# Structures of tall buildings

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## Abstract

Tall buildings have lately become a problem to design because of the development of light weighted steel frames and wind loads. An important task of a structure is the absorption of horizontal loads and the capacity to transfer the resulting moment down into the ground. It is the engineer's task to make sure that the structure's performance is sufficient when it comes to safety and serviceability. The most important thing to consider is the human comfort. In this thesis the program 3D Structures were used and three different structures were built up and compared regarding their behaviour against wind loads. Frequencies and displacements were calculated and summed up. In addition three different codes were studied regarding calculations of acceleration to see the difference between countries. It is shown that the frequency increases when the building is getting stiffer and it decreases when the building is getting heavier. Finally the stiffness between the structures was compared.

Key words: Tall buildings, Wind loads, Comfort, Frequency, Acceleration

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# Preface

This thesis is done in cooperation with ELU Konsult AB in Stockholm and is made for the Division of Structural Engineering at the Faculty of Engineering at Lund University. I would like to thank professor Sven Thelanderson from Division of Structural Engineering and Peter Granberg from ELU Konsult AB, for all the help and support. A big thank to my family and friends for their support and encouragement.

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# Contents

# Summary

There are three periods to divide the development of tall buildings into, the first period starts around 1895, the second around 1930 and the third around 1950. The periods represent the development of different kinds of structures.

There is no specific definition of a tall building but different countries have their own definition. There is an international guideline for the height, which considers a building to be a tall building when it is taller than 35 metres or 12 storeys.

The two basic sources for loads are geophysical and man-made. The geophysical consists of gravitational, meteorological and seismological forces and the man-made depends on shocks. The weight of the material of a building is called dead loads while loads that are variable over time and variable over location are called live loads. Live loads are variable over time as well as of location

It was first in the 50's, when the development of new system such as light weighted steel frames, that the wind load became a problem when designing a tall building. Previous to this, the weight limited the height of a building. The number one concern when constructing a tall building is human comfort. The wind affects the building in three ways. There is an along-wind force, an across-wind force and torsional moments.

The structure of a tall building contains linear elements, surface elements and spatial elements. To design the structure, knowledge of the loads to be carried, properties of the building materials and the way the forces are being transferred trough the structure down into the ground is needed. The primary task of a structure is the absorption of horizontal loads and the capacity to transfer the resulting moment down into the ground. There are many different kinds of structures used in a building.

Comfort has to consider the frequency, type of activity and the location of the occupant. An early reference about human comfort has stated that: for buildings to be comfortably habitable, the structure should be designed in such way to prevent great bending. The first criterion of occupant's response to acceleration was given by Reed and he was the one who was primarily responsible for the criteria in ISO 6897.

The motion characteristics of tall buildings depend on structural design, aerodynamic conditions and damping.

Different countries have different codes, in which calculations of acceleration is presented. After calculations, according to the codes, it is shown that the results from the different codes are almost the same.

A case study of structures built up in 3D Structures gives the frequencies of the structure. It shows that for a heavier building the frequency decreases and for a stiffer building the frequency increases.

# 1. Introduction

## 1.1 Goal

The major goal of this thesis is to understand the best way to build a tall building regarding the comfort of its inhabitants. This goal will be divided into three parts. The first part is to study and compare codes from different countries, to see their way of calculating acceleration for tall buildings. The second part is to build up structures in the program 3D-Structures and study how they resist wind loads. Finally, the third part will contain a comparison between the structures regarding frequencies and accelerations.

## 1.2 Method

Literature searching and studying was the first thing that was made for this thesis. The literature was read to get an understanding and a background of the theory.

Three different codes have been studied regarding the parts about calculations of acceleration.

For the case study, a program called 3D Structures was used. The three structures, which were studied in the thesis, were built up in 3D Structures with beams, columns, walls and slabs. Forces were applied on the structure and then calculations for displacement and frequencies were made. The result was summed up in tables and then compared and analyzed.

Finally the frequencies from the case study were used in calculation of the acceleration with the equations from the codes. The calculations were made with the program MathCAD.

## 1.3 Limitations

The studies have been limited to study only the parts regarding buildings over 100 metres in the codes and only calculation of acceleration has been studied.

To be able to make a comparison of the structures the same conditions has been used. The wind loads are assumed to be the same on every storey, a load of 5 kN/m. The same dimensions have been used for all the beams and columns in the building even if this is not the best solution regarding economy. The dimension is VKR 400x400x16 with steel S355J0 as material. The walls in the cores are concrete C50/60 and the slabs are concrete C30/37. Safety class is 3 and exposure class is X0. The thickness of the walls of the cores and the slabs, which will be studied are 0.2 m, 0.4 m and 0.6 m.

## 1.4 History

The development of tall buildings can be divided into three periods. The first period starts around 1895 when most of the buildings were built up with masonry walls that carried the entire structure. The walls were thick and messy and they alone mainly resisted the horizontal and lateral loading. The walls could be close to two metres thick. About 1930 the second period started with an evolution of steel structures. Corporations started to see advantages of publicity when connecting their company name with imposing tall buildings and the demand of tall buildings increased. In the third period, around 1950, reinforced concrete started to establish its own identity. The third period is now considered as modernism in construction history and put the architectural emphasis on reasons, functional and technological facts as a contrast to the first and the second period which put the emphasis on external dressing and historical style, (Chew, Y.L.M, 2001). Figure 1 show three different buildings built in each period.

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*Figure 1. Reliance Building 1894 (GNU), Empire State Building 1931 (GNU) and Sears Tower 1974.*

# 2 Theory

## *2.1 Definition*

There is no definition of a tall building. Although, general consensus seems to be, when the structure and the design of the building are affected by lateral loads, that it may be considered as a tall building. It is particularly the sway caused by such loads that are affected, (Chew, Y.L.M, 2001).

In Germany the following definition has been made, by dictates of fire protection and the effective use of fire escapes:
“High-rises are buildings in which the floor of at least one occupied room is more than 22 m above the natural or a prescribed ground level”.
There are other building laws in other countries that define a tall building; these definitions are between 13 metres and 50 metres. It is therefore difficult to generalize the definition of tall buildings internationally. As a guideline, international databases such as Skyscrapers.com, has chosen a height of 35 metres or 12 storeys of the building, (Eisele, J., Kloft, E.). The tallest buildings in the world so far are presented in Figure 2.

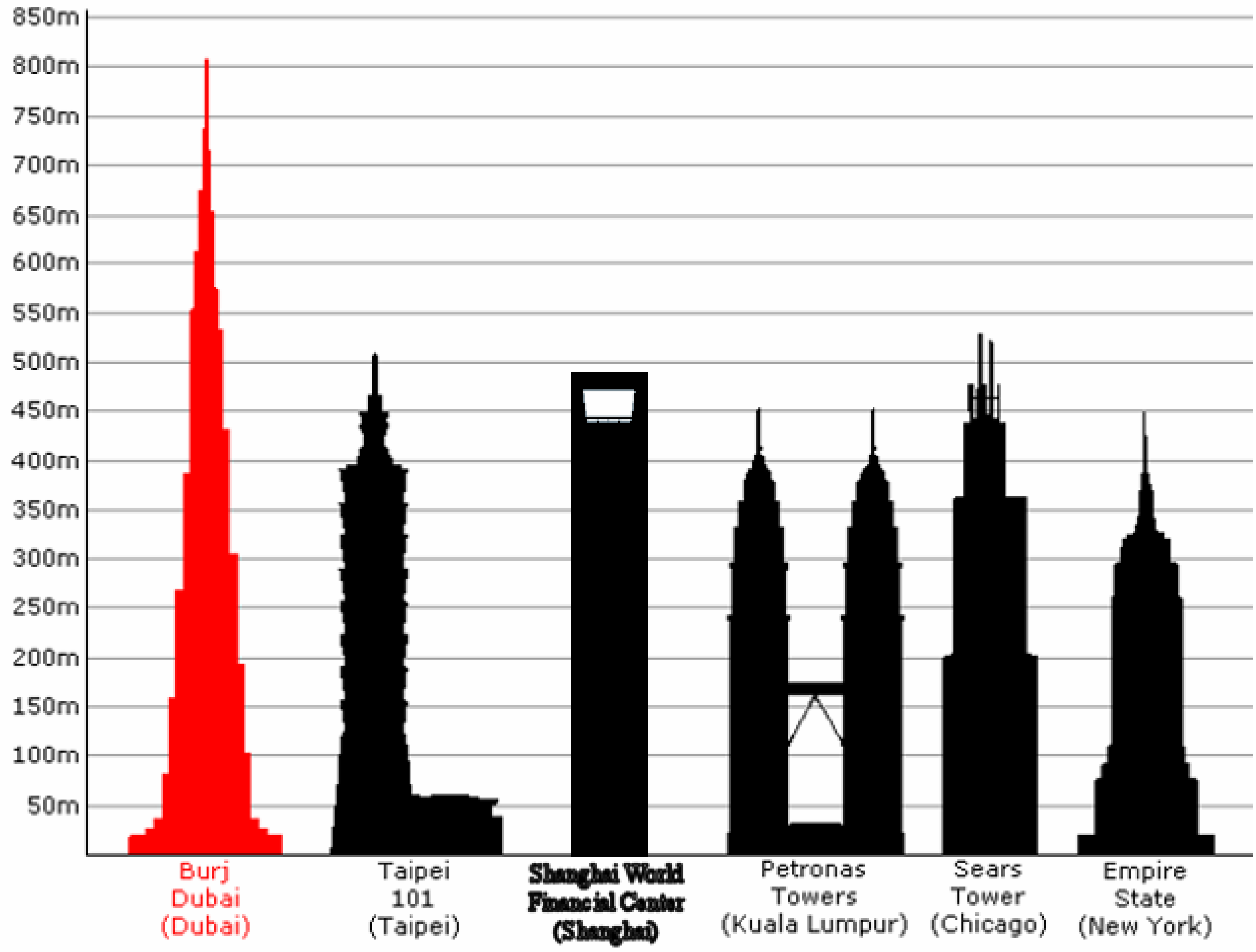



*Figure 2. The tallest buildings in the world so far. (GNU)*

## 2.2 Loads

Loads have two basic sources, geophysical and man-made. Processes in nature are what constitute the geophysical load and this can further be divided into gravitational, meteorological and seismological forces. The weight of the building is producing by itself a force on the structure that is called dead load and is a result of the gravity. The dead load is constant throughout the building's life span. A variation of loads over a period of time is produced of the ever-changing occupancy and is also a result of gravity. The meteorological forces are appearing as wind, snow, rain, humidity and ice and they vary with time and location. Finally in the geophysical loads there are seismological forces that are produced of example earthquakes. The mass, size, shape and material of a building is what influence the geophysical force action.

Man-made loads depend of different kinds of shocks. These shocks are generated by cars, elevators, machines or the movement of people and equipment or the result of blast and impact.

Forces can also be induced by getting locked into the structure during the manufacturing and construction or by prestressing that may be required for the stability of the building, (Wolfgang, S., 1977).

### 2.2.1 Dead loads

The weight of every element in the structure cause static forces that are defined as dead loads. The dead loads, that is to say the weight of the material seems to be easy to determine, but the estimation of dead loads may differ with 15-20% or more. It is difficult to determine the dead loads because when an accurate analysis of the loads is made, for example in the beginning of the building process, it is impossible for the engineer to predict the exact weight of material that has not been chosen yet, (Wolfgang, S., 1977).

### 2.2.2 Live loads

Live loads are variable over time as well as of location. Examples of live loads are snow and imposed load on floors.

### 2.2.3 Wind loads

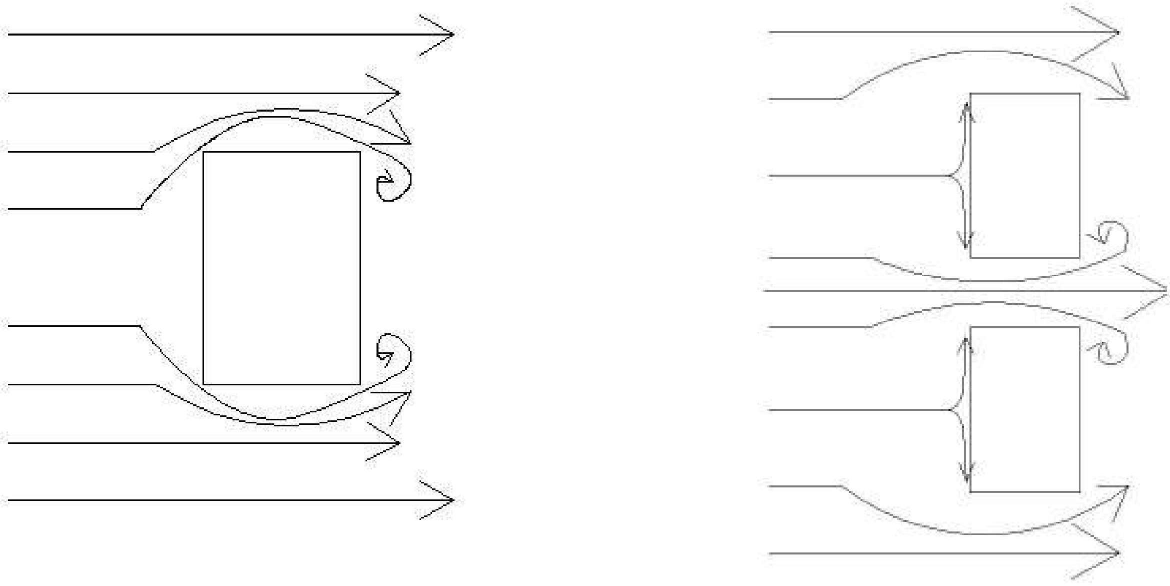
The first skyscrapers that were built had bearing masonry walls, which had huge weights. This was the reason why the wind loads could not overcome the locked in gravitational forces and therefore the buildings were not very sensitive to wind loads. The wind loads first became a problem in the 1950's, (Wolfgang, S., 1977). The development of new structural systems but also the usage of structural materials with high strength and light non-structural components has made modern buildings more flexible and more lightly damped than before, (Islam, M.S., et al., 1990). It was no longer the weight that limited the height of the building when new systems like light weight steel frame became more and more ordinary. The wind loads had become the main problem for constructing tall buildings. The lateral stiffness could be more significant than the strength of the building because of the new development. For engineering of tall buildings, the wind loads have become a huge problem to consider in design. Environmental factors, such as large-scale roughness and form of terrain, the shape, slenderness and façade texture of the structure itself, are what influence the wind action, (Wolfgang, S., 1977).

When designing a tall building the human comfort is a big concern, (Islam, M.S., et al., 1990). It is the engineer's job to reassure that the performance of the structure affected by wind loads is sufficient when it comes to safety and serviceability. To achieve this, information is needed for the engineer. The information consists of wind environment, relation between environment and the forces it induces on the structure and the structure's behaviour under the influence of the forces, (Simiu, E., et al., 1996).

Wind pressure is an overturning force that is created when an air mass is moving in a given direction and touches a surface to a building. The pressure depends on the wind velocity and the surface area. Double flexure in a building occurs when there is significant wind action on more than one surface of the building. It means that the original wind direction is being split into two components that apply the resulting wind on each side of the building. There may be both negative and positive effects on a building's motion regarding double flexure. Positive effects may be that the multi-directional displacement is less than it would be if the same amount of wind flow meets only one of the building's sides. Another positive effect is that for double flexure, the wind direction differs from 90 $^{o}$, and therefore much of the wind force is naturally dissipated. When the wind direction is perpendicular to the side of the building, the wind pressure is always greatest. A negative effect is that double flexure creates shear and torsional stresses on the structural parts and these stresses would not occur during unidirectional displacement.

The two components that wind pressure originates from are mean velocity and gust velocity. The resulting wind pressure corresponds to mean velocity averaged over a longer period of time. The resulting wind pressure is wielding a steady deflection on the building. There is another possible deflection created by a corresponding dynamic wind pressure, which is produced by the dynamic gust velocity. For slender buildings, this deflection could be dominant. Gust action creates random forces, which induce oscillation of buildings parallel to the wind direction.

An air mass in motion will move to each side of the building and then rejoin the major airflow when it meets a building. Turbulent air current will appear when the larger air mass moves in a steady flow with a high wind velocity towards a building, see Figure 3a. If the moving air mass is channelled trough a narrow space between two tall buildings, turbulence will occur. This is called the Venturi effect and is a type of turbulent wind action. The corresponding wind velocity in the space between the two tall buildings will exceed the wind velocity of the major air flow. Figure 3b shows the Venturi effect. As long as the air is in contact with the surface of the building, there are positive air pressures in any turbulent airflow. If the airflow is too rapid on the other hand, the air flow will leave the surface and negative pressure areas will occur. Turbulent winds generate air currents in these low pressure areas called vortices and eddies. Vortices create circular updrafts and suction streams nearby the building and they are high velocity air currents. Eddies are formed much the same as vortices but are slow moving circular air currents that creates little perceivable motion of a building, see figure 4, (Wolfgang, S., 1977).



*a* *b*

*Figure 3. a) Turbulence b) Venturi effect*

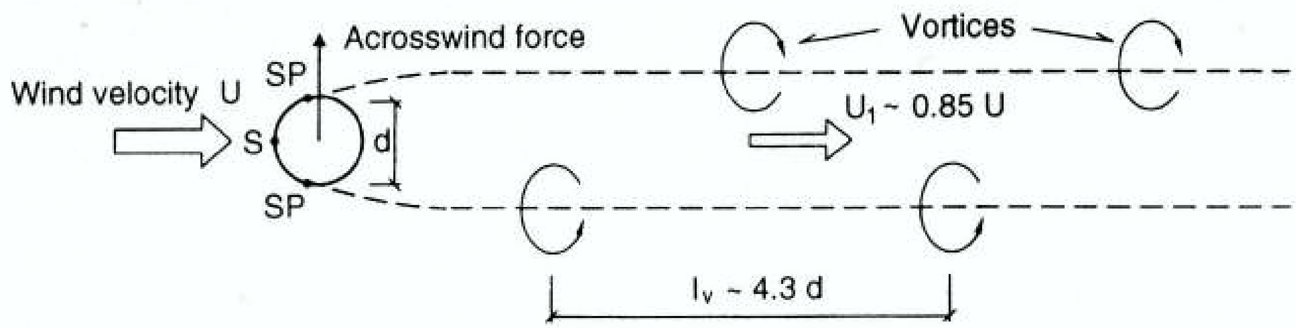



*Figure 4. Shows vortex shedding*

There is one force called along-wind, which goes in the same direction as the wind flow, and one force called across-wind, which goes perpendicular to the wind flow, both these forces are included in the aerodynamic forces. The building is also an object for fluctuating torsional moments because the resulting wind force rarely coincides with the elastic centre of the structure. It is easy to calculate the along-wind force by using classical theories of fluid mechanics. It has been shown on the other hand that across-wind and torsional forces cannot be calculated with analytical formulations. They have been proved as being too complex. The only method to use for calculation of these forces is wind tunnel test. To get the statistics of building response to fluctuating wind is to make the building as a multi-degree-of-freedom system, (Islam, M.S., et al., 1990).

## *2.3 Structures*

Structures contains of different kinds of members and the basic elements in a structure are linear elements, like columns and beams, surface elements, like walls and slabs and spatial elements, like façade envelope or core. Deciding the structure for a building depends on the knowledge of the following factors.

- The loads to be carried.
- Properties of the building materials
- The way the forces, developed from the loads, are being transferred through the structure down into the ground

(Wolfgang, S., 1977)

The answer to why a tall building's structure differs from other building's structure is fairly obvious; it is because they are higher. That the building is higher leads to higher vertical and lateral loads. The moment towards the base increases more rapidly than it would have done, if the horizontal load would have been constant, because in reality the horizontal load is increasing with the height, (Eisele, J., Kloft, E.). The columns weight per unit area increases linearly with the building height because the gravity load is increasing down the height of the building. On the other hand, there is a bending moment that the lateral forces are inducing that increases with at least the square of the height. This makes the lateral forces significant, (Stafford Smith, B., et al., 1991). I.e. if a building is twice as high as another it has to resist four times as large wind effects, (Chew, Y.L.M., 2001).

The primary task for a structural design of a building is the absorption of horizontal loads and the capacity to transfer the resulting moment down into the foundation. For the latter, stairwell cores are highly suitable, because of the continuous vertical elements. To treat the building as a clamped tube is another variant to transfer the loads, (Eisele, J., Kloft, E.)

The higher a building is, the stiffer it has to be to resist lateral forces. Almost all tall buildings were, up to the 1960's, designed as rigid frames but as the structural engineering, numerical modelling, wind tunnel tests etc. progressed, more efficient structural systems could be developed. These systems include the frame tube, semi-rigid frame and the composite structure combining the use of reinforced concrete and steel, (Chew, Y.L.M., 2001).

The stiffness of the building often comes from the walls of the core. The core is placed in the middle of the ground plan to minimize torsion stresses and enlarge lateral stiffening. Fire safety regulations demands the walls of the stairwell core to be firewalls and this is normally made with reinforced concrete. Reinforced concrete shear walls are also used, when considering the stiffness of the building, because they are suitable to transfer the shear forces induced by lateral loads.
The horizontal loads are transferred to the structural frame by the slabs and then through the structural frame, horizontal loads are transferred further down to the ground, (Eisele, J., Kloft, E.). Examples of different kinds of structural systems are shown in Table 1, (Wolfgang, S., 1977, Gustafsson, D., et al., 2005)

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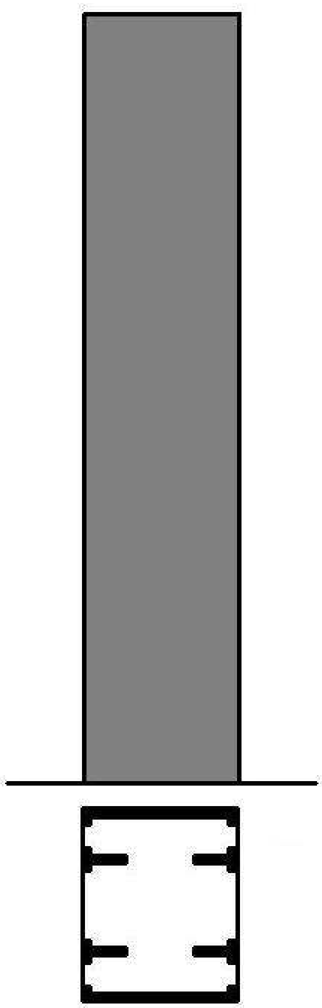
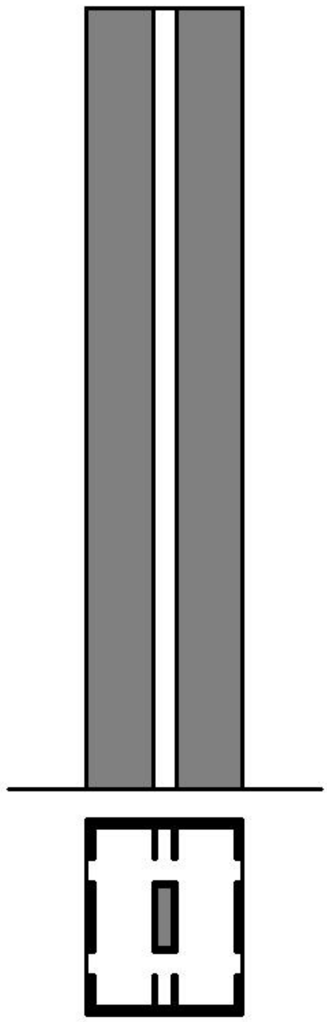
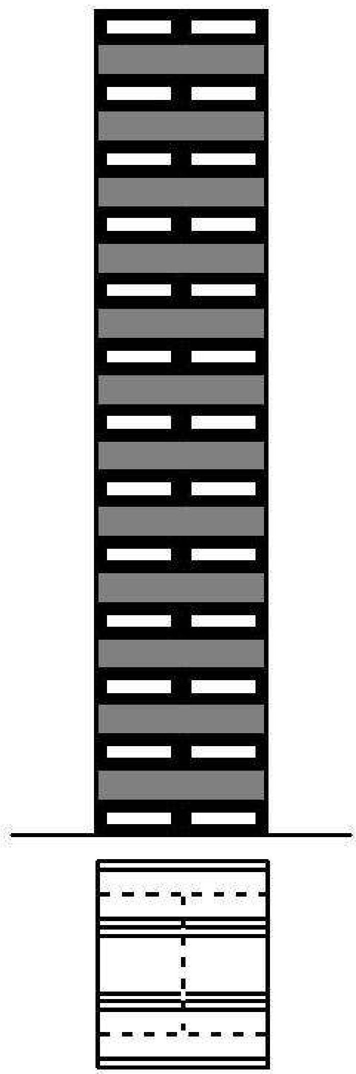
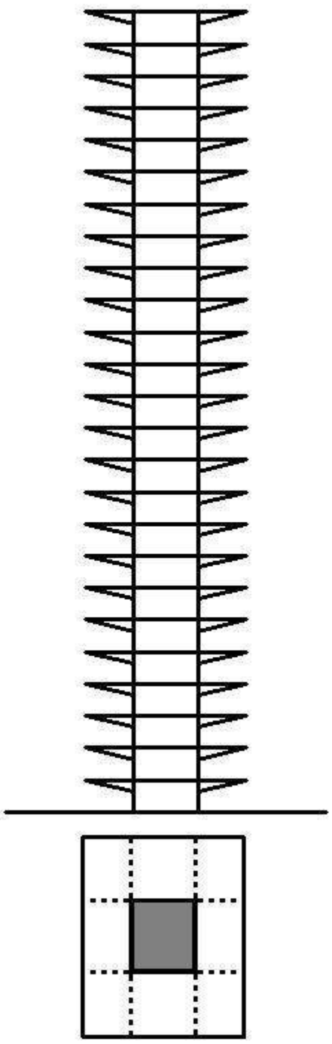
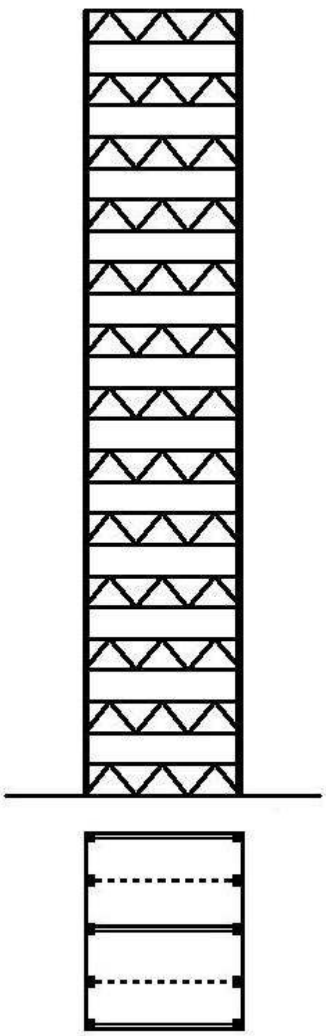
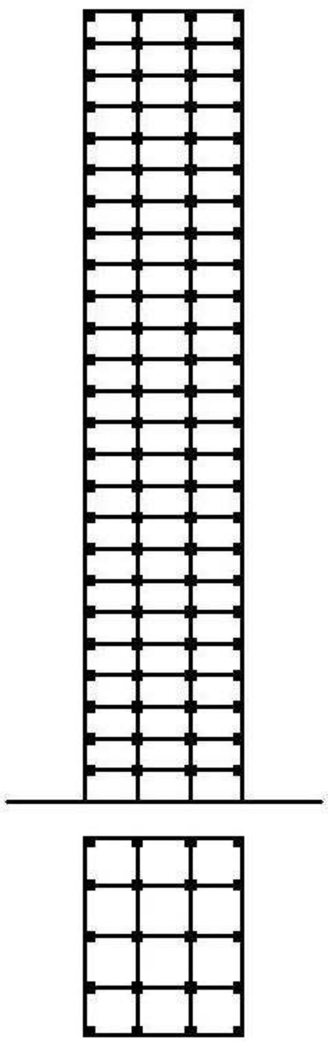
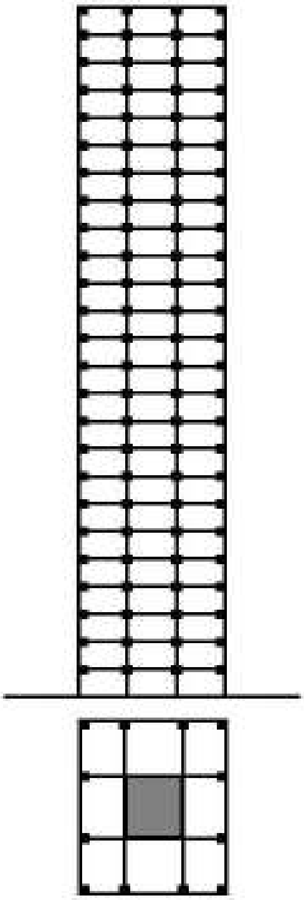
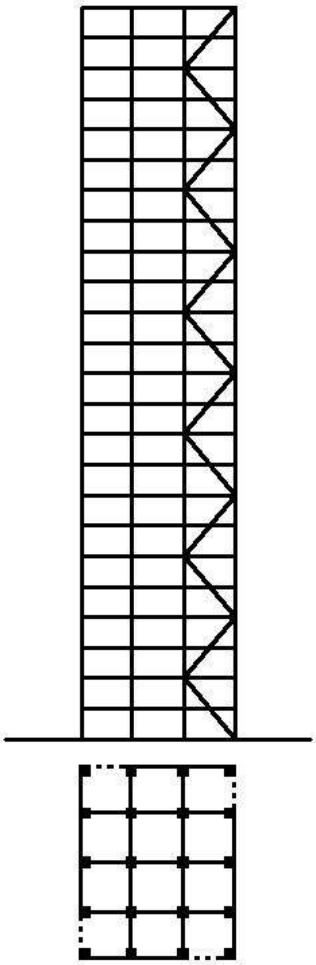
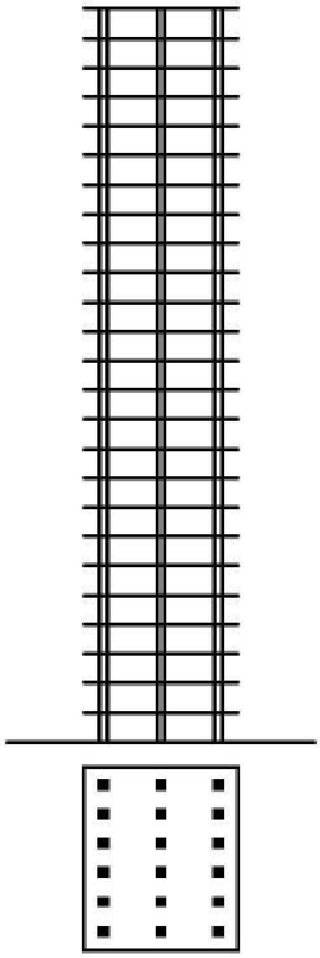
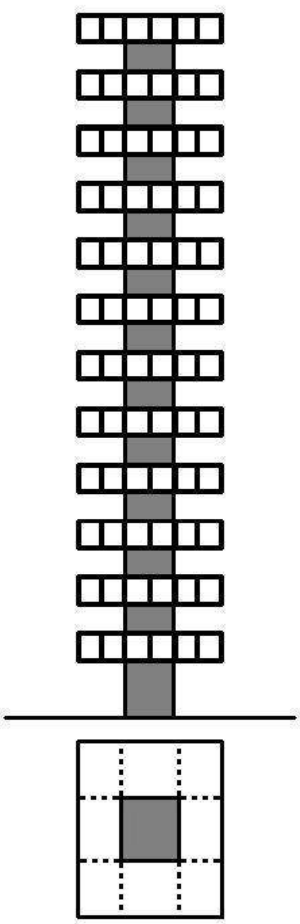
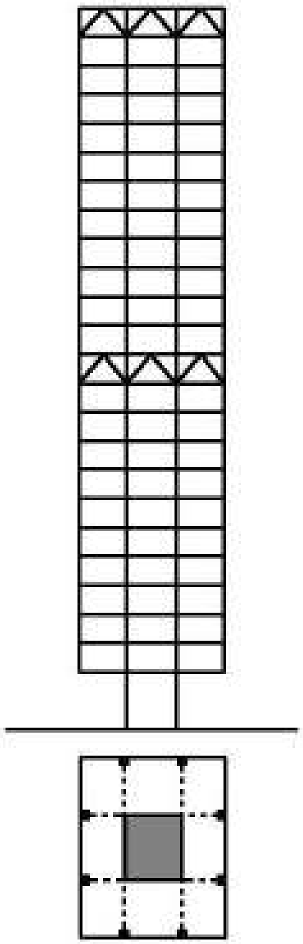
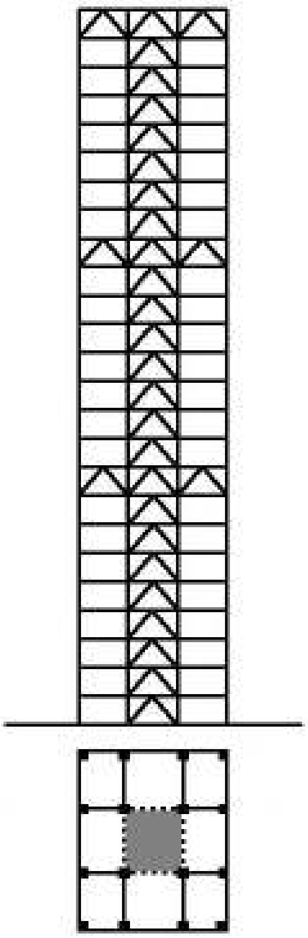
| **Parallel Bearing Walls** | **Cores and Facade Bearing Walls** | **Self-Supporting Boxes** |
|---|---|---|
| This system consists of vertical elements, prestressed by their weight. This means that they are efficient to absorb lateral forces. This system is used for apartment buildings where large spaces are not necessary and mechanical systems do not need core structures. | Around a core, vertical elements are formed as outer walls. The spanning capacity of the floor structure allows large interior spaces. Mechanical and vertical transport systems are placed in the core. The core also supplements the stiffness of the building. | This system is similar to the bearing wall system when it is assembled. The boxes are prefabricated three-dimensional units. They are put together like bricks and results in a criss-crossed wall beam system. |
|  |  |  |

| **Cantilever Slab** | **Staggered Truss** | **Rigid Frame** |
| --- | --- | --- |
| It is the slab that decides the size of the building. The system allows a column free area because of the supporting of the floor structure from a central core with the strength of the slab. There is a large amount of steel needed and the slab stiffness can increase by prestressing techniques. | Each floor is resting, partly on the top chord and partly on the bottom of the next one, in arranged story-high trusses. To transfer the wind loads through the structure down to the ground, the truss organize minimal wind bracing. It also carries the vertical loads. | This system is formed as vertical and horizontal planes. To manufacture this, rigid joints are used between linear elements. Both of the planes are organized the same way with columns and beams as a rectangular grid.<br>This system is mostly used for approximately 15-storey buildings. |
|  |  |  |

| **<u>Rigid frame and core</u>** | **<u>Trussed Frame</u>** | **<u>Flat Slab</u>** |
|---|---|---|
| By introducing a core in the rigid frame system the lateral resistance of the building will increase significantly. This is a good solution because the rigid frame reacts to the lateral loads by bending beams and columns. And this behaviour results in large lateral drift of building of a certain height. This system is mostly used for approximately 25-story buildings.  | The strength and stiffness will increase in a structure by combining a rigid frame with vertical shear trusses. The system is similar to rigid frame and core by using the frame for gravity loads and the vertical truss for wind loads. This system is mostly used for approximately 100-story buildings.  | This system is a horizontal storey-high system that contains uniformly thick concrete slabs that are supported by columns.  |

| **Interspatial** | **Suspended** | **Belt-Trussed Frame and Core** |
|---|---|---|
| On every other floor, cantilevered story-high framed structures are utilized. The construction is made in such a way to create useful space between and over the frame. Fixed operations are made in the space within the frame and any type of activity can be applied in the space above the frame.  | By using hangers instead of columns to carry the floor loads, the system offers efficient usage of material. A member that is exposed to compression has to have the strength decreased due to buckling while a member that is exposed for tension can use its full capacity.  | The facade columns and the core are bound together by the belt trusses. By binding the trusses and the core, the individual action of frame and core is eliminated. This system is mostly used for approximately 60-story buildings.  |

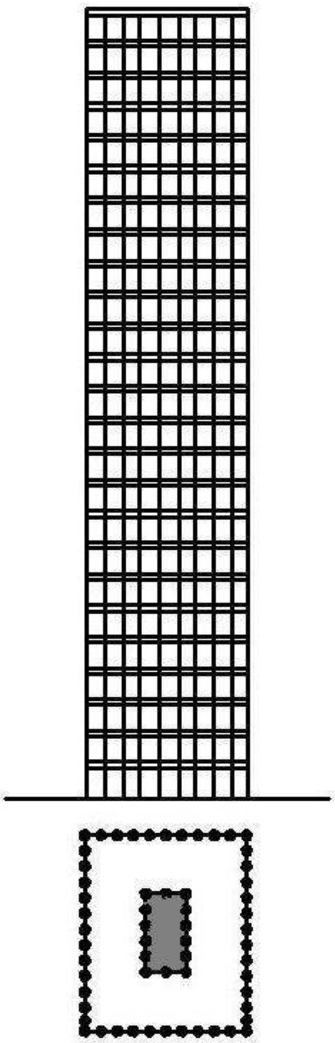
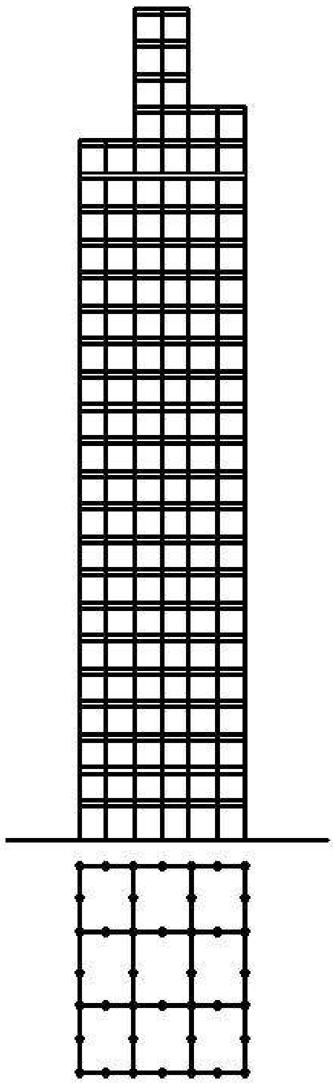
| **<u>Tube in Tube</u>** | **<u>Bundled Tube</u>** |
|---|---|
| The facade of the system looks like a wall with perforated window openings because the columns and beams are spaced so closely. It is from this that the system got its name the tube because the entire building looks like a hollow tube that cantilever from the ground. By sharing the loads with the exterior tube, the interior tube increases the stiffness  | A collection of tubes results in a multiple-cell tube and the increase in stiffness is obvious. The best height and the largest floor area are allowed. This system is mostly used for approximately 110-story buildings.  |

*Table 1. Structural systems*, (Wolfgang, S., 1977, Gustafsson, D., et al., 2005)

## 2.4 Comfort

Frequency, type of activity and the occupant's location are things that comfort has to consider and is measured in dynamic quantities, (Eisele, J., Kloft, E.).

Horizontal movements may arise in tall buildings. In an experiment by Takeshi Goto a motion simulator was built up to recreate these movements. From this experiment the following conclusions could be summed up as following:

1. There are thresholds for motion that can be measured for people.
2. Perception and tolerance seems to be related to the acceleration of the head.
3. People can sense motion before movement or sound of furniture.
4. The acceleration of the floor affects movements of object and human task performance.
5. Nausea may appear even if acceleration is not high because of the duration of motion.
6. Threshold accelerations are the following, based on investigation:
    - Perception threshold, less than 0,05 $m/s^2$,
    - Psychological and task performance, about 0,4 $m/s^2$,
    - Walking, 0,5 $m/s^2$ -0,7 $m/s^2$ and
    - Building motion for safety considerations should not exceed 0,8 $m/s^2$,

    (Goto, T., 1983).

"The Structural Division ASCE Subcommittee" made an early reference about human comfort that stated: for buildings to be comfortably habitable, the structure should be designed to prevent great bending, (Hansen, R.J., et al., 1973). To describe acceleration, peak values or rms (root-mean-square) is used. The peak value is an immediate event during a storm while rms is a median value during the worst ten minutes or one hour of the storm. It is the peak acceleration that the norms are referring to. To get an understanding of the values some normal experienced values are summarized in Table 2.

| milli-g | $m/s^2$ | |
|---|---|---|
| ~25~30 | 0,245~0,294 | start of interruption to walk normally in buildings |
| 80~100 | 0,78~0,98 | some loose object may fall |
| ~120 | ~1,18 | typical acceleration during a train ride |
| ~200 | ~1,96 | typical acceleration during a city car journey |
| ~260 | ~2,55 | typical acceleration during a boat ride. 10% experience nausea after an hour |
| 1000 | 9,8 | acceleration due to gravity |
| 4000-5000 | 39,2~49 | acceleration to experience in fairground rides |

*Table 2. Values of acceleration*
(Denoon, R., 2006*).*

Motion of a building may be compared with storms, which increases in velocity the major part of the day until they reach a peak and then they slowly loose momentum. Regarding human discomfort, it is the rms acceleration, which represents a twenty minutes average of the peak of the storm and an average over the top building floor that best characterizes the severity of the storm. It is also appropriate to calculate an average rms acceleration considering the top floor, hence observation has been made, that shows that buildings have motion in two directions and may twist.

There are many motion cues that people's response to wind-induced motion depends on. Some of them are motion, noise, visual observation and comments from co-workers. The motion tolerance varies from person to person because it is a subjective value and the permitted building sway may differ between owner and occupants. After asking the following question to several owners of buildings and an engineer, an answer of 2 % was found to be a reasonable level; "Assume you are building a new office building and you are worried over the human discomfort because of the motion of the building. You do not want to spend extra money on it, how many percent of the people, living on the top third part of the building; will you accept to protest against the motions?"

People from two buildings that had been exposed to a storm were interviewed and after that an rms acceleration of 0,049 $m/s^2$ with a return period of 6 years could be found to be a recommended level of acceleration, (Hansen, R.J., et al., 1973).

Information about human reception of sinusoidal excitations as a function of frequency was given by Chen and Robertson. They suggested using an occupancy sensitivity quotient, which defines the ratio of tolerable amplitudes of motions to the threshold motion for half the population.

Reed did the first evaluation of occupant's response to acceleration and this gave the first criteria, which was in terms of standard deviation of acceleration for a return period for the frequencies of the buildings.

Afterwards, Irwin continued Reed's study about response and came up with sinusoidal acceleration over a range of frequencies. He was the one who was primarily responsible for the criteria of standard deviation of acceleration in ISO 6897.

Melbourne continued the study using the work of Chen and Robertson, Reed and some full scale observations to develop criteria for tall buildings. His work showed that it was the peak acceleration that was the most important factor and the criterion was that the structure should be designed in a way such that the horizontal peak acceleration does not exceed 10 milli-g ~ 0,1 $m/s^2$ with a return period of one year for frequencies between 0,2 Hz-0,3 Hz, (Melbourne W.H., et al., 1992). The criteria for apartment buildings are stricter because they are more likely to be occupied during a storm.

For flexible buildings, the across-wind force is the one to contribute the most to the total rms acceleration. For stiffer buildings the torsion-induced acceleration is more important because it is insensitive to variations in stiffness, (Islam, M.S., et al., 1990).

It is possible, for analytical convenience, to identify the motion that is significant for comfort from different kinds of sources. Either one source is dominant or several of them together form a resulting motion felt by the occupant. The sources are identified as followed:

- motion in the first two bending modes about orthogonal axes (for the worst case scenario these can be divided into along-wind and across-wind motions)
- rotational motion about a vertical axis, often either a second or third mode but can be higher
- for buildings with stiffness and mass asymmetry, complex bending and rotational motion in one or both of the lower modes, and any one of these motions may not be normally distributed, (Melbourne W.H., et al., 1992)

## *2.5 Measures*

To prevent buildings from getting large motions there are different measures that could be used. One of them is to use structural design such as changing the modal mass. If the modal mass increases the frequency decreases, and therefore the acceleration decreases, it will also be harder for the wind to set the building in motion. Another measure to reduce accelerations is to modify aerodynamic conditions. The across-wind often gives the highest accelerations, also called vortex-shedding, and in slender buildings with a height-to-width ratio of 6 or more, that is the most dominant mechanism. By introducing tapers, corner set-backs or make the shape of the building vary with height it is possible to limit vortex shedding.

### 2.5.1 Damping

Another measure to decrease the movement in a tall building is damping. Damping is getting more and more common in tall buildings to ease wind-induced motion with the intention of improving the comfort of the occupants or at least reduce complaints. To decide whether dampers are needed or not, wind tunnel tests are applied. There are a lot of different kinds of dampers and to decide which one that suits which building depends on the response, free space and the cost. The most expensive one but the one that reacts intelligently on building motions is active damper system. In the upper storeys of the building, passive liquid dampers may be inserted. They are being incorporated by modifying existing water areas. The most effective dampers may be the viscous dampers because they will engage during the first cycle of motion, (Denoon), R., 2006). Also to mention is tuned mass dampers.

# 3. Codes

Three codes have been studied regarding calculation of the acceleration; the Canadian code, NBCC, the European code, Eurocode and the International standard, ISO. Most of the codes in the world give an approximate upper limit of 0,15 m/s$^2$ for the peak acceleration of a normal building. But it depends a little on the frequency of the building.

## *3.1 NBCC - National Building Code of Canada*

1. Buildings whose height is greater than 4 times their minimum effective width, w (eq. 3.1), or greater than 120 m and other buildings whose light weight, low frequency and low damping properties make them susceptible to vibration shall be designed
    - By experimental methods for the danger of dynamic overloading, vibration and the effects of fatigue, or
    - By using a dynamic approach to the action of wind gusts

$$w = \frac{\sum h_i \cdot w_i}{\sum h_i} \tag{3.1}$$

$h_i$= height above grade to level i
$w_i$= width normal to the wind direction at height $h_i$
*(NBCC, 2005)*

### 3.1.1 Calculation of the acceleration

A general expression for the maximum or peak loading effect, denoted by $W_p$, is as follows:

$$W_p = \mu + g_p \cdot \sigma \tag{3.2}$$

μ = mean loading effect
σ = root-mean square loading effect
$g_p$ = statistical peak factor for the loading effect

$$g_p = \sqrt{2 \cdot \log_e(v \cdot T)} + \frac{0{,}577}{\sqrt{2 \cdot \log_e(v \cdot T)}} \tag{3.3}$$

v = the natural frequency
T = the period, 600 s

If this expression 3.2 is rearranged, the following expression for the gust effect factor, $C_g = W_p/ \mu$, is obtained by 3.4.

$$C_g = 1 + g_p \cdot \left(\frac{\sigma}{\mu}\right) \tag{3.4}$$

The form of the fluctuating wind loading effect, σ, varies with the excitation, whether it is due to gusts, wake pressures or motion-induced forces.

Value of $\sigma/ \mu$, the coefficient of variation can be expressed by 3.5.

$$\frac{\sigma}{\mu} = \sqrt{\frac{K}{C_{eH}}\left(B + \frac{s \cdot F}{\beta}\right)} \tag{3.5}$$

K = a factor related to the surface roughness coefficient of the terrain
= 0,08 for exposure A
= 0,10 for exposure B
= 0,14 for exposure C
$C_{eH}$ = exposure factor at the top of the building has been assumed to be 1,74 for the case studies.
B = background turbulence factor is determined by 3.6

$$B = \frac{4}{3} \cdot \int^{\frac{914}{H}} \left[\frac{1}{1 + \frac{x \cdot H}{457}}\right] \cdot \left[\frac{1}{1 + \frac{x \cdot w}{122}}\right] \cdot \left[\frac{x}{\left(1 + x^2\right)^{\frac{4}{3}}}\right] dx \tag{3.6}$$

w = effective width of the windward face of the building
H = height of the windward face of the building, m
s = size reduction factor as a function of w/H and the reduces frequency $f_{nD}H/V_H$, 3.7

$$s = \frac{\pi}{3}\left[\frac{1}{1 + \frac{8 \cdot f_{nD} \cdot H}{3 \cdot V_H}}\right] \cdot \left[\frac{1}{1 + \frac{10 \cdot f_{nD} \cdot w}{V_H}}\right] \tag{3.7}$$

$f_{nD}$ = natural frequency of vibration in the along-wind direction in Hz
$V_H$ = mean wind speed, in m/s, at the top of structure, H, evaluated using equation 3.10 below
F = gust energy ratio at the natural frequency of the structure as a function of the wave number $f_{nD}/V_H$

$$F = \frac{x_0^2}{(1+x_0^2)^{\frac{4}{3}}} \qquad x_0 = (1220 \cdot f_n / V_H) \qquad (3.8)\ (3.9)$$

β = critical damping ratio in the along-wind direction

The mean wind speed at the top of the structure, $V_H$ in m/s

$$V_H = \bar{V} \cdot \sqrt{C_{eH}} \qquad (3.10)$$

where $\bar{V} = 39{,}2 \cdot \sqrt{q}$ the reference wind speed in m/s at the height 10 m, is determined from the reference velocity pressure q (kPa)

While the maximum lateral wind loading and deflection are generally in the direction parallel to the wind, the maximum acceleration of a building leading to possible human perception of motion or even discomfort may occur in the direction perpendicular to the wind. Across-wind accelerations are likely to exceed along-wind accelerations if the building is slender about both axes, that is if $\sqrt{w \cdot d} / H$ is less than one third, where w and d are the across-wind effective width and along-wind effective depth, respectively, and H is the height of the building. The along-wind effective depth, d, is calculated as above replacing $w_i$ by $d_i$.

The accelerations in a building are very dependent on the building's shape, orientation and buffeting from surrounding structures. However, data on the peak across-wind acceleration, $a_w$, at the top of the building from a variety of turbulent boundary-layer wind-tunnel studies exhibit much scatter around the following empirical formula 3.11.

$$a_w = f_{nW}^2 \cdot g_p \cdot \sqrt{w \cdot d} \cdot \left( \frac{a_r}{\rho_B \cdot g \cdot \sqrt{\beta_W}} \right) \qquad (3.11)$$

In less slender structures or for lower wind speeds, the maximum acceleration may be in the along-wind direction and may be estimated from expression 3.12.

$$a_D = 4 \cdot \pi^2 \cdot f_{nD}^2 \cdot g_p \cdot \sqrt{\frac{K \cdot s \cdot F}{C_{eH} \cdot \beta_D}} \cdot \frac{\Delta}{C_g} \qquad (3.12)$$

w,d = across-wind effective width and along-wind effective depth respectively, in m

$a_W$,$a_D$ = peak acceleration in across-wind and along-wind directions respectively, in m/s$^2$

$$a_r = 78{,}5 \cdot 10^{-3} \cdot \left[V_H / \left(f_{nW} \cdot \sqrt{w \cdot d}\right)\right]^{3,3} \text{ in N/m}^3 \qquad (3.13)$$

$\rho_B$ = average density of the building, in kg/m$^3$

$\beta_W$, $\beta_D$ = fraction of critical damping in across-wind and along-wind directions respectively

$f_{nW}$, $f_{nD}$ = fundamental natural frequencies in across-wind and along-wind direction respectively, in Hz

$\Delta$ = maximum wind-induced lateral deflection at the top of the building in along-wind direction, in m

g = acceleration due to gravity = 9,81 m/s$^2$, (User's guide-NBC, 2005)

## *3.2 Eurocode*

The wind actions calculated using EN1991-1-4 are characteristic values. They are determined from the basic values of wind velocity or the velocity pressure. In accordance with EN 1990 the basic values are characteristic values having annual probabilities of exceedence of 0,02, which is equivalent to a mean return period of 50 years.

### 3.2.1 Calculation of the acceleration

The turbulent length scale L(z) represents the average gust size for natural winds. For heights z below 200 m the turbulent length scale may be calculated using expression 3.14 or 3.15.

$$L(z) = L_t \left(\frac{z}{z_t}\right)^{\alpha} \qquad \text{for } z > z_{min} \qquad (3.14)$$

$$L(z) = L(z_{\min}) \text{ for } z < z_{\min} \qquad (3.15)$$

with a reference height of $z_t$ = 200m, a reference length scale of $L_t$ = 300 m, and with $\alpha$=0,67+0,05 ln($z_0$), where the roughness length $z_0$ is in m. The minimum height is $z_{min}$, which depends on the terrain category.

The wind distribution over frequencies is expressed by the non-dimensional power spectral density function $S_L(z,n)$, which should be determined using expression 3.16.

$$S_L(z,n) = \frac{n \cdot S_v(z,n)}{\sigma_v^2} = \frac{6{,}8 \cdot f_L(z,n)}{(1 + 10{,}2 \cdot f_L(z,n))^{5/3}} \qquad (3.16)$$

where $S_v(z,n)$ is the one-sided variance spectrum, and

$f_L(z,n) = \dfrac{n \cdot L(z)}{v_m(z)}$ is a non-dimensional frequency determined by the frequency $n=n_{1,x}$, the natural frequency of the structure in Hz, by the mean velocity $v_m(z)$ and the turbulence length scale L(z).

The background factor $B^2$ allowing for the lack of full correlation of the pressure on the structure surface may be calculated using expression 3.17.

$$B^2 = \frac{1}{1 + 0{,}9 \cdot \left( \dfrac{b+h}{L(z_s)} \right)^{0{,}63}} \qquad (3.17)$$

where: b,h is the width and height of the structure
$L(z_s)$ is the turbulent length scale at reference height $z_s$ = 0,6·h. It is on the safe side to use $B^2$=1

The peak factor $k_p$, defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation, should be obtained by 3.18.

$$k_p = \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0,6}{\sqrt{2 \cdot \ln(v \cdot T)}} \qquad (3.18)$$

or

$k_p = 3$ whichever is larger

where:
v is the up-crossing frequency given in 3.19.
T is the averaging time for the mean wind velocity, T = 600 s

The up-crossing frequency v should be obtained by 3.19.

$$v = n_{1,x} \sqrt{\frac{R^2}{B^2 + R^2}} ; v \geq 0,08Hz \qquad (3.19)$$

where $n_{1,x}$ is the natural frequency of the structure. The limit of $v \geq 0,08Hz$ corresponds to a peak factor of 3,0.

The resonance response factor $R^2$ allowing for turbulence in resonance with the considered vibration mode of the structure should be determined using expression 3.20.

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n_{1,x}) \cdot R_h(\eta_h) \cdot R_b(\eta_b) \qquad (3.20)$$

where:
δ is the total logarithmic decrement of damping
$S_L$ is the non-dimensional power spectral density function given in 3.16
$R_h$,$R_b$ is the aerodynamic admittance functions.

The aerodynamic admittance functions $R_h$ and $R_b$ for a fundamental mode shape may be approximated using expression 3.17 and 3.18.

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot \eta_h^2}(1 - e^{-2 \cdot \eta_h}) \text{; } R_h = 1 \text{ for } \eta_h = 0 \qquad (3.21)$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2}(1 - e^{-2 \cdot \eta b}) \text{; } R_b = 1 \text{ for } \eta_b = 0 \qquad (3.22)$$

with: $\eta_h = \frac{4,6 \cdot h}{L(z_s)} \cdot f_L(z_s, n_{1,x}) and \eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n_{1,x})$ (3.23) and (3.24)

Figure 5 shows the number of times $N_g$, that the value $\Delta S$ of an effect of wind is reached or exceeded during a period of 50 years. $\Delta S$ is expressed as a percentage of the value $S_k$, where $S_k$ is the effect due to a 50 years return period wind action.

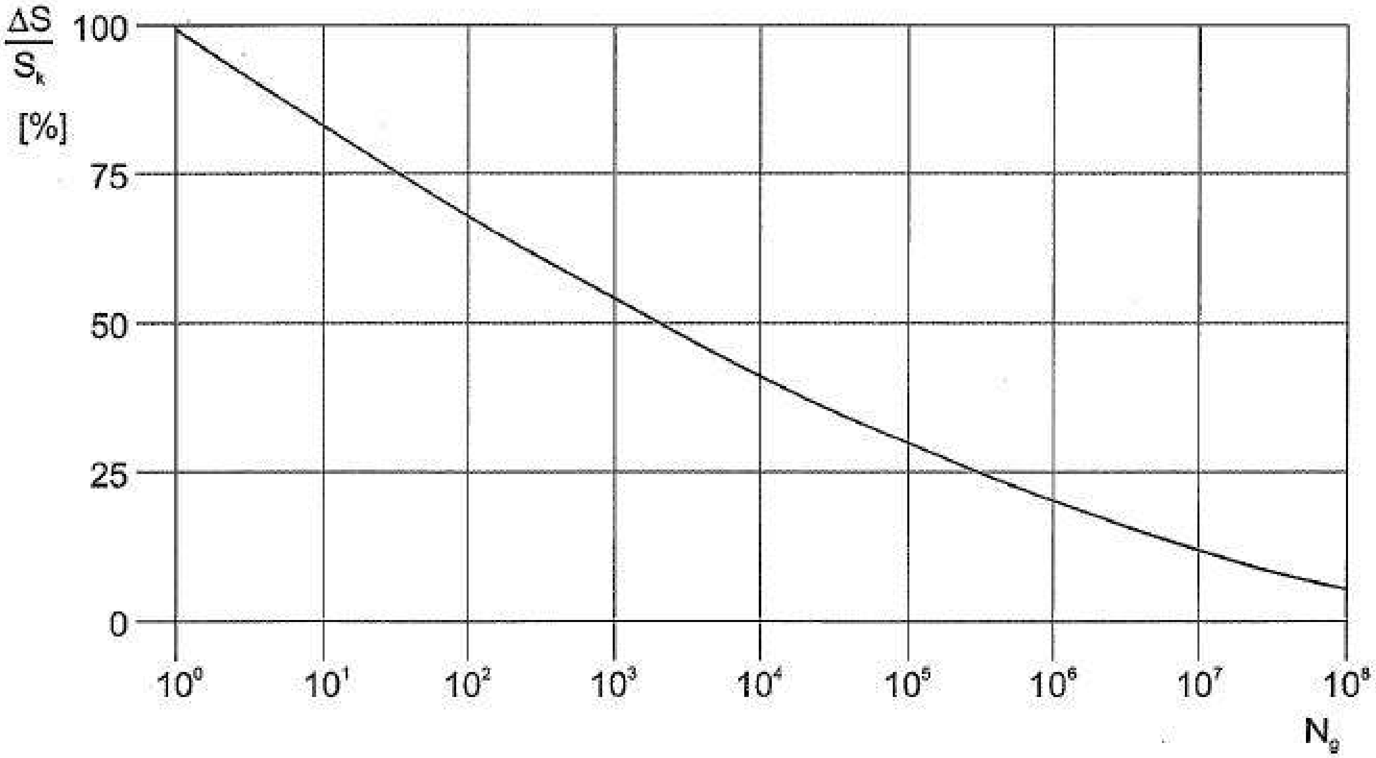



*Figure 5. Number of gust loads $N_g$ for an effect $\Delta S/ S_k$, during a 50 years period*

The relationship between $\Delta S/S_k$ and $N_g$ is given by expression 3.25.

$$\frac{\Delta S}{S_k} = 0,7 \cdot (\log(N_g))^2 - 17,4 \cdot \log(N_g) + 100 \qquad (3.25)$$

The maximum along-wind displacement is determined from the equivalent static wind force

The standard deviation $\sigma_{a,x}$ of the characteristic along-wind acceleration of the structural point at height z should be obtained using expression 3.26.

$$\sigma_{a,x}(z) = \frac{c_f \cdot \rho \cdot b \cdot I_v(z_s) \cdot v_m^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \cdot \Phi_{1,x}(z) \qquad (3.26)$$

where:

$c_f$ is the force coefficient, which is represented in section 7 in EN 1991-1-4:2005

$\rho$ is the air density

b is the width of the structure

$I_v(z_s)$ is the turbulence intensity at the height $z=z_s$ above ground

$$I_v(z) = \frac{K_1}{C_0(z) \cdot \ln(z/z_0)} \qquad (3.27)$$

$K_1$ is the turbulence factor

$C_0(z)$ is the orography factor, suggested to be 1,0

$v_m(zs)$ is the mean wind velocity for $z=z_s$,

$$v_m(z) = C_r(z) \cdot C_0(z) \cdot v_b \qquad (3.28)$$

$C_r(z)$ is the roughness factor

$v_b$ is the basic wind velocity

$$C_r(z) = k_r \cdot \ln(\frac{z}{z_0}) \qquad (3.29)$$

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,11}}\right)^{0.07} \qquad (3.30)$$

$z_s$ is the reference height, 0,6·h, according to the code

R is the square root of resonant response, see 3.20

$K_x$ is a non-dimensionall coefficient, see 3.31

$m_{1,x}$ is the along wind fundamental equivalent mass

$n_{1,x}$ is the fundamental frequency of along wind vibration of the structure
$\Phi_{1,x}$ is the fundamental along wind modal shape

The non dimensional coefficient, $K_x$, is defined by 3.31.

$$K_x = \frac{\int_0^h v_m^2(z) \cdot \Phi_{1,x}(z)dz}{v_m^2(z_s) \cdot \int_0^h \Phi_{1,x}^2(z)dz} \qquad (3.31)$$

where:
h is the height of the structure
$\Phi(z) = z/h$ assuming a linear modal shape

The characteristic peak accelerations are obtained by multiplying the standard deviation by the peak factor in using the natural frequency as up crossing frequency, i.e $v=n_{1,x}$,
(EN 1991-1-4, 2005)

## *3.3 International Standard, ISO 6897*

### 3.3.1 Limits for acceleration

The criterion for the acceleration is as follows:

$$\sigma_a < \sigma_{ISO}(f_e) \qquad (3.32)$$

where:
$\sigma_a$ = the standard deviation for acceleration based on a 5 years or more return period.
$\sigma_{ISO}$ = threshold value, depends on the frequency according to Figures 6 and 7
$f_e$= frequency of the structure

The relation between peak acceleration and $\sigma_a$ is given by 3.33 and 3.34.

$$a = g \cdot \sigma_a \tag{3.33}$$

$$g = \sqrt{2 \cdot \ln(f_e \cdot T)} + \frac{1}{\sqrt{3}} \cdot \frac{1}{\sqrt{2 \cdot \ln(f_e \cdot T)}} \tag{3.34}$$

T is the period in seconds

| **Frequency (centre frequency of one-third octave band) Hz** | **Acceleration m/s$^2$ $\sigma_{ISO}$ rms** | | | |
|---|---|---|---|---|
| | Curve 1 Figure 1 | Curve 2 Figure 1 | Curve 1 Figure 2 | Curve 2 Figure 2 |
| 0,063 | 0,081 5 | 0,489 0 | 0,012 6 | 0,050 4 |
| 0,08 | 0,073 5 | 0,441 0 | 0,011 4 | 0,045 0 |
| 0,100 | 0,067 0 | 0,400 0 | 0,010 3 | 0,040 9 |
| 0,125 | 0,061 0 | 0,366 0 | 0,009 2 | 0,037 0 |
| 0,160 | 0,055 0 | 0,330 0 | 0,008 3 | 0,033 0 |
| 0,200 | 0,050 0 | 0,300 0 | 0,007 5 | 0,030 0 |
| 0,250 | 0,046 0 | 0,276 0 | 0,006 9 | 0,027 0 |
| 0,315 | 0,041 8 | 0,250 0 | 0,006 1 | 0,024 0 |
| 0,400 | 0,037 9 | 0,228 0 | 0,005 5 | 0,021 9 |
| 0,500 | 0,034 5 | 0,207 0 | 0,004 9 | 0,019 8 |
| 0,630 | 0,031 5 | 0,189 0 | 0,004 45 | 0,017 8 |
| 0,800 | 0,028 5 | 0,167 0 | 0,003 98 | 0,015 9 |
| 1,000 | 0,026 0 | 0,156 0 | 0,003 60 | 0,014 4 |

*Table 3 Acceleration/frequency values at the one-third points for the curves in figure 6 and 7*

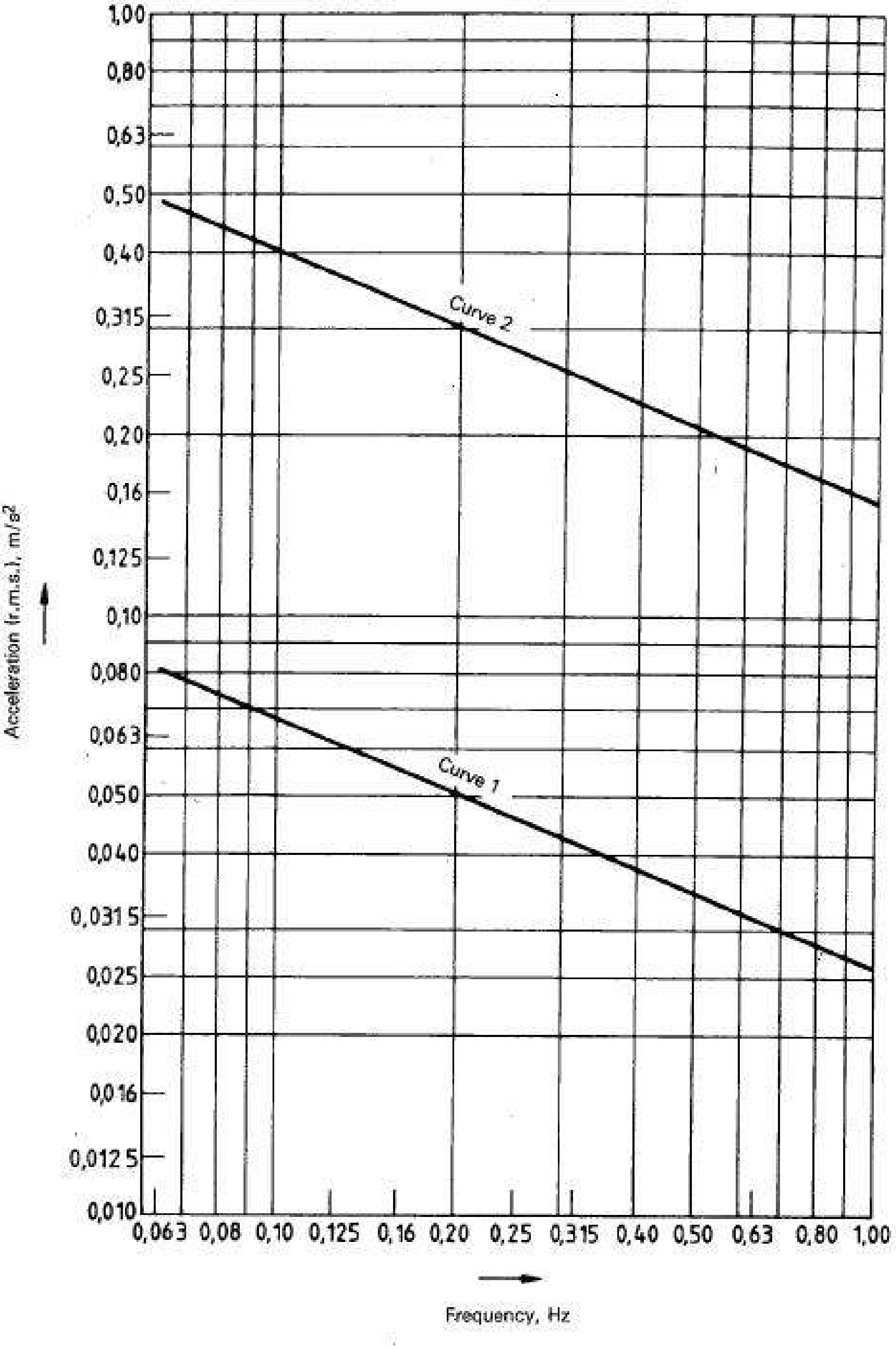



*Figure 6. Suggested satisfactory magnitudes of horizontal motion of buildings used for general purposes (curve 1) and of off-shore fixed structures (curve 2)*

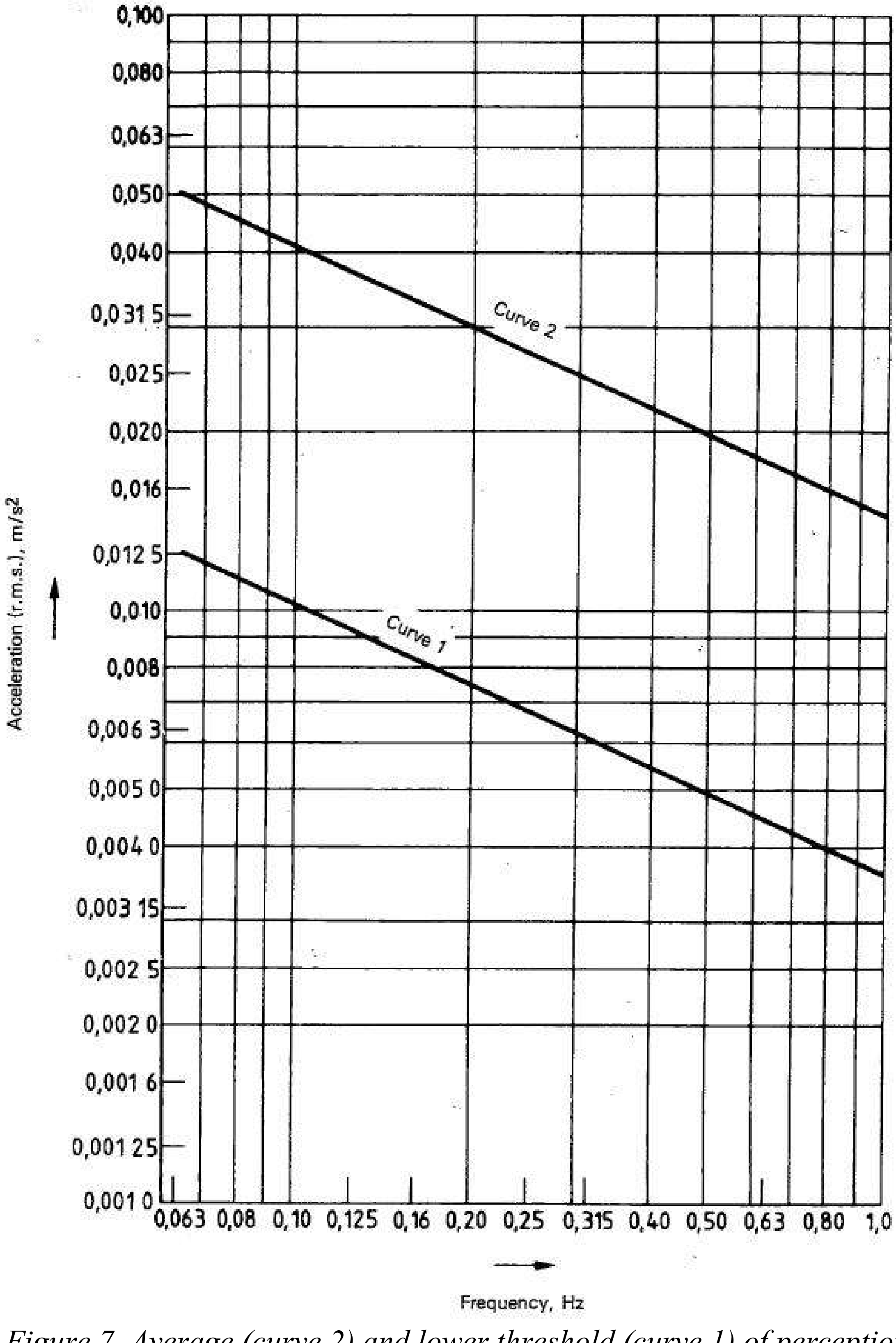



*Figure 7. Average (curve 2) and lower threshold (curve 1) of perception of horizontal motion by humans*
(ISO, 1984).

# 4. Case studies

Different kinds of structures are studied to see which one of them that resists wind loads the best. Displacement, torsion, frequencies and accelerations are presented in this thesis. Structure 1 is a tube in tube system, structure 2 is a trussed frame with a core and structure 3 is like structure 2 but with two cores on each side of the building. The thicknesses 0,2 m, 0,4 m and 0,6 m of the walls that is presented in Tables 6, 7 and 8 can be equivalent to moment of inertia, I, of about I, 2I and 3I respectively. The height of the structure is 72 m, the width is 18 m and each storey is 6 m high. The core is 6x6 m and the dimensions for the columns, beams and slabs are given in 1.3 Limitations. Table 4 shows the masses in ton/m for the structures' each case. The structures' EI in $GNm^2$ has been calculated for each case by treating the structure as a cantilever, see table 5. The equation for displacement is as follows:

$$y = \frac{q \cdot L^3 \cdot x}{24 \cdot EI} \cdot \left( 4 - \frac{x^3}{L^3} \right)$$

x is equal to L and y is taken from Table 6,7 and 8, rearranging the equation gives:

$$EI = \frac{q \cdot L^4}{8 \cdot y}$$

To evaluate the bending stiffness EI easch system was analysed by the computer program 3D Structures with a lateral load 15 kN/m to get the corresponding static deflection y.

| **Thicknesses** | **Structure 1** | **Structure 2** | **Structure 3** |
|---|---|---|---|
| **s= 0,2 ; w = 0,2** | 41,38 | 41,25 | 47,64 |
| **s= 0,2 ; w = 0,4** | 52,90 | 52,77 | 70,68 |
| **s= 0,2 ; w = 0,6** | 64,42 | 64,29 | 93,72 |
| **s= 0,4 ; w = 0,2** | 64,42 | 64,29 | 67,80 |
| **s= 0,4 ; w = 0,4** | 75,94 | 75,81 | 90,84 |
| **s= 0,4 ; w = 0,6** | 87,46 | 87,33 | 113,88 |
| **s= 0,6 ; w = 0,2** | 87,46 | 87,33 | 87,96 |
| **s= 0,6 ; w = 0,4** | 98,98 | 98,85 | 111,00 |
| **s= 0,6 ; w = 0,6** | 110,50 | 110,37 | 134,04 |

*Table 4. Shows the structures' masses in ton/m*

| **Thicknesses** | **Structure 1** | **Structure 2** | **Structure 3** |
|---|---|---|---|
| **s= 0,2 ; w = 0,2** | 1938 | 3599 | 2400 |
| **s= 0,2 ; w = 0,4** | 2964 | 4581 | 4199 |

| | | | |
|---|---|---|---|
| **s= 0,2 ; w = 0,6** | 3876 | 5599 | 6299 |
| **s= 0,4 ; w = 0,2** | 2964 | 3599 | 2964 |
| **s= 0,4 ; w = 0,4** | 3876 | 4581 | 5039 |
| **s= 0,4 ; w = 0,6** | 5039 | 5599 | 7198 |
| **s= 0,6 ; w = 0,2** | 3599 | 3876 | 3599 |
| **s= 0,6 ; w = 0,4** | 5039 | 4581 | 5599 |
| **s= 0,6 ; w = 0,6** | 5599 | 5599 | 7198 |

*Table 5. Shows EI of the structures' in $GNm^2$*

## 4.1 Structure 1

### 4.1.1 Description

The tube concept has been an efficient framing system for many slender tall buildings. The perimeter of the building is working as a vertical stiff tube. It consists of vertical bearing such as beams or bracing members and stands against the horizontal forces such as wind or earthquake. It will also give lateral support to all vertical supports against buckling. If shortening and lengthening of the columns in the tube take place, chord drift will appear. Also, web drift appears, created by shear and bending deformations of the individual tube members. The deflections of the tube consist of these chord and web drifts. The sides of the tube has to be stiff to reduce shear lag, this to get all the columns on the windward and leeward side to play a part in resisting overturning moments. They are doing this by direct forces in the columns. Two different ways of making the side of the building stiff are:

- By putting diagonal bracing across each side.
- By forming a rigid frame with spandrel beams and the columns, (Monograph, 1978).

By using the interior tube for gravity loads but also for lateral loads the stiffness of the exterior tube will increase, which will make the system look like a tube in tube. The tubes together respond to lateral forces as a unit and are tied together by the floor structure. The wind on the upper part of the building is resisted by the exterior tube and the wind on the lower part of the building is resisted by the interior tube, (Schueller, W., 1977). The core, the interior tube, is made of bearing concrete walls, (Stafford Smith, B., et al., 1991).

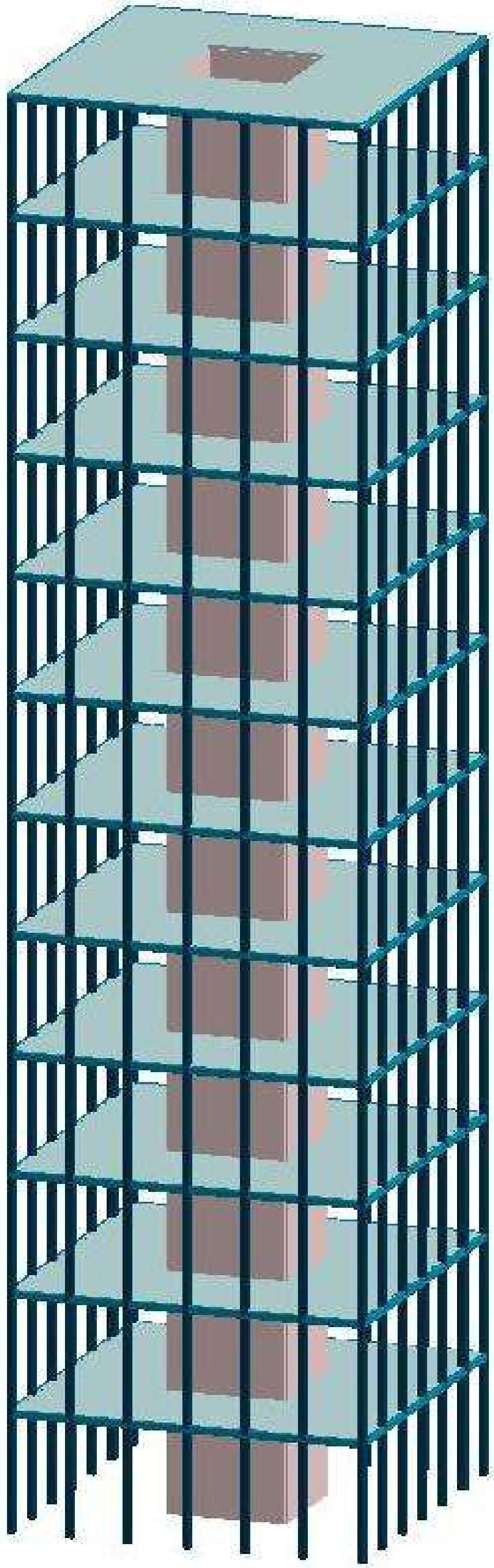


*Figure 8. Model of the tube in tube system*

### 4.1.2 3D Structures

The structure was built up in the program 3D structures, see Figure 7. Calculations of frequencies, in the first, second and third mode, and displacement, for wind load in one direction and in two directions, were made. The result is presented in Table 6.

| **Tube in tube** | | **Frequency/Displacement (Hz)/(m)** | **Walls, t, (m)** | | |
|---|---|---|---|---|---|
| | | | **0,2** | **0,4** | **0,6** |
| **Slabs, t, (m)** | **0,2** | 1 | 0,73 | 0,8 | 0,85 |
| | | 2 | 0,73 | 0,8 | 0,85 |
| | | 3 | 1,92 | 2,58 | 3,06 |
| | | Displ./comb. wind and weight | 0,026/0,046 | 0,017/0,031 | 0,013/0,024 |
| | | 2-direct. Displ. | 0,065 | 0,044 | 0,034 |
| | **0,4** | 1 | 0,70 | 0,77 | 0,81 |
| | | 2 | 0,70 | 0,77 | 0,81 |
| | | 3 | 1,52 | 2,06 | 2,46 |
| | | Displ./comb. wind and weight | 0,017/0,031 | 0,013/0,023 | 0,010/0,019 |
| | | 2-direct. Displ. | 0,044 | 0,032 | 0,026 |
| | **0,6** | 1 | 0,66 | 0,73 | 0,78 |
| | | 2 | 0,66 | 0,73 | 0,78 |
| | | 3 | 1,31 | 1,77 | 2,12 |
| | | Displ./comb. wind and weight | 0,014/0,027 | 0,010/0,020 | 0,009/0,017 |
| | | 2-direct. Displ. | 0,037 | 0,028 | 0,023 |

*Table 6. Presents the frequency and the displacements for one direction wind load and for 2 directions wind load for the tube in tube structure*

### 4.1.3 Conclusion

If the thickness of the walls increases the building gets more stiff, which makes the displacement smaller. This results in increasing the frequency. If increasing the thickness of the slabs, in other words increasing the dead weight of the building, the displacement will be smaller as well but the frequency will decrease. The frequency decreases because the building gets heavier and therefore harder to move. Mode one and two is directions perpendicular towards the building and the third mode is torsion. Mode one and two are the same because the structure is symmetric. This structure is the least stiff of the structures because the difference between the frequencies, regarding thickness of the walls, is the largest.

## 4.2 Structure 2

### 4.2.1 Description

Trussed frames includes of beams and columns, which primary task is to resist gravity loads. The lateral stability of the frames is created by diagonals that are put together to resist horizontal loads. As one with the beams the diagonals are creating the web of the vertical truss and the columns acts as the chords. Trusses are very efficient to resist lateral loads. The diagonals are exposed of tension in either of the lateral load direction and that is why mostly steel is used for this structure. It is an economical system because a structure can be constructed very stiff for all heights with the least possible material. The placement of doors and windows and the interior of a building are hard to plan for this type of structure, (Stafford Smith, B., et al., 1991).

The trussed frame may be connected with an interior core where many times the elevator and other connections are placed. The core is made of bearing walls of concrete and helps the system to resist torsions of the building, (Monograph, 1978).

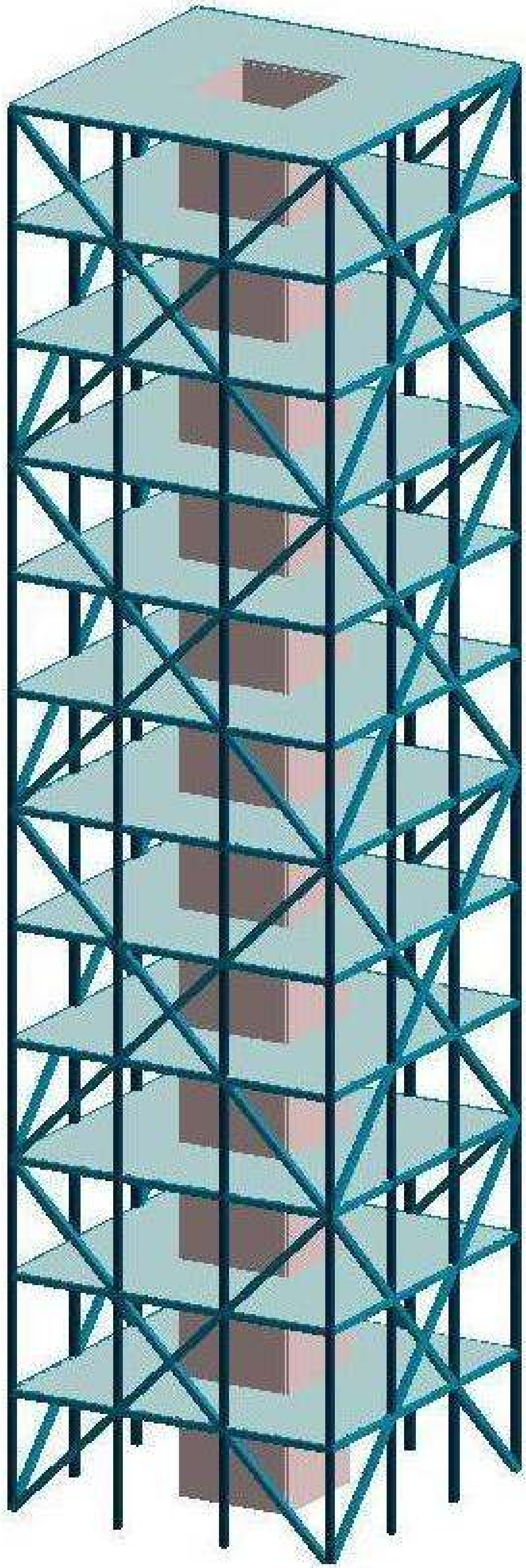


*Figure 9. Model of the trussed frame system.*

### 4.2.2 3D Structures

For the model that was built up in 3D structures, see Figure 8, and for the results of the calculations see Table 7.

| **Trussed frame w. 1 core** | | **Frequency/Displacement (Hz)/(m)** | **Walls, t, (m)** | | |
|---|---|---|---|---|---|
| | | | **0,2** | **0,4** | **0,6** |
| **Slabs, t, (m)** | **0,2** | 1 | 0,99 | 1,00 | 1,01 |
| | | 2 | 0,99 | 1,00 | 1,01 |
| | | 3 | 3,03 | 3,44 | 3,78 |
| | | Displ./comb. wind and weight | 0,014/0,025 | 0,011/0,021 | 0,009/0,018 |
| | | 2-direct. Displ. | 0,035 | 0,029 | 0,025 |
| | **0,4** | 1 | 0,80 | 0,83 | 0,86 |
| | | 2 | 0,80 | 0,83 | 0,86 |
| | | 3 | 2,38 | 2,74 | 3,03 |
| | | Displ./comb. wind and weight | 0,014/0,026 | 0,011/0,021 | 0,009/0,019 |
| | | 2-direct. Displ. | 0,035 | 0,029 | 0,025 |
| | **0,6** | 1 | 0,69 | 0,74 | 0,77 |
| | | 2 | 0,69 | 0,74 | 0,77 |
| | | 3 | 2,03 | 2,34 | 2,61 |
| | | Displ./comb. wind and weight | 0,013/0,027 | 0,011/0,022 | 0,009/0,019 |
| | | 2-direct. Displ. | 0,036 | 0,029 | 0,025 |

*Table 7. Presents the frequency and the displacements for one direction wind load and for 2 directions wind load for the trussed frame with one core structure*

### 4.2.3 Conclusion

Also here the frequency is increasing when the thickness of the wall is increasing and the building gets stiffer. The frequency is decreasing when the thickness of the slabs is increasing and so the building gets heavier and harder to move. Mode one and two are the same because the house is symmetric. This structure is the stiffest of the three structures. This can be seen by looking at the frequencies when increasing the thickness of the walls. The difference between the frequencies is small, which indicates that it is already stiff.

## *4.3 Structure 3*

### 4.3.1 Description

This structure is the same as structure 2 but the interior core is replaced with two cores that are placed at each side of the building. And the trusses have been removed on these two sides.

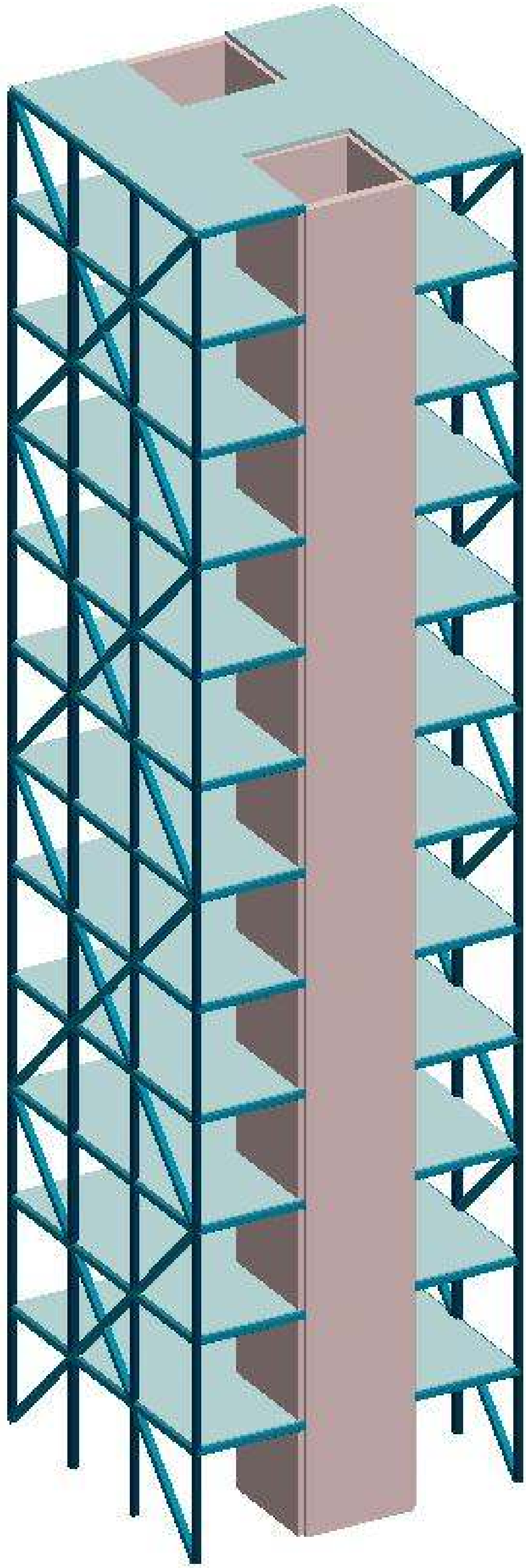


*Figure 10. Model of the trussed frame system with two cores.*

### 4.3.2 3D Structures

Also this structure was built up with 3D structures, see Figure 9, and the results is presented in Table 8.

| Trussed frame w. 2 cores | | Frequency/Displacement (Hz)/(m) | Walls, t, (m) | | |
|---|---|---|---|---|---|
| | | | **0,2** | **0,4** | **0,6** |
| **Slabs, t, (m)** | **0,2** | 1 | 0,77 | 0,86 | 0,90 |
| | | 2 | 1,00 | 1,01 | 1,02 |
| | | 3 | 2,75 | 3,19 | 3,42 |
| | | Displ./comb. wind and weight | 0,021/0,039 | 0,012/0,023 | 0,008/0,016 |
| | | 2-direct. Displ. | 0,044 | 0,027 | 0,020 |
| | **0,4** | 1 | 0,71 | 0,81 | 0,86 |
| | | 2 | 0,98 | 1,07 | 1,09 |
| | | 3 | 2,36 | 2,85 | 3,12 |
| | | Displ./comb. wind and weight | 0,017/0,032 | 0,01/0,020 | 0,007/0,015 |
| | | 2-direct. Displ. | 0,035 | 0,022 | 0,017 |
| | **0,6** | 1 | 0,67 | 0,77 | 0,83 |
| | | 2 | 0,96 | 1,11 | 1,18 |
| | | 3 | 2,13 | 2,63 | 2,92 |
| | | Displ./comb. wind and weight | 0,014/0,029 | 0,009/0,019 | 0,007/0,015 |
| | | 2-direct. Displ. | 0,030 | 0,020 | 0,015 |

*Table 8. Presents the frequency and the displacements for one direction wind load and for 2 directions wind load for the trussed frame with two cores structure*

### 4.3.3 Conclusion

This structure's behaviour is the same, regarding the frequency, as structure 1 and structure 2. The frequency is different in the modes because the structure is not symmetric. This structure is stiff in one of the directions but not as stiff in the other direction.

# 5. Calculations of the accelerations

To compare the norms calculations of acceleration, the frequencies from the case study is used. The results from ISO are permitted values of acceleration in the structure and the results from NBCC and Eurocode are actual values of acceleration in the structure. NBCC and Eurocode refer to ISO for comparison of the values. Damping has been chosen to 0,02 for NBCC and 0,08 for Eurocode after the codes' suggestions. It is the along-wind acceleration that has been calculated. The results of the calculations can be seen in table 9. The same calculations have been made for all the structures.

## *5.1.1 NBCC*

K is assumed to be 0,10

$$a = 4 \cdot \pi^2 \cdot f_{nD}^2 \cdot g_p \cdot \sqrt{\frac{K \cdot s \cdot F}{C_{eH} \cdot \beta_D}} \cdot \frac{\Delta}{C_g}$$

For further calculations see appendix 1.

## *5.1.2 Eurocode*

$$a = \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0{,}6}{\sqrt{2 \cdot \ln(v \cdot T)}} \cdot \frac{c_f \cdot \rho \cdot b \cdot l_v(z_s) \cdot v_m^2(z_s)}{m_{1,x}} \cdot R \cdot K_x \cdot \Phi_{1,x}(z)$$

For further calculations see appendix 2.

## *5.1.3 ISO*

$$a = g \cdot \sigma_a$$

$$g = \sqrt{2 \cdot \ln(f_e \cdot T)} + \frac{1}{\sqrt{3}} \cdot \frac{1}{\sqrt{2 \cdot \ln(f_e \cdot T)}}$$

T = 600 s

$\sigma_a$ is given by Figure 6, curve 1.

## 5.4 Compilation

The values of the peak accelerations in m/s$^2$, for NBCC and Eurocode, are tabulated in Table 9 and the values of the limited accelerations in m/s$^2$, for ISO, are tabulated in Table 10.

| **Thicknesses** | **Code** | **Structure 1** | **Structure 2** | **Structure 3** |
|---|---|---|---|---|
| **s=0,2; w=0,2** | NBCC | 0,110 | 0,080 | 0,095 |
| **s=0,2; w=0,4** | NBCC | 0,080 | 0,063 | 0,060 |
| **s=0,2; w=0,6** | NBCC | 0,060 | 0,052 | 0,042 |
| **s=0,4; w=0,2** | NBCC | 0,070 | 0,065 | 0,071 |
| **s=0,4; w=0,4** | NBCC | 0,059 | 0,053 | 0,047 |
| **s=0,4; w=0,6** | NBCC | 0,047 | 0,045 | 0,035 |
| **s=0,6; w=0,2** | NBCC | 0,054 | 0,053 | 0,055 |
| **s=0,6; w=0,4** | NBCC | 0,043 | 0,048 | 0,041 |
| **s=0,6; w=0,6** | NBCC | 0,041 | 0,041 | 0,034 |
| **s=0,2; w=0,2** | Eurocode | 0,13 | 0,095 | 0,110 |
| **s=0,2; w=0,4** | Eurocode | 0,11 | 0,083 | 0,074 |
| **s=0,2; w=0,6** | Eurocode | 0,08 | 0,068 | 0,053 |
| **s=0,4; w=0,2** | Eurocode | 0,10 | 0,088 | 0,096 |
| **s=0,4; w=0,4** | Eurocode | 0,08 | 0,072 | 0,062 |
| **s=0,4; w=0,6** | Eurocode | 0,06 | 0,060 | 0,046 |
| **s=0,6; w=0,2** | Eurocode | 0,08 | 0,077 | 0,079 |
| **s=0,6; w=0,4** | Eurocode | 0,06 | 0,063 | 0,053 |
| **s=0,6; w=0,6** | Eurocode | 0,05 | 0,054 | 0,041 |

*Table 9. Shows the accelerations calculated with the codes' equations for all the cases*

| | | | | |
|---|---|---|---|---|
| **s=0,2; w=0,2** | ISO | 0,11 | 0,097 | 0,099 |
| **s=0,2; w=0,4** | ISO | 0,10 | 0,097 | 0,10 |
| **s=0,2; w=0,6** | ISO | 0,10 | 0,097 | 0,10 |
| **s=0,4; w=0,2** | ISO | 0,11 | 0,10 | 0,11 |
| **s=0,4; w=0,4** | ISO | 0,11 | 0,10 | 0,10 |
| **s=0,4; w=0,6** | ISO | 0,10 | 0,11 | 0,11 |
| **s=0,6; w=0,2** | ISO | 0,11 | 0,11 | 0,11 |
| **s=0,6; w=0,4** | ISO | 0,11 | 0,11 | 0,10 |
| **s=0,6; w=0,6** | ISO | 0,10 | 0,10 | 0,10 |

*Table 10. Shows the limited accelerations for the frequencies*

## 5.5 Conclusion

The difference between NBCC’s and Eurocode’s result is not that big even though they are calculated in different ways. Hence ISO is the accepted accelerations in the building, NBBC’s and Eurocode’s accelerations will be compared with them. And as can be shown in Table 9 and 10, the accelerations are very similar even though they are calculated in different ways and in most cases they are below ISO’s accelerations why the structures are accepted.

# 6. Discussion and conclusions

After doing this thesis I have learned that there are a lot of factors to think about when constructing tall buildings. The taller the building gets the harder it is to see which structure that suits it the best. You have to think about the stability, safety and comfort of the building and the economy of the construction. Nowadays you want to build with light weight frames and therefore the wind loads are a large problem.

From the case study the following conclusions could be made: When the building is getting stiffer the frequency is increasing and the deflection is getting smaller. The frequency will decrease on the other hand if you are making the building heavier. Structure 2 is the stiffest of the three structures which can be seen by looking at the frequencies regarding the walls´ thicknesses. The frequency is increasing with small margins, which means that it is already very stiff and therefore it does not matter whether the thickness of the wall is changing. The least stiff structure is structure 1 because the difference between the frequencies is the largest. Structure 3 is very stiff in one direction but not as stiff in the other because of the placement of the cores, which makes it asymmetric.

My own conclusions on the thesis are that the building should be stiff to keep the displacement small, but the stiffer the building becomes the larger the frequency of the building becomes. The frequency should not be large and therefore to decrease the frequency, more dead weight is needed to make the building heavier. The building should not get too heavy for economical and practical reasons, so it is important to find an optimal equilibrium. The heaviest structure is the one with two cores and the least heaviest is the one with trusses on the outside of the building and with the core placed in the middle, which is also the stiffest structure. This conclusion is taken from the case study. It is not easy to figure out how the frequency and acceleration will change just of doing structural changes but you can figure out that a change will happen when putting more weight on the building.

It is also shown that the accelerations that have been calculated are similar even though the codes account different ways of calculations. The three structures have also been shown to be accepted when looking at the accelerations.

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## *7.1 Copyrights for the pictures*

# Appendix 1

The same calculations have been made for all the cases except for the frequency and displacement. The following case is for Structure 3 with s = 0,6 m and
w = 0,6 m.

$f := 0.83Hz$

$T_1 := 600s$

$$g_p := \sqrt{2 \cdot \ln(f \cdot T_1)} + \frac{1}{\sqrt{3}} \cdot \frac{1}{\sqrt{2 \cdot \ln(f \cdot T_1)}} \qquad g_p = 3.688$$

$\Delta := 0.007m$

$h := 72m$

$b := 18m$

$K := 0.10$

$C_{eh} := 1.74$

$\beta := 0.02$

$$\rho := 1.2 \frac{kg}{m^3}$$

$$V' := 24 \frac{m}{s} \qquad V' = 24 \frac{m}{s}$$

$$V_h := V' \cdot \sqrt{C_{eh}} \qquad V_h = 31.7 \frac{m}{s}$$

$$s_1 := \frac{\pi}{3} \cdot \left( \frac{1}{1 + \frac{8 \cdot f \cdot h}{3 \cdot V_h}} \right) \cdot \left( \frac{1}{1 + \frac{10 \cdot f \cdot b}{V_h}} \right) \qquad s_1 = 0.03$$

$$B := \frac{4}{3} \cdot \int_0^{\frac{914}{h}} \left( \frac{1}{1 + \frac{x \cdot h}{457}} \right) \cdot \left( \frac{1}{1 + \frac{x \cdot b}{122}} \right) \cdot \left[ \frac{x}{\left(1 + x^2\right)^{\frac{4}{3}}} \right] dx \qquad B = 0.925$$

$$x_0 := \frac{1220 \cdot f}{V_h} \qquad x_0 = 31.985 \frac{1}{m}$$

$$F := \frac{{x_0}^2}{\left(1 + {x_0}^2\right)^{\frac{4}{3}}} \qquad F = 0.099$$

$$\frac{\sigma}{\mu} = \sqrt{\frac{K}{C_{eh}} \cdot \left(B + \frac{s \cdot F}{\beta}\right)}$$

$$C_g := 1 + g_p \cdot \frac{\sigma}{\mu} \qquad C_g = 1.917$$

$$a := 4 \cdot \pi^2 \cdot f^2 \cdot g_p \cdot \sqrt{\frac{K \cdot s_1 \cdot F}{C_{eh} \cdot \beta}} \cdot \frac{\Delta}{C_g}$$

$$a = 0.034 \frac{m}{s^2}$$

# Appendix 2

The same calculations have been made for all the cases except for the frequency and mass. The following case is for Structure 3 with s = 0,4 m and w = 0,2 m.

$n := 0.71\,Hz$

$T := 600\,s$

$$k_p := \sqrt{2 \cdot \ln(n \cdot T)} + \frac{1}{\sqrt{3}} \cdot \frac{1}{\sqrt{2 \cdot \ln(n \cdot T)}} \qquad k_p = 3.646$$

$$m_q := \frac{33647000\,N}{72 \cdot m \cdot g} \qquad m_q = 52.529\,\frac{ton}{m}$$

$c_f := 1.5$

$\rho := 1.2 \cdot \frac{kg}{m^3}$ $\qquad z_0 := 1\,m$

$b := 18\,m$ $\qquad z_{0.II} := 0.05\,m$

$h := 72\,m$

$z_s := h \cdot 0.6$ $\qquad z_s = 43.2\,m$

$$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} \qquad k_r = 0.234$$

$k_1 := 1$

$c_0 := 1$

$z := h$

$$I_v := \frac{k_1}{c_0 \cdot \ln\left(\frac{z_s}{z_0}\right)} \qquad I_v = 0.266$$

$$c_r(z) := k_r \cdot \ln\left(\frac{z}{z_0}\right)$$

$$v_b := 24\,\frac{m}{s}$$

$$v_m(z) := c_r(z)\cdot c_0\cdot v_b$$

$$\phi(z) := \frac{z}{h}$$

$$K_x := \frac{\int_0^h v_m(z)^2\cdot\phi(z)\,dz}{v_m(z_s)^2\cdot\int_0^h \phi(z)^2\,dz} \qquad K_x = 1.535$$

$$\delta := 0.08$$

$$L_t := 300m$$

$$\alpha := 0.67 + 0.05\cdot\ln(z_0)$$

$$z_t := 200m$$

$$L(z) := L_t\cdot\left(\frac{z}{z_t}\right)^{\alpha}$$

$$f_L(z) := \frac{n\cdot L(z)}{v_m(z)}$$

$$\eta_h := \frac{4.6\cdot h}{L(z_s)}\cdot f_L(z_s) \qquad \eta_h = 11.103$$

$$\eta_b := \frac{4.6\cdot b}{L(z_s)}\cdot f_L(z_s) \qquad \eta_b = 2.776$$

$$R_h := \frac{1}{\eta_h} - \frac{1}{2\cdot\eta_h^{\,2}}\cdot\left(1 - e^{-2\cdot\eta_h}\right) \qquad R_h = 0.086$$

$$R_b := \frac{1}{\eta_b} - \frac{1}{2\cdot\eta_b^{\,2}}\cdot\left(1 - e^{-2\cdot\eta_b}\right) \qquad R_b = 0.296$$

$$S_L(z) := \frac{6.8\cdot f_L(z)}{\left(1 + 10.2\cdot f_L(z)\right)^{\frac{5}{3}}}$$

$$R := \sqrt{\frac{\pi^2}{2\cdot\delta}\cdot S_L(z_s)\cdot R_h\cdot R_b} \qquad R = 0.301$$

$$\sigma(z) := \left(\frac{c_f\cdot\rho\cdot b\cdot I_v\cdot v_m(z_s)^2}{m_q}\right)\cdot R\cdot K_x\cdot\phi(z)$$

$$a := k_p\cdot\sigma(z)$$

$$a = 0.136\frac{m}{s^2}$$