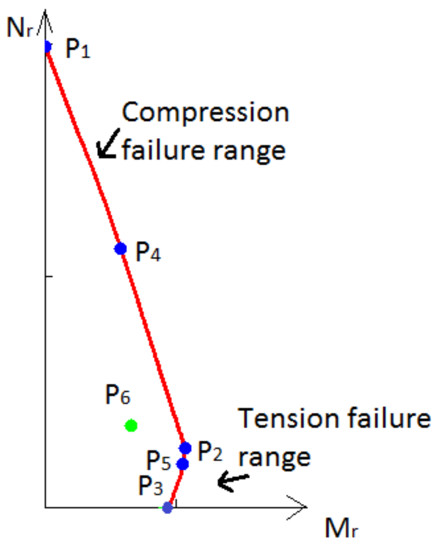


Figure 4.5 Strength interaction diagram for a concrete column

Points between them, for example P_4 and P_5, can be determined by varying distance, $\boldsymbol{x}$, of neutral layer to the outer edge, see Figure 4.6 which will give a better resolution of the diagram in Figure 4.5. As distance of the neutral layer is chosen the strain ($\boldsymbol{\varepsilon_s}$) and the stress ($\boldsymbol{\sigma_s}$) in reinforcements can be calculated by equation (4.5.1) and (4.5.2). Therefore there are none unknown and eventually moment ($\boldsymbol{M_r}$) and axial ($\boldsymbol{N_r}$) capacity can be calculated for the chosen $\boldsymbol{x}$ by equations (4.5.3) and (4.5.4).

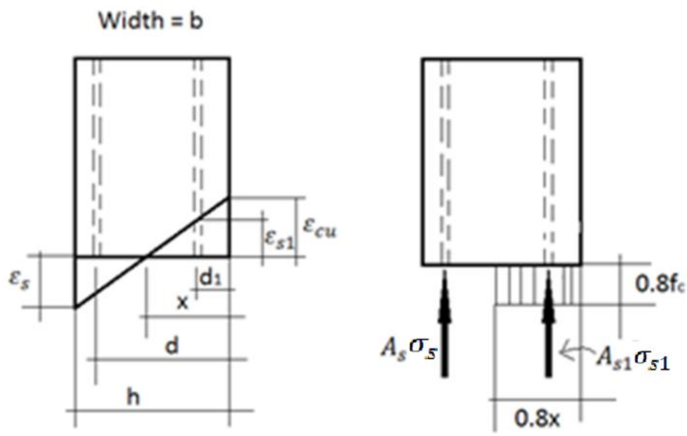


Figure 4.6 Distances and forces in a concrete column

$$\sigma_s = \varepsilon_s E_s \quad if \quad |\varepsilon_s| \leq \varepsilon_y$$

$$\sigma_s = f_y \quad if \quad |\varepsilon_s| > \varepsilon_y \tag{4.5.1}$$

$$\varepsilon_s = \frac{x-d}{x}\varepsilon_{cu} \tag{4.5.2}$$

$$M_r = 0.8f_c \cdot b \cdot 0.8x \cdot \left(\frac{h}{2} - \frac{0.8x}{2}\right) + A_{s1}\sigma_{s1} \cdot \left(\frac{h}{2} - d_1\right) - A_s\sigma_s \cdot \left(d - \frac{h}{2}\right) \tag{4.5.3}$$

$$N_r = 0.8f_c \cdot b \cdot 0.8x + A_{s1}\sigma_{s1} + A_s\sigma_s \tag{4.5.4}$$

P_6 in the interaction diagram represents a real load case, as this point is inside of the curve the column will hold. If this point is outside the curve the column would fail.

A similar diagram can be made even for walls, because walls are actually elongated columns. There will be more layers of reinforcements which will make the calculation of points longer but the principal is the same.

In Figure 4.7 the moment capacity is considered around one axis, if considering of the moment and capacity around two axes is demanded a similar curve in the other direction can be constructed. Combining these two curves in one diagram will provide a graphic surface, Figure 4.8, which can be used to control a column capacity with the same principal as with the curve.

According to EC 2 (euro code) the second order effects can be neglected if the effects are less than 10 % of the first order effects. The slenderness of a column is then the measure of whether these effects are of smaller magnitude or not.

Important aspects of the slenderness of a concrete column are its cracking and creeping effects. As concrete cracks the overall stiffness of the column gets reduced and the deflections increase. The concrete creeps during a long time – up to 70 years, all these years the deflections are increasing and so does the additional moment. (Engström, 2007)

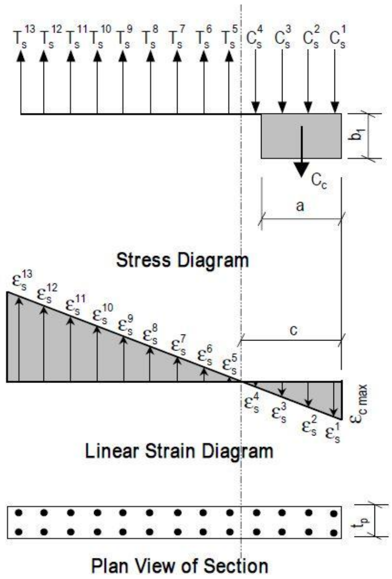


Figure 4.7 Stress and strain for a concrete wall (ETABS MANUAL)

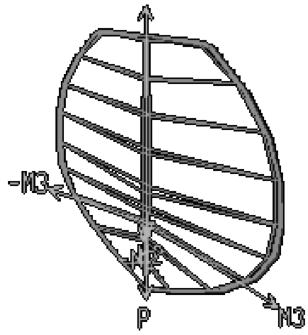


Figure 4.8 Graphical interaction surface made in ETABS

Walls are often designed with openings. When using ETABS this may require additional concern. The wall is divided into several pieces when an opening is made, see Figure 4.9. It is necessary to make ETABS design the overhead piece S1 as a beam instead of a column.

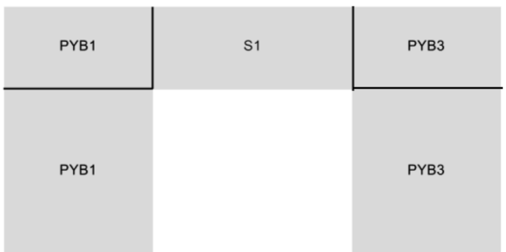


Figure 4.9 Wall with opening, divided into several pieces

4.6 Designing steel columns

The design of the steel columns is simpler than of the concrete because the steel has same material properties in tension and compression, no cracking, shrinkage and negligible creep effects are developed over time.
For a steel column the axial capacity is shown in equation (4.6.1) and the moment capacity in equation (4.6.2).

$$N_{Rd} = \frac{\chi \cdot A \cdot f_y}{\varphi_{M1}} \tag{4.6.1}$$

$$M_{Rd} = \frac{W \cdot f_y}{\varphi_{M0}} \tag{4.6.2}$$

When handling both an axial force and a bending moment a steel column will hold as long as equation (3.5.3) is true. The interaction diagram for a steel column will therefore be a lot less complex than that of a concrete column, as shown in Figure 4.10. (Stålbyggnadsinstitutet, 2008)

$$\frac{M_{Ed}}{M_{Rd}} + \frac{N_{Ed}}{N_{Rd}} \leq 1 \tag{4.6.3}$$

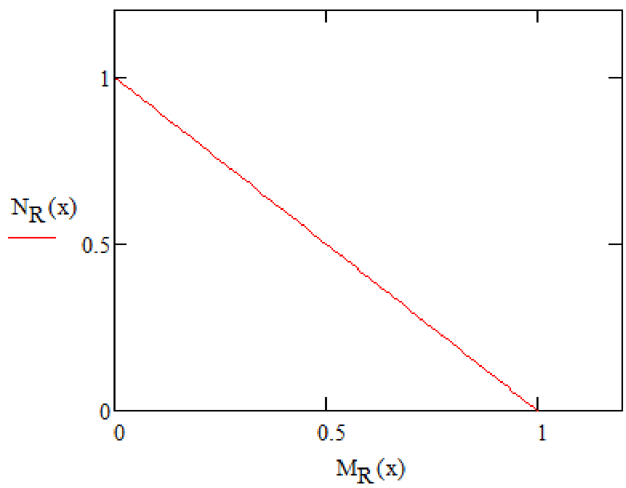


Figure 4.10 Interaction diagram for a steel column

4.7 **Design of a concrete beam**

Unlike columns beams are rarely exposed for high axial forces, the risk of buckling is low. The beam capacity for shear forces and moment is vital.

Determination of the moment capacity of a beam is not much different from that of the columns which are not subjected to axial force, see the previous section. Characteristic for the beams is that they are often subjected to tensile stresses at the bottom and compressive stresses in the top in field and the opposite at supports. Reinforcement in the tension zone is crucial for the resistance as the concrete has low tensile capacity. Even in pressure zones reinforcement may be of importance if the compressive stresses exceed the concrete's compressive strength.

Shear resistance of a concrete beam can be calculated using the truss model where shear reinforcement represents tensile struts and the concrete compression struts, as diagonals between the shear reinforcement, see Figure 4.11. As a designer there is a possibility to choose the slope compression struts should have, these are recommended to be limited in the range of 22 to 45 degrees. Selection of the slope affects the requisite amount of shear reinforcement and the requisite concrete strength. Generally, flat slope required less shear reinforcement quantity but higher concrete strength. Two conditions must be met; compressive stresses in the pressure strut do not exceed the concrete strength that tensile stresses in tensile strut do not exceed the shear reinforcement tensile strength. (Engström, 2007)

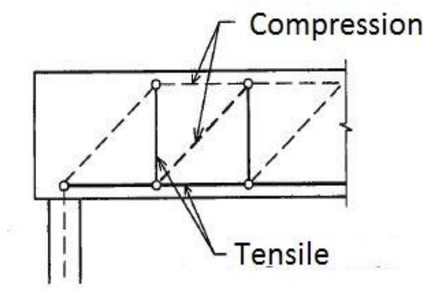


Figure 4.11 Truss model

4.8 Model check

After completing a model using a FEM-program such as SAP2000 or ETABS it may be necessary to perform a model check. This is done by hand calculating the responsive forces for a few cases and control that they are similar to that of the model. A vertical control is done by calculating the weight of the structure and then the reaction forces. Shear force and overturning moment with its reacting forces can be controlled using the acting wind force. Appendix B demonstrates a model check done for the models made in ETABS.

5 ANALYSIS

For greater understanding of the structural systems different analysis are made. Stabilization is the primary subject looked upon but other things are measured as well.

The first analysis is of systems in 2D using SAP2000 with a 20 story structure. A simplification is made when converting 3D systems to 2D system. Comparison is made from the deflection from wind. Other compared subjects are weight, utilization and connection (as pieces).

The second analysis is made using ETABS in 3D primary with a 40 story building. A 40 instead of 20 story building is used because the systems measured are not applicable for lower buildings. In this analysis the stabilization is compared from both deflection because of wind and the stiffness calculated from the natural frequency. It first compares the tube in a tube with and the outrigger system. From there these two systems are compared to the Tubed mega frame (TMF) with a few different premises; designed to meet requirements and then using the same amount of concrete. A 160 story building is created with the TMF and the Outrigger systems as a premium for height study.

The third analysis, which is also made in ETABS with a 40 story building, measures the effectiveness of the TMF with different geometries. It first compares geometries using about the same amount of concrete and then designing them to get another comparison.

For further information about how the modeling and design was done see APPENDIX.

5.1 2D Analysis using SAP2000

The structures have geometry according to Figure 5.1. Adding only a realistically calculated horizontal wind load of 50 kN per floor (calculated with a building width of 30 meters), see APPENDIX A, and then making the program auto-design the frames with selected profiles, all W21 for beams and all W14 for columns, a comparable value on weight, displacement and the ratio of utilization is given, see Table 5.1. The graph in Figure 5.2 shows the deflection-curve when increasing the number of floors, while Table 5.2 lists the biggest required beam and column as well as the amount of different steel pieces in the frames.

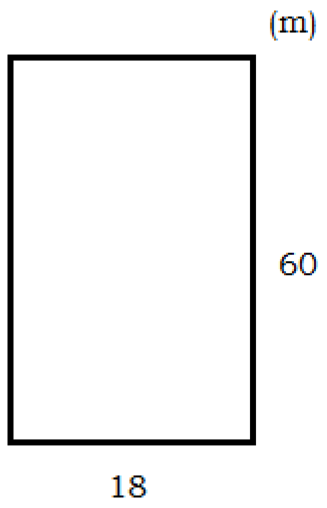


Figure 5.1 Geometry of 2D analysis model (elevation)

The W-shaped profiles are a steel-type found in the Steel Construction Manual, from the American institute of steel construction (AISC). W-shapes have parallel inner and outer flange surfaces and have the form of an “I”. The first number is displaying the nominal depth and the second number is displaying the nominal weight in lb./ft. E.g. a W21x101 is a W-shape that is nominally 21 inches (533 mm) deep and weighs 101 lb/ft (150 kg/m).

Table 5.1 weight, displacement and average ratio of utilization of steel members for different systems

Type of system	Weight [kN]	Displacement [m]	Ratio of utilization
Rigid frame [RF]	408	0,525	0,70
Completely braced frame [CBF]	479	0,124	0,33
Partially braced frame [PBF]	419	0,650	0,36
Bracing + outrigger [B+Out]	452	0,304	0,52
Braced rigid frame [BRF]	391	0,273	0,57
Shear walls + rigid frame [SW+RF]	194	0,187	0,45
concrete C30	2121		
Shear walls + outrigger arms [SW+O	213	0,149	0,39
concrete C30	2121		

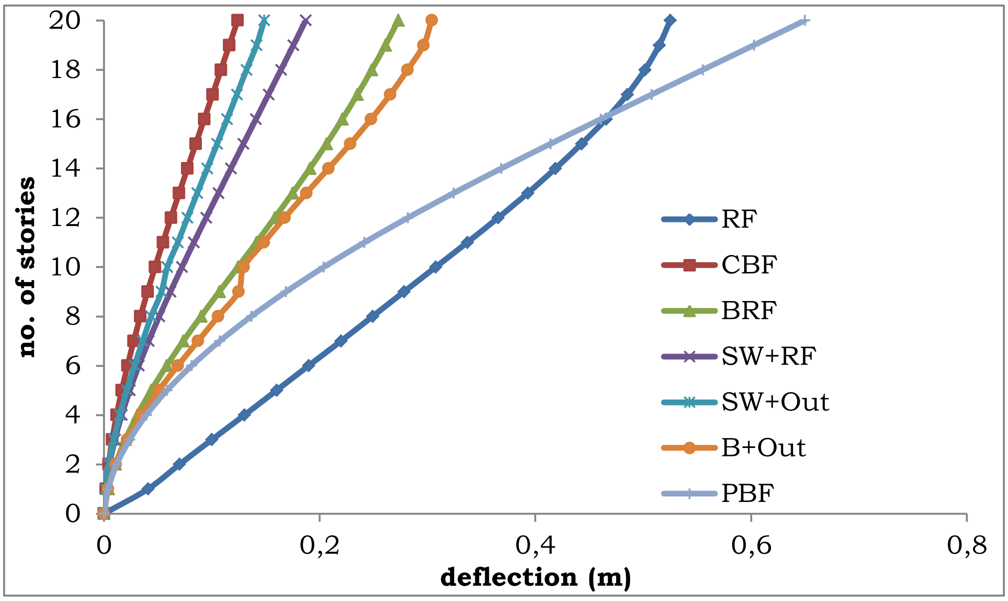


Figure 5.2 Graph showing the difference in deflection between systems and number of stories

Table 5.2 Steel pieces needed for each system and the biggest required beam and column profiles.

Type of system	No.of pieces	Biggest beam	Biggest column
Rigid frame [RF]	140	W21x101	W14x109
Completely braced frame [CBF]	260	W21x44	W14x34
Partially braced frame [PBF]	180	W21x57	W14x90
Bracing + outrigger [B+Out]	188	W21x73	W14x61
Braced rigid frame [BRF]	180	W21x50	W21x48
Shear walls + rigid frame [SW+RF]	80	W21x44	W14x22
Shear walls + outrigger arms [SW+Out]	88	W21x62	W14x30

Every system except the rigid frame demonstrates the same way of deflection, the bending way of deforming, with the deformation accelerating with higher stories. The rigid frames deformation is decelerating with each floor which is excellent in terms of allowed deflection. However the bending moment in the beams and the rigid connections requires huge steel profiles and massive connections at the bottom of the frame.

The biggest beam and biggest column can be found at the bottom of every system except for the braced rigid frame and the outrigger systems. In the braced rigid frame the biggest beam is found near the middle of the structure. Both the biggest beam and column in the outrigger systems can be found in the lowest outrigger arm.

When looking at the graph in Figure 5.2 and Table 5.1 the rigid braced frame seems like a better choice than using the outrigger system, with less deflection, less weight and about the same rate of utilization. This is not entirely correct however, since the outrigger system uses a lot more steel than it has to because of the way it is assembled. If looking at the ratio of utilization at the beams in between the outrigger arms the beams show a utilization of zero, meaning they’re not being used at all against the wind load, see Figure 5.3. In reality these beams are really the floors connected between the core and the column at the perimeter, preventing the column from buckling. If the weights of these non-utilized beams are withdrawn, this system would show to be much more beneficial. The second analysis displays a fairer image of the outrigger system.

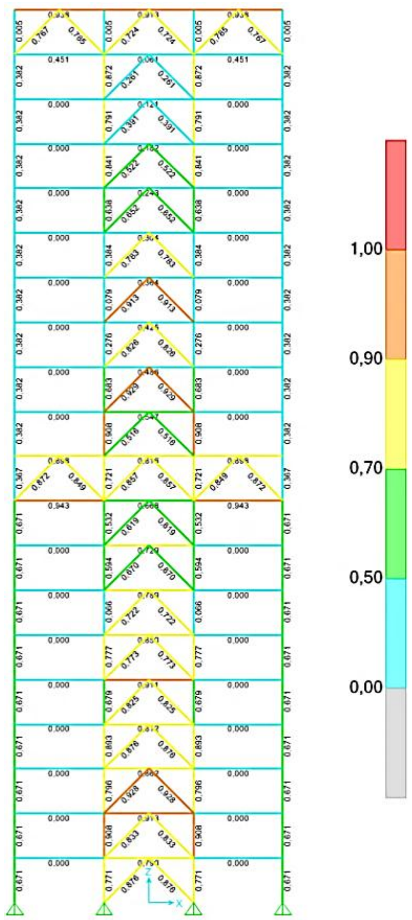


Figure 5.3 The ratio of utilization in an outrigger system. The beams between the outrigger arms showing zero utilization

5.2 **3D Analysis using ETABS**

The model used in the second analysis is similar to that of the one used in SAP2000 but with a changed length of 26 meters and a width of 18 meters.

5.2.1 *Tube in a tube model*

Since this analysis is done in 3D a core is designed with geometry of 6 by 6 meters located in the center of the structure. A model of the structure can be seen in Figure 5.4. With this new geometry a new wind load is calculated. Other loads such as dead, live and snow is also added to better approach reality, see Table 5.3. Determination of loads is done according to Eurocode, calculations for wind and snow can be seen in APPENDIX A. Steel components used are still W14 as columns and W21 as beams which are first auto designed and then manually chosen to decrease variation. Concrete quality of C40/50 is used in the core. Thickness of the core walls are varied between 250 and 700 mm depending on story with the thickest being at the bottom.

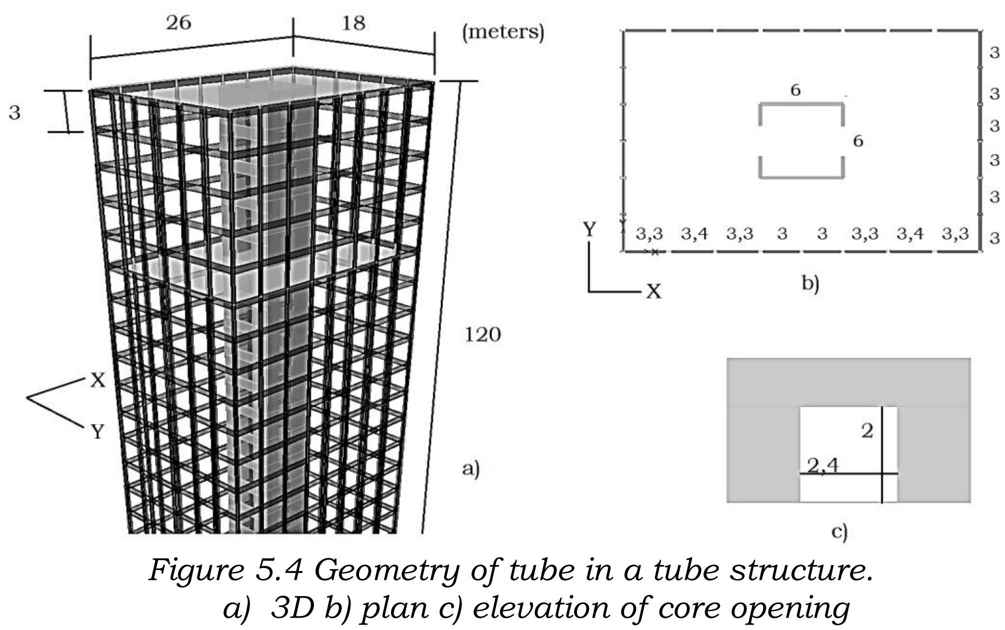


Figure 5.4 Geometry of tube in a tube structure.
a) 3D b) plan c) elevation of core opening

Table 5.3 Used loads in ETABS model

Load	Type	Distribution	Size
Wind	Wind	Joint load	42-120 kN
Live	Reducible live	Area load	2,0 kN/m^2
Installation	Live	Area load	0,8 kN/m^2
Façade	Super dead	Line load	2,0 kN/m
Snow	Live	Area load	1,28 kN/m^2
Dead	Dead		

In this analysis stability is met through deflection of wind and equivalent stiffness of the structure in its weakest mode shape in both the X and Y direction. The deflection because of wind is analyzed in the serviceability limit state. The equivalent stiffness can be calculated by using equation (2.3.1) in Chapter 2.3, through ETABS a structures time period for different modes and mass can be known.

5.2.2 *Outrigger model*

An Outrigger model is created using the tube in a tube model as base, with outriggers at stories 19-20 and 39-40. The outriggers are formed as X-braces and cover two stories in height. Number of columns has been reduced by three on each side in Y-direction and five in X-direction, see Figure 5.5.

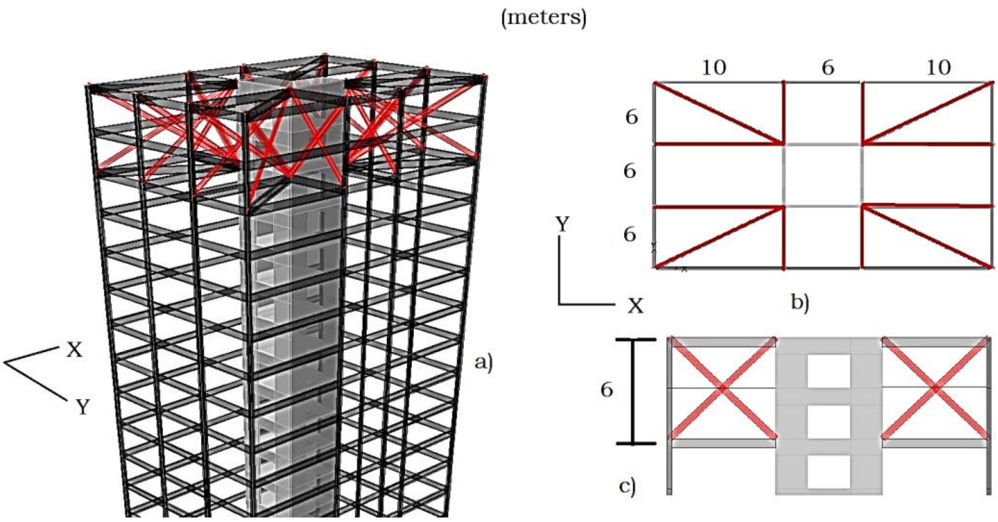


Figure 5.5 Geometry of outrigger structure.
a) 3D b) plan c) elevation of outriggers

5.2.3 *Tube in a tube versus Outrigger*

A first analysis of the outrigger shows that the concrete core is experiencing much larger stresses than the tube in a tube. Its shear capacity is exceeded at several stories. To make a comparable analysis the core is redesigned to fit the outrigger system increasing the thickness of the core by 50 mm at stories 17-22 and 13. The same core is then used in the tube in a tube system. After this change the outrigger system is more efficient in terms of stabilization, which can be seen in Table 5.4 and Figure 5.6, with less deflection and greater stiffness. The stiffness and deflection is improved by an average of about 21%.

There are a few drawbacks however; the outrigger requires more steel and two stories in height for each outrigger. An equivalent utilization of steel, shown in Table 5.5, can be used to get a better comparison for the increase of stiffness. About 17 % more steel is used in the outrigger system meaning the actual increase of stiffness is only about 4 %. A tube in a tube system generally has better torsional stiffness than the outrigger without belt trusses.

Table 5.4 Time period (Y and X), Max deflection (m) and Stiffness (EI) for the Outrigger and Tube in a tube

	T (s), Mode		Max Deflection		Stiffness	
System	Y	X	Y	X	Y	X
Outrigger	4,44	3,98	0,27	0,15	3,65E+10	4,55E+10
Tube in a tube	4,77	4,46	0,32	0,19	3,16E+10	3,61E+10

Table 5.5 Weigh of steel (kN), utilization and equivalent steel utilization weight (kN)

System	Weight of steel	Utilization	Utilized steel weight
Outrigger	7224	0,62	4508
Tube in tube	6629	0,58	3858

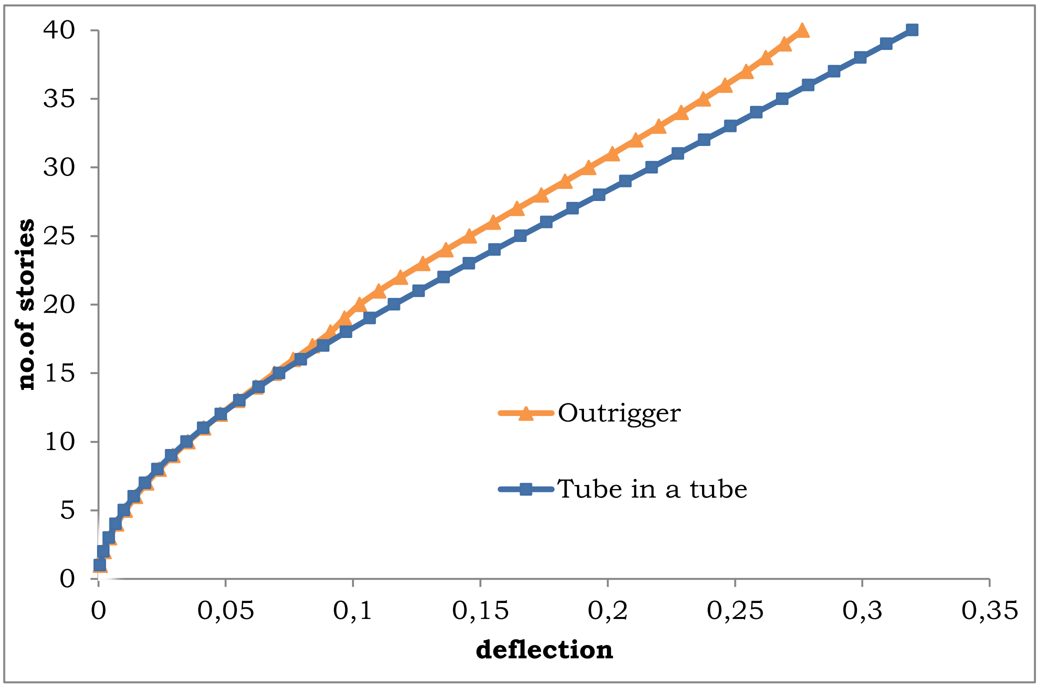


Figure 5.6 Deflection of the outrigger and tube in a tube systems in their stiff direction, X

5.2.4 *Tubed mega frame model*

This model is created using geometry according to Figure 5.7. The outriggers are made up of concrete walls and are placed at story 19-20 and 40. Steel columns are placed at the corners and the center of the model and made to redistribute gravitational loads to the mega columns at outrigger floors.

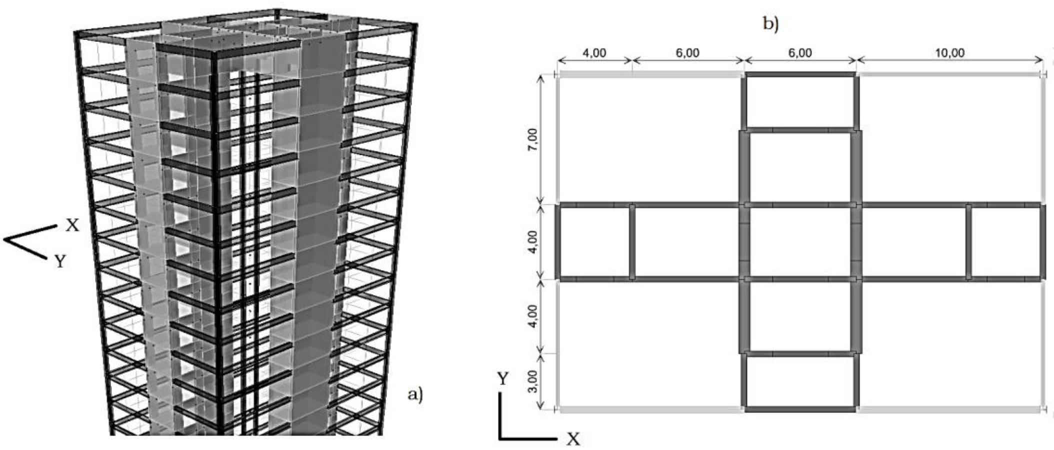


Figure 5.7 Tubed mega frame model. a) 3D b) Plan

5.2.5 *TMF versus Tube in a tube and Outrigger*

The previously used Outrigger and Tube in a tube model is reshaped with a bigger core of 6x12.6 meters to better fit a comparison to the Tubed mega frame model. The TMF model is made using about the same floor area and area boxed in by the core as the other two models. If the TMF would use the same area as the smaller cores (6x6 meters) the cores in the TMF model would have been unrealistically small. All models are designed to meet the requirements in ultimate limit state and maximum allowed deflection.

TMF shows greater stiffness in the structures weak direction with a 16,5 % increase compared to the Outrigger and 55 % compared to the Tube in a tube. In the other direction, X, the Outrigger has better stiffness and the torsional stiffness is lower in the TMF than in the both other systems, Table 5.6.

Material quantities vary depending on the system. While the Tubed mega frame uses a lot more concrete it uses considerably less steel that that of an Outrigger system. The tube in a tube utilizes the least amount of concrete and only a slightly increase in the weight of steel required, Table 5.7.

Table 5.6 Time period (longest), Stiffness (EI) for the TMF, Outrigger and Tube in a tube systems.

System	T (s)	Stiffness Y	Stiffness X	Stiffness R
TMF	3,5617	7,20E+10	1,27E+11	4,56E+11
Outrigger	3,6407	6,18,E+10	1,56,E+11	6,72E+11
Tube in a tube	4,0808	4,64,E+10	1,08,E+11	5,85E+11

Table 5.7 The weight of steel and concrete in the different modeled systems

System	Steel weight [kN]	Concrete weight [kN]
TMF	5702	58818
Outrigger	9509	36888
Tube in a tube	6658	31120

For further analysis the Outrigger and Tube models are remodeled using about the same amount of concrete for their cores as the TMF does. The TMF model will still have slightly more concrete weight since it uses concrete for its outriggers. The weight of steel remains the same as in previous models.

Equivalent stiffness becomes fairly less in comparison with the Outrigger system now being the stiffest in both X and Y directions, seen in Table 5.8. The tube system has the best resistance against torsion, being a little stiffer than the outrigger system.

Table 5.8 Time period (longest) and stiffness (EI) when using about the same amount of concrete

System	T (s)	Stiffness Y	Stiffness X	Stiffness R
TMF	3,345	8,42E+10	1,49E+11	6,08E+11
Outrigger	3,331	8,44E+10	2,38E+11	2,05E+12
Tube in a tube	3,504	7,51E+10	2,13E+11	2,10E+12

To test the Outrigger and TMF systems at greater heights models are created by adding floors to previous models, making them grow vertically to 480 meters with 160 stories. These models are not designed to meet the requirements for drift and deflection but are simply done to receive a general view of how the systems respond to an increase of height. The Outrigger system uses roughly about 14 % more weight with four times as much weight of steel as the TMF does. Stiffness in the two general directions is greater in the TMF while torsional stiffness is four times greater in the Outrigger system. Table 5.9 displays time period and stiffness and Figure 5.8 displays the deflection curve.

Table 5.9 Time period (longest) and stiffness (EI) for the 480 meter models

System	T (s)	Stiffness Y	Stiffness X	Stiffness R
TMF480m	27,50	7,22E+09	1,25E+10	2,40E+11
Outrigger480m	35,63	4,94E+09	1,13E+10	9,21E+11

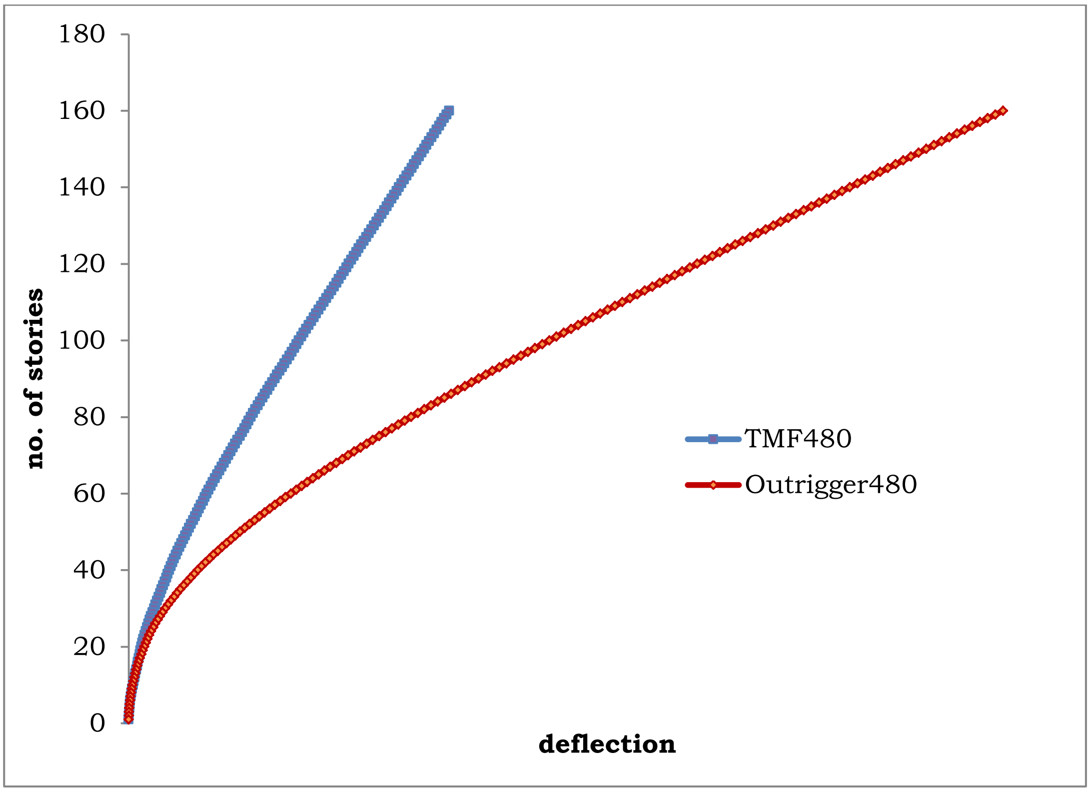


Figure 5.8 Deflection curves for the 160 story models

5.3 **TMF with different geometries**

The TMF model used in the previous analysis is one of the earliest versions of how such a system might look like, adapted for the new elevator system. Although this is not representative of the TMF system as a whole because as shown in the Figure 3.6 the TMF may be present in many different shaping to meet stipulated requirements. This analysis is aimed to evaluate the placement of the tubes with varied geometries at the facade on the basis of the idea that increased lever arm from the building's center should provide a more stable system. Different design of connections between the tubes is also applied in this analysis.

Geometry adjustments have been made for the building of the dimensions 18x26 m in order to relate to the previous comparison with the other selected building system. The models are created in the same context as previous models with respect to concrete strength, load assigns, story height and the story placement of the tube connections. The exception is that the steel elements and door openings have been removed to speed up the modeling process. The analysis is first done with no consideration of designing the walls, 200 mm thick concrete walls is used all way up. To estimate that results are acceptable a design of walls is done too. Different versions with usable facade and floor area ratio are illustrated in Figure 5.8 to 5.15. The versions V1 – V5 are based on the same amount of the material, versions 1A -1C is the further development of the V1 because the usable area ratio for this model differs from the V4 - V5.

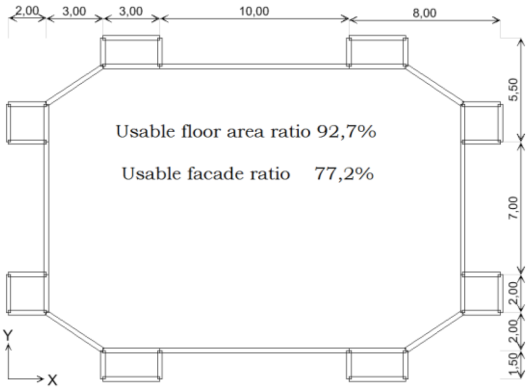


Figure 5.9 V1

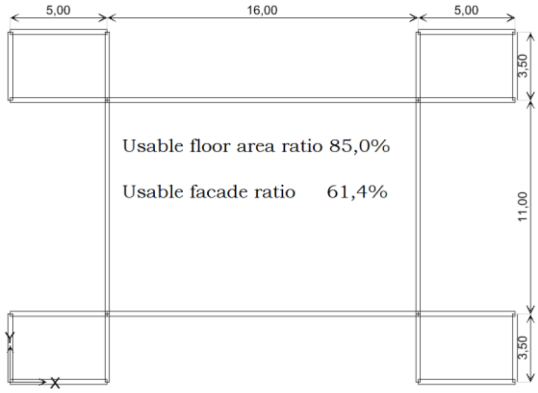


Figure 5.10 V2

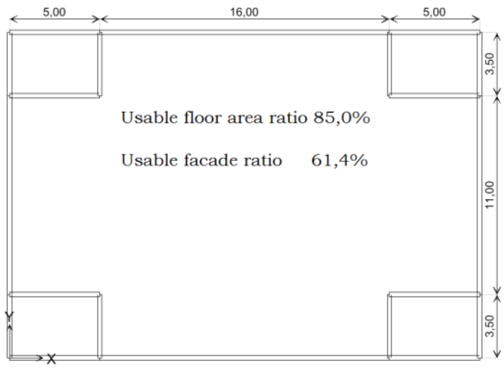


Figure 5.11 V3

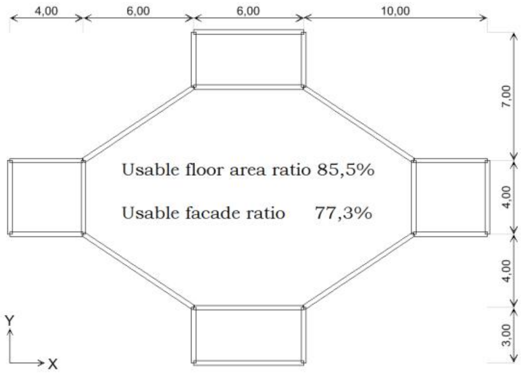


Figure 5.12 V4

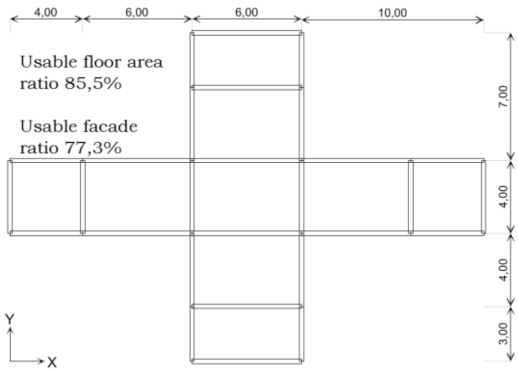


Figure 5.13 V5

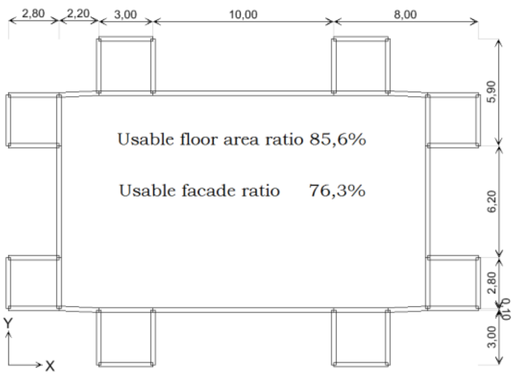


Figure 5.14 V1A

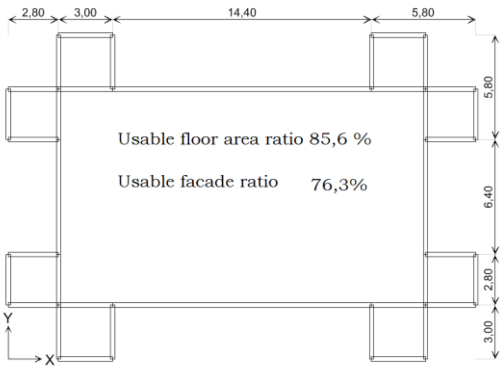


Figure 5.15 V1B

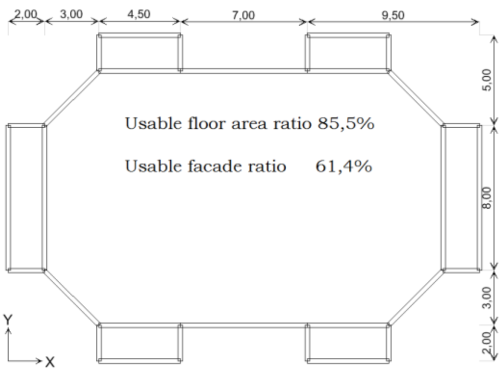


Figure 5.16 V1C

The first result of the analysis of V1-V5, with no consideration of designing the walls, is expressed as buildings equivalent stiffness in different directions; X, Y and rotational stiffness R in Table 5.10 and Table 5.11. Based on the first result V5, which represents the TMF model used in Chapter 5.2.5, is about as stiff as the other models, besides the V1, in all directions. The V1A – V1C generally inhabits much greater stiffness but also contains more material, about 34%, than in the V1-V5 due to the geometry of the tubes.

Table 5.10 Equivalent stiffness (EI) for V1-V5

	Stiffness Y	Stifness X	Stiffness R
V1	3,93E+10	6,57E+10	1,77E+11
V2	7,30E+10	1,21E+11	4,15E+11
V3	7,29E+10	1,21E+11	4,75E+11
V4	6,38E+10	1,17E+11	4,75E+11
V5	6,41E+10	1,15E+11	4,54E+11

Table 5.11 Equivalent stiffness (EI) for V1A-C

	Stiffness Y	Stiffness X	Stiffness R
V1A	8,52E+10	1,21E+11	4,01E+11
V1B	1,18E+11	1,46E+11	5,50E+11
V1C	1,20E+11	1,29E+11	6,21E+11

As the V1 does not seem to be a stiffer version than V5, which was expected a different analysis is done. The tubes in models V1-V5 is connected at every other floor. The results are illustrated in Table 5.12. In this case the V1 becomes stiffer than V5.

Table 5.12 Stiffness (EI) for V1-5 with Outriggers connected at every other floor

	Stiffness Y	Stifness X	Stiffness R
V1	1,88E+11	3,55E+11	2,34E+12
V2	2,12E+11	4,08E+11	1,17E+12
V3	2,16E+11	4,15E+11	3,05E+12
V4	1,35E+11	2,56E+11	1,98E+12
V5	1,37E+11	2,53E+11	1,35E+12

In view of the results five promising models have been selected for the design of walls with the requirement that the tubes shall withstand the forces which have occurred by increasing wall thickness. From Table 5.13 it can be seen that V1 has the lowest stiffness while V1C has greatest stiffness but uses more material. All of these models succeed in meeting the requirements for the highest allowed drift and deflection which is 0.25%. In this case V3 cope with the requirements using least material.

Table 5.13 Designed Tubed mega frame models, displaying stiffness (EI) and weight

				Weight [kN]	
	Stiffness Y	Stiffness X	Stiffness R	Walls	Total
V1	4,56E+10	7,59E+10	1,97E+11	54,1E+3	156,6E+3
V1A	8,76E+10	1,21E+11	4,06E+11	68,2E+3	162,9E+3
V1C	1,21E+11	1,32E+11	6,32E+11	66,9E+3	161,7E+3
V3	7,90E+10	1,27E+11	4,92E+11	52,4E+3	146,5E+3
V5	6,84E+10	1,21E+11	4,79E+11	52,4E+3	147,1E+3

6 CONCLUSIONS AND FURTHER STUDIES

6.1 Conclusions

It is not a simple task to determine which of the stabilization system that is most effective because there appears to be no universal solution to meet all possible requirements that may arise. Some systems are best suited taking into account certain factors, but has disadvantages over others. Based on the limitations that are defined in Chapter 1.3 and Chapter 5 following conclusions can be drawn.

The 2D analysis shows that Figure 2.23 appears to be fairly correct when generally looking at high-rise buildings. Compared to Figure 5.2, which shows deflection, it is possible to approximately put these systems in the graph at Figure 2.23 with the system that has the most deflection having the least amount of stories attainable.

In the ETABS analysis of the 40 story building the three different systems each seem to have their respective benefits and disadvantages. Whereas the Outrigger system displays great stiffness its drawbacks are the amount of steel it uses and the extra number of stories the outrigger arms inhabit. The Tubed mega frame demands a high amount of concrete, more because of its geometry than the handling of loads. As with the Outrigger system some floors are sacrificed to make room for the outriggers. The TMF does however use less steel and still has good stiffness, especially in the structures weak direction that is of outmost importance. Perhaps the most effective system at 40 stories is the Tube in a tube which does not require that much more steel than the TMF and the least amount of concrete, still being able to meet the requirements for deflection and drift. The tube in a tube system also has the benefit of not having to use outriggers, taking up space at certain floors, but has a drawback in that it uses a lot of room at the façade.

Geometry is one of the most important aspects when it comes to the TMF. With other systems the geometry is often more static, not changeable other than with a wider or thinner core. The Tubed mega frame can adapt its shape depending on the situation, as seen in Chapter 5.3. One conclusion can be drawn; that the stiffer the TMF has to become the more façade area has to be sacrificed or a longer outrigger arm be used. Compared to systems using a central core the TMF has less rotational stiffness. This can be increased by better connecting the mega columns to each other; either by using outriggers at more stories or better connecting the floors to the mega columns.

After comparing higher structures in ETABS it becomes evident that the Tubed mega frame does increase in effectiveness compared to other structures the higher the structure gets. The statement made from (Ching, Onouye, & Zuberbuhler, 2009) that a mega frame structure is suited for buildings with extreme height seems viable. The graph in Figure 2.23 can therefore be complemented as seen in Figure 6.1.

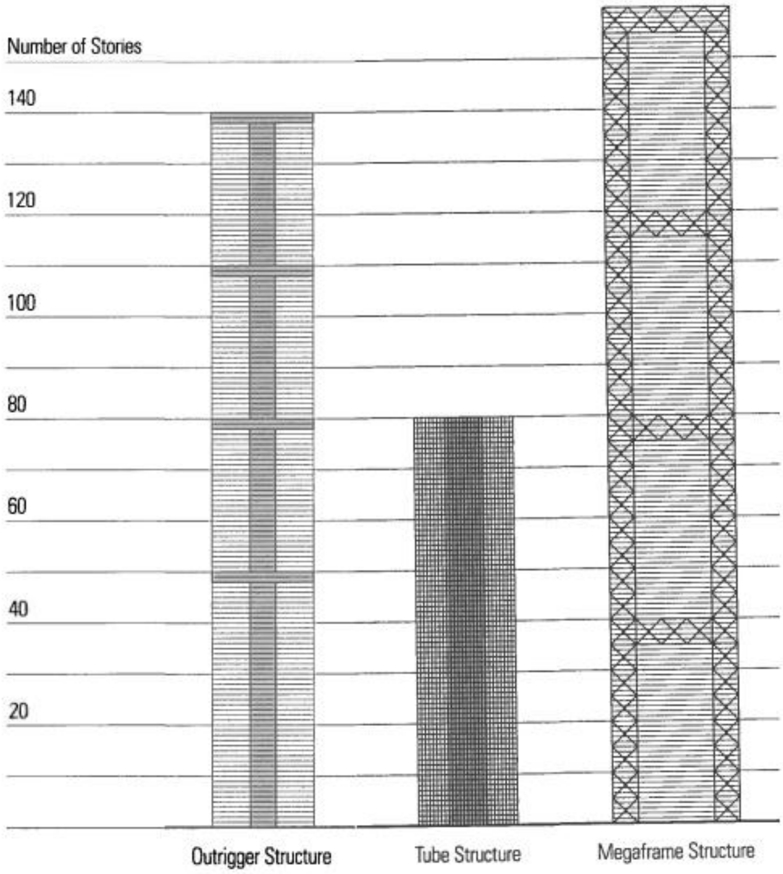


Figure 6.1 Complemented reasonable attainable no. of stories, as according to Ching, Onouye & Zuberbuhler. Here describing a mega frame system using vertical trusses as mega columns

6.2 **Further Studies**

A limitation made with the outriggers in the TMF models is that they are designed as walls rather than high beams which perhaps might be the correct way to analyze them. The long spans might be hard to achieve and may need to be more looked upon. Analyzing these outriggers as walls showed great shear forces that demands very thick walls along with a great modulus of elasticity. Further studies are required for these arms, their connection to the concrete mega columns and to minimize the shear forces.

The premium for height, first shown in Figure 2.1, can be developed further to fit a general picture of the necessary material quantities for each individual system. Figure 6.2 is a vague picture of what the systems analyzed with ETABS in this report could look like.

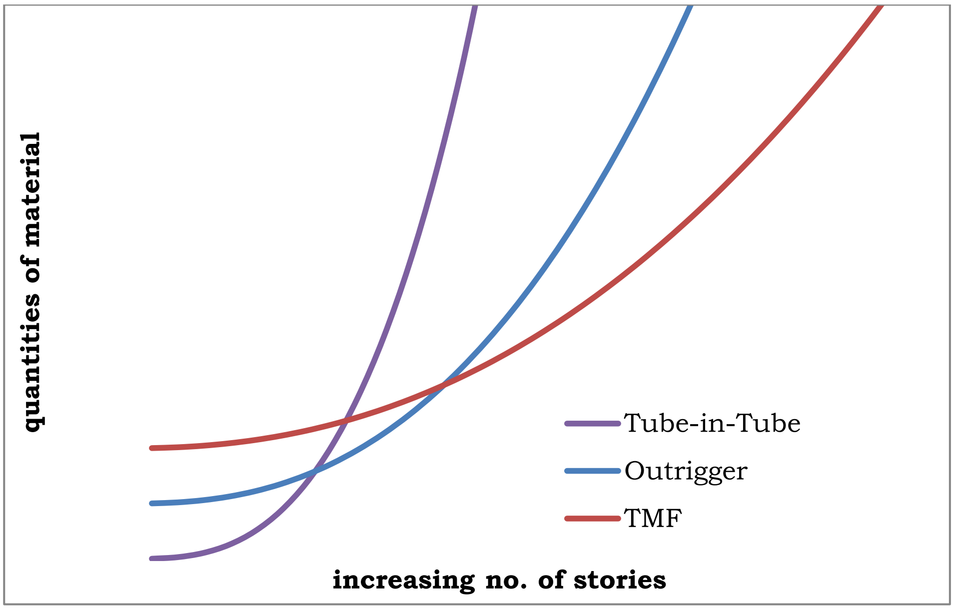


Figure 6.2 Cost of height diagram for what the Tube in a tube-, Outrigger and TMF systems might look like

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8 APPENDIX A: CALCULATIONS FOR LOADS

8.1 Approximate wind load for an 18x30m building with 20 stories.

The wind is calculated having its direction through the buildings far side and 180° towards the roof.

Data

- Terrain type IV
- Total height $\boldsymbol{z = 60\ m}$
- Story height $\boldsymbol{h = 3\ m}$
- width $\boldsymbol{b = 30\ m}$
- depth $\boldsymbol{d = 18\ m}$
- direction factor $\boldsymbol{c_{dir} = 1,0}$
- season factor $\boldsymbol{c_{season} = 1,0}$
- Referenced wind speed $\boldsymbol{v_{,0} = 24\ m/s}$
- Air density $\boldsymbol{\rho = 1,25\ kg/m^3}$
- Exposure factor $\boldsymbol{c_e(z) = 2,5}$
- Factor for outside wind

$$\boldsymbol{c_{pe,10,wall,far\ side,windward} = 0,8}$$
$$\boldsymbol{c_{pe,10,wall,far\ side,leeward} = 0,5}$$

Loads

Referenced wind speed $\boldsymbol{v_b = c_{dir}c_{season}v_{,0} = 24\ m/s}$

Referenced pressure $\boldsymbol{q_b = \frac{1}{2}\rho {v_b}^2 = 360\ N/m^2}$

Characteristic pressure $\boldsymbol{q_p = c_e(z)q_b = 900\ N/m^2}$

Characteristic outside wind load

$$\boldsymbol{W_{e,wall,far\ side,windward} = q_p c_{pe,wall,far\ side,windward} = 720\ N/m^2}$$
$$\boldsymbol{W_{e,wall,far\ side,leeward} = q_p c_{pe,wall,far\ side,leeward} = 450\ N/m^2}$$
$$\boldsymbol{W_{e,tot} = W_{e,wall,far\ side,windward} + W_{e,wall,far\ side,lee} = 1170\ N/m^2}$$

Characteristic outside wind load as joint load

$$\boldsymbol{H_{e,i} = W_{e,tot}\frac{b}{2}h = 52650\ N \approx 50\ kN}$$

8.2 Approximate wind load for an 18x26m building with 40 stories.

Data

- Terrain type IV
- Total height $\boldsymbol{z = 120\ m}$
- Story height $\boldsymbol{h = 3\ m}$
- width $\boldsymbol{b = 26\ m}$
- depth $\boldsymbol{d = 18\ m}$
- direction factor $\boldsymbol{c_{dir} = 1,0}$
- season factor $\boldsymbol{c_{season} = 1,0}$
- Referenced wind speed $\boldsymbol{v_{,0} = 24\ m/s}$
- Air density $\boldsymbol{\rho = 1,25\ kg/m^3}$
- Exposure factor $\boldsymbol{c_e(120) = 3,3}$
- Exposure factor zmin $\boldsymbol{c(10) = 1,2}$
- Factor for outside wind

$$\boldsymbol{c_{pe,10,wall,far\ side,windward} = 0,8}$$
$$\boldsymbol{c_{pe,10,wall,far\ side,leeward} = 0,5}$$

Loads

Referenced wind speed $\boldsymbol{v_b = c_{dir}c_{season}v_{,0} = 24\ m/s}$

Referenced pressure $\boldsymbol{q_b = \frac{1}{2}\rho v_b{}^2 = 360\ N/m^2}$

Characteristic pressure $\boldsymbol{q_{p1} = c_e(z)q_b = 1188\ N/m^2}$

$$\boldsymbol{q_{p2} = 432\ N/m^2}$$

Characteristic outside wind load

$$W_{e,wall,far\ side,windward1} = q_{p1}c_{pe,wall,far\ side,windward} = 950\ N/m^2$$
$$W_{e,wall,far\ side,windward2} = 345\ N/m^2$$
$$W_{e,wall,far\ side,leeward1} = q_{p1}c_{pe,wall,far\ side,leeward} = 594 N/m^2$$
$$W_{e,wall,far\ side,leeward2} = 216$$
$$W_{e,tot1} = W_{e,wall,far\ side,windward1} + W_{e,wall,far\ side,lee1} = 1544,4\ N/m^2$$
$$W_{e,tot2} = 648\ N/m^2$$

Characteristic outside wind load as joint load on story 4-40

$$H_y = W_e \cdot 26 = 40154,4\ N/m$$
$$H_x = W_e \cdot 18 = 27799,2\ N/m$$

Characteristic outside wind load as joint load on story 1-3

$$H_y = 16848\ N/m$$
$$H_x = 11664\ N/m$$

8.3 Approximate snow load for an 18x26m building with 40 stories.

Data

• Location	Stockholm
• Topography	Normal
• Roof pitch	0°
• Snowload shapefactor	$\mu_1 = 0{,}8$
• Exposure factor	$C_e = 0{,}8$
• Thermal coefficient factor	$C_t = 1{,}0$
• Characteristic ground snowload	$s_k = 2{,}0$

Characteristic snowload

$$s = \mu_1 \cdot C_e \cdot C_t \cdot s_k = 1{,}28\ kN/m^2$$

9 APPENDIX B: DESIGN OF COMPONENTS

Model check

Tube in a tube

Dead load

$\text{Floor} := (26 \cdot 18 - 6 \cdot 6) \cdot 0.25 \cdot 23.56 = 2544.48$ kN

$\text{Walls} := 2 \cdot (6 \cdot 3 + 6 \cdot 3 - 2.4 \cdot 2) \cdot 0.35 \cdot 23.56 = 514.55$ kN

$\text{Steel\_Beams} := (26 + 18) \cdot 2 \cdot 44 \cdot 0.015 = 58.08$ kN

$\text{Steel\_Columns} := 28 \cdot 283 \cdot 0.015 = 118.86$ kN

$\text{Total} := (\text{Floor} + \text{Walls} + \text{Steel\_Beams} + \text{Steel\_Columns}) \cdot 40 = 129438.816$ kN

According to ETABS Total support reaction = 128662 kN

Check OK

Wind load - shear

$\text{Hy} := 120 \cdot 40 = 4800$ kN

According to ETABS Total support reaction = 4558 kN

Check OK

Wind load - moment

$\text{Mx} := \text{Hy} \cdot \frac{120}{2} = 288000$ kNm

According to ETABS Total support reaction = 286833 kNm

Check OK

Outrigger

Dead load

$Floors := (26\cdot18 - 6\cdot6)\cdot0.25\cdot23.56 = 2544.48$ kN $Floors\cdot40 = 101779.2$

$Walls := 2\cdot(6\cdot3 + 6\cdot3 - 2.4\cdot2)\cdot0.35\cdot23.56 = 514.55$ kN $Walls\cdot40 = 20582.016$

$Steel\_Beams := (26 + 18)\cdot2\cdot70\cdot0.015 = 92.4$ kN $Steel\_Beams\cdot40 = 3696$

$Steel\_Columns := 12\cdot3\cdot108\cdot0.015 = 58.32$ kN $Steel\_Columns\cdot40 = 2332.8$

$Steel\_Braces := 12\cdot53\cdot0.015 = 9.54$ kN $Steel\_Braces\cdot40 = 381.6$

$Total := (Floors + Walls + Steel\_Beams + Steel\_Columns)\cdot40 + Steel\_Braces\cdot2 = 128409.096$

According to ETABS Total support reaction = 129800 kN

Check OK

Wind load - shear

$Hy := 120\cdot40 = 4800$ kN

According to ETABS Total support reaction = 4558 kN

Check OK

Wind load - moment

$Mx := Hy\cdot\frac{120}{2} = 288000$ kNm

According to ETABS Total support reaction = 286833 kNm

Check OK

10 APPENDIX C: SYSTEMS IN SAP2000

The following figures shows the models designed in SAP2000 with their respective releases at endpoints.

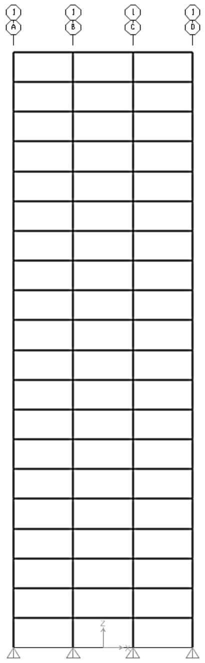

Figure 10.1 Rigid Frame

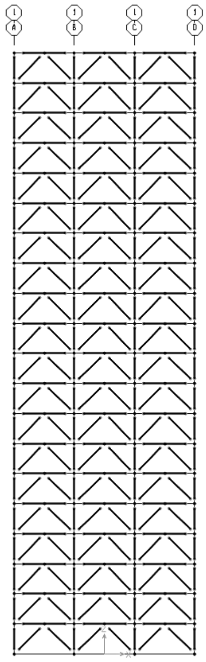

Figure 10.2 Completely Braced Frame

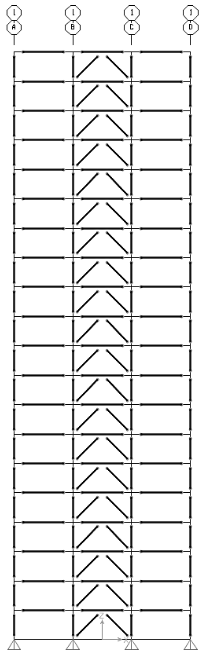

Figure 10.3 Partially Braced Frame

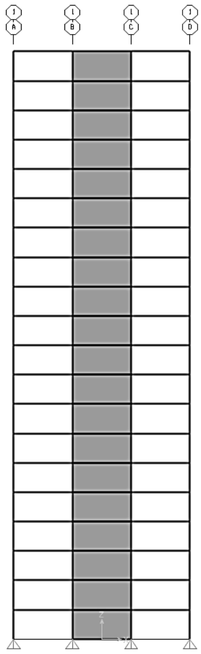

Figure 10.4 Shear Wall and Rigid Frame

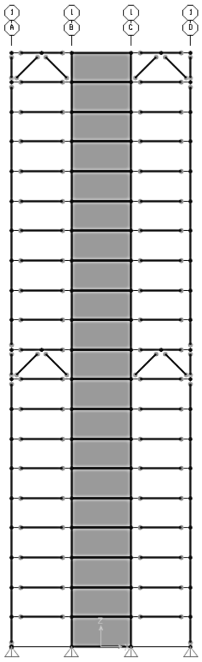

Figure 10.5 Shear Wall and Outrigger

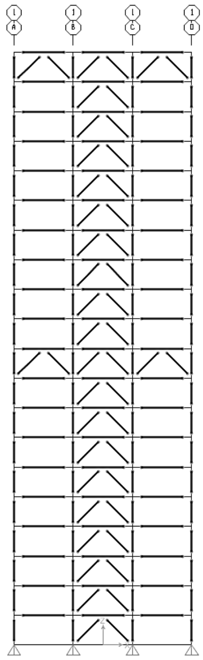

Figure 10.6 Braced Core and Outrigger

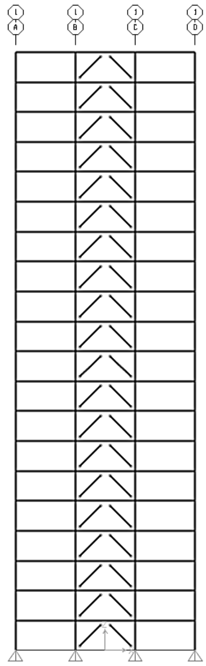

Figure 10.7 Rigid Braced Frame

11 APPENDIX D: SYSTEMS IN ETABS

Following figures show the models made in ETABS.

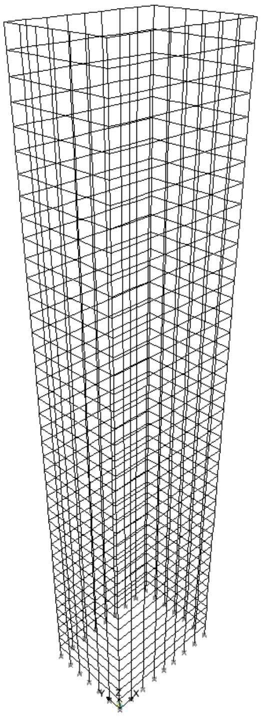

Figure 11.1 Framed tube

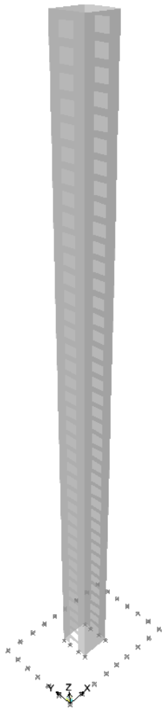

Figure 11.2 Core

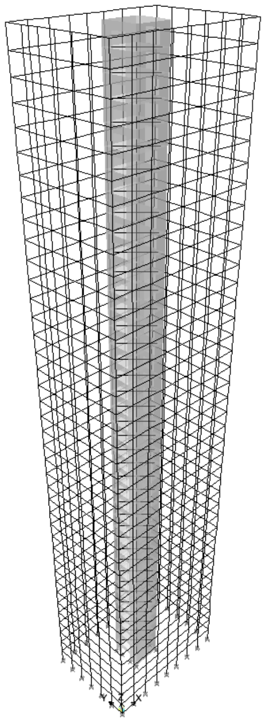

Figure 11.3 Tube in a tube

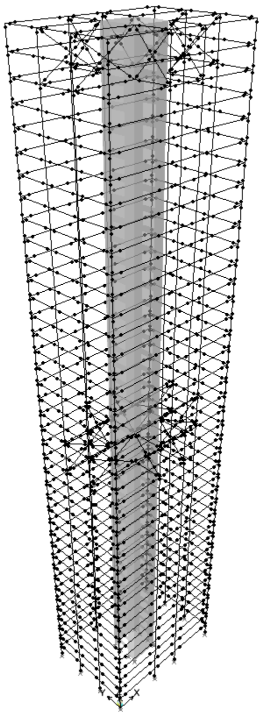

Figure 11.4 Outrigger

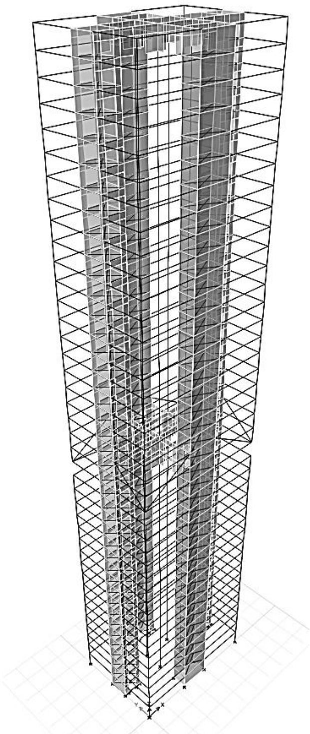

Figure 11.5 Tubed mega frame

12 APPENDIX E: EXCEL SHEETS

	Weak	y	x	Torsion		Deflection		Total		Stiffness				
	Mode	T, mode1	T, mode2	T, mode3		Wind X	Wind Y	Weight	Y	X	Torsion			
Framed tube	1	1,869	1,440	1,260	3	0,393	0,960	6629	9,92E+09	1,67E+10	2,18E+10			
Core	2	2,153	2,489	0,337	5	0,544	0,575	19562	2,20E+10	1,65E+10	9,00E+11			
Tube in tube	1	1,927	1,784	0,702	3	0,194	0,330	26191	3,69E+10	4,30E+10	2,78E+11			
												Steel Columns		Concrete
												Ratio	Failure	Failure
TinT	1	4,800	4,466	1,941	3	0,194	0,330	136452	3,09E+10	3,58E+10	1,89E+11	0,743	no	no
TinTuncracked	1	4,799	4,465	1,939	3	0,193	0,324	136452	3,10E+10	3,58E+10	1,90E+11			
TinT without openings	1	4,763	4,173	1,354	3	0,168	0,316	137924	3,18E+10	4,14E+10	3,93E+11			
TinT bigger core	1	3,992	2,614	1,144		0,068	0,224	149212	4,89E+10	1,14E+11	5,96E+11			
TinT P-delta	1	5,046	4,665	1,984	3	0,211	0,360	136452	2,80E+10	3,28E+10	1,81E+11	0,756	no	yes
TinT sequence	1	4,800	4,466	1,941	3	0,193	0,324	136664	3,10E+10	3,58E+10	1,89E+11	0,706	yes	no
												Steel		
												Ratio	Weight	
Outrigger	1	4,480	4,011	2,336	3	0,017	0,017	137258	3,57E+10	4,46E+10	1,31E+11			
Outrigger redim.	1	4,440	3,977	2,285	3	0,017	0,017	137846	3,65E+10	4,55E+10	1,38E+11	0,624	7224	
TinT redim.	1	4,767	4,455	1,906	3	0,018	0,017	137252	3,16E+10	3,61E+10	1,97E+11	0,582	6629	
												Weight		
												Steel	Concrete	
TMF	1	3,562	2,676	1,415	3	0,013	0,014	174781	7,20E+10	1,27E+11	4,56E+11	5702	58818	
Outrigger	1	3,641	2,290	1,104	3	0,017	0,017	156657	6,18E+10	1,56E+11	6,72E+11	9509	36888	
TinT	1	4,081	2,681	1,150	3	0,017	0,018	148039	4,64E+10	1,08E+11	5,85E+11	6658	31120	
TMF	1	3,345	2,511	1,244	3			180190	8,42E+10	1,49E+11	6,08E+11	5702	64226	
Outrigger	1	3,331	1,984	0,676	3			179237	8,44E+10	2,38E+11	2,05E+12	9509	59422	
TinT	1	3,504	2,079	0,662	3			176341	7,51E+10	2,13E+11	2,10E+12	6658	59422	
TMF_480		27,503	20,932	4,774				1045475	7,22E+09	1,25E+10	2,40E+11			
Outrigger_480		35,639	23,577	2,609				1199716	4,94E+09	1,13E+10	9,21E+11			

	y T, mode1	x T, mode2	Torsion T, mode3	Stiffness Y	Stifness X	Stiffness R	Total Weight
V1_out	2,1612	1,5706	0,6118	1,88E+11	3,55E+11	2,34E+12	167625
V2_out	1,993	1,4375	0,8478	2,12E+11	4,08E+11	1,17E+12	161327
V3_out	1,9744	1,4247	0,5257	2,16E+11	4,15E+11	3,05E+12	161327
V4_out	2,4365	1,7677	0,6357	1,35E+11	2,56E+11	1,98E+12	152897
V5_out	2,5034	1,8386	0,7964	1,37E+11	2,53E+11	1,35E+12	163907

	y T, mode1	x T, mode2	Torsion T, mode3	Stiffness Y	Stifness X	Stiffness R	Weight Total	Weight Walls	Area of the tubes	Ratio of facade
V1	4,511	3,4894	2,1231	3,93E+10	6,57E+10	1,77E+11	153080	50629	0,927	0,773
V2	3,2232	2,5055	1,3516	7,30E+10	1,21E+11	4,15E+11	145106	50925	0,850	0,614
V3	3,2255	2,5042	1,2639	7,29E+10	1,21E+11	4,75E+11	145106	50925	0,850	0,614
V4	3,4362	2,5361	1,2599	6,38E+10	1,17E+11	4,75E+11	144232	49591	0,855	0,773
V5	3,4485	2,5787	1,2952	6,41E+10	1,15E+11	4,54E+11	145884	51243	0,855	0,773

	y T, mode1	x T, mode2	Torsion T, mode3	Stiffness Y	Stifness X	Stiffness R	Weight Total	Weight Walls	Area of the tubes	Ratio of facade
V1A	3,1582	2,6522	1,4553	8,52E+10	1,21E+11	4,01E+11	162563	67776	0,856	0,736
V1B	2,6842	2,4155	1,2432	1,18E+11	1,46E+11	5,50E+11	162584	67796	0,856	0,736
V1C	2,6497	2,5563	1,1651	1,20E+11	1,29E+11	6,21E+11	161308	66520	0,855	0,614

	y T, mode1	x T, mode2	Torsion T, mode3	Stiffness Y	Stifness X	Stiffness R	Weight Total	Weight Walls	Area of the tubes	Ratio of facade
V1_des	4,2376	3,2828	2,0391	4,56E+10	7,59E+10	1,97E+11	156586	54135	0,927	0,773
V1A_des	3,1172	2,6534	1,4489	8,76E+10	1,21E+11	4,06E+11	162945	68157	0,856	0,736
V3_des	3,1123	2,4594	1,2476	7,90E+10	1,27E+11	4,92E+11	146534	52353	0,850	0,614
V1C_des	2,643	2,5261	1,1561	1,21E+11	1,32E+11	6,32E+11	161668	66880	0,855	0,614
V5_des	3,3517	2,5243	1,2664	6,84E+10	1,21E+11	4,79E+11	147085	52445	0,855	0,773