Feasibility study of Air Traffic Control Towers around the globe

Literature report

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Challenge the future

Cover: This photo has been taken at Amsterdam Schiphol Airport. In the back (left) the main air traffic control tower with a height of 100 meter is shown and the secondary tower (right) is the old control tower and operates at this moment as air traffic controller training facility. The main tower is constructed by Bureau De Weger in cooperation with architect Netherlands Airports Consultants (NACO) in 1991. In front an Airbus A320-200 of Swiss International Air Lines is taxiing and ready for departure. [www.airchive.com]

Literature report

"Feasibility study of air traffic control towers around the globe"

"International research regarding the local influences providing an
optimal structural design for air traffic control towers
around the globe in an economical perspective"

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August, 2014

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Preface

This literature report summarises all the findings made during the first phase of the master thesis research "Feasibility study of air traffic control towers around the globe''. This report will form the base of phase II, written in the main research report.

The basic principles of designing medium or high-rise towers considering local circumstances are discussed, in order to be able to answer the sub-question *"What are the design characteristics of high and tall buildings in the chosen countries around the globe?"*

In chapter 1 the mathematical background of tall and high buildings are presented and also several structural systems suitable to construct these buildings are mentioned.

From chapter 1 it is substantiated that horizontal loading is the most important load configuration for tall and high building. Chapter 2 and 3 will discuss the causes of wind and earthquakes loading and their application in the structural design of buildings, by analysing the international building regulations.

From this point, in mainline, enough information is gathered to be able to make a structural design. But to develop a concept that meets all functional requirements, boundary conditions and at the same time is an economical solution, a methodology should be follow during the conceptual phase. Chapter 4 presents the important aspects of this methodology, which will influence the whole design process.

A short summary of this literature report is given in chapter 2 of the research report.

My literature study was carried out in cooperation with Delft University of Technology, Faculty of Civil Engineering & Geosciences, Department Building Engineering and Royal HaskoningDHV, Business line Buildings, Structural design.

The Hague, August 2014

Joost Hartmann

Keywords: structural engineering, high-rise design, foundation, international construction industry, international building codes, The Netherlands, Japan, China, Turkey, Indonesia, Nigeria, wind engineering, earthquake engineering, construction technology,

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1. General tower design

1.1 Introduction

The aim of a structural designer is to make the most optimal structural solution which results in different load bearing systems. Therefore it is important to investigate the different design aspects and the most common types of load bearing structures are described in this chapter.

1.2 Definition

In the worldwide construction industry towers are not defined equally. Around the globe they are categorized on their height, tallness, slenderness and multiple building experts have got their own personal definition. Some are expressed below:

"A building with a slenderness of the lateral load resisting structure larger than 5 is defined as a high-rise building, (hr,eff = h/deff>5) and in general the effect of lateral forces will play an important role in the structural design of these building" [Hoenderkamp, J.C.D, 2007]

"A tall building may be defined as one that, because of its height, is affected by lateral force due to wind or earthquake actions to an extent that they play an important role in the structural design. The influence of these actions must therefore be considered from the very beginning of the design process." [Smith & Coull, B.S, 1991]

1.3 Design aspects

As mentioned the lateral forces, caused by wind or earthquake action, plays a vital role on the structural design of a tower. To provide enough structural safety, the design has to fulfil structural requirements regarding stability, strength and stiffness. These design requirements are illustrated below considering a rectangular building stabilized by a central core, which is constant over the height and loaded in its weak direction. Note that these considerations are made for wind loading; earthquake loading will be discussed in chapter 3.

1.3.1 Stability

The first and most fundamental requirement in structural design is stability. The building may not tip over as a result of the applied horizontal loads. In order to prevent tipping over, the overturning moment caused by the horizontal loads must be smaller than the resisting moment of the dead load of the building. In figure 1.1 a schematization is given and equations 1.1 to 1.3 [Berg vd, R., 2012] give the stability equations. If equation 1.3 is not met, the foundation must be designed to resist tensile forces and this must be prevented as much as possible.

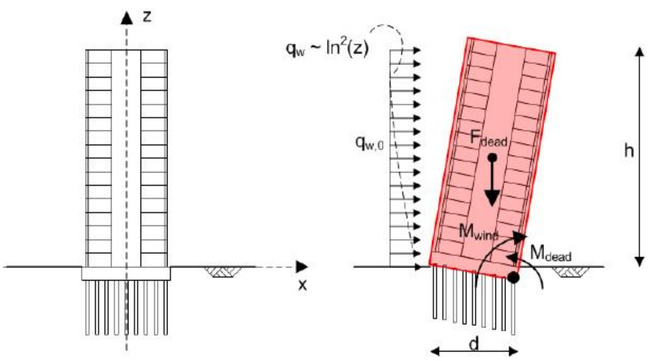


Figure 1.1: Stability configuration [Berg vd, R. 2012]

$$Mwind = \int_0^h qw(z) \times z \, dz = \frac{1}{2} \times qw,0 \times z^2 \tag{1.1}$$

$$Mdead = \frac{1}{2} \times d \times Fdead \tag{1.2}$$

$$Mdead > Mwind \tag{1.3}$$

1.3.2 Strength

The second structural requirement is strength. The building must have enough strength to resist the most critical load combinations in order to avoid failure. The highest stresses in the building must be lower than the allowable stresses of the loaded material to provide enough safety for the users. In figure 1.2 a schematization of the vertical and horizontal stresses is presented.

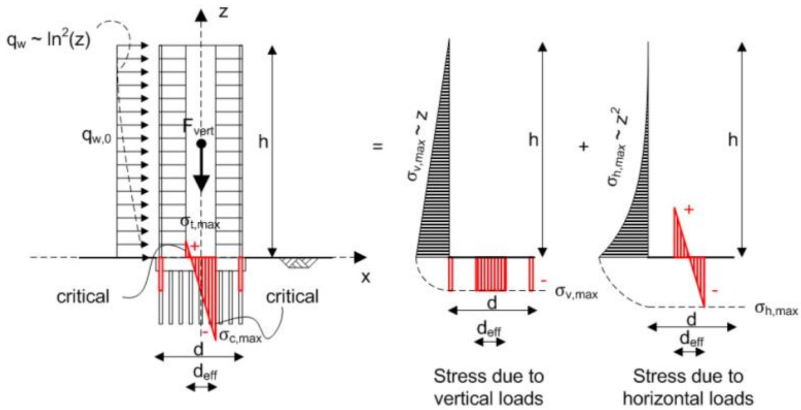


Figure 1.2: Stress configuration [Berg vd, R., 2012]

The main purpose of the load bearing structure is to transfer horizontal and vertical loads to the foundation.

Horizontal loads caused by e.g. the wind causes a bending moment at the bottom of the building. This bending moment causes compressive and tensile stresses at the edge of the stability system. The maximum stress at the edge of the stability system increases quadric with the building height, given by equation 1.1.

Vertical loads consist of a combination of dead load (self-weight) and live loads causing a compressive stress distribution at the bottom of the building. The maximum compressive stress at the bottom of the building increases linear with the building height.

As a result of the horizontal and vertical loads, a stress summation at the base of the building can be made, giving the maximum compressive and/or tensile stress distribution. These stresses can be modelled with equations 1.4 to 1.6 [Berg vd, R., 2012].

$$\sigma h, \max = \pm \frac{Mwind \times \frac{1}{2} deff}{Icore} \tag{1.4}$$

$$\sigma v, \max = -\frac{Fvert}{Acore + columns} \tag{1.5}$$

$$\sigma \max = \frac{Fvert}{Acore + columns} \pm \frac{Mwind \times \frac{1}{2} deff}{Icore} \tag{1.6}$$

In general the required lifetime of a building is long; 50 to 100 years are common numbers. During the lifetime local stresses can occur due to creep, shrinkage and temperature differences which cause elastic deformations and can lead to higher local stresses in elements. All these effects should be taken into account in the structural design. [Smith, B.S., 1991]

1.3.3 Stiffness

The third structural requirement is the stiffness. Due to horizontal loads, buildings deform during the time and its degree is depending on the stiffness of the building. In general the maximum deflection of the building is been restricted to 1/500 of the structural height, the so called drift index [Smith, B.S., 1991] in order to prevent malfunction of non-structural components, large second order effects and cracking of concrete.

The deformation of a tower can be considered as a clamped beam that deforms due to bending and shear deformations. The two deformation shapes are given in figures 1.3 and 1.4.

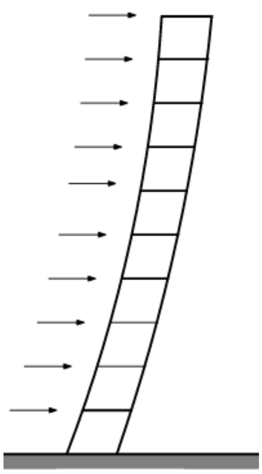


Figure 1.3: Shear deformation [Simone, A., 2011]

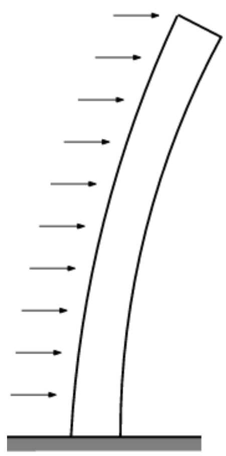


Figure 1.4: Bending deformation [Simone, A., 2011]

Using the reader of Simone [Simone, A., 2011] both differential equations are expressed for bending and shear deformation. These equations are expressed in equations 1.7 and 1.11.

The bending deformation has been derived considering the Euler Bernoulli beam differential equation:

$$EI\,\frac{d^4v}{dx^4} = q(x) \tag{1.7}$$

After integrating equation 1.7 four times, equation 1.8 is obtained:

$$EIv = \frac{1}{24}qx^4 + \frac{1}{6}\,C1x^3 + \frac{1}{2}\,C2x^2 + C3x + C4 \tag{1.8}$$

Because equation 1.8 is fourth order, four boundary conditions are needed to solve this problem and are expressed for a clamped beam below:

$for\ x = 0$ $\quad$ $for\ x = l$

$u = 0$ $\quad$ $M = 0$
$\varphi = 0$ $\quad$ $V = 0$

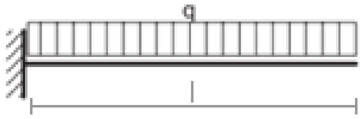


Figure 1.5: Clamped beam [Hartmann, J., 2013]

Gives values:

$C1 = -ql$ $\quad$ $C2 = \frac{1}{2}ql$
$C3 = 0$ $\quad$ $C4 = 0$

Gives the deformation equation:

$$u = \frac{q}{24EI}(x^4 - 4x^3l + 6x^2l^2) \tag{1.9}$$

When x = h (structural height), equation 1.9 becomes:

$$u = \frac{qh^4}{8EI} \tag{1.10}$$

The shear deformation has been derived considering the shear beam differential equation:

$$-GAs\,\frac{d^2v}{dx^2} = q(x) \tag{1.11}$$

After integrating equation 1.11 two times, equation 1.12 is obtained:

$$-GAsv(x) = \frac{1}{2}qx^2 + C1x + C2 \tag{1.12}$$

Because equation 1.12 is second order, two boundary conditions are needed to solve this problem and are expressed for the clamped beam

$for\ x = 0$ $\quad$ $for\ x = l$

$u = 0$ $\quad$ $\varphi = 0$

Gives values:

$C1 = ql$ $\quad$ $C2 = 0$

Gives the deformation equation:

$$u = \frac{q}{2GAs}(2lx - x^2) \tag{1.13}$$

When x = h (structural height), equation 1.13 becomes:

$$u = \frac{qh^2}{2GAs} \tag{1.14}$$

Combine equations 1.10 and 1.14 and the total deformation of the building is equal to:

$$u = \frac{qh^4}{8EI} + \frac{qh^2}{2GAs} \tag{1.15}$$

Usually, for very slender beams, the shear component can be neglected [Simone, A., 2011] and equation 1.10 fulfils. For rigid frames the shear deformation is larger and cannot be neglected and equation 1.15 is governed.

Now the lateral deformation configuration of the structure is known. Beside the stiffness of the load bearing structure also the foundation plays an important role regarding the deformation of a building [Hoenderkamp, J.C.D, 2002]. Foundations are sensitive to rotation due to e.g. shortening of piles. Therefore the foundation is modelled as a rotation spring and can be integrated into equation 1.15 and a schematization is made of the total deformation in figure 1.6.

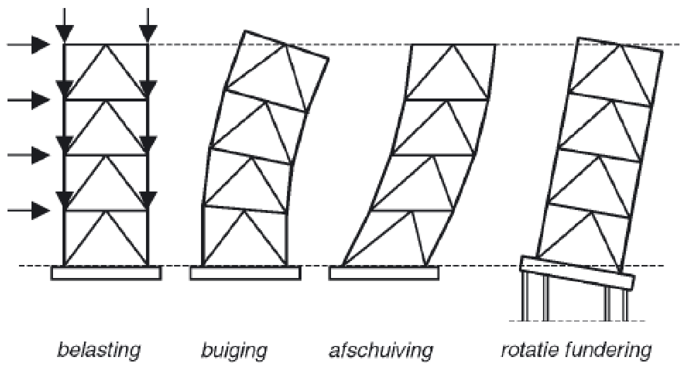


Figure 1.6: Total deformation building [Hoenderkamp, J.C.D., 2002]

The deformation of the building caused by the rotation of the foundation is expressed by the next equations [Hoenderkamp, J.C.D., 2002]

$$u = h \times \frac{Mwind}{C} = \frac{qh^3}{2C} \tag{1.16}$$

$$C = k \times \sum d^2 \tag{1.17}$$

Combining equations 1.15 and 1.16 gives the total deformation:

$$utot = \frac{qh^3}{2C} + \frac{qh^4}{8EI} + \frac{qh^2}{2GAs} \tag{1.18}$$

Taking into account the maximal drift index of 1/500, the minimum required stiffness becomes

$$EI \geq \frac{500\ \times q \times\ h^3}{8} - \frac{h\ \times C}{4} - \frac{h^2 \times GAs}{4} \tag{1.19}$$

1.3.4 Dimension influences

The influence of the vertical dimension of the building on its structural behaviour is an important aspect. If no horizontal load existed, a building with double the height, but 2 times as slender, would have approximately the same material demand as normal Building. Due to the lateral forces this is not applicable and the material demand increases along with the height. Doubling the height has the following consequences with respect to stability, strength and stiffness [Hoenderkamp, J.C.D, 2007]

- The uniform stress due to vertical loads at ground level, increases with a factor $2^1 = 2$
- The shear load at base level increases to the power one, increases with $2^1 = 2$
- The bending moment increases to the power two, increases with $2^2 = 4$
- The drift index at the top increases to the power three, increases with $2^3 = 8$
- The deflection at the top increases to the power four, increases with $2^4 = 16$

1.3.5 Accidental design situations

Another important design feature of the structural design of a building is accidental design situations, often causing progressive collapse. A part of the structure is not able to bear the load and fails and progressive collapse starts. This increases the load on the surrounding bearing structure even more resulting eventually in total collapse. These progressive collapses often appear after unforeseen situations like, explosions, fires, collisions and earthquakes.

Two different strategies are taken in the design phase to limit these events. The first strategy is identification of accidental actions and measurements should be taken to mitigate the risk of accidental actions. The second strategy is limiting the extent of localised failure.

Air traffic control towers are in general slender towers consisting of a single structural system. Often no second load bearing path is provided. In those cases the Eurocode states that the structural elements must be designed as key elements, on which the stability of the structure depends to sustain the effects of a model of accidental action A_d. The recommended value for the uniformly distributed load A_d is 34 kN/m^2 for building structures. These key elements should be capable of sustaining this load in horizontal and vertical direction (in one direction at a time) to the member. This load may be a concentrated or distributed load.

1.4 Types of load bearing structures

To fulfil the design requirements of strength, stiffness and stability, different main load bearing structures are developed to transfer the horizontal and vertical loading to the foundation. The most used materials for the load bearing structures are reinforced concrete and steel. The different types of structures are presented in the figures below and are indicated with their maximum height of application expressed in stories [Hoenderkamp, J.C.D, 2007]. Many variations of these types of structures are possible, but the main essence of each structure is described in the following sections.

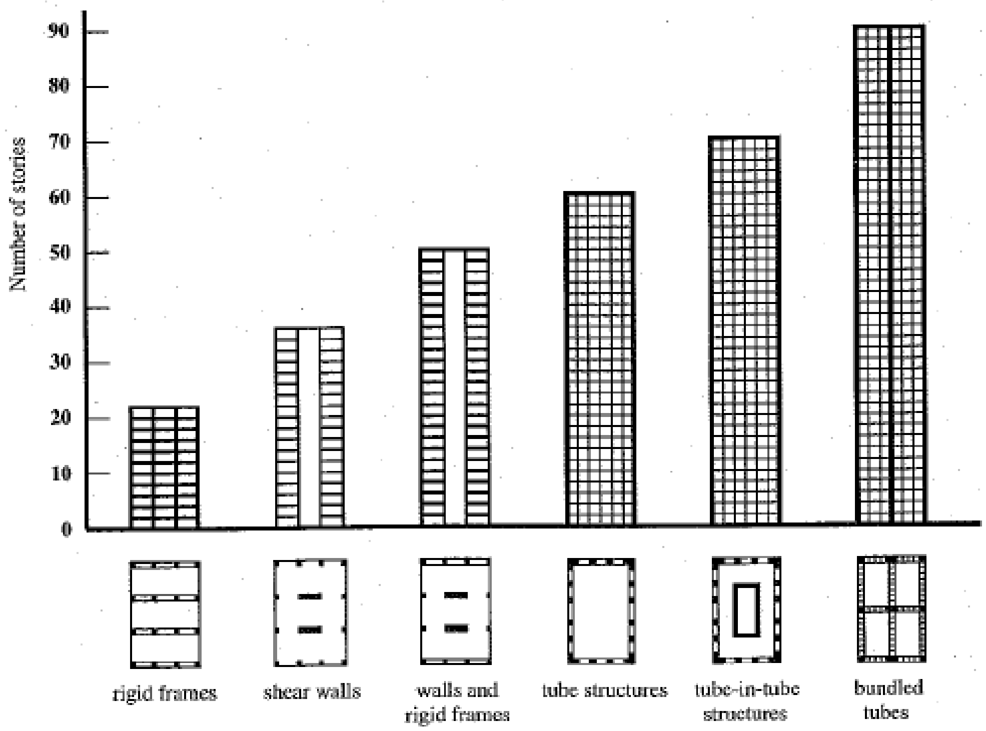


Figure 1.7: Reinforced concrete load bearing structures [Hoenderkamp, J.C.D, 2007]

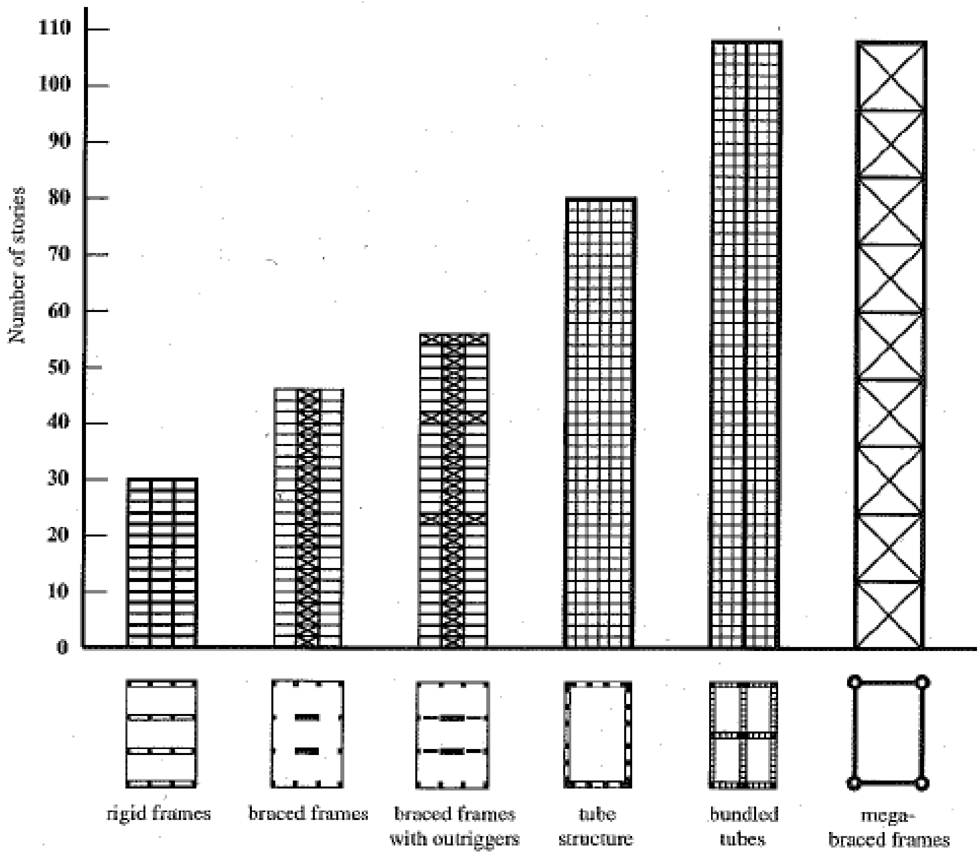


Figure 1.8: Steel load bearing structures [Hoenderkamp, J.C.D, 2007]

1.4.1 Rigid Frame systems

The first system is the rigid frame system. This system is most suitable for lower towers and can be erected from both reinforced concrete and steel. The economic height for a concrete structure is limited around 80 meters and for a steel structure 120 meters. The horizontal loads are transferred to the foundation by bending action of the columns and beams. The connection between the columns and beams are moment resistant and provide horizontal stiffness for the un-braced frames. An advantage of the un-braced frames is that they provide more open functional space. The major disadvantage of is this system is the low shear rigidity, because the horizontal load is transferred by bending causing large shear formations.

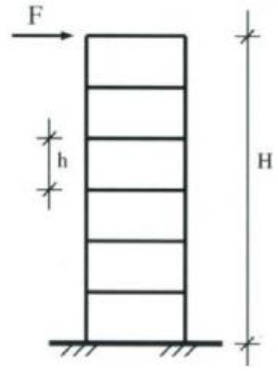


Figure 1.9: Rigid frame system [Hoenderkamp, J.C.D, 2007]

1.4.2 Shear wall systems

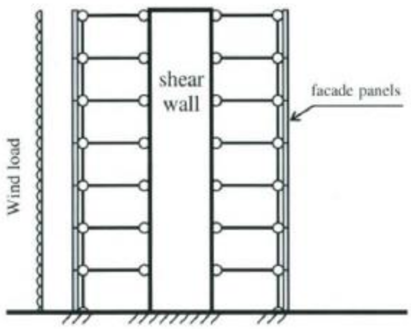


Figure 1.10: Shear wall system [Hoenderkamp, J.C.D., 2007]

The second system is the shear wall system. This system is mainly erected from concrete and has an economic height of 160 meters. The horizontal loads are transferred to the foundation by bending of the walls. These walls are clamped to the foundation and provided the horizontal stiffness. Walls in different directions are necessary to provide enough stiffness in each direction. It is possible to couple several walls in one direction to increase the stiffness. The cooperation between the coupled shear walls depends on the stiffness of the connection, e.g. stiffness of floors or floor beams.

1.4.3 Core structures

An equivalent to the coupled shear wall system is the core structure. In this case the shear walls are integrated together shaping a tree-dimensional core structure. Also core structures can only be erected from reinforced concrete and has an economic height of 160 meters. The horizontal loads are transferred to the foundation by bending of the core. Inside the core often elevators, stairs and equipment risers are accommodated.

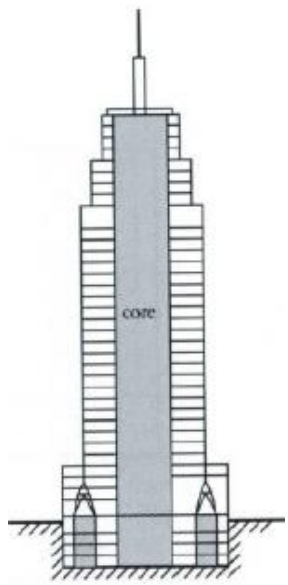


Figure 1.11: Core structure [Hoenderkamp, J.C.D, 2007]

1.4.4 Braced frame systems

The fourth load bearing system is the braced frame system. This system is mainly erected from steel and has an economic height of 200 meters. The horizontal load is transferred to the foundation by pure axial deformation of the elements. The connections between the columns, beams and diagonals are hinges. Bending of the frame activates axial deformation of the vertical elements, while shear of the frame activates the diagonals and provide the horizontal stiffness for the braced frames. The major disadvantage of this system is the diagonal bracing, providing less open functional spaces.

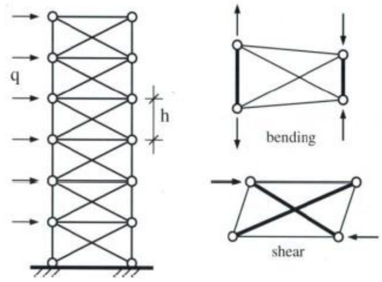


Figure 1.12: Braced frame system [Hoenderkamp, J.C.D., 2007]

1.4.5 Outrigger system

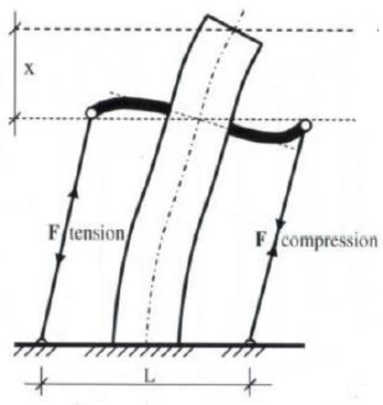


Figure 1.13 Outrigger system [Hoenderkamp, J.C.D, 2007]

The outrigger system is often erected from concrete combined with steel and has an economic height of 240 meters. The horizontal load is transferred to the foundation by a central core or shear wall system combined with the façade columns. The façade columns are connected to the core system by an outrigger system. The façade columns transfer tension and compression forces from the outrigger to the foundation, reducing the maximum moment located at the base. The advantage of this system is the dimension reduction of the central stabilizing system. The major disadvantage of this system is the functional loss of space at the outrigger level.

1.4.6 Tube system

The tube system can be erected from both reinforced concrete and steel. The economic height for a concrete structure is limited to approximately 240 meters and for a steel structure to 320 meters, making it less attractive for the use of an air traffic control tower. The horizontal loads are transferred to the foundation by the tube system that acts as a rigid frame and providing horizontal stiffness. The principle behind the tube system is to enlarge the building's moment of inertia, by placing as much as possible load bearing material around the perimeter. The major disadvantage of this system is the occurrence of peak stresses, the so called shear lag, in the corners of the building. To minimize shear lag in the structure, mega diagonals, belt structures or division in smaller tubes can be applied.

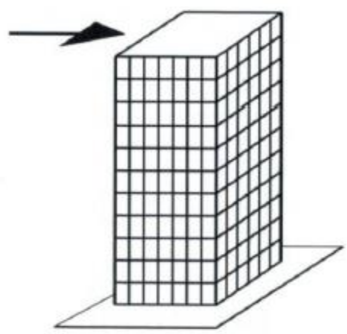


Figure 1.14: Tube structure [Hoenderkamp, J.C.D, 2007]

1.4.7 Cable-stayed structures

The cable-stayed structure is not expressed in the load bearing structure overview of Hoenderkamp, but it is a conventional used structural system to stabilize TV/radio towers. This method is also used for air traffic control towers (London, Sydney). Therefore this system will also be analysed.

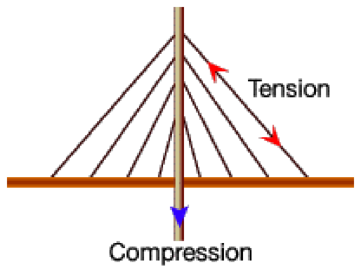


Figure 1.15: Cable-stayed principle [images-google, 2013]

The idea behind a cable-stayed structure is to provide more stability to a (slender) core structure. The vertical loads are transferred to the foundation in compression by a core structure. A part of the horizontal load is transferred to the foundation by bending of the core system and the remaining horizontal forces are transferred to the foundation by wire stay cables in tension. The advantage of this system is to design very slender core structures, reducing the structure perimeter and improving the visual impact. The disadvantage of this system is to construct extra (tensile) foundation for the cable-stays ends, the dynamic performance is harder to control and it type of structure is more sensitive for terroristic attacks.

1.5 Types of foundations

A pre-requisite to the success of a foundation project is a thorough understanding on the ground conditions. The overall objective of a site investigation for foundation design is to determine the site constraints, geological profile and the properties of the various strata. The geological sequence can be established by making boreholes from which the soil and rock samples are collected for testing and identification. Also an in-situ test can be carried out to determine the properties of the ground. [Coduto, D., 2001]

The importance of a properly planned and executed ground investigation cannot be over-emphasises. The information obtained from the investigation will allow an appropriate geological model to be constructed. This determines the selection of the optimum foundation system for the proposed structure.

In the foundation engineering two main types of foundations are classified; the shallow and deep foundations. Both classes will be determined and the common foundation structures will be described.

1.5.1 Shallow foundations

Shallow foundations are generally more economical than deep foundations. Shallow foundations are structures founded near the ground surface, so these structures don't have to be installed deep into the ground and extensive ground improvement works are not required. In general the foundation depth is less than 3 meters. They are often used to support structures at sites where subsurface materials are sufficiently strong and they are less suitable in weak or highly compressible souls, such as peat and poorly-compacted fill. Shallow foundations are mainly used to support light-medium weighted buildings. High-rise structures or the presence of weak ground bearing materials do not necessarily prohibit the use of shallow foundations. Suitable design provision or ground improvement could be considered to overcome the difficulties. Below the most common shallow foundations are presented.

Spread foundations

This type of foundation is used to support an individual point load, e.g. column. It forms an enlargement at the bottom of a column to spreads the applied load sufficiently over the soil area. Spread foundations are the most common foundations and are built in different shapes and sizes.

- Square spread footings
- Rectangular spread footings
- Circular spread footings
- Continuous spread footings
- Combined footings
- Ring spread footings

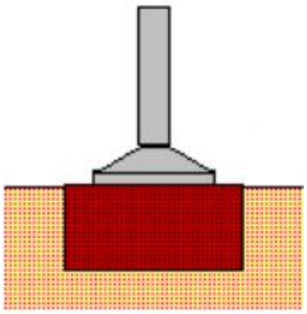


Figure 1.16: Spread foundation [Hartmann, J., 2013]

Mat/Raft foundations

This type of foundation is used to spread the load over a larger soil area, normally the entire area of the structure. Mat foundations are usually continuous in two directions and are suitable when the under laying souls have a low bearing capacity or large differential settlements are anticipated. Mat foundations are also used when column loads or other loads are close together and individual spread foundations would interact. In some cases, the mat foundation is designed as a cellular mat foundation, where hollow boxes are formed in the concrete slab. The main advantage is to reduce the overall weight of the foundation. Mat foundations could be stiffened by using ribs are beams.

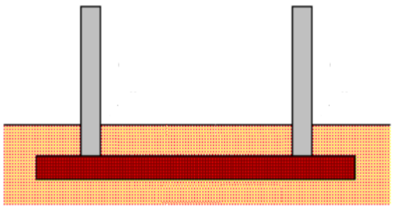


Figure 1.17: Mat foundation [Hartmann, J, 2013]

1.5.2 Deep foundations

Deep foundations are foundations founded deeply below the ground surface. The depth is usually higher than 3 meter and can be used to transfer the load to a stiffer deeper soil, in cases unsuitable soils are presented near the surface. Deep foundations are suitable when a large uplift is required (tensile forces), but also have a large bearing capacity downwards. Another major advantage of deep foundations is that this type of foundations will retain stable during future excavation, where shallow foundation would subjected to settlements. The disadvantage of deep foundations is that they are the most expensive foundations. Below the most deep foundations are presented.

Pile foundation

Foundation piles are long and slender elements that transmit load-bearing loads to deeper soil with higher bearing capacities, through soils with lower capacities. They are often used in the Netherlands and are applied when shallow foundations are not applicable. Piles can transfer besides compressive forces also tensile forces, which make it suitable for anchor structures.

Different types of piles exist, each having their own capacity. The capacity is generated by end bearing and skin friction of the pile. The most common types are: End bearing piles, friction piles, settlement reducing piles, tension piles, laterally loaded piles and piles in fill.

Drilled shafts

The second group of deep foundation are drilled shafts. The difference between piles and drilled shafts is that piles are prefabricated elements driven into the ground, whereas drilled shafts are cast in place. This type of deep foundation is very suitable for heavy structural loads and several types are common used: pier, drilled pier, bored pier and cast-in-place pile.

Caissons

Caissons are prefabricated boxes or cylinders which are sunk into the ground to the required level by excavating or dredging. Afterwards the prefabricated boxes are filled up with concrete.

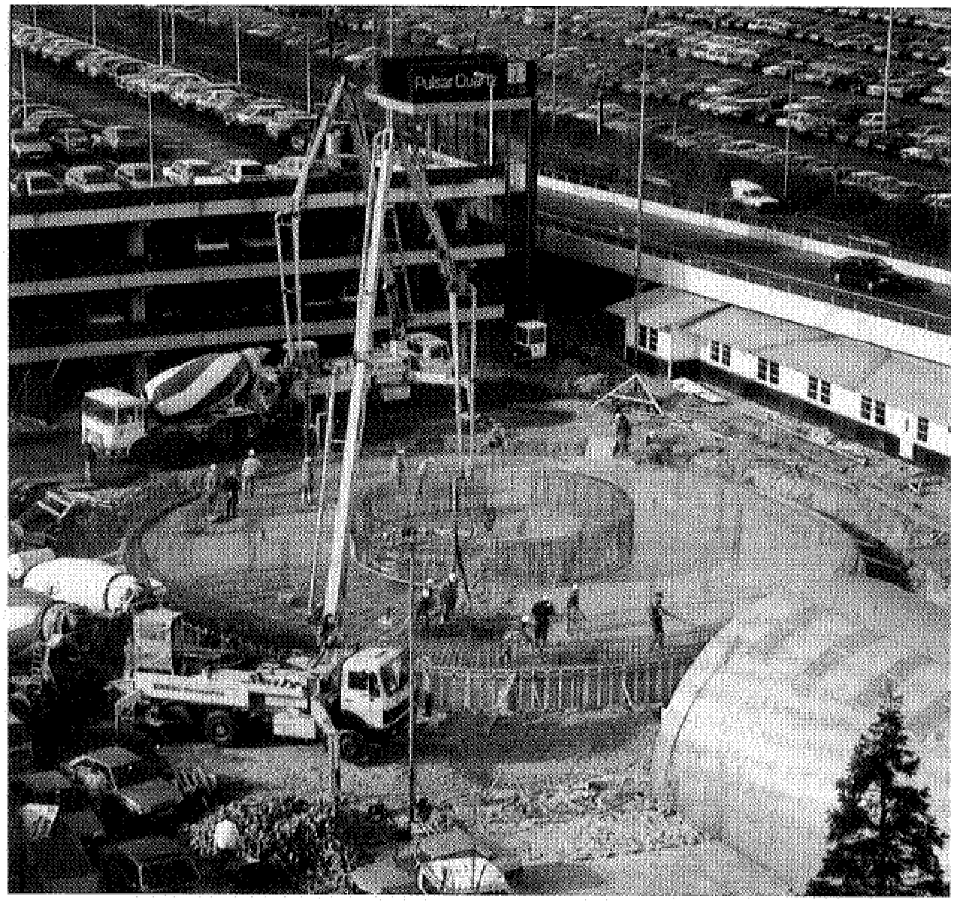


Figure 1.18: Construction foundation slab air traffic control tower Schiphol [Weersma, S., 1991]

2. Wind Engineering

2.1 Introduction

Wind actions fluctuate with time and act directly on the surface of the structure and as mentioned in chapter 1 wind plays an important role in the structural design of large towers, resulting in normal forces to the surface or individual cladding components.

To meet all the structural requirements in service limit state (SLS) and ultimate limit state (ULS) it is important to investigate all the wind parameters in order to design an efficient and economic structure. To make an efficient wind engineering research first the Eurocode will be analysed in detail. The calculating method, wind parameters and other important wind issues will be discussed. On basis of the Eurocode the other "local" building codes will be analysed in short. The main comparisons and difference are investigated and expressed in the research report. Afterwards basic design principles for buildings are presented to reduce wind action and their disadvantageous properties.

2.2 Wind phenomenon

Wind is a constantly appearing phenomenon along the earth surface. Wind is caused by air flowing from high pressure areas to areas with low air pressures. This process is constantly changing from direction and magnitude. Since the earth is rotating the air does not flow directly between these two areas, but it is deflected and therefore the wind flows mainly around the high and low pressure areas. The magnitude of the wind is determined by the pressure gradient and is drawn on weather maps by lines of constant pressure. These lines are called isobars and are labelled with pressure values in millibars and the closer the isobars the stronger the wind. In figure 2.1 a typical weather map is presented.

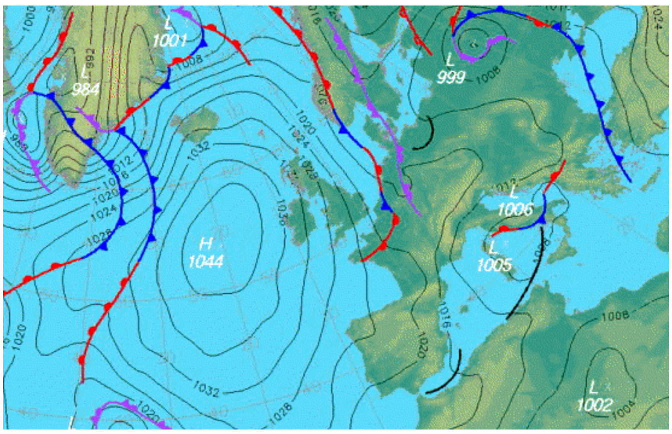


Figure 2.1: Weather map with isobars Europe [images.google.com]

Around the globe there are different methods to classify the magnitude of the wind. Globally the Beaufort wind force scale is used. This scale is presented in table 2.1 and the wind speeds are classified in 12 scales. As been noticed the scale is limited to the Beaufort wind scale of magnitude 12, representing a wind speed of 33 m/s.

Higher magnitudes are rated by a different scale, namely by the Saffir-Simpson Hurricane wind scale, presented in table 2.2. This scale estimates the potential property damage. This damage is caused by exceptional storms and are named dependently on their geographically path, like typhoons, cyclones and hurricanes. These storms are formed over warm oceans in the summer and early fall and are intense low pressure areas that gets his energy by evaporation of warm water from the ocean surface

Table 2.1: Beaufort Wind Force Scale [www.metoffice.gov.uk]

Beaufort wind scale	Mean Wind Speed		Limits of wind speed		Wind descriptive terms
	Knots	ms^{-1}	Knots	ms^{-1}	
0	0	0	<1	<1	Calm
1	2	1	1–3	1-2	Light air
2	5	3	4–6	2-3	Light breeze
3	9	5	7–10	4-5	Gentle breeze
4	13	7	11–16	6-8	Moderate breeze
5	19	10	17–21	9-11	Fresh breeze
6	24	12	22–27	11-14	Strong breeze
7	30	15	28–33	14-17	Near gale
8	37	19	34–40	17-21	Gale
9	44	23	41–47	21-24	Severe gale
10	52	27	48–55	25-28	Storm
11	60	31	56–63	29-32	Violent storm
12	-	-	64+	33+	Hurricane

Table 2.2: Saffir-Simpson Hurricane Wind Scale [www.nhc.noaa.gov]

Category	Sustained Winds	Types of Damage Due to Hurricane Winds
1	74-95 mph 64-82 kt 119-153 km/h	**Very dangerous winds will produce some damage:** Well-constructed frame homes could have damage to roof, shingles, vinyl siding and gutters. Large branches of trees will snap and shallowly rooted trees may be toppled. Extensive damage to power lines and poles likely will result in power outages that could last a few to several days.
2	96-110 mph 83-95 kt 154-177 km/h	**Extremely dangerous winds will cause extensive damage:** Well-constructed frame homes could sustain major roof and siding damage. Many shallowly rooted trees will be snapped or uprooted and block numerous roads. Near-total power loss is expected with outages that could last from several days to weeks.
3 (major)	111-129 mph 96-112 kt 178-208 km/h	**Devastating damage will occur:** Well-built framed homes may incur major damage or removal of roof decking and gable ends. Many trees will be snapped or uprooted, blocking numerous roads. Electricity and water will be unavailable for several days to weeks after the storm passes.
4 (major)	130-156 mph 113-136 kt 209-251 km/h	**Catastrophic damage will occur:** Well-built framed homes can sustain severe damage with loss of most of the roof structure and/or some exterior walls. Most trees will be snapped or uprooted and power poles downed. Fallen trees and power poles will isolate residential areas. Power outages will last weeks to possibly months. Most of the area will be uninhabitable for weeks or months.
5 (major)	157 mph or higher 137 kt or higher 252 km/h or higher	**Catastrophic damage will occur:** A high percentage of framed homes will be destroyed, with total roof failure and wall collapse. Fallen trees and power poles will isolate residential areas. Power outages will last for weeks to possibly months. Most of the area will be uninhabitable for weeks or months.

2.3 Eurocode Wind Actions Part 1998-1-1-4

2.3.1 Characteristic values

The wind actions calculated by the Eurocode are characteristic values. They are determined from the basic values of wind velocity or the velocity pressure. In accordance with the Eurocode the characteristic values have annual probabilities of accidence of 0,02, which is equivalent to a mean return period of 50 years [Part 3.4, NEN-EN 1991-1-4].

2.3.2 Model

The effect of the wind on the structure depends on the size, shape and dynamic properties of the structure. The Eurocode covers dynamic response due to along-wind turbulence in resonance with the along-wind vibrations of a fundamental mode shape. The entire calculation method is presented in the following sections.

2.3.3 Basic wind velocity Vb

The basic wind velocity Vb is defined as the characteristic 10 minute mean wind velocity, irrespective of the wind direction and season at 10 meter above the ground and is determined by equation 2.1:

$$Vb = Cdir \times Cseason \times Vb,0 \tag{2.1}$$

Wherein:

$Vb,0$	Is the fundamental value of the basic wind velocity
$Cdir$	Is the wind direction factor, recommended value $Cdir = 1.0$
$Cseason$	Is the season factor, recommended value $Cseason = 1.0$

2.3.4 Average wind velocity Vm

The average wind velocity Vm(z) at a height z above ground depends on the terrain roughness, orography and the basic wind velocity and is determined by equation 2.2:

$$Vm(z) = Cr(h) \times Co(h) \times Vb \tag{2.2}$$

Wherein:

$Cr(z)$	Is the roughness factor, (section 2.3.5)
$Co(z)$	Is the orography factor, recommended value $Co(z) = 1.0$
Vb	Is the basic wind velocity, (section 2.3.3)

2.3.5 Roughness factor Cz

The roughness factor $Cr(z)$ takes into account the height and the roughness of the terrain wind wards of the structure and is determined by equations 2.3 or 2.4 using equation 2.5:

$$Cr(z) = Kr \times \ln(\frac{h}{z0}) \quad for\ zmin \leq z \leq zmax \tag{2.3}$$

$$Cr(z) = Cr \times (zmin) \quad for\ z \leq zmin \tag{2.4}$$

$$Kr = 0.19 \times (\frac{z0}{z0,II})^{0.07} \tag{2.5}$$

Wherein:

z	Is the height of the structure, also h
$z0$	Is the roughness length, (Table 2.3)
Kr	Is the terrain factor
$z0, II$	Is the reference value, recommended 0.05 [m]
$zmin$	Is the minimum height, (Table 2.3)
$zmax$	Is the maximum height, 200 [m]

2.3.6 Orography factor Co

The effects of the orography $Co(z)$ may be neglected when the average ground slope is less than 3°. In this thesis the airport and surrounding surfaces are considered to be flat and the effect of orography can be neglected during this research. The recommended value will be $Co(z) = 1.0$.

2.3.7 Terrain categories

In table 2.3 the terrain categories are presented. In this thesis the airport surfaces are considered to be category I, due to functional and safety requirements the surfaces are open and without obstacles.

Table 2.3: Terrain categories [NEN-EN 1994-1-4:2005, Table 4]

Terreincategorie	z_0 m	z_{min} m
0 Zee of kustgebied met wind aanstromend over open zee	0,003	1
I Meren of vlak en horizontaal gebied met verwaarloosbare vegetatie en zonder obstakels	0,01	1
II Gebied met lage begroeiing als gras en vrijstaande obstakels (bomen, gebouwen) met een tussenruimte van ten minste 20 obstakelhoogtes	0,05	2
III Gebied met regelmatige begroeiing of gebouwen of vrijstaande obstakels met een tussenruimte van ten hoogste 20 obstakelhoogtes (zoals dorpen, voorstedelijk terrein, blijvend bos)	0,3	5
IV Gebied waar ten minste 15 % van de oppervlakte is bedekt met gebouwen met een gemiddelde hoogte boven 15 m	1,0	10
De terreincategorieën zijn geïllustreerd in A.1.		

2.3.8 Wind turbulence intensity Iv

The wind turbulence intensity $Iv(z)$ is determined as the standard deviation σv of the turbulence divided by the average wind velocity and is determined by equations 2.6 or 2.7 using equation 2.8:

$$Iv(z) = \frac{\sigma v}{Vm(h)} = \frac{KI}{Co(z) \times \ln\frac{h}{z0}} \quad for\ zmin \leq z \leq zmax \tag{2.6}$$

$$Iv(z) = Lv(zmin) \quad for\ z < zmin \tag{2.7}$$

$$\sigma v = Kr \times Vb \; x \; KI \tag{2.8}$$

Wherein:

σv	Is the standard deviation of turbulence
Vm	Is the average wind velocity, (section 2.4.4)
KI	Is the turbulence factor, recommended value $KI = 1.0$

2.3.9 Peak velocity pressure qp

The peak velocity pressure qp at reference height z includes mean and short velocity fluctuations and is given by equation 2.9 using equations 2.10 and 2.11:

$$qp(z) = (1 + 7 \times Iv(z)) \times \frac{1}{2} \times \rho \times Vm^2(z) = Ce(z) \times qb \tag{2.9}$$

$$Ce(z) = \frac{qp(z)}{qb} \tag{2.10}$$

$$qb = \frac{1}{2} \times \rho \times Vb^2 \tag{2.11}$$

Wherein:

Iv	Is the wind turbulence (section 2.3.8)
ρ	Is the air density, recommended value $\rho = 1.25 \; kg/m^3$
Ce	Is the exposure factor
qb	Is the basis velocity pressure

With figure 2.2 the exposure factor Ce can be determined to calculate the peak velocity pressure qp quickly.

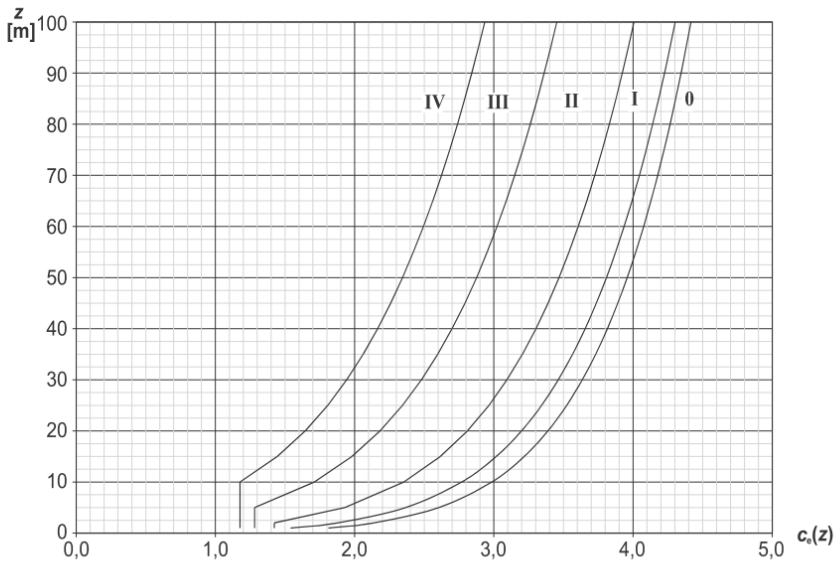


Figure 2.2: Exposure factor Ce(z) for C0 = 1,0 and Kl = 1,0 [NEN-EN 1994-1-4:2005, Figure 4.2]

2.3.10 Structural factor $CsCd$

The structural factor $CsCd$ takes into account the effect of wind actions from the non-simultaneous occurrence from extreme wind pressure at the surface of the structure Cs together with the vibration effect of the structure due to turbulence Cd. In the Eurocode it is taken that the reference height is calculated by equation 2.13 and figure 2.3 and therefore the structural factor is valid given by equation 2.12

$$CsCd = \frac{1 + 2 \times Kp \times Iv(zs) \times \sqrt{B^2 + R^2}}{1 + 7 \times Iv(zs)} \tag{2.12}$$

$$zs = 0.6 \times z \geq zmin \tag{2.13}$$

Wherein:

zs	Is the reference height for the structural factor
Kp	Is the peak factor (section 2.3.11)
$Iv(zs)$	Is the wind turbulence intensity (section 2.3.8)
B^2	Is the background response factor (section 2.3.12)
R^2	Is the resonance response factor (section 2.3.13)
h	Is the height of the structure

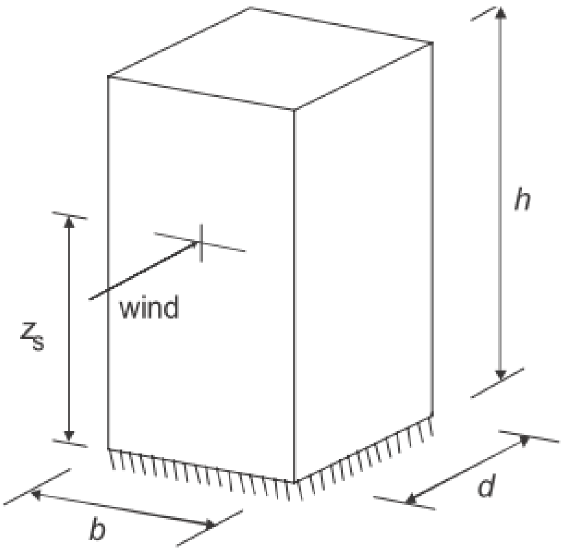


Figure 2.3: Building configuration [NEN-EN 1994-1-4:2005, Figure 6.1)

2.3.11 Peak factor Kp

The peak factor Kp is determined as the ratio between the maximum value of the fluctuating response part and its standard deviation and is given by equation 2.14 using equations 2.15 and 2.16. The natural frequency of the structure is estimated in this phase. The exact natural frequency can be determined by calculation software more accurate.

$$Kp = \sqrt{2 \times \ln(v \times T)} + \frac{0,6}{\sqrt{2 \times \ln(v \times T)}} \quad or \quad Kp = 3 \tag{2.14}$$

$$v = n1,x \sqrt{\frac{R^2}{B^2 + R^2}} \quad ; \quad v \geq 0.08\ Hz \tag{2.15}$$

$$n1,x = \frac{46}{h} \tag{2.16}$$

v	Is the gust frequency, limit $v \geq 0.08\ Hz$
T	Is the average time of the reference wind velocity, T = 600 [sec]
$n1,x$	Natural frequency of the structure

2.3.12 Background response factor B^2

The background response factor B^2 subscribes the lack of full correlation of the wind pressure on the surface of the structure and is given by equation 2.17. The turbulence length scale $L(zs)$ determines the average size of the wind gust and is given by equations 2.18, 2.19 and 2.20.

$$B^2 = \frac{1}{1 + \frac{3}{2} \times \sqrt{\left(\frac{b}{L(zs)}\right)^2 + \left(\frac{h}{L(zs)}\right)^2 + \left(\frac{b}{L(zs)}\frac{h}{L(zs)}\right)^2}} \tag{2.17}$$

$$L(zs) = Lt \times \left(\frac{zs}{zt}\right)^{\alpha} \quad for\ z \geq zmin \tag{2.18}$$

$$L(zs) = L(zmin) \quad for\ z < zmin \tag{2.19}$$

$$\alpha = 0,67 + 0,05\ ln(z0) \tag{2.20}$$

Wherein:

b	Is the width of the building
h	Is the height of the building
$L(zs)$	Is the turbulence length scale
zt	Is the reference height, 200 [m]
Lt	Is the reference scale length, 300 [m]
$z0$	Is the roughness length, (table 2.3)
$zmin$	Is the minimum height, (table 2.3)

2.3.13 Resonance response factor R^2

The resonance response factor R^2 takes into account turbulence in resonance with the considered vibration mode of the structure and is given by equation 2.21.

$$R^2 = \frac{\pi^2}{2 \times \delta} \times Sl(zs, n) \times Ks(n) \tag{2.21}$$

Wherein:

δ	Is the logarithmic decrement of the total damping (section 2.4.16)
Sl	Is the spectra density function (section 2.4.15)
Ks	Is the size reduction factor (section 2.4.14)

2.3.14 Size reduction factor Ks

The size reduction factor Ks is given by equation 2.22 using equations 2.23.1 and 2.23.2:

$$Ks(n) = \frac{1}{1 + \sqrt{(Gy \times \emptyset y)^2 + (Gz \times \emptyset z)^2 + (\frac{2}{\pi} \times Gy \times \emptyset y \times Gz \times \emptyset z)^2}} \tag{2.22}$$

$$\emptyset y = \frac{Cy \times b \times n}{Vm(zs)} \tag{2.23.1}$$

$$\emptyset z = \frac{Cz \times h \times n}{Vm(zs)} \tag{2.23.2}$$

Wherein:

Cy	Is the decay constant in y-direction, recommended 11.5 [-]
Cz	Is the decay constant in z-direction, recommended 11.5 [-]
Gy	Is the uniform horizontal mode vibration shape, (table 2.4)
Gz	Is the parabolic vertical mode vibration shape, (table 2.4)
n	Is the natural frequency of the structure, $n = n1, x$

Table 2.4: Vibration shape modes [NEN-EN 1994-1-1-4:2005, Table C1]

Mode shape	Uniform	Linear	Parabolic	Sinusoidal
G:	1/2	3/8	5/18	$4/\pi^2$
K:	1	3/2	5/3	$4/\pi$

NOTE 1 For buildings with a uniform horizontal mode shape variation and a linear vertical mode shape variation $\Phi(y,z) = z/h$, $G_y = 1/2$, $G_z = 3/8$, $K_y = 1$ and $K_z = 3/2$.

NOTE 2 For chimneys with a uniform horizontal mode shape variation and a parabolic vertical mode shape variation $\Phi(y,z) = z^2/h^2$, $G_y = 1/2$, $G_z = 5/18$, $K_y = 1$ and $K_z = 5/3$.

NOTE 3 For bridges with a sinusoidal horizontal mode shape variation $\Phi(y,z) = \sin(\pi \cdot y/b)$, $G_y = 4/\pi^2$, $G_z = 1/2$, $K_y = 4/\pi$ and $K_z = 1$.

2.3.15 Spectra density function Sl

The spectra density function Sl is given by equation 2.24 by using formula 2.25:

$$Sl(zs,n) = \frac{6.8 \times fl(zs,n)}{(1 + 10{,}2 \times fl(zs,n))^{\frac{5}{3}}} \quad (2{,}24)$$

$$fl(zs,n) = \frac{n \times L(zs)}{vm(z)} \quad (2.25)$$

Wherein:

$Sv(z,n)$	Is the single-sided variance spectrum
n	Is the natural frequency of the structure, $n = n1, x$
L(zs)	Is the turbulence length scale (section 2.4.12)
$Vm(z)$	Is the average wind velocity (section 2.4.4)

2.3.16 Logarithmic decrement of total damping δ

The logarithmic decrement of total damping δ of the structure is given by equation 2.26 using equations 2.27, 2.28, 2.29 and table's 2.5 and 2.6:

$$\delta = \delta s + \delta a + \delta d \quad (2.26)$$

$$\delta a = \frac{Cf \times \rho \times b \times vm(zs)}{2 \times n1 \times me} \quad (2.27)$$

$$me = \frac{\int_0^l m(z) \times \Phi 1^2(z) ds}{\int_0^l \Phi 1^2(z) ds} \quad (2.28)$$

$$\Phi 1(z) = \left(\frac{z}{h}\right)^{\xi} \quad (2.29)$$

Wherein:

δs	Is the logarithmic decrement of the structural damping, table 2.5
δa	Is the logarithmic decrement of the aerodynamic damping
δd	Is the Logarithmic decrement of special damping devices, special treatment
Cf	Is the force coefficient for the structure or structural element
ρ	Is the air density, recommended value $\rho = 1.25\ kg/m^3$
b	Is the width of the building
$vm(zs)$	Is the average wind velocity
$n1$	Is the natural frequency of the structure
me	Is the equivalent mass per unit of length
m	Is the mass per unit of length
l	Is the length of the structure or structural element

Table 2.5: Logarithmic decrement of structural damping per structure type [NEN-EN 1994-1-4:2005. Table F.2]

Constructietype		Constructieve demping δ_s
Gebouwen van gewapend beton		0,10
Stalen gebouwen		0,05
Constructies uit beton en staal		0,08
Torens en schoorstenen van gewapend beton		0,03
Onbeklede gelaste stalen schoorstenen zonder uitwendige thermische isolatie		0,012
Onbeklede gelaste stalen schoorstenen met uitwendige thermische isolatie		0,020
Stalen schoorstenen met een voering met uitwendige thermische isolatie	$h/b < 18$	0,020
	$20 \leq h/b < 24$	0,040
	$h/b \geq 26$	0,014
Stalen schoorstenen met twee of meer bekledingen met uitwendige thermische isolatie [a]	$h/b < 18$	0,020
	$20 \leq h/b < 24$	0,040
	$h/b \geq 26$	0,025
Stalen schoorsteen met interne bakstenen bekleding		0,070
Stalen schoorsteen met intern spuitbeton		0,030
Gekoppelde schoorsteen zonder bekleding		0,015
Getuide stalen schoorsteen zonder bekleding		0,04
Stalen bruggen en stalen vakwerktorens	Gelast	0,02
	Bouten met hoge weerstand	0,03
	Gewone bouten	0,05
Samengestelde bruggen		0,04
Betonnen bruggen	Voorgespannen zonder scheurvorming	0,04
	Met scheurvorming	0,10
Houten bruggen		0,06 - 0,12
Bruggen, aluminium legeringen		0,02
Bruggen, glas- of vezelversterkte kunststof		0,04 - 0,08
Kabels	Parallelle kabels	0,006
	Getordeerde kabels	0,020

OPMERKING 1 De waarden voor houten en kunststofcomposieten zijn alleen indicatief. In geval de aerodynamische effecten significant worden gevonden in de berekening, zijn meer verfijnde gegevens noodzakelijk via specialistisch advies (in overeenstemming met de bevoegde overheid indien van toepassing).

OPMERKING 2 Bij hang- en tuibruggen worden de waarden gegeven in tabel F.2 vermenigvuldigd met 0,75.

[a] Voor tussenliggende waarden van h/b mag lineaire interpolatie zijn gebruikt.

Table 2.6: vibration shape factors [NEN-EN 1994-1-4:2005, Section F.3]

Value in ξ	Building type
0.6	Slender truss structure without load bearing walls
1,0	Central core surrounded with columns
1,5	Slender clamped buildings and buildings with central reinforced concrete core
2,0	Towers and chimneys
2,5	Steel truss towers

2.3.17 Wind forces Fw calculated by force coefficient method

The wind force Fw on a structure or structural element can be calculated directly by equation 2.30 or by vectorial summation given by equation 2.31:

$$Fw = CsCd \times Cf \times qp(zs) \times Aref \tag{2.30}$$

$$Fw = CsCd \times \sum_{Elements} Cf \times qp(zs) \times Aref \tag{2.31}$$

Wherein:

$CsCd$	Is the structural factor
Cf	Is the force coefficient for the structure or structural element (section 2.3.19)
$qp(zs)$	Is the peak velocity pressure at reference height (section 2.3.9)
$Aref$	Is the reference area of the surface

2.3.18 Wind forces Fw calculated by surface pressure method

The wind force Fw on a structure or structural element can be calculated by vectorial summation of the forces Fw, e, Fw, i and Ffr given respectively by formula's 2.32, 2.33 and 2.34:

$$Fw,e = CsCd \times \sum_{Surfaces} We \times Aref \tag{2.32}$$

$$Fw,i = \sum_{Surfaces} Wi \times Aref \tag{2.33}$$

$$Ffr = Cfr \times qp(ze) \times Afr \tag{2.34}$$

Wherein:

$CsCd$	Is the structural factor
We	Is the external pressure on a surface at height ze
Wi	Is the internal pressure of a surface at height zi
$Aref$	Is the reference area of the surface
Cfr	Is the friction coefficient
Afr	Is the friction area of the surface

2.3.19 Force coefficients Cf

The force coefficient Cf is mainly determined by the building shape. In several shape configurations will be analysed and therefore three different force coefficients Cf will be discussed in this section.

Rectangular

$$Cf\ (rectangular) = Cf, 0, rect\ \times\ \psi r\ \times\ \psi\lambda \tag{2.35}$$

Wherein:

$Cf, 0, rect$	Is the force coefficient for rectangular sections (figure 2.4)
ψr	Is the reduction factor (figure 2.5)
$\psi\lambda$	Is the final effect factor (figure 2.6)

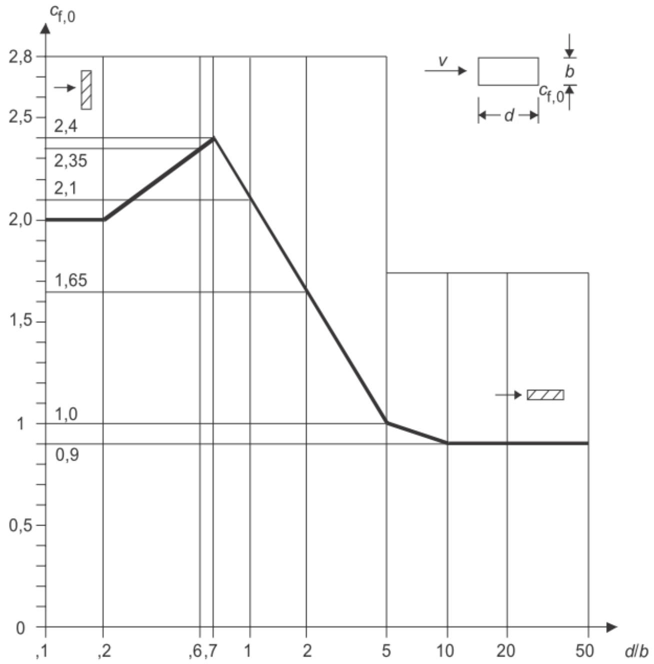


Figure 2.4: Force coefficient $Cf, 0$ for rectangular sections [NEN-EN 1994-1-4:2005, Figure 7.23]

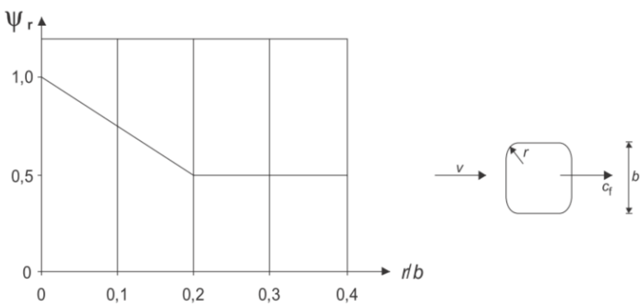


Figure 2.5: Reduction factor ψr for rectangular sections [NEN-EN 1994-1-4:2005, Figure 7.24]

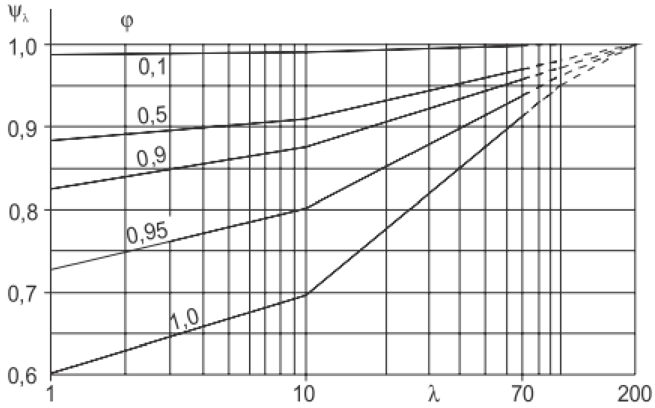


Figure 2.6: Final effect $\psi\lambda$ factor [NEN-EN 1994-1-4:2005, Figure 7.36]

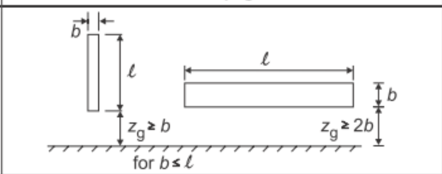
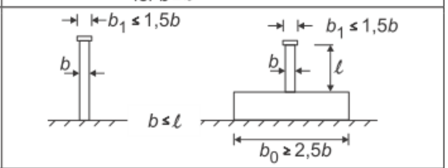
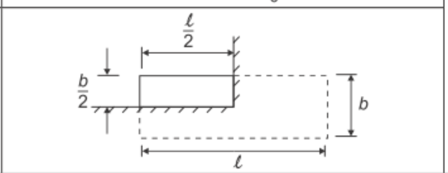
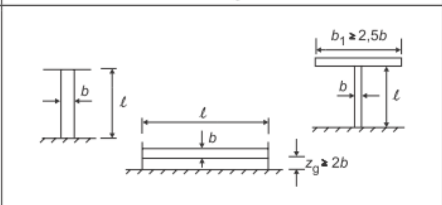
To determine the Final effect $\psi\lambda$ factor, first the effective slenderness λ of the structure will be calculated, using equation 2.36 and table 2.7

$$\varphi = \frac{A}{Ac} \tag{2.36}$$

Wherein:

A	Is the effected surface of the building
Ac	Total surface building

Table 2.7: Effective slenderness λ structures [NEN-EN 1994-1-4:2005, Table 7.16]

Nr.	Positie van de constructie, wind normaal op het vlak van de pagina	Effectieve slankheid λ
1		Voor veelhoekige, rechthoekige doorsneden met scherpe hoeken en vakwerkconstructies: voor $\ell \geq 50$ m: neem de kleinste van $\lambda = 1,4\ \ell/b$ of $\lambda = 70$,
2		voor $\ell < 15$ m: neem de kleinste van $\lambda = 2\ \ell/b$ of $\lambda = 70$. Voor cirkelvormige cilinders: voor $\ell \geq 50$ m: neem de kleinste van $\lambda = 0,7\ \ell/b$ of $\lambda = 70$,
3		voor $\ell < 15$ m: neem de kleinste van $\lambda = \ell/b$ of $\lambda = 70$. Voor tussenliggende waarden van ℓ behoort lineair te zijn geïnterpoleerd.
4		voor $\ell \geq 50$ m: neem de grootste waarde van $\lambda = 0,7\ \ell/b$ of $\lambda = 70$; voor $\ell < 15$ m: neem de grootste waarde van $\lambda = \ell/b$ of $\lambda = 70$. Voor tussenliggende waarden van ℓ behoort lineair te zijn geïnterpoleerd.

Polygonal

$$Cf\ (polygonal) = Cf, 0, poly\ \times \psi\lambda \tag{2.37}$$

Wherein:

$Cf, 0, poly$	Is the force coefficient for polygonal sections (table 2.8)
$\psi\lambda$	Is the final effect factor (figure 2.6)

Table 2.8: Force coefficient $Cf, 0$ for polygonal sections [NEN:EN 1994-1-4:2005, Table 7.11]

Aantal zijden	Doorsnede	Afwerking van het oppervlak en de hoeken	Reynoldsgetal Re [a]	$c_{f,0}$
5	Pentagonaal	Alle	Alle	1,80
6	Hexagonaal	Alle	Alle	1,60
8	Octagonaal	Glad oppervlak r/b < 0,075 [b]	$Re \leq 2{,}4 \cdot 10^5$	1,45
			$Re \geq 3 \cdot 10^5$	1,30
		Glad oppervlak $r/b \geq 0{,}075$ [b]	$Re \leq 2 \cdot 10^5$	1,30
			$Re \geq 7 \cdot 10^5$	1,10
10	Decagonaal	Alle	Alle	1,30
12	Dodecagonaal	Glad oppervlak [c] afgeronde hoeken	$2 \cdot 10^5 < Re < 1{,}2 \cdot 10^6$	0,90
		Alle andere	$Re < 4 \cdot 10^5$	1,30
			$Re > 4 \cdot 10^5$	1,10
16-18	Hexadecagonaal tot octadecagonaal	Glad oppervlak [c] afgeronde hoeken	$Re < 2 \cdot 10^5$	Behandel als cirkelvormige cilinders, zie (7.9)
			$2 \cdot 10^5 \leq Re < 1{,}2 \cdot 10^6$	0,70

[a] Reynoldsgetal met $v = v_m$ en v_m gegeven in 4.3, Re is vastgesteld in 7.9.

[b] r = hoekstraal, b = diameter van de omschreven cirkel, zie figuur 7.26.

[c] Uit windtunnelproeven op modellen met een oppervlak van gegalvaniseerd staal en een doorsnede met b = 0,3 m en hoekstraal = 0,06 b.

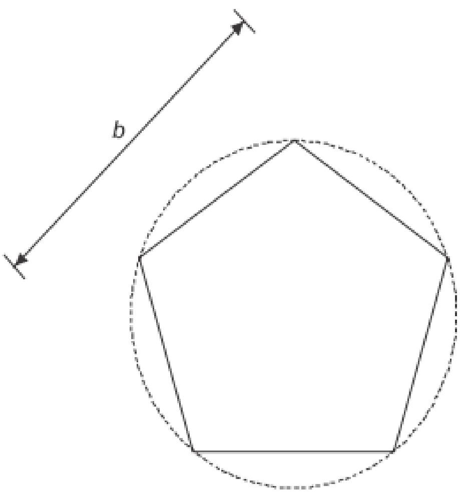


Figure 2.7: Polygonal section [NEN-EN 1994-1-4:2005, Figure 7.26]

Circular

$$Cf\ (circular) = Cf, 0, cir \ \times \ \psi\lambda \tag{2.38}$$

$$Cf, 0, cir = \frac{0{,}11}{\left(\frac{Re}{10^6}\right)^{1.4}} \tag{2.39}$$

$$Cf, 0, cir = 1{,}2 + \frac{0{,}18 \ \times \ \log(10 - \frac{k}{b})}{1 + 0{,}4 \ \times \log(\frac{Re}{10^6})} \tag{2.40}$$

$$Re = \frac{b \ \times v(ze)}{\upsilon} \tag{2.41}$$

$$v(ze) = \sqrt{\frac{2 \ \times qp}{\rho}} \tag{2.42}$$

Wherein:

$Cf, 0, cir$	Is the force coefficient for circular sections (figure 2.8)
$\psi\lambda$	Is the final effect factor (figure 2.6)
Re	Is the Reynolds number
b	Is the diameter of the circle
υ	Is the kinematic viscosity of air, $\nu = 15 \ \times 10^6 \ [m^2/s]$
qp	Is the peak velocity pressure (section 2.3.9)
k	Is the roughness

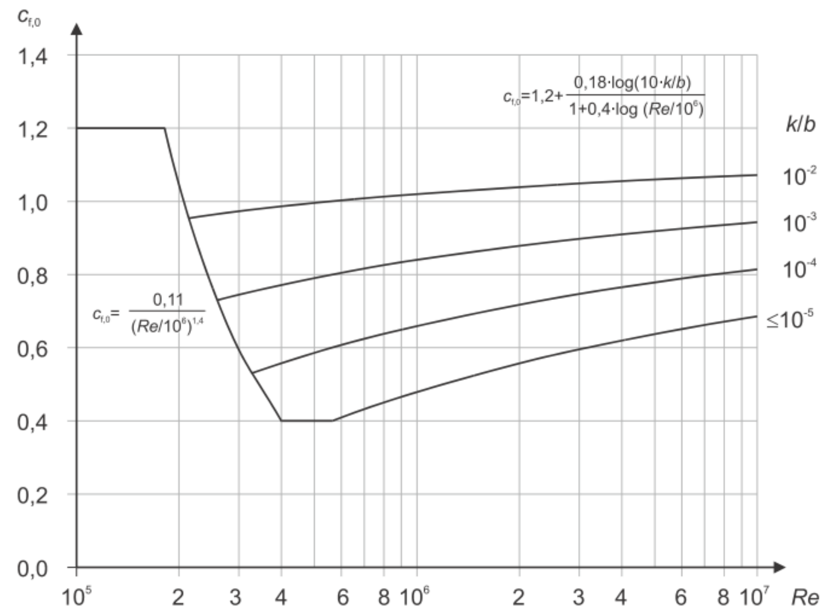


Figure 2.8: Force coefficient $Cf, 0$ for circular sections [NEN-EN 1994-1-4:2005, Figure 7.28]

Table 2.9: Roughness k [NEN-EN 1994-1-4:2005, Table 7.13]

Oppervlaktetype	Ruwheidshoogte* k mm	Oppervlaktetype	Ruwheidshoogte* k mm
Glas	0,0015	Glad beton	0,2
Gepolijst metaal	0,002	Geschaafd hout	0,5
Gladde verflaag	0,006	Ruw beton	1,0
Gespoten verflaag	0,02	Ruw hout	2,0
Blank staal	0,05	Roest	2,0
Gietijzer	0,2	Metselwerk	3,0
Gegalvaniseerd staal	0,2		

2.3.20 Vortex shedding $Vcrit, i$

Vortex-shedding occurs when vortices are shed alternately from opposite sides of the structure. Structural vibrations may occur if the frequency of vortex-shedding is the same as the natural frequency of the structure. This gives rise to a fluctuating load perpendicular to the wind direction. This condition occurs when the wind velocity is (almost) equal to the critical wind velocity. The effects of vortex shedding don't need to be investigated when equation 2.43 does not occur.

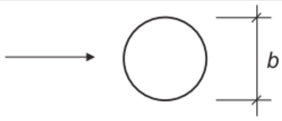
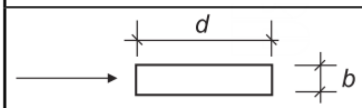
$$Vcrit, i > 1{,}25 \times vm \tag{2.43}$$

$$Vcrit, i = \frac{b \times n1, x}{St} \tag{2.44}$$

Wherein:

b Is the width of the building or outer diameter
$n1, x$ Is the natural frequency of the structure (section 2.2.11)
St Is the Strouhal number
vm Is the average wind velocity, (section 2.3.4)

Table 2.10: Strouhal numbers St [NEN-EN 1991-1-4:2005, Table E.1]

Cross-section	St
 for all Re-numbers	0,18
 $0{,}5 \leq d/b \leq 10$	from Figure E.1

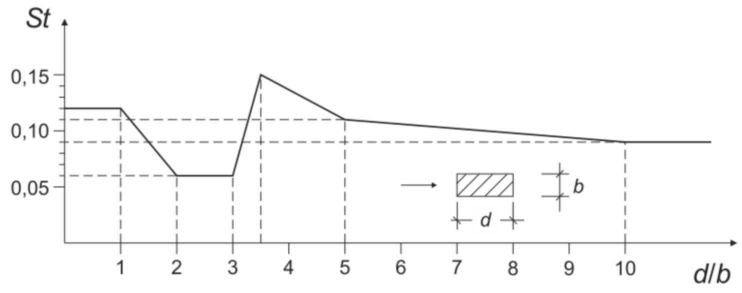


Figure 2.9 : Strouhal number St for rectangular cross-sections with sharp corners [NEN-EN 1991-1-4:2005, Figure E.1]

2.3.21 Galloping

Galloping is a self/induced vibration of a flexible structure in cross wind bending mode. Non circular-cross sections are prone to galloping. Ice may cause a stable cross section to become unstable. Galloping oscillation starts at a special onset wind velocity Vcg and normally the amplitudes increase rapidly with increasing wind velocity. Galloping is calculated with equations 2.45 and 2.46.

$$Sc = \frac{2\, x\, \delta s\ \times mi, e}{p\ \times b^2} \tag{2.45}$$

$$Vcg = \frac{2\ \times Sc}{ag} \times n1, y\ \times b \tag{2.46}$$

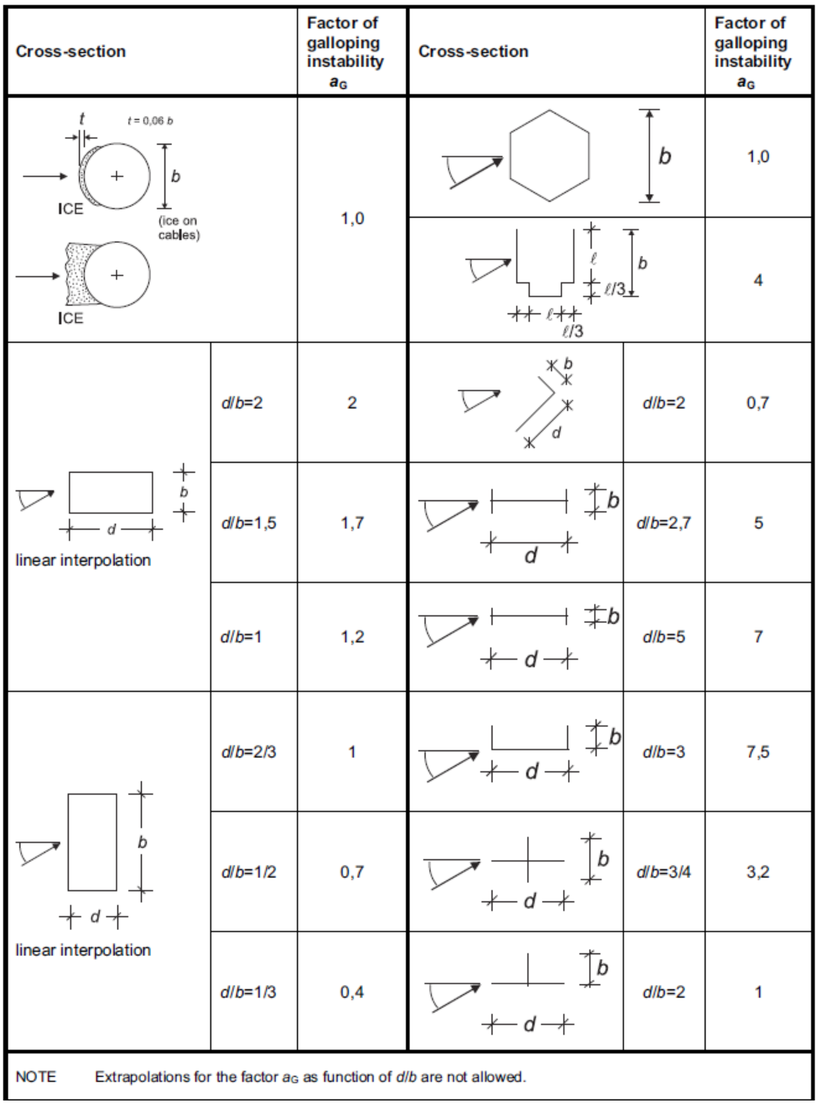
Wherein:

δs	Is the logarithmic decrement of the structural damping, (table 2.5)
ρ	Is the air density, recommended value $\rho = 1.25\ kg/m^3$
me	Is the equivalent mass per unit of length
b	Is the width of the building or outer diameter
Sc	Is the Scruton number
ag	Is the factor of galloping instability, (figure 2.10)
$n1, x$	Is the natural frequency of the structure (section 2.3.11)
vm	Is the average wind velocity, (section 2.3.4)

It should be ensured that equation 2.47 fulfils. Extra attention is required when the effects between vortex shedding and galloping is small, in case the structure is in the range of equation 2.48.

$$Vcg > 1{,}25\ \times vm \tag{2.47}$$

$$0{,}7 < \frac{Vcg}{Vcrit} < 1{,}5 \tag{2.48}$$



Cross-section		Factor of galloping instability a_G	Cross-section		Factor of galloping instability a_G
$t = 0,06\,b$ ICE (ice on cables) ICE		1,0			1,0
					4
linear interpolation	$d/b=2$	2		$d/b=2$	0,7
	$d/b=1,5$	1,7		$d/b=2,7$	5
	$d/b=1$	1,2		$d/b=5$	7
linear interpolation	$d/b=2/3$	1		$d/b=3$	7,5
	$d/b=1/2$	0,7		$d/b=3/4$	3,2
	$d/b=1/3$	0,4		$d/b=2$	1

NOTE Extrapolations for the factor a_G as function of d/b are not allowed.

Figure 2.10: Factor of galloping instability [NEN-EN 1-1:4:2005, Table E.7]

2.3.22 Peak accelerations a

The characteristic peak accelerations are obtained by multiplying the standard deviation of the characteristic along-wind acceleration by the peak factor and are given by equation 2.49 by using equations 2.50 and 2.51.

$$a = Kp \times \sigma a, x(y,z) \tag{2.49}$$

$$\sigma a, x(y,z) = Cf \times \rho \times Iv(zs) \times vm^2(zs) \times R \times \frac{Ky \times Kz \times \Phi(y,z)}{\mu ref \times \Phi max} \tag{2.50}$$

$$\frac{\Phi(y,z)}{\Phi max} = 1 \tag{2.51}$$

Wherein:

Kp	Is the peak factor (section 2.3.11)
$\sigma a, x(y,z)$	Is the standard deviation along wind acceleration
Cf	Is the force coefficient (section 2.3.19)
ρ	Is the air density, recommended value $\rho = 1.25\ kg/m^3$
$Iv(zs)$	Is the wind turbulence intensity (section 2.3.8)
vm	Is the average wind velocity, (section 2.3.4)
R	Is the square root of resonance response factor (section 2.3.13)
Ky	Is the uniform mode shape (table 2.4)
Kz	Is the Parabolic mode shape (table 2.4)
$\Phi(y,z)$	Is the vibration mode
μref	Is the reference mass per unit area at the point of largest amplitude
Φmax	Is the vibration mode at the point of largest amplitude

2.4 Summary Eurocode wind

The response of structures will be calculated with the characteristic wind load qw (equation 2.52). Below all the parameters affecting the value of qw are given.

$$qw = CsCd \times Cf \times qp(z) \tag{2.52}$$

$CsCd$ is the structural factor and is highly effected by the characteristics of the building structure. It can be divided into two parts. The first part is Cs, the size factor and takes into account the reduction effect on the wind action due to the non-simultaneity of occurrence of the peak wind pressures on the surface. The second part is Cd, the dynamic factor and takes into account the increasing effect from vibrations due to turbulence in resonance with the structure.

Cf is the force coefficient and is highly effect by the shape of the building structure. Force coefficients give the overall effect of the wind on a structure, structural element or component as a whole, including friction, if not specifically excluded.

$qp(z)$ is the peak velocity pressure at height z and is highly effected by the location of the building site (geographical). The peak velocity pressure includes mean and short-term velocity fluctuations prescribed on 50 years return period and as a 10 minutes average wind speed at a reference height of 10 meters above ground over the flat open terrain.

Table 2.11: Parameters [Hartmann, J., 2013]

Building parameters	Structural parameters	Geographical parameters
Depth	Damping devices	Air density
Diameter	Decay constants	Average time for reference wind velocity
Height	Natural frequency	Fundamental wind value
Mass	Structural damping	Kinematic viscosity
Radius	Surface roughness	Orography factor
Shape	Vibration shapes	Reference height
Width		Reference scale length
		Reference value
		Roughness length
		Season factor
		Turbulence factor
		Wind direction factor

Next to the characteristic wind load, the Eurocode also checks the vibration of the structure due to the wind action. These vibration modes are vortex-shedding, galloping and peak acceleration. When the structure does not fulfil the vibration requirements adjustments are needed.

2.5 Dutch wind code

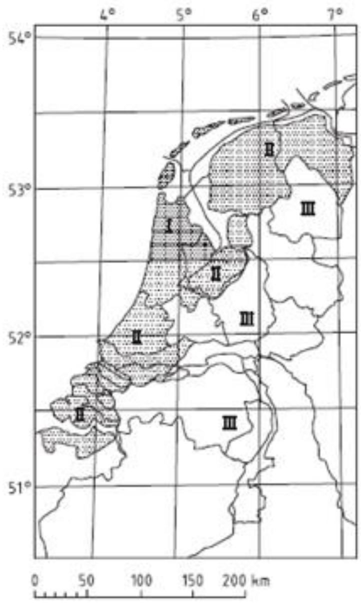
2.5.1 General information

The Dutch building code is: NEN-EN 1994-1-1-4:2005/NB:2007, Algemene belastingen – wind belasting. The Dutch code consists of two parts, the general Eurocode and the national annex.

The wind speed is prescribed on 50 years return period and as a 10 minutes average wind speed at a reference height of 10 meters above ground over the flat open terrain.

2.5.2 Main differences between the Eurocode and the Dutch national annex

Basic wind velocity $Vb,0$



Windgebied	$v_{b,0}$ m/s
I	29,5
II	27,0
III	24,5

Figure 2.11 Basic wind velocity the Netherlands [NEN-EN 1994-1-1-4:2005/NB:2007, figure NB.1]

Terrain categories

Table 2.12 Terrain categories and terrain parameters [NEN-EN 1994-1-1-4:2005/NB:2007, table 4.1]

Terreincategorie		z_0 m	z_{min} m
0	Zee of kustgebied aan zee	0,005	1
II	Onbebouwd gebied	0,2	4
III	Bebouwd gebied	0,5	7

Basic wind velocity $CsCd$

When the structural factor $CsCd$, becomes smaller than 0.85. A minimum value for $CsCd$ of 0.85 must be taken.

2.6 British/Nigerian wind code

2.6.1 General information

The British building code is: BS-EN 1994-1-1-4:2005+A1:2010, General actions – wind actions. The British code consists of two parts, the general Eurocode and the national annex.

Also here wind speed is prescribed on 50 years return period and as a 10 minutes average wind speed at a reference height of 10 meters above ground over the flat open terrain.

2.6.2 Main differences between the Eurocode and the British national annex

<u>Basic wind velocity $Vb,0$</u>

The fundamental value of the basic wind velocity $Vb,0$ should be determined with equation 2.53 using equation 2.54.

$$Vb,0 = b,vmap \ \times Calt \qquad (2.53)$$

$$Calt = 1 + \ 0{,}001 \ \times A \times \left(\frac{10}{z}\right)^{0.2} \qquad (2.54)$$

Wherein:

$b,vmap$	Is the value of the fundamental basic wind velocity (figure 2.12)
$Calt$	Is the attitude factor
A	Is the attitude of the site in metres above mean sea level
z	Is the height above ground

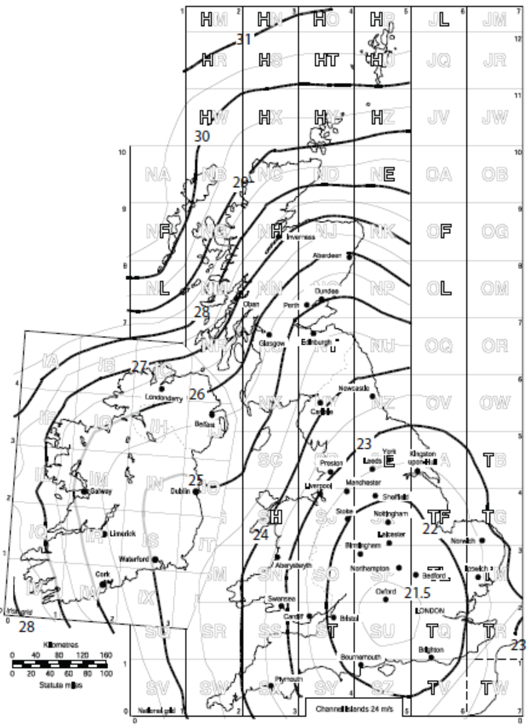


Figure 2.12: Fundamental basic wind velocity [BS EN 1991-1-1-4:2005+a1:2010, figure NA.1]

Directional factor $Cdir$

Table 2.13: Directional factor $Cdir$ [BS EN 1991-1-1-4:2005+a1:2010, table NA.1]

Direction	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°
c_{dir}	0,78	0,73	0,73	0,74	0,73	0,80	0,85	0,93	1,00	0,99	0,91	0,82

NOTE 1 Interpolation may be used within Table NA.1.

NOTE 2 The directions are defined by angles from due North in a clockwise direction.

NOTE 3 Where the wind loading on a building is assessed only for orthogonal load cases, the maximum value of the factor for the directions that lie ±45° either side of the normal to the face of the building is to be used.

NOTE 4 Conservatively, c_{dir} may be taken as 1,0 for all directions.

Roughness factor $Cr(z)$

The roughness factor $Cr(z)$ depends on upwind distance to sea and the distance upwind to the edge of the urban area like cities. Both numbers of figure 2.13 should be multiplied to attain the correct roughness factor.

$$Cr(z) = Cr \times Cr,t \qquad (2.55)$$

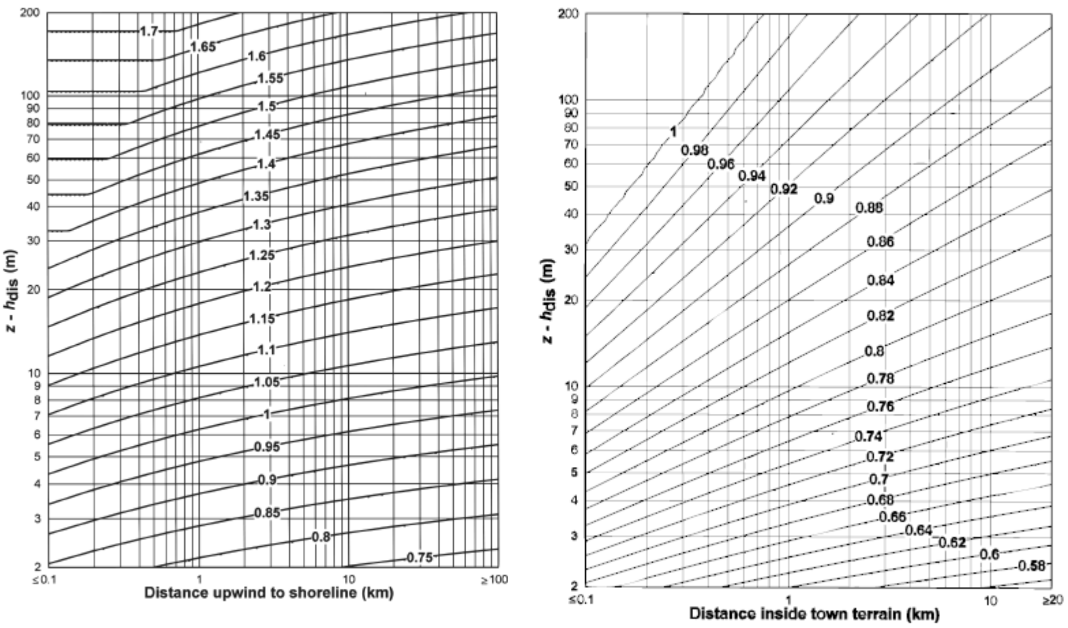


Figure 2.13: Roughness figures Cr (left) and Cr,t (right) [BS EN 1991-1-1-4:2005+a1:2010, figure NA.3 and NA.4]

Turbulence intensity $Iv(z)$

The turbulence intensity $Iv(z)$ depends on upwind distance to sea and the distance upwind to the edge of the urban area like cities. Both numbers of figure 2.14 should be multiplied to attain the correct turbulence intensity.

$$Iv(z) = Iv(z)flat \times KI,t \qquad (2.56)$$

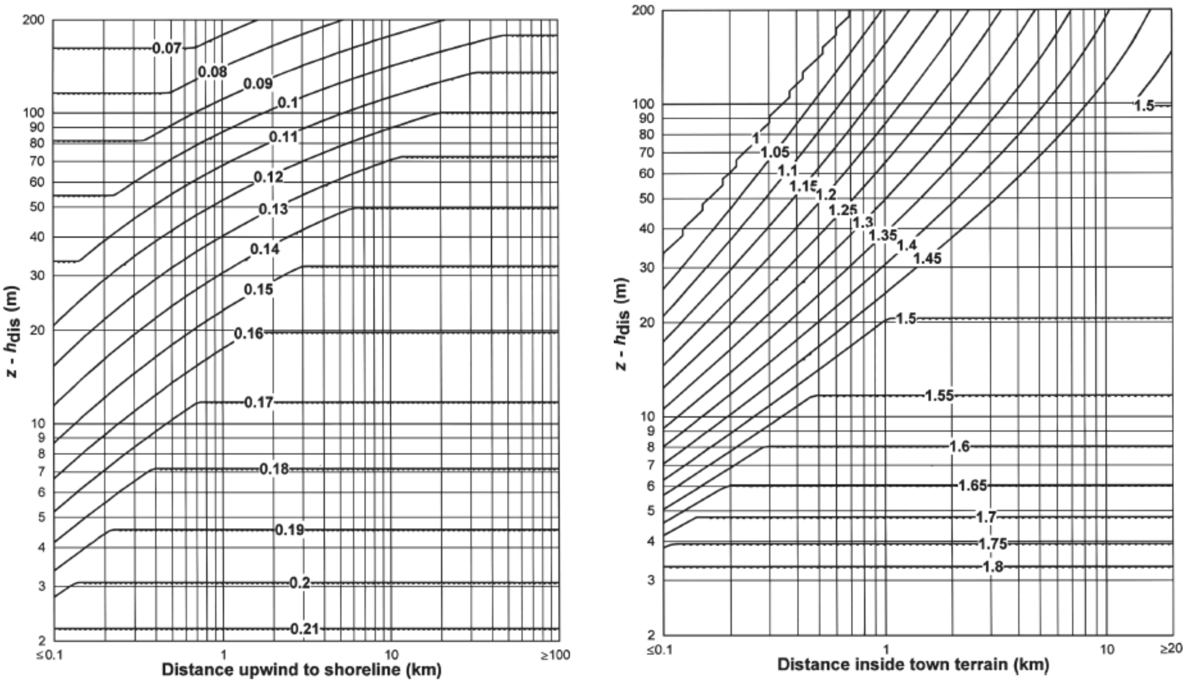


Figure 2.14: Turbulence factors $Iv(z)flat\ (left)$ and KI,t (right) [BS EN 1991-1-1-4:2005+a1:2010, figure NA.5 and NA.6]

Peak velocity pressure $qp(z)$

The peak approach (when orography factor $C0$ =1) in the British annex is determined by equations 2.56 and 2.57 using equation 2.58.

$$qp(z) = Ce(z) \times qb \qquad for\ country\ terrein \qquad (2.56)$$

$$qp(z) = Ce(z) \times Ce,t \ \times qb \qquad for\ town\ terrein \qquad (2.57)$$

$$qb = \frac{1}{2} \times \rho \ \times vb^2 \qquad (2.58)$$

Wherein:

$Ce(z)$	Is the exposure factor, figure 2.15
Ce,t	Is the exposure correction factor figure 2.15
ρ	Is the air density, recommended value is 1.226 kg/m3
vb	Is the basic wind velocity (section 2.3.3)

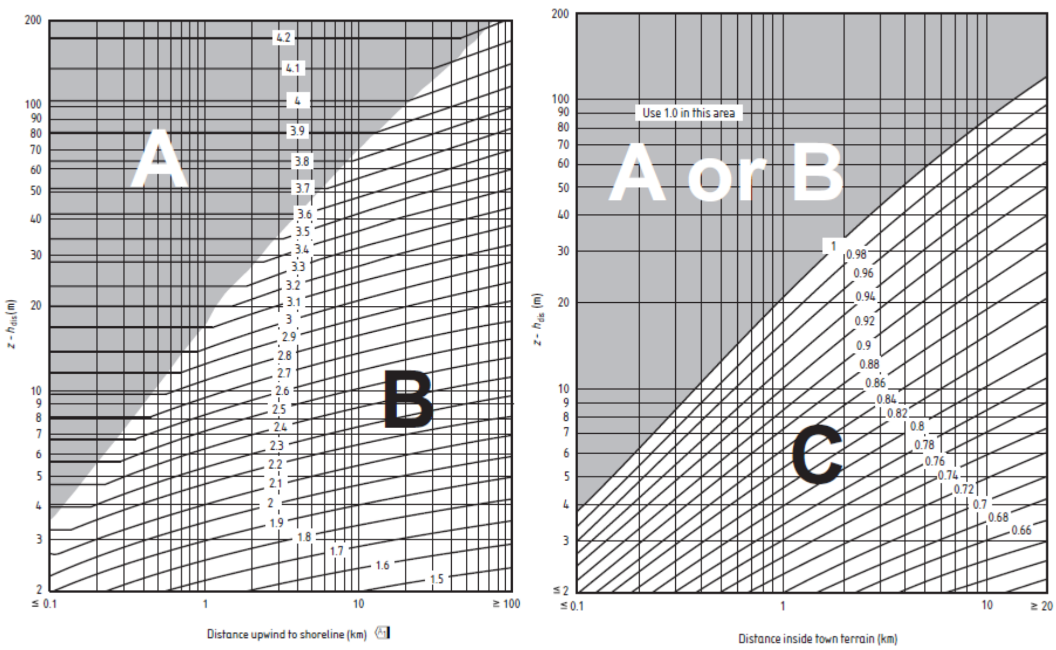


Figure 2.15: Exposure (correction) factors $Ce(z)$ $(left)$ and Ce,t $(right)$ [BS EN 1991-1-1-4:2005+a1:2010, figure NA.7 and NA.8]

Background factor B^2

Replace the background factor B^2 (section 2.4.12) in the structural factor $CsCd$ with equation 2.59.

$$B^2 = \frac{1}{1 + 0{,}9 \times \left(\frac{B+H}{L(zs)}\right)^{0{,}63}} \tag{2.59}$$

Wherein:

B	Is the height of the structure
H	Is the height of the structure
$L(zs)$	Is the turbulent length scale (section 2.3.12)

Resonance response factor R^2

Replace the resonance response factor R^2 (section 2.3.13) in the structural factor $CsCd$ with equation 2.60.

$$R^2 = \frac{\pi^2}{2 \times \delta} \times Sl(zs, n\,) \times Rh(\eta h) \times Rb(\eta b) \tag{2.60}$$

Wherein:

δ	Is the logarithmic decrement of the total damping (section 2.3.16)
Sl	Is the spectra density function (section 2.3.15)
Rh, Rb	Size reduction factor (section 2.3.14)

Effective slenderness factor λ

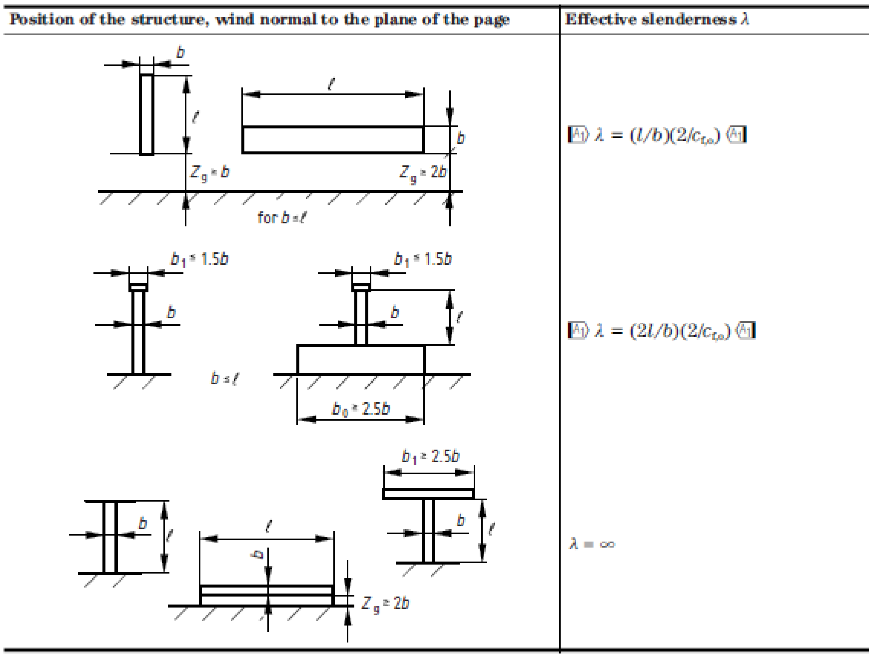


Figure 2.16: Effective slenderness λ structure [BS EN 1991-1-1-4:2005+a1:2010, figure NA.10]

Peak acceleration α

The peak acceleration α in the British annex is determined with equation 2.61.

$$\sigma a, x(z) = \frac{Cf \times \rho \times Iv(zs) \times vm^2(zs)}{m1, x} \times R \times Kx \times \Phi 1(z) \tag{2.61}$$

Cf	Is the force coefficient (section 2.3.19)
ρ	Is the air density, recommended value $\rho = 1.226\ kg/m^3$
$Iv(zs)$	Is the wind turbulence intensity (section 2.3.8)
vm	Is the average wind velocity, (section 2.3.4)
R	Is the square root of resonance response factor (equation 2.59)
Kx	is the non-dimensional coefficient (figure 2.17)
$m1, x$	Is the along fundamental equivalent mass
$\Phi 1(z)$	Is the fundamental along wind modal shape

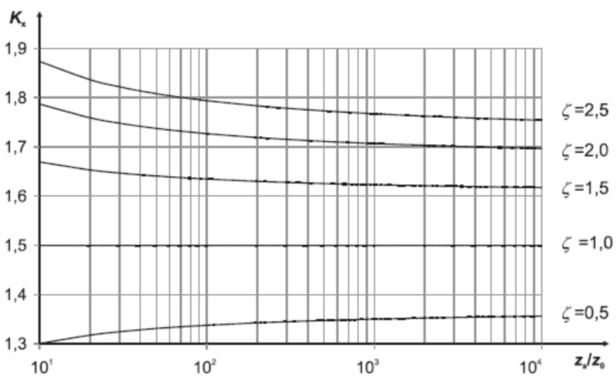


Figure 2.17: Non dimensional coefficient Kx [BS EN 1991-1-1-4:2005+a1:2010, figure B.4]

2.7 Chinese wind code

2.7.1 General information

The Chinese building code is: GB 50009-2001, load code for the design of building structures.

The wind speed is prescribed on 50 years return period and as a 10 minutes average wind speed at a reference height of 10 meters above ground over the flat open terrain.

2.7.2 Characteristic wind load wk

The wind loading in China is defined as a force over unit area for buildings and the characteristic value of wind load vertical to building surfaces shall be calculated with the following equation:

$$wk = \beta z \times \mu s \times \mu z \times w0 \tag{2.62}$$

Wherein:

βz	Is the dynamic wind effect factor (section 2.7.6)
μs	Is the shape factor of wind load (section 2.7.5)
μz	Is the exposure factor for wind pressure (section 2.7.4)
$w0$	Is the reference wind pressure (section 2.7.3)

2.7.3 Reference wind pressure $w0$

The reference wind pressure is determined from the basic wind speed and the air density.

$$w0 = \frac{1}{2} \times \rho \times v0^2 \tag{2.63}$$

$$\rho = 0.001225 e^{-0.0001 \times z} \tag{2.64}$$

Wherein:

ρ	Is the air density
c$v0$	Is the basic wind speed, figure 2.18

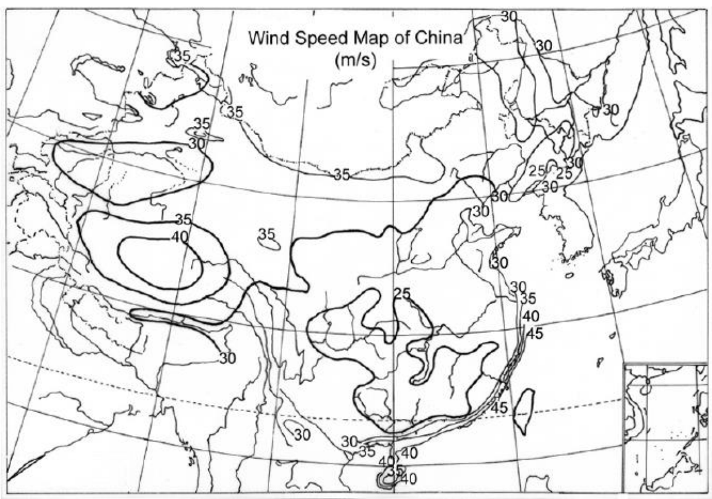


Figure 2.18: Wind pressure map of China [Yaojun, G., Xinyang, J., 2004]

The wind pressure map of China is made on basis of statically records made in more than 350 stations with an average year wind speed of 35 – 40 m/s. On this map there are two strong wind areas located. The southeast coast of China, in which the maximum wind speed is mainly caused by typhoons. The other area is in northwest of China, where the maximum wind speed is basically influenced by cold storms.

2.7.4 Exposure factor μz

The exposure factor for wind shall be determined on the basis of the categories of terrain roughness according to equations 2.65 to 2.68 or table 2.15 for structure height z.

Table 2.14: Terrain roughness category [GB 50009-2001, section 7.6.2]

Category	Terrain	Value α
A	shore sea surfaces, islands, sea shores, lake shores and deserts	0.12
B	open fields, villages, forests, hills, sparsely-populated towns and city suburbs	0.16
C	urban districts in densely-populated cities	0.22
D	Densely-populated cities with high building urban districts	0.30

$$\mu z^A = 1.379 \times \left(\frac{z}{10}\right)^{2\times\alpha} \tag{2.65}$$

$$\mu z^B = 1.000 \times \left(\frac{z}{10}\right)^{2\times\alpha} \tag{2.66}$$

$$\mu z^C = 0.616 \times \left(\frac{z}{10}\right)^{2\times\alpha} \tag{2.67}$$

$$\mu z^D = 0.318 \times \left(\frac{z}{10}\right)^{2\times\alpha} \tag{2.68}$$

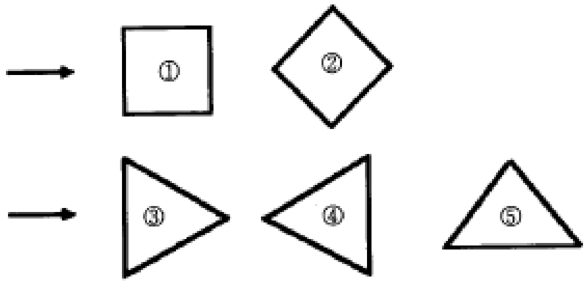
Table 2.15: Exposure factor μz [GB 50009-2001, table 7.2.1]

Height above terrain or sea level (m)	Terrain roughness categories			
	A	B	C	D
5	1.17	1.00	0.74	0.62
10	1.38	1.00	0.74	0.62
15	1.52	1.14	0.74	0.62
20	1.63	1.25	0.84	0.62
30	1.80	1.42	1.00	0.62
40	1.92	1.56	1.13	0.73
50	2.03	1.67	1.25	0.84
60	2.12	1.77	1.35	0.93
70	2.20	1.86	1.45	1.02
80	2.27	1.95	1.54	1.11
90	2.34	2.02	1.62	1.19
100	2.40	2.09	1.70	1.27
150	2.64	2.38	2.03	1.61
200	2.83	2.61	2.30	1.92
250	2.99	2.80	2.54	2.19
300	3.12	2.97	2.75	2.45
350	3.12	3.12	2.94	2.68
400	3.12	3.12	3.12	2.91
≥450	3.12	3.12	3.12	3.12

2.7.5 Shape factor μs

The shape factor μs for the wind load can be adopted according to the following configurations, presented in tables 2.16 and 2.17. The shape factors for important and complicated shaped buildings or structural constructions shall be determined by the wind tunnel test.

Table 2.16: Rectangular and triangular shape factors μs [GB 50009-2001, table 7.3.1]



(*a*) Shape factor μ_s for integrated calculating the angle steel tower frame

wind shielding coefficient	square			triangular wind direction ③④⑤
	wind direction ①	wind direction ②		
		single angle steel	combined angle steel	
≤0.1	2.6	2.9	3.1	2.4
0.2	2.4	2.7	2.9	2.2
0.3	2.2	2.4	2.7	2.0
0.4	2.0	2.2	2.4	1.8
0.5	1.9	1.9	2.0	1.6

(*b*) Shape factor μ_s for integrated calculating the pipe and round steel tower frame

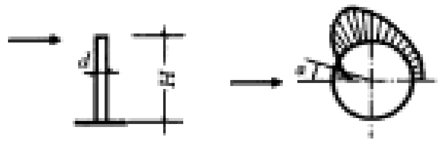
When $\mu_s w_0 d^2 \leqslant 0.002$, the value μ_s of angle steel tower frame multiplied by 0.8 can be used as μ_s;

When $\mu_s w_0 d^2 \geqslant 0.015$, the value μ_s of angle steel tower frame multiplied by 0.6 can be used as μ_s;

For medium value, calculated by interpolation

Table 2.17: Circular shape factors μs [GB 50009-2001, table 7.3.1]

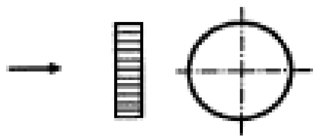
(a) For local calculation, the shape factors μ_s distributed around the surface



	$H/d \geqslant 25$	$H/d = 7$	$H/d = 1$
0°	+1.0	+1.0	+1.0
15°	+0.8	+0.8	+0.8
30°	+0.1	+0.1	+0.1
45°	−0.9	−0.8	−0.7
60°	−1.9	−1.7	−1.2
75°	−2.5	−2.2	−1.5
90°	−2.6	−2.2	−1.7
105°	−1.9	−1.7	−1.2
120°	−0.9	−0.8	−0.7
135°	−0.7	−0.6	−0.5
150°	−0.6	−0.5	−0.4
165°	−0.6	−0.5	−0.4
180°	−0.6	−0.5	−0.4

The values given above are applicable to the smooth surface with $\mu_z w_0 d^2 \geqslant 0.015$, where w_0 is measured by kN/m^2, and d is measured by m

(b) The shape factors μ_s for integrated calculating



$\mu_z w_0 d^2$	Surface condition	$H/d \geqslant 25$	$H/d = 7$	$H/d = 1$
$\geqslant 0.015$	$\Delta \approx 0$ $\Delta = 0.02d$ $\Delta = 0.08d$	0.6 0.9 1.2	0.5 0.8 1.0	0.5 0.7 0.8
$\leqslant 0.002$		1.2	0.8	0.7

2.7.6 Dynamic wind effect factor βz

For buildings with a natural fundamental frequency greater than 0.25 seconds, as well as buildings with the height greater than 30 meters and height-width ratio greater than 1.5, in along wind direction, the wind excitation due to the fluctuation effect of wind pressure shall be considered. This effect will be calculated according the following equation.

$$\beta z = 1 + \frac{\xi \times \upsilon \times \varphi z}{\mu z} \tag{2.68}$$

Wherein:

ξ	Is the magnification factor of wind fluctuation (table 2.18)
υ	Is the wind fluctuation factor (table 2.19)
φz	Is the vibration mode factor
μz	Is the exposure factor for wind pressure (section 2.7.4)
$T1$	Is the fundamental natural period of vibration for the structure

Table 2.18: magnification wind fluctuation ξ [GB 50009-2001, table 7.4.3]

$w_0 T_1^2$ (kNs²/m²)	0.01	0.02	0.04	0.06	0.08	0.10	0.20	0.40	0.60
Steel structure	1.47	1.57	1.69	1.77	1.83	1.88	2.04	2.24	2.36
Steel structure buildings with filler wall	1.26	1.32	1.39	1.44	1.47	1.50	1.61	1.73	1.81
Concrete, masonary structures	1.11	1.14	1.17	1.19	1.21	1.23	1.28	1.34	1.38
$w_0 T_1^2$ (kNs²/m²)	0.80	1.00	2.00	4.00	6.00	8.00	10.00	20.00	30.00
Steel structure	2.46	2.53	2.80	3.09	3.28	3.42	3.54	3.91	4.14
Steel structure buildings with filler wall	1.88	1.93	2.10	2.30	2.43	2.52	2.60	2.85	3.01
Concrete, masonary structures	1.42	1.44	1.54	1.65	1.72	1.77	1.82	1.96	2.06

For structures which having a large height/width ratio the wind fluctuation factor υ is determined by table 2.19.

Table 2.19: wind fluctuation factor ν [GB 50009-2001, table 7.4.4-1]

Total height H(m)		10	20	30	40	50	60	70	80
Categories of terrain roughness	A	0.78	0.83	0.86	0.87	0.88	0.89	0.89	0.89
	B	0.72	0.79	0.83	0.85	0.87	0.88	0.89	0.89
	C	0.64	0.73	0.78	0.82	0.85	0.87	0.88	0.90
	D	0.53	0.65	0.72	0.77	0.81	0.84	0.87	0.89

Total height H(m)		90	100	150	200	250	300	350	400	450
Categories of terrain roughness	A	0.89	0.89	0.87	0.84	0.82	0.79	0.79	0.79	0.79
	B	0.90	0.90	0.89	0.88	0.86	0.84	0.83	0.83	0.83
	C	0.91	0.91	0.93	0.93	0.92	0.91	0.90	0.89	0.91
	D	0.91	0.92	0.97	1.00	1.01	1.01	1.01	1.00	1.00

2.7.7 Vortex shedding

For structures with round sections, vortex shedding occurrence must be checked. The checking shall be carried out on the basis of the different Reynolds number conditions. The Reynolds number can be determined according equation 2.69.

$$Re = 69000 \times vz \times D \tag{2.69}$$

When $Re < 3 \times 10^5$ (gentle breeze resonance of sub-critical), the wind velocity at the top of the structure vh may not exceed the critical wind velocity vcr, given by equations 2.70 and 2.71. Otherwise precautions against vibration must be taken.

$$vcr = \frac{D}{T1 \times St} \tag{2.70}$$

$$vz = \sqrt{\frac{2000 \times \gamma w \times \mu z \times w0}{\rho}} \tag{2.71}$$

Wherein:

D	Is the diameter of the structure
St	Is the Strouhal number, recommended value 0.2
$T1$	Is the fundamental natural period of vibration for the structure
γw	Is the partial safety factor of wind loading, recommended value 1,4
μz	Is the exposure factor (section 2.7.4)
$w0$	Is the reference wind pressure (section 2.7.3)

When Re $\geq 3.5 \times 10^6$ (strong breeze resonance of trans-critical) and $vh > vcr$, the load effects caused by cross wind direction wind load shall be considered with equations 2.72 and 2.73.

$$Wc = \frac{|\lambda| \times vcr^2 \times \varphi z}{12800 \times \xi} \tag{2.72}$$

$$\frac{H1}{H} = \left(\frac{vcr}{vz}\right)^{\frac{1}{\alpha}} \tag{2.73}$$

Wherein:

λ	Is the calculating coefficient (table 2.20)
φz	Is the vibration mode factor
vcr	Is the crital wind speed of vortec-shedding vibration
vz	Is the wind velocity at height z
ξ	Is the structural damping ratio, recommended values for steel 0.02 and concrete 0.05
H1/H	Is the height ratio
α	Is the index of the terrain roughness (table 2.14)

Table 2.20: Table for calculation λ [GB 50009-2001, table 7.6.2]

Type of structures	Number of vibration mode	H_1/H										
		0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
High-rise structures	1	1.56	1.55	1.54	1.49	1.42	1.31	1.15	0.94	0.68	0.37	0
	2	0.83	0.82	0.76	0.60	0.37	0.09	−0.16	−0.33	−0.38	−0.27	0
	3	0.52	0.48	0.32	0.06	−0.19	−0.30	−0.21	0.00	0.20	0.23	0
	4	0.30	0.33	0.02	−0.20	−0.23	0.03	0.16	0.15	−0.05	−0.18	0
Tall buildings	1	1.56	1.56	1.54	1.49	1.41	1.28	1.12	0.91	0.65	0.35	0
	2	0.73	0.72	0.63	0.45	0.19	−0.11	−0.36	−0.52	−0.53	−0.36	0

2.8 Japanese wind code

2.8.1 General information

The Japanese building codes are AIJ-RLB-2004 and the Building Standard Law of Japan (BSLJ-2000)

The design of buildings in Japan is based on the Building Standard Law of Japan (BSLJ), which specifies the minimum building design requirements. However BSLJ does not specify the wind loads and therefore the building code AIJ-RLB exist. It is not a formal national standard law, but it is used by designers to require more sophisticated building designs or compensating parts how are not covered by the BSLJ. When a building is constructed in Japan, the building owner has to submit the plan to the local government for approval. Is the building is higher than 60m the plan has to be approved by the ministry of Land, Infrastructure and Transport. Structural designers of tall buildings can use both building codes, but it is common that they use the more comprehensive AIJ-RLB code.

2.8.2 BSLJ-2000

In table 2.21 the required performance and design wind load levels are presented. Two recurrence periods are presented. The first, 50 years, represents medium wind levels and the most important performance requirements for structures are the next two occurrences, no damage and cladding must not fall down. No damage means that the stresses in the main structure are less than the yield stress of the material.

The second period, 500 years, takes into account strong wind levels, due to typhoons. The performance requirement is in this case much higher. The building may never collapse, which means that the stresses of the main structure are less than 1.1 times the yield stress of the material.

Also the buildings are divided into 2 height regions, higher requirements are demanded for buildings higher than 60 meters. Next to along wind load, also crosswind loads and torsional loads have to be taken into account, presented in table 2.21.

Table 2.21: [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Wind Load Levels		Medium-level Winds	Strongest-level Winds
Recurrence Period		50 years	500 years
Required Performance		- No damage to main frames - Cladding does not fall down	Buildings never collapse
$H \leq 60$m	Design Methods	- Allowable Stress Method - Limit Strength Method	Limit Strength Method
	Load Factor	1	1.6
	Wind Loads	Along-wind shall be checked.	
$H > 60$m	Design Method	Dynamic response analyses in time domain	
	Wind Speed Factor	1	1.25
	Wind Loads	Along-wind, crosswind, torsional, and vertical loads shall be checked.	

Wind load Wf

The wind load for BSLJ-2000 is based on a 50 years occurrence with a 10 minute mean wind speed at a reference height of 10 above ground over open flat terrain. The level of wind load Wf is calculated by equation 2.74.

$$Wf = 0.6 \times Er^2 \times Gf \times V0^2 \times Cf \qquad (2.74)$$

Wherein:

Er	Is the vertical distribution coefficient for mean wind speed
Gf	Is the gust loading factor
Vo	Is the datum wind speed (figure 2.19)
Cf	Is the wind force coefficient

Datum wind speed $V0$

The datum wind speed of Japan is presented in figure 2.19, varying between 30 m/s and 46 m/s. Design wind load for no damage (return period of 50 years) is prescribed on these datum wind speed. The design wind load for no collapse (return period of 500 years) is prescribed as $1.6 \times$ $no\ damage\ design\ wind\ load$.

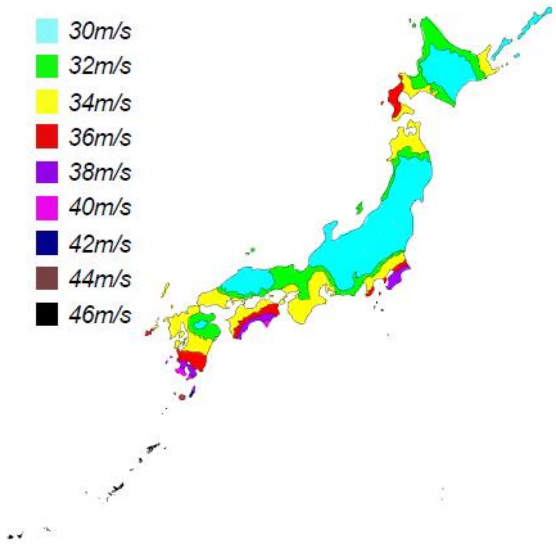


Figure 2.19: Map of datum wind speed Vo Japan [Okada, H., Okuda, Y., Kikitsu, H., 2001]

Vertical distribution coefficient for mean wind speed Er

The vertical distribution coefficient for mean wind speed Er is given with the equation

$$Er = 1.7 \times \left(\frac{Zb}{Zg}\right)^{\alpha} \qquad for\ H \leq Zb \tag{2.75}$$

$$Er = 1.7 \times \left(\frac{H}{Zg}\right)^{\alpha} \qquad for\ H > Zb \tag{2.76}$$

Wherein:

H	Is the height of the roof
Zb	(table 2.22)
Zb	(table 2.22)
α	(table 2.22)

Table 2.22: Parameters of Er [Okada, H., Okuda, Y., Kikitsu, H., 2001]

Terrain category	Z_b (m)	$Z_G(m)$	α
I	5	250	0.10
II	5	350	0.15
III	5	450	0.20
IV	10	550	0.27

Table 2.23: Terrain categories Japan [Okuda, Y., 2010]

Category	Terrain
I	Open sea, lakes or unobstructed coastal areas on land
II	Terrain wit scattered obstructions up to 10m
III	Closely spaced obstructions up to 10m
IV	Local central cities with 4-9 story buildings

Gust loading factor Gf

Table 2.24: Gust loading factors [Okada, H., Okuda, Y., Kikitsu, H., 2001]

Terrain category	Mean height of roof		
	H≤10m	10<H<40m	40m≤H
I	2.0	Linear interpolation	1.8
II	2.2		2.0
III	2.5		2.1
IV	3.1		2.3

Wind force coefficient Cf

The wind force coefficient Cf is determined by the external and internal wind pressure coefficients and is given with the equation 2.77.

$$Cf = Cpe - Cpi \tag{2.77}$$

Cpi Is the internal wind pressure coefficient

Cpe Is the external wind pressure coefficient (closed building 0, -0.2)

2.8.3 AIJ-RLB-2004

<u>The design wind speed UH</u>

The design wind speed UH for AIJ-RLB-2004 is based on basic wind speed $U0$ with 100 years occurrence with a 10 minute mean wind speed at a reference height of 10m above ground over open flat terrain. The level of wind load UH is calculated by equation 2.78.

$$UH = U0 \times KD \times EH \times Krw \qquad (2.78)$$

Wherein:

$U0$	Is the basic wind speed
KD	Is the wind directionality factor
EH	Is the wind speed profile factor
Krw	Is the return period conversion factor

<u>The basis wind $U0$</u>

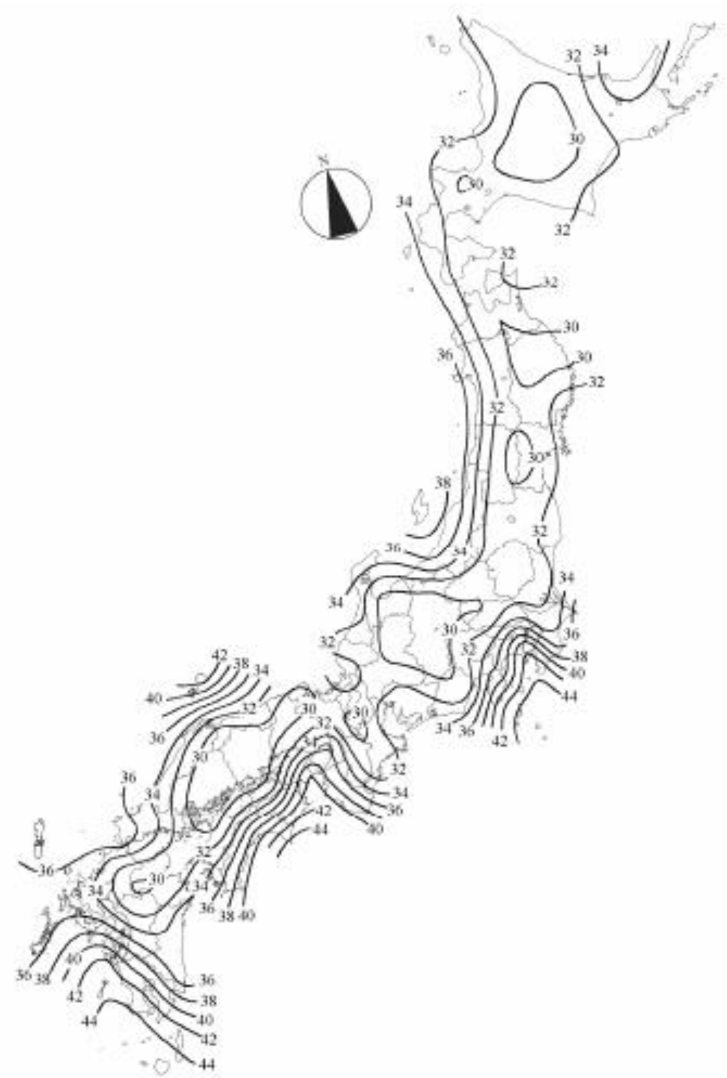


Figure 2.20: Basic wind speed Japan, UO [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Wind directionality factor KD

The building orientation can play an economic role in the structural design of a building. The building can be oriented in such a way that the pressure/force coefficients can be taken smaller for the wind certain directions. If no site information is available, take a wind directionality factor with value 1.

Table 2.25: Example of the wind directionality factor [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Direction	Sapporo	Tokyo	Kyoto	Osaka	Fukuoka
NE	0.85	0.85	1	0.9	0.85
E	0.85	0.85	0.95	0.85	0.85
SE	1	0.85	0.85	0.85	1
S	1	0.85	0.85	1	1
SW	0.85	0.85	0.85	1	0.85
W	0.95	0.85	0.85	1	1
NW	1	1	0.95	1	1
N	0.85	1	0.95	1	1

Wind speed profile factor E

The wind speed profile factor E takes into account the change in wind speed with the height, topographical features and surface roughness and is given with the equations. The topography factor is taken as value 1.0, because Air Traffic Control Towers will not be built near mountains.

$$E = Er \times Eg \tag{2.79}$$

$$Er = 1.7 \times \left(\frac{H}{Zg}\right)^{\alpha} \qquad for\ Zb < H \leq Zb \tag{2.80}$$

$$Er = 1.7 \times \left(\frac{zb}{Zg}\right)^{\alpha} \qquad for\ H \leq Zb \tag{2.81}$$

Wherein:

H	Is the height of the roof
Eg	Is the topography factor, taken as 1.0
Zb	(table 2.26)
Zb	(table 2.26)
α	(table 2.26)

Table 2.26: Parameters for exposure factor [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Flat Terrain Subcategories	I	II	III	IV	V
Z_b (m)	5	5	10	20	30
Z_G (m)	250	350	450	550	650
α	0.1	0.15	0.2	0.27	0.35

Return period conversion factor

The return period conversion factor krw is given with following equations.

$$Krw = 0.62\ \times (\lambda u - 1) \times \ln r - 2.9\ \lambda u + 3.9 \tag{2.82}$$

$$\lambda u = \frac{U500}{U0} \tag{2.83}$$

Wherein:

$U500$	Is the 500-year-accurence wind speed (figure 2.21)
$U0$	Is the basic wind speed (figure 2.20)
r	Is the r-year return period

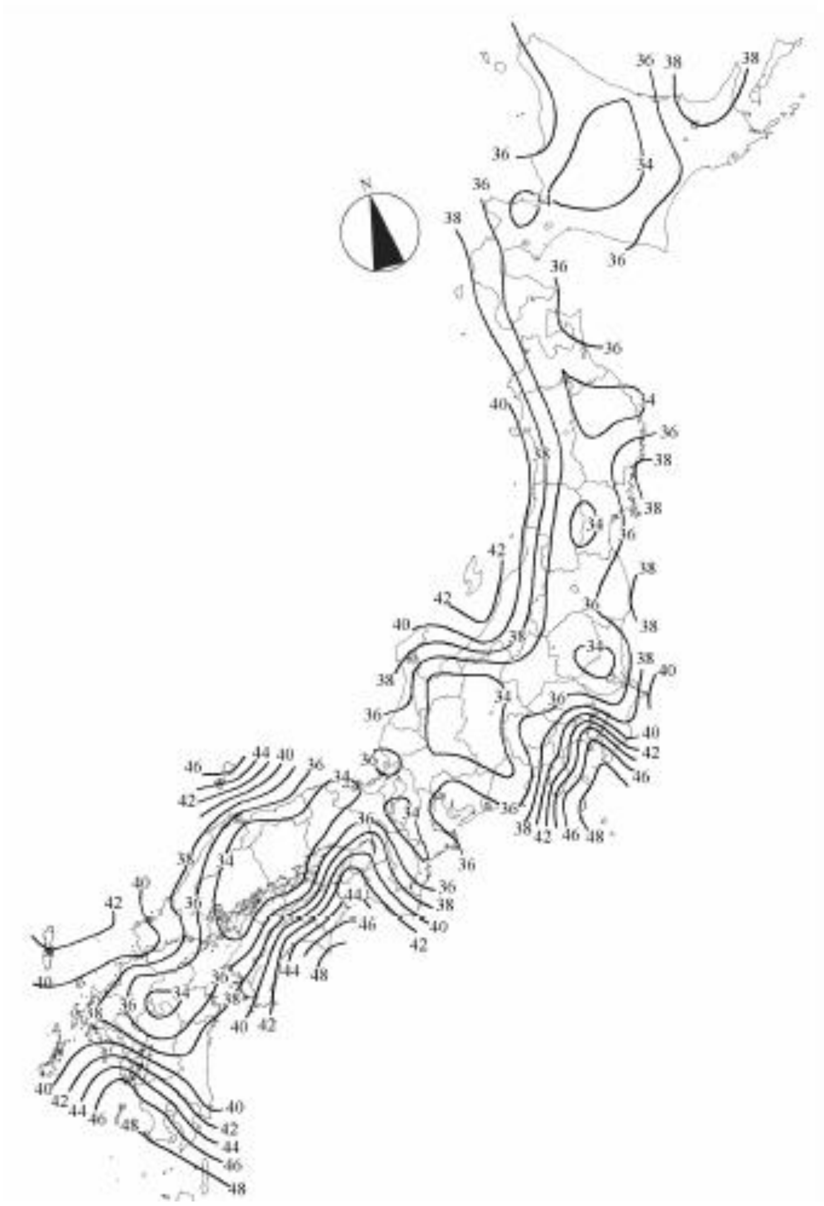


Figure 2.21: Basic wind speed Japan, $U500$ [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Along-wind loads WD

The along-wind load $WD$is calculated using the AIJ-RLB-2004 following equations.

$$WD = qh \times CD \times GD \times A \tag{2.84}$$

$$qh = \frac{1}{2} \times \rho \times UH^2 \tag{2.85}$$

$$GD = 1 + gd \, \frac{C'g}{Cg} \sqrt{1 + \emptyset d^2 Rd} \tag{2.86}$$

$$\phi d = \frac{1 - 0.4 \ln \beta}{2 + \beta} \frac{M}{Md} \tag{2.87}$$

Wherein:

Uh	Is the design wind speed
CD	Is the aerodynamic factor
GD	Is the BBM-based gust loading factor
A	Is the projected area
gd	Is the peak factor
$C'g$	Is the fluctuating coefficient along wind overturning moment
Cg	Is the mean coefficient along wind overturning moment
$\emptyset d^2$	Is the correction factor
Rd	Is the resonance factor
M	Is the total building mass excluding underground part
Md	Is the generalized mass of fundamental mode

Cross wind loads WL and torsional loads WT

The crosswind load and torsional win loads should be taken into account in design of slender and flexible buildings that fulfil the following equation:

$$\frac{H}{\sqrt{B \times D}} \geq 3 \tag{2.88}$$

H	Is the building height
B	Is the building width
D	Is the building depth

Cross wind loads WL

The crosswind load Wl at height z is given by the equation 2.89 using 2.90 and 2.91.

$$WL = 3 \times qh \times C'l \times A \times \frac{Z}{H} \times gl \times \sqrt{1 + \emptyset l^2 \times Rl} \tag{2.89}$$

$$C'L = 0.0082 \times (\frac{D}{B})^3 - 0.071 \times (\frac{D}{B})^2 + 0.22 \times \left(\frac{D}{B}\right) \tag{2.90}$$

$$\emptyset L = \frac{M}{3ML} \times \left(\frac{Z}{H}\right)^{\beta - 1} \times (1 - 0.4 \times \ln \beta) \tag{2.91}$$

Torsional loads WT

The torsional load WT at height z is given by the equation 2.92 using 2.93 and 2.94.

$$WT = 1.8 \times qh \times C't \times AB \times \frac{Z}{H} \times gt \times \sqrt{1 + \emptyset r^2 \times Rt} \tag{2.92}$$

$$C'T = \left\{0.0066 + 0.015 \times \left(\frac{Z}{H}\right)^2\right\}^{0.78} \tag{2.93}$$

$$\emptyset T = \frac{M(B^2 + D^2)}{36It} \times \left(\frac{Z}{H}\right)^{\beta-1} \times (1 - 0.4 \times \ln\beta) \tag{2.94}$$

Wind load combinations

For the calculation of the total wind load, 3 load combinations can be considered. The value ρlt is the correlation coefficient between the cross wind response and the torsional response. This value it tabulated in AIJ-RLB-2004.

Table 2.27: Basic wind speed Japan [Tamura, Y., Kawai, H., Uematsu, Y., Okada, H., Ohkuma, T., 2004]

Combination	Along-wind Load	Crosswind Load	Torsional Load
1	W_D	$0.4W_L$	$0.4W_T$
2	$W_D\left(0.4+\frac{0.6}{G_D}\right)$	W_L	$\left(\sqrt{2+2\rho_{LT}}-1\right)W_T$
3	$W_D\left(0.4+\frac{0.6}{G_D}\right)$	$\left(\sqrt{2+2\rho_{LT}}-1\right)W_L$	W_T

Vortex resonance

Vortex resonance should be examined for structures which fulfils with the following equations. Equations 2.95 is the condition for rectangular buildings and equation 2.96 for circular structures.

$$\frac{H}{\sqrt{B \times D}} \geq 4 \; and \; \left(\frac{Uh}{fl\sqrt{B \times D}} \geq 0.83 \times U^*ler \quad or \quad \frac{Uh}{ft\sqrt{B \times D}} \geq 0.83 \times U^*ter\right) \tag{2.95}$$

$$\frac{H}{Dm} \geq 7 \quad and \quad \frac{Uh}{flDm} \geq 4.2 \tag{2.96}$$

Human perception building vibration AIJ-GBV

In Japan an evaluation of habitability to wind-induced horizontal vibrations is made. The AIJ-GBV-2004 made a curve table, with in total 5 groups. H-10, H-30, H-50, H-70 and H-90. The figures of each curve indicate the perception probability as a percentage. E.g. 30 % of the people can perceive the vibration specified by the H-30 curve. Structural engineers and clients can select their design criteria with the perception design curve, presented in figure 2.22. Normally at value of H-2 / H-3 (2 and 3 %) is taken as the standard level for residential and office buildings.

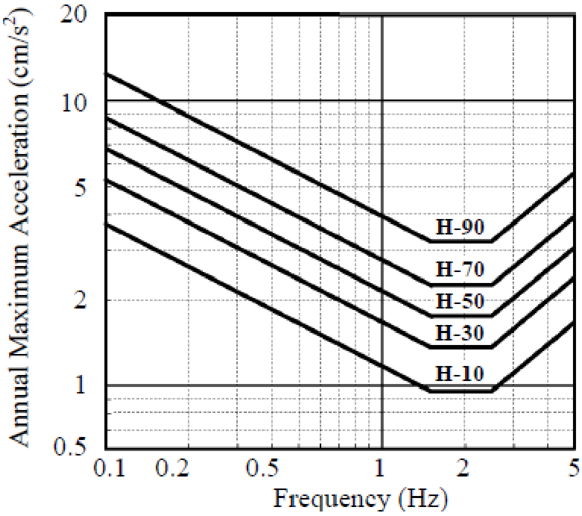


Figure 2.22: Perception design curve.

2.9 Indonesian wind code

2.9.1 General information

The Indonesian building code is: SNI-03-1727

The wind speed is prescribed on a 3 second average wind speed at a reference height of 10 meters above ground over the flat open terrain. The return period is not defined.

Unfortunately no more English wind code is available.

2.10 Turkish wind code

2.10.1 General information

Unfortunately no English Turkish wind code is available

2.11 Basic principles for the wind design of buildings

To this point the wind loading and its cause are explained. In this section the basic principles for wind design of buildings are presented to construct safe, efficient and economic structures. The main factors are discussed from table 2.11 section 2.5 and some building examples are presented.

2.11.1 Building weight (mass)

Increasing the mass of building reduce the displacement of the building and improves its stability, like galloping, under wind actions. The main disadvantage of increasing the weight is the increase of the vertical gravity loading. Compression stresses will increase and larger dimensions of beams, columns and foundations are needed. This is very costly solution.

2.11.2 Damping

In all the building codes, vibration actions like vortex shedding and galloping are important design aspects, especially for slender and flexible buildings when the natural frequency of the building reaches the wind or earthquake frequency, see figure 2.23. To reduce the vibration mode of a building, structural damping and damping devices are used. In this section the most common types of dampers are presented. They can be subdivided into passive and active damping systems. The main difference between passive and active is the requirement of an external power source. The passive system does not need power whereas the active damper can only be operated by an external power source. The mechanical properties of a passive system cannot be modified. These systems dissipate energy through the motion of the structure to produce relative motion within the damping devices. An active system can however adjust its mechanical properties based on feed from the structural system to which they are attached. [Symans, M.D., Constantinou, M.C., Taylor, D.P., Garnjost, K.D., 2004]

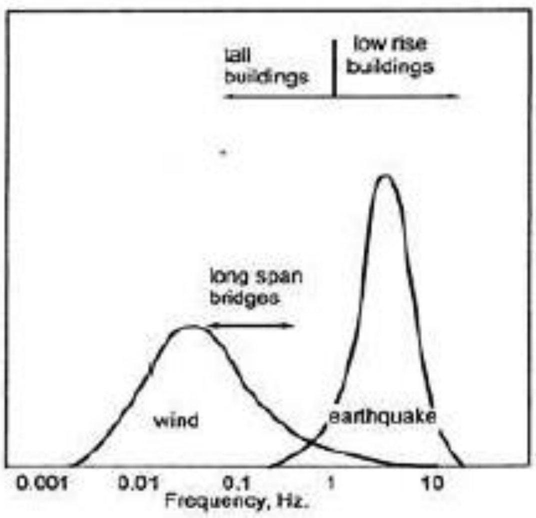


Figure 2.23: Schematization natural frequency buildings [Winter, U. 2011]

Viscous fluid damper

This type of damper is introduced in the structural building design around 1990. It consists of a hollow cylinder which is filled with a highly viscous fluid, like silicone or oil. In this cylinder a piston head with very small holes is placed. Due to motion of the structure, the piston head is moving through the fluid and energy is dissipated by friction. The damper is usually installed as a part of the building bracing system, as presented in figure 2.24 as diagonals.

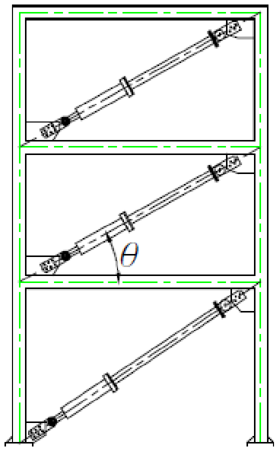


Figure 2.24: Viscous fluid damper [Hwang, J., 2002]

Friction damper

Friction dampers absorbers energy by friction between surfaces. The damper consists of several steel plates sliding in opposite direction during wind/earthquake action. The steel plates are separated by friction materials, like rubber, and dissipated energy by friction between the sliding surfaces. These dampers can be used in braced systems and rigid frame systems.

Tuned mass damper

This kind of damper works by fastening a (big) mass to the structure by a spring. This system is set up that when the structure vibrates, also the mass will follow this vibration in the same frequency. Large shock absorbers, the so-called dashpots, are also attached to the mass which will reduce the vibration. The dashpots convert the movement energy into heat, reducing the structure movement and the buildings oscillation. [Webster, 2003] The most known tuned mass damper in the world is constructed in Taipei 101

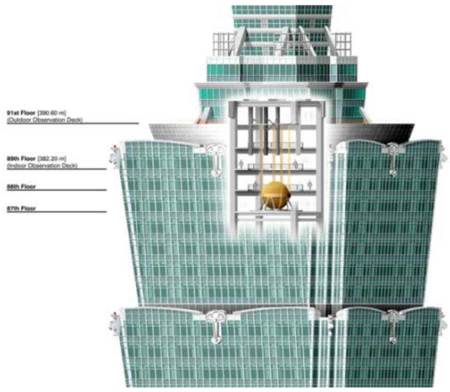


Figure 2.25: Tuned mass damper Taipei 101 [wikipedia.com, 2013]

Base isolation

Base isolation is mainly used for earthquake damping and is less suitable for wind actions. It is a passive system consisting of rubber bearings, which are placed between the building and its foundation. When an earthquake occurs, the rubber bearings offers resistance against lateral movements of the building. As a result of this damping, the forces and movement of the upper building reduces significantly. The main feature of the base isolation technology (figure 2.26) is that it introduces flexibility into the connection between the structure and its foundation. Two type of base isolation exists, elastomeric bearings and friction pendulum bearings.

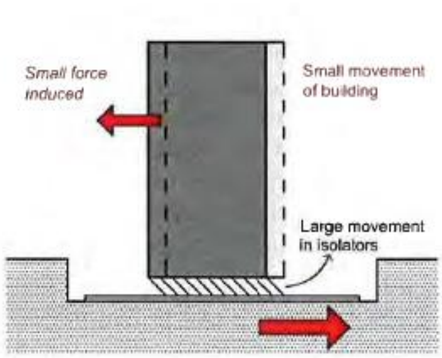


Figure 2.26: Base isolation schematization [University of Canterbury, 2011]

Table 2.28 gives an overview of some damping devices with their performance of application.

Table 2.28: Performance of damping devices [Shibuya, K., 2001]

Dampers			Damping efficiency Extremely Strong Earthquake	Medium Class Earthquake	Wind	Cost	Maintenance
Steel Damper		(SD)	VG	G		VG	VG
Oil Damper	Passive	(OD)	G	G	VG	G	
	Semi-active	(AOD)	G	VG	VG		
Viscous Fluid Damper		(VFD)	G	VG	VG	G	G
Visco-elastic Damper		(VED)	G	VG	VG	G	G
Mass Damper	Passive	(TMD)			G	G	
	Active	(AMD)			VG		

VG: Very good, G: Good

2.11.3 Shape of the building

The most important factor determining the wind load on a building is its shape. The height, width, depth, shape and diameter are all important parameters in calculating the peak velocity pressure and force coefficient. The shape is usually determined by an architect; however there are still ways to improve the wind induced behaviour of a building by (small) modifications, such as [Winter, U., 2011]:

- Porosity and openings
- Rotating or adding twists
- Tapering and setbacks
- Varying the cross section
- Corner modifications
- Applying spoilers

Below some images of buildings are presented where i.e. wind determined the shape.

Figure 2:27: Burj Khalifa [images-google.com, 2013]

Figure 2.28: Shanghai world Finance Center and Jin Mao Tower Shanghai [images-google.com, 2013]

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3. Earthquake Engineering Part 1998-1

3.1 Introduction

As mentioned in chapter 1, horizontal loading is an important loading configuration for the structural design of buildings. Also earthquake action cause horizontal loading on a structure and the magnitude of the force can be very simplified with the Newton's second law of motion [Charleson, A., 2008].

$$F = m \times a$$

The magnitude of the force is the mass multiplied with acceleration. During an earthquake the ground accelerate and the mass of the building will follow this acceleration, resulting in a horizontal force. Luckily earthquakes do not occur often, like wind actions do, but the total magnitude of the applied forces during this relatively short event is much higher. This often results in a major disaster. Buildings, infrastructure and other services severe large damages or even collapse. To meet all the structural requirements in SLS and ULS it is important to investigate all the earthquake parameters in order to design an safe, efficient and economic structure.

To make an efficient earthquake engineering research first the Eurocode will be analysed in detail. The calculating method, earthquake parameters and other important issues will be discussed. On basis of the Eurocode the other "local" building codes will be analysed in short. Afterwards basic design principles for buildings are presented to improve structural safety under earthquake action.

3.2 Earthquake phenomenon.

The thickness of the earth top layer (crust) is very thin. It is divided into several tectonic plates, which are floating on molten rock (figure 3.1 on the next page). Over the year they move relative to each other with approximately 50 mm per year. In some places, the tectonic plates slip past each other horizontally. In other places the plates are pushing against each other, whereby one plate will bend and slide under. Due to the roughness of the surfaces on the edges of the plates, potential sliding and slipping is restrained. The plates are absorbing this energy resulting in the situation where more and more compression and shear strains are accumulated. When this energy suddenly ruptures, the ground suddenly moves and an earthquake is born. The magnitude of an earthquake is often classified in the Richter scale and in the less known Mercalli scale. Both scales are presented in table 3.1 on the next page.

Seventy percent of the annual earthquakes occur around the perimeter of the Pacific plate (East coast Asia and west coast America). Twenty percent along the southern edge of the Eurasian plate (Middle East towards India) and the other ten percent is divided along the other fault lines. To give an indication of the amount of quakes per year, 200 quakes of magnitude 6, 20 quakes of magnitude 7 and 1 quake of magnitude 8 are expected. Their location is most likely along the Pacific plate but their timing is unpredictable.

An earthquake generally causes the most ground shaking at the epicentre. As the epicentre distance increases, the energy of seismic waves arriving at the distant site reduces. There are three types of waves which are generated by a fault rupture. The first wave is the P-wave. This kind of wave travels the fastest and pushes and pulls the soil in which it passes through. The second wave is the S-wave. This wave is of most concern to buildings, while they move the soil particles side to side, vertically and horizontally. The last type of wave is the Rayleigh wave, which moves along the surface.

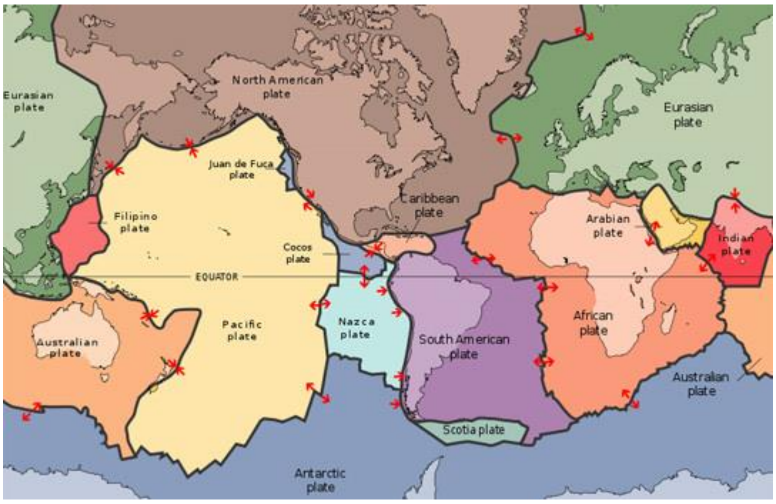

Figure 3.1: Tectonic plates around the globe [Schooltv.nl, 2012]

Table 3.1: Mercalli scale and Richter scale [thegeosphere.com, 2012]

The Mercalli Scale		Level Of Damage	The Richter Scale
1-4	Instrumental to Moderate	No damage.	</= 4.3
5	Rather Strong	Damage negligible. Small, unstable objects displaced or upset; some dishes and glassware broken.	4.4 - 4.8
6	Strong	Damage slight. Windows, dishes, glassware broken. Furniture moved or overturned. Weak plaster and masonry cracked.	4.9 - 5.4
7	Very Strong	Damage slight-moderate in well-built structures; considerable in poorly-built structures. Furniture and weak chimneys broken. Masonry damaged. Loose bricks, tiles, plaster, and stones will fall.	5.5 - 6.1
8	Destructive	Structure damage considerable, particularly to poorly built structures. Chimneys, monuments, towers, elevated tanks may fail. Frame houses moved. Trees damaged. Cracks in wet ground and steep slopes.	6.2 - 6.5
9	Ruinous	Structural damage severe; some will collapse. General damage to foundations. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground; liquefaction.	6.6 - 6.9
10	Disastrous	Most masonry and frame structures/foundations destroyed. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Sand and mud shifting on beaches and flat land.	7.0 - 7.3
11	Very Disastrous	Few or no masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Rails bent. Widespread earth slumps and landslides.	7.4 - 8.1
12	Catastrophic	Damage nearly total. Large rock masses displaces. Lines of sight and level distorted.	> 8.1

3.3 Eurocode Earthquake Actions Part 1-1-4

3.3.1 Introduction

The main scope of the Eurocode 8 is to provide structures in seismic regions, which ensure that in the event of an earthquake; the human lives are protected, the damage of the building is limited and important structures remain operational. This chapter is based on the Eurocode 8 – Design of structures for earthquake resistance.

3.3.2 Performance requirements and compliance criteria

Structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

No-collapse requirement

The structure will be designed and constructed to withstand the design seismic action without local or global collapse. This seismic action is expressed with the following terms.

- Reference seismic action is associated with:
  - probability of exceedance, $Pncr$ in 50 years, recommended value of $Pncr$ of 10%
  - return period, $Tncr$, recommended value of 475 years
- Importance factor γI

Damage limitation requirement

The structure shall be designed and constructed to withstand seismic action, having a larger probability of occurrence than the design seismic action (requirement one), without the occurrence of damage. The associated limitations reduce the repair costs, which would be disproportionately high in comparison with the cost of the structure itself. This seismic action is expressed with the following terms;

- Reference seismic action is associated with:
  - probability of exceedance, $Pncr$ in 10 years, recommended value of $Pncr$ of 10%
  - return period, $Tdir$, recommended value of 95 years

In order to satisfy the fundamental performance requirements, mentioned above, the following limit states shall be checked.

Ultimate limit states

The ultimate limit state is associated with collapse or other structural failure, which endangers the safety of people. The structural system shall be verified regarding enough resistance and energy dissipation capacity. Next the structure shall to be checked to ensure it is stable under the design seismic action. Therefore overturning and sliding instability shall be taken into account. Also foundation reaction and second order effect are considered. At last it should be verified that under seismic action the behaviour of non-structural elements does not present risks to the safety of occupants.

Damage limitation state

The deformation limits of the structure should be satisfied to avoid unacceptable damage. Next to this the structural system shall be verified to have sufficient resistance and stiffness to maintain all vital service functions in the building direct after an earthquake.

3.3.3 Ground conditions and seismic action

The first step for earthquake engineering is to investigate the local ground conditions and the seismic hazard of the building location. Ground investigations and geological studies should be performed to determine these aspects.

Ground conditions

The ground conditions in de Eurocode are classified into 5 main groups, named A to E. S1 and S2 are special ground conditions, wherefore special studies are required. The identification of these ground types describes the stratigraphic profiles and parameters of the soil and takes into account the influence of local ground conditions on the seismic action. The ground types are classified in table 3.2.

Table 3.2: Ground types [NEN-EN 1998-1:2004, Table 3.1]

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Seismic action

Seismic action is subdivided by national authorities into seismic zones depending on the local hazard. The hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration agR. Expressed in gravitational acceleration unit [g] based on ground type A. This parameter corresponds with $Tncr$ (reference return period for no-collepse) or $Pncr$ (reference probability of exceedence in 50 years). An importance factor γI value 1.0 is taken in this consideration.

In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used. This is allowed when agr is not greater than 0.78 [m/s^2] or 0.08 [g]. With 1.0 [g] = 9.81 [m/s^2]. [NEN-EN 1998-1:2005 section 3.2.1]

In cases of very low seismicity, the provisions of the Eurocode 1998 need not be observed, when agr is not greater than 0.39 [m/s^2] or 0.1 [g]. [NEN-EN 1998-1:2005 section 3.2.1]

In figure 3.3 a typical seismic hazard map is presented. This map represents the seismic hazard of Europe. In the diagram legend the reference peak ground acceleration is given. Each seismic zone is marked with a different colour. Note that these values are expressed in [m/s^2].

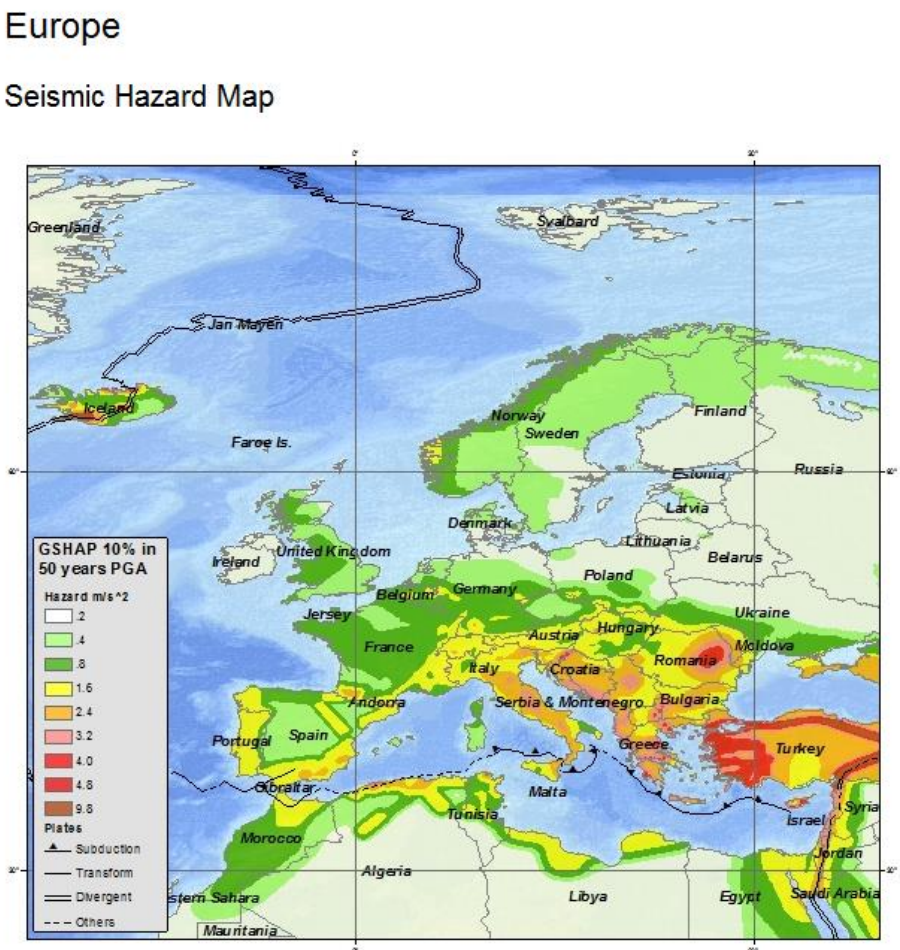

Figure 3.2: Seismic Hazard map Europe [USGS, 2013]

3.3.4 Basic representation of the seismic action

At this point the reference peak ground acceleration is known. Next this reference peak ground acceleration is transformed into an earthquake motion at a given point on the surface. Presented by the ground acceleration response spectrum, which is called the elastic response spectrum. Several spectra are generated such as; horizontal elastic, vertical elastic, horizontal design and vertical design. All spectra are presented below.

Horizontal elastic response spectrum $Se(T)$ (linear)

The horizontal elastic response spectra $Se(T)$, for horizontal structural components under seismic action are given by the equations 3.1 to 3.4 using equations 3.5 and 3.6:

$$0 \leq T \leq Tb : Se(T) = ag \times S \times \left[1 + \frac{T}{Tb} \times (\eta \times 2{,}5 - 1)\right] \quad (3.1)$$

$$Tb \leq T \leq Tc : Se(T) = ag \times S \times \eta \times 2{,}5 \quad (3.2)$$

$$Tc \leq T \leq Td : Se(T) = ag \times S \times \eta \times 2{,}5 \times \left[\frac{Tc}{T}\right] \quad (3.3)$$

$$Td \leq T \leq 4s : Se(T) = ag \times S \times \eta \times 2{,}5 \times \left[\frac{Tc \times Td}{T^2}\right] \quad (3.4)$$

$$ag = \gamma I \times agR \quad (3.5)$$

$$\eta = \sqrt{\frac{10}{(5 + \xi)}} \geq 0{,}55 \quad (3.6)$$

Wherein:

$Se(T)$	Is the elastic response centrum
T	Is the vibration period of a linear single-degree-of-freedom system
ag	Is the design ground acceleration
agR	Is the ground acceleration ground type A (section 3.3.3)
γI	Is the importance factor (table 3.5)
Tb	Is the lower limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Tc	Is the upper limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Td	Is the value defining the beginning of the constant displacement response range of the spectrum (Table 3.3 and table 3.4)
S	Is the soil factor (table 3.3 and table 3.4)
η	Is the damping correction factor, recommended $\eta = 1$ for 5 % viscous damping
ξ	Is the viscous damping ratio of the structure, recommended value of ξ 0.05

If the seismic hazard defined for the site has a surface-wave magnitude Ms greater than 5.5 values of type 1 (table 3.3) are recommended. In case Ms is lower than 5.5 values of type 2 (table 3.4) are recommended.

Table 3.3: Values of parameter type 1 [NEN-EN 1998-1:2004, Table 3.2]

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Table 3.4: Values of parameter type 2 [NEN-EN 1998-1:2005, Table 3.3]

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Importance classes

Four importance classes exist. Each having their own value for γI. The recommended values γI for importance classes I, II, III and IV, are equal to 0.8, 1.0, 1.2 and 1.4. The description of each importance class is given in table 3.5.

Table 3.5: Importance classes

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Shape of the horizontal elastic response spectrum

The values of the periods, Tb, Tc and Td and soil factor S describing the shape of the elastic response spectrum. The elastic response spectra's for both types are given in figures 3.3 and 3.4.

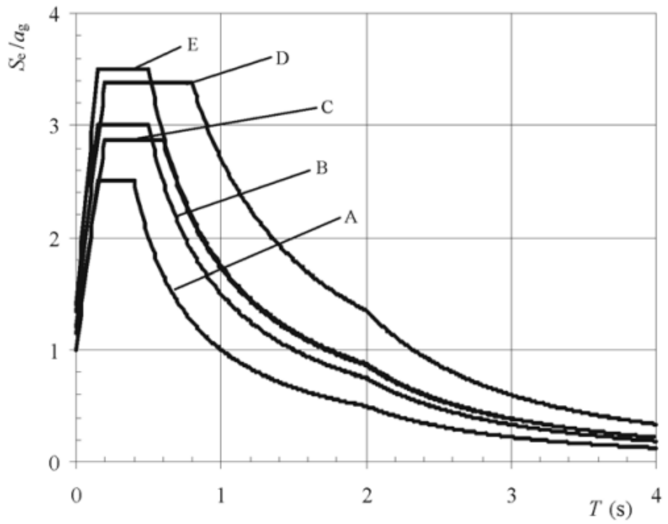

Figure 3.3: Elastic response spectra type 1 with 5% damping [NEN-EN 1998-1:2005]

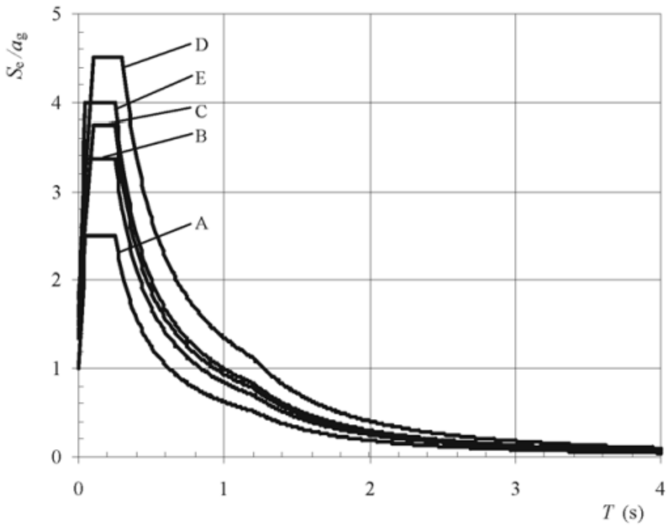

Figure 3.4: Elastic response spectra type 2 with 5% damping [NEN-EN 1998-1:2005]

Horizontal elastic displacement response spectrum $SDe(T)$

The elastic displacement response spectrum is obtained by transformation of the elastic acceleration response spectrum $Se(T)$ given by equation 3.7. This equation is applied for vibration period not exceeding 4.0 seconds.

$$SDe(T) = Se(T) \times \left[\frac{T}{2\pi}\right]^2 \tag{3.7}$$

Horizontal design ground displacement dg

The design ground displacement dg is given by equation 3.8:

$$dg = 0{,}025 \times ag \times S \times Tc \times Td \tag{3.8}$$

ag	Is the design ground acceleration on ground type A (section 3.3.3)
S	Is the soil factor (Table 3.3 and table 3.4)
Tc	Is the upper limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Td	Is the value defining the beginning of the constant displacement response range of the spectrum (Table 3.3 and table 3.4)

Vertical elastic response spectrum $Sve(T)$ Linear

The vertical elastic response spectra $Sve(T)$, for vertical structural component under seismic action are given by equations 3.9 to 3.12 using values from table 3.6:

$$0 \leq T \leq Tb : Sve(T) = avg \times \left[1 + \frac{T}{Tb} \times (\eta \times 3{,}0 - 1)\right] \tag{3.9}$$

$$Tb \leq T \leq Tc : Sve(T) = avg \times \eta \times 3{,}0 \tag{3.10}$$

$$Tc \leq T \leq Td : Sve(T) = avg \times \eta \times 3{,}0 \times \left[\frac{Tc}{T}\right] \tag{3.11}$$

$$Td \leq T \leq 4s : Sve(T) = avg \times \eta \times 3{,}0 \times \left[\frac{Tc \times Td}{T^2}\right] \tag{3.12}$$

$Se(T)$	Is the elastic response centrum
T	Is the vibration period of a linear single-degree-of-freedom system
avg	Is the design ground acceleration on ground, (table 3.6)
Tb	Is the lower limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Tc	Is the upper limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Td	Is the value defining the beginning of the constant displacement response range of the spectrum (table 3.3 and table 3.4)
η	Is the damping correction factor, recommended $\eta = 1$ for 5 % viscous damping, (equation 3.6)

Table 3.6: Values of parameters types 1 and 2 [NEN-EN 1998-1:2005, Table 3.4]

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

Horizontal design spectrum for elastic analysis $Sd(T)$ (non-linear)

The horizontal design spectrum for elastic analysis $Sd(T)$ is generated to resist seismic actions in the non-linear range. $Sd(T)$ is given by equations 3.13 to 3.16. To avoid explicitly an inelastic structural analysis in the design, the capacity of the structure to dissipate energy is taken into account by performing an elastic analysis based on a response spectrum reduced by behaviour factor q.

$$0 \leq T \leq Tb : Sd(T) = ag \times S \times \left[\frac{2}{3} + \frac{T}{Tb} \times \left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \tag{3.13}$$

$$Tb \leq T \leq Tc : Sd(T) = ag \times S \times \frac{2{,}5}{q} \tag{3.14}$$

$$Tc \leq T \leq Td : Sd(T) \begin{cases} = ag \times S \times \dfrac{2{,}5}{q} \times \left[\dfrac{Tc}{T}\right] \\ \geq \beta \times ag \end{cases} \tag{3.15}$$

$$Td \leq T: \quad Sd(T) \begin{cases} = ag \times S \times \dfrac{2{,}5}{q} \times \left[\dfrac{Tc \times Td}{T^2}\right] \\ \geq \beta \times ag \end{cases} \tag{3.16}$$

Wherein:

ag	Is the design ground acceleration on ground type A (section 3.3.3)
S a	Is the soil factor (Table 3.3 and table 3.4)
Tc	Is the upper limit of the period of the constant spectral acceleration branch (Table 3.3 and table 3.4)
Td	Is the value defining the beginning of the constant displacement response range of the spectrum (Table 3.3 and table 3.4)
q	Is the behaviour factor, depending on structure type and material
β	Is the lower bound factor for the horizontal design spectrum, recommended $\beta = 0{,}2$

The behaviour factor q is an (very) important factor in the equations mentioned above. This factor is used to reduce forces obtained from a linear analysis; in order to account for the nonlinear response of a structure associated with the materials, the structural system and the design procedures. The construction materials considered in this section are concrete, steel, composite steel / concrete, timber and masonry.

Structures are classified in two categories; Low-dissipative and dissipative structures. By low dissipative structures no account is taken of any hysteretic energy dissipation. The behaviour value is taken between 1.5 and 2. By dissipative structures the behaviour factor is taken as being greater than these limiting values accounting for the hysteretic energy dissipation that mainly occurs in specifically design zones, the so-called dissipative zones or critical regions. In the tables below all the behaviour factors are presented.

Concrete structures

The upper limit value of the behaviour factor q for concrete structures is given with equation 3.17 and table 3.7.

$$q = q_0 k_w \geq 1{,}5 \tag{3.17}$$

Wherein

q_0 Is the basic value of the behaviour factor (table 3.7)
k_w Is the factor reflecting the prevailing failure mode in structural systems with walls, value 1.0

Table 3.7: Basic behaviour factors q_0 concrete structures [NEN-EN 1998-1:2004, Table 5.1]

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4{,}0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

For buildings which are not regular in elevation, the value of q_0 should be reduced by 20 %. The expressions DCM and DCH mean ductility classes medium and high. The multiplication factor α_u/α_1 depending on the structural system given by table 3.8

Table 3.8: Multiplication factor α_u/α_1 [NEN-EN 1998-1:2004, section 5.2.2.2]

Frames or frame-equivalent dual systems	α_u/α_1
One-storey building	1,1
Multi-storey, one bay frames	1,2
Multi-storey, multi-bay frames or dual structures	1,3
Uncoupled wall systems	α_u/α_1
Wall systems with only two uncoupled walls	1,0
Other uncoupled wall systems	1,1
Wall-equivalent dual, or coupled wall systems	1,2

For precast concrete structures the same behaviour factor may be used when general provisions are taken regarding the connections of the precast elements. These provisions are; connections located away from critical regions, overdesigned connections and energy dissipating connections.

Steel structures

The upper limit value of the behaviour factor q for steel structures is given with table 3.9. Value $\alpha_u/\alpha_1$is taken as 1.2.

Table 3.9: Basic behaviour factors q steel structures [NEN-EN 1998-1:2004, Table 6.2]

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frame with concentric bracings Diagonal bracings V-bracings	 4 2	 4 2,5
c) Frame with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	See section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section 7	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_u/\alpha_1$

<u>Composite steel / concrete structures</u>

The upper limit value of the behaviour factor q for composite structures is given with table 3.10. Value $\alpha_u/\alpha_1$is taken as 1.1 and the structure types are given in figure 3.5.

Table 3.10: Basic behaviour factors q composite structures [NEN-EN 1998-1:2004, Table 7.2]

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a), b), c) and d)	See Table 6.2	
e) Composite structural systems Composite walls (Type 1 and Type 2)	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$
Composite or concrete walls coupled by steel or composite beams (Type 3)	$3\alpha_u/\alpha_1$	$4{,}5\alpha_u/\alpha_1$
f) Composite steel plate shear walls	$3\alpha_u/\alpha_1$	$4\alpha_u/\alpha_1$

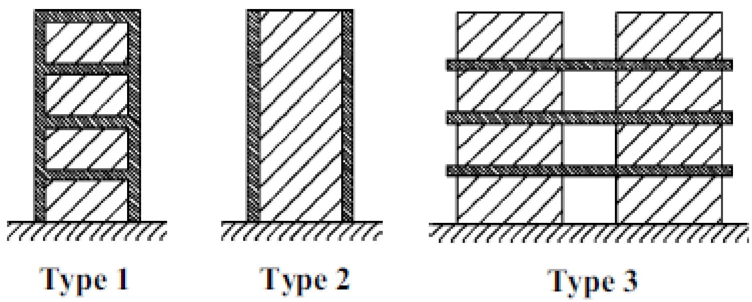

Figure 3.5: Types composite structures [NEN-EN 1998-1:2004, section 7.3.1]

Timber structures

The upper limit value of the behaviour factor q for timber structures is given with table 3.11.

Table 3.11: Basic behaviour factors q timber structures [NEN-EN 1998-1:2004, Table 8.1]

Design concept and ductility class	q	Examples of structures
Low capacity to dissipate energy - DCL	1,5	Cantilevers; Beams; Arches with two or three pinned joints; Trusses joined with connectors.
Medium capacity to dissipate energy - DCM	2	Glued wall panels with glued diaphragms, connected with nails and bolts; Trusses with doweled and bolted joints; Mixed structures consisting of timber framing (resisting the horizontal forces) and non-load bearing infill.
	2,5	Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).
High capacity to dissipate energy - DCH	3	Nailed wall panels with glued diaphragms, connected with nails and bolts; Trusses with nailed joints.
	4	Hyperstatic portal frames with doweled and bolted joints (see **8.1.3(3)**P).
	5	Nailed wall panels with nailed diaphragms, connected with nails and bolts.

Masonry structures

The upper limit value of the behaviour factor q for masonry structures is given with table 3.12.

Table 3.12: Basic behaviour factors q masonry structures [NEN-EN 1998-1:2004, Table 9.1]

Type of construction	Behaviour factor q
Unreinforced masonry in accordance with EN 1996 alone (recommended only for low seismicity cases).	1,5
Unreinforced masonry in accordance with EN 1998-1	1,5 - 2,5
Confined masonry	2,0 – 3,0
Reinforced masonry	2,5 - 3,0

Vertical design spectrum for elastic analysis $Sdve(T)$ (non-linear)

The vertical design spectrum for elastic analysis $Sdve(T)$ is given by equations to 3.18 to 3.21.

$$0 \leq T \leq T_B \quad : S_d(T) = a_{vg} \times \left[\frac{2}{3} + \frac{T}{T_B} \times \left(\frac{3,0}{q} - \frac{2}{3}\right)\right] \tag{3.18}$$

$$T_B \leq T \leq T_C \quad : S_d(T) = a_{vg} \times \frac{3,0}{q} \tag{3.19}$$

$$T_C \leq T \leq T_D \quad : S_d(T) \begin{cases} = a_{vg} \times \dfrac{3,0}{q} \times \left[\dfrac{T_C}{T}\right] \\ \geq \beta \times a_g \end{cases} \tag{3.20}$$

$$T_D \leq T : \leq 4s \quad : S_d(T) \begin{cases} = a_{vg} \times \dfrac{3,0}{q} \times \left[\dfrac{T_C \times T_D}{T^2}\right] \\ \geq \beta \times a_g \end{cases} \tag{3.21}$$

Wherein

avg	Is the design ground acceleration on ground, (table 3.6)
Tc	Is the upper limit of the period of the constant spectral acceleration branch (table 3.2 and table 3.3)
Td	Is the value defining the beginning of the constant displacement response range of the spectrum (table 3.2 and table 3.3)
q	Is the behaviour factor, depending on structure type and material
β	Is the lower bound factor for the horizontal design spectrum, recommended $\beta = 0,2$

3.3.5 Structural analysis

Several methods of analysing the structural behaviour of a building under seismic action exist. These methods are mentioned below and are explained in the next sections.

- Linear-elastic analysis
  - Lateral force method of analysis
  - Model response spectrum analysis
- Non-linear method
  - Non-linear static (pushover) analysis
  - Non-linear time history (dynamic) analysis

3.3.6 Lateral force method of analysis

This method may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in principal direction. Also it has to fulfil both of the two following conditions; equation 3.22 and the building has to be categorised as being regular in elevation, like shape, stiffness and mass.

$$T1 \leq \begin{cases} 4 \times Tc \\ 2{,}0\ s \end{cases} \tag{3.22}$$

Base shear force Fb

The seismic base shear force Fb, for each horizontal direction is given by equations 3.23 using equations 3.24 to 3.26:

$$Fb = Sd(T1) \times m \times \lambda \tag{3.23}$$

$$T1 = Ct \times H^{\frac{3}{4}} \quad (for\ building\ up\ to\ 40m) \tag{3.24}$$

$$Ct = \frac{0{,}075}{\sqrt{Ac}} \tag{3.25}$$

$$Ac = \sum \left[Ai \times \left(0{,}2 + \left(\frac{lwi}{H} \right) \right)^2 \right] \tag{3.26}$$

Wherein:

$Sd(T1)$	Is the ordinate of the design spectrum at period $T1$
$T1$	Is the fundamental period of vibration for building
m	Is the total mass of the building above the foundation
λ	Is the correction factor, choose $\lambda = 0.85\ if\ T1 \leq 2 \times Tc$, otherwise $\lambda = 1.0$
Ct	Is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and 0,050 for all other structures, for concrete shears wall choose equation 3.25

Distribution of the horizontal seismic forces Fi

The distributed horizontal seismic forces Fi acting on buildings shall be determined by two planar models given by equations 3.27 and 3.28. The horizontal force Fi shall be distributed to the lateral load resisting system.

$$Fi = Fb \times \frac{si \times mi}{\sum si \times mj} \tag{3.27}$$

$$Fi = Fb \times \frac{zi \times mi}{\sum zj \times mj} \tag{3.28}$$

Wherein:

Fi	Is the horizontal force acting on story i
Fb	Is the seismic base shear force (section 3.3.6)
si, sj	Are the displacements of masses mi, mj in the fundamental mode shape
mi, mj	Are the masses of stories mi, mj
zi, zj	Are the heights of the masses mi, mj above the level of application of the seismic action (foundation or top of a rigid basement)

<u>Torsional effect δ</u>

Torsional effects may be accounted by multiplying the horizontal seismic forces Fi with factor δ given by equation 3.29.

$$\delta = 1 + 0{,}6 \times \frac{x}{Le} \tag{3.29}$$

Wherein:

x	Is the distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action
Le	Is the distance between the two out outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action

3.3.7 Model response spectrum analysis

This method is applied when buildings do not satisfy the following conditions; equation 3.22 and the building has not been categorised as being regular in elevation. This model takes into account all the modes of vibration that significantly contribute to the global response. Also torsional effects are considered.

Model response spectrum analysis can be modelled with finite element programs, such as "SCIA engineer". The input parameters are presented in section 3.3.4.

3.3.8 Non-linear methods

By a non-linear method also the strength of structural elements and their post-elastic behaviour are modelled. By using the pushover analysis a non-linear static analysis is carried out under conditions of constant gravity load and monotonically increasing horizontal loads. The time-dependent response of the structure may be obtained through direct numerical integration of its differential equations of motion. Both methods are very complex and only sophisticated software is able to model these two methods. In this thesis only linear-elastic analyses will be modelled in the design phase of this thesis.

3.3.9 Safety verifications

<u>Ultimate limit state</u>

The ultimate limit state is a no-collapse requirement of the structure under seismic conditions. The following requirements will be considered; resistance, ductility, equilibrium, foundation stability and seismic joints. Some equations are given below. Further attention will be taken in the design phase of this thesis.

Resistance $$Ed \leq \mathrm{Rd} \tag{3.30}$$

Second order $$\theta = \frac{Ptot \times dr}{Vtot \times h} \leq 0{,}10 \tag{3.31}$$

Ductility $\sum MRe \ \geq \ 1{,}3 \sum Mrb$ (3.32)

Foundation $EFd = EF, G + \gamma Rd \ \times \Omega EF, E$ (3.33)

Damage limitation

The damage limitation is determined by limitation of interstorey drift given by equations 3.34 to 3.36:

$For\ building\ with\ nonstructural\ elements\ of\ brittle\ materials\ attacted\ to\ the\ structure$:

$$dr \ \times \ v \leq 0{,}005 \ h \quad (3.34)$$

$For\ buildings\ having\ ductile\ nonstructural\ elements$

$$dr \ \times \ v \ \leq 0{,}0075 \ h \quad (3.35)$$

$For\ building\ without\ nonstructural\ elements$

$$dr \ \times \ v \leq 0{,}010 \ h \quad (3.36)$$

Wherein:

dr Is the design interstory drift
h Is the storey height
v Is the reduction factor, recommended $v = 0{,}4$ for importance classes III and IV

3.4 Summery Eurocode earthquake

The main goal of the Eurocode is to provide safe structures in seismic regions. Therefore two important performance requirements are established for structures.

- For earthquake with a high magnitude, no-collapse requirement is normative and this requirement is based on a probability of exceedance of 50 years.
- For earthquakes with a lower magnitude, damage limitation is normative and this requirement is based on a probability of exceedance of 10 years.

The structural acceleration reaction on earthquake action is modelled with the so-called response spectrums. These response spectrums are established with calibration the total structure response by Fourier transformation, taking n-order of vibration modes. As a result several main response spectrums are generated and used in the Eurocode. These response spectrums include the ground acceleration, ground conditions, fundamental natural period of a structure and its damping. Because earthquake acceleration occurs in three-dimensions several spectrums are be provided, e.g. for horizontal or vertical acceleration.

The analyses of a structure can be done with several methods. The choice of method that shall be used depends on the shape, mass, and stiffness, but also on the wanted structural analyses exactness. Linear analyses are most common and can be generated with basic Finite Element programs like “SCIA Engineer”. A non-linear method takes besides the strength also the post-elastic behaviour of the structural elements into account. Complex calculations are generated by the use of complex Finite Element Programs.

The Eurocode prescribe an analytical method to obtain the lateral force by using a linear-elastic analysis method. With this method the characteristic earthquake load; (named in the Eurocode) **base shear force** is calculated and is presented in equation 3.37.

$$Fb = Sd(T1) \times m \times \lambda \quad (3.37)$$

All the different kinds of spectrums and base shear force Fb are determined with the following parameters.

Table 3.13: Parameters [Hartmann, J., 2014]

Building parameters	Structural parameters	Geographical parameters
Height	Behaviour factor	Ground type
Importance classes	Damping correction factor	Seismic hazard
Mass	Material	Soil factor
	Period limits	
	Response spectrum	
	Structure type	
	Vibration period structure	
	Viscous damping ratio	

Safety verifications of the two important performance requirements shall be established to verify the ultimate limit state and damage limitation. Ultimate limit state requires no-collapse. Whereas the damage limitation is determined by the limitation of interstory drift.

These requirements are specialized for concrete buildings, steel buildings, composite steel-concrete buildings, timber and masonry buildings. During the (detail) design phase, more emphasis will placed on these aspects.

3.5 Chinese earthquake code

3.5.1 General information

The Chinese building code is: GB 50011-2001, Code for seismic design of buildings.

This section is based on the Chinese code GB 50011-2001 and the International Institute of Seismology and Earthquake engineering IISEE. This is a global information network of experts for earthquake disaster and prevention technologies.

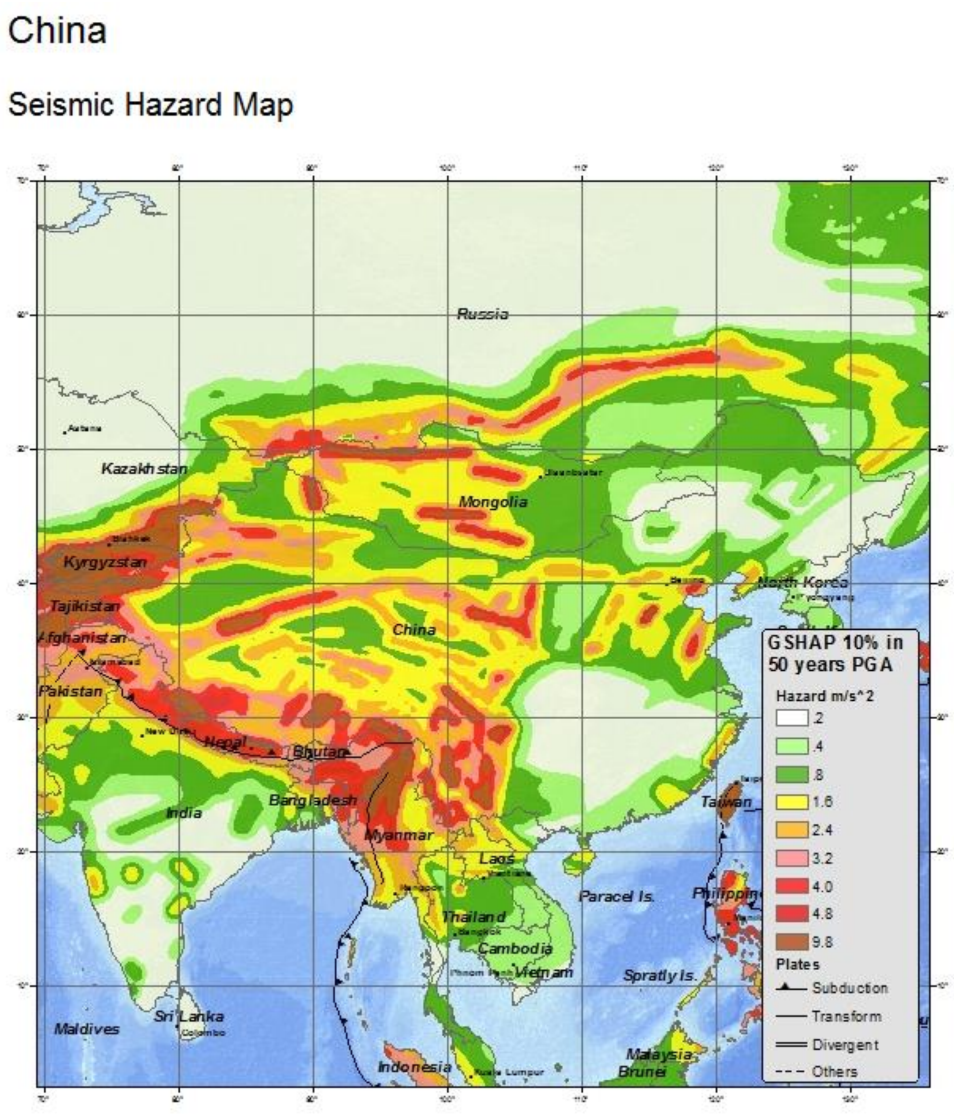

Figure 3.6: Seismic Hazard Map China [USGS, 2013]

3.5.2 Horizontal seismic effective coefficient α (response spectrum)

The horizontal seismic effective coefficient α is calculated with the response spectrum presented in figure 3.7 and using equations 3.38 to 3.40.

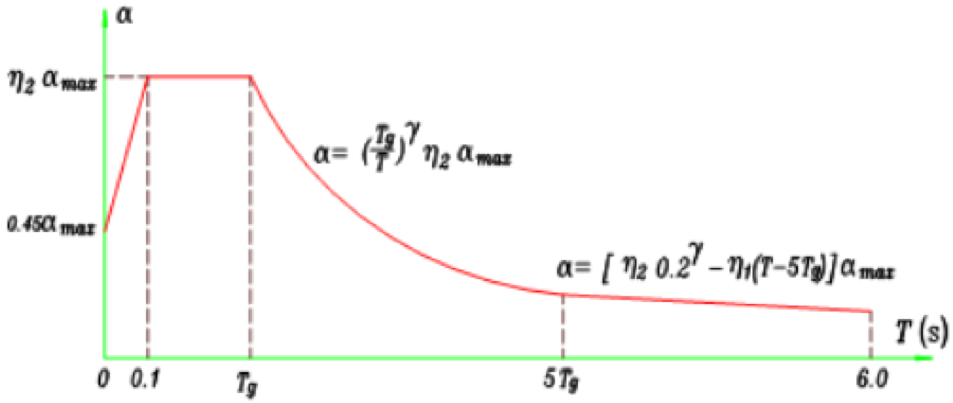

Figure 3.7: Response spectrum [IISEE, 2012]

$$0{,}1 < T < Tg \qquad \alpha = \alpha \max \tag{3.38}$$

$$Tg < T < 5\,Tg \qquad \alpha = \left(\frac{Tg}{T}\right)^{\gamma} \times \eta_2 \times \alpha \max \tag{3.39}$$

$$5\,Tg < T < 6.0 \qquad \alpha = [\eta_2 \times 0,2^{\gamma} - \eta_1 \times (T - 5\,Tg)] \times \alpha max \tag{3.40}$$

$$\gamma = 0{,}9 + \frac{0{,}05 - \xi}{0{,}3 + 6 \times \xi} \tag{3.41}$$

$$\eta_1 = 0{,}02 + \frac{0{,}05 - \xi}{4 + 32 \times \xi} \tag{3.42}$$

$$\eta_2 = 1 + \frac{0{,}05 - \xi}{0{,}08 + 1{,}6 \times \xi} \tag{3.43}$$

Wherein:

T	Is the fundamental natural period of the structure in seconds
Tg	Is the design characteristic period of ground motion (table, 3.14)
αmax	Is the maximum value of the seismic coefficient (table 3.15)
γ	Is the attenuation index
ξ	Is the damping ratio, recommended value for $\xi$is 0,05
$\eta_{1,2}$	Both parameters are the slope adjustment coefficients

Table 3.14: Design characteristic period of ground motion [IISEE, 2012]

Design Earthquake Groups	Site Category				
	I_0	I_1	II	III	IV
First Group	0.20	0.25	0.35	0.45	0.65
Second Group	0.25	0.30	0.40	0.55	0.75
Third Group	0.30	0.35	0.45	0.65	0.90

Table 3.15: maximum value of seismic coefficient [IISEE, 2012]

Seismic Effect	VI	VII	VIII	IX
Frequently Occurred Earthquakes	0.04	0.08(0.12)	0.16(0.24)	0.32
Expected Rare Earthquakes	-	0.50(0.72)	0.90(1.20)	1.40

3.5.3 Horizontal seismic forces Fek (base shear force)

The horizontal seismic force, or base shear force, is calculated with equation

$$Fek = \alpha \times Geq \tag{3.44}$$

Wherein:

Fek Is the standard value of the total horizontal seismic action of the structure
α Is the horizontal seismic effective coefficient, (section 3.5.2)
Geq Is the total equivalent seismic weight of a building

3.5.4 Horizontal seismic shear Fi of i-th story

The horizontal seismic shear over the height of the building can be calculated with expressions 3.45 and 3.46. Figure 3.8 gives an schematisation of the applied loads.

$$Fi = \frac{Gi \times Hi}{\sum_{j=1}^{n} Gj \times Hj} \times Fek \times (1 - \delta_n) \tag{3.45}$$

$$\Delta\, Fn = \delta n \times Fek \tag{3.46}$$

Wherein:

Gi, Gj Are representative values of gravity loads
Hi, Hj Are the representative height of the masses i,j
δn Is the additional seismic action coefficient ath the top level of the building (table 3.16)
Fek Is the horizontal seismic force, (section 3.5.3)
$\Delta\, Fn$ Is the additional seismic force action at the top of the building

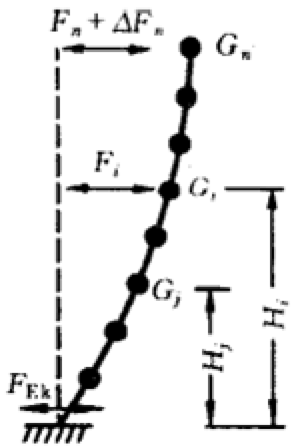

Figure 3.8: Load configuration of horizontal seismic action [IISEE, 2012]

Table 3.16: additional seismic action [GB 50011:2001, table 5.2.1]

T_g(s)	$T \geq 1.4T_g$	$T < 1.4T_g$
≤0.35	0.08T +0.07	0.0
>0.35~0.55	0.08T+0.01	0.0
>0.55	0.08T-0.02	0.0

3.6 Japanese earthquake code

3.6.1 General information

The Japanese building code is: Ministry regulation No. 14, 1997

This section is based on the International Institute of Seismology and Earthquake engineering IISEE. This is a global information network of experts for earthquake disaster and prevention technologies.

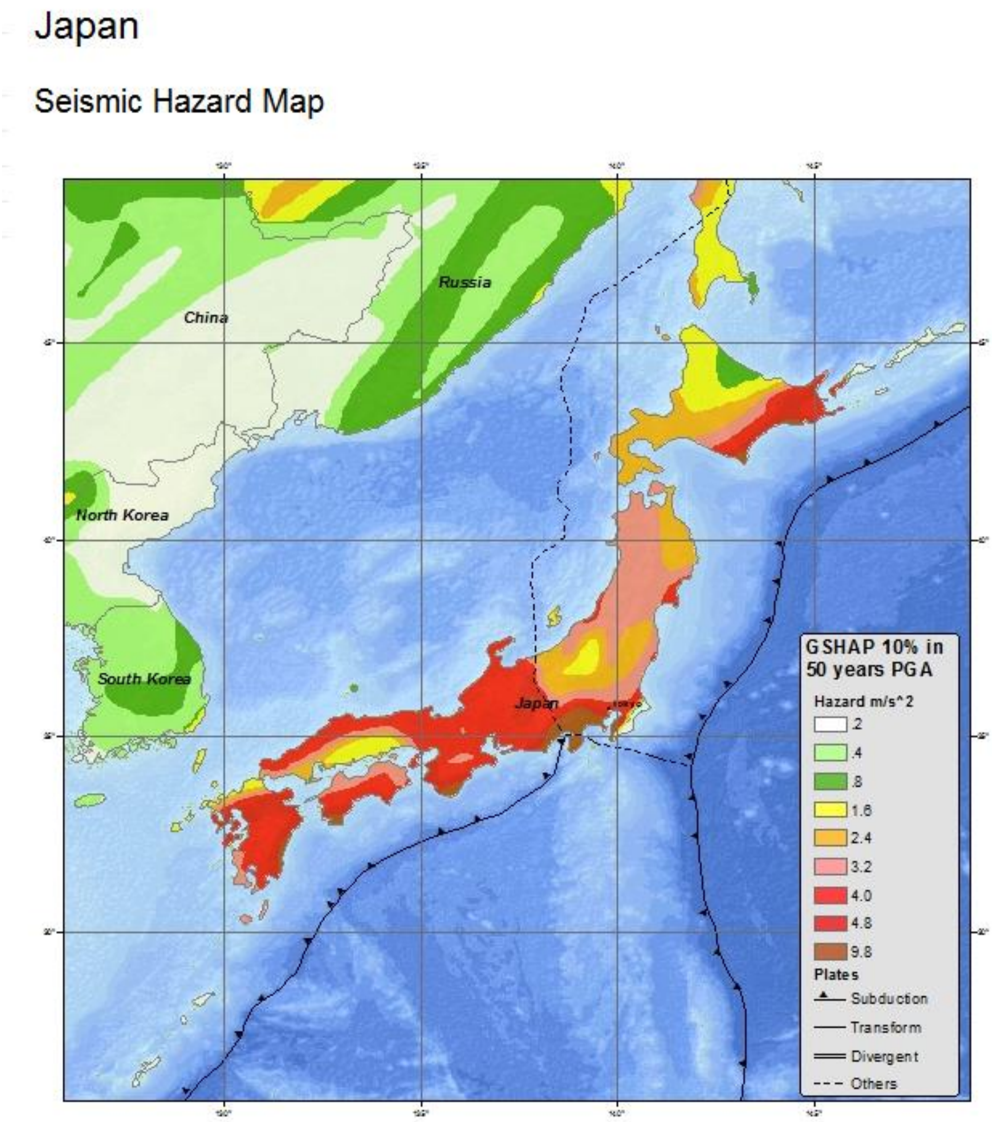

Figure 3.9: Seismic Hazard Map Japan [USGS, 2013]

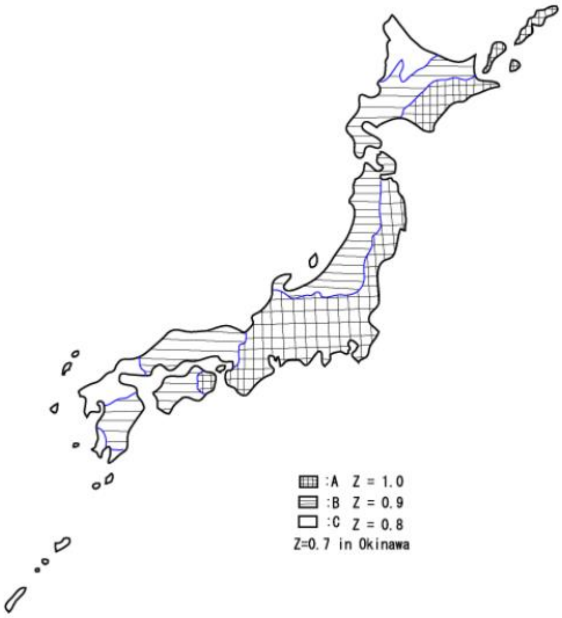

Figure 3.10: Seismic Hazard Map Japan [IISEE, 2000]

3.6.2 Design spectral coefficient Rt (response spectrum)

The design spectral coefficient for the Japanese earthquake code is given by equations 3.47 to 3.49 or figure 3.10 using table 3.17. Herein is T the fundamental natural period in seconds.

$$T < Tc \qquad Rt = 1 \tag{3.47}$$

$$Tc < T < 2\,Tc \qquad Rt = 1 - 0.2 \times \left(\frac{T}{Tc} - 1\right)^2 \tag{3.48}$$

$$2\,Tc < T \qquad Rt = \frac{1.6 \times Tc}{T} \tag{3.49}$$

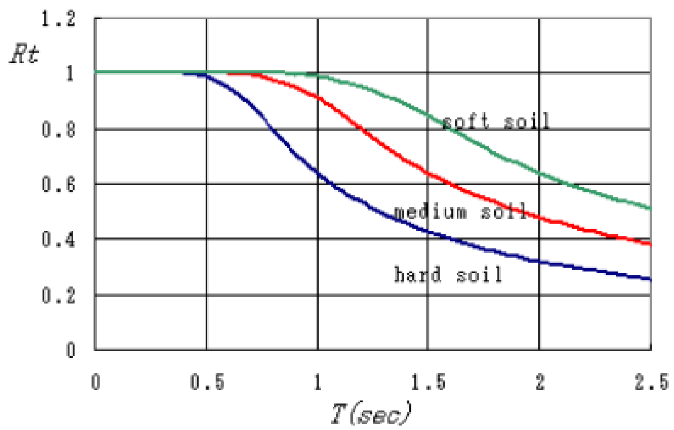

Figure 3.11: Design spectral coefficient [IISEE, 2000]

Table 3.17: Tc and soil classifications

Soil type	Ground characteristics	T_c
Hard soil	Ground consisting of rock, hard sandy gravel, etc. classified as Tertiary or older.	0.4
Medium soil	Other than hard type or soft one.	0.6
Soft soil	Alluvium consisting of soft delta deposits, topsoil, mud, or the like (including fills, if any), whose depth is 30 meters or more, land obtained by reclamation of a marsh, muddy sea bottom, etc., where the depth of the reclaimed ground is 3 meters or more and where 30 years have not yet elapsed since the time of reclamation.	0.8

3.6.3 Lateral seismic shear forces Q (base shear force)

The lateral seismic shear force, or base shear force, is calculated with equation 3.50.

$$Q = Z \times Rt \times C0 \times W \tag{3.50}$$

Wherein:

Z Is the seimic hazard zoning coefficient (figure 3.9)
Rt Is the Design spectral coefficient, (section 3.6.2)
$C0$ Is the standard shear coefficient, take
$C0 \geq 0{,}2$ for allowable stress design against moderate earthquake
$C0 \geq 1{,}0$ for ultimate lateral shear capacity design against servere earthquake
W Is the weight above ground

3.6.4 Lateral seismic shear of Qi i-th story

The lateral seismic shear for the i-th story is calculated with equation 3.51 using equations 3.52 and 3.53

$$Qi = Z \times Rt \times C0 \times W \times Ai \tag{3.51}$$

$$Ai = 1 + \left(\frac{1}{\sqrt{(\alpha i)}} - \alpha i\right) \times \frac{2 \times T}{1 + 3 \times T} \tag{3.52}$$

$$\alpha i = \frac{Wi}{W} \tag{3.53}$$

Wherein:

W Is the weight above ground
Wi Is the weight above i-th story
T Is the fundamental natural period of the structure in seconds

3.7 Indonesian earthquake code

3.7.1 General information

The Indonesian building code is: SNI-03-1726-2002

This section is based on the Indonesian code SNI-03-1726-2002 and the International Institute of Seismology and Earthquake engineering IISEE. This is a global information network of experts for earthquake disaster and prevention technologies.

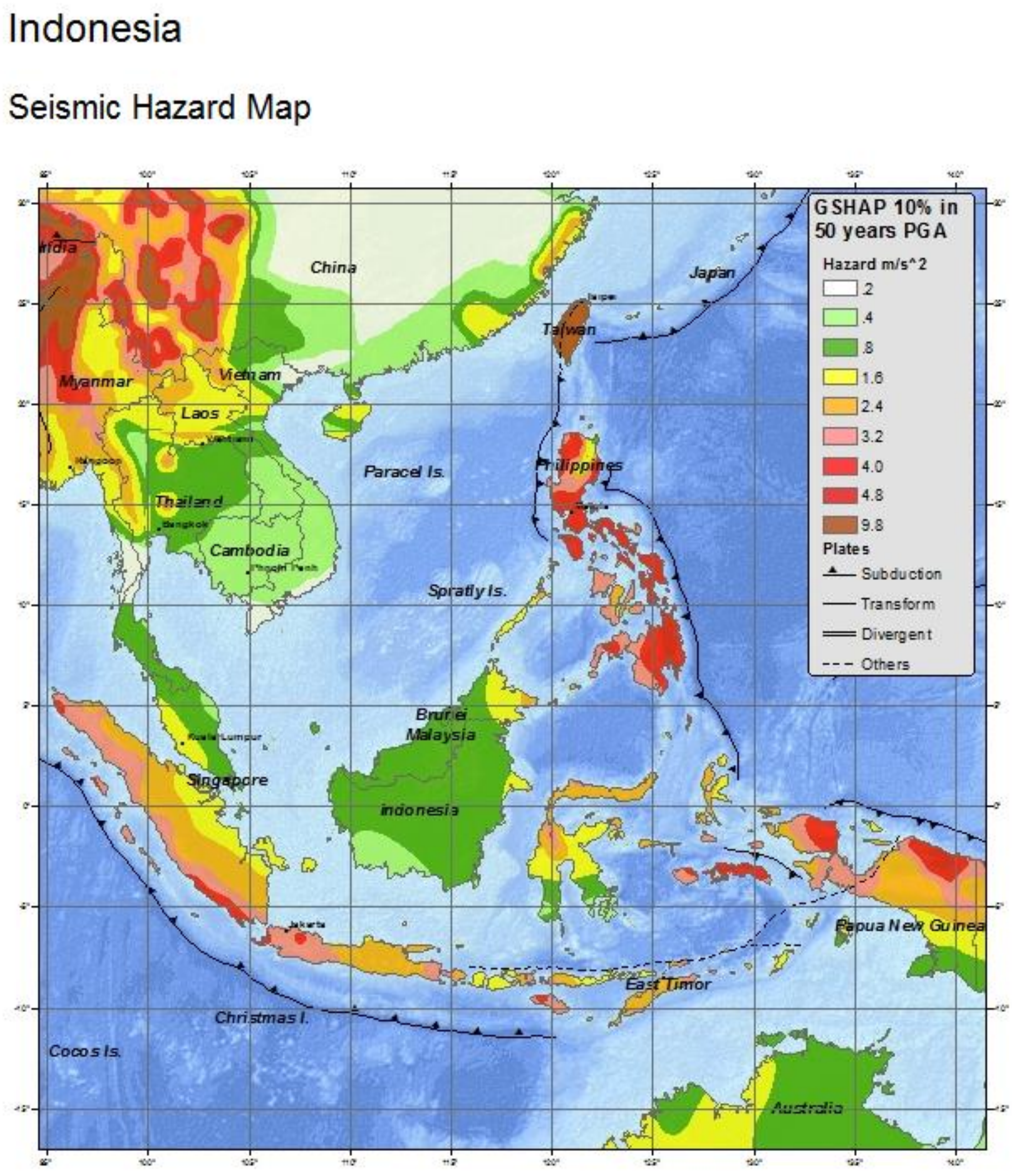

Figure 3.12: Seismic Hazard Map Indonesia [USGS, 2013]

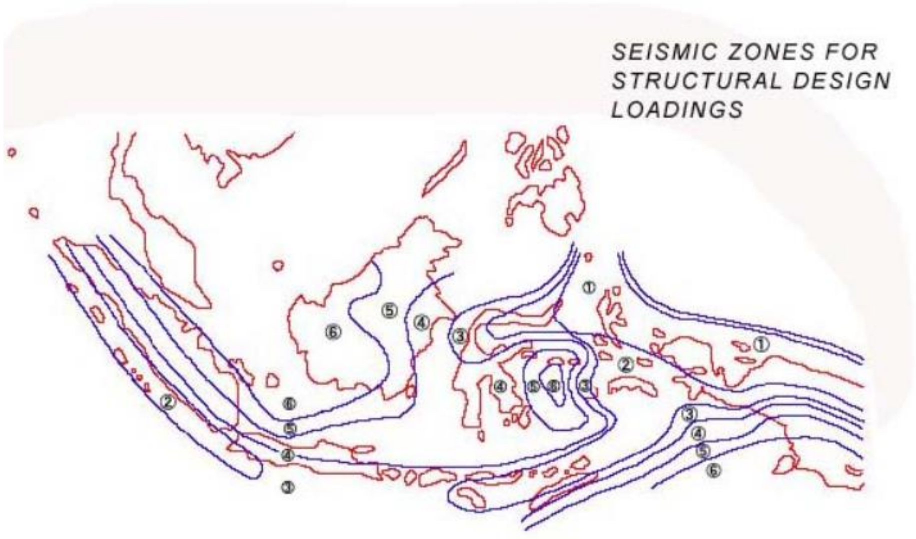

Figure 3.13: Seismic Zones Map Indonesia [IISEE, 2000]

3.7.2 Basic seismic coefficient C (response spectrum)

The basic seismic coefficient C is presented in figure 3.14 using seismic zone map of figure 3.13.

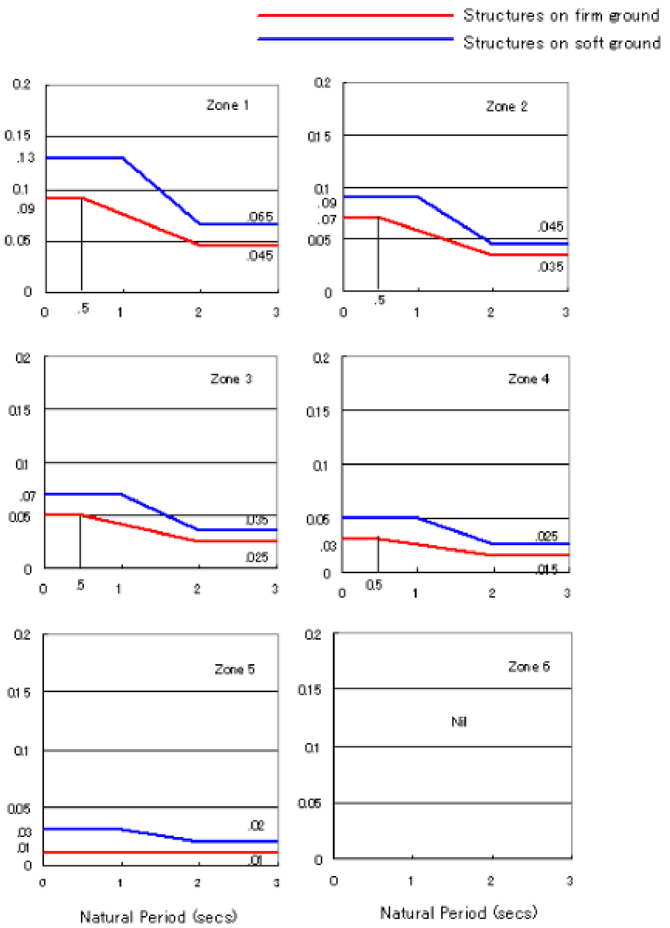

Figure 3.14 Basic seismic coefficient Indonesia IISEE, 2000]

3.7.3 Horizontal base shear V (base shear force)

The horizontal shear force, or base shear force, is calculated with equation 3.54.

$$V = C \times I \times K \times Wt \tag{3.54}$$

Wherein:

C	Is the seismic coefficient, (section 3.7.2)
I	Is the importance factor, (table 3.18)
K	Is the structure type factor, (table 3.19)
Wt	Is the total vertical load

Table 3.18: Importance factor [IISEE, 2000]

TYPE OF BUILDING	IMPORTANCE FACTOR I
a. Monumental Buildings	1.5
b. Essential facilities that should remain functional after an earthquake Examples of these facilities would be : Hospitals Food storage structures Emergency relief stores Power stations Water works Radio and Television facilities Places of assembly	1.5
c. Distribution facilities for gas or petroleum products in urban areas	2.0
d. Structures for the support or containment of dangerous substances (such as acids, toxic substances etc)	2.0
e. Other structures	1.0

Table 3.19: Structural type factor [IISEE, 2000]

STRUCTURE TYPE	CONSTRUCTION MATERIAL OF THE SEISMIC ENERGY DISSIPATING ELEMENTS	STRUCTURAL TYPE FACTOR K
DUCTILE FRAMES	Reinforced Concrete Prestressed Concrete (3) Steel Timber	1.0 1.4 1.0 1.7
COUPLED SHEAR WALLS WITH DUCTILE WALLS (1)	Reinforced Concrete	1.0
CANTILEVER DUCTILE SHEAR WALLS (1)	Reinforced Concrete Reinforced Hollow Masonry Timber (2)	1.2 2.5 2.0
CANTILEVER SHEAR WALLS OF LIMITED DUCTILITY (1)	Reinforced Concrete Reinforced Hollow Masonry Timber (2)	1.5 3.0 2.5
DIAGONALLY BRACED FRAMES	Reinforced Concrete Steel Timber	2.5 2.5 3.0
CANTILEVERED SINGLE STOREY FRAMES (5)	Reinforced Concrete Steel	2.5 2.5
CHIW4EYS, SMALL TANKS	Reinforced Concrete Steel	3.0 3.0
OTHER STRUCTURES		See Note4

3.8 Turkish earthquake code

3.8.1 General information

The Turkish building code is: Specification for buildings to be built in seismic zones 2007

This section is based on the Arabian journal for science and engineering International [Ilki, A., Celep, Z., 2012] and Institute of Seismology and Earthquake engineering IISEE. This is a global information network of experts for earthquake disaster and prevention technologies.

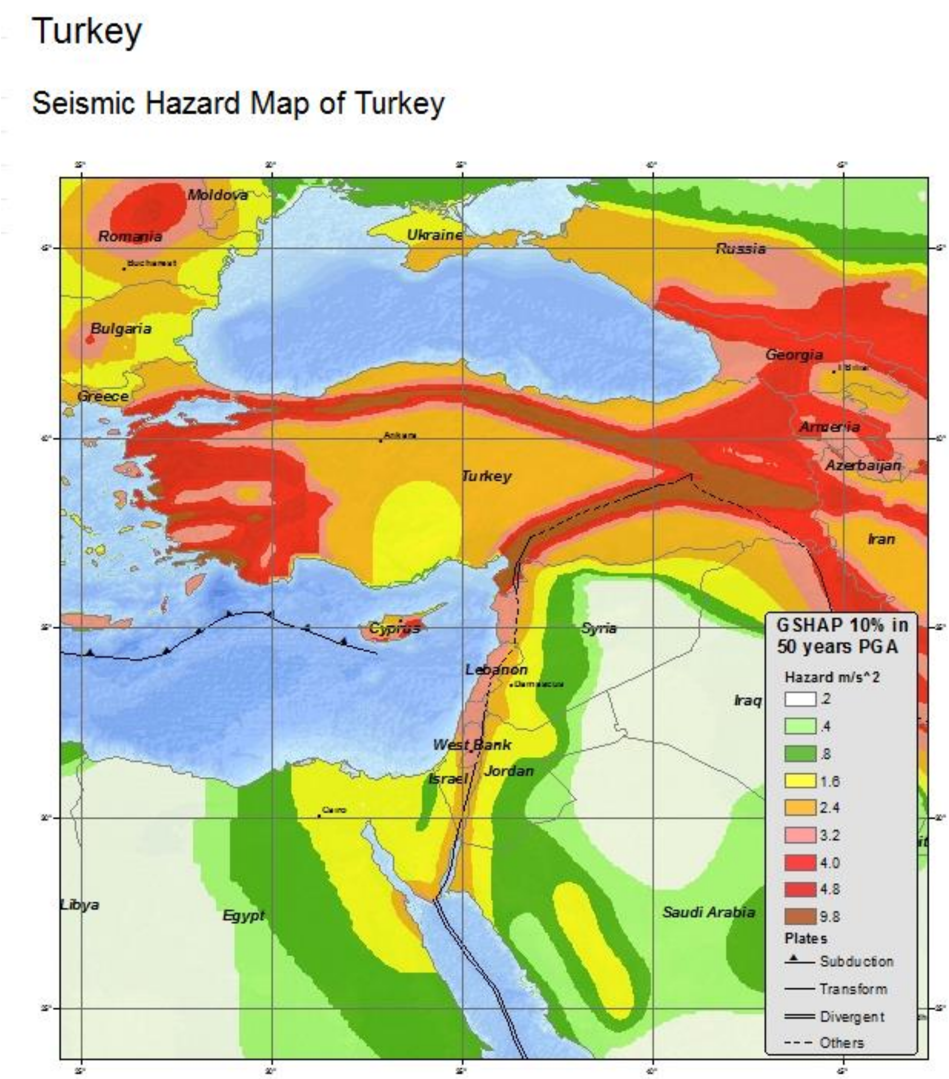

Figure 3.15: Seismic Hazard Map Indonesia [USGS, 2013]

3.8.2 Spectrum coefficient S(T) (response spectrum)

The spectral coefficient for the Turkish earthquake code is given by equations 3.55to 3.57 or figure 3.16 using table's 3.20 to 3.22. Herein is T the fundamental natural period in seconds.

$$0 < T < TA \qquad S(T) = 1 + 1.5\,\frac{T}{Ta} \tag{3.55}$$

$$TA < T < TB \qquad S(T) = 2.5 \tag{3.56}$$

$$T > TB \qquad S(T) = 2.5 \times \left(\frac{Tb}{T}\right)^{0.8} \tag{3.57}$$

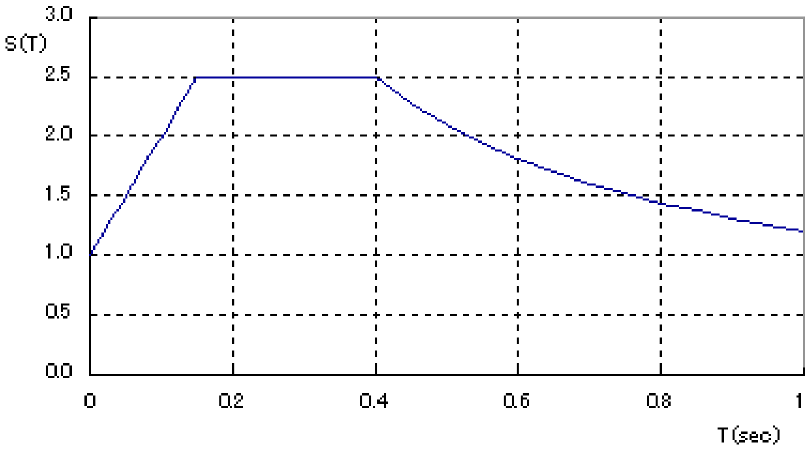

Figure 3.16: Spectral coefficient Turkey [IISEE, 2007]

Table 3.20: Spectrum characteristic period [IISEE, 2007]

Local Site Class According to Table 12.2	T_A (s)	T_B (s)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

Table 3.21: Local site classes [IISEE, 2007]

Local Site Class	Soil Group and Top Layer Thickness
Z1	Group **(A)** soils Group **(B)** soils with $h_1 \leq 15$m
Z2	Group **(B)** soils with $h_1 > 15$m Group **(C)** soils with $h_1 \leq 15$m
Z3	Group **(C)** soils with 15m $< h_1 \leq 50$m Group **(D)** soils with $h_1 < 10$m
Z4	Group **(C)** soils with $h_1 > 50$m Group **(D)** soils with $h_1 > 10$m

Table 3.22: Soil groups A-D [IISEE, 2007]

Soil Group	Description	Standard Penetration (N /30 cm)	Density Relative (%)	Unconf. Comp. Strength (kPa)	Shear Wave Veloc. (m/s)
(A)	1.Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks			>1000	>1000
	2.Very dense sand and gravel	>50	85-100		>700
	3.Hard clay, silty clay	>32		>400	>700
(B)	1.Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity			500-1000	700-1000
	2.Dense sand and gravel	30-50	65-85		400-700
	3.Very stiff clay, silty clay	16-32		200-400	300-700
(C)	1.Highly weathrered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity			<500	400-700
	2.Medium dense sand and grovel	10-30	35-65		200-400
	3.Stiff clay, silty clay	8-16		100-200	200-300
(D)	1.Soft, deep alluvial layers with high water table				<200
	2.Loose sand	0-10	<35		<200
	3.Soft clay, silty clay	<8		<100	<200

3.8.3 Horizontal base shear Vt (base shear force)

The horizontal shear force, or base shear force, is calculated with equations 3.58 to 3.61.

$$Vt = W \times \frac{A(T)}{Ra} \geq 0.1 \times A0 \times I \times W \tag{3.58}$$

$$A(T) = A0 \times I \times S(T) \tag{3.59}$$

$$0 \leq T \leq TA \qquad Ra = 1.5 + (R - 1.5) \times \frac{T}{TA} \tag{3.60}$$

$$T > TA \qquad Ra = R \tag{3.61}$$

Wherein:

W	Is the total weight of the building
$A(T)$	Is the spectral acceleration coefficient
$A0$	Is the effective ground acceleration coefficient, (table 3.23)
Ra	Is the seismic load reduction factor
R	Is the structural behaviour factor. (table 3.24)
T	Is the fundamental natural period in seconds
I	Is the building importance factor, (table 3.25)

Table 3.23: Effective ground acceleration coefficient [IISEE, 2007]

Seismic Zone	A_0
1	0.40
2	0.30
3	0.20
4	0.10

Table 3.24: Structural behaviour factor [IISEE, 2007]

Structural System	Ductility Level	
	Nominal	High
(1) CAST-IN-SITU REINFORCED CONCRETE BUILDINGS		
(1.1) Buildings in which seismic loads are fully resisted by frames	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled shear walls	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls	4	7
(2) PREFABRICATED REINFORCED CONCRETE, BUILDING		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment tranfer	3	7
(2.2) Buildings in which seismic loads are fully resisted by single-story frames with columns hinged at top	-	3
(2.3) Prefabricated buildings in which seismic loads are fully resisted by prefabricated or cast-in-situ solid and/or coupled stuctural walls with hinged frame connections	-	5
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	6
(3) STRUCTURAL STEEL BUILDINGS		
(3.1) Buildings in which seismic loads are fully resisted by frames	5	8
(3.2) Buildings in which seismic loads are fully resisted by single-story frames with columns hinged at top	4	6
(3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
(a) Concentrically braced frames	4	5
(b) Eccentrically braced frames	-	7
(c) Reinforced concrete structural walls	4	6
(3.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) Concentrically braced frames	5	6
(b) Eccentrically braced frames	-	8
(c) Reinforced concrete structural walls	4	7

Table 3.25: Building importance factor [IISEE, 2007]

Purpose of Occupancy or Type of Building	*I*
1. Buildings to be utilised after the earthquake and buildings containing hazardous material a. Buildings required to be utilised immediately after the earthquake (hospitals, dispensaries, health wards, fire fighting and facilities, PTT and other telecommunication, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations) b. Buildings containing toxic, explosive and flammable materials, etc.	**1.5**
2. Intensively and long-term occupied buildings and buildings preserving valuable goods a. Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc. b. Museums	**1.4**
3. Intensively but short-term occupied buildings Sports facilities, cinema, theatre and concert halls, etc.	**1.2**
4. Other buildings Buildings other than above-defined buildings (Residential and office buildings, hotels, building-like industrial structures, etc.)	**1.0**

3.8.4 Lateral force distribution along the height shear Fi of i-th story

The lateral seismic shear for the i-th story is calculated with equation 3.62 using equations 3.63.

$$Fi = \frac{(V - FT) \times Wi \times hi}{\sum_i^N Wi \times hi} \tag{3.62}$$

$$Ft = 0.004 \times V \times \left(\frac{H}{D}\right)^2 \leq 0.15 \times V \qquad for\ \frac{H}{D} > 3 \tag{3.63}$$

Wherein:

Wi	Is the weight at level i
hi	Is the height at level i
H	Is the height of the building
D	Is the depth of the building
V	Is the horizontal base shear (section 3.8.3)

3.9 Basic principles for the seismic design of buildings

3.9.1 Introduction

To this point of chapter 3 the earthquake loading and its causes are elaborated. In this section the basic principles for seismic design of buildings are presented to construct safe, efficient and economic structures. First the main factors are discussed from table 3.7 section 3.4. Afterwards several building configurations are discussed. Configurations which must be avoided as much as possible while designing a building.

3.9.2 Building weight (mass)

The most important factor determining the earthquake force in a building is its weight. Like Newton stated in his second law, the inertia force is proportional to mass, figure 3.17. The heavier an object, the higher the force at a certain level of acceleration and therefore it is desirable to build light-weight structures in earthquakes regions.

3.9.3 Natural period of vibration

Each building has got a natural period of vibration, which is termed as almost equal to first mode of vibration. When earthquake waves, with their chaotic period content, hits the foundation of a building, its structure responds to the various period of vibration that are all mixed-up together to comprise the shaking. The building will eventually resonate. Particularly by low to medium high buildings, most of the dynamic energy of an earthquake resonates in the first mode of vibration and interact with the natural period of vibration of the building. This interaction causes severe damage. The higher the natural period of vibration of the structure, the better earthquake resistance. Parameters of the natural period of vibration are height, mass and stiffness of the structure. The higher a building, the heavier a building, the more flexible or less stiff, the longer the natural period.

3.9.4 Ductility (structural damping)

Ductility of the structure has a large influence upon the magnitude of accelerations and seismic force response. When earthquake shaking exceeds the strength of a brittle member, this member will fail and this could lead to progressive collapse. But when the member is ductile, the material will yield and plastic behaviour will occur resulting in large deformations. During yielding ductile nature absorbs the seismic energy that would otherwise lead to the building experiencing increased accelerations. Ductility depends on material, structure type and detailing.

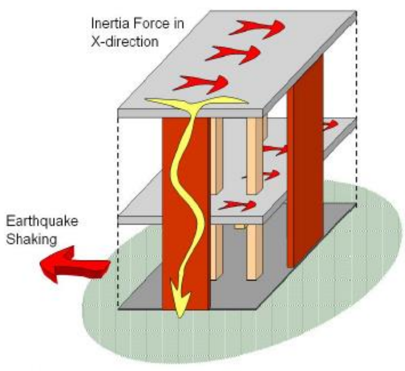

Figure 3.17: Force flow earthquake [Szakats, G., 2006]

3.9.5 Design configurations for earthquake loading

<u>Avoid soft-story ground and upper floors</u>

One of the common collapse mechanisms of buildings under seismic actions is caused by soft floors. Soft floors are storey floors that offer considerably less horizontal resistance as a result of reduced lateral bracing elements. For instance due to functional requirements. The columns between the moving soil or storey layers are damaged by cyclic displacements. Plastic deformations at the top and bottom end of the columns lead to dangerous sway mechanisms with collapse as a result. In figure 3.19 two soft-story configurations are given.

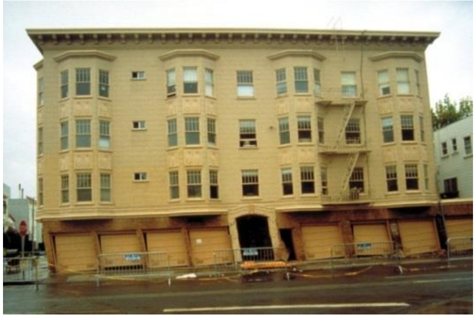

Figure 3.18: Soft ground floor collapse [images.google.nl]

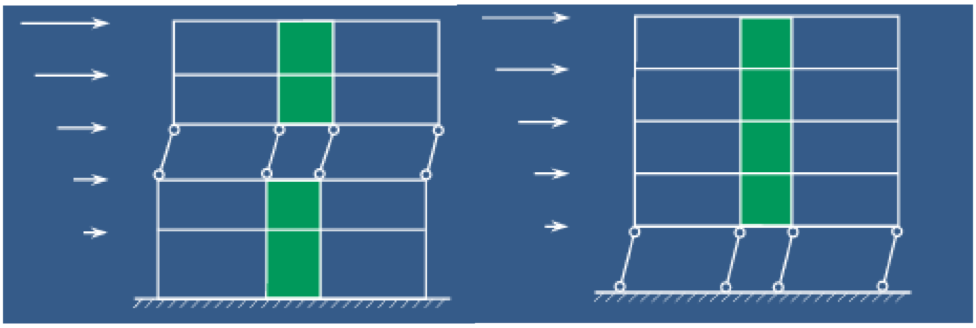

Figure 3.19: Soft-story floor [Bachmann, H., 2003]

<u>Avoid asymmetric bracing</u>

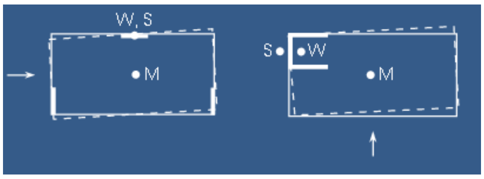

Figure 3.20: Asymmetric bracing [Bachmann, H, 2003]

Figure 3.21: Asymmetric collapse [images.google.nl]

Another frequent collapse mechanism of buildings under seismic actions is caused by asymmetric bracing. Each building has a centre of mass M, a centre of horizontal resistance W and a centre of stiffness S. If the centres of mass M and resistance W do not coincide, eccentricity and twisting occur. The building starts to twists in the horizontal plane, around the centre of stiffness S. Displacements occur between the top and bottom of the columns furthest away from the centre of stiffness, with building collapse as a result. Therefore building twist has to be limited by reducing the distance between centres M and W and sufficient torsional resistance should be available. In figure 3.20 a twisting configuration is given.

Avoid bracing offsets and discontinuities in stiffness

In case of bracing offset, the lateral bracing of a building changes from one storey to the other. These solutions are not efficient regarding the internal force flow and weaken the horizontal resistance and reduce the ductility. These types of structures are vulnerable to earthquake actions and need to be avoided.

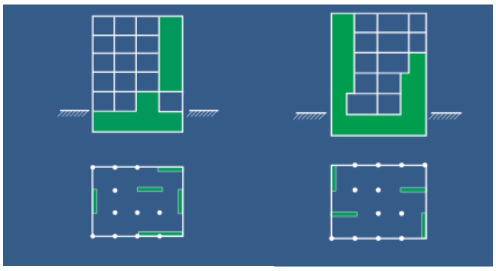

Figure 3.22: Bracing offset and discontinuities in stiffness [Bachmann, H., 2003]

Stiffness discontinuities over the height of the building lead to sudden variations in the stiffness and resistance of the building. It can cause irregularities in the dynamic behaviour of the building. A stiffness increase from the top to bottom of the structure is generally favourable. In figure 3.22 bracing offsets configurations are given.

Avoid mixed structural systems

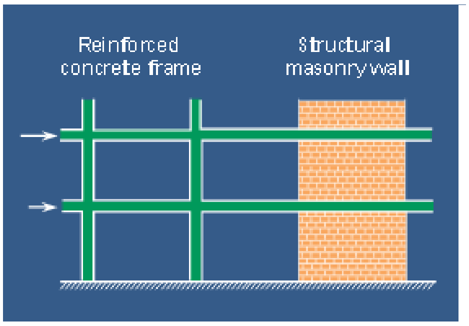

Figure 3.23: Mixed structural system [Bachmann, H., 2003]

Mixed structural systems, combinations of concrete, steel or masonry elements, behave very unfavourably during earthquakes. Stiffer systems, in figure 3.23, the masonry wall, attracts more horizontal loading due to earthquake action. In addition to their own influence zone, the walls must resist the inertia forces from parts of the building with the columns. The calculated resistance must therefore be larger and when a masonry wall fails this usually leads to a total collapse of the building and need to be avoided.

Avoid short columns

Another cause of collapse of buildings during earthquake action is the shear failure of short columns. These are columns that are relatively thick compared to their height and are often fixed in strong beams of slabs. Due to the short column an enormous moment gradient occurs resulting in a large shear force. This often leads to shear failure and this configuration is presented in figure 3.24.

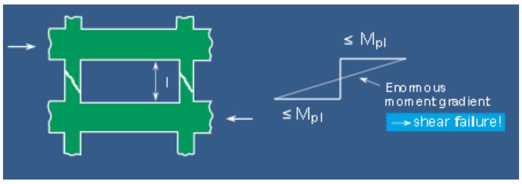

Figure 3.24: Short columns [Bachmann, H., 2003]

Brace masonry buildings

Masonry structures are not well suited for seismic actions. On one hand they are relatively stiff and therefore a high natural frequency. On the other hand unreinforced masonry buildings are brittle and generally exhibit relatively little energy dissipation. Seismic resistance of masonry buildings can be improved using reinforce concrete structural walls, figure 3.25. These walls limit the horizontal deformation of the masonry wall. A possible alternative is to reinforce long masonry walls and stiffen them in longitudinal direction.

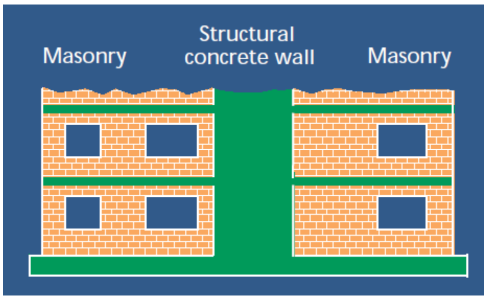

Figure 3.25: Braced masonry buildings [Bachmann, H., 2003]

Match structural and non-structural elements

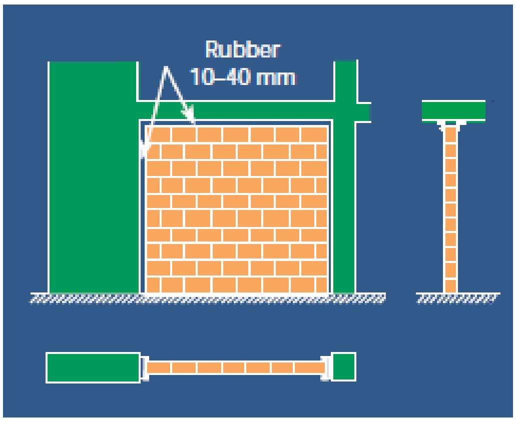

Figure 3.26: Flexible joints [Bachmann, H., 2003]

Substantial damage may occur to non-structural elements, like partition walls and façade, when the connections between structural and non-structural elements are not designed properly. Special joints are often used or the stiffness of the structure must match with the deformation capacity of the non-structural elements.

Soft joints are special joints used in building in earthquake regions. The joints runs along columns, structural walls, slabs and beams and must be filled by a very flexible soundproof material, figure 3.26.

Special design diagonal steel bracings

One of the load bearing structure systems, described in chapter 1, are braced systems. These systems must be designed in earthquake regions with special care. The common truss bracing with centre connections and slender diagonal members may show a very unfavourable behaviour under cyclic actions. The diagonals yield under tension, lengthen more by each cycle and buckle. As a result the stiffness of the structure reduces. Steel truss systems with eccentric connections behave much better than trusses with centre connections.

Separate adjacent buildings by joints

Buildings can collapse due to pounding and hammering of adjacent buildings. This risk is the highest when the floors of both buildings are not on the same level. In that case the floors will hammer against the less stiffer columns. Therefore joints between the buildings must have a minimum width. Special care is required by designing buildings in heavily dense areas.


Figure 3.27: Truss bracing failure [Bachmann, H., 2003]

Secure connections in prefabricated buildings

Prefabricated buildings can be vulnerable to earthquakes, because often the connections of prefabricated structures are designed for gravity load only. Floors and beams frequently collapse due to short support lengths, weak dowels and lateral overturning moments. Therefore bearings must have a minimum support length and the connections and the fixed dowels must have enough plastic deformation capacity. In figure 3.28 a clear representation is given where prefabricated elements failed at the connections.

Figure 3.28: Connection failure prefabricated building [Bachmann, H., 2003]

Foundations measurements

Figure 3.29: Soil liquefaction [Bachmann, H, 2003]

Figure 3.30: Soil liquefaction [Bachmann, H., 2003]

The foundation is the most vital structure of a building. If the foundation fails, the structure above will follow. Often massive rafts are used. Foundation structures should always remain elastic because plastic deformations lead to unpredictable behaviour and additional displacements. Repairing the foundation afterwards will be more difficult and very expensive and therefore the reinforcement must therefore be strengthened below the plastic zones.

Foundation also fails due to soil liquefaction. This is a process whereby the load-bearing capacity of sandy or silty soils, under earthquake vibration, changes and become significantly lower. Entire buildings or sections sink, tilt and collapse. Therefore soils must be studied for their liquefaction potential. In figures 3.29 liquefaction is presented.

Nowadays base isolation is more and more installed in buildings to provide more damping. These measurements are placed in the foundations. See section 2.11 for more information about base isolation

Anchor non-structural elements

To ensure the safety of occupants, also measurements regarding the non-structural elements in a building must be taken into account. These elements, such as façade panels, partition walls ceilings, and internal technical installations can be heavy and therefore dangerous elements during earthquakes. Propitiate anchors are used to ensure collapse. Besides it is very important to guarantee the integrity of installations and equipment that must remain operational after a strong earthquake.

4. Construction methodology

4.1 Introduction

From the previous chapters the most important structural characteristics [chapter 1] and load actions [chapters 2 and 3] are mentioned to be able to design an air traffic control tower around the globe. But these are not the only starting points in making a feasible and appropriate design. In the design of Civil Engineering structures it is the principles of interaction that is of importance. On one hand it is the interaction of design the functional requirements / boundary conditions, on the other hand the interaction between design and execution/maintenance, quoted by [Horst, A.Q.C. van der, 2013], and is presented in figure 4.1. To develop a concept that meets all functional requirement and boundary conditions and at the same time is an economical solution, a methodology should be followed during the conceptual phase. This chapter presents the most important aspects of this methodology.

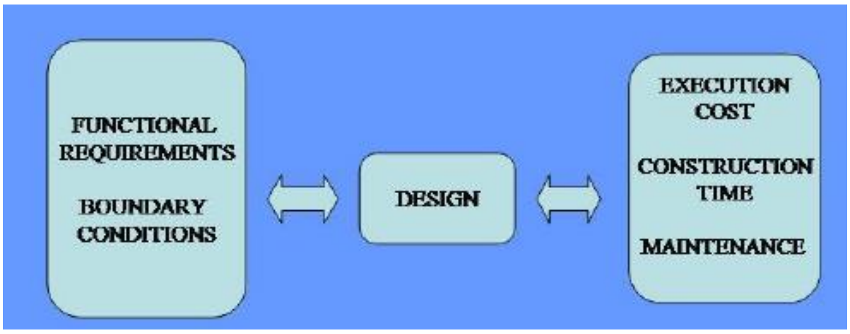

Figure 4.1: Interaction in design [Horst, A.Q.C van der, 2013]

4.2 Design criteria

Several criteria are essential for an optimal integrated structural design. In this section the most important criteria are described which will influence the choice between structural alternatives.

<u>Cost</u>

Cost is the one of the most import design criteria. The largest percentages of cost of a project are made during the execution and they consist out of labour, use of equipment and materials. Changes in the (previous) design phase can have a large impact on the costs during execution. Therefore it is very important to understand what the cost drivers are and to optimize them.

<u>Reliability and risk management</u>

The design and the design choices should allow a strict control of the construction phase. Also for the design a risk analyses shall be performed. Risk is a function of a chance multiplied by a consequence and shall be determined and reduced.

<u>Constructability</u>

Concepts must be checked on constructability aspects. These aspects comprise: Design checks, reliability, tolerances required by construction and development of method statements.

<u>Redundancy aspects</u>

Redundancy is the ability to absorb the unforeseen without disproportional damage.

<u>Maintenance cost</u>

The required amount of maintenance must be reduced as much as possible to reduce costs and nuisance. Also maintenance operations must be as simple as possible and good accessible.

<u>Durability aspects</u>

An important design aspect is the durability during the lifetime of the structure. Local weather conditions can have a large influence on the durability of a structure and have to be taken into account.

<u>Sustainability aspects</u>

The designer should consider the impact of the building, during its entire lifetime, on the local environment. Often certain sustainability levels are required by the client.

<u>Future extension capability</u>

The designer should consider future extension possibilities. E.g. Foundation modifications, to obtain more load-bearing capacity, can be very expensive.

All these design criteria have to be taken into account in all the stages of the design process. To limit the research scope, the most important design criteria will be used for the feasibility of the (preliminary) of the air traffic control tower and are discussed in the following section.

4.3 Cost

4.3.1 Cost components

During the entire life cycle of a project costs are allocated into many aspects, too much to mention. Luckily the majority of the cost is grouped together and starts to occur during the construction phase. These costs are materials, man-hours, rent or purchase of equipment and construction time, subdivided in direct cost, indirect cost and general overhead costs. In this thesis the direct costs will only be considered to determine the optimal preliminary design of an air traffic control tower, because they are in general proportionally influencing the other two cost categories.

4.3.2 Cost of materials

The largest amount of material cost of a building project is dedicated to the structural elements of the building. In chapter 1, the most vital structural elements; the load bearing structure and foundation are presented. The materials used are reinforced concrete and steel.

The cost components for concrete works in civil engineering projects are divided in three units and are given below. In figure 4.2 the cost ratio between the three units are presented.

- Concrete per m^3
- Reinforcement per kg, ton
- Formwork/ falsework per m^2

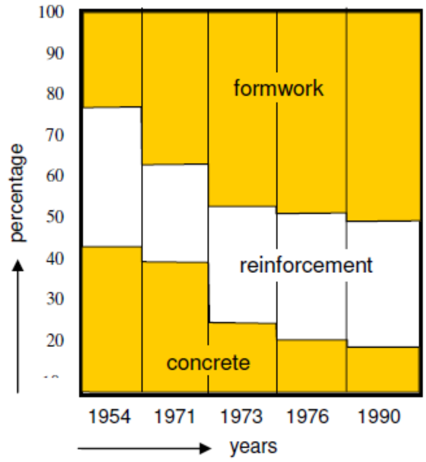

Figure 4.2 Contribution of cost components for concrete work [Horst. A.Q.C. van der, 2013]

The steel prices are given in price per ton per beam element.

These two materials will be discussed in more detail in phase II, regarding their price and availability for the investigated countries. In countries with high material cost (relative to the cost of labour) focus should be given to minimize material demand.

4.3.3 Man-hours

The construction industry is one of the most labor-intensive industries in the world. The labor cost component of a building project often ranges from 30 to 50 % and can be as high as 60 % of the overall project cost. [Elbeltagi, E]. The labour cost will be discussed in more detail in phase II for the investigated countries. In countries with high labour cost (relative to cost of materials) it makes sense to optimize man-hour demand.

4.3.4 Equipment

To optimize cost it important to take a good look at the available equipment. Also site conditions or requirements could limit the use of certain equipment's. The hoisting crane capacity shall always be considered in combination with its reach. Related to the choice of equipment, is the choice of prefabrication or even preassembly of larger units of the structure. New special techniques are nowadays used to reduce the construction time and therefore the execution costs.. Two special techniques are very suitable to construct a concrete core are explained below.

<u>Climbing formwork</u>

By this system the concrete core will be constructed per level by using formwork walls. When the concrete is hardened the same formwork will be reused over and over till the top of the structure is reached. There are two main types of climbing formwork; crane climbing and self-climbing

Slip forming

The slip form system is a hydraulically operated form system. Concrete is poured into the forms in layers of approximately 200 mm. The hydraulic jacks lift the form approximately 25 mm per stroke generally producing a slip forming rate of 300 mm per hour. Slip forming can be done continuously. Therefore it is a rapid and economical construction method with can achieve considerable cost savings.

4.3.5 Construction time

The shorter the construction time, the lower construction costs and the less nuisance in general. Several ways can be obtain to shorten the construction time. When the structure will be built in-situ efficient design considerations can be taken such as; Apply simple, straight and plane shapes. Avoid locally required modification to the formwork and apply large formwork units if possible to reduce the required man hours per m^2.

Also the construction time can be heavily reduced by application of prefab elements. The workers only have to assemble the building on the construction site. By using less different element there will be more repetition in the factory and building site. The construction process is shifted from the building site to the factory. The working conditions of the factor can be better controlled and elements with high quality can be expected. Prefabrication is also very suitable for very small buildings sites. The elements will arrive with the philosophy “just in time” and will be directed hoisted and assembled.

4.4 Interaction between the different engineering disciplines

Large building projects have often a multi-disciplinary character. Therefore attention should be given to the interaction between the different engineering disciplines during the (preliminary) design. In the building engineering these disciplines are structural engineering, electrical engineering, mechanical engineering, process engineering and HVAC engineering.

4.5 Interaction between design and QHSE

Another important interaction in the construction industry is the interaction between design and QHSE (quality, health, safety and environmental care). This interaction can have a large impact on the (preliminary) design of a building, like choice of materials, construction details and use of equipment. Safety can be addressed in the civil engineering in three main aspects. Structural safety; required level specified in building codes, safe working methods and personal safety.

4.6 Life Cycle Cost

In the construction industry nowadays more and more the Life Cycle Cost (LCC) of a project is projected. The definition of the LCC is the sum of all recurring and one-time costs over the full life span or a specified period of a project. This includes purchase price, installation cost, operating costs, and maintenance, upgrade costs and the residual value.

5. Conclusions

The sub-question elaborated in this literature report states "*What are the design characteristics of high and tall buildings in the chosen countries around the globe"* and is answered with the following conclusions.

The most important structural characteristic for high and tall building design is the vital role of the lateral forces, wind and earthquake. To provide enough structural reliability, the design has to fulfil the structural requirements regarding stability, strength and stiffness prescribed in the Eurocode.

Suitable structural systems to construct high and tall buildings with a maximum height of 150 meters are rigid frames, shears walls, braced frames, core structures, tube-structures. These structural solutions are obtained by expressing the optimal height versus optimal structural systems for both concrete and steel structures. Next deep foundations are the most common foundation systems for high and tall structures.

In order to limit progressive collapse, the identification of accidental actions and taking measures will be the main strategy in the design phase. Due to the limit amount of time, this identification will be disregarded in this thesis research.

The local building codes of the chosen countries are very similar compared with the Eurocode. The wind and earthquake codes contain the same factors and approaches and therefore in the following thesis research the Eurocode will be used as normative building code.

The wind force is determined with three parameters. Whereby, the design wind speed and the aerodynamic factor are the most important parameters regarding the structural design. The gust response factor does not have a vital role and this parameter will be disregarded.

The magnitude of the earthquake force is equal to the mass of the building multiplied with the ground acceleration. In general, this load action is (very) decisive (in comparison with wind) and stricter requirements are taken as prescribed in the Eurocode.

The criteria which will be taken into account for optimal integrated structural design are; cost, reliability, quality, constructability and future extensions capability. These criteria are considered as most important and are expected to give the most interesting results regarding the international, structural and economical focus.

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