Timber stadium engineering

a feasibility study

Annexes

31 October 2011

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bam
utiliteitsbouw
# Annexes

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Annex A Study on literature

Annex A.1 Timber as a building material

A.1.1 Introduction

Being a natural and renewable material, timber is one of the most sustainable materials available on the market. That is to say, when being harvested from a sustainable forest. Compared to steel and concrete, the production of timber products requires a very small amount of energy, see figure 170 and figure 171. It should be mentioned that the scales of these figures differ, since different reference projects have been used. Only the ratio between timber and concrete/steel should therefore be taken into consideration.

![Figure 170: Environmental harm of a timber frame construction versus a reference steel construction (measured in Minnesota)](image)

![Figure 171: Environmental harm of a timber frame construction versus a reference concrete construction (measured in Atlanta)](image)

The material has been used in building structures for ages, and it still is the most popular construction material in the world. The reason for this can be found in its many advantages. Timber is a very versatile material which is available in many different species, sizes and shapes. It can therefore be used in a wide variety of applications, such as beams, columns, trusses, piles, plating, formwork, etc. For some decades, composite and engineered timber products have become available on the market, stretching the possibilities of the utilisation of timber within building structures. These products have overcome the limitations on cross-sectional dimensions and spans. Laminations techniques have made it (theoretically) possible to produce timber products in every size, length or shape. The only limitations here are manufacturing and transportation.

Another big advantage of timber is the very high strength to weight ratio and the low ratio of costs to strength, especially when short or medium spans are considered. The material can be both used as a tensile and a compression member, although tensile members might lead to large connections. Compression joints on the other hand, are relatively simple connections.

Timber can also be an extremely durable material, when being treated, built and detailed properly. Timber is susceptible to biological attacks, from both fungi and wood-boring insects, as well as from corrosion of steel elements. Preservative measures can prevent these problems from occurring.

When fire safety is being discussed, people often tend to think that wood is a very precarious material to use. Despite the fact that wood is a combustible material, this thought is certainly not justifiable. Wood has the advantageous property to act self-protective when being ignited. By forming an insulating char
coal layer, the material protects itself from further burning. On the other hand, the emission of fumes and gasses often causes problems.

In this annex, a partial answer is provided on the first sub questions as presented in the Introduction. As a reminder, this question is shown here:

ß What kind of timber species are available to use? What are their main applications and what are their characteristics?

In this annex, the main focus will be on the material its characteristics and their backgrounds. It can therefore be regarded as being a general introduction on the material wood. At first, attention will be given to the structural composition of the material, making a distinction between softwoods and hardwoods. Next, the durability and the behaviour in fire will be discussed. Finally, available certificates and marks are introduced and a general distinction is made between soft- and hardwoods.

References: [24] [26] [27]

A.1.2 The material

Timber is obtained from the living organism that is commonly known as a tree, which exists in many different species with many different properties. Among the 30,000 known species, there are trees that can reach ages of about 5000 years, trees that can reach a height of 135 m and trees that can reach diameters up to 12 m. The timber that is generally used as a building material, is obtained from about 1500-3000 different species, with ages mainly between 60 and 140 years old.

When taking a closer look to the structural composition of a tree, one can distinguish three typical organs: roots, trunks and leaves (or needles). For building purposes, primarily the trunk is of value. The trunk consists of various elements, such as heartwood, sapwood and cambium, see figure 172.

The timber closest to the bark is called the sapwood; this is the youngest wood in the trunk and (for most species) lighter of colour. The thickness of this layer is about 25-170 mm depending on the specific species. The sapwood contains both dead and living cells, and its main purpose is the transport of water and minerals from the roots to the leaves.

The layers in the centre of the trunk are called the heartwood, consisting of more or less inactive cells. The heartwood provides the trunk with stiffness and mechanical support. Heartwood is constantly evolving from sapwood during the life of the tree.

The strength and the weights of both are more or less the same, but the sapwood has a lower natural resistance against attacks by fungi and insects.

In countries with a modest climate, a new layer of wood is produced every year at the beginning of the growth season. During the winter months, the growth stops, leaving a clear circular mark, known as a annual ring. In tropical countries, this is generally not the case. Here, the timber grows more or less constant throughout the year. The wood that is formed in the beginning of the growth season is usually
called early wood (or springwood), while the slower growing wood that is formed at the end of the season is called late wood (or summerwood).

Considering the elements that can be found in the wood of the tree, it is found that a tree consists for almost 50% out of carbon, for about 44% out of oxygen and for about 6% of hydrogen. The structure of the material is mostly determined by the various types of cells. One can distinguish support cells, conduction cells and storage cells. Most of them being relatively long and slender, and therefore known as fibres. Their direction is, for the great majority, longitudinal within the trunks’ cross-section. An exception are the so called rays, in which storage and transport takes place, see figure 172. The cells’ walls are made of cellulose and they are kept together by a bond of lignin.

This being known, the composition of timber can be compared with a bundle of tubes in the direction of the grain, considering low bonding between the various bundles. This composition leads to totally different properties in different directions, and even differences between the bundles their selves. This phenomenon is known as anisotropy, with the consequence that, as far as timber is concerned, the strengths parallel and perpendicular to the grain show great variation.

The exact grain structure of the material is determined by the position and direction of its cells in respect to each other, and the development of its growth rings. This varies for all species, leading to different behaviour and different characteristics.

Since timber is a hygroscopic material, the moisture content correlates with the surrounding climate. Changes in moisture can therefore lead to shrinkage or swelling, both parallel and perpendicular to the grain. It can also lead to changes in mechanical properties, i.e. strength and modulus of elasticity. When designing, one has to bear this in mind, since restrained shrinkage or swelling may lead to internal stresses and eventually relaxation. As a general rule, the timber being used should have a moisture content corresponding to the relative humidity of its environment. Within buildings, the maximum content will therefore be around 12-15%.

Before further elaborating, the structural composition of wood will be discussed. Due to their clear differences, a distinction will be made between two different kind of plants where timber can be obtained from, namely softwoods and hardwoods. The hardwoods are obtained from the so-called angiosperms, or deciduous trees, while the softwoods come from the gymnosperms, or coniferous trees.

References: [24][25][27]

A.1.3 Structural composition

A.1.3.1 Softwood

Softwoods have a relatively simple structure, consisting almost entirely out of (one single type of) long (2-5 mm), slender cells with closed ends. These cells are called tracheids and they are arranged in a radial pattern around the centre of the trunk, being orientated in the longitudinal (grain) direction of the trunk. Their main functions are the storage of nutrition and the transport of liquids, to wherever necessary, see figure 173.
When a tree evolves, the cell structure changes. Considering early wood, the cells have a large diameter and a thin cell wall, while with latewood, the diameters have decreased and the cell walls have become thicker, see figure 171. The difference expresses itself when the densities of latewood and early wood are compared to each other: for softwoods this ratio can become as high as 3:1.

The sap stream between cells passes through small openings in the cell walls, known as pits. The majority of these pits are bordered, which means that they prevent the water to move freely and they prevent the entry of air in sap-filled cells. These bordered pits have a negative effect when the timber is being dried, since capillary forces prevent the water from being extracted from the cells. The same behaviour impedes later impregnation treatment.

A.1.3.2 Hardwoods

Although the structural composition is similar to a large extent, hardwoods are more complicated and more varied. The tracheids are enclosed by a protective tissue, which contains so-called ‘conducting vessels’. These vessels can be thought of as pipes with lengths ranging between a few centimetres and a few meters, consisting of different elements with open or perforated ends, see figure 174. Compared with softwoods, the wall thickness of the cells are much bigger for hardwoods. In addition, the difference in density between early wood and latewood is not as big as for softwoods.

Within the heartwoods, two different types can be distinguished, namely diffuse-porous and ring-porous hardwoods. When the tree has the necessity to grow leaves every year, the demand for sap is higher in the spring. It is therefore possible, that larger vessels may occur in the springwood: ring-porous
hardwood. When there is no specific grow period, the vessels are more evenly distributed: diffuse-porous hardwood.

References: [ 24 ] [ 25 ] [ 27 ]

**A.1.4 Durability**

As mentioned earlier, timber is susceptible to both biological attacks and corrosion of metal components. The biological attacks can be divided in two types: fungal attacks and insect attacks. Corrosion can be prevented by applying protection measures, such as a protective coating.

Fungal attacks take place when the moisture content of the material increases to above 20%. The optimal moisture content varies between the various wood species. The presence of fungi can reduce the load bearing capacity to a certain extent, depending on kind of species and the extent of attack. To prevent problems from happening, in the design one should take certain measures. For example, wetting should be prevented wherever possible.

Insect attacks are influenced by the temperature, since insects tend to multiply under warm conditions. The appearance of cracks and splits may provide unprotected areas within preserved timber structures. This could lead to a reduced load-bearing capacity and should therefore be prevented as much as possible.

The natural durability of the material varies for and within all species, but generally softwoods are more susceptible to attacks than hardwoods. The reason for that can often be found in the percentage of heartwood which is present in the material. The sapwood region is often less dense (depending on species and insect type) and therefore more susceptible to insect attacks. In general, it can be said that the more sapwood, the more susceptible to insect attacks, but this has to be checked for each specific case.

If the natural durability does not suffice, a preservative treatment can be applied. The effect of such a treatment depends on the treatability of the timber, i.e. the absorption of the preservative and its penetration depth. The use of preservatives depends on several aspects, such as the natural durability of the material, its resistance to penetration by the preservatives, the application of the element, the service life and the ease of maintenance. All these aspects have to be considered to make a deliberate choice. Now, the most important treatment methods will be discussed.

**A.1.4.1 Treatment methods for timber**

Before elaborating on the possible treatment methods, a distinction should be made between preserved timber and modified timber.

When timber is preserved, a protective layer is formed around the edges of the material. This layer can be applied at the surfaces of the material, but as well within the material, depending on the specific risks that are at stake. Two methods can be named which are used to preserve the timber, being the vacuum-and-pressure method and the dipping method. Within elaborating into detail it can be said that for the first method, the wood is placed in a vacuum after which the preservative is brought in under pressure. For the latter, the wood is dipped in the specific preservative for some time. At this time, various preservatives are available on the market. Despite the fact that the preservatives are usually chemical substances, the treatment method is certified by the government, coming with high demands towards the environment.

When modified timber is at stake, two different types can be distinguished, being chemically and naturally modified timber. Their main difference lies in the applied treatment.

Timber can be naturally modified by artificially adapting the chemical structure of the substances which the timber holds. The natural chemical processes that occur in the timber, are then used as a basis for the modification. A possibility is modification by heating, which reduces the amount of glucose holding wood polymers. As a result, after drying, the timber can hardly absorb water, increasing the durability of the material. The obtained product can be considered as being environmental friendly. As a negative consequence, the strength of the material reduces a little and the colour turns darker.
When timber is chemically modified, the molecular structure of the material is changed by making use of chemicals. The exact process is determined by the used chemical, but in general, the weak hydroxyl sets in the material are replaced by a chemical set. As a result, the material can hardly absorb water, reducing the effect of fungal attacks. The timber itself becomes more heavy and stiffer, without reducing the strength or changing the colour of the material.

A relatively new modification process, is modification by making use of enzymes. Until now, the utilisation is limited to laboratories, where experiments on the material take place. The enzymes should provide an accelerated oxidation of the wood fibres, which interconnects them as a result. Their main application is expected in the board industry, since it could replace chemical bonding products.

### A.1.4.2 Use classes

Within NEN-EN 335-1 several use classes are defined with respect to the risk on biological attacks:

- **Use class 1** applies when the timber or wood-based product is under cover, fully protected from weather and not exposed to wetting.
- **Use class 2** applies when the timber or wood-based product is under cover, fully protected from weather, but when high environmental humidity can lead to occasional, but not persistent wetting.
- **Use class 3** applies when the timber or wood-based product is not covered and not in contact with the soil. It is either continually exposed to the weather or protected from weather, but subjected to frequent wetting.
- **Use class 4** applies when the timber or wood-based product is in contact with the ground or with fresh water and thus permanently exposed to wetting.
- **Use class 5** applies when the timber or wood-based product is permanently exposed to wetting by salted water.

Compared with the service classes as defined in NEN-EN 1995-1, see table 53, it can be mentioned that use class 1 complies with service class 1, use class 2 with service class 2 and use class 3,4 and 5 with service class 3.

<table>
<thead>
<tr>
<th>Service class</th>
<th>Relative moisture content</th>
<th>Moisture content timber</th>
<th>Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>max 65%, few weeks per year above 65%</td>
<td>around 12%</td>
<td>20°C</td>
</tr>
<tr>
<td>2</td>
<td>max 85%, few weeks per year above 85%</td>
<td>around 20%</td>
<td>20°C</td>
</tr>
<tr>
<td>3</td>
<td>above 85%</td>
<td>above 20%</td>
<td>20°C</td>
</tr>
</tbody>
</table>

When use classes 1 and 2 are considered, low levels of natural durability or light preservative measures are required to ensure long-term performance, see table 54. When use classes 4 and 5 are considered, special attention must be given to preservative treatment. For now, no further elaboration will be given on the available preservatives and possible treatment methods.

<table>
<thead>
<tr>
<th>Use class</th>
<th>Timber moisture content</th>
<th>Durability class(1) considering a design life span of 25 years</th>
<th>Durability class(1) considering a design life span of 10 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Permanent &lt; 20%</td>
<td>1 - 2 - 3 - 4 - 5</td>
<td>1 - 2 - 3 - 4 - 5</td>
</tr>
<tr>
<td>2</td>
<td>Incidentally, short-lived &gt; 20%</td>
<td>1 - 2 - 3</td>
<td>1 - 2 - 3</td>
</tr>
<tr>
<td>3</td>
<td>Regularly, short-lived &gt; 20%</td>
<td>1 - 2 - 2 to 3</td>
<td>1 - 2 - 3</td>
</tr>
<tr>
<td>4</td>
<td>Permanent &gt; 20%</td>
<td>1 - 2</td>
<td>1 - 2</td>
</tr>
<tr>
<td>5</td>
<td>Permanent &gt; 20% and in contact with the ground</td>
<td>1</td>
<td>1 - 2</td>
</tr>
</tbody>
</table>

(1) assuming that the material is in ground contact

When the durability classes are concerned, NEN-EN 350-2 distinguishes 5 classes, class 1 being very durable (life span exceeding 25 years) and class 5 being not durable at all (life span less than 5 years).
These classes are based on the assumption that the materials are in ground contact. Therefore, if they are not in ground contact, better behaviour can be expected.

In table 54, the relation between use classes and durability classes is shown for various design life spans. The durability classes are based on the resistance against fungal-attacks, rather than their resistance to insect attacks. For each timber species, the accompanying durability class is determined by the performance of the heartwood. The sapwood does, in almost all cases, not resist any fungal attacks at all. For the natural durability and treatability of certain softwood and hardwood species, reference is made to NEN-EN 350-2, table 2 and 3.

References: [ 24 ][ 25 ][ 26 ][ 27 ]
A.1.5 Strength grading

To gain some understanding about the differences in strength properties between the various wood-based products, it is of importance to have some knowledge on the background of the strength determination.

The strength of the material timber depends on various aspects, such as species, size, density and moisture content. Besides that, there is a big difference in strength parallel and perpendicular to the grain.

![Figure 175: Typical stress-strain diagram for timber](Image)

Without further elaborating, the strength of the material is determined non-destructively through visual or machine strength grading. Within each species or even within each tree, there is a big variance in strength properties between the various cross-sections. The ratio of strength between the strongest and the weakest section might even become 10:1 for some species.

To deal with this variance in a conservative matter, the strength of the timber is determined by its characteristic strength value, being the lower 5-percentile of its value, see figure 176. The majority of the material will therefore not be utilised to its maximum capacity. To prevent problems from happening when a piece from the lower 5-percentile is used, several modification factors are applied on the material and load factors are used on the loads.

Another important strength determining aspect within the variance is the difference between the lower 5-percentile and the mean strength. When the differences are small, the material can be used more efficient. In figure 176, strength distributions are shown for various timber species.

![Figure 176: Examples of strength distribution in structural timber](Image)

As for now, a short explanation will be given on the various strength parameters that exist within structural timber. These parameters will be mentioned later on, when discussing the various forms of appearance of structural timber.
Characteristic value of the bending strength, parallel to the grain

Characteristic value of the bending strength, perpendicular to the grain

Characteristic value of the tensile strength, parallel to the grain

Characteristic value of the tensile strength, perpendicular to the grain

Characteristic value of the compression strength, parallel to the grain

Characteristic value of the compression strength, perpendicular to the grain

Characteristic value of the (panel) shear strength

Characteristic value of the planar (rolling) shear strength parallel to the grain

Characteristic value of the planar (rolling) shear strength perp. to the grain

Mean modulus of elasticity, parallel to the grain

Mean modulus of elasticity, perpendicular to the grain

Mean value for the shear modulus

Mean value for the density

Mean value for the density
In addition, it has to be mentioned that only the most occurring situations have been shown above. For specific panels, also a difference is made in stresses acting edgewise and flat wise. These load configurations will be elaborated when they apply. References: [24][25][27]

A.1.6 Behaviour in fire

To provide a proper description of the behaviour of timber members under the influence of fire, it is of major importance to have some understanding about fire itself. A fire can be roughly divided into two phases: the developing phase and the fully developed phase. The developing phase is correlated with the ease of ignition, combustibility of the material, the heat release rate and the speed of fire-spread across its surface. The fully developed phase is reached when all combustible materials take part in the fire. The various stages within fire development can be found in figure 185.

![Figure 185: Standard fire curve](image)

Fire resistance is generally being described as the ability to resist the fully developed fire, referring to an element rather than a material. It is influenced by the end conditions of the element and the magnitude and distribution of its loading.

Though timber is a combustible material, the speed in which flames spread across its surface is reasonable (compared to other materials like steel and concrete). Considering the fully developed phase, the exposed surface will ignite, but eventually an insulating charcoal layer will be formed. This layer protects the core section from being heated, which has great advantages. For example, thermal expansion problems are being prevented and the physical properties of the material remain the same after a fire. Therefore, the load bearing capacity of timber will only reduce due to the reduced cross section of the element.

The bigger the ratio between surface area and volume, the easier the material ignites and the faster the flames spread. In addition, also defects result in less favourable fire behaviour. The higher the density though, the less combustible and the longer it takes before the material ignites. It goes without saying that the same applies for the moisture content. As a result, softwoods and beech are more combustible than glulam and LVL. The behaviour of hardwoods is comparable to that of glulam and LVL. Panels are most combustible due to their high ratio between surface area and volume.

To ameliorate the fire resistance of the timber, it is possible to apply flame-retardant chemicals. The use of these chemicals depends, amongst others, on the required performance and the conditions under which the element is used.

The combustibility and fire resistance of building materials is determined in certain regulations, such as the NEN-EN 1995-1-2 and NEN-EN 13501-1. The fire resistance is a material property which designates the time that the material remains functional, when exposed to a fire. The specific requirements on fire resistance, depends on the function of the structural component.

Within the building regulations a distinction is made between floorings and other construction products. For both, 7 Euro fire classes can be distinguished being A1, A2, B, C, D, E, F. For floorings the subscript fl is added to the class.
A1 can be considered as being the highest class, not contributing to a fire at all. Class F refers to products that do not comply with class E, or products that are not tested. Products in this class are regarded to be highly flammable. The above is elaborated for construction products in table 55 and for floorings in table 56.

In addition to the above mentioned fire classes, also certain classes for the development of smoke can be distinguished being s1, s2 and s3. Class s1 means that little smoke production is to be expected, s2 means average smoke production and class s3 means a big emission of smoke.

The same sort of classification can be applied on flaming droplets, distinguishing classes d0 to d2. Class d0 means that no flaming droplets may occur (within 600 s), class d1 means that no flaming droplets may occur (within 600 s) persisting longer than a certain time span (10 s), class d2 means that there are not limitations on the appearance flaming droplets.

<table>
<thead>
<tr>
<th>Euro class</th>
<th>Smoke class</th>
<th>Droplets class</th>
<th>Contribution to fire</th>
<th>Combustibility</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>s1, s2 or s3</td>
<td>d0, d1 or d2</td>
<td>Not at all</td>
<td>Virtually not</td>
<td>Natural stone</td>
</tr>
<tr>
<td>A2</td>
<td>s1, s2 or s3</td>
<td>d0, d1 or d2</td>
<td>Hardly</td>
<td>Hardly</td>
<td>PVC</td>
</tr>
<tr>
<td>B</td>
<td>s2 or s3</td>
<td>d0, d1 or d2</td>
<td>Limited</td>
<td>Average</td>
<td>Heavy timber</td>
</tr>
<tr>
<td>C</td>
<td>s1 or s2</td>
<td>d0, d1 or d2</td>
<td>Average</td>
<td>High</td>
<td>Most timber</td>
</tr>
<tr>
<td>D</td>
<td>s1 or s2</td>
<td>d0, d1 or d2</td>
<td>High</td>
<td>Very high</td>
<td>Some synthetic</td>
</tr>
<tr>
<td>E</td>
<td>-</td>
<td>d2</td>
<td>Not determined</td>
<td>Not determined</td>
<td>Not determined</td>
</tr>
<tr>
<td>F</td>
<td>-</td>
<td>-</td>
<td>Extremley</td>
<td>Not determined</td>
<td>Not determined</td>
</tr>
</tbody>
</table>

Table 55: Euro fire classes for construction products, except for floorings

<table>
<thead>
<tr>
<th>Euro class</th>
<th>Smoke class</th>
<th>Contribution to fire</th>
<th>Combustibility</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>None</td>
<td>Not at all</td>
<td>Natural stone</td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td>s1, s2 or s3</td>
<td>Hardly</td>
<td>Virtually not</td>
<td>Natural stone</td>
</tr>
<tr>
<td>B</td>
<td>s1, s2 or s3</td>
<td>Limited</td>
<td>Hardly</td>
<td>PVC</td>
</tr>
<tr>
<td>C</td>
<td>s1, s2 or s3</td>
<td>Average</td>
<td>Average</td>
<td>Heavy timber</td>
</tr>
<tr>
<td>D</td>
<td>s1, s2 or s3</td>
<td>High</td>
<td>High</td>
<td>Most timber</td>
</tr>
<tr>
<td>E</td>
<td>-</td>
<td>Very high</td>
<td>Very well</td>
<td>Some synthetic</td>
</tr>
<tr>
<td>F</td>
<td>-</td>
<td>Not determined</td>
<td>Not determined</td>
<td>Not determined</td>
</tr>
</tbody>
</table>

Table 56: Euro fire classes for floorings

For passable areas, at least fire class D, is demanded. For escape routes at least class C, should be taken into account. When timber is concerned, it should be mentioned that most solid timber products are not yet tested to the Euro fire class standards. The majority of engineered wood products are already certified with a CE-mark (see section certifications and marks), which also tests the compliance to the Euro fire classes.

When a product falls under a certain fire class, something can be said about the behaviour of that product when exposed to a fire. Not only quantitative, as in table 55 and table 56, but also qualitative. Aspects that are considered are, for example, the FIGRA (Fire Growth Rate index) and the THR (Total Heat Release). Since most timber products fall in class C or D, these values will be shown here, to provide the reader an idea of the order of magnitude of these values, see table 57.

Table 57: Qualitative meaning of Euro fire classes

<table>
<thead>
<tr>
<th>Euro class</th>
<th>FIGRA [W/s]</th>
<th>THR_{total} [MJ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>≤ 250</td>
<td>≤ 15</td>
</tr>
<tr>
<td>D</td>
<td>≤ 750</td>
<td>-</td>
</tr>
</tbody>
</table>

References: [24] [25] [26] [27] [46]
A.1.7 Certifications and marks
Throughout the years, various instances have committed their selves to environmental protection. On the field of timber, the major concerns are on forestry, illegal trade and extinction of certain species. To stand up against these concerns, various treaties have been concluded between several nations. For example, the Convention on International Trade in Endangered Species of Wild Fauna and Flora (CITES) is signed between 173 countries and aims to protect around 35,000 species, both flora and fauna, from extinction.

When discussing the origin of timber, a difference has to be made between demonstrable legal wood-based products and demonstrable sustainable wood-based products.

A.1.7.1 Legal wood-based products
Facts of the UN show that around 20 to 40% (350-650 million m$^3$/year) of the annual production of timber can still be considered illegal. Illegal logging can be defined as the winning or trading of timber in contradiction with the applying laws in the originating country. This concerns logging without licenses, logging of too big volumes and the logging of protected species. Around 16-19% of the total amount of timber annually imported by the EU, can be considered to be illegal, according to the WWF.

Illegal logging has negative ecological (e.g. disturbed equilibrium), social (e.g. capitalization) and economical (e.g. unfair competition) consequences. Therefore, in the European Union (EU), the Forest Law Enforcement, Governance and Trade (FLEGT) was initiated, to resist illegal logging and trading. The main goals of the covenant are: 1) to resist the ecological and social harm caused by irregular and illegal logging, 2) to raise the income of the often poor manufacturing countries and 3) to prevent illegal timber to concur against sustainable produced timber. Manufacturing countries that are interested to participate in this plan of action can sign a treaty with the EU.

The legality certification is generally approved by the producing country itself. An exception are the so-called demonstrable legality certificates which are assigned by independent institutes, having their own assessment criteria. Examples are OLB, VLO, VLC and TLTV. If the timber is provided with a ‘Keurhout-Legaal’ mark, it can be concluded that the timbers’ legality certificate complies with the demands stipulated by Keurhout, a Dutch foundation which assesses timber certificates.

![Keurhout trademarks: Keurhout Legal and Keurhout Sustainable](image)

If a wood-based product is certified as being legal, this only implies that the product is obtained in compliance with the applying legislation. The certificate guarantees the legality and origin of the timber.

A.1.7.2 Sustainable wood-based products
In addition to legal wood-based products, sustainable timber implies that attention is given to ecological, social and economical aspects as well. The focus is on sustainable forestry, which means that systems are created and implemented which allow forests to continue a sustainable continuation of environmental supplies and services [ 39 ]. Also social aspects, such as the human rights of native people, the conditions of employment and the stimulation of local employment, take major part in sustainable forestry.

Next to sustainable forestry, also the so-called Chain of Custody (COC) is concerned. COC certification means that all companies that take part in the commercial chain, can guarantee that the certified products are separated at all times from non-certified products. By this means, it is possible to trace the timber all the way back to its origin.

In the Netherlands, the government is committed to a 100% sustainable procurement as from 2010. The criteria for the purchase of sustainable timber are determined as the Dutch Procurement Criteria for
Timber. The compliance of the existing certificates is checked by the Timber Procurement Assessment Committee (TPAC). These certificates are assessed on several criteria, being Sustainable Forest Management (SFM), Chain of Custody and logo use (COC), Development, Application and Management of certification systems (DAM) and the Procedure on Endorsement of certification systems by a Meta-system (PEM).

One should note that the TPAC does not certify or assess wood products on Chain of Custody or Sustainability by itself. It only checks if the certification system complies with the Dutch purchase criteria for timber.

As far as sustainable wood-base products are concerned, more or less two different certificates can be distinguished. Both will be discussed now:

FSC: Forest Stewarding Council
The FSC is an independent international organisation, founded in 1993 by international environmental and human rights organisations, some local natives and companies with interest in forestry or timber. The FSC was the first organisation to provide international applicable principles and criteria for sustainable forestry and certification, with the aim to protect tropical forests against destruction and irreparable damage.

The certificate stands for timber which is obtained from forests with a responsible forest management. The certificate is only issued if the forestry complies with a set of guidelines drawn up by the FSC. It guarantees the forests’ compliance to (inter-)national regulations, the long-term viability of the forest, preservation of local biodiversity and the presence of forest management plans. Besides the forestry certificate, also chain of custody certificates can be obtained.

The FSC certificate complies with all the criteria set up by the TPAC and can therefore be considered as a proof for demonstrable sustainable timber. As from June 2009, the FSC certificate is also accepted as being Keurhout-duurzaam.

The FSC certificate is considered to be the only mark which covers appropriate forestry to an acceptable extent, according to the WWF, Greenpeace and Friends of the Earth.

PEFC: Programme for the Endorsement of Forest Certification schemes
The PEFC is an independent international forest certification system based on the mutual recognition of national and regional forest certification systems. The goal of the programme is the promotion of an international framing for forest certification systems in Europe and worldwide recognition of the certificates.
The PEFC was founded in 1999 by several organisations originating from 5 European countries. In 2004 the system was transformed from being a regional to a worldwide system. Currently, PEFC is the biggest forest certification system worldwide.

The certificate is awarded to timber that originates from sustainable forest management. It can be applied on both forestry and the total Chain of Custody. Although the basis for certification is standardized, the exact implementation can differ for every individual country. The criteria which are being used, originate from both the ‘Earth Summit 1992’ and several international conventions, such as ISO and CBD.

Within the PEFC, about 28 worldwide certification systems are being accepted. Among them are the SFI, CSA, ATFS, PAFC and the MTCS, see figure 190. These certificates comply with the demands asked by the PEFC and are therefore allowed to carry the PEFC-certificate.

As from June 2010, PEFC certified timber is accepted as a proof for demonstrable sustainable timber by the Dutch government. Only when the PEFC certificate is awarded to MTCS certified timber, the timber is not (yet) accepted as being demonstrable sustainable. Wood-based products supplied with a PEFC certificate are automatically accepted as being Keurhout-duurzaam.

Although the goal and approach of both the FSC and the PEFC are more or less the same, the institutes do not work together at this time.

The FSC has its origin in the protection of tropical forests, including the ameliorating of social aspects within these regions. The PEFC had its main focus on the softwoods in Northern Europe, where the social aspects were already at acceptable standards.

Another difference between the certificates is their specific appliance. The FSC certificate has the same demands for every part of the world, while the PEFC certificate has specific demands for every country based on that specific countries’ legislation.

In general, the costs for the FSC certificate are somewhat higher, leading to higher cost prices for the material. This might be a reason for some manufacturers to choose for the PEFC instead of the FSC.

In the Netherlands, both certificates are accepted as representations of sustainable timber by the TPAC and Keurhout. Therefore, it can be concluded that there are no moral restrictions on the use of both FSC and PEFC certified wood-based products.
A.1.7.3 Product certificates

Next to these ‘environmental’ certificates, wood products are often provided with certificates concerning product performance. These have nothing to do with the origin of the timber, but only with the applying standards and the products’ reliability. These certificates are issued on the basis of experiments and test results.

When product certifications are concerned, the focus is merely on the compliance to legislation and general rules. In some cases, also the quality of the product will be taken into account. For now, only the most important certifications are discussed.

CE: Conformité Européenne

The CE-mark guarantees that the product is in compliance with the European rules. In addition, the manufacturers take responsibility for possible negative effects of the use of the product. The mark is not meant as a quality warranty for customers, but it guarantees that the product complies with relevant safety demands and that the product is suitable for its purpose. One should note that this does not imply that the product complies directly with the Building Decree.

The CE-mark generally applies for timber panel products only. 3 different classes can be distinguished, according to the possible risks accompanying the product. When class 4 is considered, the risks are small and the manufacturers are allowed to perform tests by their selves and to certify their own product. The compliance to the CE standards has to be checked only once by a notified body. Products within class 4 are meant for decorating rather than for a load bearing function.

Class 2+ already requires more stringent measures: self-certification is allowed, but the compliance to the CE standards has to be verified annually by a notified body. For class 1, the tests have to be performed by the notified bodies themselves.

Equally to the CE-mark, in the United States and Canada the CSA mark is available. The CE-mark is directly linked to the Eurocode.

KOMO

KOMO is the quality label in the Dutch building industry. The label stands for independent quality checks, on the basis of objective measures. It guarantees that the products comply with the statutory requirements, such as the Building Decree and the Building Materials Environmental Decree.

The difference between KOMO and CE, lies in the fact that KOMO also guarantees quality. CE certificates only approve that the product is tested on prescribed European experiments, while KOMO also includes the performance of the product. In addition, KOMO applies specifically on the Dutch building practice, linking the product to the relevant demands from the Building Decree. KOMO certification is solely executed by an independent institute, while the CE mark class 4 can be attached by the manufacturers their selves.
Reflection
Within the Netherlands, the use of timber is often regarded as being questionable. Especially when (tropical) hardwoods are concerned, the majority of those involved simply neglect the opportunities the material has to offer.

The question which has to be raised is whether there is a lasting background behind these taken positions. The answer to this question is twofold. On one hand, people are right that there have been a lot of problems with the origin of timber in the past. Illegal timber flooded the market, being cheaper than its legal congener. In addition, a lot of stories can be told about the exploitation of natural resources and local inhabitants.

But one has to understand, that these problems are problems from the past. Of course, as mentioned earlier, there is still a lot of illegal timber available on the market. But with the responsible attitude taken up by more and more governmental bodies as well as principals, the use of these illegal products is diminished to a minimum.

When timber softwoods are concerned, one should note that the supply of certified timber exceeds the current demand. The meant timber is all harvested according to the high demands as stated above, rather ameliorating the natural resources than exploiting them. For hardwoods, the supply is still inferior, but there are a lot of certified products available on the market which can be readily used.

With the above mentioned certifications (FSC and PEFC), it has become possible to provide a demonstrable sustainable product to the market. It might take a long time before all forests in the world are certified, but by making use of certified products only, this process can even be accelerated.

As a guideline, those involved have to make sure that the products they use are demonstrably certified by a trust-worthy certificate. By taking up this responsible attitude towards the use of timber, there are no valid reasons to ignore the material timber any more.

As from June 2009, BAM Utiliteitsbouw is the first national operating contractor in the Netherlands, which received the FSC-multisite certificate. This specific certificate implies that all eleven divisions that fall under BAM Utiliteitsbouw can guarantee the use of sustainable timber for their projects. As a result, BAM only cooperates with sub-contractors and suppliers which hold the FSC certificate.

To enhance this responsible approach, in this thesis, use will be made of demonstrably certified timber products only.

References: [ 35 ] [ 36 ] [ 37 ] [ 38 ] [ 39 ] [ 41 ] [ 42 ]

A.1.8 Softwood versus hardwood
As already mentioned, the differences between softwoods and hardwoods can be found in the structural composition of the materials. All too often it is thought that the difference lies within the hardness of the material, but this is certainly not true. Hardwood species Balsawood, for example, is softer than all commercial softwoods.

For commercial purposes, there are approximately 50 softwood and 500 hardwood species available. Hardwoods can be found throughout the whole world and can be divided in temperate hardwoods and tropical hardwoods. Temperate hardwoods are found in temperate areas around the world (Europe, North America, Asia, Australia), while tropical hardwoods are to be found in tropical areas such as Central and South America, South-East Asia and parts of Africa. For the latter group, special attention should be paid to available certificates. Softwoods are usually only found in the northern hemisphere and are often adequately certified. Now, the most widely used species for both types of timber are mentioned.
A.1.8.1 Commonly used species

Table 58: Assignment of grades of softwoods and hardwoods to strength classes

<table>
<thead>
<tr>
<th>Trade name</th>
<th>Source</th>
<th>Durability class</th>
<th>Strength class</th>
<th>Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Douglas Fir</td>
<td>E.g. USA, UK, Canada, Germany, France</td>
<td>3</td>
<td>C14; C16; C18; C20; C24; C30; C35</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Larch</td>
<td>E.g. USA, UK, Canada, Germany, France</td>
<td>3</td>
<td>C14; C16; C18; C20; C24; C27; C30</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Pine</td>
<td>E.g. USA, UK, Canada, Germany, France</td>
<td>4</td>
<td>C14; C16; C18; C22; C24; C30</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Scots pine</td>
<td>Germany, Austria, Spain</td>
<td>3 to 4</td>
<td>C18; C22; C27</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Spruce</td>
<td>Various European Countries</td>
<td></td>
<td>C14; C16; C18; C24; C30</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Western Red Cedar</td>
<td>UK, Canada</td>
<td>2 to 3</td>
<td>C14; C18</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Andira</td>
<td>Brazil</td>
<td></td>
<td>D30</td>
<td>FSC</td>
</tr>
<tr>
<td>Angelim vermelho</td>
<td>Brazil</td>
<td>1</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Ash</td>
<td>Germany</td>
<td></td>
<td>D40</td>
<td>FSC /PEFC</td>
</tr>
<tr>
<td>Azobé (Ekki)</td>
<td>West Africa</td>
<td></td>
<td>2 (1 in water contact)</td>
<td>D70</td>
</tr>
<tr>
<td>Balau</td>
<td>South East Asia</td>
<td></td>
<td>D70</td>
<td>FSC</td>
</tr>
<tr>
<td>Bangkirai</td>
<td>Indonesia</td>
<td>2</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Basralocus</td>
<td>Suriname</td>
<td>2</td>
<td>C22</td>
<td>FSC</td>
</tr>
<tr>
<td>Beech</td>
<td>Germany</td>
<td></td>
<td>D35; D40</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Bilinga (Opepe)</td>
<td>West Africa</td>
<td>1</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Cumaru</td>
<td>Brazil</td>
<td>1</td>
<td>D60</td>
<td>FSC</td>
</tr>
<tr>
<td>Eucalyptus</td>
<td>Spain</td>
<td></td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Gonçalo Alves</td>
<td>Brazil</td>
<td>1</td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Greenheart (Ipé)</td>
<td>Guyana</td>
<td></td>
<td>1</td>
<td>D70</td>
</tr>
<tr>
<td>Iroko</td>
<td>Africa</td>
<td></td>
<td>1 to 2</td>
<td>D40</td>
</tr>
<tr>
<td>Itaúba</td>
<td>Brazil</td>
<td>1</td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Jarana</td>
<td>Brazil</td>
<td>2</td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Jarraíba</td>
<td>Western Australia</td>
<td>1</td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Kapur</td>
<td>South East Asia</td>
<td>1 to 2</td>
<td>D60</td>
<td>FSC</td>
</tr>
<tr>
<td>Karri</td>
<td>Western Australia</td>
<td>3</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Karri</td>
<td>South Africa</td>
<td>2</td>
<td>D35</td>
<td>FSC</td>
</tr>
<tr>
<td>Kempas</td>
<td>South East Asia</td>
<td>2</td>
<td>D60</td>
<td>FSC</td>
</tr>
<tr>
<td>Keruing, heavy</td>
<td>South East Asia</td>
<td>3</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Kopie</td>
<td>Brazil</td>
<td>1 to 2</td>
<td>D35</td>
<td>FSC</td>
</tr>
<tr>
<td>Mandioqueira</td>
<td>Brazil</td>
<td></td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Maple</td>
<td>Germany</td>
<td></td>
<td>D30</td>
<td>FSC</td>
</tr>
<tr>
<td>Massaranduba</td>
<td>Brazil</td>
<td>1</td>
<td>D60</td>
<td>FSC</td>
</tr>
<tr>
<td>Meranti, Red</td>
<td>South East Asia</td>
<td>2 to 4</td>
<td>C20</td>
<td>FSC</td>
</tr>
<tr>
<td>Merbau</td>
<td>South East Asia</td>
<td>1 to 2</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Mukulungu</td>
<td>Cameroon</td>
<td></td>
<td>D40</td>
<td>FSC</td>
</tr>
<tr>
<td>Nargusta</td>
<td>Honduras</td>
<td></td>
<td>C24</td>
<td>FSC</td>
</tr>
<tr>
<td>Oak, European</td>
<td>Germany</td>
<td>2</td>
<td>D30</td>
<td>FSC / PEFC</td>
</tr>
<tr>
<td>Okan</td>
<td>Ghana, Cameroon</td>
<td>1</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Piquia marfim</td>
<td>Brazil</td>
<td>1 to 2</td>
<td>D50</td>
<td>FSC</td>
</tr>
<tr>
<td>Red Oak, American</td>
<td>USA</td>
<td></td>
<td>D40</td>
<td>PEFC</td>
</tr>
<tr>
<td>Robinia</td>
<td>Hungary</td>
<td>1 to 2</td>
<td>D30</td>
<td>FSC / PEFC</td>
</tr>
</tbody>
</table>
A.1.8.2 Characteristics

In general, hardwoods have a slower growth rate than softwoods, which, as a consequence, often makes the material become denser and thus stronger. However, there are some (mainly) tropical species with a fast growth rate and thus a density similar to or even lower than softwoods. As a negative consequence, slower growth makes that hardwoods are not readily renewable, leading to higher material costs.

A.1.8.3 Durability

When durability is concerned, it can be concluded that most hardwoods are more durable than softwoods. Softwoods are available from durability class 2 up to 5, while hardwoods are available in class 1 as well (life span exceeding 25 years). This can be explained by the presence of extractives in the hardwoods.

A.1.8.4 Strength

As can be seen in table 58, the strength for softwoods varies between C14 and C35. For hardwoods, the strength varies between D30 and D70. Although the choice for softwoods or hardwoods depends on several aspects, it can be concluded that hardwoods are more suitable for heavy load-bearing structures than softwoods.

A.1.8.5 Fire resistance

In general, hardwoods show a better resistance against fire than softwoods. The material is more dense and thus has a lower charring rate. The charring rate for softwoods is around 0.67 mm/min, while hardwoods have a rate of about 0.5 mm/min, see table 59. The exact charring rate differs for each species and can be determined by means of tests.

### Table 59: Design charring rates for softwoods and hardwoods

<table>
<thead>
<tr>
<th>Material</th>
<th>( \beta_0 ), ([\text{mm/min}] ) (^{(1)} )</th>
<th>( \beta_n ), ([\text{mm/min}] ) (^{(2)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid softwood with ( \rho \geq 290 \text{ kg/m}^3 )</td>
<td>0.65</td>
<td>0.80</td>
</tr>
<tr>
<td>Glued laminated softwood with ( \rho \geq 290 \text{ kg/m}^3 )</td>
<td>0.65</td>
<td>0.70</td>
</tr>
<tr>
<td>Solid hardwood with ( \rho_{\text{mean}} \geq 290 \text{ kg/m}^3 )</td>
<td>0.65</td>
<td>0.70</td>
</tr>
<tr>
<td>Glued laminated hardwood with ( \rho_{\text{mean}} \geq 290 \text{ kg/m}^3 )</td>
<td>0.65</td>
<td>0.70</td>
</tr>
<tr>
<td>Solid hardwood with ( \rho_{\text{mean}} \geq 450 \text{ kg/m}^3 )</td>
<td>0.5</td>
<td>0.55</td>
</tr>
<tr>
<td>Glued laminated hardwood with ( \rho_{\text{mean}} \geq 450 \text{ kg/m}^3 )</td>
<td>0.5</td>
<td>0.55</td>
</tr>
</tbody>
</table>

\(^{(1)}\) one-dimensional design charring rate under standard fire exposure

\(^{(2)}\) notional design charring rate, the magnitude of which includes the effect of corner roundings and fissures

A.1.8.6 Certificates

Both hardwoods are softwoods can be obtained with accompanying sustainability certificates. Until recent, the origin of certain (especially tropical) hardwoods was obscure, restricting the use of these types of timber. But now, a wide variety of certified hardwood species are readily available on the market. Therefore, there are no moral limitations on the use of hardwoods any more (e.g. if being certified).
A.1.8.7 Conclusions
Since hardwoods have clear benefits over softwoods, e.g. more durable, better fire resistance, higher strength and stiffness, the utilisation of hardwoods offers great possibilities. Despite this, these benefits can only be obtained, when the material is being used properly and in the correct environment. In addition, the material is available in longer lengths with larger sections. Softwoods have the benefits over hardwoods that they are cheaper, readily available and more workable.

The choice between hardwoods and softwoods differs for each specific case. When all specific aspects are taken into account, both softwoods and hardwoods provide a reliable product.

References: [ 26 ] [ 41 ] [ 47 ]
Annex A.2 Appearances

A.2.1 Introduction
Together with the preceding annex, this annex will be given an answer to the first sub question as stated in the Introduction. As a reminder, this question will now be shown again:

What kind of timber species are available to use? What are their main applications and what are their characteristics?

This annex will mainly focus on the various appearances of timber and wood-based products. At first, solid timber is discussed, then engineered wood products and finally some composed timber products. The overview given in this chapter does not claim completeness, but is meant to provide an idea on the possibilities of the structural utilisation of timber.

Since it is important that the various products are comparable to each other, each of them will be discussed according to the same set of subjects. At first, some information will be given on the manufacturing and the characteristic properties of the individual product. Thereafter, some insight will be provided on the available dimensions and possible applications in the form of references. Also the certifications that are provided on the product will be discussed. Finally, an overview of the strength properties, the fire resistance and the durability will be given.

As mentioned above, solid timber products are discussed at first. A distinction will be made between completely raw sawn timber and minimally improved sawn timber. Within this section, attention will be given to timber piles as well.

Within the section of engineered wood products, consequently glued laminated timber, plywood, laminated veneer lumber, cross laminated timber and orientated strand boards are mentioned.

Finally, an overview of possible composed wood products will be provided. In this section thin webbed joists and beams are discussed.
A.2.2 Solid timber

Solid wood products are manufactured from wood trunks without putting much effort in improving its quality, while still providing a reliable product. The quality is determined by grading and can be improved by drying, (partial) re-sawing, removing weak spots and combining different elements. Before using them as a building material, it has to be checked which preventive measures have to be taken. In addition to that, their load bearing capacity has to be checked by making use of standards or (certified) tests. In case the options as stated above do not cover the specific use, individual approval can be obtained from the building authority.

In this annex, some different appearance forms of solid wood will be discussed. At first, attention will be given to timber foundation piles. Next, sawn solid timber will be discussed.

A.2.2.1 Timber piles

Production and characteristics

When the load bearing capacity of the soil is insufficient, pile foundations often offer a solution. Amongst the various types of piles that can be used, also the timber pile finds itself. Timber piles were frequently used in the past, but throughout the years the focus has shifted to (reinforced) concrete piles. The timber piles are produced from solid tree trunks, without putting any effort in the improvement of the material, see figure 193.

![Figure 193: Timber foundation piles (and accompanying futtocks)](image)

The toe of the pile is driven to a depth 1.5 m below the top of the load bearing layer. It has to be made sure, that the tip of the pile is placed at least 500 mm below the lowest expected ground water layer. In this case the pile remains under water in all circumstances, preventing it from rotting.

To connect the piles with the upper-structure, concrete futtocks (in Dutch: oplangers) are placed on top of the piles. In figure 193, these concrete futtocks are shown as well.

Available dimensions

The various suppliers of timber foundation piles have drawn up a standard for the available dimensions. This resulted in available lengths up to 23 m, which is usually sufficient to reach the load bearing soil layers and to transfer the loads. The piles are tapered, having a diameter of 300 mm at the top and a diameter of 110-160 mm at the bottom. This is elaborated in table 60.

<table>
<thead>
<tr>
<th>Pile toe diameter [mm]</th>
<th>Toe circumference [mm]</th>
<th>Maximum length [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>280-340</td>
<td>12</td>
</tr>
<tr>
<td>110</td>
<td>340-400</td>
<td>23</td>
</tr>
<tr>
<td>130</td>
<td>400-430</td>
<td>22</td>
</tr>
<tr>
<td>140</td>
<td>430-460</td>
<td>20</td>
</tr>
<tr>
<td>150</td>
<td>460-490</td>
<td>20</td>
</tr>
<tr>
<td>160</td>
<td>&gt; 490</td>
<td>19</td>
</tr>
</tbody>
</table>

For the concrete futtocks that connect the pile tip with the superstructure, the following diameters are available, see table 61. These futtocks, in general, have a length of 3 to 4 m.
Table 61: Available diameters for concrete futtocks

<table>
<thead>
<tr>
<th>Pile diameter, 1 m below tip [mm]</th>
<th>Diameter concrete futtock [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 ≤ D ≤ 160</td>
<td>230</td>
</tr>
<tr>
<td>160 &lt; D ≤ 190</td>
<td>250</td>
</tr>
<tr>
<td>190 ≤ D ≤ 220</td>
<td>280</td>
</tr>
<tr>
<td>220 &lt; D ≤ 250</td>
<td>310</td>
</tr>
<tr>
<td>D &gt; 250</td>
<td>350</td>
</tr>
</tbody>
</table>

Material use
For pile foundations usually only softwoods are being used, since they are cheap, lightweight and have sufficient load bearing capacity. The use of hardwoods would also give satisfying results, but against higher costs.
The most commonly used species are pine, spruce, larch and Douglas Fir. Scots pine is not used anymore due to the high percentage of sapwood, which influences the load bearing capacity badly.

Certifications
PEFC
FSC
CE
KOMO

Strength properties

<table>
<thead>
<tr>
<th>Load duration class: Design compression strength $f_{c,0;d}$ [N/mm²]</th>
<th>Permanent</th>
<th>Long-term</th>
<th>Medium-term</th>
<th>Short-term</th>
<th>Instantaneous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter [mm]</td>
<td>7.5</td>
<td>8.8</td>
<td>10.0</td>
<td>11.3</td>
<td>13.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Diameter [mm]</th>
<th>Design strength of a timber foundation pile [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>47.7 55.9 63.6 71.9 87.8</td>
</tr>
<tr>
<td>110</td>
<td>71.2 83.6 95.0 107.3 131.1</td>
</tr>
<tr>
<td>130</td>
<td>99.5 116.8 132.7 149.9 183.1</td>
</tr>
<tr>
<td>140</td>
<td>115.4 135.4 153.9 173.9 212.4</td>
</tr>
<tr>
<td>150</td>
<td>132.5 155.5 176.7 199.6 243.8</td>
</tr>
<tr>
<td>160</td>
<td>150.7 176.9 201.0 227.2 277.4</td>
</tr>
</tbody>
</table>

Note: next to the strength verification of the piles, also the soil strength has to be verified in accordance with Eurocode 7.
A.2.2.2 Sawn solid timber

Production and characteristics
Sawn solid timber is made from wooden logs, which are being sawn into the right dimensions. These logs can be hardwood or softwood logs. After drying, the strength of the timber is graded visually. The exact production method depends on both sawmill and species. Most important factor during production is the optimisation of the volume, and thus reducing losses.

![Figure 195: Sawn solid timber](image)

Strength improvements
More beneficial production techniques have been developed throughout the years to improve yields of higher grades and to offer special cuts. If necessary, individual members can be finger-jointed or glued together.

Material use and available dimensions
Sawn solid timber can be made from both softwoods and hardwoods. In table 62 and table 63 the readily available dimensions are shown, sorted on species and depending on manufacturer (or literature).

<table>
<thead>
<tr>
<th>Species</th>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>NEN-EN 1313-1</td>
<td>38-100</td>
<td>50-275</td>
<td></td>
</tr>
<tr>
<td>Pine</td>
<td>Centrum Hout</td>
<td>16-100</td>
<td>38-275</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td></td>
<td>Jongeneel</td>
<td>16-100</td>
<td>50-250</td>
<td>2.70-5.40</td>
</tr>
<tr>
<td></td>
<td>Gras</td>
<td>22-100</td>
<td>42-275</td>
<td>2.70-7.50</td>
</tr>
<tr>
<td></td>
<td>Pontmeyer</td>
<td>11-100</td>
<td>32-275</td>
<td>2.70-6.60</td>
</tr>
<tr>
<td></td>
<td>Van Drimmelen</td>
<td>11-100</td>
<td>33-250</td>
<td>unknown</td>
</tr>
<tr>
<td></td>
<td>Finnforest</td>
<td>15-75</td>
<td>45-225</td>
<td>2.55-6.00</td>
</tr>
<tr>
<td></td>
<td>Stiho</td>
<td>22-100</td>
<td>50-275</td>
<td>1.80-6.00</td>
</tr>
<tr>
<td>Scots Pine</td>
<td>Centrum Hout</td>
<td>16-100</td>
<td>38-275</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td></td>
<td>Jongeneel</td>
<td>22-100</td>
<td>100-225</td>
<td>2.70-5.40</td>
</tr>
<tr>
<td></td>
<td>Gras</td>
<td>19-300</td>
<td>75-300</td>
<td>2.70-5.40</td>
</tr>
<tr>
<td></td>
<td>Finnforest</td>
<td>15-75</td>
<td>45-225</td>
<td>2.55-6.00</td>
</tr>
<tr>
<td></td>
<td>Stiho</td>
<td>25-100</td>
<td>100-200</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td>Western Red Cedar</td>
<td>Centrum Hout</td>
<td>22-150</td>
<td>100-300</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td></td>
<td>Jongeneel</td>
<td>22-150</td>
<td>100-300</td>
<td>1.85-6.10</td>
</tr>
<tr>
<td></td>
<td>Pontmeyer</td>
<td>22</td>
<td>150</td>
<td>4.60-5.90</td>
</tr>
<tr>
<td></td>
<td>Gras</td>
<td>23-105</td>
<td>105-310</td>
<td>2.70-5.40</td>
</tr>
<tr>
<td>Larch</td>
<td>Centrum Hout</td>
<td>19-75</td>
<td>100-275</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td></td>
<td>Jongeneel</td>
<td>unknown</td>
<td>unknown</td>
<td>2.70-6.00</td>
</tr>
<tr>
<td></td>
<td>Gras</td>
<td>25-75</td>
<td>150-200</td>
<td>2.70-5.40</td>
</tr>
<tr>
<td>Douglas Fir</td>
<td>Centrum Hout</td>
<td>custom made</td>
<td>custom made</td>
<td>custom made</td>
</tr>
<tr>
<td></td>
<td>Jongeneel</td>
<td>&lt; 500</td>
<td>&lt; 500</td>
<td>up to 10.00</td>
</tr>
<tr>
<td></td>
<td>Gras</td>
<td>25-150</td>
<td>90-400</td>
<td>2.45-6.70</td>
</tr>
</tbody>
</table>
Table 63: Available hardwood species and accompanying dimensions for sawn solid timber

<table>
<thead>
<tr>
<th>Species</th>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>NEN-EN 1313-2</td>
<td>20-100</td>
<td>50-300</td>
<td>2.00-6.00</td>
</tr>
<tr>
<td>Angelim Vermelho</td>
<td>De Ru houtimport</td>
<td>30-150</td>
<td>100-250</td>
<td>unknown</td>
</tr>
<tr>
<td>Bankirai</td>
<td>Centrum Hout</td>
<td>6-100</td>
<td>50-300</td>
<td>2.45-6.70</td>
</tr>
<tr>
<td>Merbau</td>
<td>Centrum Hout</td>
<td>22-100</td>
<td>75-300</td>
<td>1.80-5.40</td>
</tr>
<tr>
<td></td>
<td>Stiho</td>
<td>25-100</td>
<td>90-300</td>
<td>Unknown</td>
</tr>
<tr>
<td>Meranti</td>
<td>Centrum Hout</td>
<td>16-125</td>
<td>50-325</td>
<td>Unknown</td>
</tr>
<tr>
<td></td>
<td>Stiho</td>
<td>25-150</td>
<td>75-300</td>
<td>Unknown</td>
</tr>
<tr>
<td>Azobé (Ekki)</td>
<td>Timbersource</td>
<td>25-100</td>
<td>100-305</td>
<td>2.10-5.50</td>
</tr>
<tr>
<td></td>
<td>De Ru houtimport</td>
<td>50-150</td>
<td>100-250</td>
<td>unknown</td>
</tr>
<tr>
<td>Iroko</td>
<td>Brooks Bros</td>
<td>25-255</td>
<td>150-255</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stiho</td>
<td>44-100</td>
<td>125-300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Timbersource</td>
<td>25-100</td>
<td>130-350</td>
<td>2.00-4.50</td>
</tr>
<tr>
<td>American hardwoods&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>American Hardwood Export Council</td>
<td>19-100</td>
<td>75-300</td>
<td>1.20-4.90</td>
</tr>
</tbody>
</table>

<sup>(1)</sup> As American hardwoods, one can think of Alder, Ash, Beech, Cherry, Hard and Soft Maple, Walnut, Red Oak and White Oak

**Applications**

- Load bearing sections in floors, walls and roofs
- Timber-frame construction
- Cross-laminated timber
- Edge-glued floors and walls
- Scaffolding and support constructions

**Certifications**

- PEFC
- FSC
- CE
- KOMO

**Strength**

Due to the huge amount of different species, sources and grades within structural timber, a strength class system was developed and published in NEN-EN 338. A division has been made between hardwoods and softwoods, denoted by D- and C-classes (Deciduous and Coniferous respectively). Individual structural timber species can be attached to a matching strength class.
### Softwoods or Coniferous

Table 64: Strength classes and characteristic values for softwoods according to NEN-EN 338

<table>
<thead>
<tr>
<th>Characteristic values</th>
<th>C14</th>
<th>C16</th>
<th>C18</th>
<th>C20</th>
<th>C22</th>
<th>C24</th>
<th>C27</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{m,0} ) [N/mm²]</td>
<td>14</td>
<td>16</td>
<td>18</td>
<td>20</td>
<td>22</td>
<td>24</td>
<td>27</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>45</td>
<td>50</td>
</tr>
<tr>
<td>( f_{t,0} ) [N/mm²]</td>
<td>8</td>
<td>10</td>
<td>11</td>
<td>12</td>
<td>13</td>
<td>14</td>
<td>16</td>
<td>18</td>
<td>21</td>
<td>24</td>
<td>27</td>
<td>30</td>
</tr>
<tr>
<td>( f_{t,90} ) [N/mm²]</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>( f_{c,0} ) [N/mm²]</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>19</td>
<td>20</td>
<td>21</td>
<td>22</td>
<td>23</td>
<td>25</td>
<td>26</td>
<td>27</td>
<td>29</td>
</tr>
<tr>
<td>( f_{c,90} ) [N/mm²]</td>
<td>2.0</td>
<td>2.2</td>
<td>2.2</td>
<td>2.3</td>
<td>2.4</td>
<td>2.5</td>
<td>2.5</td>
<td>2.7</td>
<td>2.8</td>
<td>2.9</td>
<td>3.1</td>
<td>3.2</td>
</tr>
<tr>
<td>( f_{v,0} ) [N/mm²]</td>
<td>3.0</td>
<td>3.2</td>
<td>3.4</td>
<td>3.6</td>
<td>3.8</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>( E_{0;\text{mean}} ) [kN/mm²]</td>
<td>7.0</td>
<td>8.0</td>
<td>9.0</td>
<td>9.5</td>
<td>10.0</td>
<td>11.0</td>
<td>11.5</td>
<td>12.0</td>
<td>13.0</td>
<td>14.0</td>
<td>15.0</td>
<td>16.0</td>
</tr>
<tr>
<td>( E_{0;0.05} ) [kN/mm²]</td>
<td>4.7</td>
<td>5.4</td>
<td>6.0</td>
<td>6.4</td>
<td>6.7</td>
<td>7.4</td>
<td>7.7</td>
<td>8.0</td>
<td>8.7</td>
<td>9.4</td>
<td>10.0</td>
<td>10.7</td>
</tr>
<tr>
<td>( E_{90;\text{mean}} ) [kN/mm²]</td>
<td>0.23</td>
<td>0.27</td>
<td>0.30</td>
<td>0.32</td>
<td>0.33</td>
<td>0.37</td>
<td>0.38</td>
<td>0.40</td>
<td>0.43</td>
<td>0.47</td>
<td>0.50</td>
<td>0.53</td>
</tr>
<tr>
<td>( G_{\text{mean}} ) [kN/mm²]</td>
<td>0.44</td>
<td>0.50</td>
<td>0.56</td>
<td>0.59</td>
<td>0.63</td>
<td>0.69</td>
<td>0.72</td>
<td>0.75</td>
<td>0.81</td>
<td>0.88</td>
<td>0.94</td>
<td>1.00</td>
</tr>
<tr>
<td>( \rho ) [kg/m³]</td>
<td>290</td>
<td>310</td>
<td>320</td>
<td>330</td>
<td>340</td>
<td>350</td>
<td>370</td>
<td>380</td>
<td>400</td>
<td>420</td>
<td>440</td>
<td>460</td>
</tr>
<tr>
<td>( \rho_{\text{mean}} ) [kg/m³]</td>
<td>350</td>
<td>370</td>
<td>380</td>
<td>390</td>
<td>410</td>
<td>420</td>
<td>450</td>
<td>460</td>
<td>480</td>
<td>500</td>
<td>520</td>
<td>550</td>
</tr>
</tbody>
</table>

(1) Not readily available

### Hardwoods or Deciduous

Table 65: Strength classes and characteristic values for hardwoods according to NEN-EN 338

<table>
<thead>
<tr>
<th>Characteristic values</th>
<th>D18</th>
<th>D24</th>
<th>D30</th>
<th>D35</th>
<th>D40</th>
<th>D50</th>
<th>D60</th>
<th>D70</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{m,0} ) [N/mm²]</td>
<td>18</td>
<td>24</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>50</td>
<td>60</td>
<td>70</td>
</tr>
<tr>
<td>( f_{t,0} ) [N/mm²]</td>
<td>11</td>
<td>14</td>
<td>18</td>
<td>21</td>
<td>24</td>
<td>30</td>
<td>36</td>
<td>42</td>
</tr>
<tr>
<td>( f_{c,0} ) [N/mm²]</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>( f_{t,90} ) [N/mm²]</td>
<td>18</td>
<td>21</td>
<td>23</td>
<td>25</td>
<td>26</td>
<td>29</td>
<td>32</td>
<td>34</td>
</tr>
<tr>
<td>( f_{c,90} ) [N/mm²]</td>
<td>7.5</td>
<td>7.8</td>
<td>8.0</td>
<td>8.1</td>
<td>8.3</td>
<td>9.3</td>
<td>10.5</td>
<td>13.5</td>
</tr>
<tr>
<td>( f_{v,0} ) [N/mm²]</td>
<td>3.4</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.5</td>
<td>5.0</td>
</tr>
<tr>
<td>( E_{0;\text{mean}} ) [kN/mm²]</td>
<td>9.5</td>
<td>10.0</td>
<td>11.0</td>
<td>12.0</td>
<td>13.0</td>
<td>14.0</td>
<td>17.0</td>
<td>20.0</td>
</tr>
<tr>
<td>( E_{0;0.05} ) [kN/mm²]</td>
<td>8.0</td>
<td>8.5</td>
<td>9.2</td>
<td>10.1</td>
<td>10.9</td>
<td>11.8</td>
<td>14.3</td>
<td>16.8</td>
</tr>
<tr>
<td>( E_{90;\text{mean}} ) [kN/mm²]</td>
<td>0.63</td>
<td>0.67</td>
<td>0.73</td>
<td>0.80</td>
<td>0.86</td>
<td>0.93</td>
<td>1.13</td>
<td>1.33</td>
</tr>
<tr>
<td>( G_{\text{mean}} ) [kN/mm²]</td>
<td>0.59</td>
<td>0.62</td>
<td>0.69</td>
<td>0.75</td>
<td>0.81</td>
<td>0.88</td>
<td>1.06</td>
<td>1.25</td>
</tr>
<tr>
<td>( \rho ) [kg/m³]</td>
<td>475</td>
<td>485</td>
<td>530</td>
<td>540</td>
<td>550</td>
<td>620</td>
<td>700</td>
<td>900</td>
</tr>
<tr>
<td>( \rho_{\text{mean}} ) [kg/m³]</td>
<td>570</td>
<td>580</td>
<td>640</td>
<td>650</td>
<td>660</td>
<td>750</td>
<td>840</td>
<td>1080</td>
</tr>
</tbody>
</table>

(2) Probably not readily available
Related products
Next to the above mentioned sawn timber products, also related products can be distinguished such as modified timber products.

.Modified timber
As mentioned in the section about durability, modified timber can be described as timber that is either thermally or chemically modified, thereby improving the properties of the wood product. The material obtains a better resistance against moisture, better behaviour when durability is concerned, an improved dimensional stability and lower cost for maintenance. As a disadvantage, the strength of the material is reduced up to 30% in some cases.

The material should therefore only be used for non-load bearing purposes, such as outside cladding or floor decking.

.Bamboo
In addition to the earlier mentioned softwoods and hardwoods, also bamboo can be used as a building material. As an example, the product solid bamboo can be mentioned, which is made by high-pressurised compression of bamboo strips. In this way, a solid product is formed coming with a high strength. The product is available in the appearance of beams and plates.

References: [24][25][34][44]
A.2.3 Engineered wood products

Engineered wood products (EWPs) are wood-based products in which the effect of quality reducing elements (such as knots) has been taken away or at least reduced. In addition, these products can be combined to form an element with large dimensions. Hereby, the limits of sawn solid woods (e.g. limited size and quality) are resolved.

The main advantages of EWPs are that they can be manufactured for specific use, they are available in long lengths with high quality and they are often stronger and less sensitive to humidity. In addition, for EWPs made from veneers and flakes, the amount of waste is reduced to a minimum.

A disadvantage can be found in the production costs, which are a lot higher than for sawn solid timber. Now, several different types of EWPs will be discussed.

A.2.3.1 Glued laminated timber

Production and characteristics

Glued laminated timber (glulam) is created by gluing several (at least three) solid timber sections (so-called laminates) together, after taking away defects that reduce the strength of the individual members. The distinct sections are being bonded in such a way, that the direction of their grains is essentially parallel to the longitudinal axis of the member, see figure 197.

The individual laminates have a thickness of 12-50 mm, a length of 1.5-5 m, and they are end-jointed by means of finger-jointing. The individual laminates are kiln dried to a moisture content of around 12-15% before being assembled. Higher moisture contents might complicate high quality bonding by adhesives.

Within the manufacturing process it is possible to create curved members, by using thin laminates of about 12-33 mm. These members can also be double curved or twisted. In addition, also members with varying cross sections, such as tapering beams, can be produced.

Roughly two types of glulam can be distinguished: homogeneous glulam (GL h) consisting of laminates having the same strength, and combined glulam (GL c) consisting of laminates with varying strengths. With the latter type, it is possible to match the quality of the member to the acting design stresses. For example, by providing a beam with higher grade laminates at high stressed regions and low grade laminates at low stressed regions. The timber will then be used more efficiently, compared with normal timber members. Comparing, for example, GL24h and GL24c to each other, it can be found that the bending strength and the Young’s modulus are equal, while all other strength properties for the combined members have lower values.
**Strength improvements**

Although the strength of glulam is determined by the strength of the individual laminates and the strength of the finger joints, the product has a higher strength and stiffness than solid timber members. The reason for this lies in the more evenly spread defects, i.e. the defects are reinforced by adjacent laminates, leading to a more homogeneous material (less variance in strength). The effect of failure due to individual defects is thus reduced, giving the timber a higher mean strength. In A.1.5 more information can be found on the determination of the strength of timber.

For curved members, the cracks that appear in the material are less compared to curved solid timber members, leading to higher characteristic tensile strengths perpendicular to the grain. For the characteristic compression strength perpendicular to the grain, the strengths become higher due to the smaller variance in density of the glulam. The characteristic density of the glulam therefore comes close to the mean density of individual laminates.

**Available dimensions**

In theory, there are no limits on the dimensions of glulam. Spans of up to 300 m are known from practice, though these members have to be custom-made, coming with higher costs. In practice, the limits on dimensions follow from both limitations on transport and limitations on manufacturing.

The maximum dimensions which are feasible with respect to transport can be found in Annex F. It should be mentioned that glulam is usually transported over road, due to the low self-weight of the product.

The maximum dimensions following from production can be found underneath. These are obtained from several manufacturers in the Netherlands as well as from literature.

<table>
<thead>
<tr>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
<th>Radius [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>100-3000</td>
<td>60-300</td>
<td>I ≤ 30</td>
<td>R &gt; 150*d, R &gt; 3500</td>
</tr>
<tr>
<td>De Groot Vroomshoop</td>
<td>99-2260</td>
<td>60-240</td>
<td>I ≤ 70</td>
<td></td>
</tr>
<tr>
<td>Withagen</td>
<td>100-1000</td>
<td>60-300</td>
<td>I ≤ 18</td>
<td></td>
</tr>
<tr>
<td>Derix</td>
<td>100-1000</td>
<td>60-300</td>
<td>I ≤ 18</td>
<td></td>
</tr>
<tr>
<td>GLC</td>
<td>100-880</td>
<td>60-400</td>
<td>I ≤ 24</td>
<td></td>
</tr>
<tr>
<td>Eurban</td>
<td>up to 2000</td>
<td>80-270</td>
<td>I ≤ 21</td>
<td></td>
</tr>
</tbody>
</table>

It can be concluded that the governing limitations on height and width follow from production restrictions. The available lengths are generally limited by transport restrictions.

It has to be mentioned that when the conditions on the construction site are suitable for finger-jointing, there are no restrictions on the lengths of the members.

**Material use and applications**

In general, glued laminated timber is produced from softwoods. In the Netherlands often European softwoods, such as pine and larch, are being used. It is also possible to obtain products made from hardwoods, such as Iroko. The latter products are mainly used when outdoor conditions apply.

![Figure 199: Typical applications of glued laminated timber](image)

- Heavily loaded, long span components (e.g. beams, columns)
- Components with high demands on form stability and appearance

**Certification**
Strength properties

Table 67: Strength classes and characteristic values for glued laminated timber according to NEN-EN 1194

<table>
<thead>
<tr>
<th>Characteristic values</th>
<th>GL24h</th>
<th>GL24c</th>
<th>GL28h</th>
<th>GL28c</th>
<th>GL32h</th>
<th>GL32c</th>
<th>GL36h</th>
<th>GL36c</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{m,0;k}$ [N/mm²]</td>
<td>24</td>
<td>24</td>
<td>28</td>
<td>28</td>
<td>32</td>
<td>32</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td>$f_{t,0;k}$ [N/mm²]</td>
<td>16.5</td>
<td>14</td>
<td>19.5</td>
<td>22.5</td>
<td>19.5</td>
<td>26</td>
<td>22.5</td>
<td></td>
</tr>
<tr>
<td>$f_{t,90;k}$ [N/mm²]</td>
<td>0.4</td>
<td>0.35</td>
<td>0.45</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>$f_{c,0;k}$ [N/mm²]</td>
<td>24</td>
<td>21</td>
<td>26.5</td>
<td>24</td>
<td>29</td>
<td>26.5</td>
<td>31</td>
<td></td>
</tr>
<tr>
<td>$f_{c,90;k}$ [N/mm²]</td>
<td>2.7</td>
<td>2.4</td>
<td>3.0</td>
<td>2.7</td>
<td>3.3</td>
<td>3.0</td>
<td>3.6</td>
<td></td>
</tr>
<tr>
<td>$E_{0;\text{mean}}$ [N/mm²]</td>
<td>11600</td>
<td>11600</td>
<td>12600</td>
<td>12600</td>
<td>13700</td>
<td>13700</td>
<td>14700</td>
<td>14700</td>
</tr>
<tr>
<td>$E_{0;0.05}$ [N/mm²]</td>
<td>9400</td>
<td>9400</td>
<td>10200</td>
<td>10200</td>
<td>11100</td>
<td>11100</td>
<td>11900</td>
<td>11900</td>
</tr>
<tr>
<td>$G_{\text{mean}}$ [N/mm²]</td>
<td>390</td>
<td>320</td>
<td>420</td>
<td>390</td>
<td>460</td>
<td>420</td>
<td>490</td>
<td></td>
</tr>
<tr>
<td>$G_{0,0.05}$ [N/mm²]</td>
<td>720</td>
<td>590</td>
<td>780</td>
<td>720</td>
<td>850</td>
<td>780</td>
<td>910</td>
<td></td>
</tr>
<tr>
<td>$\rho_k$ [kg/m³]</td>
<td>380</td>
<td>350</td>
<td>410</td>
<td>380</td>
<td>430</td>
<td>410</td>
<td>450</td>
<td></td>
</tr>
</tbody>
</table>

Reaction to fire
Release of formaldehyde
D-s2, do (1)
E1

(1) Assuming a minimum mean density of 380 kg/m³ and a minimum overall thickness of 40 mm (NEN-EN 14080)

When fire safety is concerned, the same behaviour can be expected as for solid timber. The charring rate will be low and the uncharred section preserves its strength properties.

Related products

As mentioned above, glulam can be composed of laminates having differing strength classes. In succession, several products have been engineered to meet even higher design standards.

Hybrid glulam

For example, one can think of hybrid glulam products, being composed of laminates from different timber species. In Germany, a glulam product composed of both Beech and (an arbitrary) softwood is brought on the market. The outer, heavier stressed, laminates are made from beeches, being the stronger timber species. The core laminates (around 1/3 of the cross-section) are made from softwoods, see figure 200.

As a result, the bending strength of the material remains almost the same as for a glulam member made of beech only. The Young’s modulus will only be reduced by a minimum (~3%). Advantages of this product are the lower self-weight and the lower costs.

Figure 200: Hybrid glulam product composed of beech and softwood (left) and a combined glulam product made of beech (right)

Another type of hybrid glulam, can be found by the composition of a member from glulam and LVL. The idea behind this product is the same as for the above mentioned hybrid glulam product: the most stressed
fibres are made from high-strength lamellas, while lightweight and cheaper lamellas are being used in the less stressed areas.

**FRP Glulam**
Another product that is highly related to glulam, is the so-called FRP Glulam, which stands for Fibre Reinforced Polymer Glulam. Here, ‘normal’ glulam beams are strengthened by the attachment of a (pre-stressed) fibre at locations where high tensile stresses are to be expected. The system can therefore be compared with that of (pre-stressed) reinforcement within concrete.

The main reason to use FRP is that lower laminate grades are significantly stronger in compression than in tension. Failure will therefore almost always follow directly from tension. The use of 1 to 3 % of FRP can already double the bending strength of the product.

At this time, the product is mainly being used in renovation projects to reinforce beams that do not comply with the modern building standards. To the author’s notice, the product is not (yet) suitable to be used within new buildings.

The main problem that has to be solved for this product, is the debonding between the fibres and the timber near the supports due to high shear stresses. Research [48] has provided several solutions to this problem, but at this time the utilisation of the product is still limited.

**Multilam**
Multilam is a product that is made from laminates that are glued adjacent to each other (instead of on top of each other as for glulam). Hereby horizontal elements are formed, having a heights of 60 to 260 mm and widths of maximal 1 m. Distinct elements can be connected easily, both in the longitudinal and the transverse direction. The elements can span up to 16 m. Their main application is for floorings and roof covering.
A.2.3.2 Plywood

Production and characteristics

Plywood is produced by the pressurized bonding of several thin layers of veneers. These layers are produced by rotary peeling of steamed logs, by which veneer layers with a thickness of 2-4 mm are obtained. These distinct layers are laid up with an angle of 90° to the direction of the grain of adjacent layers, see figure 201.

![Figure 201: Structural composition of plywood](image)

The veneer layers are usually cut in sheets with a length of 2 m, before they are being bonded together. To form stable plywood products, at least 3 layers are necessary. The sheets that are applied at the faces of the product are usually orientated with their grain in longitudinal direction, i.e. the direction in which the product has the biggest length. This leads to the highest stiffness in this direction. An exception can be found with Finnish ply woods, in which the face veneers are in transverse direction.

![Figure 202: Plywood](image)

Strength improvements

The strength of the product is higher than that of a solid timber panel, since defects are more evenly distributed over the cross-section. In addition, the density of the product has improved compared with the solid timber. Another improvement is obtained by shifting the adjacent veneer layers to an angel of 90° to each other. Hereby, the panel becomes more stable in two directions. This configuration also prevents grain movements due to shrinkage and swelling.

Due to it structural composition, the product has a very high strength to weight ratio and is capable of carrying high concentrated loads. The exact strength of the plywood depends on the number of layers, their individual thicknesses, the species and the grade of the material. Also the moisture content, which is usually about 7-12%, is of influence.
Available dimensions
The dimensions in which the product is available, is determined by production limitations. In general, the veneer layers are cut in widths of about 2 m. The possible lengths depend on the specific manufacturer and the lengths of the logs used. The height is determined by the number and thickness of the veneer layers.

<table>
<thead>
<tr>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>3-50</td>
<td>1200-1500</td>
<td>1.20-3.60</td>
</tr>
<tr>
<td>International plywood</td>
<td>4-50</td>
<td>1200-1900</td>
<td>1.20-4.00</td>
</tr>
<tr>
<td>Javor Nederland</td>
<td>4-60</td>
<td>max. 2000</td>
<td>max. 4.00</td>
</tr>
<tr>
<td>Maiburg Hout</td>
<td>4-40</td>
<td>1220-1850</td>
<td>2.44-3.12</td>
</tr>
</tbody>
</table>

Material and applications
Ply woods can be made of both softwoods and hardwoods. For hardwoods can be thought of beech, birch, Okoumé, Meranti, Poplar and Mahoney. For softwoods, one can think Pine, Scots pine, Oregon Pine and Douglas.

Certifications
FSC
PEFC
KOMO
CE-class 2+

Strength properties
Although the NEN-EN 12369-2 provides guidelines on the characteristic bending strength and mean bending modulus values, the choice is made to provide the strength properties of various plywood products that are readily available on the market. The reason for this lies in the fact that the strength classes provided by the regulations are not (yet) implemented by manufacturers and thus, they provide no surplus value.

<table>
<thead>
<tr>
<th>Characteristic values</th>
<th>S-plywood&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>FiN-plywood&lt;sup&gt;(2)&lt;/sup&gt;</th>
<th>US-plywood&lt;sup&gt;(3)&lt;/sup&gt;</th>
<th>CAN-plywood&lt;sup&gt;(4)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T_f;0 [N/mm²]</td>
<td>f&lt;sub&gt;k&lt;/sub&gt;</td>
<td>t_f;90 [N/mm²]</td>
<td>f&lt;sub&gt;k&lt;/sub&gt;</td>
</tr>
<tr>
<td>f&lt;sub&gt;_m,0&lt;/sub&gt; [N/mm²]</td>
<td>12.0</td>
<td>23.0</td>
<td>12.0</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>21.6</td>
<td>24.0</td>
<td>34.8</td>
</tr>
<tr>
<td>f&lt;sub&gt;_m,90&lt;/sub&gt; [N/mm²]</td>
<td>12.0</td>
<td>11.4</td>
<td>12.0</td>
<td>14.8</td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>12.4</td>
<td>24.0</td>
<td>21.0</td>
</tr>
<tr>
<td>f&lt;sub&gt;_t,0&lt;/sub&gt; [N/mm²]</td>
<td>12.0</td>
<td>15.0</td>
<td>12.0</td>
<td>23.5</td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>15.4</td>
<td>24.0</td>
<td>34.8</td>
</tr>
<tr>
<td>f&lt;sub&gt;_t,90&lt;/sub&gt; [N/mm²]</td>
<td>12.0</td>
<td>12.0</td>
<td>12.0</td>
<td>13.6</td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>11.4</td>
<td>24.0</td>
<td>34.1</td>
</tr>
</tbody>
</table>

234
According to NEN-EN 13986, plywood panels fit in fire class D-s2, d0. For floorings the class becomes D-1-s1. The minimum applied thickness of a panel should be 9 mm, to comply with these fire classes.

The release of formaldehyde depends on the specific product and the way it is manufactured. The exact class to which the product belongs should be obtained from the manufacturer.

When the durability against biological attacks is concerned, one should made a distinction between the future purposes of the product. Considering the three service classes as mentioned in the section durability, the following should be mentioned:

<table>
<thead>
<tr>
<th>Purpose</th>
<th>Service class</th>
<th>Suitable for use class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry conditions</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Humid conditions</td>
<td>2</td>
<td>1 and 2</td>
</tr>
<tr>
<td>Exterior conditions</td>
<td>3</td>
<td>1, 2 and 3</td>
</tr>
</tbody>
</table>

In addition, it should be mentioned that plywood is generally not suitable to use when use class 4 or 5 applies.

**Related products**

Next to plywood composed of only soft- or hardwood veneers, also combinations of soft- and hardwood veneers are being applied. The face veneers are made from hardwood, while the core veneers are made from softwoods. By doing this, the material is used more efficiently, reducing costs and weight. At this time, the product is not readily available on the (Dutch) market.

References: [ 24 ][ 25 ][ 27 ][ 32 ][ 45 ]
A.2.3.3 Laminated veneer lumber

**Production and characteristics**

Laminated veneer lumber (LVL) products are composed of equally oriented veneer layers which are glued together, see figure 204. The veneer layers that are being used are somewhat thicker than those that are used in plywood. These veneers are obtained by the rotary peeling of logs, resulting in layers of about 2 m wide. These individual layers have a thickness of about 3-4 mm. The veneer layers are jointed by means of scarf-joints, except for the core layers which are butt-jointed. To minimize the strength reducing effects of these joints, they are staggered above each other. After bonding, the plates are cut and sawn into the favoured dimensions.

![Figure 204: Composition of LVL](image)

After production, the material has a relatively good strength and stiffness, and a very high resistance against bending. The moisture content after production is around 11%, which, to a large extent, prevents the material from warping. The charring rate is considerably low, being only 0.70 mm/min.

The material comes under wood preservative class HWS 100, which means that the maximum allowable moisture content is 18%. After treatment this can be increased to 21% (HWS 100 G).

The product LVL has great potential due to its possible dimensions, strength and finish quality. In Europe the product is commonly known as Kerto®. Kerto is supplied in two types, namely Kerto-S® (Straight) and Kerto-Q® (Quer). The difference can be found in the directions in which the veneers are orientated. For Kerto-S® all veneers are orientated in the same direction, as for Kerto-Q®, approximately a fifth of the veneers is oriented in transverse direction.

![Figure 205: Kerto-S®](image)  ![Figure 206: Kerto-Q®](image)

**Strength improvements**

As for plywood, the peeling of logs results in a better spread of defects in the material. As a result, especially the tensile strength of the material is increased. Since all (or at least 80% for Kerto-Q, see above) of the veneer layers are staggered with the grain in one single direction, the material has a very high bending strength.

In some cases, as for Kerto-products, the veneers are compressed up to 10%, resulting in even higher values for strength and stiffness.

**Available dimensions**

<table>
<thead>
<tr>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>Type S</td>
<td>27 - 90</td>
<td>200 - 2500</td>
</tr>
<tr>
<td></td>
<td>Type Q</td>
<td>27 - 69</td>
<td>200 - 2500</td>
</tr>
<tr>
<td>Finnforest</td>
<td>Kerto-S®</td>
<td>21 - 90</td>
<td>200 - 2500</td>
</tr>
<tr>
<td></td>
<td>Kerto-Q®</td>
<td>27 - 69</td>
<td>900 - 2500</td>
</tr>
</tbody>
</table>
Material use and applications

LVL is usually composed from softwoods, rather than from hardwoods. When Kerto products are concerned, the softwood used is Finnish Pine wood. Another species that is regularly used is Southern yellow pine.

Kerto-S® is mainly being used for horizontal load-bearing applications, such as rails or beams.
Kerto-Q® is being used for plates, columns and curved elements.

Certifications
CE-Class 1+
PEFC, FSC

Strength properties

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Kerto-S®</th>
<th>Kerto-Q®</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>21-90 mm</td>
<td>27-69 mm</td>
</tr>
<tr>
<td>$f_{m,0;edge}$ [N/mm²]</td>
<td>44.0</td>
<td>32.0</td>
</tr>
<tr>
<td>$f_{m,0;flat}$ [N/mm²]</td>
<td>50.0</td>
<td>36.0</td>
</tr>
<tr>
<td>$f_{c,0;edge}$ [N/mm²]</td>
<td>35.0</td>
<td>26.0</td>
</tr>
<tr>
<td>$f_{c,0;flat}$ [N/mm²]</td>
<td>0.8</td>
<td>6.0</td>
</tr>
<tr>
<td>$f_{t,0;edge}$ [N/mm²]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$f_{t,0;flat}$ [N/mm²]</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$E_{0;mean}$ [N/mm²]</td>
<td>13800</td>
<td>10500</td>
</tr>
<tr>
<td>$E_{0}$ [N/mm²]</td>
<td>11600</td>
<td>8800</td>
</tr>
<tr>
<td>$G_{0;edge;mean}$ [N/mm²]</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td>$G_{0;edge}$ [N/mm²]</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>$G_{0;flat;mean}$ [N/mm²]</td>
<td>600</td>
<td>-</td>
</tr>
<tr>
<td>$G_{0;flat}$ [N/mm²]</td>
<td>400</td>
<td>-</td>
</tr>
<tr>
<td>$\rho_{mean}$ [kg/m³]</td>
<td>510</td>
<td>510</td>
</tr>
<tr>
<td>$\rho_{k}$ [kg/m³]</td>
<td>480</td>
<td>480</td>
</tr>
</tbody>
</table>

Reaction to fire  
D-s1, d0  
D-s1, d0

Release of formaldehyde  
E1  
E1

Natural durability against biological attack  
4  
4
**Related products**

In addition to the above mentioned products, one can also name (among others) LVL type T, Parallel Strand Lumber (PSL) and Laminated Strand Lumber (LSL). These products are all closely related to LVL, being composed of veneers, strands or flakes.

LVL type T can be considered the same as type S, except for that the veneers being used are lighter (lower density) and thus have a lower load bearing capacity. The veneers are generally bonded by a scarf joint or a simple overlap.

LSL (for example Intrallam®) is produced from 300 mm long strands, with widths of 30 mm. They are orientated in a parallel direction and formed into mats with dimensions of 2.44 m by 14.63 m. By compressing these mats under steam injection, the desirable thickness can be obtained, see figure 213.

PSL (for example Parallam®) is produced by peeling small-diameter logs into veneer sheets. These are then dried to a moisture content of about 3% and then cut into thin long strands, see figure 214. By stranding these sheets, the defects which reduce strength characteristics are diminished. After being stranded, the various veneers are bonded in a microwave process to dimensions of 275 mm by 475 mm, with lengths up to 20 m.

![Figure 213: Intrallam®](image1)

![Figure 214: Parallam®](image2)

Compared to Kerto® LVL, the bending strength of Parallam® is in the same range, while in compression and shear Parallam® has a higher strength.

References: [24] [25] [27] [29] [30]
A.2.3.4 Cross Laminated Timber (CLT)

**Production and characteristics**

Cross Laminated Timber (CLT) is manufactured by bonding various (3, 5, 7 or more) single layered panels, which are in turn made by the bonding of various softwood lamellas with a thickness of 4-84 mm. The orientation of the panels is shifted for each successive layer, as with plywood, creating a massive structural timber element. By means of vacuum compression, the panels can even be bent. The product has a high resemblance to plywood, except for it being composed of laminated panels instead of veneers.

![Figure 215: Composition of Cross-Laminated Timber](image)

A commonly used product within Europe is Leno®. It has a higher strength than traditional timber elements, while also the warping of the timber is limited due to the moisture content of around 11%. The charring rate is around 0.7 mm/min.

![Figure 216: Cross Laminated Timber](image)

**Strength improvements**

Compared to solid timber, the strength of the product is highly increased due to the more even spread of defects. Due to the crosswise arrangement of the individual panels, the product becomes stiff in two directions. This leads to a totally new set of load bearing possibilities compared to conventional timber products. In addition, shrinking and swelling is reduced to a minimum.

**Available dimensions**

<table>
<thead>
<tr>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
<th>Radius [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>50 - 500</td>
<td>up to 4800</td>
<td>up to 20.00</td>
<td>R &gt; 8000</td>
</tr>
<tr>
<td>StoraEnso CLT</td>
<td>51 - 400</td>
<td>up to 2950</td>
<td>up to 16.00</td>
<td></td>
</tr>
<tr>
<td>KLH</td>
<td>57 - 500</td>
<td>2400 - 2950</td>
<td>8.00 - 16.50</td>
<td></td>
</tr>
<tr>
<td>Lenotec</td>
<td>51 - 500</td>
<td>up to 4800</td>
<td>up to 14.80</td>
<td></td>
</tr>
<tr>
<td>Binder Jones</td>
<td>65 - 282</td>
<td>1250</td>
<td>up to 13.00</td>
<td></td>
</tr>
</tbody>
</table>
Material use and applications
The cross laminated timber elements are usually made from kiln-dried spruce, but also White Fir and Douglas Fir are being used. Hardwoods are generally not being used.

![Figure 217: Typical applications of cross laminated timber](image)

β Load-bearing walls
β Floors
β Roofs
β Stairs
β Lift shafts

Certification
PEFC
FSC
CE

Strength properties
The strength properties of cross laminated timber depend on the thickness and number of the layers and the strength of the individual lamellas. In general, lamellas with strength classes up to C24 are being used in CLT. The product complies with formaldehyde emission class E1.

For the exact strength properties, reference is made to the above mentioned manufacturers. At this time, there are no design standards available on CLT. Therefore, it is impossible to provide guidelines on the design.

Related products
Lenostrand is a cross laminated timber product composed of OSB-3 and OSB-4 layers, see figure 218. The thickness of the elements varies between 66 and 200 mm, while a width of up to 4800 mm can be obtained. They are usually composed of 3 to 7 layers. The weight is approximately 650 kg/m³ (interior/exterior wall) or 800 kg/m³ (partition wall).

![Figure 218: Lenostrand](image)

References: [ 29 ][ 31 ]
A.2.3.5 Oriented strand boards

**Production and characteristics**

Oriented strand boards (OSB), in the United States referred to as structural wood panels (SWP), are manufactured from small pieces of wood (strands) which are bonded under heat and pressure. The used strands are on average 70-130 mm long and 35 mm wide. Board thicknesses vary between 5-40 mm with densities of 550 to 750 kg/m$^3$. The boards consist for about 97% out of timber.

Near the surfaces of the board, the strands are in general lying in the longitudinal direction of the board. These outer layers each have a thickness of about 25% of the total thickness of the board. In the middle the layers are generally placed in transverse direction. OSB has the advantage over plywood that it is more cost-effective (return 80-90% for OSB compared to 30-40% for plywood), environmentally friendlier (half the energy required during production) and more stable.

![Figure 219: Structural composition of Oriented Strand Board](image)

Four different types of OSB can be distinguished, being referred to as OSB/1 up to OSB/4. The difference lies in the applications and environment in which the board can be used. For example, OSB/1 is suitable for general purposes in dry conditions, while OSB/4 can have a heavy duty, load bearing function in a humid climate. In the Netherlands, the use of OSB/1 and OSB/4 is very rare, since they are not manufactured here nor imported from abroad. OSB/2 can be used as a load bearing board under dry conditions, while OSB/3 can also be applied in humid conditions.

![Figure 220: Oriented Strand Board](image)

**Strength improvements**

Since the material is composed of several strands bonded under pressure, the effect of strength reducing defects is reduced to the utmost extent. This leads to OSB being a very homogeneous and uniform material, able to concur against plywood. The strands are bonded together using a specific glue, which gives the boards a moisture proof character.

**Available dimensions**

<table>
<thead>
<tr>
<th>Source</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>5-40</td>
<td>600 - 2440</td>
<td>1.22 - 7.32</td>
</tr>
<tr>
<td>Sterling</td>
<td>8-25</td>
<td>590 - 1200</td>
<td>2.44</td>
</tr>
<tr>
<td>International plywood</td>
<td>9-18</td>
<td>590 - 1220</td>
<td>2.44</td>
</tr>
<tr>
<td>Pontmeyer</td>
<td>18</td>
<td>590 - 1220</td>
<td>2.44</td>
</tr>
<tr>
<td>Jongeneel</td>
<td>18</td>
<td>610 - 1220</td>
<td>2.44</td>
</tr>
</tbody>
</table>
Material use and applications
In Europe mainly softwoods are being used, such as Pine, Scots pine and Douglas. In the United States of America, also hardwoods are being used.

![Figure 221: Typical applications of OSB](image)

- Floor decking
- Roof cladding
- Composite constructions (e.g. webs of I-joists)

Certification
- FSC
- PEFC
- CE-class 2+
- KOMO

Strength properties
According to NEN-EN 12369, the following strength properties can be applied to OSB/2-3 and OSB/4. OSB/1 is neglected in this case, since this material is not suitable to be used as a structural member.

<table>
<thead>
<tr>
<th>Characteristic values</th>
<th>OSB/2-3</th>
<th>OSB/4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_{t;0;k}$ [N/mm$^2$]</td>
<td>$f_{t;90;k}$ [N/mm$^2$]</td>
</tr>
<tr>
<td></td>
<td>$6-10$</td>
<td>$10-18$</td>
</tr>
<tr>
<td>$f_{m,0;k}$ [N/mm$^2$]</td>
<td>$18.0$</td>
<td>$24.5$</td>
</tr>
<tr>
<td>$f_{m,90;k}$ [N/mm$^2$]</td>
<td>$9.0$</td>
<td>$13.0$</td>
</tr>
<tr>
<td></td>
<td>$6-10$</td>
<td>$10-18$</td>
</tr>
<tr>
<td>$f_{c,0;k}$ [N/mm$^2$]</td>
<td>$15.9$</td>
<td>$18.1$</td>
</tr>
<tr>
<td>$f_{c,90;k}$ [N/mm$^2$]</td>
<td>$12.9$</td>
<td>$14.3$</td>
</tr>
<tr>
<td></td>
<td>$6-10$</td>
<td>$10-18$</td>
</tr>
<tr>
<td>$f_{v,0;k}$ [N/mm$^2$]</td>
<td>$6.8$</td>
<td>$6.9$</td>
</tr>
<tr>
<td>$f_{v,90;k}$ [N/mm$^2$]</td>
<td>$1.0$</td>
<td>$1.1$</td>
</tr>
<tr>
<td>$E_{m,0;mean}$ [N/mm$^2$]</td>
<td>$4930$</td>
<td>$6780$</td>
</tr>
<tr>
<td>$E_{m,90;mean}$ [N/mm$^2$]</td>
<td>$3800$</td>
<td>$4300$</td>
</tr>
</tbody>
</table>

242
Table 1. Initial properties of OSB boards

<table>
<thead>
<tr>
<th>Parameter</th>
<th>6-25</th>
<th>3000</th>
<th>6-25</th>
<th>3200</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{\text{30,mean}}$ [N/mm²]</td>
<td>6-25</td>
<td>3000</td>
<td>6-25</td>
<td>3200</td>
</tr>
<tr>
<td>$G_{\text{v,mean}}$ [N/mm²]</td>
<td>6-25</td>
<td>1080</td>
<td>6-25</td>
<td>1090</td>
</tr>
<tr>
<td>$G_{\text{r,mean}}$ [N/mm²]</td>
<td>6-25</td>
<td>50</td>
<td>6-25</td>
<td>60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_k$ [kg/m³]</td>
<td>550</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reaction to fire</td>
<td>D-s2, d0; D-s2, d0</td>
</tr>
<tr>
<td></td>
<td>D-s1, d0; D-s1, d1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Release of formaldehyde</td>
<td>E1</td>
</tr>
<tr>
<td>Natural durability against</td>
<td>4</td>
</tr>
<tr>
<td>biological attack</td>
<td>4</td>
</tr>
</tbody>
</table>

In addition to the above, NEN-EN 300 gives additional bending strength properties for boards with a thickness up to 40 mm.

At this time, there are no OSB products available that are able to comply with the demands of use classes 4 and 5.

**Related products**

Structural insulated panels (SIP) are composed of two OSB panels, filled with insulating material in-between, see figure 222. These types of panels are already commonly used in the USA and have the advantage that their use reduces the building time to a great extent. SIPs are commonly used for flooring, roofs and as wall elements.

![Figure 222: Structural Insulated Panel](image)

References: [24][25][33]
A.2.4 Composed wood products

In the previous sections, the currently available solid timber and engineered wood products have been discussed. In addition, there are also manufacturers who combine the earlier mentioned products to form a new product, the so-called composed wood products. Some of these products have already been discussed briefly under the headings ‘related products’. One can think of hybrid glulam and timber I-joists.

In this section, a more detailed approach will be shown. Again, attention will be given to the production process and the characteristics, strength improvements, available dimensions, material use and possible applications, certification, strength properties and related products. For some products, not all of these aspects are being discussed.

At first, thin webbed joist, better known as I-joists, will be discussed. After that, attention will be given to thin webbed box beams and finally boxed elements are considered.

A.2.4.1 Thin webbed joists

Production and characteristics

Thin webbed joists (I-joists) are composite construction elements, consisting of flanges of solid timber or LVL and webs of OSB or plywood. Without elaborating on the production process of the separate timber products, it can be said that the flanges and the webs are bonded by a waterproof adhesive, creating an I-section member.

The flanges provide moment capacity of the beam, while the web carries the shear force to a great extent. The top flange is usually composed of the same material and grade as the bottom flange. Besides, I-joists are lightweight and they possess more strength than conventional structural timber members of the same size. A weight reduction of 30 to 40% can be expected. In addition, the beam can be produced in exact dimensions, being straight in two dimensions and within small tolerances.

The product is sensitive when it comes to shear buckling. In addition, the product is instable by itself and should therefore be braced laterally. At locations where high shear forces are to be expected, it is advised to apply shear stiffeners.

Strength improvements

As mentioned earlier, I-joists are economically efficient because of the efficient material use. The greatest bending stresses in a beam occur at the outer edges, where the high strength flanges are located.
**Available dimensions**

As for most composed wood products, the available dimensions follow from the specific manufacturer. A distinction has to be made on the material that is being used for both flanges and webs. An overview is given in table 74.

<table>
<thead>
<tr>
<th>Source</th>
<th>Flange / Web</th>
<th>Height h [mm]</th>
<th>Width b [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Literature</td>
<td>LVL / Solid timber</td>
<td>220 - 350</td>
<td>8 - 40</td>
<td>3.50 - 14.00</td>
</tr>
<tr>
<td></td>
<td>OSB / Plywood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Finnjoist</td>
<td>Kerto-S</td>
<td>39</td>
<td>13 - 89</td>
<td>12.00 - 14.00</td>
</tr>
<tr>
<td></td>
<td>OSB/3</td>
<td>122 - 322</td>
<td>8 - 25</td>
<td></td>
</tr>
<tr>
<td>JJI</td>
<td>Solid timber C24</td>
<td>45</td>
<td>45 - 97</td>
<td>up to 12.00</td>
</tr>
<tr>
<td></td>
<td>OSB/3</td>
<td>55 - 360</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>Kaufmann</td>
<td>Solid Timber</td>
<td>40</td>
<td>80</td>
<td>2.45 - 10.00</td>
</tr>
<tr>
<td></td>
<td>Plywood</td>
<td>120</td>
<td>26.8</td>
<td></td>
</tr>
<tr>
<td>Masonite</td>
<td>Solid Timber</td>
<td>47 - 60</td>
<td>47 - 90</td>
<td>up to 8.00</td>
</tr>
<tr>
<td></td>
<td>OSB / Plywood</td>
<td>100 - 306</td>
<td>8 - 15</td>
<td></td>
</tr>
</tbody>
</table>

**Material use and applications**

The webs of the I-joist are composed of OSB or plywood. The flanges are made from sawn solid timber or LVL. The composition (species) of these individual products can be found in their distinctive section.

![Typical applications of I-joists](image)

Figure 225: Typical applications of I-joists

**Certification**

PEFC, FSC
CE-class 1

**Strength properties**

The strength properties of an I-joist depend on the (strength of the) used materials and their dimensions. Since these differ for every manufacturer, specific strength information should be obtained from manufacturers. One should be aware that the strength is mainly determined by the strength of the flanges.

**Related products**

In addition to this product, also double I-joists and metal-web I-joists are available on the market. The double I-joist will not be discussed here, since the product is very similar to thin webbed box beams, which will discussed later on.

Metal webbed I-joists are, as the name implies, I-joist composed of webs made of metal. An example is given in figure 226.
The product combines the low weight of timber with the strength of metal. Therefore, bigger continuous spans become possible and the beam show better behaviour on dimensional stability. In addition, the product has the advantage that technical installations can be carried through without reducing the strength of the beam.

The product is available in widths of 72 to 147 mm, by heights of 219 to 417 mm. Spans bigger than 8 m are possible, when lateral bracing is introduced.

References: [27] [29] [40]
A.2.4.2 Thin webbed beams (box beams)

**Production and characteristics**
Box beams are similar to I-joists, being composed of LVL, solid timber or glulam flanges and OSB or plywood webs. The flanges are being connected by webs on both ends, forming a box shape. The hollow core can be used to hide services from the sight. As with I-joists, at the top and bottom of the beam, the biggest cross-sectional area is present to resist flexural stresses.

![Figure 227: Structural composition of a box beam](image)

Generally, box beams have to be designed and manufactured for every specific project.

![Figure 228: Box beam](image)

**Strength improvements**
Compared to I-joists, the box beams is more stable because of its symmetrical shape, but web stiffeners might still be necessary. For technical installations, box beams provide the possibility to be placed in the voids of the beam.

**Available dimensions**
Since box beams are not readily available on the market, the dimensions can be specified for each project individually. The cross sectional dimensions are currently limited by production to a maximum of 1200 by 1200 mm. For the lengths of the elements, transport and factory sizes are governing.

**Strength properties**
The strength properties of a box beam depend on the (strength of the) used materials and their dimensions. Since these differ for every manufacturer, specific strength information should be obtained from manufacturers.

**Related products**
Next to box beam, also boxed columns can be mentioned. At this time, these (square) columns are mainly being used in the USA because of their aesthetic appearance. For structural solutions, elements which are closer related to box beams could be used.

References: [27]
A.2.4.3 Box elements

**Production and characteristics**
Next to the above mentioned box beams, also box elements can be mentioned. These elements, also known as thin flanged beams, can be considered as multiple box beam elements. The product is closely related to the concrete hollow core slabs as they exist at this time.

The element can be composed of multiple boxes, which can be easily combined to form an element. Each box is composed of two horizontal and two vertical planks with a thickness of 21 mm which are glued together, see figure 229.

![Lignatur box element](image)

**Strength improvements**
Compared to solid timber elements, the used material is being used more efficient. The product is known for its rigid load-bearing characteristics. When being compared to a concrete floor, it can be said that the necessary height is in the same range.

As another advantage, the hollow cores can be filled with insulation or fire retardant materials, leading to an improved functionality.

**Available dimensions**
Currently, Lignatur is the only product that is readily available at the market. The product has not been widely used in Netherlands up to now.

<table>
<thead>
<tr>
<th>Source</th>
<th>Height $h$ [mm]</th>
<th>Width $b$ [mm]</th>
<th>Lengths [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lignatur</td>
<td>160 - 320</td>
<td>Multiple of 200</td>
<td>up to 12</td>
</tr>
</tbody>
</table>

**Material use and applications**
Lignatur is made from Swiss Spruce and is mainly being used as a floor or roof element.

![Typical applications of box elements](image)
Certification
There is no information available on awarded certificates

Strength properties
The strength of the material depends on the grades of material used and is specified by the supplier. Lignatur makes use of strength grade C24. The product weighs about 38 to 68 kg/m².

Related products
In relation with the above mentioned box elements, also elements composed of timber plates which are connected by ribs can be distinguished. These will be referred to as multiple box elements.

Multiple box elements
As mentioned above, multiple box elements are composed of an lower and upper plate, connected in between by means of ribs. Currently two products are on the market, being the Kerto-Ripa Box element and the Lignatur Surface element.

The Kerto-Ripa Box element is composed of Kerto-Q surfaces combined with Kerto-S ribs. The surface plates have a thickness of 27mm or 33 mm, while the ribs have dimensions of 39 mm to 75 mm by 200 mm to 600 mm. The standard width for the element is 2400 mm and it can span up 14 m for floors and up to 20 m for roofs. An example is given in figure 231.

![Figure 231: Kerto-Ripa Box element](image)

The Lignatur Surface element is composed of planks made from spruce, glued together to form a massive element. The element is available in standard widths of 514 mm and 1000 mm, with height varying between 120 mm and 320 mm. The product shows a high resemblance to the Kerto-Ripa box element, except for it not being formed by joining several box elements together.

![Figure 232: Lignatur Surface element](image)

References: [ 29 ]
Annex B Design standards

Annex B.1 Building Decree

In this annex, only the demands which are directly relevant for a stadium are shown. Within the stadium structure, there is space available for the use functions congregation, office, education, stores, remaining use function for stalling motorised vehicles (car park) and structures not being a building (grandstands). The demands that comply with these functions are shown in this annex.

B.1.1 Degree of occupation

The demands which are stated in the Building Decree, often coincide with the specific use function of a (part of) a building and the so-called ‘degree of occupation’ (in Dutch: ‘bezettingsgraad’). Before further elaborating on the demands as stated in the Building Decree, this term is explained now.

The degree of occupation tells something about the amount of people that can possibly be present, at a certain functional area, at any given time. To clarify the term, table 76 shows some examples.

<table>
<thead>
<tr>
<th>Use Function</th>
<th>Degree of occupation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Living</td>
<td>-</td>
</tr>
<tr>
<td>Congregation:</td>
<td></td>
</tr>
<tr>
<td>- to watch sports</td>
<td>B2</td>
</tr>
<tr>
<td>- other activities</td>
<td>B3</td>
</tr>
<tr>
<td>Cell function</td>
<td></td>
</tr>
<tr>
<td>- for visitors</td>
<td>B3</td>
</tr>
<tr>
<td>- other</td>
<td>B4</td>
</tr>
<tr>
<td>Health care function:</td>
<td></td>
</tr>
<tr>
<td>- for visitors</td>
<td>B3</td>
</tr>
<tr>
<td>- other</td>
<td>B4</td>
</tr>
<tr>
<td>Industrial</td>
<td>B5</td>
</tr>
<tr>
<td>Office</td>
<td>B4</td>
</tr>
<tr>
<td>Accommodation</td>
<td>B4</td>
</tr>
<tr>
<td>Education</td>
<td>B3</td>
</tr>
<tr>
<td>Sports</td>
<td>B5</td>
</tr>
<tr>
<td>Stores</td>
<td>B5</td>
</tr>
<tr>
<td>Remaining use functions</td>
<td>B5</td>
</tr>
<tr>
<td>Structure not being a building</td>
<td>-</td>
</tr>
</tbody>
</table>

Each degree of occupation refers to a standardized amount of space that should be accounted for per person. The higher the degree of occupation, the more space that should be accounted for. For more information on the requirements of use space for each specific function, reference is made to the Dutch Building Decree.

B.1.2 Strength requirements

B.1.2.1 General

Section 2.1 of the Dutch Building Decree demands that the structure should be able to withstand all the loads that might occur during the life span of the specific building, as defined by the NEN 6700. For the prevailing load combinations reference is made to NEN 6702, while for timber structures NEN 6760 should be accounted for. Since the Eurocode is adopted in this thesis, use is made of the equivalents of the NENs above. This is acceptable on behalf of the provision of equivalence conform article 1.5.

B.1.2.2 Fire

According to article 2.8, a building structure should be built in such a way, that the building can be evacuated and searched within an acceptable time, without collapse being a risk.
Since there are lots of restrictions made behaviour in fire, here, only a summary is provided on the most important factors. The aspect of evacuation is neglected within this summary.

**Resistance against fire**

- **B** The load-bearing structure should be fire resistant for at least 90 minutes. This value can be reduced by 30 minutes when the fire load is less than 500 MJ/m$^2$.

- **B** The grandstand structure should be fire resistant for at least 30 minutes (depending on the fire load underneath the grandstand).

- **B** When the roof structure is concerned, no demands are made concerning fire.

**Restricting fire development**

To restrict a fire from developing, demands have been made on the surfaces of various (structural) elements within the building envelope. The most important demands are shown in Table 77.

<table>
<thead>
<tr>
<th>Euro-class</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>for a structural element (not being a door, a window, etc.) bordering open air at ground level having a height of at least 2.5 m</td>
</tr>
<tr>
<td>B</td>
<td>for a structural element 1. within a confined fire and smoke free area (including an elevator shaft), 2. bordering open air, being situated at a height of 13 m or more above ground level</td>
</tr>
<tr>
<td>C</td>
<td>for a structural element within a non-confined fire and smoke free area</td>
</tr>
<tr>
<td>D</td>
<td>for a structural element within all remaining spaces</td>
</tr>
<tr>
<td>D$_{fl}$</td>
<td>for doors, windows or equivalent elements bordering open air</td>
</tr>
<tr>
<td>C$_{fl}$</td>
<td>for the topside of floors which provide fire and smoke free evacuation routes</td>
</tr>
<tr>
<td>D$_{fl}$</td>
<td>for the topside of remaining floors</td>
</tr>
</tbody>
</table>

**Restricting fire extension**

In between a fire compartment and: 1. another fire compartment, 2. a confined space including a fire and smoke free escape route or 3. a non-confined staircase, a resistance of at least 60 minutes should be guaranteed.

**Overview**

When the above stated demands are implemented in the stadium structure, the following is found, see Figure 233 and Table 78. For the main load-bearing structure, excluding the roof, a fire resistance of 90 minutes is required. For the roof structure no demands are made. When the grandstand is concerned, the fire resistance is linked to the potential fire load of the underlying space.
Table 78: Overview of demands on fire safety sorted by space

<table>
<thead>
<tr>
<th>Space</th>
<th>Degree of occupation</th>
<th>Floors</th>
<th>Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B4: Office</td>
<td>D₄</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>B3: Education</td>
<td>D₃</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>B4: Office</td>
<td>D₄</td>
<td>D/B</td>
</tr>
<tr>
<td></td>
<td>B3: Education</td>
<td>D₃</td>
<td>D/B</td>
</tr>
<tr>
<td>3</td>
<td>B3: Congregation: other use⁽¹⁾</td>
<td>C₃ or D₄</td>
<td>D/B</td>
</tr>
<tr>
<td>4</td>
<td>B2: Congregation: to watch sport⁽²⁾</td>
<td>C₄</td>
<td>B/D</td>
</tr>
<tr>
<td>5</td>
<td>B5: Other use function</td>
<td>D₅</td>
<td>D/B</td>
</tr>
<tr>
<td>6</td>
<td>B5: Other use function</td>
<td>D₅</td>
<td>D/B</td>
</tr>
<tr>
<td>7</td>
<td>- : Roof</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>- : Structure not being a building:</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Grandstand</td>
<td>C₅</td>
<td>B</td>
</tr>
</tbody>
</table>

⁽¹⁾When fire and smoke free evacuation routes are concerned, fire class C₃ is demanded

B.1.3 Serviceability requirements

Also in this section, only the most important demands are discussed, since it goes beyond the goal of this thesis to provide a complete overview. Subjects like toilets, bathrooms, bicycle sheds, etc. are therefore neglected.

B.1.3.1 Free passage and accessibility (article 4.10)

- A building should be reasonably accessible
- The entrances to a certain space should be at least 0.85 m wide, having a free height of 2.3 m.
- The difference in height between ground level and an entrance to an accessible space is at most 1 m.

B.1.3.2 Occupied zones and space

A building contains at least one occupied zone, in which activities are employed which are specific for the use function concerned. For all use functions that can be found within the stadium, except for a structure not being a building, the demand is made that at least 55% of the useable area for the use function is an occupied zone. When a congregation area for the purpose of alcohol consumption is considered, the occupied zone should be at least 35 m².

Article 4.24 shows the requirements on the minimum dimensions for each specific use function within an occupied zone. A summary can be found in table 79.

Table 79: Minimal dimensions for the various use functions within an occupied zone

<table>
<thead>
<tr>
<th>Use Function</th>
<th>Area [m²]</th>
<th>Width [m]</th>
<th>Free height [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Congregation:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- to consume alcohol</td>
<td>10</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>- other activities</td>
<td>10</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Office</td>
<td>10</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Education</td>
<td>8</td>
<td>1.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Stores (Not a liquor store)</td>
<td>10</td>
<td>1.8</td>
<td>2.6</td>
</tr>
</tbody>
</table>

Grandstand

Within the Building Decree, no demands have been proposed for the dimensions of the grandstands. A grandstand should be considered as a crow-stepped floor, as mentioned in article 2.28. When such a floor is accessible for visitors, the dimensions can be found in table 80.

Table 80: Dimension for a crow-stepped floor, accessible for visitors

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum width</td>
<td>1.1</td>
</tr>
<tr>
<td>Minimum free height above the floor</td>
<td>2.3</td>
</tr>
<tr>
<td>Maximum height</td>
<td>4.0</td>
</tr>
<tr>
<td>Maximum height of the step</td>
<td>0.21</td>
</tr>
<tr>
<td>Minimum width of the step</td>
<td>0.23</td>
</tr>
</tbody>
</table>
Annex B.2 Building codes

This section contains a summary of all relevant standards and loads according to the NEN-EN 1990, the NEN-EN 1991-1, and the NEN-EN 1995-1. Only those regulations which are related to relevant stadium parts, such as grandstands, offices, car parks etc., are discussed. For detailed background information, use can be made of the specific parts of the codes, which is referred to.

B.2.1 General aspects

- A stadium is classified as a life span class 4 building, with an accompanying reference period of 50 years (section 2.3 of the NEN-EN 1990-1-1).
- For replaceable parts of the structure, reference class 2 applies with a reference life span of 15 years (Dutch National Annex to NEN-EN 1990-1-1).
- A stadium is classified in reliability class 3 which coincides with consequence class 3.

The latter implies that structural failure has major consequences concerning life loss (some tens) and/or huge economical or social consequences.

B.2.1.1 Ultimate limit state (ULS)

The ultimate limit state is defined as ‘the limit state concerning the safety of persons and/or structure.

Three different limit states have to be checked, being:

- Loss of equilibrium for (a part of) the structure, while treated as a rigid body (EQU)
- Failure through excessive deformations, change of (a part of) the structure into mechanisms, cracks, (partial) loss of structural stability, including supports and foundations (STR)
- Failure caused by fatigue or other time-dependent effects (FAT)

Section 6.4.3 of NEN-EN 1900-1-1 defines the following load combinations:

\[ \sum_{j=1}^{n} \psi G_{k,j} + \gamma_{p} + \gamma_{Q,k}Q_{k,j} + \sum_{i=1}^{m} \psi_{0,i}Q_{k,i} \text{ (Permanent)} \]

\[ \sum_{j=1}^{n} \xi G_{k,j} + \gamma_{p} + \gamma_{Q,k}Q_{k,j} + \sum_{i=1}^{m} \psi_{0,i}Q_{k,i} \text{ (Quasi-permanent)} \]

\[ \sum_{j=1}^{n} G_{k,j} + P + A_{d} + (\psi_{11} / \psi_{21})Q_{k,j} + \sum_{i=1}^{m} \psi_{2,i}Q_{k,i} \text{ (Accidental)} \]

Since earthquakes are not likely to happen in the Netherlands, this specific load combination is neglected.

The various \( \gamma \)-factors (safety factors), are defined in the national annex and are shown in table 81.

Table 81: Design values for loads

<table>
<thead>
<tr>
<th>Load situations</th>
<th>Permanent loads</th>
<th>Extreme variable load</th>
<th>Other variable load</th>
<th>Others</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQU (6.10)</td>
<td>1.2 ( \psi_{k} )</td>
<td>0.9 ( \psi_{k} )</td>
<td>1.65 ( Q_{k} )</td>
<td>1.65 ( \psi_{k} )</td>
</tr>
<tr>
<td>STR (6.10a)</td>
<td>1.49 ( \psi_{k} ) (1.1 * 1.35)</td>
<td>0.9 ( \psi_{k} )</td>
<td>1.65 ( Q_{k} )</td>
<td>1.65 ( \psi_{k} ) (i &gt; 1)</td>
</tr>
<tr>
<td>STR (6.10b)</td>
<td>1.32 ( \psi_{k} ) (1.1 * 1.2)</td>
<td>0.9 ( \psi_{k} )</td>
<td>1.65 ( Q_{k} )</td>
<td>1.65 ( \psi_{k} )</td>
</tr>
<tr>
<td>Extreme (6.11a/b)</td>
<td>1.0 ( \psi_{k} )</td>
<td>1.0 ( \psi_{k} )</td>
<td>( A_{k} ) or ( \psi_{k} )</td>
<td>( \psi_{k} )</td>
</tr>
</tbody>
</table>

B.2.1.2 Serviceability limit state (SLS)

The serviceability limit state is defined as ‘the limit state concerning the serviceability of the structure, or parts of it, under normal service circumstances, the comfort of its users and the appearance of the structure’.

Three different limit states have to be checked, being:
β Deformations affecting appearance, service comfort or functionality, or causing damage to non-structural elements
β Vibrations causing discomfort or limiting functional effectiveness
β Damage having a negative effect on appearance, durability or functionality

Section 6.5.3 defines the following load combinations:

\[ \sum_{j=1}^{i} G_{k,j} + P + \psi_{j}Q_{k,j} \] (Permanent)

\[ \sum_{j=1}^{i} G_{k,j} + P + \psi_{j}Q_{k,j} \] (Frequent)

\[ \sum_{j=1}^{i} G_{k,j} + P + \psi_{j}Q_{k,j} \] (Quasi-Permanent)

The \( \psi \) factors follow from the NEN-EN 1900 Annex A and are shown in table 86.

B.2.2 Timber specific aspects

The basics for the design in timber is specified in NEN-EN 1995-1-1-2005, also referred to as the Eurocode 5. For now, only the most important aspects are elaborated on.

B.2.2.1 Ultimate Limit State (ULS)

When the ultimate limit state is concerned, the following is mentioned in article 2.2.2 about the stiffness properties:

β The mean values should be used when a first order linear elastic calculation is concerned, in which the internal force distribution is not influenced by the stiffness distribution within the structure.

β The ultimate mean values, adapted to the force component that introduces the highest stress-strength ratio, should be used when the distribution of internal forces is influenced by the stiffness distribution.

β The characteristic values, not adapted to the load duration, should be used for second order linear elastic calculations.

β The slip modulus (in Dutch: verschuivingsmodulus) of a connection in the ultimate limit state should be taken as \( K_{u} = \frac{2}{3} * K_{ser} \).

B.2.2.2 Serviceability Limit State (SLS)

When the serviceability limit state is concerned, article 2.2.3 states that:

β The deformation as a consequence of loads and moisture should be limited, accounting for possible damage, serviceability and appearance.

β The instantaneous deformation, \( u_{inst} \), should be calculated for the characteristic load combination, using the mean values for the modulus of elasticity, shear and translation.

β The final deformation, \( u_{fin} \), should be calculated for the quasi-static load combination.

β When the structure consists of members having a different behaviour on creep, the final deformation should be calculated with the ultimate mean values of the modulus of elasticity, shear and translation.

β For structure having members with equal behaviour on creep, while assuming a linear relation between load and deformation, the ultimate deformation can be taken as:

\[
\begin{align*}
\text{for permanent loads } G & : u_{in,G} = u_{in,G} + u_{in,Q1} + u_{in,Qi} \\
\text{with: } u_{in,G} & = u_{inst,G} (1 + k_{def}) \\
\text{for governing variable loads } Q & : u_{in,Q1} = u_{inst,Q1} (1 + \phi_{1,2} k_{def}) \\
\text{for simultaneous variable loads } Q & : u_{in,Qi} = u_{inst,Qi} (1 + \phi_{2,1} k_{def})
\end{align*}
\]

β When vibrations are considered, the mean values of the suitable stiffness modulus should be used.

B.2.2.3 Load duration classes

The load duration classes are characterised by the effect of a constant load which acts during a certain amount of time within the lifespan of a structure. For calculations on strength and stiffness, all loads should be assigned to one of the follow load duration classes, see table 82.
Table 82: Load duration classes

<table>
<thead>
<tr>
<th>Load duration class</th>
<th>order of magnitude for the cumulative duration of the characteristic load</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>Over 10 years</td>
<td>Self-weight</td>
</tr>
<tr>
<td>Long-term</td>
<td>6 months up to 10 years</td>
<td>Storage</td>
</tr>
<tr>
<td>Medium-term</td>
<td>1 week up to 6 months</td>
<td>Imposed floor loads</td>
</tr>
<tr>
<td>Short-term</td>
<td>less than 1 week</td>
<td>Snow, wind</td>
</tr>
<tr>
<td>Instantaneous</td>
<td></td>
<td>Accidental loads</td>
</tr>
</tbody>
</table>

B.2.2.4 Design values

The design value of a strength property should be determined as:

\[ X_d = k_{mod} \times \frac{X_K}{\gamma_m} \]

in which:

- \( k_{mod} \) = a factor incorporating the influence of load duration and moisture content, see table 83.
- \( X_K \) = the characteristic value for a strength property
- \( \gamma_m \) = a partial factor for a specific material, see table 84.

The design value of a stiffness property is determined as:

\[ E_d = \frac{E_{mean}}{\gamma_m} \quad \text{and} \quad G_d = \frac{G_{mean}}{\gamma_m} \]

Table 83: \( k_{mod} \) for various materials depending on service class and load duration class\(^{(1),(2)}\)

<table>
<thead>
<tr>
<th>Material</th>
<th>Service class</th>
<th>Permanent</th>
<th>Long-term</th>
<th>Medium-term</th>
<th>Short-term</th>
<th>Instantaneous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sawn timber, Glulam, LVL</td>
<td>1</td>
<td>0.60</td>
<td>0.50</td>
<td>0.70</td>
<td>0.50</td>
<td>0.80</td>
</tr>
<tr>
<td>OSB/2</td>
<td>1</td>
<td>0.30</td>
<td>0.50</td>
<td>0.45</td>
<td>0.50</td>
<td>0.85</td>
</tr>
<tr>
<td>OSB/3, OSB/4</td>
<td>2</td>
<td>0.40</td>
<td>0.50</td>
<td>0.70</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>

\(^{(1)}\) the values shown in italic should be used when tensile forces perpendicular to the grain occur, as mentioned in the Draft version of the Dutch Annex 2010.

\(^{(2)}\) When a load combination is comprised of several loads, having various load duration classes, the \( k_{mod} \) value that belongs to the shortest load duration should be used.

Table 84: \( \gamma_m \) factor for various materials

<table>
<thead>
<tr>
<th>Fundamental combination:</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sawn timber</td>
<td>1.3</td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>1.25</td>
</tr>
<tr>
<td>LVL, plywood, OSB</td>
<td>1.2</td>
</tr>
<tr>
<td>Connections</td>
<td>1.3</td>
</tr>
<tr>
<td>Metal plate connectors</td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Extreme combination:</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>All types</td>
<td>1.0</td>
</tr>
</tbody>
</table>

When deformation is at stake, the deformation factor \( k_{def} \) should be incorporated. For various materials, in combination with the available service classes, the value of \( k_{def} \) is given in table 85.

Table 85: \( k_{def} \) for various materials depending on service class

<table>
<thead>
<tr>
<th>Material</th>
<th>Service class</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sawn timber, Glulam, LVL</td>
<td>0.60</td>
<td>0.80</td>
<td>2.00</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>0.80</td>
<td>1.00</td>
<td>2.50</td>
<td></td>
</tr>
<tr>
<td>OSB/2</td>
<td>2.25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OSB/3, OSB/4</td>
<td>1.50</td>
<td>2.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
B.2.2.5 Material specific properties

B. Sawn timber
For rectangular sawn timber, having a characteristic density $\rho \leq 700 \text{ kg/m}^3$, the reference height for bending or the reference width for tension is 150 mm. When the dimensions of a sawn timber member are smaller than 150 mm, the characteristic values of $f_{m;k}$ and $f_{t0;k}$ may be multiplied with a factor $k_h$.

$$k_h = \min \left[ \left( \frac{150}{h} \right)^{0.2}, \frac{1.3}{h} \right]$$

in which $h$ is the height (bending) or the width (tension)

For sawn timber, placed with a moisture content at or around the point of saturation for the fibre, likely to dry under loading, the above given values for $k_{w,r}$ should be raised by 1.0.

B. Glued laminated timber
For rectangular glued laminated timber the reference height for bending or the reference width for tension is 600 mm. When the dimensions of a glulam member are smaller than 600 mm, the characteristic values of $f_{m;k}$ and $f_{t0;k}$ may be multiplied with a factor $k_h$.

$$k_h = \min \left[ \left( \frac{600}{h} \right)^{0.1}, \frac{1.1}{h} \right]$$

in which $h$ is the height (bending) or the width (tension)

B. Laminated Veneer Lumber (LVL)
For LVL members, having the same fibre direction for all veneers, the reference height for bending is 300 mm. When the dimensions of a LVL member are smaller than 300 mm, the characteristic value of $f_{m;k}$ may be multiplied with a factor $k_h$.

$$k_h = \min \left[ \left( \frac{300}{h} \right)^{0.2}, \frac{1.2}{h} \right]$$

in which $h$ is the height (bending) and $s$ is the size-effect parameter

For LVL members, having the same fibre direction for all veneers, the reference width for tension is 3000 mm. When the length of a LVL member is smaller than 3000 mm, the characteristic value of $f_{t0;k}$ may be multiplied with a factor $k_l$.

$$k_l = \min \left[ \left( \frac{3000}{l} \right)^{0.2}, \frac{1.1}{l} \right]$$

B.2.3 Imposed loads
In the Eurocode, a distinction is made between various functional areas which provides the loads that should be accounted for, see table 86. For the sake of completeness, all categories are displayed here, but only the values to be used are marked (Italic).

<table>
<thead>
<tr>
<th>Cat.</th>
<th>Specific Use</th>
<th>$q_k$ [kN/m$^2$]</th>
<th>$Q_k$ [kN]</th>
<th>$\psi_0$</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Domestic and residential areas</td>
<td>1.75</td>
<td>3</td>
<td>0.4</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>A-floors</td>
<td>2.0</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A-stairs</td>
<td>2.5</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Office areas</td>
<td>2.5</td>
<td>3</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>C</td>
<td>Congregating areas</td>
<td>4.0</td>
<td>7</td>
<td>0.25</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>C1-tables</td>
<td>4.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C2-fixed seats</td>
<td>5.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3-no moving obstacles</td>
<td>5.0</td>
<td>7</td>
<td>0.25</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>C4-physical activities</td>
<td>5.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C5-large crowds</td>
<td>5.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Shopping areas</td>
<td>4.0</td>
<td>7</td>
<td>0.4</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>D1-retail</td>
<td>4.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D2-department</td>
<td>4.0</td>
<td>7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>Storage</td>
<td>$\geq 5.0$</td>
<td>$\geq 7$</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>E1-shops</td>
<td>$\geq 2.5$</td>
<td>$\geq 3$</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>E1-libraries</td>
<td>$\geq 5.0$</td>
<td>$\geq 10$</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>E2-industrial use</td>
<td>over 3</td>
<td>over 7</td>
<td>1.0</td>
<td>0.9</td>
<td>0.8</td>
</tr>
</tbody>
</table>
For areas which play an important part when safety is concerned (escape routes), the design loads are even more stringent. These values are shown in Table 87.

### Table 87: Characteristic values of actions on corridors of various categorised areas

<table>
<thead>
<tr>
<th>Cat.</th>
<th>Specific Use</th>
<th>( q_i ) [kN/m²]</th>
<th>( Q_i ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Domestic and residential areas</td>
<td>2.0</td>
<td>3.0(1)</td>
</tr>
<tr>
<td>B</td>
<td>Offices</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>C</td>
<td>Congregation areas</td>
<td>5.0</td>
<td>7.0</td>
</tr>
<tr>
<td>D</td>
<td>Shopping areas</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>E</td>
<td>Storage</td>
<td>E1-Libraries</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E1-Others</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E2-Industrial use</td>
<td>4.0</td>
</tr>
</tbody>
</table>

(1) Active on an area of 0.5 m * 0.5 m

Within these tables, various \( \psi \) factors can be distinguished. These factors deal with the simultaneous chance of occurrence for the various loads:

- \( \psi_0 \) = factor due to the combination value of a variable load
- \( \psi_1 \) = factor due to the frequent value of a variable load
- \( \psi_2 \) = factor due to the quasi-permanent value of a variable load

### B.2.4 Wind loads

The wind loads are determined according to the values given in the NEN-EN 1991-1.4. Wind loads are variable, fixed actions, being composed of both a basic and a fluctuating component.

To determine the force induced by the wind, at first the mean wind speed \( \left( v_{m} \right) \) has to be determined, by making use of the basic wind speed \( \left( v_{b} \right) \). This mean wind speed depends on the local wind climate as well as on the height variations of the wind.

#### B.2.4.1 Basic wind speed

The basic wind speed is determined as a function of the governing wind direction and the season, taken at a reference height of 10 m above ground level. Since a conservative approach is desirable, the wind direction \( \left( c_{\text{dir}} \right) \) and season factors \( \left( c_{\text{season}} \right) \) are taken 1.0. Therefore, the basic wind speed \( \left( v_{b} \right) \) is equal to the fundamental value of the basic wind speed \( \left( v_{b;0} \right) \).

As far as the Netherlands is concerned, three different wind areas are distinguished. Each area has its own fundamental wind speed, see figure 72. The exact division of the country in these areas is found in the national annex to Eurocode 1, part 1.4.
Table 88: Values of the fundamental wind speed for the various wind areas in the Netherlands

<table>
<thead>
<tr>
<th>Area</th>
<th>$v_b$ [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>29.5</td>
</tr>
<tr>
<td>II</td>
<td>27.0</td>
</tr>
<tr>
<td>III</td>
<td>24.5</td>
</tr>
</tbody>
</table>

When the Euroborg stadium is concerned, wind area II is at stake (Groningen). Therefore, the fundamental (and thus the basic) wind speed is 27.0 m/s.

B.2.4.2 Mean wind speed

The average wind speed $v_m(z)$, at a height of $z$ [m] above ground level, depends on the terrain harshness, the orography and the basic wind speed $v_b$. This leads to the following formula:

$$v_m(z) = c_r(z) * c_o(z) * v_b$$

The orography factor, $c_o$, is taken to be 1.0, the terrain harshness factor ($c_r$) is defined as:

$$c_r = 0.19 * \left( \frac{z_0}{0.05} \right)^{0.7}$$

where $z_0$ has to be determined in relation to the terrain harshness (see table 89).

<table>
<thead>
<tr>
<th>Terrain category</th>
<th>$z_0$ [m]</th>
<th>$z_{min}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Sea or coast</td>
<td>0.005</td>
<td>1</td>
</tr>
<tr>
<td>II Uncultivated regions</td>
<td>0.2</td>
<td>4</td>
</tr>
<tr>
<td>III Cultivated regions</td>
<td>0.5</td>
<td>7</td>
</tr>
</tbody>
</table>

This results in:

$$v_m(z) = 0.19 * \left( \frac{z_0}{0.05} \right)^{0.7} * \ln \left( \frac{z}{z_0} \right) * v_b$$

The environment of the stadium is regarded an uncultivated area, since it is a desolate terrain during construction. This leads to the governing $z_r$, being 0.2 m in this case. With a total height of around 25 m for the Euroborg, the height of the stadium remains within limits. Accounting for these values, incorporating the basic wind speed, it is found that the mean wind speed $v_m$ is around 27.3 m/s.

B.2.4.3 Turbulence

The turbulence intensity $l_v(z)$ at a height $z$, is determined as the standard deviation from the turbulence divided by the mean wind speed. For the standard deviation of wind turbulence it is found that:

$$\sigma_v = k_t \cdot v_b \cdot k_i = 0.19 \cdot \left( \frac{z_0}{0.05} \right)^{0.7} \cdot v_b$$

since it is suggested to take $k_i = 1.0$:

This results in:

$$l_v(z) = \frac{1}{\ln \left( \frac{z}{z_0} \right)}$$

assuming $z_{min} \leq z \leq z_{max}$

Accounting for the Euroborg stadium, with a height of 25 m, this leads to $l_v(25) = 0.2071$

B.2.4.4 Extreme wind driving pressure

The extreme wind driving pressure at a height $z$, including the mean wind speed and the short-term speed fluctuations, is determined now:

$$q_p(z) = (1 + 7l_v(z))^* \left( 0.5 \rho_{air} * v_m^2 \right) = c_d(z) * q_b$$

in which $\rho_{air} = 1.25$kg/m$^3$; $c_d(z) = \frac{q_b(z)}{q_b}$; $q_b = 0.5 \rho_{air} * v_b^2$

For the Euroborg stadium, this leads to:

$$q_p(25) = (1 + 7*0.2071)^* \left( 0.5 * 1.25 * 27.3^2 \right) = 1.14$${kN/m$^2$}

This value corresponds to the value found in Eurocode 1, part 1.4, National Annex table 4. It is therefore concluded that these tables can be used for further design purposes.
B.2.4.5 Wind pressure

To determine the wind pressure on a building, a deviation is made between internal and external pressure. Therefore, the pressure coefficients, $c_{pe}$ and $c_{pi}$, are introduced. In addition, for roofs and coverings or detached walls and facades, a net pressure coefficient is introduced, $c_{p,net}$.

**Internal and external pressure coefficients**

The external pressure coefficient is dependent on the dimensions of the surface ($A$) under action. The coefficient are given for areas of 1 m$^2$ and 10 m$^2$ for the specific building configuration, respectively as $c_{pe,1}$ for local coefficients and $c_{pe,10}$ for global coefficients. The factor $c_{pe,1}$ concerns small parts of the structure, such as facade elements. Since the structure of the stadium is at stake within this thesis, only the factor $c_{pe,10}$ is of importance.

To determine the pressure coefficients that apply on the stadium structure, at first the shape of the stadium is determined. The stadium consists of 4 building parts, referred to as the North, East, South or West part. Within this thesis, only the East part is considered, see figure 234.

**Wind in EW-direction**

This East part of the stadium is considered having a rectangular plan, while the roof is a so-called shed roof. It shall be clear that the height of the stadium is not equal to the width (in this case $h < b \approx 25 \text{ m} < 34 \text{ m}$). Therefore, for the facade, the following situation applies:

![Diagram](image)

This leads to the following external coefficients for specific zones of the building, see figure 236.
The coefficients concerning this specific situation are extracted from table 7.1 of the National Annex to Eurocode 1, part 1-4. The results are shown in table 90.

Table 90: External wind pressure coefficients for specific building zones.

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{pe;10}$</td>
<td>$c_{pe;1}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;1}$</td>
<td>$c_{pe;10}$</td>
</tr>
<tr>
<td>-1.2</td>
<td>-1.4</td>
<td>-0.8</td>
<td>-1.1</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

When the roof is considered, a distinction is made between the possible load directions, which are referred to as $\theta = 0^\circ$ and $\theta = 180^\circ$. The reason for this can be found in the cantilevering part of the roof structure. When the wind approaches the structure from the cantilevering part, problems might occur due to the high wind coefficients.

When the Euroborg stadium is concerned, the roof has an angle of $8^\circ$. Making use of table 7.3a of the National Annex to Eurocode 1, part 1-4, the following can be obtained by interpolating between the given values:

Table 91: External load coefficients on the roof ($\theta = 0^\circ$)

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;1}$</td>
</tr>
<tr>
<td>-1.46</td>
<td>-2.35</td>
<td>-1.08</td>
<td>-1.85</td>
<td>-0.51</td>
</tr>
</tbody>
</table>

Making use of table 7.3a and table 8 of the National Annex to Eurocode 1, part 1-4, the following is obtained for the roof coefficients, by interpolating between the given values:

Table 92: External load coefficients on the roof ($\theta = 180^\circ$)

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;10}$</td>
<td>$c_{pe;1}$</td>
</tr>
<tr>
<td>-2.00</td>
<td>-1.60</td>
<td>-2.36</td>
<td>-2.59</td>
<td>-1.30</td>
</tr>
</tbody>
</table>

| + 0.7 | + 0.3 |

Figure 236: External load coefficients on the façade and the roof
It has to be mentioned, that not all areas A to H are considered in this situation. For example, when the wind coefficients on the roof are considered for $\theta = 0^\circ$, it is found that the coefficients for area F are not used. The reason for this lies in the fact, that these roof areas are located near edges, and only influence a small part of the roof. These local coefficients have been neglected, since the focus is on the average structure and not on local influences.

In addition, it should be mentioned that the figures and values as shown in figure 236 and table 92, are obtained by combining two distinct load situations, i.e. the combination of a wind load on a canopy and the wind load on a shed roof.

**Wind in NS-direction**

When wind in the NS-direction is concerned, the following external coefficients are found for the specific zones of the building, see figure 237.

The coefficients concerning this specific situation are extracted from table 7.1 of the National Annex to Eurocode 1, part 1-4. The results are shown in table 93. It has to be mentioned, that for a global overview, only the $c_{pe10}$ values have to be considered.

![Figure 237: Areas where the external load coefficients on the roof apply ($\theta = 90^\circ$)](image)

The values that apply near the edges of this building part, the above mentioned F areas, are neglected here. The focus will be on the centre of this part of the structure.
B.2.4.6 Structural coefficient

The structural factor \( c_{sc} \) takes into account the effect of wind actions from the non-simultaneous occurrence of peak wind pressures on the surfaces \( (c_s) \) together with the effect of the vibrations of the structure due to turbulence \( (c_d) \). For the first approach, it is acceptable to take \( c_{sc} \) conservatively as 1.0.

B.2.4.7 Wind pressure on surfaces

By making use of the various values obtained earlier in this section, the wind pressure on both internal and external surfaces are obtained, see figure 238.

The wind acting on the external surfaces:
\[
w_e = q_p(z_e) \cdot c_{pe} \quad [\text{kN/m}^2]
\]

The wind acting on the internal surfaces:
\[
w_i = q_p(z_i) \cdot c_{pi} \quad [\text{kN/m}^2]
\]

Figure 238: Wind pressure on surfaces

B.2.4.8 Wind forces

According to Eurocode 1, part 1-4, the governing wind force is calculated by:
\[
F_w = c_{sc} \cdot c_{sd} \cdot c_f \cdot q_p(z_e) \cdot A_{ref}
\]

The values for \( c_{sc}, c_d, \) and \( q_p(z_e) \) are determined earlier in this section, and are used to determine the wind force on the structure. For structural designing, use is made of an equally divided wind load on a structure:
\[
p_w = 1 \cdot 1.14 \cdot b \quad [\text{kN/m}] \text{ (excluding pressure coefficients)}
\]

The governing \( \Psi \)-factors for wind loads on buildings follow from the NEN-EN 1990 (see A1.2.2):
\[\Psi_0 = 0, \quad \Psi_1 = 0.2 \text{ and } \Psi_2 = 0\]

B.2.5 Snow loads

Due to the moderate climate, the National Annex section 5.2 to the NEN-EN 1991-1-3:2003 states, that in the Netherlands no extraordinary snowfall or snowdrift has to be accounted for. Snow loads only have to be accounted for when permanent situations are at stake. The characteristic snow load is defined as:
\[
s = \mu_1 \cdot C_e \cdot C_t \cdot s_k
\]

In which:
\[\mu_1 = \text{roof shape coefficient}\]
\[C_e = \text{exposing coefficient}\]
\[C_t = \text{thermal coefficient}\]
\[s_k = \text{characteristic value of the snow load on the ground}\]

Paragraph 5.2 of the National Annex states, that in the Netherlands, \( C_e = 1.0 \) for every building and \( C_t = 1.0 \) for every location. For a shed roof having an angle of 8°, as for the Euroborg stadium, paragraph 5.3.2 of NEN-EN 1991-1-3:2003 states that \( \mu_1 = 0.8 \).

For every area in the Netherlands, the characteristic snow load \( s_k \) on the ground should be taken 0.7 kN/m².

This leads to a characteristic snow load on the roof of 0.56 kN/m² (0.8 * 0.7 kN/m²).

The governing \( \Psi \)-factors for snow loads on buildings follow from the NEN-EN 1990 (see A1.2.2):
\[\Psi_0 = 0, \quad \Psi_1 = 0.2 \text{ and } \Psi_2 = 0\]
**B.2.6 Thermal actions**

Thermal actions on buildings caused by climate and industrial processes are determined in compliance to national (or regional) data and experience. These actions are taken into account when the possibility exists, that the ULS or SLS is exceeded by thermal deformations or thermal stresses.

Next to a variance in temperature, these actions can be influenced by shading of neighbouring structures, combined use of several materials with differing thermal expansion coefficients or the use of various cross-sectional shapes.

- $T$ is the average temperature of a structural element caused by climatological temperatures in summer or winter and by industrial processes. The governing values for $T$ are given in table 95.
- The governing inside temperatures in the Netherlands, for both summer and winter, are $T_1$ and $T_2$, both being 17°C.

The governing outside temperatures in the Netherlands, for structural elements above ground level, are shown in table 95. These values depend on the orientation of the outer face and the absorbing capacity.

<table>
<thead>
<tr>
<th>Season</th>
<th>Significant factor</th>
<th>Temperature $T_{out}$ in °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summer</td>
<td>Relative absorbing capacity depending on outside colour</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.5 (very light)</td>
<td>$T_{max} + T_1 = 30°C + 20°C$</td>
</tr>
<tr>
<td></td>
<td>0.7 (light)</td>
<td>$T_{max} + T_1 = 30°C + 30°C$</td>
</tr>
<tr>
<td></td>
<td>0.9 (dark)</td>
<td>$T_{max} + T_1 = 30°C + 45°C$</td>
</tr>
<tr>
<td>Winter</td>
<td>-</td>
<td>$T_{max} = -25°C$</td>
</tr>
</tbody>
</table>

These values are maximum values and are usually only obtained for surfaces that face the west or southwest side or for horizontal surfaces.

For structural elements below ground level, the governing outside temperatures are shown in table 96.

<table>
<thead>
<tr>
<th>Season</th>
<th>Significant factor</th>
<th>Temperature $T_{out}$ in °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summer</td>
<td>Less than 1 m below ground level</td>
<td>$T_1 = 10°C$</td>
</tr>
<tr>
<td></td>
<td>More than 1 m below ground level</td>
<td>$T_1 = 10°C$</td>
</tr>
<tr>
<td>Winter</td>
<td>Less than 1 m below ground level</td>
<td>$T_1 = 10°C$</td>
</tr>
<tr>
<td></td>
<td>More than 1 m below ground level</td>
<td>$T_1 = 10°C$</td>
</tr>
</tbody>
</table>

**Linear expansion coefficient**

For the linear expansion coefficient $\alpha$, the following values are stated in Annex C of NEN-EN 1991-1-5:

- Timber, parallel to the grain: $5 \times 10^{-6}/°C$
- Timber, perpendicular to the grain: $30-70 \times 10^{-6}/°C$ (depending on the species)
- Structural Steel: $12 \times 10^{-6}/°C$ (10 for hybrid structures)
- Concrete: $10 \times 10^{-6}/°C$

**B.2.7 Serviceability limit conditions**

**B.2.7.1 Deflections**

The maximum values for vertical deflections are defined in NEN 6702, section 10.

**Requirements on additional deflections**

- Floors: $u \leq 0.003* l_{rep}$
- Cantilevering floors: $u \leq 0.003* l_{rep}$ (with $l_{rep} = 2*l$)
- Floors with partition: $u \leq 0.002* l_{rep}$
- Roofs: $u \leq 0.004* l_{rep}$
- Cantilevering roofs: $u \leq 0.004* l_{rep}$ (with $l_{rep} = 2*l$)
Requirements on ultimate deflections

- Floors: \( u \leq 0.004 \times l_{\text{rep}} \)
- Cantilevering floors: \( u \leq 0.004 \times l_{\text{rep}} \) (with \( l_{\text{rep}} = 2 \times l \))
- Roofs: \( u \leq 0.004 \times l_{\text{rep}} \)
- Cantilevering roofs: \( u \leq 0.004 \times l_{\text{rep}} \) (with \( l_{\text{rep}} = 2 \times l \))

Requirements on horizontal displacements

Considering a multi-storey building, the following is stated in 10.3:

- Typical storey: \( u \leq h/300 \), in which \( h \) is the maximum height of the storey
- Whole building: \( u \leq h/500 \), in which \( h \) is the smallest façade height

B.2.7.2 Vibrations

Within the serviceability limit state, the structure has to be checked for vibrations. These vibrations might cause significant problems, when not been dealt with properly. Since the focus is on a stadium structure, the focus will be on the vibrations of the timber floors and vibrations of the grandstands. Vibrations on the structure caused by the wind or machines are neglected.

Vibrations might be disturbing when they are perceptible, which can be both visual and sensible. In the design standards the focus is set to the vibrations that can be felt, since it is hardly possible to provide a guideline to prevent disturbance caused by visual vibrations. The reason for that can be found in the wide range of natural frequencies and the damping of all the objects within a building. Therefore, when vibrations are mentioned within this thesis, only sensible vibrations are being meant, unless stated otherwise.

Since not all design standards provide the same guidelines on vibrations, at first, a summary will be given of the guidelines as stated in the Dutch NEN, the Eurocodes and the SBR ‘Trillingen op vloeren door lopen’. Finally, it will be determined which guideline will be followed in this thesis.

**NEN 6702**

Section 10.5 of the NEN 6702 provides guidelines to prevent resonance caused by moving people on floors. It is stated that ‘vibrations should never obstruct the functional use of a structural element and should cause no damage. Hindrance should be avoided to the greatest extent.’

- For floors that are mainly used for walking, the minimal natural frequency should exceed 3 Hz, since this is the maximum frequency that can be caused by walking people.
- When a load of 5 kN/m^2 or 150 kN is available at the floor, one is allowed to assume that no sensible vibrations will be caused by people walking on the floor.
- When a floor is concerned which is meant for dancing, the frequency should exceed 5 Hz.

Accounting for the specific support conditions (see NEN 6702 Annex A.4), these values can be converted to a maximal acceptable deflection \( \delta \):

\[
f_e = \frac{\alpha}{\delta}
\]

in which \( \alpha \) is the numerical value of the vibration acceleration

**NEN-EN 1990-1 and 1995-1**

Annex A.1.4.4 of the NEN-EN 1990-1 states that ‘to provide an acceptable behaviour of buildings and structural elements on vibrations in the serviceability limit state, the aspects of user comfort and functional usability should be considered. Other aspects should be discussed with the project’s principal.’

As within the NEN, this situation is reached by preventing the first natural frequency of the structure to drop beneath the natural frequencies caused by the use of the structure. These values are determined together with the project’s principal and/or with a governmental body.

For calculations concerning vibrations, the mean value of the stiffness modulus should be used, according to NEN-EN 1995-1. In addition to the above, this guideline states that the level of vibration should be determined by measurements or calculations including the mean damping factor (\( \xi \)). For floors, this factor should be taken 1%. 
For floors in housing, additional research should be performed when the natural frequency is lower than 8 Hz. When the natural frequency is above 8 Hz, it should be checked that:

\[ \frac{w_a}{F} \leq 1 \text{ [mm/kN]} \]  
\[ v \leq b^{1/5} \text{ [m/NS]} \]  

in which \( w \) is the initial deflection by a vertical static load \( F \)

\( v \) is the velocity response [m/s] cause by a unity impulse load (1 Ns)

\( b = 120 \) as stated in Draft National Annex 2010

8 Papers on vibrations due to human activities

Floors vibrations due to human activity [60]

This guideline provides some design criteria for rhythmic events, which are shown in table 97.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Floor weight [kN/m²]</th>
<th>Minimal required ( f_e ) [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dining and dancing</td>
<td>5.0</td>
<td>6.4</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>8.1</td>
</tr>
<tr>
<td>Lively concerts or sports events</td>
<td>5.0</td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>6.4</td>
</tr>
<tr>
<td>Aerobics only</td>
<td>5.0</td>
<td>8.8</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>9.2</td>
</tr>
<tr>
<td>Jumping exercises shared with weight lifting</td>
<td>5.0</td>
<td>9.2</td>
</tr>
<tr>
<td></td>
<td>2.5</td>
<td>10.6</td>
</tr>
</tbody>
</table>

Dynamic performance of stands [61]

Within this paper, two distinct human dynamic actions are distinguished; being a sudden concerted action (standing) and repeated actions (jumping). The first action produces transient vibrations which die away quickly. The second action is often in response to a musical beat, leading to higher natural frequencies.

For large groups, it is hard to jump with good coordination at frequencies above 2.75 Hz. Despite that, stands are only suitable for any type of event, when the vertical natural frequency is above 6 Hz. It is questionable if such a high demand should be incorporated in the design, since it asks a lot of the grandstand structure. Table 98 provides acceptable design values for a grandstand.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Vertical [Hz]</th>
<th>Horizontal [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sports</td>
<td>3.5</td>
<td>1.75</td>
</tr>
<tr>
<td>Pop concerts</td>
<td>7.0</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Conclusion

When the building codes as stated above are considered, it is found that these provide highly simplified design procedures. Although the focus in this thesis is mainly on the Eurocodes, for the dynamic behaviour the procedure as stated in the NEN is adopted. The reason lies in the fact that the provided design procedure directly provides insight in the natural frequency of a specific structural element. As a result, the obtained values can directly be compared to the required values.

In the Eurocode, for residential floors, a minimal natural frequency of 8 Hz is required. In the NEN, a minimal natural frequency of 3 Hz is required for floors that have walking as their main purpose. Dialogue with Mark Spanenburg, an expert of BAM A&E on dynamic behaviour, clarified that it is highly conservative to consider a natural frequency exceeding 8 Hz for the floors within a stadium structure. It is recommended to consider a value of 5 Hz as a first assumption. This value has therefore been adopted in the design of floors.

When the grandstands are concerned, it is concluded from the literature, that there is still a lot of indistinctness on the exact behaviour and the requirements. Consultation with the projects’ principal clarified that the stadium is not meant for hosting pop concerts. As a result, lower natural frequencies may be adopted in the design.

In this thesis, a minimal required natural frequency of 5 Hz is adopted in the design. This value is somewhat larger than minimal required value as stated in table 98, and somewhat smaller than the value for light-weight floors as presented in table 97.
Annex C Structural systems

In this annex, all backgrounds on the design of the structural systems is provided. It is therefore highly recommended to consider this Annex in the light of chapter 4 Structural system.

In this Annex, the same build-up as in chapter 4 is used. At first attention is paid to the timber central core system (Annex C.1), subsequently to the system of small shear walls (Annex C.2) and finally to the system consisting of trusses (Annex C.3).
Annex C.1 Central core system

C.1.1 Introduction
To provide the reader with some understanding on the structural build-up of the timber core, a section of the core is provided in figure 239. For all relevant information on the core lay-out, reference is made to section 4.4.

Figure 239: Sections of the fully enclosed core and the core with openings

C.1.2 Loads
In this section, the various loads that might act on the structure are shown. The loads follow directly from the weight calculation as shown in Annex G. These loads have been determined while accounting for the structural plan of the Euroborg stadium and should therefore be considered preliminary only.

One should note, that a total of three cores is incorporated in the design. Therefore, the loads are equally divided over these cores.

Making use of the weight calculation, the total amount of additional loading is determined. This load consists of a permanent part (dead load) and a variable part (live loads). The eccentric point of action of this vertical load has been neglected here.

At first, it is determined which part of this load, is actually transferred to the central cores. Therefore, the area of influence of the core should be determined. This has been performed for the 3rd, the 2nd, the 1st, the ground and the -1st floor, resulting in the following overview:

<table>
<thead>
<tr>
<th>Situation:</th>
<th>Floor</th>
<th>Stabilized area by one core [m²]</th>
<th>Area directly influenced [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3rd</td>
<td>765</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>903</td>
<td>108</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>765</td>
<td>108</td>
</tr>
</tbody>
</table>
It is assumed that the load following from the roof, is directly carried by the roof trusses to the columns. Therefore, the core does not play a role in the transfer of these loads.

\[
\begin{align*}
G_{\text{dead;core;rep}} &= (5 \times 0.27 \times 2 + 5 \times 0.249 \times 2) \times 19.22 \times 5.1 = 510 \text{ kN (self-weight core)} \\
G_{3\text{rd floor}} &= (0.78 \times 2.91 + 0.22 \times 1.91) \times 108 = 290 \text{ kN} \\
G_{2\text{nd floor}} &= (1.0 \times 3.14) \times 108 = 339 \text{ kN} \\
G_{1\text{st floor}} &= (1.0 \times 3.14) \times 108 = 339 \text{ kN} \\
G_{-1\text{st floor}} &= (0.64 \times 1.62 + 0.36 \times 3.14) \times 134 = 290 \text{ kN} \\
G_{\text{total}} &= 510 + 290 + 339 + 339 + 420 + 290 = 2188 \text{ kN}
\end{align*}
\]

For the variable part of these loads is found:

\[
\begin{align*}
Q_{3\text{rd floor}} &= 4.0 \times 108 = 432 \text{ kN} \quad \left(\Psi_0 = 1.0; \Psi_1 = 0.9; \Psi_2 = 0.8\right) \\
Q_{2\text{nd floor}} &= 2.5 \times 108 = 270 \text{ kN} \quad \left(\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3\right) \\
Q_{1\text{st floor}} &= 2.5 \times 108 = 270 \text{ kN} \quad \left(\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3\right) \\
Q_{\text{ground floor}} &= 5.0 \times 134 = 670 \text{ kN} \quad \left(\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6\right) \\
Q_{-1\text{st floor}} &= 2.0 \times 134 = 268 \text{ kN} \quad \left(\Psi_0 = 0.7; \Psi_1 = 0.7; \Psi_2 = 0.6\right)
\end{align*}
\]

The wind loads have been determined in B.2.4:

\[
\begin{align*}
Q_{w;\text{EW-direction;rep}} &= \frac{1}{3} \times 1.14 \times 1.3 \times 164 = 81 \text{ kN/m} \quad \left(\Psi_0 = 0.0; \Psi_1 = 0.2; \Psi_2 = 0.0\right) \\
Q_{w;\text{NS-direction;rep}} &= \frac{1}{3} \times 1.14 \times 1.3 \times 34 = 17 \text{ kN/m} \quad \left(\Psi_0 = 0.0; \Psi_1 = 0.2; \Psi_2 = 0.0\right)
\end{align*}
\]

For the grandstand structure, a horizontal load should be introduced which equals 10% of the vertical load. This load is assumed to be divided over the -2nd, -1st and the ground floor for the lower grandstand, and over the column in front of the 1st floor and the 2nd floor for the upper grandstand. The load per floor, divided for the lower (1) and upper (2) grandstand, then becomes:

\[
\begin{align*}
Q_{\text{grandstand 1; hor;rep}} &= \frac{1}{3} \times \frac{1}{3} \times 0.10 \times 11309 = +/\/- 126 \text{ kN} \quad \left(\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6\right) \\
Q_{\text{grandstand 2; hor;rep}} &= \frac{1}{3} \times \frac{1}{3} \times 0.10 \times 9266 = +/\/- 155 \text{ kN} \quad \left(\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6\right)
\end{align*}
\]

Since the worst combination is obtained when the direction of this load coincides with the wind direction, this values will be taken positive.

### C.1.3 Load combinations

Since the stability of the structure is considered, use have to be made of the following load combinations:

- **ULS 1:** \[1.49 \times G_{\text{k;sup}} + 1.65 \times \Psi_{0;i} \times Q_{k;i}\]
- **ULS 2:** \[0.90 \times G_{\text{k;sup}} + 1.65 \times \Psi_{0;i} \times Q_{k;i}\]
- **ULS 3:** \[1.32 \times G_{\text{k;sup}} + 1.65 \times \Psi_{0;i} \times Q_{k;i}\]
- **SLS 1:** \[1.0 \times G_{\text{k;sup}} + 1.0 \times Q_{k;i}\]
- **SLS 2:** \[1.0 \times G_{\text{k;sup}} + 1.0 \times \Psi_{1;i} \times Q_{k;i}\]
- **SLS 3:** \[1.0 \times G_{\text{k;sup}} + 1.0 \times \Psi_{2;i} \times Q_{k;i}\]
This leads to the following combinations for the EW-direction:

### Table 99: Load combinations for the EW-direction

<table>
<thead>
<tr>
<th>Situation</th>
<th>Permanent</th>
<th>Extreme variable</th>
<th>Other variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS 1a:</td>
<td>1.49*2188 (, )</td>
<td>-</td>
<td>1.65 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, ) + 1.65 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>ULS 2a:</td>
<td>0.9 * 2188 (, )</td>
<td>-</td>
<td>1.65 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, ) + 1.65 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>ULS 3a:</td>
<td>1.32 * 2188 (, )</td>
<td>1.65 * 670 (, )</td>
<td>1.65 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.7 * 268) (, ) + 1.65 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>ULS 3b:</td>
<td>1.32 * 2188 (, )</td>
<td>1.65 * 81 (#)</td>
<td>1.65 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, ) + 1.65 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>ULS 3c:</td>
<td>1.32 * 2188 (, )</td>
<td>1.65 * 155 + 1.65 * 126 (#)</td>
<td>1.65 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, )</td>
</tr>
<tr>
<td>SLS 1a:</td>
<td>1.0 * 2188 (, )</td>
<td>1.0 * 670 (, )</td>
<td>1.0 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, ) + 1.0 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>SLS 1b:</td>
<td>1.0 * 2188 (, )</td>
<td>1.0 * 81 (#)</td>
<td>1.0 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, ) + 1.0 * (0.25 * 126 + 0.25 * 126 + 0.25 * 126 + 0.25 * 126) (, )</td>
</tr>
<tr>
<td>SLS 1c:</td>
<td>1.0 * 2188 (, )</td>
<td>1.0 * 155 + 1.0 * 126 (#)</td>
<td>1.0 * (1.0 * 432 + 0.5 * 270 + 0.5 * 270 + 0.25 * 670 + 0.7 * 268) (, )</td>
</tr>
<tr>
<td>SLS 2a:</td>
<td>1.0 * 2188 (, )</td>
<td>0.7 * 670 (, )</td>
<td>1.0 * (0.8 * 432 + 0.3 * 270 + 0.3 * 270 + 0.6 * 268 (, ) + 1.0 * (0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126) (, )</td>
</tr>
<tr>
<td>SLS 2b:</td>
<td>1.0 * 2188 (, )</td>
<td>0.2 * 81 (#)</td>
<td>1.0 * (0.8 * 432 + 0.3 * 270 + 0.3 * 270 + 0.6 * 268 (, ) + 1.0 * (0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126) (, )</td>
</tr>
<tr>
<td>SLS 2c:</td>
<td>1.0 * 2188 (, )</td>
<td>0.7 * 155 + 0.7 * 126 (#)</td>
<td>1.0 * (0.8 * 432 + 0.3 * 270 + 0.3 * 270 + 0.6 * 268 (, ) + 1.0 * (0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126) (, )</td>
</tr>
<tr>
<td>SLS 3a:</td>
<td>1.0 * 2188 (, )</td>
<td>-</td>
<td>1.0 * (0.8 * 432 + 0.3 * 270 + 0.3 * 270 + 0.6 * 268 (, ) + 1.0 * (0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126 + 0.6 * 126) (, )</td>
</tr>
</tbody>
</table>

These combinations lead to the following values for the acting forces:

### Table 100: Acting forces in the EW-direction, following from the governing load combinations

<table>
<thead>
<tr>
<th>Situation</th>
<th>N [kN]</th>
<th>q_w [kN/m]</th>
<th>F_{n2,2nd} [kN]</th>
<th>F_{n2,1st} [kN]</th>
<th>F_{n2,1st} [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS 1a:</td>
<td>5005</td>
<td>-</td>
<td>64</td>
<td>52</td>
<td>52</td>
</tr>
<tr>
<td>ULS 2a:</td>
<td>3713</td>
<td>-</td>
<td>64</td>
<td>52</td>
<td>52</td>
</tr>
<tr>
<td>ULS 3a:</td>
<td>5461</td>
<td>-</td>
<td>64</td>
<td>52</td>
<td>52</td>
</tr>
<tr>
<td>ULS 3b:</td>
<td>4632</td>
<td>134</td>
<td>64</td>
<td>52</td>
<td>52</td>
</tr>
<tr>
<td>ULS 3c:</td>
<td>4632</td>
<td>-</td>
<td>256</td>
<td>208</td>
<td>208</td>
</tr>
<tr>
<td>SLS 1a:</td>
<td>3748</td>
<td>-</td>
<td>39</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>SLS 1b:</td>
<td>3245</td>
<td>81</td>
<td>39</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>SLS 1c:</td>
<td>3245</td>
<td>-</td>
<td>155</td>
<td>126</td>
<td>126</td>
</tr>
<tr>
<td>SLS 2a:</td>
<td>3326</td>
<td>-</td>
<td>93</td>
<td>76</td>
<td>76</td>
</tr>
<tr>
<td>SLS 2b:</td>
<td>3258</td>
<td>16.2</td>
<td>93</td>
<td>76</td>
<td>76</td>
</tr>
<tr>
<td>SLS 2c:</td>
<td>3258</td>
<td>-</td>
<td>109</td>
<td>88</td>
<td>88</td>
</tr>
<tr>
<td>SLS 3a:</td>
<td>3258</td>
<td>-</td>
<td>93</td>
<td>76</td>
<td>76</td>
</tr>
</tbody>
</table>
When the same procedure is applied for the NS-direction, the following loads can be obtained, by making use of the earlier mentioned loads and combinations.

### Table 101: Acting forces in the NS-direction

<table>
<thead>
<tr>
<th>Situation</th>
<th>N [kN]</th>
<th>q_w [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS 4a:</td>
<td>5005</td>
<td>-</td>
</tr>
<tr>
<td>ULS 5a:</td>
<td>3713</td>
<td>-</td>
</tr>
<tr>
<td>ULS 6a:</td>
<td>5461</td>
<td>-</td>
</tr>
<tr>
<td>ULS 6b:</td>
<td>4632</td>
<td>28</td>
</tr>
<tr>
<td>SLS 4a:</td>
<td>3748</td>
<td>-</td>
</tr>
<tr>
<td>SLS 4b:</td>
<td>3245</td>
<td>17</td>
</tr>
<tr>
<td>SLS 5a:</td>
<td>3326</td>
<td>3.4</td>
</tr>
<tr>
<td>SLS 5b:</td>
<td>3258</td>
<td>3.4</td>
</tr>
<tr>
<td>SLS 6a:</td>
<td>3258</td>
<td>-</td>
</tr>
</tbody>
</table>

### C.1.4 Sectional properties

The cores are composed of several Kerto-LVL elements. Since some of these panels contain openings, two distinct sections can be distinguished, being a section of the fully enclosed core (see figure 239) and a section of the core including an opening (see figure 239).

At first, the section without openings is taken into consideration.

#### C.1.4.1 Section without openings

From figure 239 and section 0, it can be determined that:

\[ EA_1 = EA_3 = \frac{13800 \, \text{N/mm}^2 \times 5000 \times 180 \, \text{mm}^3 + 2400 \, \text{N/mm}^2 \times 5000 \times 69 \, \text{mm}^3}{2} = 1.325 \times 10^{10} \, \text{Nmm}^2 \]

\[ EA_2 = EA_4 = \frac{13800 \, \text{N/mm}^2 \times 1.350 \times 10^6 \, \text{mm}^2}{2} = 1.863 \times 10^{10} \, \text{Nmm}^2 \]

with \( a_{x1} = a_{x3} = 2624.5 \, \text{mm} \)

and \( a_{y2} = a_{y4} = 2365 \, \text{mm} \)

As a first assumption for the connection between the distinct walls, the elements are considered to be bolted together, having a spacing of 150 mm between the bolts. This leads to the following values for the slip moduli:

\[ K_{ser} = \frac{1}{23} \pi \eta \frac{d_{bolt}}{2} = \frac{1}{23} \pi \frac{1.5}{2} \times 510 \times 0.15 = 12020 \, \text{N/mm} \]

\[ K_u = \frac{2}{3} K_{ser} = 8012 \, \text{N/mm} \]

With help of these slip moduli, the connection factor \( \gamma \) can be determined. This leads to the following connection factors, both for the serviceability as the ultimate limit state:

\[ \gamma_{lyser} = \gamma_{3:lyser} = \sqrt{1 + \pi^2 \frac{E_1 \cdot A_1 \cdot s}{2 \cdot K_{ser} \cdot l_{eff}^2}} = \sqrt{1 + \pi^2 \frac{1.325 \times 10^{10} \times 150}{2 \times 12020 \times 38440^2}} = 0.644 \]

\[ \gamma_{lxser} = \gamma_{3:lxser} = \sqrt{1 + \pi^2 \frac{E_2 \cdot A_2 \cdot s}{2 \cdot K_{ser} \cdot l_{eff}^2}} = \sqrt{1 + \pi^2 \frac{1.863 \times 10^{10} \times 150}{2 \times 12020 \times 38440^2}} = 0.563 \]

\[ \gamma_{2:lyser} = \gamma_{4:lyser} = \gamma_{2:lxser} = \gamma_{4:lxser} = 1.0 \]

and

\[ \gamma_{lysu} = \gamma_{3:lys} = \sqrt{1 + \pi^2 \frac{E_1 \cdot A_1 \cdot s}{2 \cdot K_u \cdot l_{eff}^2}} = \sqrt{1 + \pi^2 \frac{1.325 \times 10^{10} \times 150}{2 \times 8012 \times 38440^2}} = 0.547 \]

\[ \gamma_{lxsu} = \gamma_{3:lhs} = \sqrt{1 + \pi^2 \frac{E_2 \cdot A_2 \cdot s}{2 \cdot K_u \cdot l_{eff}^2}} = \sqrt{1 + \pi^2 \frac{1.863 \times 10^{10} \times 150}{2 \times 8012 \times 38440^2}} = 0.462 \]
Making use of these factors, the effective values of the second moments of area can be found:

\[
(\text{EI})_{ef,ys} = \sum (\text{EI})_{ys} + \gamma_{ys} \cdot (EA)_h \cdot a_{ys}^2 = 2 \cdot 13800 \cdot \frac{1}{12} \cdot 270 \cdot 5000^3 + 2 \cdot (13800 \cdot \frac{1}{12} \cdot 5000 \cdot 180^3 + 2400 \cdot \frac{1}{12} \cdot 5000 \cdot 69^3 + 0.644 \cdot 1.325 \cdot 10^{10} \cdot 2624.5^2)
\]

\[= 19.54 \cdot 10^{16} \text{ Nmm}^2\]

\[
(\text{EI})_{ef,xs} = \sum (\text{EI})_{xs} + \gamma_{xs} \cdot (EA)_h \cdot a_{xs}^2 = 2 \cdot 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 270^3 + 2 \cdot 0.563 \cdot 1.863 \cdot 10^{10} \cdot 2365^2
\]

\[= 17.27 \cdot 10^{16} \text{ Nmm}^2\]

\[
(\text{EI})_{ef,yy} = \sum (\text{EI})_{yy} + \gamma_{yy} \cdot (EA)_h \cdot a_{yy}^2 = 2 \cdot 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 270^3 + 2 \cdot 0.563 \cdot 1.863 \cdot 10^{10} \cdot 2365^2
\]

\[= 17.76 \cdot 10^{16} \text{ Nmm}^2\]

\[
(\text{EI})_{ef,xu} = \sum (\text{EI})_{xu} + \gamma_{xu} \cdot (EA)_h \cdot a_{xu}^2 = 2 \cdot 13800 \cdot \frac{1}{12} \cdot 180 \cdot 5000^3 + 2 \cdot 2400 \cdot \frac{1}{12} \cdot 69 \cdot 5000^3 + 2 \cdot 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 270^3 + 2 \cdot 0.462 \cdot 1.863 \cdot 10^{10} \cdot 2365^2
\]

\[= 15.17 \cdot 10^{16} \text{ Nmm}^2\]

C.1.4.2 Section including openings

When this situation applies, the sectional properties can be derived from figure 239. One should note that in this case \(A_1 \neq A_3\).

\[
EA_1 = 13800 \text{ N/mm}^2 \cdot 5000 \cdot 180 \text{ mm}^2 + 2400 \text{ N/mm}^2 \cdot 5000 \cdot 69 \text{ mm}^2 = 1.325 \cdot 10^{10} \text{ Nmm}^2
\]

\[
EA_2 = EA_4 = 13800 \text{ N/mm}^2 \cdot 1.350 \cdot 10^4 \text{ mm}^2 = 1.863 \cdot 10^{10} \text{ Nmm}^2
\]

\[
EA_3 = 13800 \text{ N/mm}^2 \cdot 3080 \cdot 180 \text{ mm}^2 + 2400 \text{ N/mm}^2 \cdot 3080 \cdot 69 \text{ mm}^2 = 0.816 \cdot 10^{10} \text{ Nmm}^2
\]

with \(a_{1,1} = 2396.9 \text{ mm}\)

\(a_{2,1} = a_{y,4} = 227.6 \text{ mm}\)

\(a_{3,1} = 2852.1\)

and \(a_{1,3} = a_{x,4} = 2365 \text{ mm}\)

\(a_{2,3} = a_{x,3} = 0 \text{ mm}\)

The slip moduli \(K_{ww}\) and \(K_s\) are still 12020 N/mm and 8012 N/mm respectively.

With help of these slip moduli, the connection factor \(\gamma\) can be determined. This leads to the following connection factors, both for the serviceability as the ultimate limit state:

\[
\gamma_{1,y,ser} = \left[1 + \pi^2 \cdot \frac{E_1 \cdot A_1 \cdot s}{2 \cdot K_{ser} \cdot s_{ef}}\right]^{-1} = \left[1 + \pi^2 \cdot \frac{1.325 \cdot 10^{10} \cdot 150}{2 \cdot 12020 \cdot 38440^2}\right]^{-1} = 0.644
\]

\[
\gamma_{3,y,ser} = \left[1 + \pi^2 \cdot \frac{E_3 \cdot A_3 \cdot s}{2 \cdot K_{ser} \cdot s_{ef}}\right]^{-1} = \left[1 + \pi^2 \cdot \frac{0.816 \cdot 10^{10} \cdot 150}{2 \cdot 12020 \cdot 38440^2}\right]^{-1} = 0.746
\]

\[
\gamma_{1,x,ser} = \gamma_{3,x,ser} = \left[1 + \pi^2 \cdot \frac{E_2 \cdot A_2 \cdot s}{2 \cdot K_{ser} \cdot s_{ef}}\right]^{-1} = \left[1 + \pi^2 \cdot \frac{1.863 \cdot 10^{10} \cdot 150}{2 \cdot 12020 \cdot 38440^2}\right]^{-1} = 0.563
\]

\[
\gamma_{2,y,ser} = \gamma_{4,y,ser} = \gamma_{2,x,ser} = \gamma_{4,x,ser} = 1.0
\]

and
Making use of these factors, the effective values of the second moments of area can be found:

\[(\text{EI})_{e,yz} = \sum (\text{EI})_{yj} + \gamma_{yjs} \cdot (\text{EA})_{j} \cdot a_{yz} = 2 \left( \frac{1}{12} \left( 13800 \cdot 0.1 \cdot 700 \cdot 5000^3 + 1.863 \cdot 10^{10} \cdot 270^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 180^3 + 2400 \cdot \frac{1}{12} \cdot 5000 \cdot 69^3 + 0.644 \cdot 1.325 \cdot 10^{10} \cdot 2396.9^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 3080 \cdot 180^3 + 2400 \cdot \frac{1}{12} \cdot 3080 \cdot 69^3 + 0.746 \cdot 0.816 \cdot 10^{10} \cdot 2852.1^2 \right) \right) = 17.83 \cdot 10^{16} \text{ Nmm}^2 \]

\[(\text{EI})_{e,zx} = \sum (\text{EI})_{xj} + \gamma_{xz} \cdot (\text{EA})_{j} \cdot a_{xz} = \left( \frac{1}{12} \left( 13800 \cdot 0.1 \cdot 5000 \cdot 5000^3 + 1.863 \cdot 10^{10} \cdot 270^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 180 \cdot (5000^3 - 1920^3) + 2400 \cdot \frac{1}{12} \cdot 69 \cdot (5000^3 - 1920^3) \right) + \left( 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 270^3 + 0.563 \cdot 1.863 \cdot 10^{10} \cdot 2365^2 \right) \right) = 17.12 \cdot 10^{16} \text{ Nmm}^2 \]

\[(\text{EI})_{e,yx} = \sum (\text{EI})_{yj} + \gamma_{yx} \cdot (\text{EA})_{j} \cdot a_{yx} = 2 \left( \frac{1}{12} \left( 13800 \cdot 0.1 \cdot 700 \cdot 5000^3 + 1.863 \cdot 10^{10} \cdot 270^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 180^3 + 2400 \cdot \frac{1}{12} \cdot 5000 \cdot 69^3 + 0.547 \cdot 1.325 \cdot 10^{10} \cdot 2396.9^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 3080 \cdot 180^3 + 2400 \cdot \frac{1}{12} \cdot 3080 \cdot 69^3 + 0.662 \cdot 0.816 \cdot 10^{10} \cdot 2852.1^2 \right) \right) = 16.43 \cdot 10^{16} \text{ Nmm}^2 \]

\[(\text{EI})_{e,xu} = \sum (\text{EI})_{xj} + \gamma_{xu} \cdot (\text{EA})_{j} \cdot a_{xu} = \left( \frac{1}{12} \left( 13800 \cdot 0.1 \cdot 5000 \cdot 5000^3 + 1.863 \cdot 10^{10} \cdot 270^2 \right) + \left( 13800 \cdot \frac{1}{12} \cdot 180 \cdot (5000^3 - 1920^3) + 2400 \cdot \frac{1}{12} \cdot 69 \cdot (5000^3 - 1920^3) \right) + \left( 13800 \cdot \frac{1}{12} \cdot 5000 \cdot 270^3 + 0.462 \cdot 1.863 \cdot 10^{10} \cdot 2365^2 \right) \right) = 15.01 \cdot 10^{16} \text{ Nmm}^2 \]
\section*{C.1.5 2\textsuperscript{nd} order effect}

The second-order effect may be neglected provided that:

\[ L_c \leq \frac{E_{lf}}{N_{vd}} \]

Conservatively, the smallest value for $E_l$ will be considered, being $15.01 \times 10^{16}$ Nmm\(^2\).

The additional loading (has been determined in the weight calculation, and it can be found that $N_{vd,\text{max}} = 5461$ kN. Using this value in the formula above, this leads to:

\[ L_c \leq \frac{15.01 \times 10^{16}}{5461 \times 10^{4}} = 165788 \text{m} \]

The effective length $L_c$ is determined as follows:

\[ L_c = l^* \sqrt{1.12^2 + \frac{\pi^2}{2\rho}} \]

in which:

\[ \rho = \frac{C}{E_{lf}} \]

The rotational stiffness of the foundation, $C$, is obtained from design calculations of BAM A\&E on the Euroborg stadium, and seemed to be $19975147$ kNm/rad.

Hereby it is found that $\rho = 2.56$ and thereby:

\[ L_c = 19.22 \times \sqrt{1.12^2 + \frac{\pi^2}{2\times 2.56}} = 34.29 \text{m} \]

Since $L_c < 165788 \text{ m}$, it can be concluded that a second-order calculation can be neglected within the design.

\textbf{Alternative calculation}

The second-order effect may be neglected when

\[ \frac{n}{n-1} \leq 1 \quad \text{or} \quad n \geq 11 \]

in which

\[ n = \frac{N_{cr}}{N_{vd}} \]

\[ \frac{1}{N_{cr}} = \frac{1}{N_{c1}} + \frac{1}{N_{c2}} = \frac{(1.12)^2}{\pi^2 C} + \frac{1}{2 \times 15.01 \times 10^7} + \frac{19.22}{2 \times 19975147} = \frac{1}{3196866} + \frac{1}{2078579} \]

By making use of these values, it is found that $n = 231$ and therefore the 2\textsuperscript{nd} order effect may be neglected.
C.1.6 Global strength and stiffness calculation

By making use of the earlier obtained sectional properties, the strength and the stiffness of the structure is checked. At first, the overall stiffness is checked, by making use of the maximum acceptable deflections.

C.1.6.1 Global stiffness calculation

With reference to B.2.7, it can be found that these values should be limited to:

\[ u_{\text{max, total}} = \frac{1}{500} \times l \text{ for structures as a whole} \]
\[ u_{\text{max, storey}} = \frac{1}{300} \times l \text{ for each storey} \]

It should be mentioned that these requirements can be considered as very conservative for a stadium structure, since these kind of buildings only contain few storeys. Despite that, for now, use will be made of these requirements.

Since wind should be regarded as being a short-term action, the effect of shrinkage and creep can be neglected. With a total height of the structure of 19.22 m, it can be found that they maximum acceptable deflection at the top may therefore be 40 mm.

When the rotational stiffness of the foundation is considered, the rotation of the foundation in y-direction can be determined as:

\[ \varphi = \frac{M_{\text{rep}}}{C} = \frac{0.5 \times 81 \times 19.22^2}{19975147} = 0.00075 \text{ rad} \]

With a height of 19.22 m, this leads to a deflection of 14.4 mm caused by the stiffness of the foundation. The remaining deflection is therefore restricted to 25.6 mm. In the same way it can be found that for the x-direction, the remaining deflection is 37 mm.

This leads to a required moment of inertia of:

\[ I_{\text{required, y}} = \frac{q l^4}{8 u_{\text{max, total}}} = \frac{81 \times 19220^4}{8 \times 25.6} = 5.398 \times 10^{16} \text{ Nmm}^2 \]
\[ I_{\text{required, x}} = \frac{q l^4}{8 u_{\text{max, total}}} = \frac{17 \times 19220^4}{8 \times 37} = 0.784 \times 10^{16} \text{ Nmm}^2 \]

When the reduced (i.e. including openings) values of \( E I \) are considered, it is found that:

\[ (E I)_{\text{eff, y}} = 17.83 \times 10^{16} \text{ Nmm}^2 \]
\[ (E I)_{\text{eff, x}} = 17.12 \times 10^{16} \text{ Nmm}^2 \]

Therefore, it can be concluded that the overall stiffness of the stabilizing structure is not at stake.

C.1.6.2 Global strength calculation

Now, the strength of the proposed central core will be taken into consideration. It is of importance that checks will be performed on the governing loads from both the EW-direction and the NS-direction.

At first, the EW-direction will be considered.

**EW-direction**

By making use of the earlier mentioned load combinations, the governing loads can be determined. On one hand the combination of the maximum bending moment and the corresponding normal force should be accounted for. On the other hand the maximum normal force should be considered, in combination with the corresponding bending moment. From C.1.3 it can be found that the governing loads are:
It is concluded that the bending moment for ULS 3a becomes far smaller than for ULS 3b, i.e. the bending moment for ULS 3b follows almost directly from the acting wind force. Therefore, it is concluded that, when the structure suffices for ULS 3b, it is probably satisfying for ULS 3a as well. This is checked later on.

Since ULS 3b is governing for the design, this load combination is used to determine the acting stresses.

Making use of the above found values, and the build-up of the core, see figure 239, the acting forces on the structure are determined. For the heaviest loaded part of the core, near the foundation, this leads to:

\[ N_d = 4633 \text{kN} \]
\[ M_d = 26280 \text{kNm} \]
\[ V_d = 2743 \text{kN} \]
\[ N_{d;\text{max}} = 5461 \text{kN} \]

This value will be used in the design calculations.

Now, it has to be checked whether the structure is able to withstand these forces. By making use of the sectional properties as found earlier, stated in table 102, and the formulae as shown underneath, the acting stresses are determined (see figure 240). An overview is provided in table 103.

### Table 102: Properties for the individual walls

<table>
<thead>
<tr>
<th>( i )</th>
<th>( h_i ) [mm]</th>
<th>( \gamma_i ) [-]</th>
<th>( E_{\text{eff},i} ) [N/mm²]</th>
<th>( a_i ) [mm]</th>
<th>( E_i ) [Nmm²]</th>
<th>( A_i ) [N]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>249</td>
<td>0.547</td>
<td>10641</td>
<td>2396.93</td>
<td>16.53*10^{10}</td>
<td>1.325*10^{10}</td>
</tr>
<tr>
<td>2</td>
<td>5000</td>
<td>1.0</td>
<td>13800</td>
<td>227.57</td>
<td>16.53*10^{14}</td>
<td>1.863*10^{10}</td>
</tr>
<tr>
<td>3</td>
<td>249</td>
<td>0.662</td>
<td>10641</td>
<td>2852.07</td>
<td>16.53*10^{14}</td>
<td>0.816*10^{10}</td>
</tr>
<tr>
<td>4</td>
<td>5000</td>
<td>1.0</td>
<td>13800</td>
<td>227.57</td>
<td>16.53*10^{14}</td>
<td>1.863*10^{10}</td>
</tr>
</tbody>
</table>

\[
\sigma_{\text{m};i} = \frac{0.5 E_i h_i M_{i;\text{m}}}{E_{\text{eff},i}}; \sigma_i = \frac{E_i a_i M_{i;\text{m}}}{E_{\text{eff},i}}; \sigma_N = \frac{N_i}{E A_{\text{tot}}} \\
\tau_{1/3;\text{m;max}} = \frac{\gamma_1 E_i A_{i;\text{m}} a_i}{b_i^3 E_{\text{eff}}}; \tau_{2/4;\text{m;max}} = \frac{\gamma_2 E_i A_{i;\text{m}} a_i + 0.5 E_2 b_2 (0.5 h_2 + a_2)^2}{b_2^3 E_{\text{eff}}}
\]
### Table 103: Normal stresses due to bending moment and normal forces

<table>
<thead>
<tr>
<th>i</th>
<th>( \sigma_{xy} ) [N/mm²]</th>
<th>( \sigma_{yz} ) [N/mm²]</th>
<th>( \sigma_{xy} ) [N/mm²]</th>
<th>( \tau_{x1\min} ) [N/mm²]</th>
<th>( \tau_{x1\max} ) [N/mm²]</th>
<th>( \tau_{y3\min} ) [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+/- 0.21</td>
<td>+/- 2.22</td>
<td>0.84</td>
<td>0</td>
<td>0.58</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>+/- 5.49</td>
<td>+/- 0.50</td>
<td>1.09</td>
<td>0.53</td>
<td>1.23</td>
<td>0.47</td>
</tr>
<tr>
<td>3</td>
<td>+/- 0.21</td>
<td>+/- 3.20</td>
<td>0.84</td>
<td>0</td>
<td>0.51</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>+/- 5.49</td>
<td>+/- 0.50</td>
<td>1.09</td>
<td>0.53</td>
<td>1.23</td>
<td>0.47</td>
</tr>
</tbody>
</table>

For the sake of clarity, figure 241 shows how these values act over the cross-section.
It appears that the normal stresses following from the bending moment exceed the stresses from the normal (compression) force. As a result, tensile stresses act at the core’s cross-section.

For the stresses found above, it has to be checked if the core is able to withstand them.

### Walls 2 and 4

\[ \sigma_{2,\text{max}} = 5.99 \text{ N/mm}^2 < f_{\text{m,ed}} = 44 \times 0.9 / 1.2 = 33.00 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.18 < 1 \]
\[ \sigma_{2,\text{max}} = 1.09 \text{ N/mm}^2 < f_{\text{c,cd}} = 35 \times 0.9 / 1.2 = 26.25 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.05 < 1 \]
\[ \tau_{2,\text{max}} = 1.23 \text{ N/mm}^2 < f_{\text{v,ed}} = 4.1 \times 0.9 / 1.2 = 3.07 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.40 < 1 \]

In addition to the above, also the combination of bending moment and normal force should be checked:

\[ \frac{\sigma_{\text{cd}}}{f_{\text{c,cd}}} + \frac{\sigma_{\text{m,d}}}{f_{\text{m,d}}} \leq 1 \]

To perform this check, at first the value for \( k_c \) is determined:

\[ \lambda_{\text{ef}} = l_{\text{ef}} * \sqrt{\frac{E_{\text{tot}}}{E_{\text{eff}}}} = 38440 * \sqrt[16]{\frac{1.325 \times 10^{10}}{16.43 \times 10^6}} = 10.92 \]
\[ \lambda_{\text{ref}} = \frac{\lambda_{\text{ef}}}{\pi} * \sqrt{\frac{f_{\text{c,0,k}}}{E_{0.05}}} = 10.92 * \sqrt{\frac{35}{11600}} = 0.19 \]

Since \( \lambda_{\text{ref}} < 0.3 \), \( k_c \) can be taken 1.0. This results in:

\[ \frac{1.09}{1.0 \times 26.25} + \frac{5.99}{33} = 0.23 < 1 \]

### Walls 1 and 3

\[ \sigma_{1,\text{max}} = 3.41 \text{ N/mm}^2 < f_{\text{m,ed}} = 44 \times 2 \times 90 \times 13800 / (2 \times 90 \times 13800 + 1 \times 69 \times 2400) \times 0.9 / 1.2 = 30.94 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.11 < 1 \]
\[ \sigma_{1,\text{max}} = 0.84 \text{ N/mm}^2 < f_{\text{c,cd}} = 35 \times 2 \times 90 \times 13800 + 9 \times 1 \times 69 \times 2400 / (2 \times 90 \times 13800 + 1 \times 69 \times 2400) \times 0.9 / 1.2 = 25.03 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.04 < 1 \]
\[ \tau_{1,\text{max}} = 0.53 \text{ N/mm}^2 < f_{\text{v,ed}} = 4.1 \times 2 \times 90 \times 13800 + 4.5 \times 1 \times 69 \times 2400 / (2 \times 90 \times 13800 + 1 \times 69 \times 2400) \times 0.9 / 1.2 = 3.1 \text{ N/mm}^2 \quad \ddagger \quad \text{U.C.} = 0.18 < 1 \]

Also here, the combination of bending moment and normal force is checked:

\[ \frac{\sigma_{\text{cd}}}{k \times f_{\text{c,cd}}} + \frac{\sigma_{\text{m,d}}}{f_{\text{m,d}}} \leq 1 \]

To perform this check, at first the value for \( k_c \) is determined:

\[ \lambda_{\text{ef}} = l_{\text{ef}} * \sqrt{\frac{E_{\text{tot}}}{E_{\text{eff}}}} = 38440 * \sqrt[16]{\frac{1.325 \times 10^{10}}{16.43 \times 10^6}} = 10.92 \]
\[ \lambda_{\text{ref}} = \frac{\lambda_{\text{ef}}}{\pi} * \sqrt{\frac{f_{\text{c,0,k}}}{E_{0.05}}} = 10.92 * \sqrt{\frac{27.80}{8940}} = 0.20 \]

Since \( \lambda_{\text{ref}} < 0.3 \), \( k_c \) can be taken 1.0. This results in:

\[ \frac{0.84}{1.0 \times 30.94} + \frac{3.41}{25.03} = 0.17 < 1 \]
In addition to the above, it is checked whether tensile forces occur in the core. Therefore, it has to be checked whether $\sigma_{N_{\min}} < \sigma_{M_{\text{max}}}$ when the combination of the simultaneous acting loads $M_{\text{max}}$ and $N_{\min}$ (ULS 3b) is considered, it is found that:

$$\sigma_{N_{\min}} = 0.84 \text{N/mm}^2 < \sigma_{M_{\text{max}}} = 3.4 \text{N/mm}^2$$

and thus tensile stresses should be accounted for:

$$\sigma_{N_{\text{tensile}}} = 2.57 \text{N/mm}^2 < f_t = 2.1/9.0*2400*69*113800*90*2/35 = 24.89 \text{N/mm}^2$$

**Fasteners**

Finally, it has to be checked if the chosen fasteners are able to withstand the shear forces, which is defined as:

$$f_{vcd} = \tau_{2_{\min}} + b_2 \times s = 0.53 * 270 * 150 = 21.47 \text{kN}$$

Without performing any checks at this very moment, it is expected that the fasteners are able to withstand these forces.

**NS-direction**

When the same procedure as used for the EW-direction is applied, the following loads are obtained:

ULS 6b:

$$N_d = 4633 \text{kN}$$

$$M_d = 5006 \text{kNm}$$

$$V_d = 538 \text{kN}$$

By making use of the sectional properties as found earlier, stated in table 104, and the formulae as shown underneath, the acting stresses can be determined. An overview can be found in table 105.

Table 104: Properties for the individual walls

<table>
<thead>
<tr>
<th>i</th>
<th>$h_1$ [mm]</th>
<th>$\gamma_z$</th>
<th>$E_{\text{eff}}$ [N/mm$^2$]</th>
<th>$a_{z1}$ [mm]</th>
<th>$E_I$ [Nmm$^2$]</th>
<th>$E_A$ [N]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5000</td>
<td>0.462</td>
<td>10641</td>
<td>0</td>
<td>$15.01*10^{16}$</td>
<td>$1.325*10^{10}$</td>
</tr>
<tr>
<td>2</td>
<td>270</td>
<td>1.0</td>
<td>13800</td>
<td>2365</td>
<td>$15.01*10^{14}$</td>
<td>$1.863*10^{10}$</td>
</tr>
<tr>
<td>3</td>
<td>1540</td>
<td>0.462</td>
<td>10641</td>
<td>0</td>
<td>$15.01*10^{14}$</td>
<td>$0.816*10^{10}$</td>
</tr>
<tr>
<td>4</td>
<td>270</td>
<td>1.0</td>
<td>13800</td>
<td>2365</td>
<td>$15.01*10^{14}$</td>
<td>$1.863*10^{10}$</td>
</tr>
</tbody>
</table>

Table 105: Normal stresses due to bending moment and normal forces

<table>
<thead>
<tr>
<th>i</th>
<th>$\sigma_{m_{z1}}$ [N/mm$^2$]</th>
<th>$\sigma_{\gamma_2}$ [N/mm$^2$]</th>
<th>$\sigma_{Nz}$ [N/mm$^2$]</th>
<th>$\tau_{yz_{\min}}$ [N/mm$^2$]</th>
<th>$\tau_{yz_{\max}}$ [N/mm$^2$]</th>
<th>$\tau_{yz_{\min}}$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>+/- 0.89</td>
<td>+/ 1.09</td>
<td>0.84</td>
<td>0.15</td>
<td>0.27</td>
<td>0.15</td>
</tr>
<tr>
<td>2</td>
<td>+/- 0.06</td>
<td>+/- 1.09</td>
<td>1.09</td>
<td>0</td>
<td>0.14</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>+/- 0.89</td>
<td>+/ 1.09</td>
<td>0.84</td>
<td>0.15</td>
<td>0.14</td>
<td>0.15</td>
</tr>
<tr>
<td>4</td>
<td>+/- 0.06</td>
<td>+/- 1.09</td>
<td>1.09</td>
<td>0</td>
<td>0.14</td>
<td>0</td>
</tr>
</tbody>
</table>

As shown in table 105, the found stresses in this direction are far smaller than those found for the perpendicular direction, see table 103. It is therefore concluded without showing, that the obtained values for the stresses in z-direction (NS) suffice to all requirements.

Despite that, it should be mentioned that, also here, tensile stresses should be accounted for!
Annex C.2 Small shear walls

To provide the reader with some understanding on the structural build-up of the small wall system, a section is provided in figure 242. For all relevant information on the lay-out, reference is made to Annex C.2.

Figure 242: Proposed small stabilizing wall

C.2.1 Loads

In this section, the various loads that might act on the structure will be shown. These loads follow directly from the weight calculation as shown in Annex G. The loads follow from the structural plan of the Euroborg stadium and should therefore be considered preliminary.

For this solution, a total of 6 walls is incorporated in the design. Therefore, the loads are equally divided over these walls.

Making use of the weight calculation, the total amount of additional loading is determined. This load consists of a permanent part (dead load) and a variable part (live loads). Again, there is not accounted for the eccentric point of action of this load.

At first, it is determined which part of this load, is actually transferred to the walls. Therefore, the area of influence of each wall is determined. This has been performed for the 3rd, the 2nd, the 1st, the ground and the -1st floor, resulting in the following overview:

<table>
<thead>
<tr>
<th>Situation: Floor</th>
<th>Stabilized area by one core [m²]</th>
<th>Area directly influenced [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd</td>
<td>394</td>
<td>99</td>
</tr>
<tr>
<td>2nd</td>
<td>462</td>
<td>99</td>
</tr>
<tr>
<td>1st</td>
<td>394</td>
<td>99</td>
</tr>
</tbody>
</table>
It is assumed that the load following from the roof, are directly carried by the roof trusses to the columns. Therefore, the walls do not play a role in the transfer of these loads.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Gdead;wall;rep</th>
<th>= (4.8 * 0.297) * 19.22 * 4.9</th>
<th>= 134 kN</th>
<th>(self-weight wall)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G3rd</td>
<td>(0.78<em>2.91 + 0.22</em>1.91) * 99</td>
<td>= 266 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G2nd</td>
<td>(1.0*3.14) * 99</td>
<td>= 311 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G1st</td>
<td>(1.0*3.14) * 99</td>
<td>= 311 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gground</td>
<td>(1.0*3.14) * 124</td>
<td>= 389 kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G-1st</td>
<td>(0.64<em>1.62 + 0.36</em>3.14) * 124</td>
<td>= 268 kN</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the total load:

\[ G_{total; \text{rep}} = 134 + 266 + 311 + 311 + 389 + 268 = 1679 \text{ kN} \]

For the variable part of these loads is found:

\[ Q_{3rd \text{ floor;rep}} = 4.0 * 99 = 396 \text{ kN} \]
\[ (\Psi_0 = 1.0; \Psi_1 = 0.9; \Psi_2 = 0.8) \]
\[ Q_{2nd \text{ floor;rep}} = 2.5 * 99 = 248 \text{ kN} \]
\[ (\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3) \]
\[ Q_{1st \text{ floor;rep}} = 2.5 * 99 = 248 \text{ kN} \]
\[ (\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3) \]
\[ Q_{ground \text{ floor;rep}} = 5.0 * 124 = 620 \text{ kN} \]
\[ (\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6) \]
\[ Q_{-1st \text{ floor;rep}} = 2.0 * 124 = 248 \text{ kN} \]
\[ (\Psi_0 = 0.7; \Psi_1 = 0.7; \Psi_2 = 0.6) \]

The wind loads have been determined in B.2.4:

\[ Q_{\text{wind direction;rep}} = \frac{1}{6} * 1.14 * 1.3 * 164 \rightarrow 40.5 \text{ kN/m} \]
\[ (\Psi_0 = 0.0; \Psi_1 = 0.2; \Psi_2 = 0.0) \]

For the grandstand structure, a horizontal load should be introduced which equals 10% of the vertical load. This load is assumed to be divided over the -2nd, -1st and the ground floor for the lower grandstand, and over the column in front of the 1st floor and the 2nd floor for the upper grandstand. The load per floor, divided for the lower (1) and upper (2) grandstand, then becomes:

\[ Q_{\text{grandstand 1; hor;rep}} = \frac{1}{6} * \frac{1}{2} * 0.10 * 11309 \rightarrow +/ - 63 \text{ kN} \]
\[ (\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6) \]
\[ Q_{\text{grandstand 2; hor;rep}} = \frac{1}{6} * \frac{1}{2} * 0.10 * 9266 \rightarrow +/ - 78 \text{ kN} \]
\[ (\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6) \]

Since the worst combination is obtained when the direction of this load coincides with the wind direction, this values will be taken positive.

**C.2.2 Load combinations**

Since the stability of the structure is considered, use is made of the following load combinations:

ULS 1: \[ 1.49 * G_{1,\text{sup}} + 1.65 * \Psi_0 * Q_{k1} \]

ULS 2: \[ 0.90 * G_{1,\text{sup}} + 1.65 * \Psi_0 * Q_{k2} \]

ULS 3: \[ 1.32 * G_{1,\text{sup}} + 1.65 * \Psi_0 * Q_{k3} \]

SLS 1: \[ 1.0 * G_{1,\text{sup}} + 1.0 * \Psi_{1,1} * Q_{k1} + 1.0 * \Psi_{2,1} * Q_{k1} \]

SLS 2: \[ 1.0 * G_{1,\text{sup}} + 1.0 * \Psi_{1,1} * Q_{k2} + 1.0 * \Psi_{2,1} * Q_{k2} \]

SLS 3: \[ 1.0 * G_{1,\text{sup}} + 1.0 * \Psi_{1,1} * Q_{k3} + 1.0 * \Psi_{2,1} * Q_{k3} \]

These combinations lead to the following values for the acting forces:
Table 106: Acting forces in the EW-direction, following from the governing load combinations

<table>
<thead>
<tr>
<th>Situation:</th>
<th>N [kN]</th>
<th>qw [kN/m]</th>
<th>F_{hor;2nd} [kN]</th>
<th>F_{hor;0} [kN]</th>
<th>F_{hor;-1st} [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS 1a:</td>
<td>4106</td>
<td>-</td>
<td>33</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>ULS 2a:</td>
<td>3115</td>
<td>-</td>
<td>33</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>ULS 3a:</td>
<td>4587</td>
<td>-</td>
<td>33</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>ULS 3b:</td>
<td>3820</td>
<td>67</td>
<td>33</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>ULS 3c:</td>
<td>3820</td>
<td>-</td>
<td>128</td>
<td>104</td>
<td>104</td>
</tr>
<tr>
<td>SLS 1a:</td>
<td>3116</td>
<td>-</td>
<td>19.5</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>SLS 1b:</td>
<td>2651</td>
<td>40.5</td>
<td>19.5</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>SLS 1c:</td>
<td>2651</td>
<td>-</td>
<td>77.5</td>
<td>63</td>
<td>63</td>
</tr>
<tr>
<td>SLS 2a:</td>
<td>2727</td>
<td>-</td>
<td>46.5</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>SLS 2b:</td>
<td>2665</td>
<td>8.1</td>
<td>46.5</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>SLS 2c:</td>
<td>2665</td>
<td>-</td>
<td>54.5</td>
<td>44</td>
<td>44</td>
</tr>
<tr>
<td>SLS 3a:</td>
<td>2665</td>
<td>-</td>
<td>46.5</td>
<td>38</td>
<td>38</td>
</tr>
</tbody>
</table>

C.2.3 Sectional properties

As mentioned a few sections back, the walls are taken CLT elements. The chosen LenoTec 297 panels are composed of 11 distinct laminates, of which 3 are oriented in transverse direction. As a result, not all laminates take an equal share of the load.

For the proposed walls, this leads to an effective bending stiffness of:

\[ E_{\text{eff}} = \sum E_i \cdot L_i = 8 \times (11000 \times \frac{1}{12} \times 27 \times 4800^3) + 3 \times (370 \times \frac{1}{12} \times 27 \times 4800^3) = 2.22 \times 10^{16} \text{Nmm}^2 \]

and a second moment of area I of:

\[ I_{\text{eff}} = \frac{2.22 \times 10^{16}}{1 \times (11000 \times 8 + 370 \times 3)} = 2.72 \times 10^{12} \text{mm}^4 \]

In addition to the above, also the axial stiffness is determined:

\[ E_{\text{A,eff}} = \sum E_i \cdot A_i = 8 \times (11000 \times 27 \times 4800) + 3 \times (370 \times 27 \times 4800) = 1.15 \times 10^{10} \text{N} \]

and a total area of:

\[ A_{\text{eff}} = \frac{E_{\text{A,eff}}}{E_{\text{rel}}} = \frac{1.15 \times 10^{10}}{1 \times (11000 \times 8 + 370 \times 3)} = 1.42 \times 10^6 \text{mm}^4 \]

C.2.4 2nd order effect

The second-order effect may be neglected provided that:

\[ L_c \leq \frac{E_{\text{eff}}}{N_{vd}} \]

With reference to the calculation of the loads earlier this section, it is found that \( N_{vd,max} = 4587 \text{kN} \). Hereby it is found that \( L_c \) should be smaller than 69568 m.

The effective length \( L_c \) is determined as follows:

\[ L_c = l^* \sqrt{1.12^2 + \frac{\pi^2}{20}} \]

in which:

\[ \rho = \frac{C \cdot l^*}{E_{\text{eff}}} \]

The rotational stiffness of the foundation, \( C \), is obtained from design calculations of BAM A&E on the Euroborg stadium, and seemed to be 19975147 kNm/rad.
Hereby, it is found that $\rho = 17.29$ and thereby:

$$ L_c = 19.22 \times \sqrt{1.12^2 + \frac{\pi^2}{2 \times 17.29}} = 23.85 \text{m} $$

Since $L_c < 69568 \text{m}$, it is concluded that a second-order calculation can be neglected within the design.

**C.2.5 Global strength and stiffness calculation**

By making use of the earlier obtained sectional properties, the strength and the stiffness of the structure is checked. At first, the overall stiffness is checked, by making use of the maximum acceptable deflections.

**C.2.5.1 Global stiffness calculation**

With reference to B.2.7, it is found that these values should be limited to:

$u_{\text{max, total}} = 1/500 \times l$ for structures as a whole

$u_{\text{max, storey}} = 1/300 \times l$ for each storey

It should be mentioned that these requirements can be considered as very conservative for a stadium structure, since these kind of buildings only contain few storeys. Despite that, these requirements are adopted in the design.

Since wind should be regarded as being a short-term action, the effect of shrinkage and creep is neglected. With a total height of the structure of 19.22 m, it is found that they maximum acceptable deflection at the top may therefore be 40 mm.

When the rotational stiffness of the foundation is considered, the rotation of the foundation in $y$-direction is determined as:

$$ \varphi = \frac{M_{\text{rot}}}{C} = \frac{0.5 \times 40.5 \times 19.22^2}{1975147} = 0.00037 \text{rad} $$

With a height of 19.22 m, this leads to a deflection of 7.2 mm caused by the stiffness of the foundation. The remaining deflection is therefore restricted to 32.8 mm.

This leads to a required second moment of area $I$ of:

$$ El_{\text{required, } y} = \frac{q l^4}{8 u_{\text{max, total}}} = \frac{40.5 \times 19220^4}{8 \times 32.8} = 2.106 \times 10^{16} \text{Nmm}^2 $$

When the value of $El_{\text{eff}}$ is considered, it is found that:

$$ (El)_{\text{eff, } x} = 2.22 \times 10^{14} \text{Nmm}^2 > El_{\text{required, } y} = 2.106 \times 10^{16} \text{Nmm}^2 $$

Therefore, it is concluded that the overall stiffness of the small wall system is satisfying.

**C.2.5.2 Global strength calculation**

Now, the strength of the proposed walls will be taken into consideration. It is mentioned that checks will be performed on the governing loads from the EW-direction, since the walls only act in this direction.

**EW-direction**

By making use of the earlier mentioned load combinations, the governing loads can be determined. On one hand the combination of the maximum bending moment and the corresponding normal force should be accounted for. On the other hand the maximum normal force should be considered, in combination with the corresponding bending moment. From C.2.1 it is determined that the governing loads are:

**ULS 3a**:

$N_d = 4587 \text{ kN}$

$F_{\text{hor, 2nd floor}} = 33 \text{ kN}$

$F_{\text{hor, ground floor}} = 26 \text{ kN}$

$F_{\text{hor, -1st floor}} = 26 \text{ kN}$
ULS 3b:

\[ N_d = 3820 \text{ kN} \]
\[ q_{w;d} = 67 \text{ kN/m} \]
\[ F_{\text{hor;2nd floor}} = 33 \text{ kN} \]
\[ F_{\text{hor;ground floor}} = 26 \text{ kN} \]
\[ F_{\text{hor;-1st floor}} = 26 \text{ kN} \]

The bending moment for ULS 3a becomes far smaller than for ULS 3b, i.e. the bending moment for ULS 3a follows almost directly from the acting wind force. Therefore, it is concluded that, when the structure suffices for ULS 3b, it probably suffices for ULS 3b as well.

Since ULS 3b is governing for the design, this load combination is used to determine the acting stresses.

Making use of the above found values, and the build-up of the wall, see section 0, the acting forces on the structure are determined. For the heaviest loaded part of the core, near the foundation, this leads to:

\[ N_d = 3820 \text{ kN} \]
\[ M_d = 13140 \text{ kNm} \]
\[ V_d = 1372 \text{ kN} \]
\[ N_{d,\text{max}} = 4587 \text{ kN} \]

This value will be used in the design calculations

With the properties of the shear wall as stated earlier, the stresses are determined from:

\[ \sigma_M = \frac{M \cdot z}{I_{\text{ef}}} ; \sigma_N = \frac{N}{A_{\text{ef}}} ; \tau_{\text{max}} = \frac{3 \cdot V}{2 \cdot A_{\text{ef}}} \]

This results in the following values:

\[ \sigma_M = \frac{13140 \cdot 10^6 \cdot 4800}{2 \cdot 7.2 \cdot 10^{12}} = 11.6 \text{ N/mm}^2 \]
\[ \sigma_N = \frac{4587 \cdot 10^3}{1.42 \cdot 10^6} = 3.2 \text{ N/mm}^2 \]
\[ \tau_{\text{max}} = \frac{3 \cdot 1372 \cdot 10^3}{2 \cdot 1.42 \cdot 10^6} = 1.45 \text{ N/mm}^2 \]

Since \( \sigma_{N,\text{ULS3B}} = 2.69 \text{ N/mm}^2 < \sigma_{M,\text{ULS3B}} = 11.60 \text{ N/mm}^2 \), tensile stresses have to be accounted for.

It is now checked, whether the chosen LenoTec panels are able to withstand the stresses that occur within the cross-section.

\[ \sigma_{M,\text{max}} = 11.6 \text{ N/mm}^2 < f_{\text{m,d},\text{d}} = \frac{24 \cdot 8 \cdot 11000 \cdot 11000}{8 \cdot 11000 + 3 \cdot 370} \cdot 0.9 / 1.25 = 17.06 \text{ N/mm}^2 \]
\[ \tau_{\text{max}} = 1.45 \text{ N/mm}^2 < f_{\text{v,d},\text{d}} = \frac{2.5 \cdot 8 \cdot 11000 + 2.5 \cdot 3 \cdot 370}{8 \cdot 11000 + 3 \cdot 370} \cdot 0.9 / 1.25 = 1.80 \text{ N/mm}^2 \]
\[ \sigma_{N,\text{max}} = 3.2 \text{ N/mm}^2 < f_{\text{c,d},\text{d}} = \frac{8 \cdot 11000 + 3 \cdot 370}{8 \cdot 11000 + 3 \cdot 370} \cdot 0.9 / 1.25 = 14.95 \text{ N/mm}^2 \]

When the combination of bending moment and normal force is considered, the following is found:
\[
\frac{\sigma_{C:d}}{k_c \cdot f_c:d} + \frac{\sigma_{m:d}}{f_{m:d}} \leq 1
\]

To perform this check, at first the value for \( k \) is determined:

\[
\lambda_{ef} = \left[ \frac{A}{A_{ef}} \right] = 38440 \cdot \frac{\sqrt{1.42 \times 10^6}}{\sqrt{2.72 \times 10^5}} = 27.79
\]

\[
\lambda_{rel} = \frac{\lambda_{ef}}{\pi} \sqrt{\frac{f_{c,0:k}}{E_{0,5}}} = \frac{27.79}{\pi} \cdot \frac{21}{\sqrt{7400}} = 0.471
\]

\[
k = 0.5(1 + \beta_c(\lambda_{rel} - 0.3 + \lambda_{rel}^2)) = 0.5(1 + 0.1(0.471 - 0.3 + 0.471^2)) = 0.62
\]

\[
k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} = \frac{1}{0.62 + \sqrt{0.62^2 - 0.471^2}} = 0.98
\]

This results in:

\[
\frac{2.69}{0.98 \times 14.95} = \frac{11.6}{17.06} = 0.86 < 1
\]

As can be seen, the chosen solution proves to be fulfilling to all requirements on strength.

When the tensile stresses are considered, it appears that a tensile stress of 8.91 N/mm\(^2\) (11.6 N/mm\(^2\) - 2.69 N/mm\(^2\)) might act in the walls. It has to be checked whether these walls are able to withstand such a stress:

\[
\sigma_{n,\text{tensile}} = 8.91 \text{ N/mm}^2 \quad > \quad f_{t,\text{cr}} = 14 \cdot \frac{8 \times 11000 + 0.4 \cdot 3 \times 370}{8 \times 11000 + 3 \times 370} \cdot 0.9 / 1.25 = 9.96 \text{ N/mm}^2
\]

Since the walls are capable of handling such a stress, no additional measures are required.
Annex C.3 Trusses

To provide the reader with some understanding on the structural build-up of the trusses, a section is provided in figure 243. For all relevant information on the exact build-up, reference is made to section 4.5.2.

![Figure 243: Build up of the proposed truss](image)

C.3.1 Loads

In this section, the various loads that act on the structure are shown. These loads follow directly from the weight calculation as shown in Annex G. These loads have been determined while accounting for the structural build-up of the Euroborg stadium and should therefore be considered preliminary only.

8 trusses are incorporated in the design, while it is assumed that the loads are equally divided over these trusses.

Intermezzo

In this situation, the trusses in the façade are being used as shear walls. Due to the eccentric wind load (which acts in the centre of the structure), a bending moment is introduced, which has to be taken by the shear walls in EW-direction. For now, it is assumed that 6 shear walls are available in EW-direction, which are equally divided over the width of the structure.

The load on each shear wall is determined from (see figure 244):

\[ N_i = \frac{\sum x_i}{x_i^2} \cdot M \]

in which \( x_i \) is the distance to the point of rotation and \( M = \frac{1}{2} \cdot 1.14 \cdot 1.3 \cdot 164 \cdot 34 \)

This leads to a maximum load, for the most distant walls, of 6.53 kN/m. When this value is compared to the wind load in NS-direction that should be taken up by these walls, it should be clear that these walls have no problem to take up these loads.
Making use of the weight calculation, the total amount of additional loading is determined. This load consists of a permanent part (dead load) and a variable part (live loads).

At first, it is determined which part of this load is actually transferred to the trusses. Therefore, the area of influence of a single wall truss is determined. This has been performed for the 3rd, the 2nd, the 1st, the ground and the -1st floor, resulting in the following overview:

<table>
<thead>
<tr>
<th>Floor</th>
<th>Stabilized area by 1 wall [m²]</th>
<th>Area directly influenced [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd</td>
<td>252</td>
<td>76</td>
</tr>
<tr>
<td>2nd</td>
<td>296</td>
<td>76</td>
</tr>
<tr>
<td>1st</td>
<td>252</td>
<td>76</td>
</tr>
<tr>
<td>ground</td>
<td>361</td>
<td>76</td>
</tr>
<tr>
<td>-1st</td>
<td>411</td>
<td>76</td>
</tr>
</tbody>
</table>

As a remark to the above, it is mentioned that the combination with small shear walls is considered. As a result, the area of influence for each portal is 76 m².

It is assumed that the load following from the roof are directly carried by the roof trusses to the columns. Therefore, the trusses do not play a role in the transfer of these loads.

\[ G_{\text{dead, frame rep}} = V_{_0} \times 4.1 = 90 \text{ kN} \] (assumed self-weight truss)

\[ G_{\text{3rd floor}} = (0.78 \times 2.91 + 0.22 \times 1.91) \times 76 = 204 \text{ kN} \]

\[ G_{\text{2nd floor}} = (1.0 \times 3.14) \times 76 = 238 \text{ kN} \]

\[ G_{\text{1st floor}} = (1.0 \times 3.14) \times 76 = 238 \text{ kN} \]

\[ G_{\text{ground floor}} = (1.0 \times 3.14) \times 76 = 238 \text{ kN} \]

\[ G_{\text{total}} = 90 + 204 + 238 + 238 + 238 + 165 = 1174 \text{ kN} \]

For the variable part of these loads is found:

\[ Q_{\text{3rd floor}} = 4.0 \times 76 = 304 \text{ kN} \] (\(\Psi_0 = 1.0; \Psi_1 = 0.9; \Psi_2 = 0.8\))

\[ Q_{\text{2nd floor}} = 2.5 \times 76 = 190 \text{ kN} \] (\(\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3\))

\[ Q_{\text{1st floor}} = 2.5 \times 76 = 190 \text{ kN} \] (\(\Psi_0 = 0.5; \Psi_1 = 0.5; \Psi_2 = 0.3\))

\[ Q_{\text{ground floor}} = 5.0 \times 76 = 380 \text{ kN} \] (\(\Psi_0 = 0.25; \Psi_1 = 0.7; \Psi_2 = 0.6\))

\[ Q_{\text{-1st floor}} = 2.0 \times 76 = 152 \text{ kN} \] (\(\Psi_0 = 0.7; \Psi_1 = 0.7; \Psi_2 = 0.6\))

The wind loads have been determined in B.2.4:

\[ Q_{\text{w;EW-direction rep}} = \frac{1}{8} \times 1.14 \times 1.3 \times 34 = 6.3 \text{ kN/m} \] (\(\Psi_0 = 0.0; \Psi_1 = 0.2; \Psi_2 = 0.0\))

**C.3.2 Load combinations**

Since the stability of the structure is considered, use has to be made of the following load combinations:

**ULS 1:**

\[ 1.49 \times G_{\text{sup}} + 1.65 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \]

**ULS 2:**

\[ 0.90 \times G_{\text{sup}} + 1.65 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \]

**ULS 3:**

\[ 1.32 \times G_{\text{sup}} + 1.65 \times Q_{\text{ki}} + 1.65 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \]

**SLS 1:**

\[ 1.0 \times G_{\text{sup}} + 1.0 \times Q_{\text{ki}} + 1.0 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \] (permanent)

**SLS 2:**

\[ 1.0 \times G_{\text{sup}} + 1.0 \times \Psi_{_{2;1}} \times Q_{\text{ki}} + 1.0 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \] (frequent)

**SLS 3:**

\[ 1.0 \times G_{\text{sup}} + 1.0 \times \Psi_{_{2;1}} \times Q_{\text{ki}} + 1.0 \times \Psi_{_{2;1}} \times Q_{\text{ki}} \] (quasi-permanent)
These combinations lead to the following values for the acting forces:

<table>
<thead>
<tr>
<th>Situation</th>
<th>( N [\text{kN}] )</th>
<th>( q_w [\text{kN/m}] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS 1a</td>
<td>2897</td>
<td>-</td>
</tr>
<tr>
<td>ULS 2a</td>
<td>2204</td>
<td>-</td>
</tr>
<tr>
<td>ULS 3a</td>
<td>3168</td>
<td>-</td>
</tr>
<tr>
<td>ULS 3b</td>
<td>2697 10.4</td>
<td></td>
</tr>
<tr>
<td>SLS 1a</td>
<td>2155</td>
<td>-</td>
</tr>
<tr>
<td>SLS 1b</td>
<td>1870 6.3</td>
<td></td>
</tr>
<tr>
<td>SLS 2a</td>
<td>1889</td>
<td>-</td>
</tr>
<tr>
<td>SLS 2b</td>
<td>1851 1.3</td>
<td></td>
</tr>
<tr>
<td>SLS 3a</td>
<td>1854</td>
<td>-</td>
</tr>
</tbody>
</table>

### C.3.3 Sectional properties

As mentioned a few sections back, the frames are composed of columns, bracings and a beam. For these elements, the sectional properties will be determined now.

<table>
<thead>
<tr>
<th></th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A = b \cdot h = 350 \cdot 350 = 0.12 \cdot 10^6 \text{mm}^2 )</td>
<td></td>
</tr>
<tr>
<td>( I = \frac{1}{12} b \cdot h^3 = \frac{1}{12} \cdot 350 \cdot 350^3 = 1.25 \cdot 10^9 \text{mm}^4 )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A = b \cdot h = 200 \cdot 1150 = 0.23 \cdot 10^6 \text{mm}^2 )</td>
<td></td>
</tr>
<tr>
<td>( I = \frac{1}{12} b \cdot h^3 = \frac{1}{12} \cdot 200 \cdot 1150^3 = 25.34 \cdot 10^9 \text{mm}^4 )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Bracings</th>
</tr>
</thead>
<tbody>
<tr>
<td>( A = b \cdot h = 150 \cdot 150 = 0.02 \cdot 10^6 \text{mm}^2 )</td>
<td></td>
</tr>
</tbody>
</table>

### C.3.4 Global strength and stiffness calculation

By making use of the earlier obtained sectional properties, the stiffness and the strength of the structure is checked.

#### C.3.4.1 Global stiffness calculation

In this section, the stiffness of the proposed stabilizing system is determined. With reference to B.2.7, it can be found that these values should be limited to:

- \( u_{\text{max,total}} = \frac{1}{500} \cdot I \) for structures as a whole
- \( u_{\text{max,storey}} = \frac{1}{300} \cdot I \) for each storey

These requirements can be considered as very conservative for a stadium structure, since these kind of buildings only contain few storeys. Despite that, for now, use is made of these requirements.

Since wind should be regarded as being a short-term action, the effect of shrinkage and creep can be neglected. With a total height of the structure of 19.22 m, it is found that they maximum acceptable deflection at the top may therefore be 40 mm.

When the rotational stiffness of the foundation is considered, the rotation of the foundation in \( y \)-direction can be determined as:

\[
\varphi = \frac{M_{\text{rep}}}{C} = \frac{(0.5 \times 10.4 \times 19.22^2)}{19975147} = 0.000095 \text{rad}
\]
With a height of 19.22 m, this leads to a deflection of 1.9 mm caused by the stiffness of the foundation. The remaining deflection is therefore restricted to 38.1 mm.

It is now checked whether the proposed stabilizing system is able to satisfy this requirement. Therefore, both the governing load situation as well as the proposed sectional properties is implemented in a computer model, resulting in the following outcome, see figure 245.

![Figure 245: Deflections of the proposed portal frame](image)

It is concluded that the maximum horizontal deflection is only 27.5 mm, which is smaller than the maximum allowable value of 38.1 mm. Therefore, it is concluded that the global stiffness of the proposed solution does not lead to any problems.

When the deflection of the individual storeys is considered, it can be found that the maximum horizontal deflection is about 11 mm, at the 1st floor. Since this floor has a height of 4520 mm, it appears that the maximum acceptable deflection is about 15 mm. Therefore, there are no problems to be expected concerning stiffness.

C.3.4.2 Global strength calculation

Now, the strength of the proposed walls are taken into consideration. It is mentioned that checks are performed on the governing loads from the NS-direction, since the walls only act in this direction.

**NS-direction**

By making use of the earlier mentioned load combinations, the governing loads can be determined. On one hand the combination of the maximum bending moment and the corresponding normal force should be accounted for. On the other hand the maximum normal force should be considered, in combination with the corresponding bending moment. From C.3.2, it appears that the governing loads are:

\[
\begin{align*}
\text{ULS 3a:} & \\
N_d &= 3168 \text{ kN} \\
\text{ULS 3b:} & \\
N_d &= 2697 \text{ kN} \\
q_w \text{d} &= 10.4 \text{ kN/m}
\end{align*}
\]
Making use of the above found values, and the build-up of the frames, see section 0, the acting forces on
the structure are determined. This leads to the following results for the governing frame:

![Figure 246: Forces in the frame due to the maximum normal force](image)

![Figure 247: Forces in the frame due to the combination of wind and normal force](image)

With the properties of the frames as stated earlier, the stresses are determined from:

\[ \sigma_M = \frac{M \cdot z}{I}; \sigma_N = \frac{N}{A} \]

The acting stresses are now checked for the various elements. Shear is neglected here.

**Column**

\[ \sigma_c = \frac{1584 \cdot 10^3}{0.12 \cdot 10^6} = 13.20 \, \text{N/mm}^2 \]

It is now checked whether the structure is able to resist these forces:

\[ \frac{\sigma_{cd}}{k_c \cdot t_{cd}} \leq 1 \]

To perform this check, at first the value for \( k_c \) is determined:

\[ \lambda_{ef} = l_{ef} \cdot \sqrt{\frac{A_{tot}}{l_{ef}}} = 7300 \cdot \sqrt{\frac{0.12 \cdot 10^6}{1.25 \cdot 10^9}} = 71.52 \]
\[ \lambda_{\text{rel}} = \frac{\lambda_{\text{ef}} \cdot f_{c0k}}{\pi \cdot E_{0.05}} = \frac{71.52 \cdot 26.5}{10200} = 1.16 \]
\[ k_y = 0.5(1 + \beta_c (\lambda_{\text{rel}} - 0.5) + \lambda_{\text{rel}}^2) = 0.5(1 + 0.65(1.16 - 0.5) + 1.16^2) = 1.206 \]
\[ k_c = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel}}^2}} = \frac{1}{1.206 + \sqrt{1.206^2 - 1.16^2}} = 0.65 \]

This results in:
\[ \sigma_t = 13.20 \text{ N/mm}^2 \quad < k_c f_{c0d} = 0.65 \cdot 26.5 \cdot 0.9 / 1.25 = 12.40 \text{ N/mm}^2 \]
‡ U.C. = 1.06 > 1

\[ \beta \quad \text{Beam} \]
\[ \sigma_M = \frac{693 \cdot 10^6 \cdot 1150}{25.34 \cdot 10^9} = 15.73 \text{ N/mm}^2 \]
\[ \sigma_n = 15.73 \text{ N/mm}^2 \quad < f_{c0d} = 28 \cdot 0.9 / 1.25 = 20.16 \text{ N/mm}^2 \]
‡ U.C. = 0.78 < 1

\[ \beta \quad \text{Bracing} \]
\[ \sigma_{t,\text{max}} = \frac{196 \cdot 10^3}{0.02 \cdot 10^6} = 9.80 \text{ N/mm}^2 \]
\[ \sigma_{t,\text{max}} = 9.80 \text{ N/mm}^2 \quad < f_{c0d} = 26.5 \cdot 0.9 / 1.25 = 19.08 \text{ N/mm}^2 \]
‡ U.C. = 0.52 < 1
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Annex D Design of floors

Annex D.1 Sectional properties

To be able to determine the performance of the distinct floor products under certain loading conditions, at first the sectional properties of all these elements are determined. In this section, the formulae that are used to determine these properties are shown.

D.1.1 Timber hollow core slabs: Lignatur

A Lignatur element is a timber hollow core element. Within the element, space is available to implement, for example, a fire retardant layer.

The sectional properties are determined with reference to figure 248, in which all relevant parameters can be found. One should note that both an acoustic layer (t_i) and a fire retardant layer (h_i) is incorporated here. In case these layers are not present, their heights should be taken 0.

\[
I = \frac{m^* d^* h^3}{12} + \frac{m^* d^* h^* (h - s_y)^2}{2} + (n^* d_i - b_u) * \frac{t_i^3}{12} + (n^* d_i - b_u) * t_i * (s_y - \frac{b_u}{2}) + \frac{n^* d_i * t_i^3}{12} + (n^* d_i * t_i) * (h_s - s_y - \frac{t_i}{2})^2
\]

in which the centre of gravity s_y can be defined as:

\[
s_y = \frac{m^* d_i * h^2}{2} + (n^* d_i - b_u) * \frac{t_i^2}{2} + n^* d_i * t_i * (t_i + h_i + \frac{t_i}{2}) + n^* d_i * t_i * (h - \frac{t_i}{2})
\]

A_total = b * h - (n^* d_i) * (h_i + h_i) - b_u * t_i

Making use of these formulae, the following values are found to determine the strength of the elements:

\[
V_{d,\text{max}} = A_w * f_{v:d}
\]

\[
M_{d,\text{max}} = \min \left[ \frac{1}{h - s_y} * f_{m:d}, \frac{1}{s_y} * f_{m:d} \right]
\]

in which:

\[
A_w = \frac{m^* d^* I}{S_y}
\]
for \( s_y \leq t_1 \):
\[
S_y = \frac{m \cdot d \cdot s_y^2}{2} + (n \cdot d \cdot b_u) \cdot \frac{s_y^2}{2}
\]

for \( t_1 < s_y \leq t_1 + h_i \)
\[
S_y = \frac{m \cdot d \cdot s_y^2}{2} + (n \cdot d \cdot b_u) \cdot t_1 \cdot \left( s_y - \frac{t_1}{2} \right)
\]

for \( t_1 + h_i < s_y \leq t_1 + h_i + t_{ii} \)
\[
S_y = \frac{m \cdot d \cdot s_y^2}{2} + (n \cdot d \cdot b_u) \cdot t_1 \cdot \left( s_y - \frac{t_1}{2} \right) + n \cdot d \cdot t_{ii} \cdot \left( s_y - t_{ii} \right) \cdot \frac{(s_y - t_{ii})^2}{2}
\]

for \( t_1 + h_i + t_{ii} < s_y \leq h - t_{ii} \):
\[
S_y = \frac{m \cdot d \cdot (h - s_y)^2}{2} + n \cdot d \cdot t_{ii} \cdot \left( h - s_y - \frac{t_{ii}}{2} \right)
\]

for \( h - t_{ii} \leq s_y \):
\[
S_y = \frac{m \cdot d \cdot (h - s_y)^2}{2} + n \cdot d \cdot \left( h - s_y \right) \cdot \frac{(h - s_y)^2}{2}
\]

D.1.2 Cross-Laminated Timber: BSP Crossplan

A cross-laminated timber panel consists of various layers which are glued together. Some of these layers are oriented in the main load-bearing direction, while others only act in transverse direction. Here, it is assumed that only the layers oriented in the main load-bearing direction are able to transfer the loads.

The sectional properties are determined with reference to figure 249, in which the distinct layers and their parameters are shown. It is mentioned that the same procedure may be applied when 3 or 5 layers are used, though one should note which layers take part in the transfer of the loads.

The design procedure as proposed here follows from [55] and [56].

\[
l_{\text{eff}} = \sum_{i=1}^{n} \left( h_i \cdot l_i + n_i \cdot A_i \cdot a_i^2 \right)
\]

The flexibility factor \( \gamma \) is determined as follows:
\[ \gamma_1 = \left( 1 + \frac{\pi^2 EA_i h_i}{l_{eff}^2 G R_i b} \right)^{-1} \text{ while } \gamma_2 = 1. \]

The value of \( l_{eff} \) depends on the support conditions of the element. When the element is supported over two supports, the length is taken equal to the span. When the element is a continuous beam, see figure 250, the span is taken \( 4/5 \) of the total span. For cantilevering beams, the length \( l \) is taken double the span of the cantilever \([57]\).

The available cross-section is determined while accounting for the whole height of the element.

\[ A_{total} = b \times h_{total} \]

Making use of these formulae, the following values are found for the maximum allowable forces:

\[ V_{d,max} = \sum A_i \times f_{Rd} \]
\[ M_{d,max} = \frac{f_{M,d} \times l_{eff}}{\gamma_i \times a_i + h_i / 2} \]

**D.1.3 Stressed skin panels: Kerto-Ripa**

A Kerto-Ripa element is a stressed skin panel, consisting of webs made from Kerto-S and a collective flange made of Kerto-Q. The webs are connected to the flange by means of glue, where it is assumed that there is no slip at the connection, i.e. \( \gamma = 1.0 \).

The sectional properties are determined with reference to figure 251, in which all relevant parameters are found. The formulae are deducted by making use of common sense. The results are checked with the product specification as provided by the manufacturer \([59]\) and correspond fairly well.
\[ E_{\text{eff}} = E_{\text{flange}} + E_{\text{web}} \]
\[ E_{\text{flange}} = E_{\text{flange}} \frac{b*t_{iii}^3}{12} \frac{b*1000}{b} \]
\[ E_{\text{web}} = E_{\text{web}} \frac{m*d*(h-t_{iii})^3}{12} + m*d*(h-t_{iii})* \frac{(h-t_{iii}+t_{iii}+s_{y})}{2} \]

in which the centre of gravity \( s_{y} \) is defined as:

\[ s_{y} = \frac{E_{\text{flange}} * b * t_{iii}^2/2 + E_{\text{web}} * (m*d) * (h-t_{iii}) * \frac{(h-t_{iii})}{2} + t_{iii} + s_{y}}{E_{\text{flange}} * b * t_{iii} + E_{\text{web}} * (m*d) * (h-t_{iii})} \]

Making use of these formulae, the following values are found for the maximum allowable forces:

\[ V_{d,max} = \sum A_{i} * f_{vd} \]
\[ M_{d,max} = \min \left[ \frac{f_{m,flange:d} * l_{eff}}{s_{y} - t_{iii}/2}, \frac{f_{m,web:d} * l_{eff}}{(h-t_{iii})/2 - s_{y}} \right] \]

It is mentioned that for the determination of the maximum deflection, shear deformations should be taken into consideration. This has been neglected at this stage of the design, since it would only introduce additional difficulties.

**D.1.4 Massive glulam floor element: Bresta Decke**

A Decke floor consist of multiple timber laminates which are glued together at their edges. As a result, a horizontally glued laminated element is found. Also here, it is assumed that there is not slip at the intermediate connections. A distinction can be made between the ‘Brett Lammellen’ and the ‘Brettstosse’ type. For the latter, joints are included in between the distinct laminates, resulting in a reduced load-bearing capacity. In this thesis, only the Brett Lammellen type are considered.

The sectional properties are determined with reference to figure 252, in which all relevant parameters are found. The formulae are deducted by making use of common sense.
Annex D.2 Design of floor elements

In this section, the design calculations concerning the floor elements are discussed. Consecutively, the stretcher bond pattern, the stacked bond pattern (NS) and the stacked bond pattern (EW) are taken into consideration.

The floor products that are considered were already discussed in the chapter Floors. Their sectional properties, as discussed in Annex D.1, are used to determine the maximum span for the various floor products. This resulted in the load-to-span diagrams as shown in chapter Floors.

The load combinations are determined from the Eurocodes and the NEN, of which an extensive overview is provided in Annex B.

D.2.1 Stretcher bond pattern

When the stretcher bond pattern is concerned, the floor elements span over three supports, see figure 74.

As a consequence, a bending moment arises above the middle support. The presence of this moment influences the structural behaviour to a certain extent. In this section, the strength distribution, the deflections and the dynamic behaviour are determined.

D.2.1.1 Strength distribution

Analysis

According to section 6.3.3 and 6.3.4 of the NEN, for a beam over three supports the following load configurations are to be considered, assuming that redistribution between the support moment and the field moment may occur:

1. One field loaded with the extreme value of the variable floor load, one field loaded with the combination (in Dutch: momentaan) value of the variable load

2. One field loaded with the combination (in Dutch: momentaan) value of the variable load and one field unloaded

Loads

Configuration 1:

\[ q_{d1} = 1.32 \cdot q_{\text{perm,1}} + 1.65 \cdot q_{\text{var,1}} \] (extreme value)

\[ q_{d2} = 1.32 \cdot q_{\text{perm,2}} + 1.65 \cdot \psi_0 \cdot q_{\text{var,2}} \] (combination value)

Configuration 2:

\[ q_{d1} = 1.32 \cdot q_{\text{perm,1}} + 1.65 \cdot \psi_0 \cdot q_{\text{var,1}} \] (combination value)

\[ q_{d2} = 0 \]
In this section, the focus is on both the office floor and the ground floor:

2nd floor Offices: $q_{var} = 2.5 \text{ kN/m}^2$, $\Psi_1 = 0.5$, $\Psi_1 = 0.5$

Ground floor Congregation: $q_{var} = 5.0 \text{ kN/m}^2$, $\Psi_0 = 0.25$, $\Psi_1 = 0.7$

The permanent loading follows from the self-weight of the elements.

**Distribution of loads**

$$M_B = -\frac{1}{8} \left[ \frac{q_1 l_1^2}{EI_1} + \frac{q_2 l_2^2}{EI_2} \right]$$

$$M_{\text{span},1} = \frac{MB}{2} + \frac{1}{8} q_1 l_1^2$$

$$M_{\text{span},2} = \frac{MB}{2} + \frac{1}{8} q_2 l_2^2$$

**Deflections**

These loads are incorporated in the following formulae, where $M_B$ is determined by making use of the formula as presented in D.2.1.2:

$$w_1 = \frac{5}{384} q_1 l_1^4 + \frac{1}{16} \frac{M_B}{EI_1} l_1^2 < w_{max1} = \frac{l_1}{500}$$

$$w_2 = \frac{5}{384} q_2 l_2^4 + \frac{1}{16} \frac{M_B}{EI_2} l_2^2 < w_{max2} = \frac{l_2}{500}$$

The maximum acceptable additional deflection is limited to the values as shown above, see Annex B, where has been accounted for stiff divider walls.

To obtain the load-to-span diagrams as incorporated in the chapter Floors, the sectional properties and strength properties of the floor products are used to determine the maximum span that satisfies the requirements on strength.

**D.2.1.2 Deflections**

**Loads**

Permanent loading does not have to be considered when additional deflections are concerned. Therefore, the following loads are found:

$q_{k,1} = 1.0 * \Psi_1 * q_{var,1}$

$q_{k,2} = 0$

**Deflections**

These loads are incorporated in the following formulae, where $M_B$ is determined by making use of the formula as presented in D.2.1.1:

To obtain the load-to-span diagrams as incorporated in the chapter Floors, the sectional properties of the distinct floor products are used to determine the maximum span that satisfies the requirements on deflections.

**D.2.1.3 Behaviour on vibrations**

**Analysis**

The governing load combination is determined while accounting for the permanent load and the combination value of the variable load. Since the first mode of vibration for a beam over three supports is identical to that of a beam over two supports, the following situation is considered, see figure 255.
For this situation, a vibration acceleration $\alpha$ of 0.315 m/s² has to be accounted for.

### Loads

$q_{k.1} = 1.0 \times q_{\text{perm},1} + 1.0 \times \psi_1 \times q_{\text{var},1}$

$q_{k.2} = 1.0 \times q_{\text{perm},2} + 1.0 \times \psi_1 \times q_{\text{var},2}$

### Fundamental frequency

$$f_{e1} = \sqrt{\frac{a}{w_1}} = \sqrt{\frac{0.315}{w_1}} > 5 \text{ Hz}$$

$$f_{e2} = \sqrt{\frac{a}{w_2}} = \sqrt{\frac{0.315}{w_2}} > 5 \text{ Hz}$$

In which the deflections are defined as:

$$w_1 = \frac{5}{384} \frac{q_{d1}^4}{E_l}$$

$$w_2 = \frac{5}{384} \frac{q_{d2}^4}{E_l}$$

To obtain the load-to-span diagrams as incorporated in the chapter Floors, the sectional properties of the distinct floor products are used to determine the maximum span that satisfies the requirements on vibrations.

### D.2.2 Stacked bond pattern (NS-direction)

When the stacked bond pattern is concerned, the floor elements span over two supports, see figure 88.

Since an extensive explanation has already been given for the beam over three supports, in this section, only the formulae and loads to be used are shown.

#### D.2.2.1 Strength distribution

**Loads**

$q_d = 1.32 \times q_{\text{perm}} + 1.65 \times q_{\text{var}}$ (extreme value)

The focus is again on both the governing office floor and the ground floor:

- **2nd floor** Offices: $q = 2.5 \text{ kN/m}^2$  \hspace{1cm} $\psi_1 = 0.5$  \hspace{1cm} $\psi_1 = 0.5$
- **Ground floor** Congregation: $q = 5.0 \text{ kN/m}^2$  \hspace{1cm} $\psi_1 = 0.25$  \hspace{1cm} $\psi_1 = 0.7$

The permanent loading follows from the self-weight of the elements.
### Load distribution

\[ M_{\text{span}} = \frac{1}{8}ql^2 \]

\[ A_v = B_v = \frac{1}{2}ql \]

#### D.2.2.2 Deflections

**Loads**

Permanent load does not have to be considered when the additional deflections are determined. This being known, the following load is found:

\[ q_k = 1.0 \times v_1 \times q_{\text{var}} \]

**Deflections**

\[ w = \frac{5}{384} \frac{ql^4}{EI} \quad < w_{\text{max}} = \frac{1}{500} \]

The maximum acceptable additional deflection is limited to the value above, where is accounted for divider walls.

#### D.2.2.3 Behaviour on vibrations

Since the first mode of vibration for a beam over two supports is identical to that of a beam over three supports, the same situation applies as discussed in D.2.1.3, see figure 255.

### D.2.3 Stacked bond pattern (EW-direction)

When the stacked bond pattern (EW-direction) is concerned, the floor elements span over three supports, see figure 257.

**Distribution of loads**

\[
\begin{align*}
M_B &= -\frac{1}{8} \left( \frac{q_1 l_1^3}{E_1} + \frac{q_2 l_2^3}{E_2} \right) \\
M_{\text{span,1}} &= \frac{1}{2} \left( \frac{q_1 l_1^2}{E_1} \right) \\
M_{\text{span,2}} &= \frac{1}{2} \left( \frac{q_2 l_2^2}{E_2} \right) \\
A_v &= \frac{M_{\text{span,1}} + \frac{1}{8}q_1 l_1^2}{0.5l_1} \\
B_v &= q_1 l_1 + q_2 l_2 - A_v - C_v \\
C_v &= \frac{M_{\text{span,2}} + \frac{1}{8}q_2 l_2^2}{0.5l_2}
\end{align*}
\]

Figure 258: Element over three support with unequal spans
Accounting for the spans as shown in figure 258, the following load distribution is found:

\[ A_v = 1.28q \quad [\text{kN/m}] \]
\[ B_v = 9.53q \quad [\text{kN/m}] \]
\[ C_v = 3.65q \quad [\text{kN/m}] \]

The value for \( q \) is determined from the governing load combination, which is determined in a similar way as described in D.2.1.
Annex D.3 Floor support beams

In this section, the beams that support the floor elements are designed. As it appeared from Annex D.2 that the stacked bond pattern (NS-direction) does not provide a beneficial solution, only the stretcher bond pattern and the stacked bond pattern (EW-direction) are considered.

Accounting for the load distribution as discussed in Annex D.2, the loads on the support beams are determined. Subsequently, the required beam dimensions (height) are determined. In case these dimensions exceed the maximum acceptable dimensions (due to free height restrictions), there has to be accounted for additional columns and the calculations are to be repeated while accounting for smaller spans.

Material properties
As a general assumption, the support beams are taken glulam members, GL28h, having a characteristic bending strength of \( f_{mk} = 28 \text{N/mm}^2 \).

Since floor loads are concerned, the modification factor \( k_{mod} \) is taken 0.8. Together with a material factor \( \gamma_M = 1.25 \) this leads to:

\[
f_{md} = f_{mk} \cdot k_{mod} = 28 \cdot \frac{0.8}{1.25} = 17.92 \text{N/mm}^2
\]

The mean modulus of elasticity for this strength class is taken \( E = 12.600 \text{N/mm}^2 \).

D.3.1 Stretcher bond pattern

D.3.1.1 Analysis

As elaborated on in the chapter Floors, the decision is made to consider a Lignatur 320 element in the design calculations, which span two times 8.0 m (elements over 3 supports, see figure 259).

Considering the load distribution, as mentioned in D.2.1, it is found that for equal span (= 8 m) and loading conditions the loads are distributed as follows:

\[
\begin{align*}
A_v &= 0.375q_I = 3q \quad \text{[kN/m]} \\
B_v &= 1.25q_I = 10q \quad \text{[kN/m]} \\
C_v &= 0.375q_I = 3q \quad \text{[kN/m]}
\end{align*}
\]

Due to the stretcher bond pattern, see figure 74, the support beams support the Lignatur elements in turn at both their ends (\( A_v \) and \( C_v \)), and at their midpoint (\( B_v \)). The actual load is therefore the mean of these values:

\[q_{beam} = \frac{(A_v + C_v) + B_v}{2} \]

This beam load is applied directly on the support beam, which spans between two supports (columns). As a first assumption, this beam spans a total of 14.4 m directly, see figure 74. In case this implies beams with dimensions that interfere with the requirements on free height (see 2.3.1), intermediate support columns are accounted for. As a result, the span decreases from 14.4 m to smaller values.

D.3.1.2 Strength

Loads
The extreme value for the loading is considered for strength calculations:

\[q_{d1} = 1.32 \cdot q_{perm1} + 1.65 \cdot q_{var1} \]
Accounting for this combination, the support reactions \((A_v, B_v, \text{and } C_v)\) are determined and thereby \(q_{beam}\).

For a beam on two supports, the bending moment is defined as:

\[
M_{beam} = \frac{1}{8} q_{beam} \cdot b_{beam}^2
\]

**Required sectional properties**

Accounting for the bending strength of the beam, the required sectional modulus is determined:

\[
W_{req} = \frac{M_{beam}}{f_{md}}
\]

As a generally used value, the width is taken 220 mm. This leads to a required height of at least:

\[
h_{min} = \frac{\sqrt{W_{req} \cdot 6}}{220}
\]

**D.3.1.3 Deflections**

**Loads**

\(q_{k,1} = 1.0 \cdot \psi_1 \cdot q_{var,1}\)

Making use of this combination, the support reactions \((A_v, B_v, \text{and } C_v)\) are determined and thereby \(q_{beam}\).

**Required sectional properties**

The maximum acceptable additional deflection is limited to the following values, where has been accounted for stiff partition walls:

\[
w_{max} = \frac{h_{beam}}{500}
\]

Incorporating this value and the mean beam load in the following formulae, the required second moment of inertia is found:

\[
l_{req} = \frac{5}{384} q_{beam} \cdot b_{beam}^4
\]

Assuming a beam thickness of 220 mm again, the required height is at least:

\[
h_{min} = \frac{l_{req} \cdot 12}{220}
\]

**D.3.1.4 Behaviour on vibrations**

**Loads**

\(q_k = 1.0 \cdot q_{perm} + 1.0 \cdot \psi_1 \cdot q_{var}\)

Making use of this combination, the support reactions \((A_v, B_v, \text{and } C_v)\) are determined and thereby \(q_{beam}\).

**Required sectional properties**

As mentioned earlier, the natural frequency should be at least 5 Hz, to prevent problems concerning the dynamic behaviour from happening. Incorporating this value in the following formula, the maximum deflection under the earlier mentioned loading condition can be found:

\[
w_{beam,max} = \frac{a}{f_e^2} = \frac{0.315}{5^2} = 12.6\text{mm}
\]

Incorporating this value and the mean beam load in the following formulae, the required second moment of inertia can be found:

\[
l_{req} = \frac{5}{384} q_{beam} \cdot b_{beam}^4
\]

Assuming a beam thickness of 220 mm again, the required height is determined from:
D.3.1.5 Conclusion

From the design calculations, it appeared that vibrations were governing in the design of the floor support beams.

Due to free height restrictions, it appeared that for the office floors, an additional column is required at a distance of 9 m from the outer façade. For the ground floor, two additional columns are to be accounted for: one at a distance of 5 m and one at a distance of 9 m from the outer façade.

An overview of this situation is provided in figure 82.

**D.3.2 Stacked bond pattern (EW-direction)**

D.3.2.1 Analysis

As elaborated on in the chapter Floors, the decision was made to consider a Lignatur 320 element in the design calculations for the upper floors, which span a total of 14.4 m. For the lower floors, it was decided to consider a Lignatur 200 element.

Following the procedure of D.2.3, it is found that for this pattern the following applies:

At the lower floors the chosen floor elements span over 4 supports with unequal spans. When the upper floors are considered, the floor elements span over 3 supports with unequal spans.

Accounting for the different span-widths, this results in the following load distribution.

**Ground floor**

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>Av</td>
<td>5.400</td>
<td>4.000</td>
<td>5.000</td>
</tr>
<tr>
<td>Bv</td>
<td>2.24q</td>
<td>5.27q</td>
<td>4.80q</td>
</tr>
</tbody>
</table>

Figure 260: Floor element over four support beams (ground floor)

**Office floors**

<p>| | | |</p>
<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>Av</td>
<td>5.400</td>
<td>9.000</td>
</tr>
<tr>
<td>Bv</td>
<td>1.28q</td>
<td>9.53q</td>
</tr>
</tbody>
</table>

Figure 261: Floor element over three support beams (office floors)

Since the floor elements are all arranged in a similar way, the governing load follows directly from the support reactions as shown above. The maximum beam height can therefore be determined from the governing support reaction, which seems to be B, here.

The value for q is determined conform to the governing load combination as mentioned in Annex D.2.
D.3.2.2 Strength, deflections and behaviour on vibrations

Accounting for the load distribution as mentioned in the preceding section, the structural behaviour of the support beams is considered.

Since the support beams are again considered to span between two supports (columns), the structural behaviour is equal to that described in D.3.1.
Annex D.4 Columns

In this section, calculations are performed on the most heavily loaded column within the stadium structure. Goal is to provide an idea on the dimensions of these columns, rather than to perform a complete design calculation.

Material properties
It is assumed here that the columns are all glulam members of strength class GL28h. This leads to a characteristic compressive strength:

\[ f_{c,k} = 26.5 \text{N/mm}^2 \]

Since floor loads are concerned, the modification factor \( k_{mod} \) is 0.8. Together with a material factor \( \gamma_m = 1.25 \) this leads to:

\[ f_{c,d} = f_{c,0,k} \cdot k_{mod} \cdot \gamma_m = 26.5 \cdot 0.8 \cdot 1.25 = 16.96 \text{N/mm}^2 \]

D.4.1 Stretcher bond pattern

D.4.1.1 Analysis
The columns that carry the heaviest loading, are those that support the -1\textsuperscript{st} floor. The maximum load follows from the load distribution as discussed in D.2.1 and D.3.1.

Accounting for equal span of 8 m again, the beam load is determined as:

\[ q_{beam} = \frac{A_y + C_y + B_y}{2} + q_{deadload,beam} = \frac{(0.375q + 0.375q) + 1.25q}{2} + q_{deadload,beam} = 8q + q_{deadload,beam} \]

The value for \( q \) is determined from the governing load combination (see D.2.1 and D.3.1), in which use is made of the permanent load (self-weight of the floor element) and the acting variable loads.

Accounting for the found dimensions for the support beams and their weight, \( q_{deadload,beam} \) is determined. Implementing this value and the earlier found value for \( q \) in the formula above, \( q_{beam} \) is determined for each of the floors within the stadium structure.

D.4.1.2 Strength

Loads
Making use of the values for \( q_{beam} \) at each floor, and the spans of the support beams at the distinct floors, the maximum vertical force in the columns can be determined:

\[ N_{total} = \sum_{i=1}^{n} q_{beam}(l_i + l_{i+1})/2 \]

Required sectional properties
Accounting for the strength of the beam, the required cross-sectional area then becomes:

\[ A_{req} = \frac{N_{total}}{f_{c,d}} \quad [\text{mm}^2] \]

D.4.1.3 Behaviour on buckling
It is assumed that the columns are hinged at both of their ends. The buckling length is thereby equal to their length and no bending moment acts on the column. Since the columns can buckle in both directions, the cross-section is taken square.
Making use of the following formulae, the reduction factor for buckling, \( k_c \), is determined:

\[
\lambda_y = \frac{l_{\text{column}}}{\sqrt{\frac{l}{h} \times \frac{l}{12}}}
\]

\[
\sigma_{\text{crit},y} = \pi^2 \frac{E_{0.05}}{\lambda_y^2}
\]

\[
\lambda_{\text{rel}} = \sqrt{\frac{f_{\text{c,0,k}}}{\sigma_{\text{crit},y}}}
\]

\[
k_y = 0.5(1 + 0.1(\lambda_{\text{rel}} - 0.5) + \lambda_{\text{rel}}^2)
\]

\[
k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{\text{rel}}^2}}
\]

Making use of the found value, it is determined whether the columns are satisfying the requirements on buckling:

\[
\frac{\sigma_{c,0,d}}{k_c \times f_{c,0,d}} \leq 1
\]

**D.4.2 Stacked bond pattern (EW-direction)**

**D.4.2.1 Analysis**

Considering the load distribution, as discussed in D.2.3 and D.3.2, it is found that for the lower and upper floors:

\[
q_{\text{lower}} = B_{v,\text{lower}} + q_{\text{deadload,beam}} = 5.27q + q_{\text{deadload,beam}}\]

\[
q_{\text{upper}} = B_{v,\text{upper}} + q_{\text{deadload,beam}} = 9.53q + q_{\text{deadload,beam}}
\]

**D.4.2.2 Strength and behaviour on buckling**

Accounting for the load distribution as mentioned above, the same procedure as shown in D.4.1 is applied.
Annex D.5 Global diaphragm-action

In this section, the procedure and calculations concerning the global diaphragm-action are shown. This section is valid for all configurations considered.

D.5.1 Theory

- DIN 1052:2004-08 – 8.7 Vereinfachte Berechnung von scheibenartig beanspruchten Tafeln
  - The diaphragm has to be confined by chords at all edges.
  - These chords are designed accounting for buckling and lateral torsional buckling.
  - These checks may be neglected when planking is applied on both sides of the chords.
  - Free ends of floor plates are not allowed.
  - There has to be accounted for shear transfer between the floor elements.
  - Continuous flow of shear force is accounted for when the c.t.c. distance of the fasteners is at most 150 mm for nails, 200 mm for screws or 300 mm for other types of fasteners.
  - A simplified calculation is made according to the bending theory (if \( h_{\text{max}} \leq l/2 \)).
  - Chords on top and bottom of the diaphragm should resist the acting moment.
  - Shear force is assumed constant over the height of the diaphragm.
  - Also valid for multiple span diaphragms.

- SKH-publicatie 97-11: Dwarsstabiliteit Houtskeletbouw
  - The diaphragm is designed on the maximum acting forces in both longitudinal and end joints. These forces follow directly from the bending moment and shear distribution.
  - Top and bottom chord are designed on the maximum tensile force following from bending.
  - These chords may not be interrupted, except for when appropriate coupling is applied.
  - For the calculations, the diaphragm can be regarded as a deep beam.
  - Shear stresses are distributed uniformly (instead of parabolic) over the height of the diaphragm.
  - Bending moment is taken completely by the upper and bottom chords.
  - It is sufficient to account for bending moment and shear in the design calculations.
  - Generally, there is no need to check the shear stresses in the planking or horizontal deflections of the diaphragm as a whole.
  - Horizontal deformations can be determined by adding up deformations due to bending, shear and slip of the connectors.

- Timber designers’ manual: Diaphragm-action (Practical considerations to BSS268-2)
  - Only the situation where all four edges of the diaphragm are supported is considered (blocked).
    - To limit deflections of the diaphragm, the height-to-length ratio should not exceed 1:4.
    - Top and bottom chords are designed to resist tensile forces due to bending.
    - These chords may not be interrupted, except for when appropriate coupling (usually a backing plate of steel/timber) is applied.
    - The shear distribution over the joints is assumed to be parallel, rather than parabolic.

Theory applied in this thesis

As will be clear from the above, there is no such thing as a clear guideline on diaphragm-action of large floor slabs. Despite that, there is some guidance in designing such a diaphragm, which makes use of a highly simplified design method. It is rather questionable if the presented theory also applies for the large-scale diaphragm as used in this thesis. But due to the absence of additional guidance on this subject, this following theory is adopted:

- The (blocked) diaphragm is schematised as a deep beam
The upper and bottom chords are designed on the maximum acting tensile force.

The joints are designed to resist the maximum acting shear force.

The deflections are determined making use of a conservative analysis, adding up the deformations from bending, shear and fastener slip.

The outcome of this latter calculation is highly conservative, but also highly simplified, and should be treated with a certain reservation though.

D.5.2 Loads

The wind load has already been determined in B.2.4:

\[
p_{\text{wind-direction,rep}} = 1.14 \times 1.3 = 1.482 \text{ kN/m}^2 \quad (\Psi_0 = 0.0; \Psi_1 = 0.2; \Psi_2 = 0.0)
\]

It is assumed that this horizontal load is transferred by the floors to the central cores. This leads to the following load distribution on the distinct floors within the stadium structure, see table 108.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Effective height [m]</th>
<th>q_{wind-direction,rep} [kN/m]</th>
<th>q_{wind-direction,d} [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd floor</td>
<td>1.95</td>
<td>2.89</td>
<td>4.77</td>
</tr>
<tr>
<td>2nd floor</td>
<td>3.90</td>
<td>5.78</td>
<td>9.54</td>
</tr>
<tr>
<td>1st floor</td>
<td>4.21</td>
<td>6.24</td>
<td>10.30</td>
</tr>
<tr>
<td>Ground floor</td>
<td>3.89</td>
<td>5.76</td>
<td>9.50</td>
</tr>
<tr>
<td>-1st floor</td>
<td>3.45</td>
<td>5.11</td>
<td>8.43</td>
</tr>
</tbody>
</table>

In addition, there is accounted for a horizontal load following from the crowd movement on the grandstand. The value of this load amounts 10% for the vertical loading, and is extracted from Annex G, see table 109.

<table>
<thead>
<tr>
<th>Floor</th>
<th>q_{grandstand,hor,rep} [kN/m]</th>
<th>q_{grandstand,hor,d} [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd floor</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2nd floor</td>
<td>4.09</td>
<td>6.75</td>
</tr>
<tr>
<td>1st floor</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Ground floor</td>
<td>2.46</td>
<td>4.06</td>
</tr>
<tr>
<td>-1st floor</td>
<td>2.46</td>
<td>4.06</td>
</tr>
</tbody>
</table>

As shown in table 108 and table 109, the 2nd floor is governing for the design, wherefore it is decided to consider this floor in detail.

Since the simultaneous occurrence of wind and crowd movement is unlikely to happen, use is made of the quasi-frequent load combinations:

ULS 1: \[ 1.65 \times q_{k1} + 1.65 \times \Psi_{sl} \times q_{k1} \]

SLS 1: \[ 1.0 \times q_{k1} + 1.0 \times \Psi_{sl} \times q_{k1} \]

This leads to:

ULS 1a: \[ 9.54 + 0.25 \times 6.75 = 11.23 \text{ kN/m} \]

ULS 1b: \[ 6.75 + 0.0 \times 9.54 = 6.75 \text{ kN/m} \]

SLS 1a: \[ 5.78 + 0.25 \times 4.09 = 6.80 \text{ kN/m} \]

SLS 2a: \[ 4.09 + 0.0 \times 5.78 = 4.09 \text{ kN/m} \]

As can be seen, ULS 1a and SLS 1b are the governing load combinations and are therefore considered in the design.

D.5.3 Global diaphragm-action

**Loads**

As can be found in chapter 4, three cores are incorporated in the design, on a centre to centre distance of 55 m to each other. The total width of the stadium structure is 164 m.
Accounting for these values and the acting loads, the maximum bending moment and shear force is determined.\(^1\)

\[ M_d = \frac{1}{8} q_d * l^2 = \frac{1}{8} * 11.23 * 55^2 = 4250 kNm \]

\[ V_d = \frac{1}{2} q_d * l = \frac{1}{2} * 11.23 * 55 = 309 kN \]

\(^1\)The calculations are performed according to the theory as presented above and stated in [55], [56] and [57].

Without paying attention to the shown element pattern, it is found from figure 262, that the structure holds two main beams (chords) at a distance of 14.4 m distance from each other. These beams are considered to act as tensile ties.

Having a leverage arm of 14.4 m, a normal force of \(N_t = N_e = \frac{M_d}{a} = \frac{4250 \cdot 10^6}{14400} = 295 kN\) is found.

\[ \text{Required sectional properties of the chords} \]

It is assumed that these chords are glulam members of strength class GL28h. Considering both the wind load and the grandstand load as short-term loads, a value of 0.90 is found for \(k_{mod}\). Since the material factor for glulam is 1.25, the following is stated:

\[ A_{\text{edgebeam}} = \frac{295 \cdot 10^3}{19.5 \cdot 0.90 \cdot 0.90} = 21000 mm^2 \]

Assuming a beam width of 150 mm, it can be found that a height of only 150 mm is required. As a result, both beams should be taken at least 150 by 150 mm.

Having floor support beams with dimensions of 220 by 750 mm for the Stacked bond pattern, is concluded that these beams comply with the requirements largely.

The maximum shear force, is taken by so-called end beams. These end beams are located at a centre to centre distance of 50 m to each other, as indicated in figure 262.

Depending on the loading direction, the acting shear force acts as a tensile force or a compression force on the central cores. Since tension is governing in the design, the end beams are designed likewise:

\[ A_{\text{endbeam}} = \frac{309 \cdot 10^3}{19.5 \cdot 0.90 \cdot 0.90} = 22000 mm^2 \]

Assuming a beam width of 150 mm, it can be found that a height of only 150 mm is required. As a result, both beams should be taken at least 150 by 150 mm.
Having floor support beams with dimensions of 220 by 800 mm for the Stretcher bond pattern, is concluded that these beams comply with the requirements largely.

§ Connection between the floor element and the chords
To be able to transfer the acting tensile and shear forces from the floor element to the chords/end beams, there has to be accounted for a connection. The floor elements are laid on the chords, where both structural elements are connected to each other by means of fasteners.

Transfer of tensile force
The tensile force has to be introduced in the chords near the supports of the diaphragm (the cores), since this is the location where the tensile force acts, see figure 262.

As a first assumption, the proposed connection is made by means of screws (Φ12*160 mm, \( f_{uk} = 640 \) N/mm²). The situation as shown in the upperleft of figure 263 is considered governing (smallest thickness). The following design assumptions apply:

Lignatur bottom flange (C24):
- \( \rho_{k1} = 350 \) kg/m³
- \( t_1 = 82 \) mm
- \( \gamma_M;1 = 1.30 \)

Glulam support beam (GL28h):
- \( \rho_{k2} = 410 \) kg/m³
- \( t_2 = 60 \) mm
- \( \gamma_M;2 = 1.25 \)

- \( k_{mod} = 0.90 \) (short-term loading)

The Lignatur elements are loaded perpendicular to their grain, while the Glulam elements are loaded parallel to their grain. This results in the following values for the embedment strength:

\[
f_{h90k,1} = \frac{0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k}{k_90 \sin^2 90 + \cos^2 90} = \frac{0.082 \cdot (1 - 0.01 \cdot 12) \cdot 350}{1.51 \cdot \sin^2 90 + \cos^2 90} = 16.70 \text{ N/mm}^2
\]

\[
f_{h90k,2} = \frac{0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k}{k_90 \sin^2 90 + \cos^2 90} = \frac{0.082 \cdot (1 - 0.01 \cdot 12) \cdot 410}{1.51 \cdot \sin^2 90 + \cos^2 90} = 29.59 \text{ N/mm}^2
\]
\[
\beta = \frac{f_{u0,k,2}}{f_{u0,k,1}} = 1.77
\]

\[
M_y = 0.3 \cdot f_{u,k} \cdot (0.9d)^2.6 = 0.3 \cdot 640 \cdot (0.9 \cdot 12)^{2.6} = 0.093 \cdot 10^6 \text{kNm}
\]

Making use of Johansen’s equations, the following design strength is found:

\[
F_{vk} = \min\{a = 16.44; b = 21.30; c = 7.64; d = 7.53; e = 7.99; f = 7.55\} = 7.53 \text{kN/screw}
\]

and thereby:

\[
F_{vd} = F_{vk} \cdot \frac{k_{mod}}{\gamma_M} = 7.53 \cdot \frac{0.9}{1.3} = 5.21 \text{kN/screw}
\]

The maximum tensile force appeared to be 295 kN, which implies a total of 57 screws.

The minimum required spacing \(a\), amounts 60 mm (5d), whereby the tensile force is introduced in the glulam beam over a length of 3.5 m. This is considered acceptable.

Transfer of shear force

Since the shear force is almost equal to the tensile stress, the length over which this force is introduced in the end beams shall be around 3.5 m as well (when accounting for the same kind of connection). This is considered acceptable.
Annex D.6 Local diaphragm-action

In Annex D.5, the global diaphragm-action was considered. By global, the behaviour of the diaphragm as a whole is meant. This concerned the transfer of the horizontal loads from the façade to the cores, whereby the influence of the floor elements was neglected.

In this section, the focus is on the transfer of the forces by the floor elements. Since the floor elements should be able to transfer the shear forces to each other, there has to be accounted for connections where adjacent elements meet. By local diaphragm-action, the focus is therefore on the transfer the acting forces between adjacent elements.

At first, the stretcher bond pattern is considered, thereafter attention is paid to the stacked bond pattern.

D.6.1 Stretcher bond pattern

Accounting for the acting forces as determined in Annex D.5, a detailed analysis is made on the stretcher bond pattern, see figure 264.

To act as a diaphragm, the distinct floor elements should be able to exchange shear forces at their joints, which is only possible when they are interconnected.

Accounting for the acting forces within these joints, in this section, the required amount of fasteners is determined. Two alternative solutions are discussed: a doweled connection and a connection with an OSB layer nailed on top. For both solutions, consecutively, the end and the longitudinal joints are considered.

D.6.1.1 Doweled connection

End joints

Since Lignatur elements are concerned, the maximum width of the floor elements is 1000 mm. The maximum shear force that should be accounted for, totals a 309 kN, see D.5.3. Having a width of 14.4 m, this leads to a shear load of \( q_{V,d} = \frac{309}{14.4} = 21.5 \text{kN/m} \).

This load is transferred by dowels having the following characteristics:

\[
d = 30 \text{ mm} \\
f_{uk} = 600 \text{ N/mm}^2
\]

This leads to:

\[
M_{yk} = 0.3 \times f_{uk} \times d^{2.6} = 0.3 \times 600 \times 30^{2.6} = 1.247 \times 10^6 \text{kNm}
\]

To be able to transfer the acting shear loads, the Lignatur elements should be enclosed at their ends. It is decided to implement an end beam at the ends of all elements, having a thickness of 50 mm. The height of the element is limited to 198 mm, which is determined by the free height in between the flanges of the element, see D.1.1.

The end beam is taken a solid timber member, C24, having a characteristic density of 350 kg/m³.
This leads to:

\[ f_{h:0,k:1} = f_{h:0,k:2} = 0.082 \times (1 - 0.01 \times d) \times \rho_k = 0.082 \times (1 - 0.01 \times 30) \times 350 = 20.09 \text{N/mm}^2 \]

Since \( f_{h:0,k:1} = f_{h:0,k:2} \) (both end members are equal), \( \beta = \frac{f_{h:0,k:1}}{f_{h:0,k:3}} = 1.0 \)

The shear capacity of the dowels is determined by making use of the Johansen’s Equations for fasteners in single shear:

\[
F_{vRk} = \min[a; b; c; d; e; f] \quad \text{where:}
\]

\[
a = f_{h:1,k} \times t_1 \times d \\
b = f_{h:2,k} \times t_2 \times d \\
c = \frac{f_{h:2,k} \times t_1 \times d}{1 + \beta} \times \sqrt{\frac{\beta + 2 \beta^2}{1 + 2 \beta^2} + \left( \frac{t_2}{t_1} \right)^2 + \beta^2 \left( \frac{t_2}{t_1} \right)^2 - \beta \left( \frac{1 + t_2}{t_1} \right)} \\
d = 1.05 \frac{f_{h:1,k} \times t_1 \times d}{2 + \beta} \times \sqrt{\frac{4 \beta (1 + \beta) \times M_{y,k}}{f_{h:1,k} \times t_1^2 \times d} - \beta} \\
e = 1.05 \frac{f_{h:2,k} \times t_2 \times d}{1 + 2 \beta} \times \sqrt{\frac{2 \beta (1 + \beta) \times M_{y,k}}{f_{h:2,k} \times t_2^2 \times d} - \beta} \\
f = 1.15 \times \frac{2 \beta}{1 + \beta} \times \frac{2 M_{y,k} \times f_{h:2,k} \times d}{d}
\]

Implementing the values found earlier, the following is found:

\[ F_{vRk} = \min[a = 30.13; b = 30.13; c = 12.48; d = 28.82; e = 28.82; f = 44.59] = 12.48 \text{kN/dowel} \]

This leads to:

\[ F_{v,d} = \frac{F_{vRk} \times M}{f_{\text{mod}}} = 12.48 \times \frac{0.9}{1.3} = 8.64 \text{kN/dowel} \]

With an acting shear load of 21.5 kN/m, it is found that at least 3 (2.49) dowels are required each linear m. This leads to a spacing of about 330 mm which is far larger than the minimum required spacing of 4d (120 mm).

Since the bolts are arranged parallel to the grain direction, the effective number of bolts has to be checked:

\[ n_{ef} = \min\left[ n \times 0.9 \times \frac{s}{\sqrt{13 + d}} \right] = 2.58 \]

Since 2.58 > 2.49, the total of 3 bolts per linear m are sufficient for the end joints.

**Longitudinal Joints**

The maximum shear force that acts in the longitudinal joints follows from the distribution of the bending moment over the diaphragm. As found earlier, the maximum force here is 295 kN.

This force should be transferred by the longitudinal joint, having a total length of 50 m, which leads to a shear load of 5.9 kN/m.

Accounting for the same type of dowels as before (d=30 mm; \( f_{uk} = 600 \text{ N/mm}^2 \)), and an element thickness of only 31 mm (see D.1.1.), it is found that:

\[ F_{v,d} = \min[a = 18.68; b = 18.68; c = 7.74; d = 29.18; e = 29.18; f = 44.59] = 7.74 \text{kN/dowel} \]
This leads to a required amount of 56 dowels, spaced at a distance of 890 m. Due to this large spacing, all dowels are being effective.

D.6.1.2 Nailed OSB layer

In the preceding section, it was found that dowels are required in both the end joints and the longitudinal joints. One can imagine, though, that this brings along difficulties during the erection phase. An alternative solution is found by nailing an OSB layer on top of the floor elements.

As a first assumption, the OSB layer is taken 20 mm thick. Use is made of a smooth round nails, having a diameter of 5 mm (end joints) and 3 mm (longitudinal joints). The timber will not be pre-drilled.

End Joints

Accounting for OSB and a nail diameter of 5 mm, it is found that:

\[
F_{vd} = F_{vRk} \frac{w}{k_{nod}} = 7.74 \times \frac{0.9}{1.3} = 5.35 \text{kN/dowel}
\]

For the timber of the Lignatur element, it is found that:

\[
f_{h0,k1} = 65 \times d^{-0.7} \times t_1^{0.1} = 65 \times 5^{-0.7} \times 20^{0.1} = 28.43 \text{N/mm}^2
\]

The timber member should be penetrated over a length of at least 8 d, or 40 mm. Having an OSB layer with a thickness of 20 mm, the minimum nail length becomes 60 mm.

Implementing these values in the Johansen’s equations, it is found that:

\[
F_{vk} = \min(a = 2.84; b = 3.54; c = 1.37; d = 1.38; e = 1.65; f = 1.85) = 1.37 \text{kN/nail}
\]

\[
F_{vd} = F_{vRk} \frac{w}{k_{nod}} = 1.37 \times \frac{0.9}{1.3} = 0.95 \text{kN/dowel}
\]

As a result, a total of \(23 \left( \frac{21.5}{0.95} \right) = 22.63\) nails should be accounted for on each side of the joint, per linear m. This results in a spacing of 43 mm, which is already somewhat smaller than the maximum allowable value of 51 mm. This is neglected here.

With a spacing of about 8.5*d, the Eurocode (table 8.1) states that \(k_{ef} = 0.78\) and thus \(n_{ef} = n^{0.78} = 11.54\). By implementing additional nails, the spacing decreases even further, leading to even lower values of \(k_{ef}\). It is therefore concluded that the chosen configurations does not lead to satisfying results.

Alternative design

By implementing larger end beams, larger diameter nails could be introduced, whereby the spacing can be increased. Considering the largest nail diameter, 8 mm, a beam with a thickness of about 70 mm should be accounted for \((8 \times 0.85 \times 10 = 68 \text{mm})\).

Accounting for this situation, the capacity of the nails becomes 1.79 kN/nail. This leads to a total of 12 nails and a spacing of about 83 mm.

The number of effective nails, only becomes 8.3. By implementing additional nails, the spacing decreases even further, leading to even lower values of \(k_{ef}\) and thus less effective nails. It is therefore concluded that the proposed solution is not feasible when the end joints are concerned.

Longitudinal Joints

When the longitudinal joints are concerned, the width of the end beams within the Lignatur elements is 31 mm, see D.1.1. The maximum nail diameter is therefore limited to 3.64 m. Accounting for nails having a diameter of 3 mm, the following results are found for the OSB layer:
For the end beams, it is found that:

\[ f_{h:0;k:2} = 0.082 \times (1 - 0.01 \times d) \times p_{ck} = 0.082 \times (1 - 0.01 \times 3) \times 350 = 20.64 \text{N/mm}^2 \]

The timber member should be penetrated over a length of at least 8 \( d \), or 24 mm. With an OSB layer with a thickness of 20 mm, this leads to a minimum nail length of 44 mm.

Implementing these values in Johansen’s equations, it is found that:

\[
R_{k;v} = \frac{\text{mod } M}{\text{dowel } k_{\text{mod}}} = 0.78 \times \frac{0.9}{1.3} = 0.53 \text{kN/dowel}
\]

Accounting for the acting load of 295 kN, it is found that at least 557 nails are required on each side of the joint. This leads to a spacing of about 90 mm, whereby \( k_{ef} = 1 \) and \( n_{ef} = n \).

It is therefore concluded that a total of 90 nails is required, on each side of a longitudinal joint.

**D.6.2 Stacked bond pattern**

Also for this pattern, see figure 265, there has to be accounted for the forces as determined in Annex D.5.

![Figure 265: Stacked bond pattern (EW-direction)](image)

To act as a diaphragm, again, the distinct floor elements should be able to exchange shear forces at their joints, which is only possible when they are interconnected. Since only longitudinal joints are required for this solution, only these joints have to be considered.

Accounting for the acting forces within these joints, in this section, the required amount of fasteners is determined. Two alternative solutions are discussed again: a doweled connection and a connection with an OSB layer nailed on top.

**D.6.2.1 Doweled connection**

Since Lignatur elements are concerned, the maximum width of the floor elements is 1000 mm. The maximum shear force that should be accounted for, totals a 309 kN, see D.5.3. Having a width of 14.4 m, this leads to a shear load of \( q_{v;f} = \frac{309}{14.4} = 21.5 \text{kN/m} \).

Accounting for the same type of dowels as before \( (d=30 \text{ mm}; f_{uk} = 600 \text{ N/mm}^2) \), and an element thickness of only 31 mm (see D.1.1.), it is found that:

\[ F_{v;d} = \min[a = 18.68; b = 18.68; c = 7.74; d = 29.18; e = 29.18; f = 44.59] = 7.74 \text{kN/dowel and} \]
\[ F_{v;d} = F_{v;Rk} \frac{\text{mod } M}{k_{\text{mod}}} = 7.74 \times \frac{0.9}{1.3} = 5.35 \text{kN/dowel} \]

This leads to a required amount of 58 dowels, spaced at a distance of 245 mm. Since the bolts are arranged parallel to the grain direction, the effective number of bolts has to be checked:
\[ n_{\text{eff}} = \min \left[ n^{0.9 + \frac{d}{4}} \frac{s}{\sqrt{13 + d}} = \frac{58}{34.41} \right] = 34.41 \]

Since the effective number is smaller than the minimal required number of fasteners, additional fasteners are to be implemented. It is therefore decided to implement 2 rows of 58 fasteners.

**D.6.2.2 Nailed OSB layer**

Considering the results as shown in D.6.1.2, it is stated that it is not a feasible solution to implement a nailed OSB layer over the longitudinal joints. This solution is therefore neglected here.
Annex D.7 Stiffness of the diaphragm

To obtain a first idea on the stiffness of the diaphragm, a simplified analysis is made on the proposed element configurations. The diaphragm is considered a beam that is subjected to bending and shear.

D.7.1 Introduction

The diaphragm is schematised as a beam that is simply supported by two timber cores, which are on a c.t.c-distance of 55 m. Having a height of 14.4 m, the diaphragm is considered a slender beam.

The load follows directly from D.5.2, where it appeared that SLS 1a is the governing load combination:

\[ q_{\text{rep}} = 6.80 \text{ kN/m}^2 \]

Accounting for this load, the following load distribution is found (SLS), see figure 266.

Accounting for the different floor element configurations, the deformation of these diaphragms is determined. The deformation follows from the combined system (serial system) of shear deformation, slip of the fasteners and bending deformation. The theory that is applied here follows from [62].

For the purpose of the calculation, the behaviour of the floor diaphragm is highly simplified. It is assumed here that the chords (at the bottom and top) of the diaphragm provide the bending stiffness, while the floor elements provide the shear stiffness.

D.7.2 Deformations due to shear

The shear stiffness GA is assumed to be provided by the Lignatur elements, more specifically by the bottom and top plate of these elements.
For timber quality C24 the shear modulus $G_{\text{mean}}$ amounts 690 N/mm$^2$. The area that is addressed follows from a section of the Lignatur element. Due to warping of the section, only $5/6$ of the total area can be addressed:

$$\frac{5}{6} A = (t_{\text{bottom}} + t_{\text{top}}) \cdot h_{\text{plate}} = \frac{5}{6} (82 + 31) \cdot 14400 \, \text{mm}^2 = 1.35 \cdot 10^6 \, \text{mm}^2$$

where $h_{\text{diag}}$ is the length of the diaphragm parallel to the wind direction.

Since the exactly the same elements are used for both configurations, the deformations due to shear are equal.

The deflection at mid-span due to shear is determined from:

$$w_{\text{shear}}(\frac{1}{2} l) = \frac{q l^2}{8 G A} = \frac{6.80 \cdot 10^3 \cdot 55^2}{8 \cdot 690 \cdot 1.35 \cdot 10^6} = 2.75 \, \text{mm}$$

**D.7.3 Deflections due to bending**

The bending stiffness is assumed to be provided by the chords of the diaphragm only. These are located on a centre-to-centre distance of 14.4 m to each other, see figure 267.

![Figure 267: Section of the floor diaphragm](image)

The chords are therefore considered as flanges, which are connected to the web (the Lignatur elements) by means of fasteners. A first calculation on this connection was carried out in D.5.3. It is assumed here that this connection suffices for both floor element configurations.

The deflection at mid-span due to bending is determined from:

$$w_{\text{bending}}(\frac{1}{2} l) = \frac{5}{384} \frac{q l^4}{E I_{\text{eff}}} \, \text{[mm]}$$

Now, both structural configurations are taken into consideration.

**D.7.3.1 Stretcher bond pattern**

When the stretcher bond pattern is concerned, the chords (GL28h) have dimensions of 150 by 150 mm$^2$. Accounting for these dimensions, the connection as proposed in D.5.3 and the section as shown in figure 267, the connections efficiency is determined:

$$K_{\text{ser}} = \frac{1}{23} \sqrt[1.5]{\rho_{\text{m3}p_{\text{m1}}} \cdot d_{\text{screw}}} = \frac{1}{23} \sqrt[1.5]{420 \cdot 410 \cdot 12} = 4410 \, \text{N/mm}$$

$$\gamma_{\text{ser}} = \left[1 + \pi^2 \frac{E_1 \cdot A_1 \cdot s}{K_{\text{ser}} \cdot I_{\text{eff}}} \right]^{-1} = \left[1 + \pi^2 \frac{12600 \cdot 150 \cdot 150 \cdot 60}{4410 \cdot 55000} \right]^{-1} = 0.987$$
It appears that 98.7% of the bending stiffness of the flange is addressed when there is accounted for the proposed connection. The effective bending stiffness thereby becomes:

\[
E_{ldf} = E \cdot \frac{1}{2} \cdot \gamma_{ser} \cdot A \cdot h^2 = 12600 \cdot \frac{1}{2} \cdot 0.987 \cdot 150 \cdot 150 \cdot 14400^2 = 2.90 \cdot 10^{16} \text{Nmm}^2
\]

And the deflection at mid-span due to bending becomes:

\[
w_{\text{bending}}(\frac{1}{2}l) = \frac{5}{384} \cdot \frac{6.80 \cdot 10^3 \cdot 55^4}{2.90 \cdot 10^{10}} = 27.94 \text{ mm}
\]

D.7.3.2 Stacked bond pattern in EW-direction

When the stacked bond pattern is concerned, the dimensions of the chords follow directly from the dimensions of the support beams, which have dimensions of 220 by 750 mm².

Since the proposed connections holds from this configuration as well, the connection efficiency follows from:

\[
\gamma_{\text{lyser}} = \gamma_{\text{lyser}} = \left[ 1 + \pi^2 \cdot \frac{E_1 \cdot A_1 \cdot s}{K_{ser} \cdot \gamma_{ser}} \right]^{-1} = \left[ 1 + \pi^2 \cdot \frac{12600 \cdot 750 \cdot 220 \cdot 60}{4410 \cdot 55000^2} \right]^{-1} = 0.915
\]

The effective bending stiffness thereby becomes:

\[
E_{ldf} = E \cdot \frac{1}{2} \cdot \gamma_{ser} \cdot A \cdot h^2 = 12600 \cdot \frac{1}{2} \cdot 0.915 \cdot 220 \cdot 750 \cdot 14400^2 = 19.72 \cdot 10^{16} \text{Nmm}^2
\]

And the deflection at mid-span due to bending becomes:

\[
w_{\text{bending}}(\frac{1}{2}l) = \frac{5}{384} \cdot \frac{6.80 \cdot 10^3 \cdot 55^4}{19.72 \cdot 10^{10}} = 4.11 \text{ mm}
\]

D.7.4 Deflections due to slip of the fasteners

In addition to the deflections due to the deformation of the elements their selves, also the slip at the joints has to be accounted for.

The individual Lignatur elements are connected to each other by means of dowel-type fasteners. These fasteners hold a certain stiffness k, resulting in a certain amount of slip at each of the joints, see figure 268.

![Figure 268: Stiffness at the joints](image)

The amount of slip at each joint follows from the acting shear force:

\[
\Delta w = \frac{V}{k}
\]

The elements thereby shift in the direction of the horizontal loads, but remain parallel to each other. Since the elements are connected to each other, as a consequence, a shear deformation occurs in the elements. Accounting for this deformation, an equivalent value of the shear stiffness is determined.
Regarding figure 269, it is stated that for small deformations:

\[ \gamma = \frac{\Delta w}{\Delta x} = \frac{\Delta w}{b_{\text{element}}} \]

Combining this expression with the earlier found expression for the slip of the joint, it is found that:

\[ \gamma = \frac{V}{k^* b_{\text{element}}} \]

Since a linear-elastic material is at stake, there is a linear relation between the shear deformation and the acting shear stress:

\[ \tau = G \gamma = G \frac{V}{k^* b_{\text{element}}} \]

The shear stress over the height of the element is defined as:

\[ \tau = \frac{3V}{2A} \]

Combining both expressions, the following appear to be valid:

\[ G \frac{V}{k^* b_{\text{element}}} = \frac{3V}{2A} \]

Whereby the equivalent shear stiffness can be determined from:

\[ G A = \frac{3}{2} k^* b_{\text{element}} \]

In conclusion it is stated that the shear stiffness of the connection follows directly from the stiffness of the fasteners and the element width.

D.7.4.1 Stretcher bond pattern

Considering the stretcher bond pattern, see figure 270, it is found that the joints parallel to the wind direction are staggered and located at distances of 8.0 m to each other.
As a result, an average of 7.2 joints is found each 8.0 m, where each joint holds 3 connectors (see D.6.1.1). This leads to an average of 21.6 fasteners at each joint.

Accounting for the proposed doweled connection (d=30 mm, timber quality C24), $K_{ser}$ is determined from:

$$K_{ser} = \frac{1}{23} \rho m^{-1.5} d = \frac{1}{23} \times 420^{1.5} \times 30 = 11.23 \text{ kN/mm} \quad [\text{N/mm}]$$

This results in a value for $GA$ of:

$$GA = \frac{3}{2} \times 21.6 \times 11.23 \times 8000 = 2.91 \times 10^6 \text{ kN}$$

The deflection at mid-span due to the slip of the fasteners is:

$$w_{\text{shear}} = \frac{ql^2}{8GA} = \frac{6.80 \times 55^2}{8 \times 2.91 \times 10^6} = 0.89 \text{ mm}$$

**Note:** When this pattern is at stake, joints are present in both direction. The effect of the joints perpendicular to the main wind direction has been neglected in this analysis. The results should therefore be considered with a certain reservation.

**D.7.4.2 Stacked bond pattern**

Considering the stacked bond pattern, it is found that the joints parallel to the wind direction are located at distances of 1.0 m to each other, see figure 270.

![Figure 271: Configuration 3: parallel joints](image)

Each of these joints holds an effective number of 58 connectors (see D.6.2).

Accounting for the proposed doweled connection (d=30 mm, timber quality C24), $K_{ser}$ is determined from:

$$K_{ser} = \frac{1}{23} \rho m^{-1.5} d = \frac{1}{23} \times 420^{1.5} \times 30 = 11.23 \text{ kN/mm} \quad [\text{N/mm}]$$

This results in a value for $GA$ of:

$$GA = \frac{3}{2} \times 58 \times 11.23 \times 1000 = 0.98 \times 10^6 \text{ kN}$$

The deflection at mid-span due to the slip of the fasteners is:

$$w_{\text{shear}} = \frac{ql^2}{8GA} = \frac{6.80 \times 55^2}{8 \times 0.98 \times 10^6} = 2.63 \text{ mm}$$
D.7.5 Conclusions on the stiffness of the diaphragm

D.7.5.1 Stretcher bond pattern
In the preceding section the deformations due to bending, shear and fastener slip have been determined. Since the system is in series, the total deformation is found by adding up the individual results:

\[ w_{\text{total}} = w_{\text{shear}} + w_{\text{bending}} + w_{\text{slip}} = 2.75 + 27.94 + 0.89 = 32 \text{ mm} \]

The maximum acceptable horizontal deflection of the 2\textsuperscript{nd} floor is limited to:

\[ w_{\text{max}} = \frac{h_{\text{gt.floor}}}{300} = \frac{3900}{300} = 13\text{mm} \]

Since \( \frac{w_{\text{total}}}{w_{\text{max}}} = \frac{32}{13} = 2.46 > 1 \), it is concluded that problems might be expected due to excessive deformations.

As can be seen, the maximum deflection follows almost directly from the bending criteria. To improve the behaviour on bending, the cross-sectional area of the chords should be increased.

D.7.5.2 Stacked bond pattern (EW-direction)
Applying the same procedure for the stacked bond pattern, the following deformation is expected:

\[ w_{\text{total}} = w_{\text{shear}} + w_{\text{bending}} + w_{\text{slip}} = 2.75 + 4.11 + 2.63 = 10 \text{ mm} \]

Since \( \frac{w_{\text{total}}}{w_{\text{max}}} = \frac{10}{13} = 0.77 < 1 \), it is concluded that no problems are to be expected due to excessive deformations.

Note: the calculations as shown in this section are highly simplified. The conclusions drawn in this section should therefore be considered with some reservation. It is highly recommended to perform additional research on the subject.
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Annex E  Design of Grandstands

Annex E.1 Boundary conditions and loads

E.1.1 Dimensions
The maximum element weight amounts 60 tons. The maximum weight per element can therefore be
determined by dividing this amount by the amount of elements per truck.
\[ m_i = \frac{m_{\text{tot}}}{n} \text{ [tons]} \]

The characteristic density and the cross-sectional area being known, the weight per linear meter can be
determined:
\[ m_{\text{cross-section}} = \rho_c A_i \text{ [kg/m]} \]

The maximum element length can be determined by dividing the maximum element weight by the cross-
sectional weight:
\[ l_{\text{max}} = \frac{m_{\text{tot}}}{m_{\text{cross-section}}} \text{ [m]} \]

E.1.2 Loads
To obtain a proper insight in the behaviour of the grandstand elements, various load combinations will be
taken into consideration. Before elaborating on these combinations, at first, the acting loads will be
presented.

E.1.2.1 Loads
- Self-weight \( q_{\text{load;self-weight}} \) depends on the cross-sectional shape
- Weight of seats and finishing layers \( q_{\text{load;seating}} \) = 0.2 kN/m²
- Downward imposed grandstand load \( q_{\text{var;grandstand;ver}} \) = 5 kN/m²
- Upward imposed grandstand load \( q_{\text{var;grandstand;ver}} \) = 50% \( q_{\text{var;grandstand;ver}} \) = 2.5 kN/m²
- Horizontal imposed grandstand load \( q_{\text{var;grandstand;hor}} \) = 10% \( q_{\text{var;grandstand;ver}} \) = 0.5 kN/m²

Making use of these loads, the governing load combinations can be determined.

E.1.2.2 Load combinations
The structure should be checked on strength, deflections and vibrations. When strength criteria apply, the
Ultimate Limit State will be considered. For deflections and vibrations, the Serviceability Limit State is at
stake.

For strength, the following combinations should be considered:
\[ q_{\text{id}} = 1.49 \times q_{\text{self-weight}} + 1.49 \times q_{\text{seating}} + 1.65 \times \psi_0 \times q_{\text{var;grandstand;ver}} + 1.65 \times \psi_0 \times q_{\text{var;grandstands;ver}} + 1.65 \times q_{\text{var;grandstand;ver}} \]
\[ q_{\text{id}} = 1.32 \times q_{\text{self-weight}} + 1.32 \times q_{\text{seating}} + 1.65 \times q_{\text{var;grandstand;ver}} + 1.65 \times q_{\text{var;grandstand;ver}} \]
\[ q_{\text{id}} = 0.90 \times q_{\text{self-weight}} + 0.90 \times q_{\text{seating}} + 1.65 \times q_{\text{var;grandstands;ver}} + 1.65 \times q_{\text{var;grandstands;ver}} \]

Considering deflections, it is mentioned that two different situations have to be considered, being the
instantaneous and the final situation. Since the effects of creep and shrinkage are neglected here, only
the instantaneous situation will be considered.
\[ q_{\text{defl}} = 1.0 \times q_{\text{self-weight}} + 1.0 \times q_{\text{seating}} + q_{\text{var;grandstands;ver}} + \psi_0 \times q_{\text{var;grandstands;ver}} \]

For vibrations the combination of dead load and the combined value of the variable loading should be
considered.
Accounting for the above, the following combinations can be found:

ULS 1a: \(1.49 \times (q_{\text{self}} + 0.20) + 1.65 \times 0.25 \times 5.0\) (↓) + 1.65 \times 0.25 \times 0.5 (←)

ULS 1b: \(1.49 \times (q_{\text{self}} + 0.20) + 1.65 \times 0.25 \times 2.5\) (↑) + 1.65 \times 0.25 \times 0.5 (→)

ULS 2a: \(1.32 \times (q_{\text{self}} + 0.20) + 1.65 \times 5.0\) (↓) + 1.65 \times 0.25 \times 0.5 (←)

ULS 2b: \(1.32 \times (q_{\text{self}} + 0.20) + 1.65 \times 0.5\) (←) + 1.65 \times 0.25 \times 5.0 (↓)

ULS 3a: \(0.90 \times (q_{\text{self}} + 0.20) + 1.65 \times 2.5\) (↑) + 1.65 \times 0.25 \times 0.5 (→)

ULS 3b: \(0.90 \times (q_{\text{self}} + 0.20) + 1.65 \times 0.5\) (→) + 1.65 \times 0.25 \times 2.5 (↑)

SLS 1a: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 5.0\) (↓) + 1.0 \times 0.25 \times 0.5 (←)

SLS 1b: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.5\) (←) + 1.0 \times 0.25 \times 5.0 (↓)

SLS 1c: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 2.5\) (↑) + 1.0 \times 0.25 \times 0.5 (→)

SLS 1d: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.5\) (→) + 1.0 \times 0.25 \times 2.5 (↑)

SLS 2a: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.7 \times 5.0\) (↓) + 1.0 \times 0.6 \times 0.5 (←)

SLS 2b: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.7 \times 0.5\) (←) + 1.0 \times 0.6 \times 5.0 (↓)

SLS 2c: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.7 \times 2.5\) (↑) + 1.0 \times 0.6 \times 0.5 (→)

SLS 2d: \(1.0 \times (q_{\text{self}} + 0.20) + 1.0 \times 0.7 \times 0.5\) (→) + 1.0 \times 0.6 \times 2.5 (↑)
## Annex F Transport

<table>
<thead>
<tr>
<th>Means of transport</th>
<th>Dimensions</th>
<th>Description</th>
<th>Cost estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>By road</strong></td>
<td>Max. dimensions for vehicle with load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without approval</td>
<td>Length 12-18.75 m, Width 2.55 m, Height 4.00 m, Weight 40 t</td>
<td>Volume of load: 2.50 m x 2.60 m x 12.00 m</td>
<td>Depends on approval requirements and escort vehicle requirements.</td>
</tr>
<tr>
<td>With approval</td>
<td>Length 25.25 m, Width 3.5 m, Height 4.2 m, Weight 60 t</td>
<td>No escort vehicle up to 3 m</td>
<td></td>
</tr>
<tr>
<td>Special transport vehicles</td>
<td>Dimensions &gt; 3.50 m or height 4.20 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>By rail</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Large container</td>
<td>Depends on standards applied (ISO, EN), e.g. Isobox container 40 x 8 ft (12.129 m x 2.438 m x 2.438 m), Eurobox container 254 (6.058 m x 2.5 m x 2.6 m)</td>
<td>Containers up to a load of 70 m³, handled with cranes or fork-lift trucks.</td>
<td>Transport by rail is more economic for very long distances. However, the last leg of the journey to the building site is almost always by road.</td>
</tr>
<tr>
<td>Interchangeable bodies</td>
<td>HGV bodies without wheels</td>
<td>Transported on flat wagons.</td>
<td></td>
</tr>
<tr>
<td>Trailer (without tractor)</td>
<td></td>
<td>Transported on hopper wagons, handled at terminals or with cranes.</td>
<td></td>
</tr>
<tr>
<td>Direct transport of vehicles</td>
<td></td>
<td>Transported on low-platform wagons, handled at terminals or on mobile ramps.</td>
<td></td>
</tr>
<tr>
<td><strong>By ship</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Handled using containers (see above)</td>
<td>The container can be loaded directly at the factory or be taken to the port.</td>
<td>The costs fluctuate depending on route and workload of the shipping company; distance, duration and fuel costs are also relevant.</td>
</tr>
<tr>
<td><strong>By helicopter</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Depends on payload, up to 900 kg, up to 1500 kg, and up to 2500 kg</td>
<td>Helicopters are primarily used for sites with difficult access, for bulky loads and when quick delivery is critical.</td>
<td>Critical here are type of helicopter, rotation time (pure flying time) and difference in altitudes.</td>
</tr>
</tbody>
</table>
Annex G Weight Calculation
BELASTINGEN Preliminary Design:

Permanente belasting

Technische verdieping

standaard
- tegels 0,05 x 20,0 1,00
- dakbedekking + isolatie 0,15
- Houten vloer c18 massief 0,41 x 3,8 1,56
- plafond + leidingen 0,20

2,91 kN/m²

lokaal
- dakbedekking + isolatie 0,15
- Houten vloer c18 massief 0,41 x 3,8 1,56
- plafond + leidingen 0,20

1,91 kN/m²

Verdieping +2

standaard
- lichte scheidingswanden 0,8 x 3,60 / 2,5 1,15
- Houten vloer C18 massief 0,47 x 3,8 1,79
- plafond + leidingen 0,20

3,14 kN/m²

Verdieping +1

standaard
- lichte scheidingswanden 0,8 x 3,60 / 2,5 1,15
- Houten vloer C18 massief 0,47 x 3,8 1,79
- plafond + leidingen 0,20

3,14 kN/m²

Omloop 0

standaard
- lichte scheidingswanden 0,8 x 3,60 / 2,5 1,15
- Houten vloer C18 massief 0,47 x 3,8 1,79
- plafond + leidingen 0,20

3,14 kN/m²

Parkeerlaag -1

standaard
- Houten vloer C18 massief 0,37 x 3,8 1,42
- leidingen 0,20

1,62 kN/m²

lokaal
- lichte scheidingswanden 0,8 x 3,60 / 2,5 1,15
- Houten vloer C18 massief 0,47 x 3,8 1,79
- plafond + leidingen 0,20

3,14 kN/m²

Parkeerlaag -2

standaard
- bestrating 0,10 x 20,0 2,00
- grondpakket 0,30 x 18,0 5,40

7,40 kN/m²
Veranderlijke belasting

Technische verdieping
standaard
- installaties 4,00 kN/m² \([y = 1,0]\)
ketelhuis
- installaties 7,50 kN/m² \([y = 1,0]\)
dak school
- sneeuw 0,56 kN/m² \([y = 0,0]\)
- personen [onderhoud] 1,00 kN/m² \([y = 0,0]\)

Verdieping +2
kantoor / skybox / school
- veranderlijke belasting 4,00 kN/m² \([y = 0,5]\)
businesslounge / restaurant / supporters / bioscoop / casino
- veranderlijke belasting 5,00 kN/m² \([y = 0,5]\)

Verdieping +1
kantoor / skybox / school
- veranderlijke belasting 4,00 kN/m² \([y = 0,5]\)
businesslounge / restaurant / bioscoop / casino
- veranderlijke belasting 5,00 kN/m² \([y = 0,5]\)
sporthal school
- veranderlijke belasting [resonantie eis voor vloer] 5,00 kN/m² \([y = 0,5]\)

Omloop 0
kantoor / school / shop
- veranderlijke belasting 4,00 kN/m² \([y = 0,5]\)
businesslounge / restaurant / bioscoop / casino / horeca / keuken / omloop
- veranderlijke belasting 5,00 kN/m² \([y = 0,5]\)

Parkeerlaag -1
parkeren [personenauto's]
- veranderlijke belasting 2,00 kN/m² \([y = 0,7]\)
verkeersklasse 30
- veranderlijke belasting 2,00 kN/m² \([y = 0,7]\)
- puntlasten [aslast vrachtauto] 3 x 100,00 kN

installatieruimte
- installatie 7,50 kN/m² \([y = 1,0]\)
**Parkeerlaag -2**
- parkeren (personauto's)
  - veranderlijke belasting

**Algemeen**
- verkeersruimte (openbaar)
  - veranderlijke belasting

**Tribune**
- veranderlijke belasting (vaste zitplaatsen)

<table>
<thead>
<tr>
<th>Belastingen</th>
<th>Belasting</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Parkeren</td>
<td>2,00 kN/m²</td>
<td>y =0,7</td>
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<tr>
<td>Verkeersruimte</td>
<td>5,00 kN/m²</td>
<td>y =0,5</td>
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<tr>
<td>Tribune</td>
<td>4,00 kN/m²</td>
<td>y =0,25</td>
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### Additional loading at the Eastern building part

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<th>Total Area [m²]</th>
<th>Eff. height [%]</th>
<th>perc. area.</th>
<th>Load [kN/m²]</th>
<th>Load [kN]</th>
<th>Load [kN]</th>
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<tbody>
<tr>
<td>3rd floor</td>
<td>765</td>
<td>14,4</td>
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<td>Standard</td>
<td>78</td>
<td>2,91</td>
<td>1735,2</td>
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**Total weight floors [kN]** 28793,8
**Total weight on building part [kN]** 86381,4
### Timber core system

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### Shear walls EW

Walls 6
Total width 164 m
Effective width 27.3 m

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### Trusses NS

Trusses 8
Total width 164 m
Effective width 17.5 m

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**BELASTINGEN Final Design:**

**Permanente belasting**

### Technische verdieping

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<tr>
<td>dakbedekking + isolatie 0,72 x 1,0</td>
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<tr>
<td>Lignatur 320</td>
<td>0,72</td>
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<tr>
<td>Support beam GL28h 220*750 0,22 x 0,75 x 4,3 / 4,8</td>
<td>0,15</td>
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<tr>
<td>plafond + leidingen</td>
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### Verdieping +2

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<tr>
<td>Lignatur 320 0,72 x 1,0</td>
<td>0,72</td>
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<tr>
<td>Support beam GL28h 220*750 0,22 x 0,75 x 4,3 / 4,8</td>
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### Verdieping +1

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<td>Lignatur 320 0,72 x 1,0</td>
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### Omloop 0

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### Parkeerlaag -1

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Veranderlijke belasting

Technische verdieping
standaard
· installaties
  # kN/m² 1-2 = 0.8

Verdieping +2
kantoor
· veranderlijke belasting
  # kN/m² 1-2 = 0.5

Verdieping +1
kantoor
· veranderlijke belasting
  # kN/m² 1-2 = 0.5

Omloop 0
omloop
· veranderlijke belasting
  # kN/m² 1-0 = 0.7

Parkeerlaag -1
parkeren [personenauto's]
· veranderlijke belasting
  # kN/m² 1-0 = 0.7

Parkeerlaag -2
parkeren [personenauto's]
· veranderlijke belasting
  # kN/m² 1-0 = 0.7

Tribune
· veranderlijke belasting [vaste zitplaatsen]
  # kN/m² 1-0 = 0.7
### Additional loading at the Eastern building part

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**Total weight floors [kN]** 23574,6

**Total weight on building part [kN]** 70723,7
### Timber core system

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<th>Nd:core</th>
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### Self-weight core

- Complete cross-section: 457 kN
- Reduction due to openings: 25 kN

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