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Structural robustness and earthquake resistant design

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Abstract

The subject of the thesis is evaluation of possible modification of the structures dynamic characteristics, within the context of increasing its level of robustness and resistance to earthquake actions. The idea and suggested approach of design in this thesis is based on the solutions to the design challenges of The Rion-Antirion Bridge. The bridge is a good example of an approach of design, which gives the structure features of adapting to various load scenarios.

The scope of this thesis is to assess the suitability of such an approach of design for buildings in Norway where earthquake actions are considered, and suggest design procedures for earthquake resistant design by alternative design methods within the context of structural robustness and the design approach of applying adaptive features to the structure.

As assumed and discussed in this thesis, there is a direct correlation between a structures stiffness characteristics and the resulting force from earthquake actions. Approach of design methods, which gives the structure features of adapting to various load scenarios are discussed, presented and analyzed in the thesis. Based on the results of the analyses, this approach of design should be considered for structures where the probability of earthquake events are relatively low, yet sets the design criteria.

Initiation to the modification of the structures dynamic characteristics are based on the principle of known failure and sacrificial elements. One of the discussed ideas is to apply viscous dampers to the wind-bracing system combined with a sacrificial element locking the damper until the system is exposed to an earthquake or similar accidental extreme events.

The suggested design procedure and robustness-increasing methods discussed in this thesis are applied to a practical example of an existing structure and assessed based on results from vibration- and response spectrum analyses. Based on the analyses results, it is concluded that the suggested design procedure along with the robustness-increasing methods discussed, results in favorable and desirable features to the building. Cost-beneficial analyses (CBA) are not performed in this thesis, but since the discussed design approach gives the opportunity to design the structure based on criteria set by often-occurring environmental loads instead of unusual earthquake loads, it is assumed that it may reduce the cost factor, without compensating on the structures safety level.

Acknowledgements

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I would like to use this opportunity to express my gratitude to my supervisor at the University of Stavanger, Professor Gerhard Ersdal for his exceptional guidance, support and for always taking the time for discussions of various problems encountered during the work with the thesis.

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Finally, I would like to express my gratitude to my family and significant other, Vanja F. Aronsen, for the extra support, motivation and patience during my time with the thesis.

In loving memory of Ingeborg Fredheim.

Stavanger, 15th June 2015

Salmir Berbic

Table of Contents

1 Introduction

1.1 Background and scope

The Rion-Antirion Bridge in Greece is considered one of the longest multi-span cable-stayed bridges in the world with a length of 2880m and spans up to 560m. The bridge deck might be considered the longest cable-stayed suspended deck. In addition to 65m water depth and seabed of mostly loose sediment, it is highly exposed to earthquake activity and significant tectonic movements expanding at a rate of about 30mm a year [1].

The bridge is a good representation of the well-known paradox among civil engineers worldwide: Design structures stiff to avoid large deformations and withstand loads from high winds, yet flexible to a degree where the structure is capable to absorb some of the loads caused by earthquakes. Special and unique construction techniques are applied to the bridge, resulting in desirable features. The piers of the bridge rest on a layer of gravel (instead of being anchored) so that they are allowed to move laterally on the seabed in case of an earthquake, where the gravel layer absorbs the energy (avoid force transmission). Beneath the deck, an innovative system of struts is installed. The main strut is designed to withstand resulting forces from wind actions, but intended to break effectively in case of an earthquake, where the installed dampers around it will take the movement. The dampers act as a shock absorber, allowing the structure to avoid damage, but at the same time keeping it from swinging too violently.

The solution that was used for the Rion-Antirion Bridge for this problem inspired the idea to this thesis. The bridge is a good example to an approach of design, which gives the structure features of adapting to various load scenarios.

The scope of this thesis is to assess the suitability of such an approach of design for buildings in Norway where earthquake actions are considered. The low probability of earthquake events in Norway may cause doubts of requirements given in structural codes setting the design basis based on earthquake actions. The intention of the thesis is to suggest design procedures for earthquake resistant design by alternative design methods within the context of structural robustness and the design approach of applying adaptive features to the structure, such as situational load-resistant characteristics based on the load scenario.

In this thesis, viscous- and material dampers are discussed to determine the damping mechanism for practical examples of existing structures. One of the discussed methods is to apply viscous dampers to the wind-bracing system combined with a sacrificial element locking the damper until the system is exposed to an earthquake or similar accidental extreme events. Due to suspicion of relatively high costs involved in the installation of dampers, material dampers (ductile elements) are discussed and compared.

1.2 Description of task

Major focus of the thesis, **Structural Robustness and earthquake resistant design**, is to get a better understanding of how levels of robustness for a structure can be used, as a basis for design within the context of structural engineering when both frequently occurring actions (wind, snow, etc) and unexpected or unusual actions (earthquake) are considered.

The work in this thesis is based on the following issues:

- How do we classify the level of robustness for a structure? Are there any correlations to the level of safety for the structure?

- Frequently used method to increase robustness is to increase the dimensions of the elements in the bearing system. Are there any other methods, which are more cost-efficient?

- Is it possible to design a structure to have the feature of adapting to the specific load scenario? (Avoid large deformation due to wind, yet be able to deform during an earthquake to absorb or reduce the resulting force).

- Is it possible to increase a wind-bracings ductility level, without compensating its level of stiffness contribution to the system?

- How effective are viscous dampers installed in wind-bracing systems when earthquake actions are considered? Are the costs involved acceptable for common structures in Norway?

- Compare effectivity and costs of viscous dampers compared to material dampers (ductile elements).

- Compare increase of redundancy (additional load paths) to increase of resistance for main load path.

2 Dynamics of structures and earthquake response

A major part of the work in this thesis is to increase level of robustness of structures exposed to earthquake activities. The resulting loads created by earthquake- or seismic actions have to be defined with our understanding of dynamics and mechanical vibrations.

This chapter covers the basic theories used to simplify dynamic systems and define our understanding of dynamics of structures. Introducing the chapter with a presentation of theory of general dynamics based on Rao's book, *Mechanical Vibrations* [2], followed by a presentation of theory used to determine a systems eigenfrequency and the characteristics of arbitrary dashpot dampers.

2.1 General dynamics

2.1.1 Undamped SDOF system

Dynamic problems are often presented with simplified models. The simplest model is a single-degree-of-freedom (SDOF) system, which includes a concentrated mass and a linear spring system with a representative stiffness, as shown in Figure 2.1.

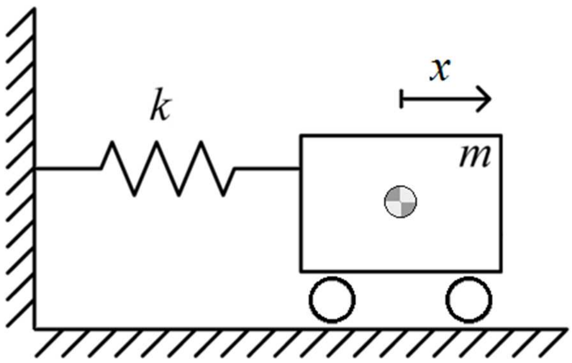


Figure 2.1. Simplified model of a single degree of freedom dynamic system, based on [2].

Deriving the equation of motion for the system shown in Figure 2.1, using Newton's second law of motion

$$F = m\,a \qquad (1.1)$$

As boundary conditions, it is chosen to set the point of equilibrium at the center of mass and the displacement, x, with positive direction as shown in the figure. The acceleration, $\ddot{x}$, at time, t, is therefore expressed as the double derivative of the displacement with respect to t

$$\ddot{x} = \frac{d^2x(t)}{dt^2} \tag{1.2}$$

Now there is an expression for the systems acceleration. Inserting equation (1.2) into equation (1.1), and since the mass is independent of time, the equation becomes

$$F(t) = m\,\ddot{x} \tag{1.3}$$

Introducing spring stiffness, k. This stiffness will during motion act as a force in the opposite direction of the motion. This phenomenon is expressed with the following equation

$$F(t) = -k\,x \tag{1.4}$$

Considering free body diagram on the system shown in Figure 2.1, the following equations may be set up

$$m\,\ddot{x} = -k\,x$$

$$m\,\ddot{x} + k\,x = 0 \tag{1.5}$$

Equation (1.5) is the general equation of motion for an un-damped single degree of freedom system. Solution to this differential can be found by assuming

$$x(t) = Ce^{st} \tag{1.6}$$

Where C and s are constants which are determined from the initial conditions. Substitution of equation (1.6) into equation (1.5) gives

$$C(ms^2 + k) = 0 \tag{1.7}$$

Since C cannot be zero ($C \neq 0$), the characteristic equation is expressed as

$$ms^2 + k = 0 \tag{1.8}$$

Solving equation (1.8) with respect to s, gives

$$s = \pm\sqrt{-\frac{k}{m}} \tag{1.9}$$

Equation (1.9) is simplified by introducing i and eigenfrequency (ω_n)

$$i = \sqrt{-1} \tag{1.10}$$

$$\omega_n = \sqrt{\frac{k}{m}} \tag{1.11}$$

$$s = \pm i\omega_n \tag{1.12}$$

Equation (1.8) is called the *auxiliary*- or *characteristic equation* corresponding to the differential equation (1.5). The two values of s given by equation (1.12) are the roots of the characteristic equation, also known as the *eigenvalues* or the *characteristic values* of the problem. Since both values of s satisfy equation (1.8), the general solution of equation (1.5) can be expressed as [2]

$$x(t) = C_1 e^{i\omega_n t} + C_2 e^{-i\omega_n t} \tag{1.13}$$

Where C_1 and C_2 are constants. Introducing the following identities

$$e^{\pm i\alpha t} = \cos\alpha t \pm i\sin\alpha t \tag{1.14}$$

Equation (1.11) is simplified by using the identities in equation (1.12)

$$x(t) = A_1\cos\omega_n t \pm A_2\sin\omega_n t \tag{1.15}$$

Where A_1 and A_2 are new constant. The constants C_i and A_i are both determined from the initial conditions of the system. The number of initial conditions to be specified is the same as the order of the governing differential equation. Considering the simplified system shown in Figure 2.1, following initial conditions may be determined [2]

Displacement, $x(t)$, at t=0 is defined as x_0. By inserting t=0 in equation (1.13) the equations is rewritten as

$$x(t = 0) = x_0 = A_1 \tag{1.16}$$

The systems velocity $\dot{x}(t)$ at t=0 is defined as $\dot{x}_0$. By deriving equation (1.13) with respect to time t, and inserting t=0, the equations is rewritten as

$$\dot{x}(t=0) = \dot{x}_0 = \omega_n A_2$$

$$A_2 = \frac{\dot{x}_0}{\omega_n} \tag{1.17}$$

Applying the boundary conditions (1.14) and (1.15) to equation (1.13) gives a solution, which is applicable for every undamped single degree of freedom system

$$x(t) = x_0 \cos \omega_n t + \frac{\dot{x}_0}{\omega_n} \sin \omega_n t \tag{1.18}$$

2.1.2 Damped SDOF system

In dynamic systems, the vibrational energy is gradually converted to heat or sound. This causes a reduction in energy, which leads to decreasing *response* and *displacement* of the system. The mechanism for this conversion of energy is known as *damping*. Even though the energy converted into heat or sound is small compared to the total energy, it is important that getting an accurate prediction of the vibration response of the system. Determining the cause of damping for a specific practical system is difficult, so we simplify it by modeling the damping as one or more of the following types: viscous damping, coulomb (dry-friction) damping, and material (hysteric) damping (this section is based on [3]).

Viscous damping is the most commonly used damping mechanism in vibrational analysis. This damping type is defined as damping due to movement of an element in a fluid medium such as air, gas, water or oil, which offers a resistance causing energy to be dissipated. Damping force is proportional to the velocity of the vibrating element (this section is based on [2]). Typical example of viscous damping is fluid flowing around a piston in a cylinder, as shown in Figure 2.2.

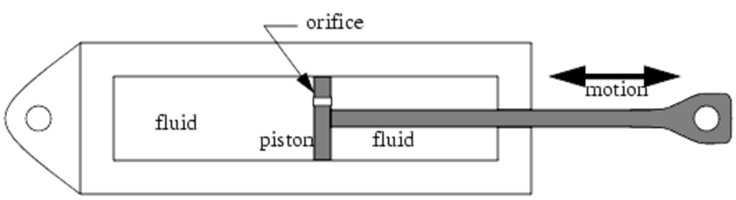


Figure 2.2. Example of viscous damping element [4].

Material damping is defined as damping due to material allowing to, or is able to deform so that energy is absorbed and dissipated by the material itself in the process. The effect of energy loss is caused by friction between the internal planes, which slide as the deformation takes place (based on [3]). When a system with material damping is exposed to vibration, the stress-strain diagram shows a hysteresis loop, as shown in Figure 2.3.

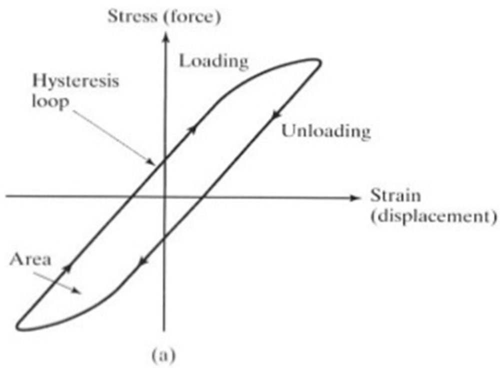


Figure 2.3. Hysteresis loop for elastic materials [2].

Procedure of deriving the equation of motion for a damped single degree of freedom system is not much different as presented in chapter 2.1.1. Figure 2.4 shows a simplified model of the damped SDOF system, where c is the damping constant for viscous damping, $\dot{x}$ is the velocity (first derivative of the position, x, with respect to time, t).

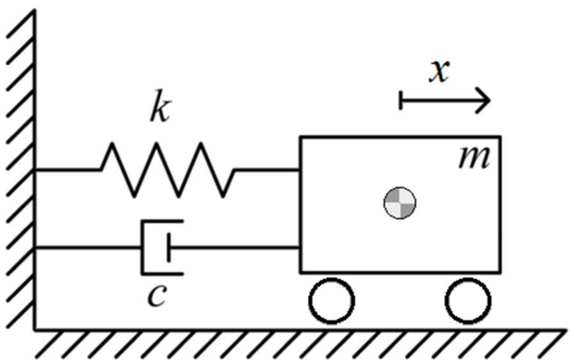


Figure 2.4. Simplified model of a single degree of freedom dynamic system with viscous damping, based on [2].

Viscous damping force, F, is negative because it acts in the opposite direction of the motion to the system. The force is proportional to the velocity, $\dot{x}$, and can be expressed by the equation

$$F(t) = -c\dot{x} \tag{2.1}$$

The position of equilibrium is set in the mass center of gravity, and Newton's law of motion is applied, resulting in the equation of motion

$$m\ddot{x} = -c\dot{x} - kx$$

$$m\ddot{x} + c\dot{x} + kx = 0 \tag{2.2}$$

To solve equation (2.2), solution in the following form is assumed

$$x(t) = Ce^{st} \tag{2.3}$$

Where C and s are constants which have to determine based on the initial conditions. Equation (2.3) inserted into (2.2) and based on that $C \neq 0$, the characteristic equation is given as

$$ms^2 + cs + k = 0 \tag{2.4}$$

Equation (2.4) solved with respect to s, gives the roots

$$s_{1,2} = \frac{-c \pm \sqrt{c^2 - 4mk}}{2m} = -\frac{c}{2m} \pm \sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}} \tag{2.5}$$

These roots give two solutions to equation (2.2), based on the assumption to the form of the solution given in (2.3)

$$x_1(t) = C_1 e^{s_1 t} \qquad \text{and} \qquad x_2(t) = C_2 e^{s_2 t} \tag{2.6}$$

Thus, the general solution of equation (2.2) is given by a combination of the two solutions presented in (2.4)

$$x(t) = C_1 e^{s_1 t} + C_2 e^{s_2 t}$$

$$x(t) = C_1 e^{\left\{-\frac{c}{2m} + \sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}}\right\}t} + C_2 e^{\left\{-\frac{c}{2m} - \sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}}\right\}t} \tag{2.7}$$

C_1 and C_2 are arbitrary constants, which are determined from the initial conditions set to the system. Equation (2.7) may be simplified by introduction of *critical camping constant* c_c, which is defined as the value for damping constant, c, for which the expression in the square root (radical) in equation (2.5) becomes zero

$$\left(\frac{c_c}{2m}\right)^2 - \frac{k}{m} = 0$$

$$c_c = 2m\sqrt{\frac{k}{m}} = 2m\omega_n \tag{2.8}$$

Introducing damping ratio, ζ, which is defined as the ratio of the damping constant to the critical damping constant

$$\zeta = \frac{c}{c_c} \tag{2.9}$$

Using equations (2.8) and (2.9), it can be rewritten as

$$\frac{c}{2m} = \frac{c}{c_c} \cdot \frac{c_c}{2m} = \zeta\omega_n \tag{2.10}$$

Which simplifies equation (2.5) to

$$s_{1,2} = \left(-\zeta \pm \sqrt{\zeta^2 - 1}\right)\omega_n \tag{2.11}$$

Thus, also simplifies equation (2.7), which can be rewritten as

$$x(t) = C_1 e^{\left(-\zeta+\sqrt{\zeta^2-1}\right)\omega_n t} + C_2 e^{\left(-\zeta-\sqrt{\zeta^2-1}\right)\omega_n t} \tag{2.12}$$

Solution to equation (2.12) depends on the magnitude of damping. In case the damping ratio, $\zeta = 0$, is inserted to equation (2.12) it leads to the undamped vibration equation (1.16) discussed in chapter 2.1.1. Thus, $\zeta \neq 0$ is assumed, giving the following three cases

Underdamped system	$\zeta < 1$	Case 1
Critically damped	$\zeta = 1$	Case 2
Overdamped system	$\zeta > 1$	Case 3

Case 1 represents an underdamped system, which is the most relevant degree of damping when the specific practical example (chapter 7) to this thesis is considered. Hence, derivation of the solution to equation (2.12) continues with applying the given value for the damping ratio, leading to

$$\zeta < 1$$

$$c < c_c \qquad \text{or} \qquad \frac{c}{2m} < \sqrt{\frac{k}{m}}$$

Due to this case, it is seen that the part, $(\zeta^2 - 1)$, in equation (2.12) becomes negative, and the roots can be expressed as

$$s_1 = \left(-\zeta + i\sqrt{1-\zeta^2}\right)\omega_n$$

$$s_2 = \left(-\zeta - i\sqrt{1-\zeta^2}\right)\omega_n$$

Solution to equation (2.12), can be written in the following forms

$$x(t) = C_1 e^{\left(-\zeta + i\sqrt{1-\zeta^2}\right)\omega_n t} + C_2 e^{\left(-\zeta - i\sqrt{1-\zeta^2}\right)\omega_n t}$$

$$x(t) = e^{-\zeta\omega_n t}\left\{C_1 e^{\left(i\sqrt{1-\zeta^2}\right)\omega_n t} + C_2 e^{\left(-i\sqrt{1-\zeta^2}\right)\omega_n t}\right\}$$

$$x(t) = e^{-\zeta\omega_n t}\left\{(C_1 + C_2)cos\left(\sqrt{1-\zeta^2}\omega_n t\right) + i(C_1 - C_2)sin\left(\sqrt{1-\zeta^2}\omega_n t\right)\right\} \qquad (2.13)$$

Introducing *damped eigenfrequency,* $\omega_d = \sqrt{1-\zeta^2}\omega_n$, and new constants, $C_1' = (C_1 + C_2)$ and $C_2' = i(C_1 - C_2)$. Applying eigenfrequency to equation (2.13) gives

$$x(t) = e^{-\zeta\omega_n t}\{C_1' cos(\omega_d t) + C_2' sin(\omega_d t)\} \qquad (2.14)$$

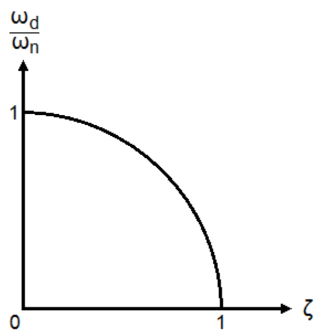


Figure 2.5. Figure illustrates variation of ω_d with damping [2].

One part of equation (2.14) is expressed with *cos*, while the other part is expressed with *sin*. This is simplified to one common part with introduction of a displacement factor, ϕ, which converts cos to sin, or the contrary. Constant, X, is the new combined constant. Rewriting equation (2.14) and applying the displacement factor and combined constant, gives

$$x(t) = X_0 e^{-\zeta\omega_n t} sin(\omega_d t + \phi_0) \tag{2.15}$$

$$x(t) = X e^{-\zeta\omega_n t} cos(\omega_d t - \phi) \tag{2.16}$$

C_1', C_2', X, X_0, ϕ, and ϕ_0 are arbitrary constants, which are determined from the initial conditions. For the initial condition, $x(t=0) = x_0$, it can be found that

$$x(0) = x_0 = e^0\{C_1' cos(0) + C_2' sin(0)\}$$

$$e^0 = 1 \qquad \cos(0) = 1 \qquad \sin(0) = 0$$

$$C_1' = x_0 \tag{2.17}$$

For the initial condition, $\dot{x}(t=0) = \dot{x}_0$, the following can be found by applying the initial condition to the first derivative of equation (2.14)

$$C_2' = \frac{\dot{x}_0 + \zeta\omega_n x_0}{\omega_d} \tag{2.18}$$

Full derivation of the first derivative to equation (2.14) is found in Appendix A.1. Applying equation (2.17) and equation (2.18) to equation (2.14), the solution becomes

$$x(t) = e^{-\zeta\omega_n t}\left\{x_0 cos(\omega_d t) + \frac{\dot{x}_0 + \zeta\omega_n x_0}{\omega_d} sin(\omega_d t)\right\} \tag{2.19}$$

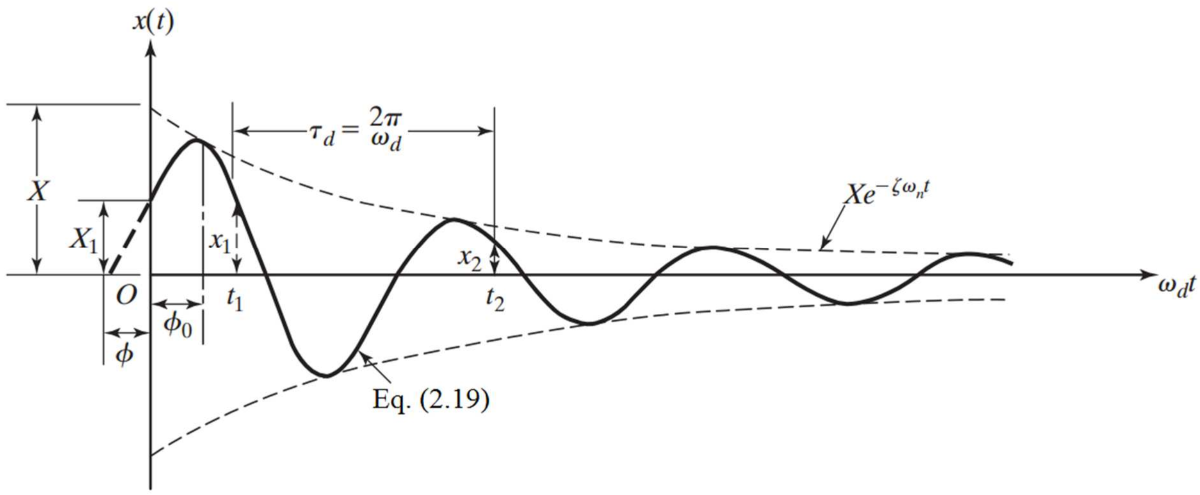


Figure 2.6. Figure illustrates the underdamped solution, based on [2].

Considering the form of the solution as presented in equation (2.15) and equation (2.16), the constants are expressed as

$$X = X_0 = \sqrt{(C_1')^2 + (C_2')^2} = \frac{\sqrt{x_0^2\omega_n^2 + \dot{x}_0^2 + 2x_0\dot{x}_0\zeta\omega_n}}{\omega_d} \tag{2.20}$$

$$\phi_0 = tan^{-1}\left(\frac{C_1'}{C_2'}\right) = tan^{-1}\left(\frac{x_0\omega_d}{\dot{x}_0 + \zeta\omega_n x_0}\right) \tag{2.21}$$

$$\phi = tan^{-1}\left(\frac{C_2'}{C_1'}\right) = tan^{-1}\left(\frac{\dot{x}_0 + \zeta\omega_n x_0}{x_0\omega_d}\right) \tag{2.22}$$

2.1.3 Response of a damped system under the harmonic motion of the base

In chapter 2.1.1 and chapter 2.1.2, the motion of dynamic systems with and without damping are discussed, but in both cases the base or support has been assumed to be in static equilibrium. However, in some cases, i.e. during an earthquake, the harmonic motion of the base or support in addition to the motion of the system itself have to be considered.

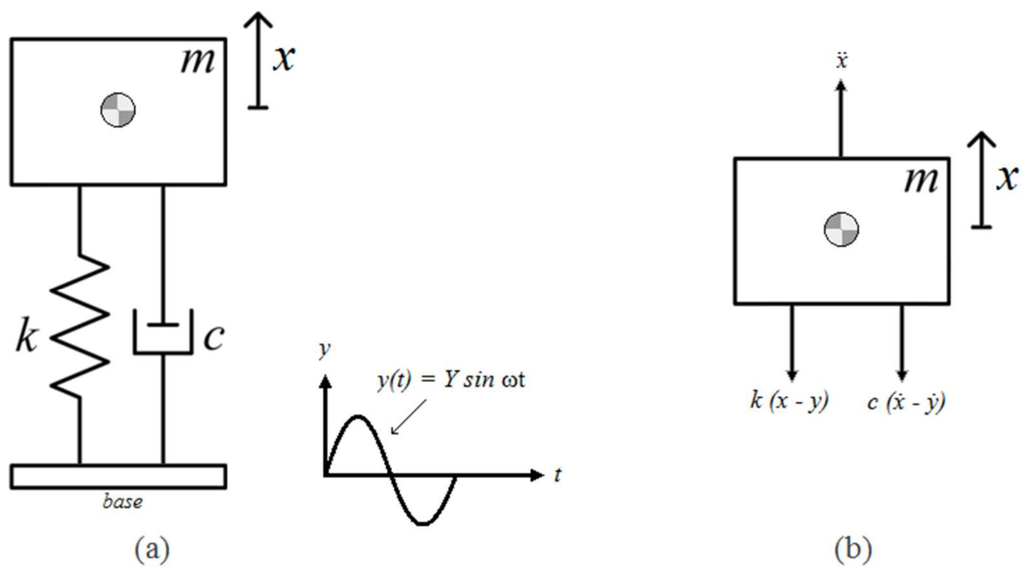


Figure 2.7. Simplified systems exposed to base excitation, based on [2].

In accordance to Figure 2.7 (a), $x(t)$ denotes the displacement of the mass and $y(t)$ denotes the displacement of the base, both with respect to the static equilibrium position at time t. Thus, the net elongation of the spring is expressed by $x - y$ and the relative velocity between the base and the mass is expressed by $\dot{x} - \dot{y}$. From the free-body diagram shown in Figure 2.7 (b), the equation of motion may be set up as

$$m\ddot{x} + c(\dot{x} - \dot{y}) + k(x - y) = 0 \tag{3.1}$$

Applying the assumed response for the base motion, $y(t) = Ysin(\omega t)$, the equation of motion may be simplified to

$$m\ddot{x} + c\dot{x} + kx = ky + c\dot{y}$$

$$m\ddot{x} + c\dot{x} + kx = kYsin(\omega t) + c\omega Ysin(\omega t)$$

$$m\ddot{x} + c\dot{x} + kx = Asin(\omega t - \alpha) \tag{3.2}$$

Where A and α are expressed with the following equations

$$A = Y\sqrt{k^2 + (c\omega)^2} \tag{3.3}$$

$$\alpha = \tan^{-1}\left[-\frac{c\omega}{k}\right] \tag{3.4}$$

This shows that giving excitation to the base is equivalent to applying a harmonic force of magnitude A to the mass.

When response of a damped system under $F(t) = F_0 e^{i\omega t}$ is considered and the particular solution is assumed to be $x_p(t) = Xe^{i\omega t}$, the steady-state solution becomes

$$x_p(t) = \frac{F_0}{[(k - m\omega^2)^2 + (c\omega)^2]^{1/2}} e^{i(\omega t - \phi)} \tag{3.5}$$

Using the steady-state solution expressed by equation (3.5), the steady-state response of the mass, $x_p(t)$, can be expressed as

$$x_p(t) = \frac{Y\sqrt{k^2 + (c\omega)^2}}{[(k - m\omega^2)^2 + (c\omega)^2]^{1/2}} sin(\omega t - \phi_1 - \alpha) \tag{3.6}$$

Where

$$\phi_1 = \tan^{-1}\left(\frac{c\omega}{k - m\omega^2}\right) \tag{3.7}$$

Using trigonometric identities, equation (3.6) can be rewritten in a more convenient form as

$$x_p(t) = X\sin(\omega t - \phi) \tag{3.8}$$

Where X and ϕ are given by

$$\frac{X}{Y} = \left[\frac{k^2 + (c\omega)^2}{(k - m\omega^2)^2 + (c\omega)^2}\right]^{1/2} = \left[\frac{1 + (2\zeta r)^2}{(1 - r^2)^2 + (2\zeta r)^2}\right]^{1/2} \tag{3.9}$$

$$\phi = \tan^{-1}\left[\frac{mc\omega^3}{k(k - m\omega^2) + (c\omega)^2}\right] = \tan^{-1}\left[\frac{2\zeta r^3}{1 + (4\zeta^2 - 1)r^2}\right] \tag{3.10}$$

The ratio of the amplitude of the response $x_p(t)$ to that of the base motion $y(t)$, $\frac{X}{Y}$, is called the *displacement transmissibility*. The variations of $\frac{X}{Y} \equiv T_d$ and ϕ given by equation (3.9) and equation (3.10) are shown in Figure 2.8 (a) and (b), respectively, for different values of r and ζ [2].

If the harmonic excitation of the base is expressed in complex form as $y(t) = Re(Ye^{i\omega t})$, the response of the system can be expressed as

$$x_p(t) = Re\left\{\left(\frac{1 + i2\zeta r}{1 - r^2 + i2\zeta r}\right)Ye^{i\omega t}\right\} \tag{3.11}$$

And the displacement transmissibility can be expressed as

$$\frac{X}{Y} = T_d = [1 + (2\zeta r)^2]^{1/2}|H(i\omega)| \tag{3.12}$$

Where $|H(i\omega)|$ is given by

$$|H(i\omega)| = \left|\frac{kX}{F_0}\right| = \frac{1}{[(1 - r^2)^2 + (2\zeta r)^2]^{1/2}} \tag{3.13}$$

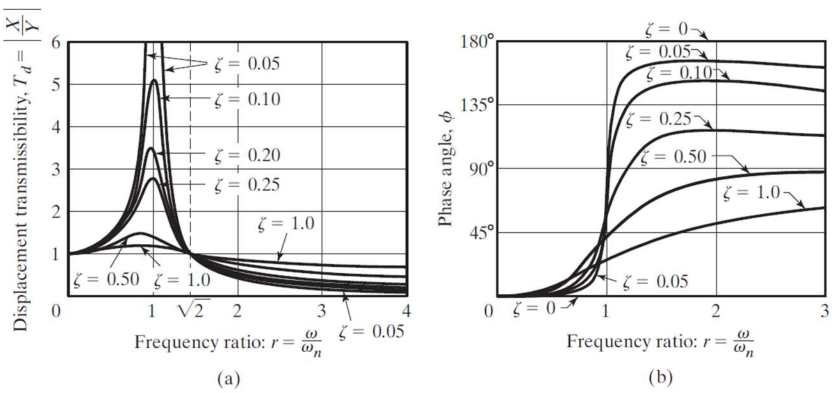


Figure 2.8. Variations of T_d and ϕ with r [2].

In Figure 2.7 (a), a force, F, is transmitted to the base or support due to the reactions from the spring and the dashpot. This force can be determined as

$$F = k(x - y) + c(\dot{x} - \dot{y}) = -m\ddot{x} \tag{3.14}$$

From equation (3.8), equation (3.14) can be rewritten as

$$F = m\omega^2 X \sin(\omega t - \phi) = F_T \sin(\omega t - \phi) \tag{3.15}$$

Where F_T is the amplitude or maximum value of the force transmitted to the base given by the following equation

$$\frac{F_T}{kY} = r^2 \left[\frac{1 + (2\zeta r)^2}{(1 - r^2)^2 + (2\zeta r)^2}\right]^{1/2} \tag{3.16}$$

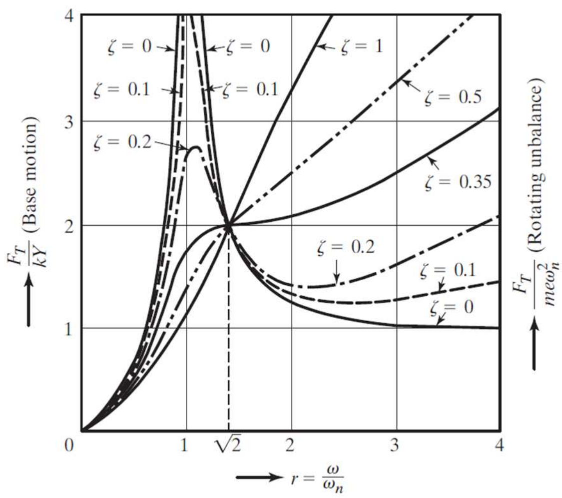


Figure 2.9. Force transmissibility [2].

The ratio $\frac{F_T}{kY}$ is known as the force transmissibility. Note that the transmitted force is in phase with the motion of the mass $x(t)$. The variation of the force transmitted to the base with the frequency ratio r is shown in Figure 2.9 for different values of ζ [2].

If $z = x - y$ denotes the motion of the mass relative to the base, the equation of motion, equation (3.1), can be rewritten as

$$m\ddot{z} + c\dot{z} + kz = -m\ddot{y} = m\omega^2 Y \sin(\omega t) \tag{3.17}$$

The steady-state solution to equation (3.17) is given by

$$z(t) = \frac{m\omega^2 Y \sin(\omega t - \phi_1)}{[(k - m\omega^2)^2 + (c\omega)^2]^{1/2}} = Z \sin(\omega t - \phi_1) \tag{3.18}$$

Where Z, the amplitude of $z(t)$, can be expressed as

$$Z = \frac{m\omega^2 Y}{[(k - m\omega^2)^2 + (c\omega)^2]^{1/2}} = Y \frac{r^2}{[(1 - r^2)^2 + (2\zeta r)^2]^{1/2}} \tag{3.19}$$

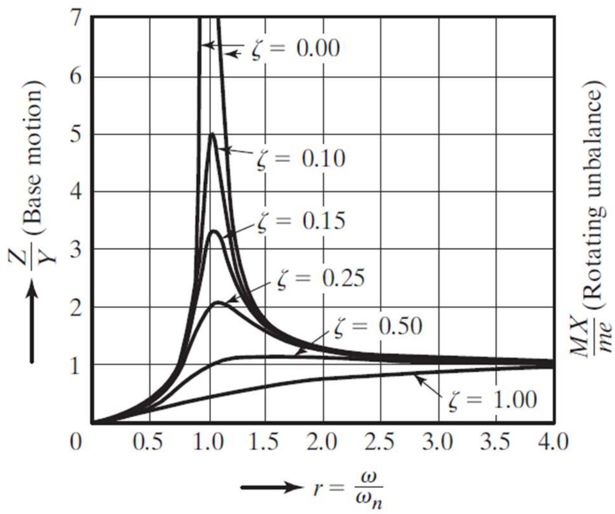


Figure 2.10. Variation of $\frac{Z}{Y}$ or $\frac{MX}{me}$ with frequency ratio $r = \frac{\omega}{\omega_n}$ [2].

And ϕ_1 is expressed by

$$\phi_1 = \tan^{-1}\left(\frac{c\omega}{k - m\omega^2}\right) = \tan^{-1}\left(\frac{2\zeta r}{1 - r^2}\right) \tag{3.20}$$

The ratio $\frac{Z}{Y}$ is shown graphically in Figure 2.10. The variation of ϕ_1 is same as that of ϕ shown in Figure 2.11.

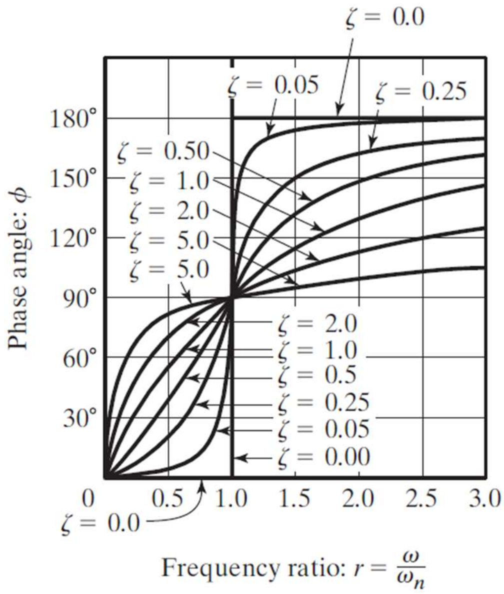


Figure 2.11. Variation of ϕ with frequency ratio $r = \frac{\omega}{\omega_n}$ [2].

In case of a ground shock, the velocity response spectrum is generally used. The displacement and acceleration spectra are then expressed in terms of the velocity spectrum. For a harmonic oscillator (an undamped system under free vibration), it is noticed that

$$\ddot{x}|_{max} = -\omega_n^2 x|_{max} \tag{3.21}$$

$$\dot{x}|_{max} = \omega_n x|_{max} \tag{3.22}$$

Thus the acceleration and displacement spectra S_a and S_d can be obtained in terms of the velocity spectrum S_v, where

$$S_d = \frac{S_v}{\omega_n} \tag{3.23}$$

$$S_a = \omega_n S_v \tag{3.24}$$

For an underdamped system subjected to base excitation, the relative displacement can be expressed with the following equation [2]

$$z(t) = -\frac{1}{\omega_d}\int_0^t \ddot{y}(\tau)e^{-\zeta\omega_n(t-\tau)}\sin(\omega_d(t-\tau))\,d\tau \tag{3.25}$$

To consider damping in the system, the maximum relative displacement is assumed to occur after the shock pulse has passed, and the subsequent motion must be harmonic. In such a case, equation (3.23) and equation (3.24) can be used. The fictitious velocity associated with this apparent harmonic motion is called the *pseudo velocity* and its response spectrum S_v, is called the *pseudo spectrum*. The velocity spectra of damped systems are used extensively in earthquake analysis. To find the relative velocity spectrum, equation (3.25) is differentiated[1]

$$\dot{z}(t) = -\frac{1}{\omega_d}\int_0^t \ddot{y}(\tau)e^{-\zeta\omega_n(t-\tau)}\left[-\zeta\omega_n \sin\left(\omega_d(t-\tau)\right) + \omega_d \cos(\omega_d(t-\tau))\right] d\tau \tag{3.26}$$

Equation (3.26) can be rewritten as

$$\dot{z}(t) = \frac{e^{-\zeta\omega_n t}}{\sqrt{1-\zeta^2}}\sqrt{P^2+Q^2}\sin(\omega_d t - \phi) \tag{3.27}$$

Where

$$P = \int_0^t \ddot{y}(\tau)e^{\zeta\omega_n t}\cos(\omega_d\tau)\, d\tau \tag{3.28}$$

$$Q = \int_0^t \ddot{y}(\tau)e^{\zeta\omega_n t}\sin(\omega_d\tau)\, d\tau \tag{3.29}$$

$$\phi = \tan^{-1}\left\{\frac{-\left(P\sqrt{1-\zeta^2}+Q\zeta\right)}{\left(P\zeta - Q\sqrt{1-\zeta^2}\right)}\right\} \tag{3.30}$$

The velocity response spectrum S_v, can be obtained from equation (3.27)

$$S_v = |\dot{z}(t)|_{max} = \left|\frac{e^{-\zeta\omega_n t}}{\sqrt{1-\zeta^2}}\sqrt{P^2+Q^2}\right|_{max} \tag{3.31}$$

Thus the pseudo response spectra are given by the following equations

$$S_d = |z|_{max} = \frac{S_v}{\omega_n} \tag{3.32}$$

$$S_v = |\dot{z}|_{max} \tag{3.33}$$

$$S_a = |\ddot{z}|_{max} = \omega_n S_v \tag{3.34}$$

[1] The following relation is used in deriving equation (3.26) from equation (3.25) [2]

$$\frac{d}{dt}\int_0^t f(t,\tau)\, d\tau = \int_0^t \frac{\partial f}{\partial t}(t,\tau)\, d\tau + f(t,\tau)|_{\tau=t}$$

2.2 Eigenfrequency

Eigenfrequency for an element or a structure is important to consider when analyzing it to any kind of harmonic loading, e.g. to prevent constructive interference (resonance). As presented in chapter 2.1.1, equation 1.11, eigenfrequency is dependent on the systems stiffness characteristics and the systems total mass. Introducing the chapter with a presentation of procedure to determine arbitrary systems stiffness characteristics, followed by a discussion of how to define active swinging mass of columns.

2.2.1 Stiffness characteristics

The stiffness characteristics of a structure is important when assessing the loads due to seismic actions. Higher stiffness is assumed to result in higher resulting forces on the structure caused by seismic actions. The assumption is based on suggested guidelines for seismic assessment and recommended equations for resulting earthquake force calculations, given in Eurocode 8 [5].

Following requirement is given in Eurocode 8, section 4.3.3.2.2. The seismic base shear force F_b, for horizontal direction in which the building is analyzed, shall be determined using the following expression: $F_b = S_d(T_1)\, m\, \lambda$, where

$S_d(T_1)$	is the ordinate of the design spectrum at period T_1
T_1	is the fundamental period of vibration of the building for lateral motion in the direction considered
m	is the total mass of the building, above the foundation or above the top of a rigid basement
λ	is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 \leq 2\, T_C$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise

The equation used to calculate base force F_b, is defined by a design spectrum based on a given value of period $S_d(T_1)$, mass of the building m, and a constant correction factor based period and amount of storeys λ.

In case the mass is considered a constant, the only variable in the equations becomes the design spectrum based on period T_1, which results to that the base force F_b directly correlates with the period T_1. In chapter 2.1.1, equation for eigenfrequency is introduced as

$$\omega_n = \sqrt{\frac{k}{m}}\ [{}^{rad}/{}_{s}] \tag{4.1}$$

Applying relation between frequency and period to equation (4.1) gives the equation for eigenperiod

$$T_n = \frac{2\pi}{\omega_n} = 2\pi\sqrt{\frac{m}{k}}\ [s] \tag{4.2}$$

Equation (4.2) solved with respect to stiffness factor k, results in a equation for stiffness factor where it is considered a variable with respect to mass and eigenperiod

$$k = \frac{4\, m\, \pi^2}{T_n^2} \tag{4.3}$$

As presented in equation (4.2) and equation (4.3), there is a direct correlation between the period T and the stiffness factor k. Thus, there has to be a correlation between the stiffness a building K and the resulting base shear force caused by seismic actions F_b.

Figure 2.12 shows a graph of relation between S/a_g and T, which represents the characteristic form of a response spectrum. a_g is a constant that defines the ground acceleration based on recommended values given in Eurocode 8 [5], further discussed in chapter 3.3.1.

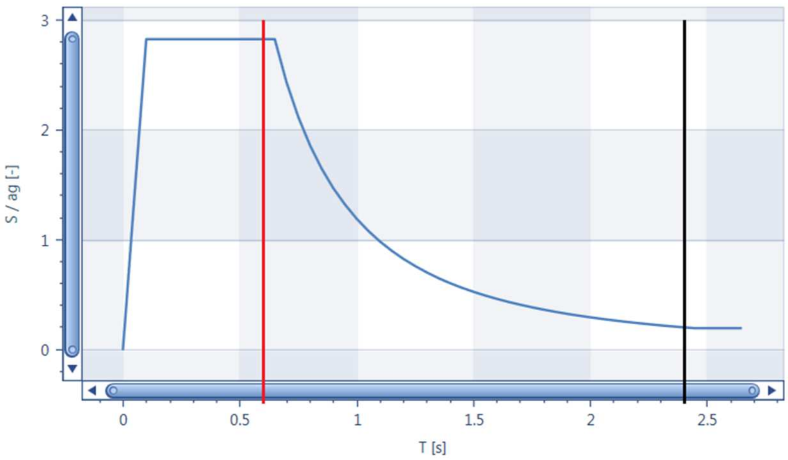


Figure 2.12. Characteristic form of response spectrum.

For a building with eigenperiod of 0.6s (marked with red), the resulting base force is at its maximum based on the value of S. In case the buildings stiffness is reduced to a degree where the period is increased to 2.4s (marked with black), the resulting base force is noticeably reduced based on the reduced value of S shown in Figure 2.12.

In case mass is considered a variable, it has an interesting effect on the resulting base force, since both the equation for base force and equation for eigenperiod correlate to the mass. Base force F_b *increases* simultaneously with increase of mass based on the equation $F_b = S_d(T_1)\, m\, \lambda$. However, period T increases when mass is increased (equation (4.2)), which gives a reduction in stiffness (equation (4.3)), causing a *reduction* of the base force. This is discussed further in chapter 8.1.

In the two following chapters, some examples of relevant systems (column and frame) are set up and hand calculated, to visualize the relation between force and stiffness where the results are plotted at the end of the examples.

2.2.1.1 Force vs. Stiffness · Column

For the first example, a simple rigid column (10m SHS100x8) is considered. Stiffness characteristics of the column is considered the same as for a cantilever beam. Derivation of the stiffness factor is done according to Rao's book, *Mechanical Vibrations* [2]. For simplicity, it is assumed that the self-weight of the element is concentrated as a point load at the free end of the element as shown in the Figure 2.13.

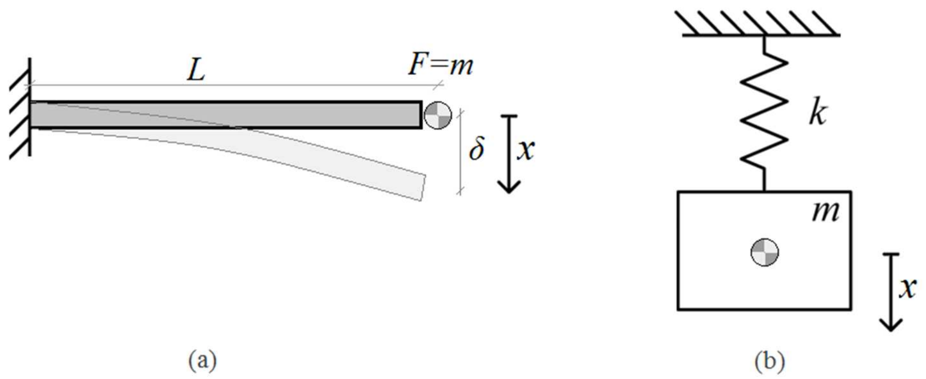


Figure 2.13. Cantilever beam (a) and an idealized model of it (b), based on [2].

From the strength of the material [6], the end deflection of the element due to the concentrated load from the mass in this case is given by

$$\delta = \frac{F\, L^3}{3\, EI} \tag{4.4}$$

Stiffness may be expressed as load divided by deflection. Considering the simplified model illustrated in Figure 2.13 (b), the spring constant of the element becomes

$$\frac{F}{\delta} = \frac{3\,EI}{L^3} \tag{4.5}$$

Where

F	is the load
δ	is the deflection
E	is Young's modulus
I	is the second moment of area
L	is the length of the element

To simplify the calculation process, the guidelines and equations recommended in Eurocode 8 [5] for seismic assessment (discussed in chapter 3.2) are programmed into a MathCad Prime 3.0 [7] sheet. Calculation of force is done by *lateral force method of analysis*, as discussed in chapter 3.3.2. A variable φ is set as a reduction factor with the value between 0.1 and 1.0 ($\varphi \in [0.1, 1.0]$). This variable indicates a factor of reduction to the systems stiffness characteristics. Response spectrum based on recommended values given in Eurocode 8 [5] for ground type E and seismic class IV is considered.

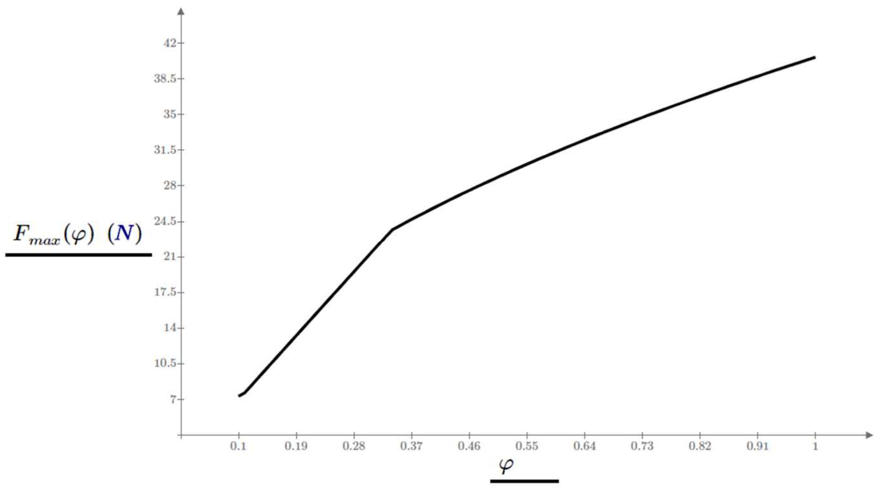


Figure 2.14. Graph of relation between force and stiffness reduction (Appendix A.2).

As assumed and discussed in chapter 2.2.1, the force is reduced by a reduction of the systems stiffness characteristics. Full calculation of the values in the graph is found in Appendix A.2.

2.2.1.2 Force vs. Stiffness · Frame

For the second example, a frame with single cross wind-bracings is considered, as discussed in chapter 7.2.2, since it represents the practical example assessed in chapter 7. Four cases of the frame are considered in this example:

1) default height with default stiffness (20.5m height)
2) reduced height with default stiffness (10.5m height)
3) default height with reduced stiffness
4) reduced height with reduced stiffness

For every case, the width of the frame is set to 24.0m and wind-bracing elements are of the type SHS140x8. Stiffness characteristics of the frame are simplified by considering single degree of freedom and only take into account the contribution to stiffness from the wind-bracing elements, since every node in the model of the practical example are hinged (chapter 7.1.2).

To simplify the calculation process, the guidelines and equations recommended in Eurocode 8 [5] for seismic assessment (discussed in chapter 3.2) are programmed into a MathCad Prime 3.0 sheet, with some modifications compared to the previous example. Calculation of force is done by *lateral force method of analysis*, as discussed in chapter 3.3.2. A variable φ is set as a reduction factor with the value between 0.05 and 1.0 ($\varphi \in [0.05,1.0]$). This variable indicates a factor of reduction to the systems stiffness characteristics. Response spectrum based on recommended values given in Eurocode 8 [5] for ground type E and seismic class IV is considered.

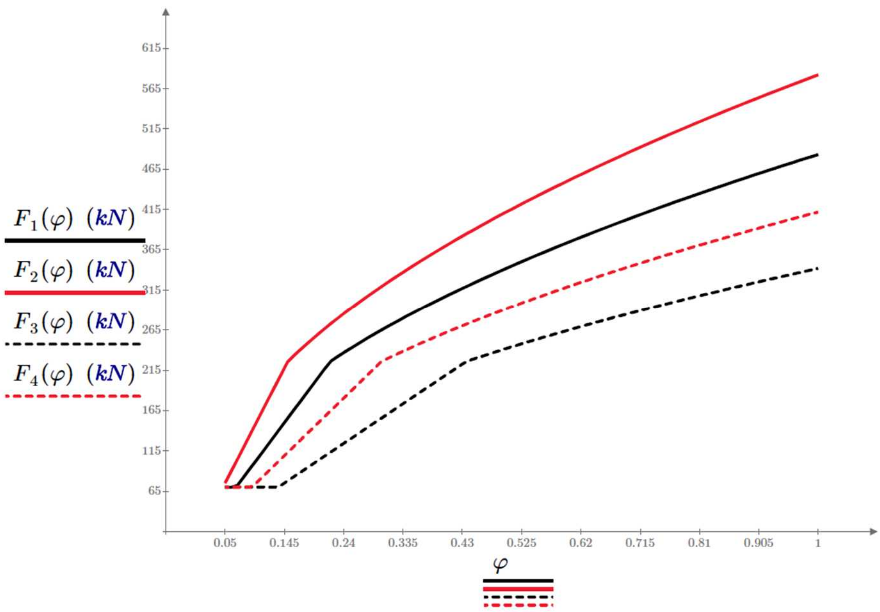


Figure 2.15. Graph of relation between force and stiffness reduction (Appendix A.3).

Index i for the force $F_i(\varphi)$ shown in Figure 2.15, indicates results for the various cases discussed in the introduction to this chapter. As assumed and discussed in chapter 2.2.1, the force is reduced by a reduction of the systems stiffness characteristics. Full calculation of the values in the graph is found in Appendix A.3.

2.2.2 Active swinging mass

For the example presented in chapter 2.2.1.1, definition of stiffness characteristics is simplified by assuming a massless rigid column and instead consider an imaginary mass on top of the column. This gives the following expression to define the stiffness of the column [2], as derived in chapter 2.2.1.1

$$k = \frac{F}{\delta} = \frac{3\,EI}{L^3} \tag{5.1}$$

For a more accurate approach, the mass due to self-weight of the element has to be considered as an evenly distributed mass along the element. For this calculation example, two arbitrary columns are set up where one is considered with a concentrated mass on top, and the other is considered with the more accurate approach.

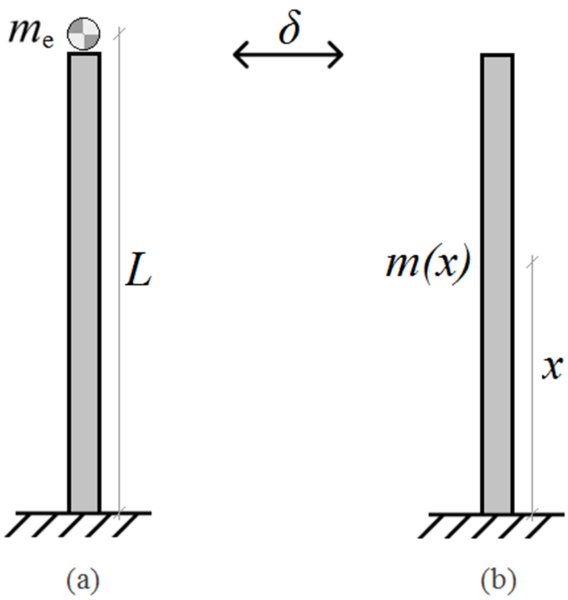


Figure 2.16. Illustration of rigid column.

Column to the left in Figure 2.16 (a) shows the simplified approach where a massless element with length L is considered, and the mass of the element, m_e, is set as a point load on top of the element. The column to the right Figure 2.16 (b) shows the more accurate approach where a column with the mass $m(x)$ as evenly distributed along the length of the element x, is considered. The similarities between the two columns is seen by $x = L$, resulting in $m(x) = m_e$.

The approach to find how much of the self-mass is active during vibration is based on setting up the equation for eigenfrequency for each case, and calculate the ratio between the masses, assuming same value of eigenfrequency for both cases. In chapter 2.1.1, equation for eigenfrequency is introduced as

$$\omega_n = \sqrt{\frac{k}{m}} \tag{5.2}$$

For case (a), the equation becomes

$$\omega_{n(a)} = \sqrt{\frac{3\ EI}{m_e\ L^3}} \tag{5.3}$$

For case (b), the equation becomes

$$\omega_{n(b)} = \sqrt{\frac{3\ EI}{m(x)\ x^3}} \tag{5.4}$$

To find out how much mass on top of the element for case (a) is equivalent to the evenly distributed mass along the element for case (b). In other words, the eigenfrequency for both cases has to be set equal. If so, equation (5.3) and equation (5.4) may be set equal to each other and solved with respect to the equivalent mass

$$\sqrt{\frac{3EI}{m_e\ L^3}} = \sqrt{\frac{3\ EI}{m(x)\ x^3}}$$

$$m_e\ L^3 = m(x)\ x^3$$

$$m_e = \frac{m(x)\ x^3}{L^3} \tag{5.5}$$

If a small element of length dx, a distance x from the ground level is considered, the elements mass will be mdx (where the value of m is defined as weight per unit length) and the part of the end mass equivalent is dm_e. Translating this into an equation, it becomes

$$d\ m_e = \left(\frac{x}{L}\right)^3 m\ dx \tag{5.6}$$

By integration along the whole length L of the element, gives

$$m_e = \int_0^L \left(\frac{x}{L}\right)^3 m\,dx = \left[\frac{x^4}{4}\frac{m}{L^3}\right]_0^L = \frac{L^4}{4}\frac{m}{L^3}$$

$$m_e = \frac{m\,L}{4} \tag{5.7}$$

Hence, when a practical column with a self-mass uniformly distributed along its length is compared to a theoretical massless column with the mass concentrated at the top, the equivalent mass on top is equal to $\frac{1}{4}$ of the total mass of the column.

2.3 Dashpot damper

Some of the case studies in this thesis include assessment of the effect from installing mechanical dampers to the wind bracing. The type of mechanical damper will be a dashpot damper, because of its well-defined damping characteristics. This chapter covers a presentation of how to calculate the characteristic damping coefficient for an arbitrary dashpot damper.

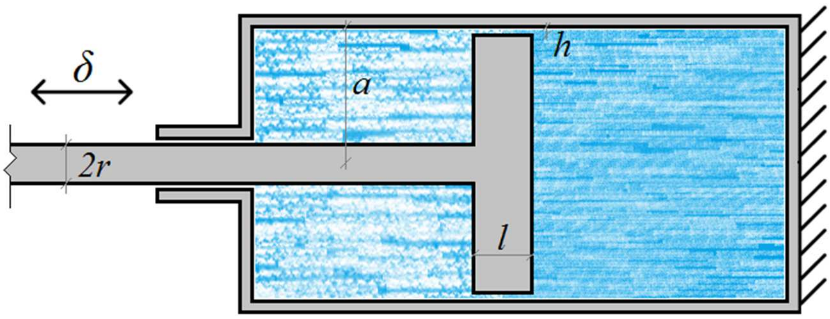


Figure 2.17. Simplified illustration of a dashpot damper, based on [4].

Figure 2.17 shows an idealized model of a dashpot damper. The model is idealized and simplified to easier be able to express the damping characteristics based on the system. The cylinder is rigid while the piston is free to move, with one degree of freedom, shown as δ in the figure. The cylinder is filled with a specific type of liquid, with a defined viscosity. The damping effect comes from the part of the piston moving through the fluid. For the piston to be able to move, the liquid on one side has to go through the small opening shown as h on the figure, to move over to the other side. This results in compression of the fluid on one side and friction between the liquid and material increases between the small openings.

The following equation is a suggest method to express the damping constant of a dashpot damper proposed by Cochin and Cadwallender, presented in their book *Analysis and Design of Dynamic Systems* [8]

$$c = \frac{6\,\pi\,\mu\,l}{h^3}\left[\left(a - \frac{h}{2}\right)^2 - r^2\right]\left[\frac{a^2 - r^2}{a - \frac{h}{2}} - h\right] \tag{6.1}$$

Where r, a, h and l are shown in Figure 2.17. The viscosity of the fluid is set as the value for μ, with typical SI unit $[Pa\ s]$. The damping effect from this dashpot damper acts theoretically as the viscous damper discussed in chapter 2.1.2.

3 Rules and standards

This chapter covers the relevant requirements, rules and guidelines given in the standards for design basis of structures with respect to robustness. Introducing the chapter with a brief summary from the transition phase from previous standards to Eurocodes, followed by a presentation of requirements within the context of robustness.

3.1 Transition to Eurocodes

The transition from the previous requirements setting the design basis for structural engineering, Norsk Standard (NS), to Eurocodes (EC) was official and active in Norway 1st of April 2010. The main purpose for this transition was to standardize the documentation of materials and elements frequently used in the industry. In other words, removing any kind of trade barriers. Leaving a bigger selection in the market, which reduces the raw material costs and thus also the cost of structures. This might as well give us a better understanding in general structural engineering. The section is based on [9].

In addition to the standardized requirements for design basis given in the Eurocodes, every country has its own *national appendix* (NA). These appendices cover the *national dependent parameters* (NDP).

3.2 Robustness in structural codes

Some of the existing structural codes do have certain requirements that the structures should be robust, however, only a few have the robustness requirements concretely defined. Since the procedure of work with analyses in the thesis is based on the Eurocodes, this chapter covers only the defined requirements to robustness set in the Eurocodes.

3.2.1 EN 1990 Basis for structural design

According to *Design for Robustness* by Franz Knoll and Thomas Vogel [10] the Eurocodes require robustness in their Basis of Design [11] only implicitly, referring to the two following requirements:

- 2.1(4) "*A structure shall be designed and executed in such a way that it will not be damaged by events such as:*
  - *explosions,*
  - *impact, and*

- *consequence of human errors,*

to an extent disproportionate to the original cause"

This basic requirement can directly be linked to some of the definitions of structural robustness, such as *damage tolerance* and *safety factors* (discussed in chapter 4). Structure designed to be damage tolerant results in a structure more robust regarding explosions and arbitrary impact loading. Safety factors in the design basis are used to mitigate the consequences caused by errors in the material, human errors and other arbitrary errors, which may emerge during the design process.

- 2.1(5) "*Potential damage shall be avoided or limited by appropriate choice of one or more of the following:*
  - *Avoiding, eliminating or reducing the hazards to which the structure can be subjected*
  - *Selecting a structural form which has low sensitivity to the hazards considered*
  - *Selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localized damage*
  - *Avoiding as far as possible structural systems that can collapse without warning*
  - *Tying the structural members together*"

One of the steps in a typical procedure of risk assessment is the *hazard identification* (hazid). When all possible hazards have been identified for the specific case, it has to be checked against the set accept criteria. In case it does not satisfy these criteria, a procedure of *risk reducing measures* is required. This procedure covers elimination or reduction of hazards that the structure may be exposed to. Thus, fulfilling the first point of requirement set by the Eurocode.

Designing a structure to have low sensitivity to the hazards considered is equivalent to design the structure in such a way that *progressive collapse* is not possible, insignificant of what member fails. Progressive collapse and methods to avoid are discussed in chapter 4.3.

The third requirement takes in consideration that the structure should survive adequately the accidental removal of an individual member (or a limited part of the structure), or the occurrence of acceptable localized damage. In other words, the structure requires higher levels of *redundancy*. One of the methods of applying situational characteristics to a structure is based on the idea of accepting localized damage to parts of the structure with adequate safety (chapter 6.2.3).

Structural systems collapsing without warnings has to be avoided. Easiest warnings to notice are visual warnings, such as cracks in the material or larger deformations to the element. Brittle material or structural systems with high stiffness are typical reasons for collapse without these kind of warnings.

The last requirement mentioned in 2.1(5) in the Eurocode is *tying the structural members together*. Difference between weld and bolts as parts connecting elements is discussed within the context of sacrificial elements in chapter 6.2.1.

The two requirements presenting robustness requirements according to Knoll and Vogel [10] are discussed above. Additional basic requirements found in the Eurocode are presented below, with suggested relevance to robustness based on their definition.

- 2.1(1) "*A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economical way:*
  - *sustain all actions and influences likely to occur during execution and use, and*
  - *meet the specified serviceability requirements for a structure or a structural element*"

Achieving an appropriate degree of reliability depends on the set accept criteria considering risks the structure may be exposed to. The structures robustness level is a key factor forming the degree of reliability. Considerations involving economics is a factor when determining the accept criteria.

- 2.1(2) "*A structure shall be designed to have adequate:*
  - *structural resistance,*
  - *serviceability, and*
  - *durability*"

Structural resistance can only be evaluated as adequate based on our understanding, ability to simplify and estimation of forces acting on the structure as realistic as possible.

In the following chapter of the Eurocode, chapter 2.2 *Reliability management*, there are requirements that also refer to robustness.

- 2.2(5) "*The levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of:*
  e) other measures relating to the following other design matters:
  - *the degree of robustness (structural integrity)*"

In this specific requirement, the Eurocode is not only referring to robustness but also to degrees of robustness, which is defined as *structural integrity*, according to the Eurocode.

3.2.2 EN 1991-1-7 Accidental actions

In Eurocode 1-7 on accidental actions [12], in section 1.5.14 (*Terms and definitions*), robustness is defined as: "*the ability of a structure to withstand events like fire, explosion, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause*".

In section 3.2 (*Accidental design situations – strategies for identified accidental actions*) the Eurocode introduces a guideline of ensuring *sufficient* robustness.

- 3.2(c) "*Ensuring that the structure has sufficient robustness by adopting one or more of the following approaches:*
  - *By designing certain components of the structure upon which stability depends as key elements to increase the likelihood of the structure's survival following an accidental event*
  - *Designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant strain energy without rupture*
  - Incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event"

Notice the second mentioned approach. One of the ideas to apply situational characteristics to structures, discussed in chapter 6.2, is based on giving parts of the structure sufficient ductility capable of absorbing significant strain energy without rupture. Methods of applying situational characteristics with use of ductile elements is discussed in chapter 6.2.3.

In section 3.3 (*Accident design situations – strategies for limiting the extent of localized failure*) the term robustness is used to define integrity and ductility. "*Applying prescriptive design/ detailing rules that provide acceptable robustness for the structure (e.g three-dimensional tying for additional integrity, or a minimum level of ductility of structural members subjected to impact)*" [12].

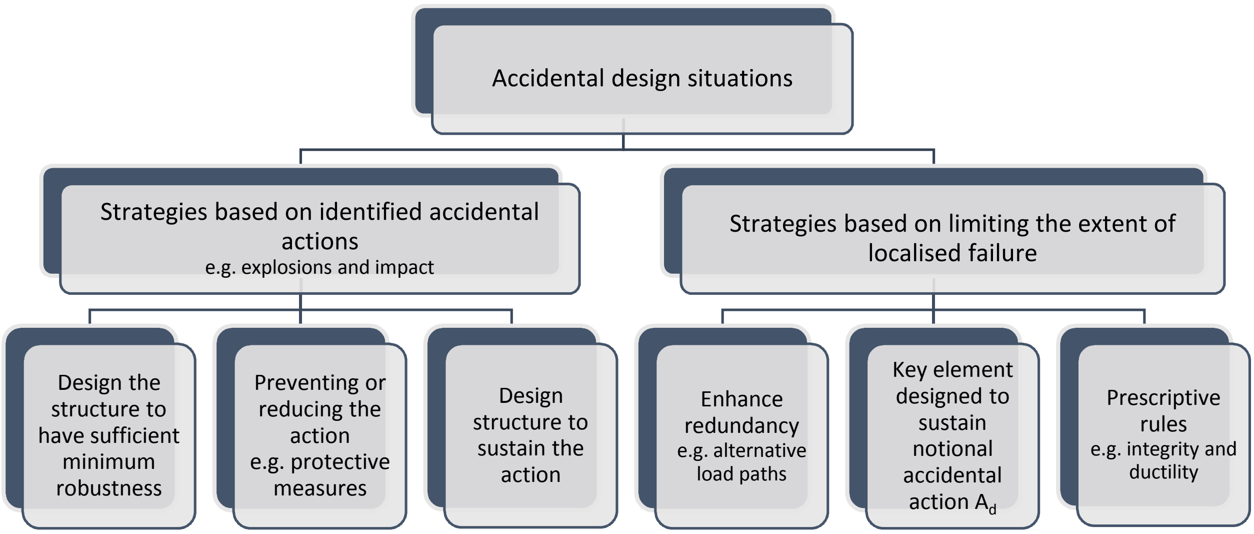


Figure 3.1. Design strategies specified in EN 1991-1-7 for accidental design situations [12].

Strategies and rules to ensure robustness are usually provided with regard to design for what are termed *accidental design situations*, which could arise due to indentified as well as unidentified or unforeseen accidental actions, as shown in Figure 3.1 (section based on [13]).

Table 3.1. Principal robustness requirements in the structural Eurocodes [13].

Consequence class	Primary requirements for robustness	Brief commentary
1 · Structures with insignificant consequence of failure	No specific requirements for robustness	
2a · lower risk group Can be seen as an intermediate class of structures with significant consequence of failure	Provision of horizontal ties or effective anchorage	· Prescriptive rules based on an assumed level of robustness · No identification of achieved robustness in different design situations
2b · upper risk group Can be seen as an intermediate class of structures with significant consequence of failure	Provision of horizontal ties and vertical ties, or	· Prescriptive rules based on an assumed level of robustness · No indication of achieved robustness in different design situations
	Notional member removal analysis and permissible limits for local damage	· Assessment approach that can be seen as performance-based with demonstration of achieved robustness · No further implementation guidance for consideration of credible design situations for application and strategies for ensuring robustness
	Key element design approach, where limits for local damage are exceeded during notional member removal analysis	· Prescriptive, when used together with the single recommended value of 34kN/m^2 · Highly scenario specific approach · No further specific guidance on the approach for determining suitable values for different design situations
3 · Structures with immensely significant consequences of failure and exceptional structures	Systematic risk assessment	· Conceptually correct approach · Rigor and detail make it impractical for the lower consequence classes

Structures (or more specifically buildings) are categorized under different consequence classes, which are primarily based on the use, occupancy and dimensions of the structures. Strategies and measures to ensure robustness are then specified for each consequence class. The principal robustness provisions in the Eurocodes [12] are given in Table 3.1 with a brief commentary (section based on [13]).

3.3 Earthquake design in Eurocode 8

This chapter covers the guidelines of analyzing structures for earthquake resistance in accordance to Eurocode 8 [5]. Introducing the chapter by a definition of *design spectrum for elastic analysis* followed by two relevant (for this thesis) methods of seismic analysis.

3.3.1 Design spectrum for elastic analysis

"The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response" [5].

According to the Eurocode, the horizontal components of the seismic action, the design spectrum, $S_d(T)$, shall be defined by the following expressions

$$0 \leq T \leq T_B \qquad S_d(T) = a_g\,S\left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \tag{7.1}$$

$$T_B \leq T \leq T_C \qquad S_d(T) = a_g\,S\,\frac{2{,}5}{q} \tag{7.2}$$

$$T_C \leq T \leq T_D \qquad S_d(T)\begin{cases} = a_g\,S\,\dfrac{2{,}5}{q}\left[\dfrac{T_C}{T}\right] \\ \geq \beta\,a_g \end{cases} \tag{7.3}$$

$$T_D \leq T \qquad S_d(T)\begin{cases} = a_g\,S\,\dfrac{2{,}5}{q}\left[\dfrac{T_C\,T_D}{T^2}\right] \\ \geq \beta\,a_g \end{cases} \tag{7.4}$$

Where

a_g	is the design ground acceleration on the specific ground type ($a_g = \gamma_1\,a_{gR}$)
S	is the soil factor
T_C	is the upper limit of the period of the constant spectral acceleration branch
T_D	is the value defining the beginning of the constant displacement response range of the spectrum
$S_d(T)$	is the design spectrum, depending on the natural period T

q	is the behavior factor, where recommended value is $q = 1.5$, more discussed below
β	is the lower bound factor for the horizontal design spectrum, where the recommended value is $\beta = 0.2$ (more specified in the national annex)

The behavior factor q is used as a reduction to avoid explicit inelastic structural analysis in the design. With the use of the behavior factor, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements, is taken into account by performing an elastic analysis based on a response spectrum with respect to the elastic one. Therefor called *design spectrum*.

The guideline above is for determining the horizontal components of the seismic action. When vertical excitation is considered, the vertical components of the seismic action are defined by the equations (7.1), (7.2), (7.3) and (7.4), with the design ground acceleration in the vertical direction a_{vg}, replacing a_g. Soil factor S, is set to be equal 1.0 and the other parameters are defined as mentioned in the national annex.

3.3.2 Lateral force method of analysis

"This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction" [5].

The requirement is satisfied in buildings, which fulfil both of the two following conditions:

- They have fundamental periods of vibration T_1 in the two main directions which are smaller than the following values

$$T_1 \leq \begin{cases} 4\,T_C \\ 2.0s \end{cases} \tag{8.1}$$

  Where T_C, is the upper limit of the period of the constant spectral acceleration branch, and is determined with respect to ground type.

- They meet the criteria for regularity in elevation, given as
  - For a building to be categorized as being regular in elevation, it shall satisfy all the conditions listed in the following paragraphs.
  - All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations on the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.
  - Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduced gradually, without abrupt changes, from the base to the top of a particular building.

- In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.

The seismic base shear force F_b, for horizontal direction in which the building is analyzed, shall be determined using the following expression

$$F_b = S_d(T_1)\, m\, \lambda \tag{8.2}$$

Where

$S_d(T_1)$	is the ordinate of the design spectrum at period T_1
T_1	is the fundamental period of vibration of the building for lateral motion in the direction considered
m	is the total mass of the building, above the foundation or above the top of a rigid basement
λ	is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 \leq 2\, T_C$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise

For buildings with heights of up to 40 meters the value of T_1 may be approximated by the following expression

$$T_1 = C_t\, H^{3/4} \tag{8.3}$$

Where

C_t	is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures
H	is the height of the building, with meters as unit, from the foundation or from the top of a rigid basement

There are two ways of determining the fundamental mode shapes in the horizontal directions of analysis of the building. It can either be calculate by using methods of structural dynamics or it can be approximated by horizontal displacements increasing linearly along the height of the building. The two following points show further process with respect to how the fundamental mode shapes are defined:

- When the mode shape is calculated by using methods of structural dynamics, the seismic action effects shall be determined by applying, to the two planar models, horizontal forces F_i to all storeys, this can be expressed with the following equation

$$F_i = F_b \frac{s_i \, m_i}{\sum s_j \, m_j} \tag{8.4}$$

Where

F_i	is the horizontal force acting on storey i
F_b	is the seismic base shear in accordance to equation (8.2)
s_i, s_j	are the displacements of masses m_i and m_j in the fundamental mode shape
m_i, m_j	are the storey masses associated with all gravity loads appearing the following combination of actions $\sum G_{k,j} + \sum \psi_{E,i} \, Q_{k,i}$. Defining representative mass combination for this type of analysis will be discussed further in chapter 7.2.

- When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces F_i should be taken as being given with the following equation

$$F_i = F_b \frac{z_i \, m_i}{\sum z_j \, m_j} \tag{8.5}$$

Where

z_i, z_j	are the heights of the masses m_i and m_j above the level of application of the seismic action (foundation or top of a rigid basement).

The following section (section 4.3.3.2.4) in Eurocode 8, takes torsional effects in consideration. I will here quote the guideline given in the Eurocode with minor adjustments: If the lateral stiffness and mass are symmetrically distributed in plan and unless the accidental eccentricity, $e_{ai} = \pm 0.05 \, L_i$, is taken into account by a more exact method[2]. The accidental torsional effects, may be accounted for by multiplying the action effects in the individual load resisting elements resulting from the application of distributing the horizontal forces F_i to the lateral load resisting system (assuming the floors are rigid in their plane), by a factor δ given by

$$\delta = 1 + 0.6 \, \frac{x}{L_e} \tag{8.6}$$

Where

x	is the distance of the element under consideration from the center of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered
L_e	is the distance between the two outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered

[2] Torsional effects e_{ai} may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i: $M_{ai} = e_{ai} \, F_i$

3.3.3 Modal response spectrum analysis

"This type of analysis shall be applied to building which do not satisfy the conditions given in (..) for applying the lateral force method of analysis" [5]. In other words, buildings who do not satisfy the conditions given to use LFMA, as discussed in chapter 3.3.2, should use the modal response spectrum analysis.

The Eurocode requires that response of all modes of vibration contributing significantly to the global response shall be taken into account. This requirement is deemed to be satisfied if either of the two following conditions can be demonstrated:

- The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure
- All modes with effective modal masses greater than 5% of the total mass are taken into account

Here a note is given that the effective modal mass, m_k, corresponding to a mode k, is determinded so that the base shear force F_{bk}, acting in the direction of application of the seismic action, may be expressed as $F_{bk} = S_d(T_k)m_k$. It can be shown that the sum of the effective modal masses (for all modes and a given direction) is equal to the mass of the structure.

In case the requirements specified above are not satisfied, e.g. in buildings with a significant contribution from torsional modes, the minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions:

- $k \geq 3\sqrt{n}$
- $T_k \leq 0.20\ s$

Where

k	is the number of modes taken into account
n	is the number of storeys above the foundation or the top of a rigid basement
T_k	is the period of vibration of mode k

In case of combination of modal responses the response in two vibration modes i and j (including both translational and torsional modes) may be taken as independent of each other, if their periods T_i and T_j satisfy the following condition

$$T_j \leq 0.9\ T_i \tag{9.1}$$

In case all relevant modal responses may be regarded as independent of each other, the maximum value E_E of a seismic action effect may be taken as

$$E_E = \sqrt{\Sigma\, E_{Ei}{}^2} \tag{9.2}$$

Where

E_E	is the seismic action effect under consideration (force, displacement, etc.)
T_k	is the value of this seismic action effect due to the vibration mode i

Method of considering torsional effects is not much different from the one discussed in the previous chapter. Whenever a spatial model is used for the analysis, the accidental torsional effects, $e_{ai} = \pm 0.05\, L_i$, may be determined as the envelope of the effects resulting from the application of static loadings, consisting of sets of torsional moments M_{ai} about the vertical axis of each storey i

$$M_{ai} = e_{ai}\, F_i \tag{9.3}$$

Where

M_{ai}	is the torsional moment applied at storey i about its vertical axis
e_{ai}	is the accidental eccentricity of storey mass i in accordance with the equation, $e_{ai} = \pm 0.05\, L_i$, for all relevant directions
F_i	is the horizontal force acting on storey i,for all relevant directions[3].

[3] Referring to equation (8.4) and equation (8.5) in chapter 3.3.2.

4 Structural robustness

This chapter covers discussion of the term *robustness*, its meaning for civil engineers and its importance for structural design basis. A major part the work in this thesis focuses on increasing a structures level of robustness based on the represented proposed robustness theories discussed in this chapter.

4.1 Introduction to robustness

The word robustness originates from the Latin word *rōbustus*, which directly translated to English means *strong*. "*Full of health and strength, powerfully built, sturdy*" are some examples of the various definitions of robustness [14].

Within the context of structural engineering, robustness is commonly understood as the ability of a structural system to withstand events such as explosion, impact or consequences of human errors without being damaged to an extent disproportional to the original cause [11], as discussed in chapter 3. Robustness is often used to describe properties such as strength, sturdiness, durability and the ability that enables them to survive unforeseen or unusual circumstances [10].

Robustness has been recognized as a desirable property in structures and systems as a result of several high profile system failure, such as the Ronan Point Apartment Building in 1968, where the consequences were deemed unacceptable relative to the initiating damage [15]. The initial incident was due to a gas explosion on the eighteenth storey, which blew out concrete panels forming part of the load-bearing wall at the corner of the building. The removal of this element caused the collapse of the corner of the block above the eighteenth floor. The weight of this as it fell caused the collapse of the corner in all the floors below.

New attention was given to robustness and design against disproportionate collapse after incidents such as the bombing of the Murrah Building in Oklahoma in 1995 and The World Trade Center collapse in 2001 [16]. As in the later case; even though a large number of load bearing members failed due to the impact of the airplanes, the buildings did not collapse immediately. First following the impact of the airplanes caused the collapse of the towers. After these incidents more focus has been given to the design against disproportionate collapse, especially related to explosions and other modern threats to buildings. Even so, no building regulations or codes of practice provide a useful guide to design for such requirements. Requirements to robustness given in the Eurocodes are discussed in chapter 3.2.

4.2 Robustness assessment methodologies

When assessing robustness it is necessary to assess the characteristics of the structural system, how the elements are linked together and how the load is transferred between them and most importantly: the effect of violations of any type of assumption made in the process of design, management and maintenance of the structural system.

The concept of robust structures is still an issue of controversy, since there are no well established and generally accepted criteria for a consistent definition and a quantitative measure of structural robustness [17]. There are however, several existing proposals of probability-based assessments of robustness. This chapter briefly covers two examples of probability-based assessment approaches.

4.2.1 Reliability-based assessment

There is a generic proposed measure of system damage tolerance, based on the increase in failure probability resulting from the occurrence of damage [18], where the vulnerability V of a system is defined as

$$V = \frac{P(r_d, S)}{P(r_0, S)} \tag{10.1}$$

Where

r_d	is the resistance of the damaged system
r_0	is the resistance of the undamaged system
S	is the protective loading on the system
$P()$	is the probability of failure of the system, as a function of the load and resistance of the system

This vulnerability parameter indicates the loss of system reliability due to damage. There are several reliability-based measures such as the one presented with equation (10.1), and they are all useful in that they quantify the increased probability of system failure caused by damage to a component. If a small level of damage significantly increases to the probability of system failure, than one could reasonably say that the system has a lack of robustness [18].

4.2.2 Risk-based assessment

Risk-based approaches are scenario based assessments with consideration of probabilities and consequences of failures and collapses, and are viewed as one of the most promising approaches when considering robustness of structures. The Eurocode also specifies a need for risk analysis when structures within class 3 are designed [4].

A metric for robustness of an engineered system is proposed by Baker, Schubert and Faber [15]. It is an attempt to identify problems caused by damage to a system within the context of probabilistic assessment, in the same matter as the procedure discussed in chapter 4.2.1. This approach however, incorporates the consequences of damage and failure, so that the calculation becomes risk-based rather than reliability-based. Robustness may be seen as the property of a structure to deny the consequences of structural failure to be disproportional to the original cause of the failure. Thus, an approach where measures of consequence are included in the calculation, is a more accurate approach, compared to the ones who do not.

Their approach divides consequences into direct consequences associated with the local component damage (that might be considered proportional to the initiating damage) and indirect consequences associated with subsequent system failure (that might be considered disproportional to the initiating damage) [18]. An index (index of robustness I_{rob}) is formulated by comparing the risk associated with direct and indirect consequences, defined as

$$I_{rob} = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \tag{10.2}$$

Where

R_{Dir} is the direct risk

R_{Ind} is the indirect risk

These risks are defined as shown in the event tree illustrated in Figure 4.1. The event tree initiates with an exposure (exposure before damage EX_{BD}), which has the potential to damage elements in a structure. In case of no damaged to the elements ($\overline{D}$), the even tree goes to zero and the analysis is finished. In case some elements are damaged (D), a variety of damage states can result. For each of these damage states, there is a probability that system failure (F) results. Consequences are associated with each of the possible damage and failure scenarios, and are classified as either direct (C_{Dir}) or indirect (C_{Ind}).

[4] Referring to consequence classes, as shown in Table 3.1.

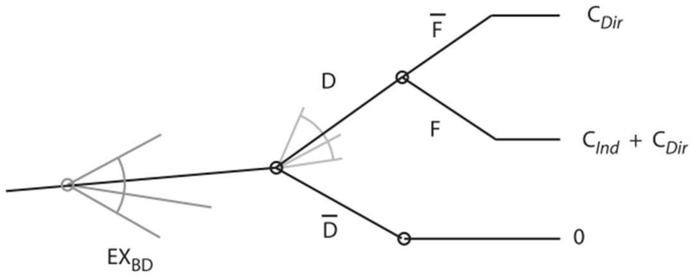


Figure 4.1. Event tree for robustness quantification [15].

Given that the needed probabilities are available, and that the consequence of each outcome can be assessed, the direct and indirect risks can be calculated with the following equations

$$R_{Dir} = \int_x \int_y C_{Dir} f_{D|EX_{BD}}(y|x) f_{EX_{BD}}(x)\, dy\, dx \tag{10.3}$$

$$R_{Indir} = \int_x \int_y C_{Indir} P(F|D = y) f_{D|EX_{BD}}(y|x) f_{EX_{BD}}(x)\, dy\, dx \tag{10.4}$$

4.3 Methods to increase robustness

Even though it does not have a consistent definition and a quantitative measure, structural robustness is both desirable and required within the context of structural engineering. In case robustness is considered as the property of a structure to deny the consequence of structural failure to be disproportional to the original cause of the failure, the method to increase robustness has to be based on the original cause but designed with respect to the progressive actions due to the original cause.

This chapter covers discussions of proposed theoretical methods to increase robustness when structures exposed to earthquake actions are considered. Some practical methods, relevant for the thesis, are discussed in chapter 6.

4.3.1 Accounting for sensitivities

Based on traditional methods of robustness in standards, G. Ersdal has set up a table (Table 4.1), in his compendium *Safety of structures* [19], summarizing ways to account for sensitivities of structures.

Table 4.1. Traditional ways of accounting for sensitivities [19].

Principle based on	Sensitivity evaluation examples	Traditionally accounted for by
Hazard event control	· Sensitivity to actions and hazards deviating unfavorable from the expected distribution.	· Action factor · ALS
	· Sensitivity to hazards and action acting in an unexpected manner.	· ALS
	· Sensitivity to errors in design (e.g. the calculations of actions, action effects, stresses, and capacity checks), material selection, fabrication, erection and use.	· Competency requirements · Quality assurance requirements
Limit sensitivity	· Sensitivity to how the assumptions that influence how the structure responds to the loads, e.g. the assumptions on boundary conditions for calculations, damping, mass distribution, etc.	· Conservative choices on values like damping
Specific load resistant design	· Materials deviating unfavorable from the expected distribution. · Different failure modes occurring from the expected (buckling, rupture, etc.) · Assumption on buckling lengths. · Fabrication, installation and erection tolerances. · Modeling of structure as a simplified mathematical model (stubs, eccentricities, etc.)	· Material faction · Conservative choices
Visual warnings	· Sensitivity to the failure sequence not being as assumed	· Joints stronger than members
Damage tolerance	· Single member failure. · Limitations in identifying the *key structural elements*. · Limitations in the assumption of where and how damage will occur. · Accelerated fatigue of damaged structure. · Ductility and impact action effects as a result of the member failure. · Escalading damage due to initial failure (failure to neighbor members and joints, *falling members*, etc.)	· Single member failure · Ductility in materials
Control of consequence	· Sensitivity to other possible consequences of a structural collapse.	

Based on the principles suggested in Table 4.1, *specific load resistant design*, *damage tolerance* and *control of consequence*, are the most relevant principles for the work in this thesis. The unique feature of installing sacrificial elements (chapter 6.1) to a structure are based on the specific load resistant design principle. Installation of mechanical dampers (chapter 6.2.2) and ductile elements (chapter 6.2.3) are based on the damage tolerance principle. The modifications to the structure assessed in the practical example to this thesis (chapter 7) is based on the control of consequence principle, where the structure is intended to change its dynamic characteristics in case of an earthquake to reduce the consequences.

4.3.2 Adapting the structure to the exposure

Adapting the structure to the exposure may be set as a general basis for the various methods to increase robustness. The first principle mentioned in Table 4.1 is *hazard event control*. Before a suitable robustness-increasing method may be determined, the potential hazards for the specific structural system need to be defined and assessed.

The main purpose of the methods is to avoid total collapse due to progressive actions caused by a disproportionate initiating action. I.e. in case of direct damage, the applied method should deny any case of indirect damage resulting in total collapse of the structure.

The structure considered in the practical example to this thesis (chapter 7) is designed with an adapting-to-exposure feature based on its dynamic characteristics. It is intended that the structure is stiff when exposed to wind actions (restricted against horizontal motion), while in case of earthquake actions the structures stiffness is reduced to allow horizontal motion to swing along with the base motion. By doing so, it is assumed that the resulting forces from the earthquake actions are reduced, as discussed in chapter 2.2.1.

This approach of increasing robustness is based on the control of consequence principle introduced in Table 4.1. In cases the resulting forces from a relatively rare extreme event (earthquake) is higher than the often-occurring forces due to wind- and snow actions, it may not be considered cost-efficient to set the design basis of the structure on the highest acting load, based on the low probability of occurrence. Therefore, the structure is initially designed based on the often-occurring environmental actions (wind and snow), with an applied method of reducing the stiffness, causing the resulting forces from the extreme event to decrease adequately to avoid total collapse of the structure.

Practical examples of methods, which are assumed to give the structure this feature are discussed in chapter 6. Some of the discussed methods are theoretically analyzed in chapter 7.2, where a discussion of suitability of the method for the specific example is discussed in chapter 8.2.

4.4 Maintenance of robustness

Applying the robustness-increasing feature to the structure or specific element is not the complete job. When the feature is applied, the procedure switches from installation to maintenance. The effect of the applied feature may decrease over time due to ageing effects or fatigue of the material. If an eventual generalized procedure for applications of robustness-increasing feature is made, a requirement for *reassessment* of the features effect should be required.

In case the robustness-increasing feature is based on sacrificial (fuse) elements, as discussed in chapter 6.1, the applied feature requires *reestablishment* after it has served its purpose the first time, if it is still intended to be accounted for.

5 Practical design procedure for earthquake analysis

Compared to other parts of the world, Norway is rarely exposed to considerable seismic activity [20]. However, due to its membership in The European Economic Community (EEC) it is required to follow some set provisions considering earthquake calculations as a part of the design basis for structural engineering.

This chapter covers the design procedure for seismic analysis and evaluation of structures using the software *Focus Konstruksjon 2015* [21] (later referred to as *Focus*), in accordance to the guidelines for seismic analysis recommended in Eurocode 8 [5], as discussed in chapter 3.3.

5.1 Focus software

Focus is a graphical calculation tool used by structural engineers to simplify the design process of structures. The software performs a variety of calculations to check suitability of chosen profiles and dimensions on elements with regard to the Eurocodes.

Focus has the ability to perform vibration analysis (to determine the modelled structures stiffness characteristics) and response spectrum analysis. To be able to rely on the results from the software, it is important to understand how the software operates. The following chapters cover the whole procedure from modelling to analysis results, with an example at the end compared to simple hand calculations.

5.1.1 Modelling

The first step in the process is to draw a good model in Focus, which represents the structure and its characteristics as realistic as possible. In cases where a specific *detail* of the structure cannot be modelled realistically, a conservative detail should be modelled to represent features less fortunate to the structures resistant features instead.

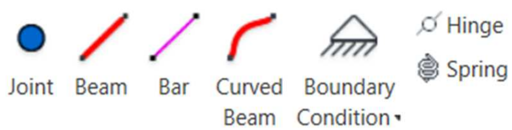


Figure 5.1. Focus modelling tools [21].

Figure 5.1 shows the options of tools available to create models in Focus. First step of modelling is to determine the position of the joints with respect to a global axis system (axis x and z when modeling in 2D, axis x, y, and z when modeling in 3D). When the joints are positioned, the next step is to define the beam- or bar elements dimensions as shown in Figure 5.2 and draw it from joint to joint. The joints position and chosen element dimensions may be modified afterwards.

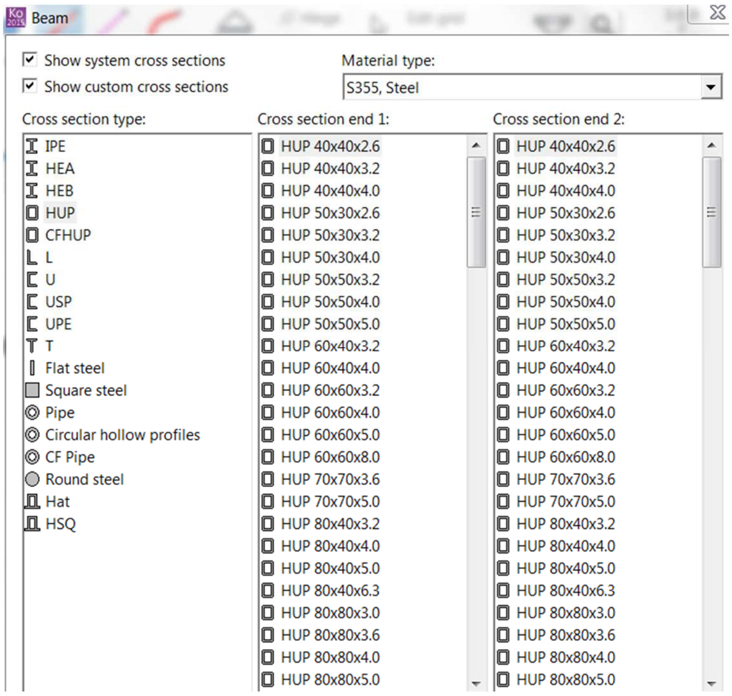


Figure 5.2. Options to define beam element dimensions [21].

When the elements dimensions and joint positions are defined, the next step is to define the boundary conditions. Important to set up the boundary conditions to represent the structures joint behavior and characteristics as realistic as possible.

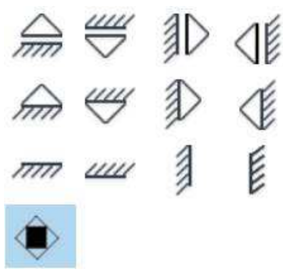


Figure 5.3. Options to define the boundary conditions [21].

Figure 5.3 shows the available options to define the boundary conditions in Focus. Among the options there are the typical pinned-, roller- and rigid conditions. In addition, there is a boundary condition called *special*. The special boundary conditions gives the user the possibility to define self-defined terms for the condition (e.g. pinned in one direction, while rigid in all other directions).

To set up boundary conditions, which represents the realistic case, may be tricky in some cases. E.g., it is not right to define the boundary condition at the bottom of a column as neither pinned nor rigid. However, with the option of special boundary condition it is possible to apply a spring stiffness to the boundary condition, which gives the ability to define a boundary condition with the characteristics of something between a pinned- and a rigid condition.

Last step of the modeling procedure is to define the amount- and type of hinges. Figure 5.4 shows the options available to define the hinges. The hinges need to be defined as either hinge or slider, and segments connected to the hinge has to be specified.

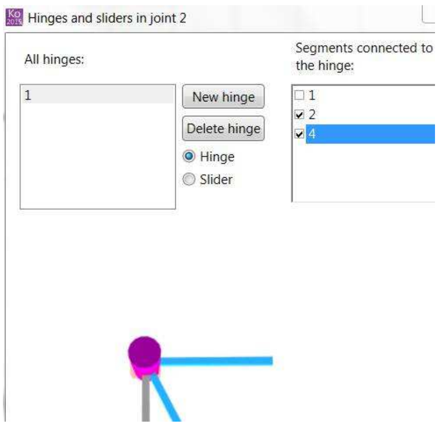


Figure 5.4. Options to define the amount and type of hinges [21].

If all steps discussed above are completed, the resulting theoretical model in Focus represents a practical structures design and characteristics. However, the dimensions of the elements are by now only a wild guess. To determine the most optimal dimensions to the elements of the model, a variety of analyses has to be performed. Before the analysis phase, however, the loads and masses (forces) acting on the structure have to be defined.

5.1.2 Load and load combinations

Before the analysis phase of the model, potential worst-case load scenarios have to be determined and how these load scenarios act when combined. The environmental loads, such as snow- and wind loads are assessed and determined according to Eurocode 1 part 1-3 [22] and Eurocode 1 part 1-4 [23], respectively.

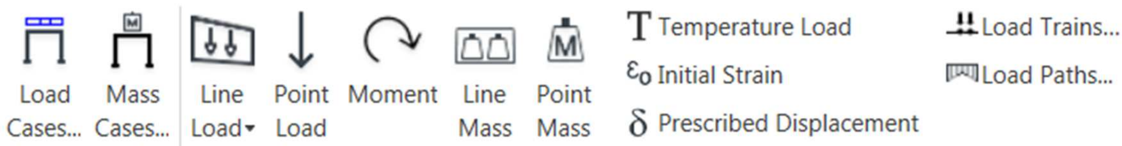


Figure 5.5. Options to define loads and masses [21].

Figure 5.5 shows the available tools to apply loads and masses to the model in Focus. Even though the software offers a variety of options to apply the loads to the model, the user has to calculate the loads manually before doing so. The software does offer *wizards*, which help define load cases and load combinations (further discussed later in this chapter).

There are various software products, which simplify the manual calculation process to determine loads. For the practical example, a software called *Ove Sletten Lastberegning* [24] is used to calculate the design wind loads based on recommended guidelines and design factors given in Eurocode 1 Part 1-4 [23]. A summary of the results from *OS Lastberegning* is given in Appendix B.2. Translation of the design forces from the software (3D) to fit the model in Focus (2D) is done afterwards in Mathcad.

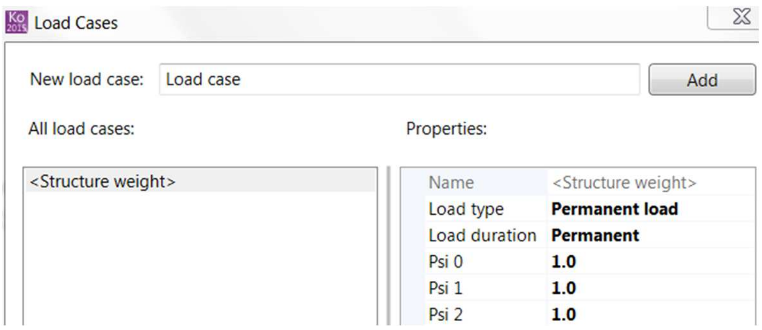


Figure 5.6. Options to define load cases [21].

Figure 5.6 shows the available options to define (or label) the specific load cases. Before the loads are applied to the model, it is important to label the load and define what type of load it represents (e.g. if it is a permanent- or variable load). This process will simplify the option to tell the software, which loads should act together, and which should not, when running the analysis (going through the code checks from the Eurocodes).

Figure 5.7. Wizard options for simplification [21].

As mentioned earlier in this chapter, Focus offers some helping wizards, as shown in Figure 5.7. The first wizard helps define snow loads acting on the roof. This is done with respect to the structures geographic location based on the characteristic values for that location, according to recommendations given in Eurocode 1 Part 1-3 [22]. The roofs geometrical characteristics has an impact on the end value for the load (e.g. if the roof is designed in a way that the snow is able to pile up at certain places).

The second wizard helps define wind loads acting on the roof, which is done in accordance to recommended characteristic values and design factors given in Eurocode 1 Part 1-4 [23]. Location of the structure gives characteristic values of loads, which afterwards are positioned in specific areas on the roof with the respective design factors. The zones are defined based to the models geometry.

Excluding load cases

In the table below you can mark the load cases that should not act simultaneously on the model

	Vindlast DogE	Vindlast AogB	Skjevstillingslast	Utilsiktet Torsjon
Snølast	☐	☐	☐	☐
Vindlast DogE		☐	☐	☐
Vindlast AogB			☐	☐
Skjevstillingslast				☐

Figure 5.8. Load combination wizard [21].

The last wizard helps determine the load combinations. This part is important because not all loads that are applied to the model should act at the same time. E.g., to check the beams and wind bracings (in a ULS analysis), a point load on top of the column (perpendicular to the column) may be set to represent the wind loads. Simultaneously, the same wind load is set up as an evenly distributed load along the column to be able to check the columns max deflection (in a SLS analysis). In other words, the wizard is a simple tool to determine the right combinations where the user has to tell the software what load cases should not act simultaneously when running the analysis, as shown in Figure 5.8.

5.1.3 Mass and mass combinations

When the steps discussed in the two previous chapters are completed, the model is ready for linear- and nonlinear analysis within ultimate limit state (ULS) and serviceability limit state (SLS). However, to perform vibration- and response spectrum analysis, masses and the mass combinations have to be defined.

First step of this process is to define (or label) mass cases as done for the loads, discussed in chapter 5.1.2. Reason to define the various masses is to later be able to determine the right mass combinations and how much of the mass is active during vibration- and response spectrum analyses of the structure.

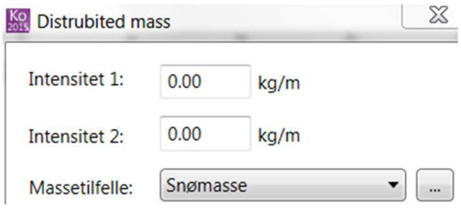


Figure 5.9. Options to define distributed mass [21].

When the mass cases are defined, the various mass value have to be determined manually (except structures mass). There are options to apply the masses as line mass or point mass. As for the loads, it is important to apply the theoretical masses to the theoretical model to fit the real masses for the practical model.

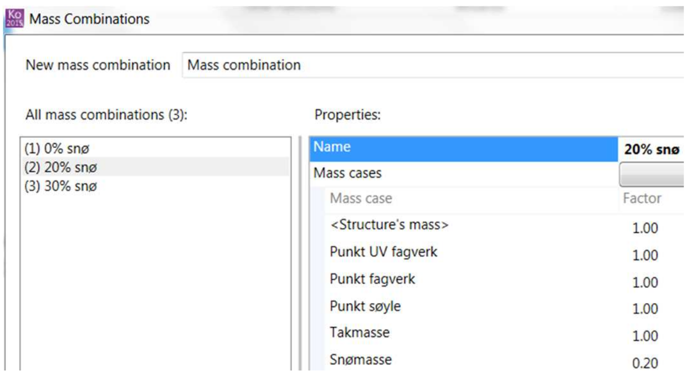


Figure 5.10. Options to define mass combinations [21].

When the mass cases are defined and applied to the model, the mass combinations have to be determined. Figure 5.10 shows the options to define the mass combinations in Focus. This gives the user the ability to check how the structure acts when extra masses from e.g. a crane is considered and how it acts when not.

5.1.4 Analyses and results

When all steps discussed in the three previous chapters are completed, the model is finally ready for analysis. Focus offers various types of analysis, as shown in Figure 5.11. For the thesis, *linear analyses* (ULS) are performed to determine the resulting forces in the elements caused by wind- and snow actions. *Vibration analyses* are performed to determine the structures stiffness characteristics for diverse scenarios, and *response spectrum analyses* are performed to determine the resulting forces in the elements caused by the earthquake actions.

Figure 5.11. Options for analysis [21].

Before running vibration analyses, definition of what the models stiffness is based on has to be set. The options are to base it on the materials stiffness characteristics only, or to base it on the material- and geometric stiffness characteristics.

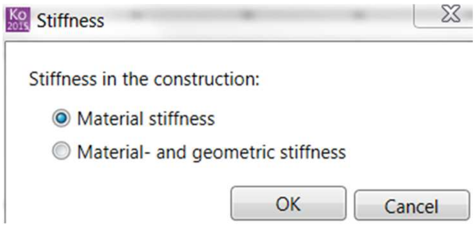


Figure 5.12. Options to define basis for stiffness [21].

Response spectrum analysis requires a defined response spectrum. There are no built-in wizards to define the spectrum, so it has to be defined manually. In cases where the ground type is categorized A, B, C, D or E, the values suggested in Eurocode 8 [5] can be applied to determine the response spectrum. For special cases, where the ground type is categorized S_1 or S_2, the values need to be determined by geotechnical engineers who evaluate the specific ground type. An example for ground evaluation, done by Multiconsult, is found in Appendix B.1.

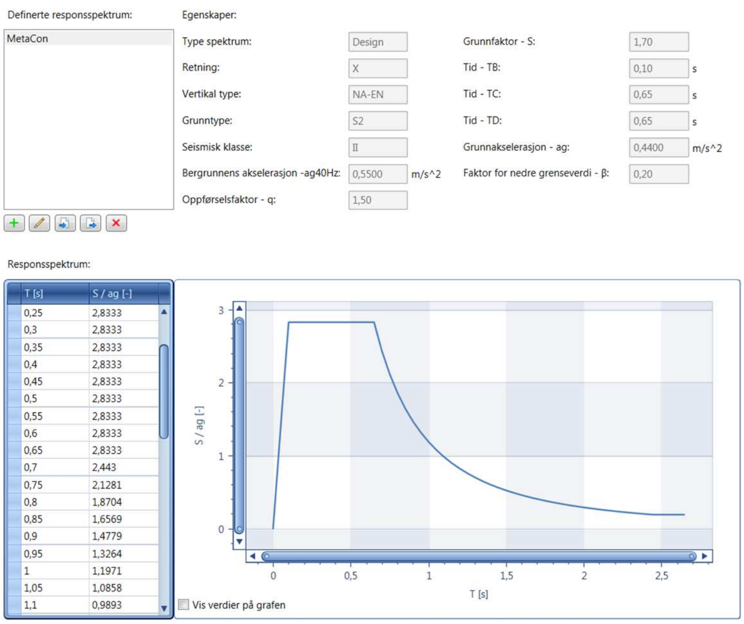


Figure 5.13. Defining response spectrums [21].

When the response spectrum is defined, and all other steps discussed above are completed, the theoretical model in Focus becomes ready for vibration- and response spectrum analyses. There are several ways to present the results, both graphically and numeric. Figure 5.14 shows some of the options to present the resulting values from the analyses.

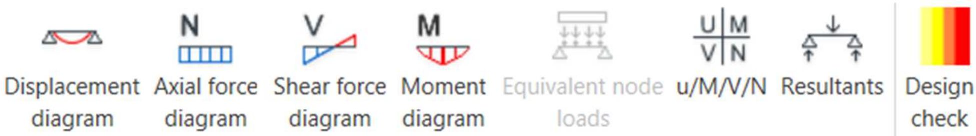


Figure 5.14. Options for presentation of results [21].

The max values of displacements, axial forces, moment forces, etc, are given in a table next to the model. In addition, the user can hoover over the elements in the model to get the values at the exact positions. Utilization of the elements are presented with a color-grading method going from white to yellow to red, where red indicates that the utilization factor is above 1.0.

In cases where it is seen that the elements dimensions chosen in advance do not meet the criteria from the code checks done by the analysis, it is possible to go back in the model and modify the elements dimensions to utilize the elements as much as reasonably possible.

5.1.5 Verification of use of software

Commercial software products often have a various set of settings and options to better fit its purpose based on the users' preferences and priorities. Because of this, it is important to do some control calculations to see if the results from hand calculations (manual calculations) are similar to the results from the software.

To simplify control procedure, a simple cantilever column with the boundary condition set to rigid is modelled and analyzed in Focus, followed by hand calculations set up in Mathcad (Appendix A.2) of the same system. Assessment of the software is based on the results from software compared to the results from hand calculations.

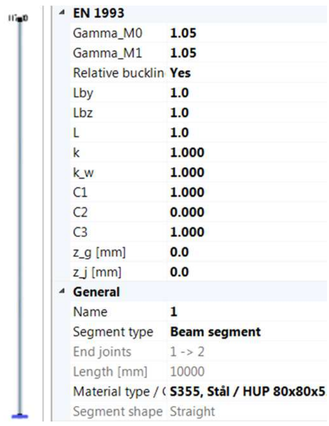


Figure 5.15. Focus model of rigid cantilever beam [21].

First step of the verification is to calculate the response spectrum manually and compare it to how the software calculates the spectrum based on the same characteristic values. This calculation example is based on the characteristic values for the specific ground type used to define *Metacon spectrum*, which will be used for response spectrum analyses in the practical example (chapter 7).

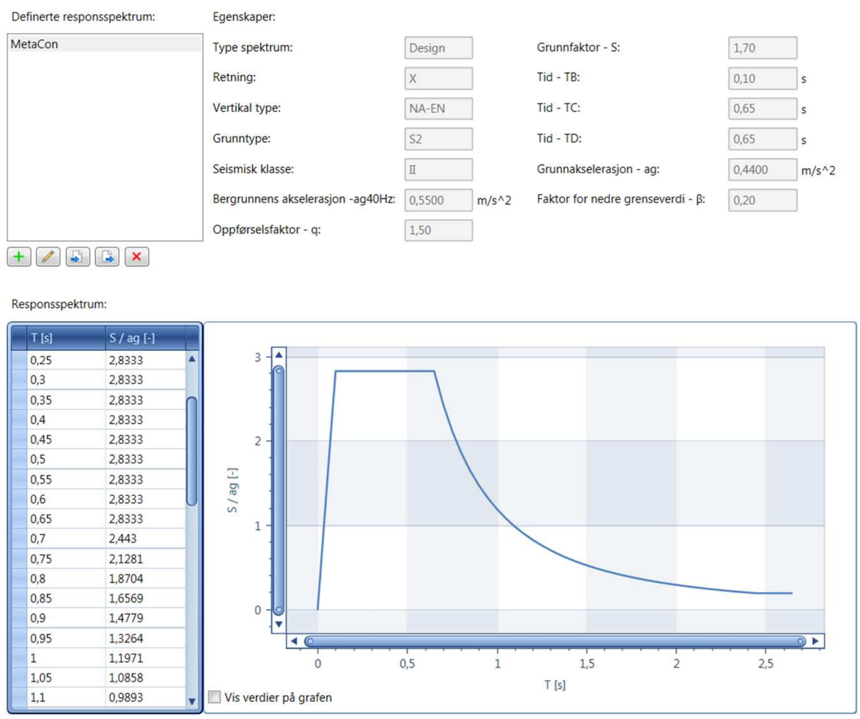


Figure 5.16. Focus results for Metacon defined response spectrum [21].

For this thesis, a calculation tool to check the relation between stiffness and EQ force is made in Mathcad Prime 3.0 (Appendix A.2) on the basis of procedure and equations given in Eurocode 8 [5]. The same calculation tool is used for this calculations-check example.

Ground type S2 · Seismic class II

$S := 1.7 \quad T_B := 0.10\ s \quad T_C := 0.65\ s \quad T_D := 0.65\ s \quad \gamma_1 := 1.0$

$a_{g40Hz} := 0.55\ \frac{m}{s^2} \quad a_{gR} := 0.8 \cdot a_{g40Hz} = 0.44\ \frac{m}{s^2} \quad a_g := a_{gR} \cdot \gamma_1 = 0.44\ \frac{m}{s^2}$

$\beta := 0.2 \quad q := 1.5$

$T(\varphi) := 1\ s$

$$S_d(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T(\varphi)}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T(\varphi) \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T(\varphi)^2}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$S_d(\varphi) = 0.527\ \frac{m}{s^2}$

$\boxed{\frac{S_d(\varphi)}{a_g} = 1.1971}$

Figure 5.17. Screenshot of calculation tool (Appendix A.2).

The table of values for the response spectrum defined by Focus (Figure 5.16) shows that at $T = 1.0s$, the value of ${}^{S_d}/_{a_g} = 1.1971$. Hand-calculated values as presented in Figure 5.17, shows the exact same value for ${}^{S_d}/_{a_g}$ when $T = 1.0s$. Thus, the response spectrum defined by Focus is same as the response spectrum defined by hand calculations.

Next step of the verification is to calculate the systems eigenfrequency (chapter 2.2) and compare it to the results of vibration analysis in Focus. From the frequency, it is possible to calculate the systems stiffness characteristics. Figure 5.18 shows the results from the vibration analysis done in Focus.

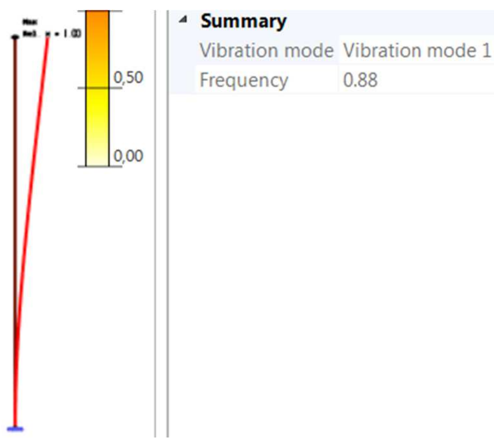
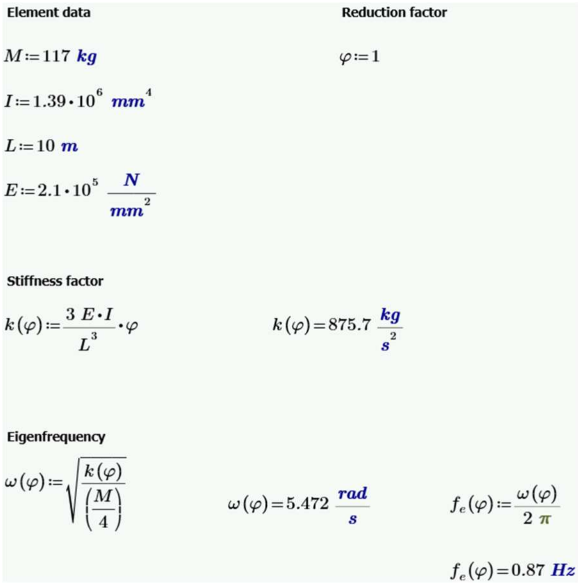


Figure 5.18. Vibration analysis of rigid column [21].

To calculate the eigenfrequency of the system manually, the calculation tool presented in Appendix A.2 is used. Main purpose of the calculation tool is to check how the resulting forces from the earthquake actions react to reduction the systems stiffness. Thus, there is a reduction factor φ, which is set as a variable between 0.1 and 1 ($\varphi \in [0.1,1]$). To calculate the systems eigenfrequency for this specific case, the reduction factor is removed (set equal to 1).



Element data

$M := 117\ kg$

$I := 1.39 \cdot 10^{6}\ mm^{4}$

$L := 10\ m$

$E := 2.1 \cdot 10^{5}\ \frac{N}{mm^{2}}$

Reduction factor

$\varphi := 1$

Stiffness factor

$k(\varphi) := \frac{3\ E \cdot I}{L^{3}} \cdot \varphi \qquad k(\varphi) = 875.7\ \frac{kg}{s^{2}}$

Eigenfrequency

$\omega(\varphi) := \sqrt{\frac{k(\varphi)}{\left(\frac{M}{4}\right)}} \qquad \omega(\varphi) = 5.472\ \frac{rad}{s} \qquad f_e(\varphi) := \frac{\omega(\varphi)}{2\ \pi}$

$f_e(\varphi) = 0.87\ Hz$

Figure 5.19. Screenshot of calculation tool (Appendix A.2).

Results from the vibration analysis done in Focus gives a value for eigenfrequency $f_e = 0.88Hz$, as shown in Figure 5.18. The results from the calculation tool as shown in Figure 5.19, gives a value for eigenfrequency $f_e = 0.87Hz$, which is not exactly the same value. Note that in the calculation tool it is assumed that $^1/_4$ of the columns mass is active during the swing motion (see chapter 2.2.2).

The mass-assumption simplifies the procedure to hand calculate the eigenfrequency of swinging columns. Based on the results of values, the mass-assumption gives the system lower stiffness characteristics. The resulting value from the software gives the structure higher stiffness, which is less desirable considering earthquake actions and is thus a safer approach.

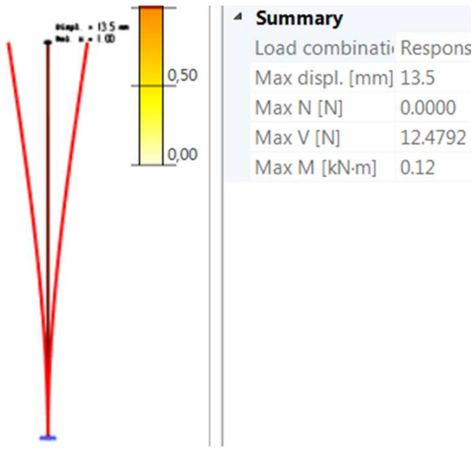


Figure 5.20. Response spectrum analysis of rigid column [21].

From the response spectrum analysis performed in Focus, the results given for max displacement $\delta = 13.5mm$ and max shear force $F = 12.48N$, as shown in Figure 5.20. The two figures below (Figure 5.21 and Figure 5.22), show the results from hand calculations of max displacement and max shear force.

Max displacement on top of column

$$\delta_{max}(\varphi) := \frac{S_d(\varphi)}{\omega(\varphi)^2} \qquad \delta_{max}(\varphi) = 13.342\ mm$$

Figure 5.21. Result of max displacement by hand calculation (Appendix A.2).

Max shearforce on foundation, assuming 2 or less storey building

$$F_{max}(\varphi) := S_d(\varphi) \cdot \frac{M}{4} \qquad F_{max}(\varphi) = 11.683\ N$$

Figure 5.22. Results of max force by hand calculation (Appendix A.2).

From the calculation tool, the result for max displacement is given as $\delta = 13.34mm$ and the result for max shear force is given as $F = 11.683N$. The small difference in the values is because of the swinging mass assumption (chapter 2.2.2).

Table 5.1. Summary of test-analysis results.

Method and case	f_e [Hz]	δ [mm]	F [N]
Focus 10m SHS 80x5	0,88	13,50	12,48
Hand calculation 10m SHS 80x5	0,87	13,34	11,68

6 Modification of dynamic characteristics

Within the analyses phase of the practical example (chapter 7.2), some modifications are done to the theoretical model created in Focus (Figure 7.4) to modify the dynamic characteristics of the model. This chapter covers a discussion- and a presentation of suggestions, to how these theoretical modifications may be applied to a practical example of a structure.

Reason to modify structures dynamic characteristics is based on the assumption that when reducing the stiffness of a structure, it will result in giving the structure more suitable properties when considering seismic actions. However, the new properties by reducing the system stiffness may not be suitable when considering actions from frequently occurring environmental loads, such as wind- and snow loads. With the use of *sacrificial elements*, it is intended that it should be possible to take benefit of reducing the systems stiffness when considering seismic actions, without the drawback of making it less suitable for the more frequently occurring environmental loads.

6.1 Sacrificial elements

To explain the meaning, use and properties of a sacrificial element, it may be considered as a type of fuse. The word *fuse* has several meanings. Within the context of electrical systems, fuse is a devise used to protect the system against excessive current. Within hydraulics, fuse is a devise used to protect against sudden loss of fluid pressure [25]. Within the context of structural engineering, sacrificial elements act as a fuse to protect the structure against high seismic activities by initiating the designed modifications to the systems dynamic characteristics to be more suitable to withstand the resulting loads from the seismic action.

The use of sacrificial elements may not be suitable for every structural system. It depends on the systems design, its positioning and assumed exposure to loads. However, there are several forms of the sacrificial elements, which gives the method some flexibility of use when assessing its suitability. The variety of forms are discussed further in chapter 6.2.

First check of suitability is to determine whether or not the loads from seismic actions are larger compared to the loads from the frequently occurring design environmental loads, such as wind- and snow loads ($F_{EQ} \gg F_{Ed}$). There are two reasons to why the earthquake load has to be greater than the environmental load. In case the environmental load would be greater than the earthquake load, modifications to the systems stiffness would not be needed, since the often-occurring loads would set the design basis for the structure. Earthquake load has to be greater than the environmental load so that the modifications to reduce the systems stiffness does not initiate unless the structure is exposed for an earthquake. The difference in value of force, when comparing often occurring environmental loads to earthquake loads, has to be relatively big. Reason for this requirement is based on uncertainties in the material.

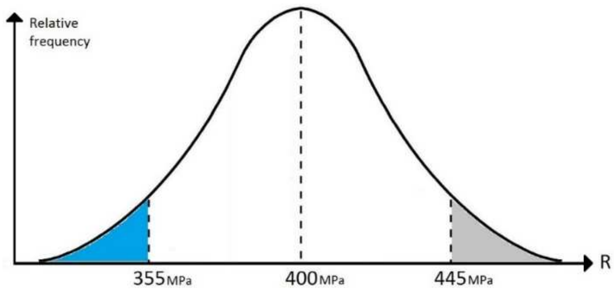


Figure 6.1. Relative frequency of yield strength from tests, based on [19].

Figure 6.1 shows a graph of a probability density function of structural steels (s355) yield strength, marked as R (resistance) in the graph. Values of yield strength shown in the graph above are only set up as an example and are not accurate realistically (an accurate example is shown in Figure 8.5). Even though steel might be considered a homogenous material compared to concrete or wood, there are uncertainties due to its microstructure, which needs to be take into account. By testing the material (e.g. tensile test) and assessing its properties, the resulting resistance (or yield strength) of the material may be presented as a normal distribution, as shown in Figure 6.1. Based on these test values, the steel is categorized by the peak of its 5% lowest values (marked with blue on the graph). Example of values in the graph represents a European standard stainless steel called s355, where the peak of its 5% lowest value is 355MPa. The proposed 5% gives an adequate safety margin for use of the material considering its yield strength is 355MPa. In other words, we can say that in 95% of the cases, the steels yield strength is equal to, or larger than 355Mpa.

As the name (sacrificial elements) states, the method of initiating the modifications of dynamic characteristic to the structure is based on certain elements failing. There are two major requirements for the sacrificial element:

- It has to withstand the loads caused by wind- and snow actions, so that the modifications of reducing the systems stiffness do not initiate when these loads are at its peak.
- Has to break (fail) due to the resulting loads from a potential earthquake, so that the modifications of reducing the system stiffness initiate and reduce the resulting loads caused by the earthquake.

Because this method is highly dependent on that the sacrificial element fails, the uncertainties of the material has to be taken into account. Thus, assessment of the sacrificial elements resistance has to be done based on; considering *lowest* yield strength (marked with blue in Figure 6.1) when checking against forces from wind and snow actions, and considering *highest* yield strength (marked with grey in Figure 6.1) when checking against forces from earthquake actions. In other words, the sacrificial element has to withstand the wind- and snow loads when at its weakest, but fail due to earthquake loads when at its strongest.

6.2 Situational characteristics

In case of an extreme event (earthquake, explosion, collision, etc), which either exposes the structure to large impact load or load that shakes the structure over relatively longer periods, it is favorable to either have a ductile structure or some type of damping element to absorb as much of the shock as possible. However, having a ductile structure is not favorable when exposed to high wind loads. Thus, the most optimal situation would be to have a structure with *situational characteristics*.

The idea of increasing robustness with the use of adaptive elements is to give the structure the ability to "*dance along"* when exposed to extreme events, by decreasing the structures level of stiffness at certain levels of load exposure, for a limited amount of time. Modifications are substantially focused on the wind-bracing system, since it is the part of the structure that contributes the most to the structures stiffness characteristics. During the following chapters, ideas of how to achieve and apply this adaptive ability to the structure in practice are discussed.

6.2.1 Smart elements

First idea discussed is the method of total removal of specific elements, which is used for the analyses of the practical example presented in chapter 7. In this thesis, these elements will be referred to as *smart elements*.

Visualize the resulting force on a structure from earthquake activity as one person frequently pushing another person. In case the person being pushed tries to stand still (be stiff), the resulting force on the person is relatively high. As a comparison, in case the person being pushed rather moves along with the pushing object, in the same direction, the resulting force on the person becomes noticeably reduced. Doing so, might be considered smart of the person. When all elements of a structure are intact, the structure is considered stiff. In case some of the elements are removed, reducing the stiffness and giving it the ability to move along, the structure acts as the smart person would do. Thus, the name, *smart elements*.

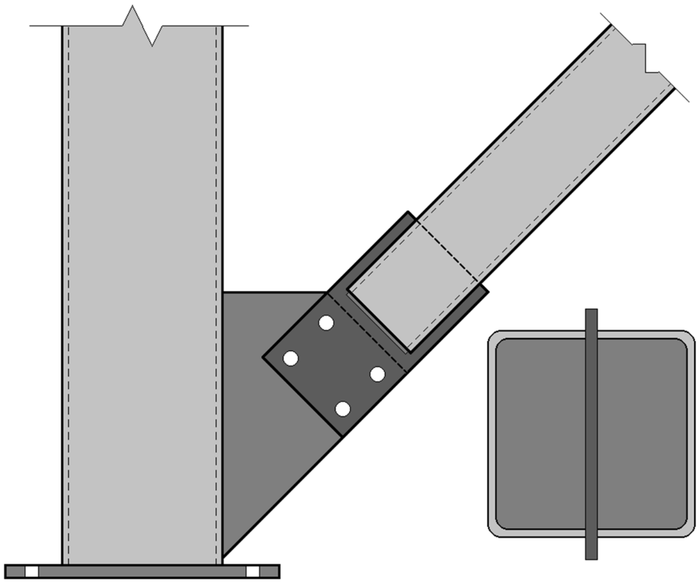


Figure 6.2. Simple illustration showing wind-bracing element connected to a column.

Figure 6.2 shows a connection between a wind-bracing element and a column. The two elements might either be connected by bolts (as shown in the figure) or by welding. When considering this method of applying situational characteristics to the structure, the connecting parts (bolts or welds) are the crucial parts. If these parts do not work as intended, the structure does not get the intended characteristics.

In chapter 8.2, the suitability of this method for the practical example presented in chapter 7 is discussed. One of the key factors to determine if it is suitable or not, is the difference in resulting force from wind- and snow actions, compared to the resulting force from earthquake actions. As discussed in the introduction of chapter 6.1, it is required to take uncertainty of material properties into account since the method is highly dependent on that the connecting parts *must* fail during the event of an earthquake.

Considering connection method between the elements as presented in Figure 6.2, the bolts set the crucial design criteria deciding whether the method is suitable or not. As a procedure of work, the first step is to determine the design shear force acting on the bolts due to wind- and show loads. Second step is to determine type and amount of bolts, to give an adequate resistance to withstand these loads when minimum value of yield strength is assumed. The resulting design of the bolts sets a boundary condition for the suitability check.

When the bolts are designed to withstand the resulting forces from wind- and snow loads in ultimate limit state with minimum value of yield strength, the next step is to determine if the resulting force from earthquake actions is large enough to break the bolts. The bolts *have* to break even when material uncertainties are taken in consideration (assuming maximum value of yield strength). Thus, the difference in force has a set minimum value, for this method to be suitable.

If it is possible to design the bolts to meet the requirements discussed above, we have only determined that it is possible to install this method to the structure, where the bolts are the parts initiating the modification of dynamic characteristics and resulting in giving the structure situational characteristics. However, we have not determined if the method is suitable for the structure yet. For the method to be suitable, we also have to determine if the modification of dynamic characteristics results in adequate reduction of the force. Since the method does remove some of the elements (smart elements), the structure has to be capable of withstanding the modified resulting forces from earthquake activity without considering contribution from the smart elements.

As an example, in case half of the elements are smart elements (being removed during an earthquake event), the resulting force from earthquake activity has to be reduced by more than 50%. More than 50%, because it is required to consider uncertainties of the material for the reasons discussed above. During the two following chapters, methods of applying situational characteristics without total removal of elements are discussed.

6.2.2 Mechanical dampers

Second idea discussed is the method to install a mechanical damper to the wind-bracing elements. Theoretically, the mechanical damper acts as an viscous damper to the system, as discussed in chapter 2.1.2, where the damping coefficient is determined by the characteristic damping ability of the mechanical damper.

There are many similarities between installing smart elements (as discussed in the previous chapter) and installing mechanical dampers to the wind-bracing element. By installation of the mechanical damper, the modifications to the structures dynamic characteristics are modified as discusses for the smart elements, with some additional benefits. There are two major additional benefit to this idea compared to the previously discussed idea:

- The system gets the ability of damping the vibration, resulting in greater absorption of the forces.
- Even though the wind-bracing elements characteristic properties are modified when the damper elements are initiated, the elements are not completely removed. Thus, it still has some contribution to the systems resistance to withstand the resulting forces from earthquake actions.

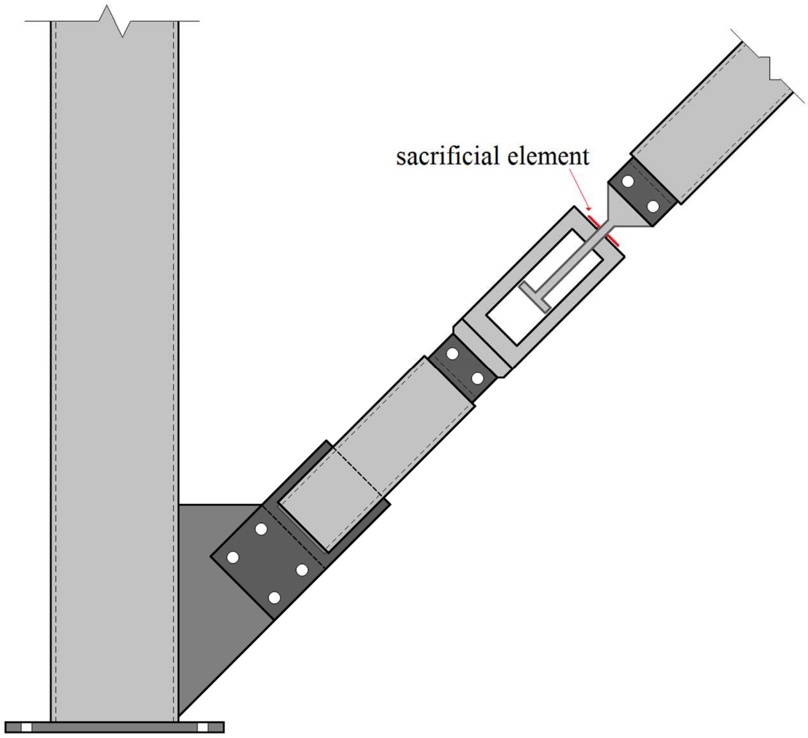


Figure 6.3. Simple illustration showing wind-bracing element with installed mechanical damper.

Figure 6.3 shows a wind-bracing element attached to a column. The mechanical damper is installed on the wind-bracing element, since this element is the main contributor to the systems stiffness characteristics (for horizontal motions). As mentioned earlier the stiffness (or dynamic characteristics) of the structure has to be reduced (modified) during, and only during, the time of a potential earthquake action. Thus, the installed damper has to be inactive, until the modifications to the dynamic characteristics are intended and needed.

A noticeably difference from this idea compared to the previously discussed idea is the method of how the modification of dynamic characteristics is initiated. For the previous idea, either the bolts or the welds connecting the wind-bracing element to the column, is designed to break when the modifications are intended to happen, resulting in the element being completely removed. For this idea, a steel plate is used as an element locking the damper. Basis of the method is to have a sacrificial element (steel plate) locking the damper until exposed to a certain degree of force (resulting earthquake force).

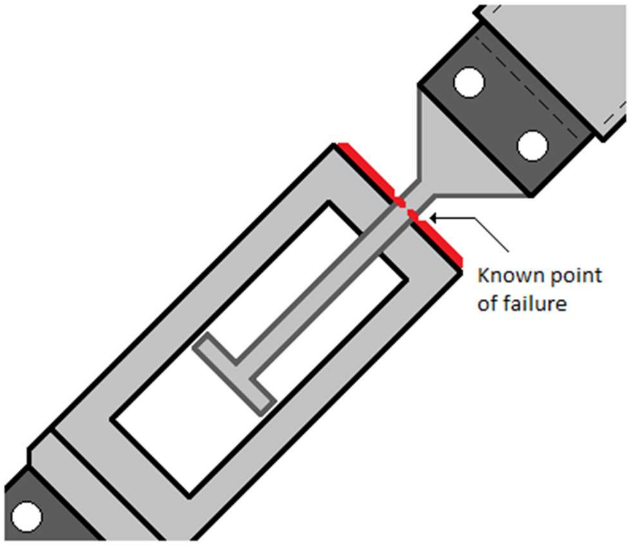


Figure 6.4. Illustration of suggested sacrificial element for dampers.

Figure 6.4 shows a suggested method of design to the sacrificial element locking the mechanical damper. Although, it is possible to lock the damper by two fillet welds as the crucial initiation pars, it is more functional to use welded steel plates and gives a wider flexibility of customization for specific practical examples.

Red element shown in Figure 6.4 represents the sacrificial steel plate. The small *cuts* in the steel plate next to the dampers bar are intended and serve an important feature. Reduced area of steel results in less resistance and potentially higher stress concentration, which is not favorable for an element with the purpose to withstand forces. However, the plate in this case is a sacrificial element intended to break at certain levels of load exposure. The benefits of the cut is that the designing engineers has the option to position the point of failure where it is most favorable.

Considering sacrificial element as shown in Figure 6.4, the steel plates set the crucial design criteria deciding whether the method is suitable or not. As a procedure of work, the first step is to determine the design shear force acting on the steel plates at its weakest points (location of the cuts) due to wind- and snow loads. Second step is to determine thickness of the plates and cuts, to give an adequate resistance to withstand these loads when minimum value of yield strength is assumed. The resulting design of the plates and cuts sets a boundary condition for the suitability check.

When the plates are designed to withstand the resulting forces from wind- and snow loads in ultimate limit state with minimum value of yield strength, the next step is to determine if the resulting force from earthquake actions is large enough to break the plates at the location of the cuts. The plates *have* to break even when material uncertainties are taken in consideration (assuming maximum value of yield strength). Thus, the difference in force has a set minimum value, for this method to be suitable.

If it is possible to design the steel plates to meet the requirements discussed above, we have only determined that it is possible to install this method to the structure, where the steel plates are the parts initiating the modification of dynamic characteristics and resulting in giving the structure situational characteristics. However, we have not determined if the method is suitable for the structure yet. For the method to be suitable, we also have to determine if the modification of dynamic characteristics results in adequate reduction of the force.

In the previously discussed idea with smart elements, the structure had to be capable of withstanding the modified resulting forces from earthquake actions without considering contribution from the smart elements. One of the major differences between the two ideas is that for the idea discussed in this chapter, the structure may rely on the contribution from the elements with dampers installed even after the initiated modification. Thus, the resulting force from earthquake actions do not have to be reduced more than 50%.

6.2.3 Ductile elements

The last idea discussed is the method to design wind-bracing systems in a way, which gives the system a certain degree of ductility. Ductile is defined as “easily drawn into wire or hammered thin, molded or shaped” [26]. The previously discussed ideas are based on the sacrificial element breaking. This idea on the other hand, is based on allowing the sacrificial element to deform within both the elastic- and plastic area of the materials properties.

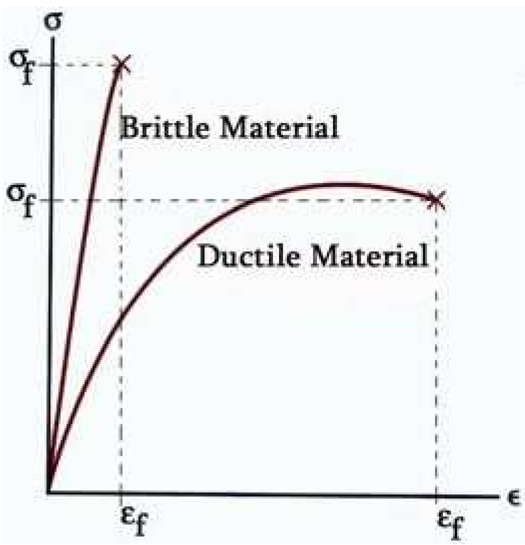


Figure 6.5. Stress-strain diagram of brittle- and ductile material [27].

Figure 6.5 shows the different properties of a brittle material compared to a ductile material with a stress-strain ($\sigma - \varepsilon$) diagram. A brittle material (e.g. a type of steel with high yield strength) has high resistance levels, but is not able to deform much until it fractures. A ductile material (e.g. a type of steel with low yield strength) has lower levels of resistance, but allows the material to deform more before it fractures.

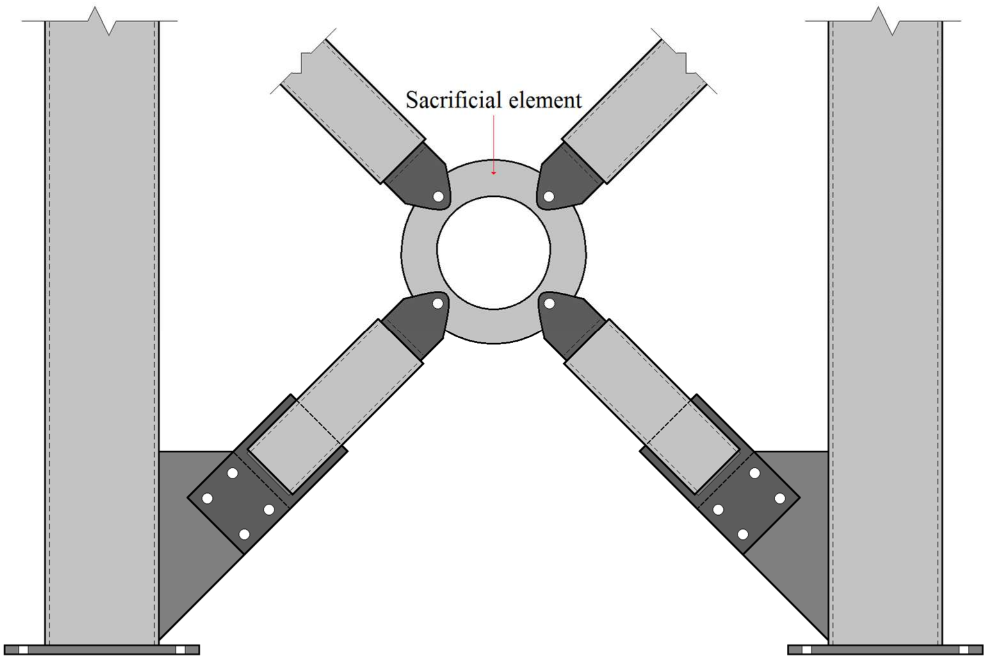


Figure 6.6. Simple illustration showing a form of adaptive wind-bracing element.

Figure 6.6 shows a suggested design of wind-bracing system with an installed ductile sacrificial element. The design is based on that the sacrificial element is able to transmit loads between the elements within limitations of elastic deformations, when exposed to often occurring environmental loads, such as wind- and snow loads. As mentioned in the introduction to this chapter, this type of sacrificial element is not designed to break during the event of earthquake actions. The element is designed to sacrifice itself in the form of deforming within limitations of plastic (permanent) deformation and in the process act as a type of damper.

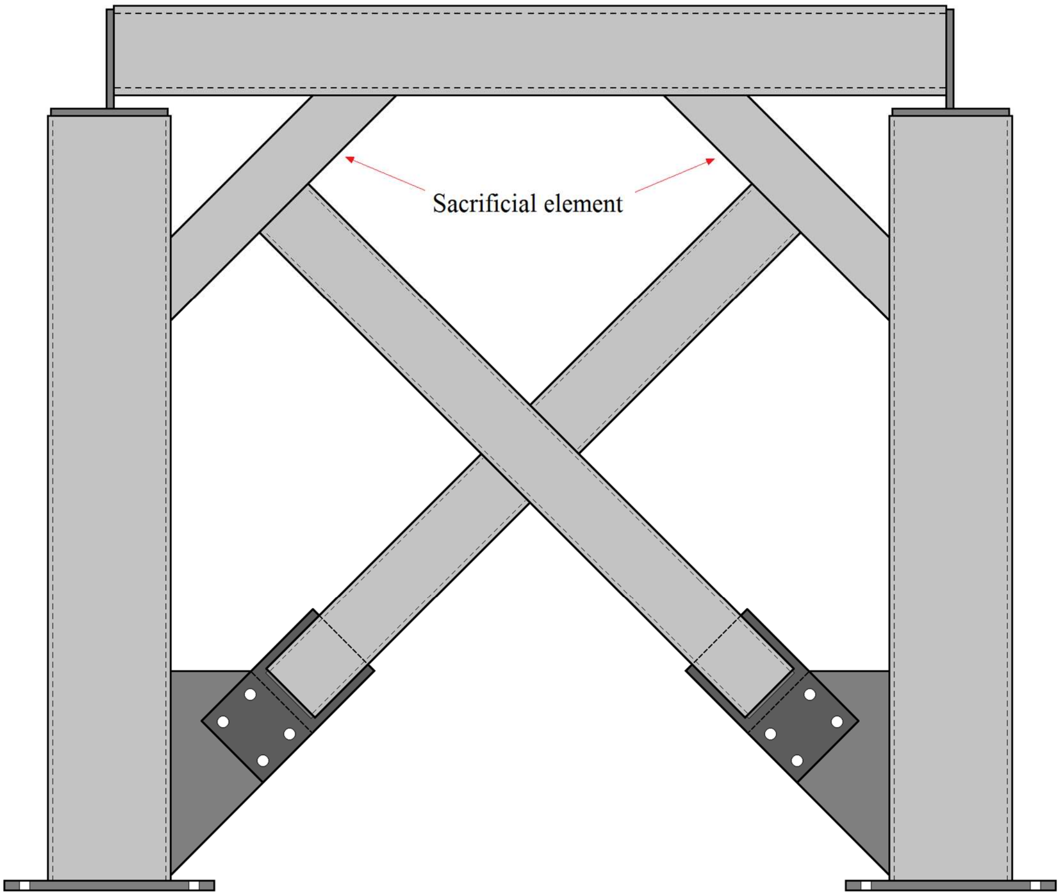


Figure 6.7. Simple illustration showing a form of adaptive wind-bracing element, based on [28].

Figure 6.7 shows an alternative method of installing ductile sacrificial elements to the wind-bracing system, based on ideas discussed in the book *Earthquake design practice for buildings* by Booth & Key [28]. This method is based on the same principles as the previously discussed method, but instead of applying the sacrificial element to the wind-bracing element itself, the sacrificial elements are added as additional element in the top corners of the frame. Assessment procedure of the sacrificial elements presented in Figure 6.7 is relatively easier compared to the relatively complex assessment of the circular sacrificial element shown in Figure 6.6.

The sacrificial elements design criteria is the maximum bending due to the resulting point load transmitted by the wind-bracing elements. Maximum bending calculated on the basis of allowing the element to deform within the area of plastic (permanent) deformation.

7 Practical example of existing structure

In the previous chapters, the benefits and methods of increasing the robustness level of a structure have been discussed, but only theoretically. Applying theoretical methods to practical examples is the art of engineering, or as Dr. A. R. Dykes described his engineering philosophy in his 1946 Chairman's address to the Scottish Branch of the Institution of Structural Engineers:

> *"Structural engineering is the art of modelling materials we do not wholly understand into shapes we cannot precisely analyze so as to withstand forces we cannot properly assess in such a way that the public at large has no reason to suspect the extent of our ignorance"* [29].

This chapter presents an attempt of applying the discussed methods to increase robustness to an existing structure. In cooperation with the company *Metacon AS*, the external supervisor for the thesis, a relatively high structure placed on potentially liquefiable soil is chosen as the practical example. Geotechnical engineers have categorized the soil as *ground type* S_2. Ground type S_2, is defined as *"Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or* S_1*"* [5]. Evaluation of the ground type will be further discussed in chapter 7.1.1.

7.1 Project 14-112

Structure analyzed for this thesis is a project from Metacon, called *project 14-112*, which is under construction simultaneously as the thesis is written. The structures has a characteristic rectangular geometry, roughly 85 meters long (along axis C), 65 meters wide (along axis 15) and 22 meters high. It is designed to be used for storage by a brewery in Sarpsborg, about 90km away from Oslo. Figure 7.1 shows a screenshot of the TEKLA model of the structure.

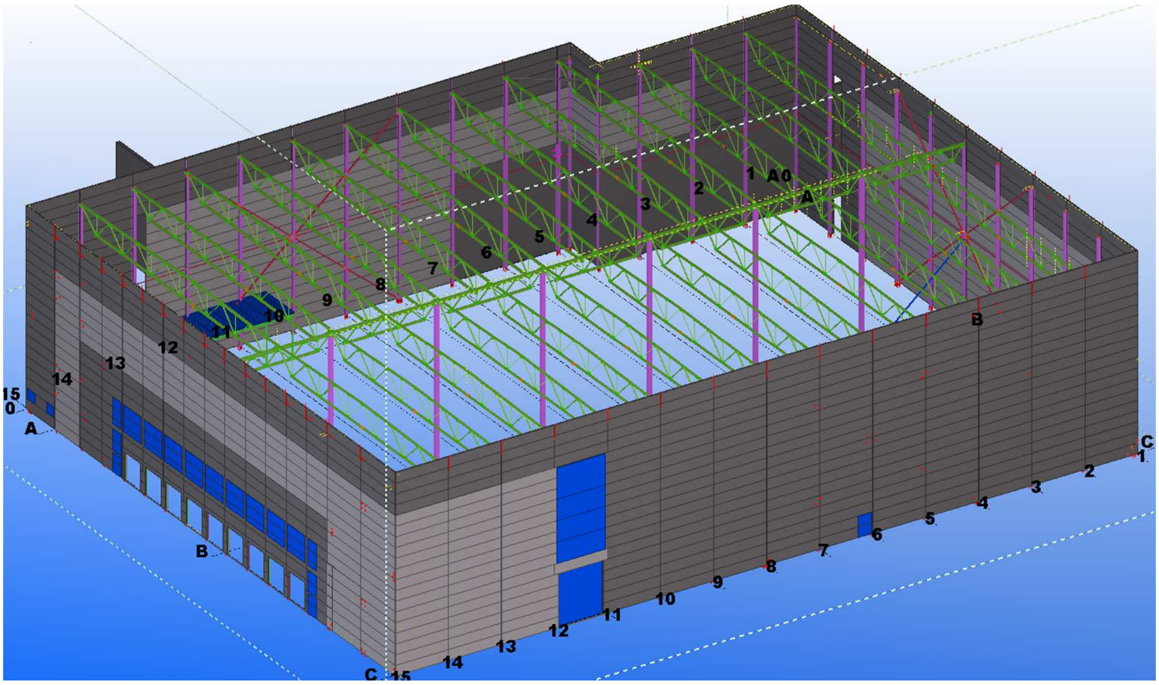


Figure 7.1. TEKLA model of project 14-112.

As seen in Figure 7.1, there are large surfaces exposed to wind, which result in high wind loads. The structures wind-bracings system have a design criterion to withstand these loads. High loads result in large dimensions on the wind-bracing elements, which thus also increase the structures stiffness. As discussed previously, high stiffness will result in higher resulting forces from seismic actions (chapter 2.2.1). Some of the major steps of assessment to the analyses of the structure is to compare loads due to wind- and snow actions with loads due to earthquake actions, and check the level of reduction of the resulting forces due to earthquake actions.

7.1.1 Define response spectrum

Evaluation of the ground type is done by *Multiconsult AS*, in accordance to Eurocode 8 [5]. Eurocode 8 has recommended values for the reinforcement factor S, and response spectrum for ground type A to E. For this specific case, ground type S_2 is being evaluated, since the structure is placed on sensitive clay. Sensitive clay (and especially quick clay) is always defined as ground type S_2. Ground type S_2, is defined as *"Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1"* [5].

Since the ground type is defined as S_2, in accordance to Eurocode 8, special studies are required to determine the seismic influence for the ground type. This requirement is localized under point 3.1.2(4) in the Norwegian national annex, saying: "*For sites with ground conditions matching either one of the two special ground types S_1 or S_2, special studies for the definition of the seismic action are required. For these types, and particularly for S_2, the possibility of soil failure under the seismic action shall be taken into account*" [5].

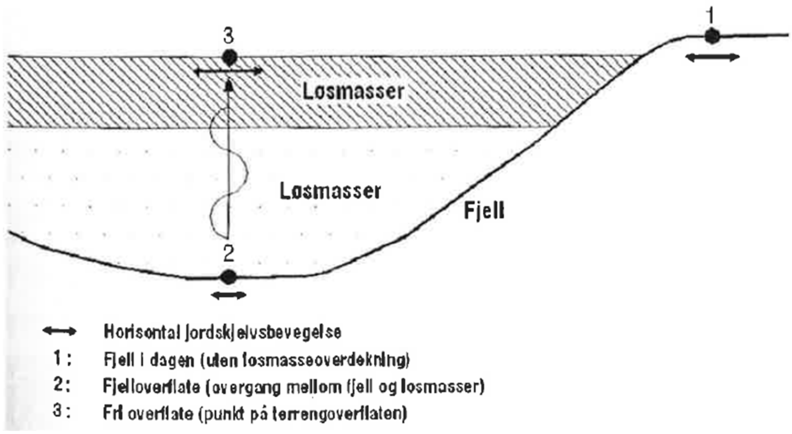


Figure 7.2. EERA example of ground response analysis (Appendix B.1).

To evaluate the seismic influence of the structure, a ground response analysis is done. The analysis is done using EERA, a computer program for equivalent-linear earthquake site response analyses of layered soil deposits.

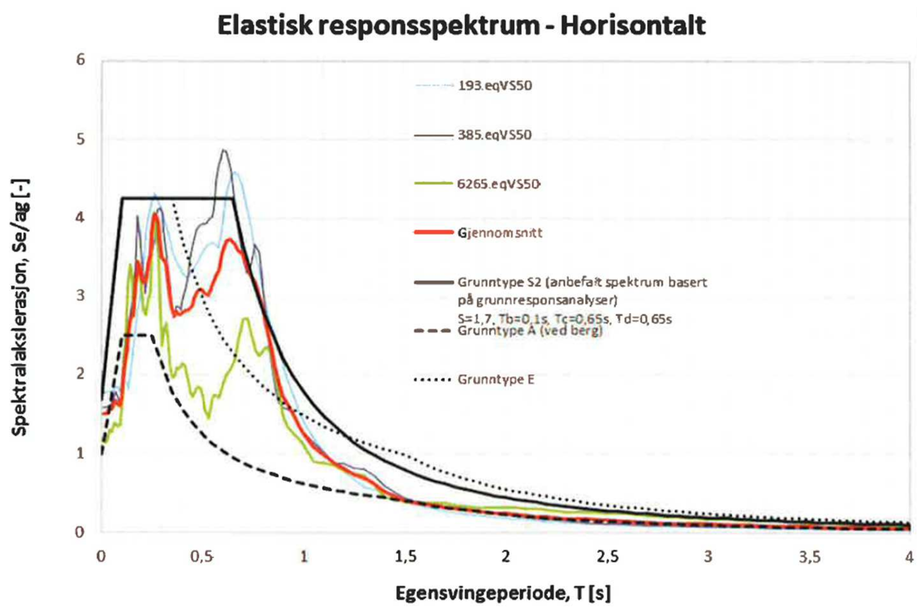


Figure 7.3. Graph representing the response spectrum defined (Appendix B.1).

This chapter only covers a brief part of the procedure for evaluation of the ground type and defining the representative response spectrum. Full procedure of evaluation of the ground type and defining the resulting response spectrum can be found in Appendix C.1.

Table 7.1. Values for parameters defining the response spectrum.

Ground type	S [-]	T_B [s]	T_C [s]	T_D [s]
S_2	1,7	0,10	0,65	0,65

7.1.2 Focus model

Before the analysis process, it is important to set up a good Focus model, which represents the practical example as realistic as possible. The procedure to set up the model is done as discussed in chapter 5.1. Lengths and dimensions of the elements are based on the TEKLA model from Metacon (Figure 7.1). Environmental loads are defined based on the structures location and the guidelines given in the Eurocodes.

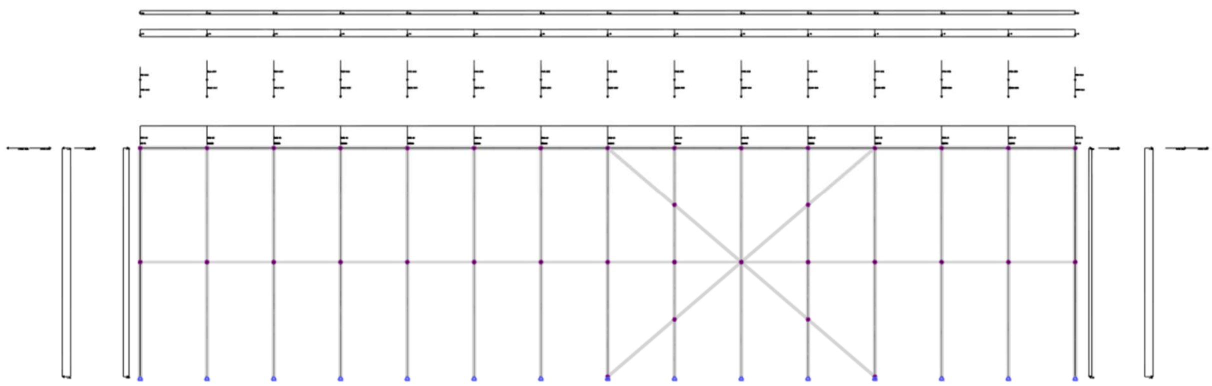


Figure 7.4. Focus model of Project 14-112.

Since it was planned to assess the structure with three different mass combinations, two different response spectrums and two different heights of the structure, it was assumed that it would take many runs of analyses to be able to assess every modification. Based on the assumption, the analyses are performed in 2D. Thus, a model of only one axis of the structure is set up, but still taking into account the masses from half the building. The reason for this is to avoid a lot of *downtime*, while the computer is running the analyses. Computers run time for each analysis is by far less for a 2D model, compared to a full 3D model. The results from a 2D analysis may of course not be as precise as from a 3D model, but the conservative method will give a decent representation of the actual result.

One of the long walls (axis C in Figure 7.1) will be analyzed in chapter 7.2. The reason for this is to get a case where the loads from the seismic actions (earthquake) is higher than the loads from the environmental actions (wind and snow). If so, adjustments and modifications to the structures stiffness characteristics will be applied to make the wind- and snow loads the main contributor to set the design criterion. The wind loads are lowest in the wind-bracings in axis C (compared to wind-bracings in axis 1 and 15), since they are resisting the loads from wind blowing on the short walls (the wind has less surface to hit, thus lower resulting force).

7.2 Analyses

Procedure for seismic analysis of project 14-112 is done as discussed in chapter 5. First analyses of the structure are done with original lengths, heights and dimensions considered, followed by analyses of the structure with some modifications, which change the structures stiffness characteristics.

Since the structure will be used as a storage unit, it is important to account for the additional masses from the items stored because they will contribute to the systems total mass, which results in contribution to the structures natural period (chapter 2.2). To account for the additional masses, every case is analyzed with three different mass combinations considered.

The first mass combination is set up as recommended in *Eurocode: Basis for structural design* [11]. Recommended combination of actions for seismic design situations is found under point 6.4.3.4. The recommendation of actions is presented with an equation (equation 6.12a in the Eurocode). General format of effects of actions should be

$$E_d = E\{G_{k,j}\ ; P\ ;\ A_{Ed}\ ;\ \psi_{2,i}\, Q_{k,i}\}\quad j \geq 1; i \geq 1 \tag{11.1}$$

Where the combination of actions in brackets { } can be expressed as (equation 6.12b in the Eurocode)

$$\sum_{j\geq 1}^{\cdot} G_{k,j} + P + A_{Ed} + \sum_{i\geq 1}^{\cdot} \psi_{2,i}\, Q_{k,i} \tag{11.2}$$

Where

E	is the effect of action (or action effects) on structural members, (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation, etc)
E_d	is the design value of action effects
$G_{k,j}$	is the characteristic value of permanent action j
P	is a relevant presentative value of a pre-stressing action
A_{Ed}	is the design value of seismic action
$\psi_{2,i}$	is the factor for quasi-permanent value of a variable action i
$Q_{k,i}$	is the characteristic value of the accompanying variable action i

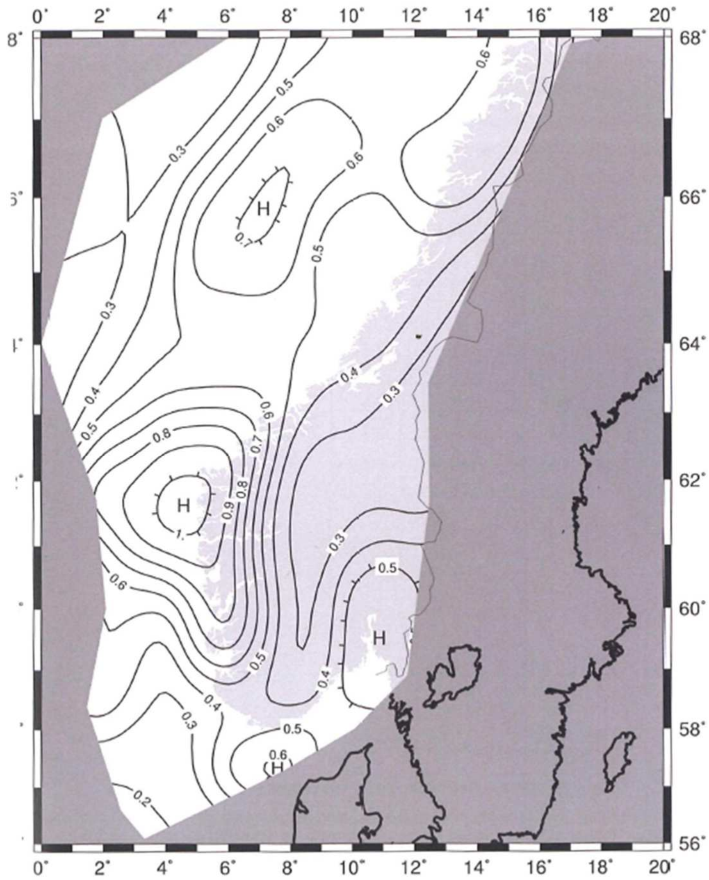
Point 6.4.4 refers to Eurocode 1, part 1-3 [22] for recommended values of ψ. According to table 4.1 in Eurocode 1, the recommended value for ψ_2, considering snow loads (variable load), is set to 0.2. Thus, for the first mass combination I have chosen to analyze the structure with 20% of the snow mass on the roof.

The second mass combination is set up to neglect the mass of the snow on the roof. Even though the Eurocode requires to account for 20% of the snow mass as a permanent load, it might be interesting to check how the structure would act in cases with no snow mass, e.g. during the summer time.

With a characteristic snow load of $2{,}4\, {}^{kN}/_{m^2} \approx 245\, {}^{kg}/_{m^2}$ for the specific location of the structure, the resulting total snow mass on the structure is $\sim 1354000\ kg$. For the simplified Focus model, half of this weight is considered ($677000\ kg$). Only 20% of the snow mass, as considered in the first mass combination, is thus $135400\ kg$. Neglecting the amount of mass is assumed to have a relatively big impact on the resulting swing motion of the structure.

For the third and last mass combination, the additional potential masses are considered. Added mass from units stored in the structure, eventual installations of cranes, etc. To simplify the step where additional masses are considered, a mass combination where the factor of snow mass on the roof set to 0.4 is created, i.e. considering 40% of the snow mass (additional $135400\ kg$). Even though the added mass by this method is placed on the roof, it should give a decent representation of the structures potential swing motion caused by the extra additional masses from the units stored.

In addition to the variety of mass combinations, the analyses are performed with two different sets of response spectrums. First run of analyses are done with the response spectrum called *Metacon*, defined for the specific ground type of the ground project 14-112 is located on (see chapter 7.1.1). For a comparison, an additional run of analyses are performed with another (worse case) response spectrum, called *Thesis*, defined according to recommendations given in Eurocode 8 [5].



Figur NA.3(901) – Seismiske soner i det sørlige Norge, a_{g40Hz} i m/s²

Figure 7.5. Seismic zones in southern part of Norway [5].

For the second response spectrum (called *Thesis*), the structures location is replaced from the eastern part of Norway to the western part of Norway (Figure 7.5). Thus, the peak ground acceleration with return period of 475 years[5] changes from $a_{g40Hz} = 0.5\, {}^{m}/_{s^2}$ to $a_{g40Hz} = 0.8\, {}^{m}/_{s^2}$. In addition, ground type E[6] and seismic class IV are assumed for the modified response spectrum, giving the corresponding values for soil factor and variety of periods, recommended in Eurocode 8 [5], which define the form of the response spectrum with respect to time. Table 7.2 shows the two response spectrums used during the analysis of the structure.

Table 7.2. Response spectrums used in analysis.

Response spectrum	Ground type	Seismic class	S [-]	T_B [s]	T_C [s]	T_D [s]
Metacon	S_2	II	1,7	0,10	0,65	0,65
Thesis	E	IV	1,55	0,15	0,40	1,60

[5] Assumed return period of seismic activity in Norway, according to Eurocode 8 [5].

[6] Ground type E defined as: "*A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with $v_s > 800\, {}^{m}/_{s}$.*" [5].

7.2.1 Test analysis

Before the model of the practical example (Figure 7.4) is analyzed and assessed, a simplified model of the frame is analyzed as a test, where the results from a vibration analysis in Focus are compared to results from hand calculations of a swinging frame.

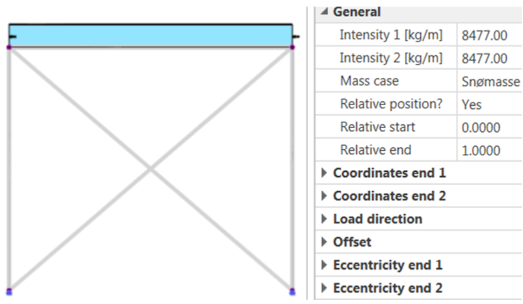


Figure 7.6. Focus model of simple frame for test analysis.

The frame shown in Figure 7.6 is a simplified representation of the wind-bracing system for the Focus model shown in Figure 7.4. Columns are 20.5m high and the distance between them is 24.0m. Roof- and snow masses are added to the model as discussed in chapter 7.2. For the vibration analysis, 20% of the added snow mass is considered.

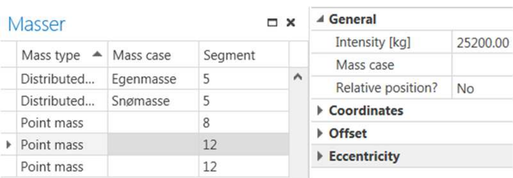


Figure 7.7. Equivalent point mass of structures mass.

The software shows its assumed equivalent masses for the structures self-weight as presented in Figure 7.7, showing one point load of 25200kg and two of 8400kg. These masses are considered for the hand calculation process.

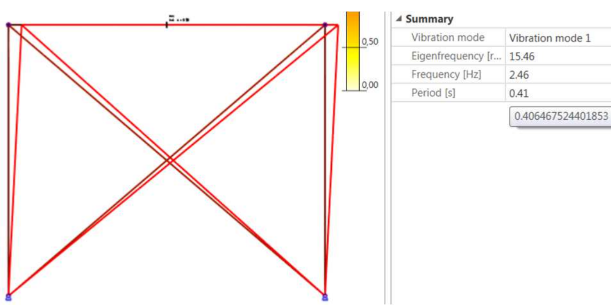


Figure 7.8. Results of vibration analysis for test frame.

Young's modulus for columns and the top beam are increased to $200 \cdot 10^5$[GPa] to get a desired vibration mode to fit the assumed vibration mode (SDOF) for the hand calculation example. Eigenperiod of the frame is $T_{n.f} = 0.406s$, according to the vibration analysis in Focus.

To simplify the hand calculation process for the test analysis, the same calculation tool (Mathcad Prime 3.0) introduced in chapter 2.2.1.2 is used, with minor modifications to fit this example. Stiffness characteristics of the frame are simplified by considering single degree of freedom and only take into account the contribution to stiffness from the wind-bracing elements, since every node in the model of the practical example are hinged (chapter 7.1.2). Thus, the columns and beams of the frame are assumed to be infinitely stiff. Complete calculation tool is found in Appendix A.3.

$$A := 40.04 \cdot 10^2\ \mathbf{mm}^2 \qquad E := 210\ \mathbf{GPa}$$

$$H := 20.5\ \mathbf{m} \qquad B := 24\ \mathbf{m}$$

$$L := \sqrt{H^2 + B^2} = 31.56\ \mathbf{m}$$

$$M_{test} := 2 \cdot 8400\ \mathbf{kg} + 25200\ \mathbf{kg} = 42000\ \mathbf{kg}$$

$$M_{snow} := 40689.6\ \mathbf{kg} \qquad M_{int.column} := 0\ \mathbf{kg} \qquad M_{truss} := 0\ \mathbf{kg} \qquad M_{beam} := 0\ \mathbf{kg}$$

$$M_{roof} := 84696\ \mathbf{kg} \qquad M_{column} := 0\ \mathbf{kg} \qquad M_{add.truss} := 0\ \mathbf{kg} \qquad M_{windb} := 0\ \mathbf{kg}$$

$$M := M_{snow} + M_{roof} + M_{int.column} + M_{column} + M_{truss} + M_{add.truss} + M_{beam} + M_{windb} = 125385.6\ \mathbf{kg}$$

$$\alpha_1 := \operatorname{atan}\left(\frac{H}{B}\right) = 0.707 \qquad K_1 := 2 \cdot \frac{A \cdot E}{L} \cdot \cos(\alpha_1)$$

$$T_{e1} := 2\ \pi \cdot \sqrt{\frac{M + M_{test}}{K_1}} = \boxed{0.4039\ \mathbf{s}} \qquad f_{e1} := \frac{1}{T_{e1}} = 2.48\ \mathbf{Hz}$$

Calculation process shown above is a simplified calculation sheet based on the calculation tool presented in Appendix A.3. The applied mass for roof- and snow are as applied to the focus model shown in Figure 7.6. Additional mass added for this example is based on the assumed equivalent point masses from structures self-weight according to Focus, as shown in Figure 7.7. Eigenperiod of the frame is $T_{n.hc} = 0.404s$, according to the hand calculations presented above.

Difference between eigenperiods is thus, $\Delta T_n = |T_{n.f} - T_{n.hc}| = 0.002s$, which may be neglected. Based on this it is safe to say that the results from vibration analysis in Focus are reliable.

7.2.2 Case 1 · Single cross wind-bracings

For the first case of analyses of the structure, the design of the wind-bracing system is set default as designed by Metacon. Heights, lengths and dimensions of the elements are based on the TEKLA model as shown in Figure 7.1.

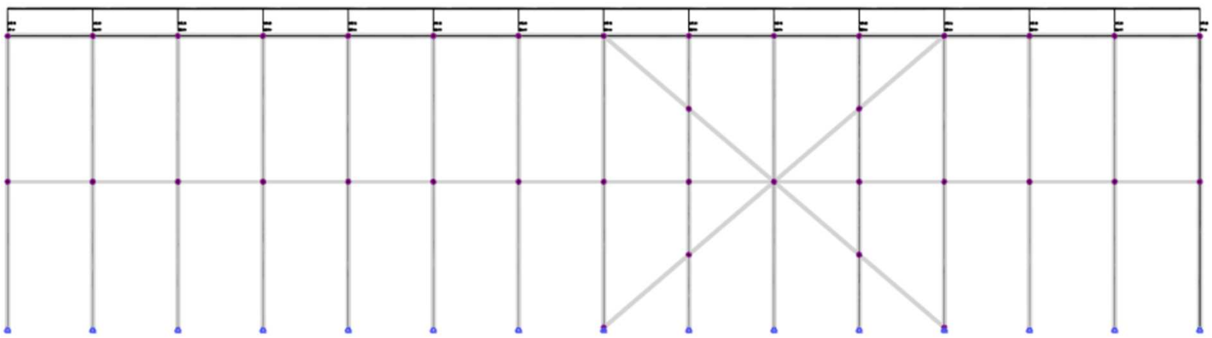


Figure 7.6. Focus model of Case 1, default stiffness.

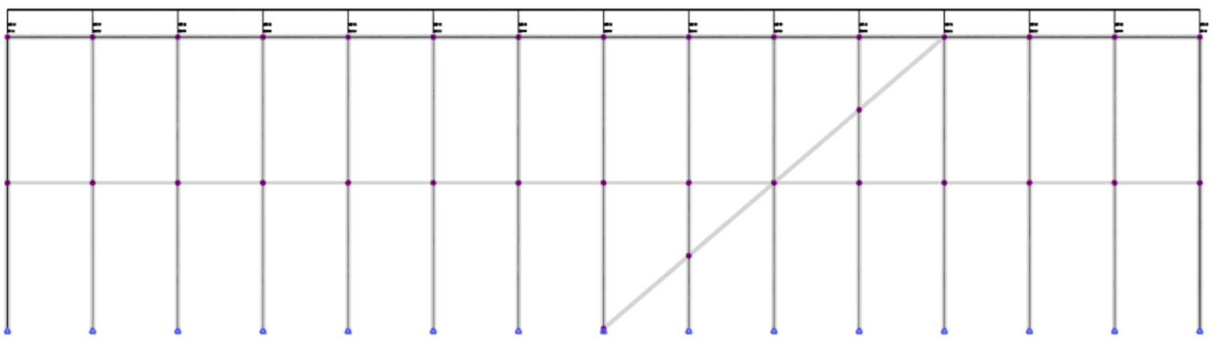


Figure 7.7. Focus model of Case 1, reduced stiffness.

For the first case, the wind-bracing system is designed as a single cross. As a modification to the systems stiffness characteristics, there is one case where the height of the structure is reduced by 10 meters and one case where one of the wind-bracing elements is removed, as shown in Figure 7.7.

7.2.2.1 ULS analysis

The case with single cross wind-bracing system is set as the default setup of the structure since this is how Metacon designed the structure. ULS (ultimate limit state) analyses are only performed for this case of wind-bracing design, but for both height scenarios. The loads from response spectrum analyses for every case however, will be compared to the design loads from the ULS analysis of the single cross case.

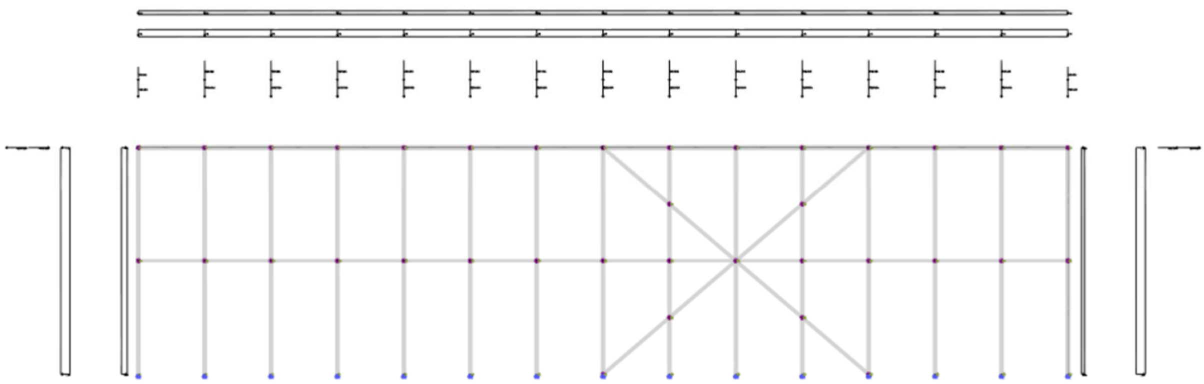


Figure 7.8. Focus model of Case 1 for ULS analysis.

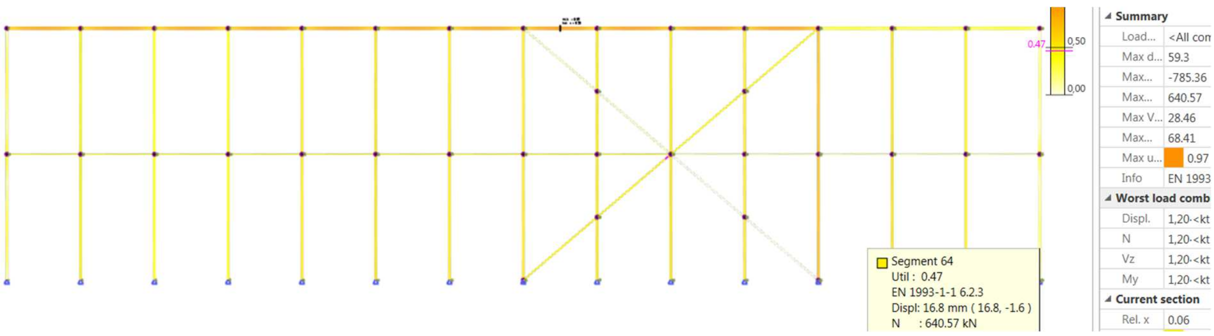


Figure 7.9. Results of ULS analysis for Case 1.

Figures 7.9 shows the Focus model used for the ULS analysis and the graphically presented results shown in the software after running the ULS analysis. The figures above only show the model and results from the scenario with default height. Same procedure is done for the scenario with reduced height. Table 7.3 is a summary of the results from the ULS analyses for both height cases. Both analyses can be found in Appendix C.1.

Table 7.3. Summary of ULS analyses results.

Case	Analysis	Max force [kN]
Default height 21.5m	ULS	640,57
Reduced height 11.5m	ULS	301,81

7.2.2.2 Response spectrum analysis · Case 1

In this chapter, the procedure of analysis is only shown for one height scenario of the single cross wind-bracing system with the Metacon defined response spectrum. Only one mass combination (20% snow mass on roof) is shown, since this is the mass combination recommended in Eurocode 8 [5]. However, analysis of both the default design of the wind-bracing system and the design with reduced stiffness by removing members in the wind-bracing system are shown as a comparison.

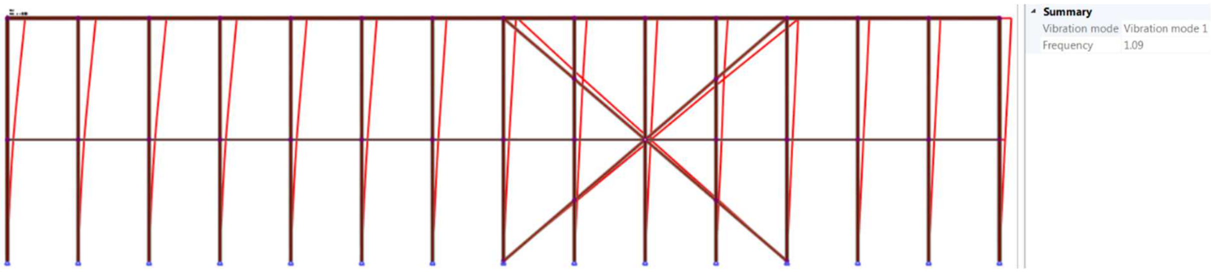


Figure 7.10. Results of Vibration analysis with full stiffness for Case 1.

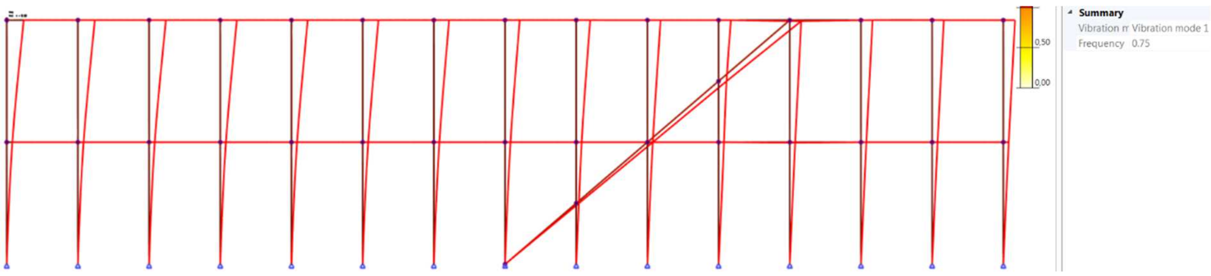


Figure 7.11. Results of Vibration analysis with reduced stiffness for Case 1.

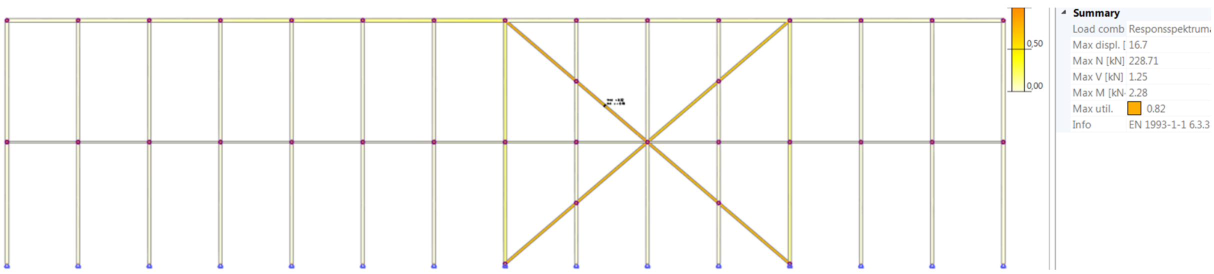


Figure 7.12. Results of Response spectrum analysis with full stiffness for Case 1.

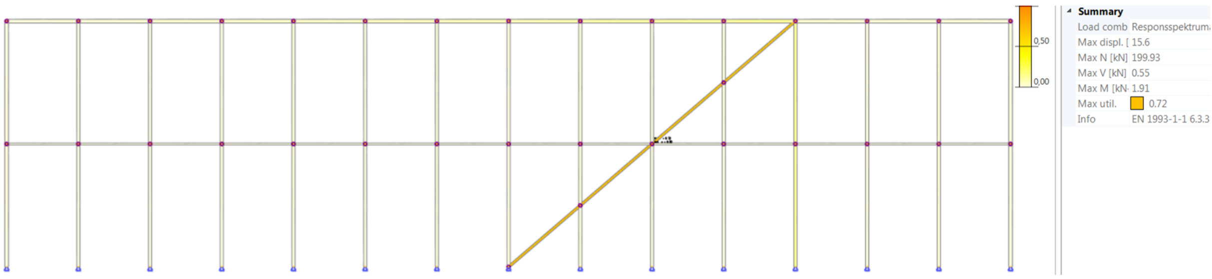


Figure 7.13. Results of Response spectrum analysis with reduced stiffness for Case 1.

Figures above show the vibration analyses and response spectrum analyses for Case 1 with default height, Metacon defined spectrum and 20% snow mass on roof considered for both default stiffness characteristics and reduced stiffness characteristics. Response spectrum analyses for every case is found in Appendix C.2.

Table 7.4. Summary of Response spectrum analyses for Case 1.

Response spectrum and height case		Default stiffness			Reduced stiffness		
		0% snow	20% snow	40% snow	0% snow	20% snow	40% snow
Metacon Default height	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66
	δ [mm]	16,70	16,70	16,60	15,70	15,60	15,60
	F [kN]	229,38	**228,71**	228,32	201,55	**199,93**	199,02
Thesis Default height	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66
	δ [mm]	26,20	30,70	34,60	35,90	42,10	43,80
	F [kN]	358,67	**420,37**	474,09	461,37	**538,03**	558,62
Metacon Reduced height	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83
	δ [mm]	16,10	18,30	18,30	16,60	16,60	16,50
	F [kN]	275,97	**313,63**	313,02	258,64	**257,53**	265,90
Thesis Reduced height	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83
	δ [mm]	22,40	26,40	29,80	30,10	35,40	40,00
	F [kN]	384,05	**450,76**	508,79	469,74	**550,74**	621,28

Table 7.4 shows a summary of the values for natural frequency, displacement and maximum axial force in the wind-bracing elements from the response spectrum analyses performed in Focus, for Case 1. The presented values of force is the maximum axial force in one of the elements (i.e. to find the total force in the system, the force presented in the results needs to be multiplied by the amount of elements in the system). A complete table of results for all cases is found in Appendix C.3. Results will be further discussed in chapter 8.

7.2.3 Case 2 · Double cross wind-bracings

For the second case of analyses of the structure, the heights, lengths and dimensions of the elements are set as presented in the TEKLA model as shown in Figure 7.1. However, some modifications to the design of the wind-bracing system are done.

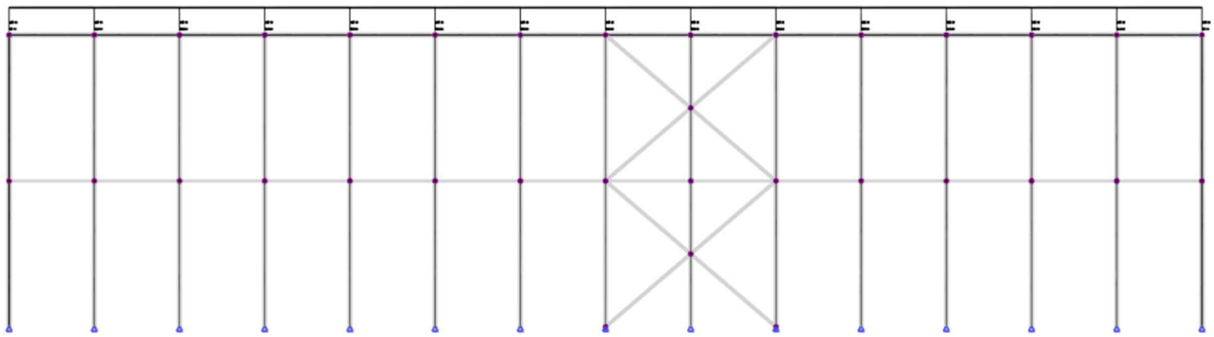


Figure 7.14. Focus model of Case 2, default stiffness.

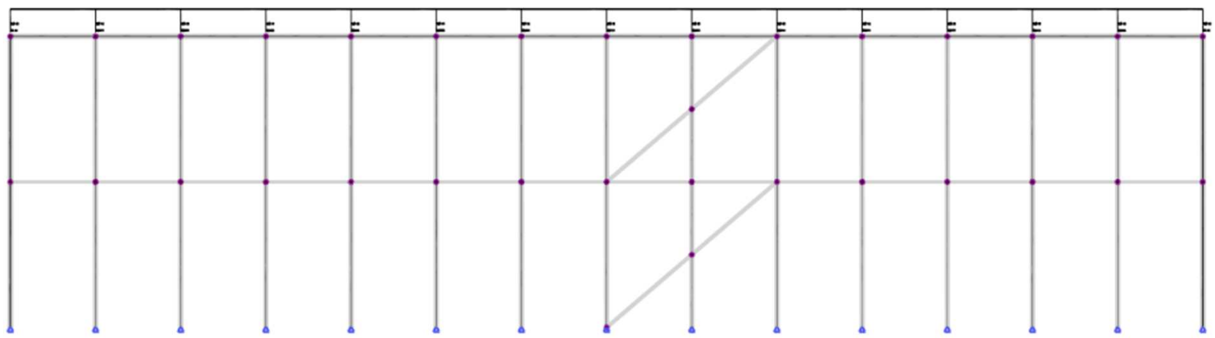


Figure 7.15. Focus model of Case 2, reduced stiffness.

For the second case, the wind-bracing system is designed as a double cross, instead of a single cross as presented in the first case. As a modification to the systems stiffness characteristics, there is one case where the height of the structure is reduced by 10 meters and one case where two of the wind-bracing elements are removed, as shown in Figure 7.15.

7.2.3.1 Response spectrum analysis · Case 2

In this chapter, the procedure of analysis is only shown for one height scenario of the double cross wind-bracing system with the Metacon defined response spectrum. Only one mass combination (20% snow mass on roof) is shown, since this is the mass combination recommended in Eurocode 8 [5]. However, analysis of both the default design of the wind-bracing system and the design with reduced stiffness by removing members in the wind-bracing system are shown as a comparison.

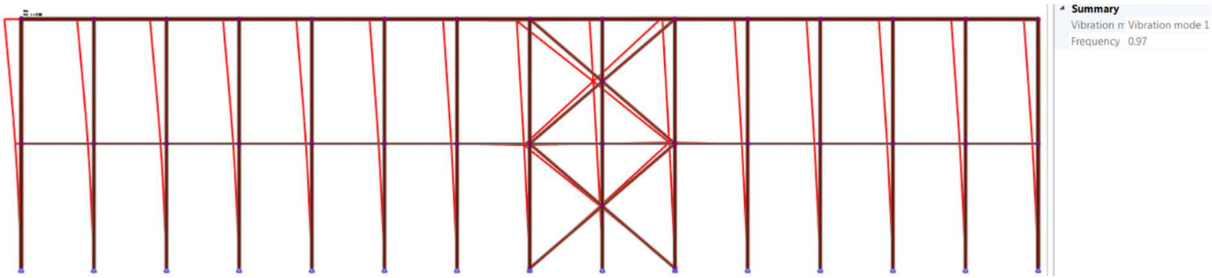


Figure 7.16. Results of Vibration analysis with full stiffness for Case 2.

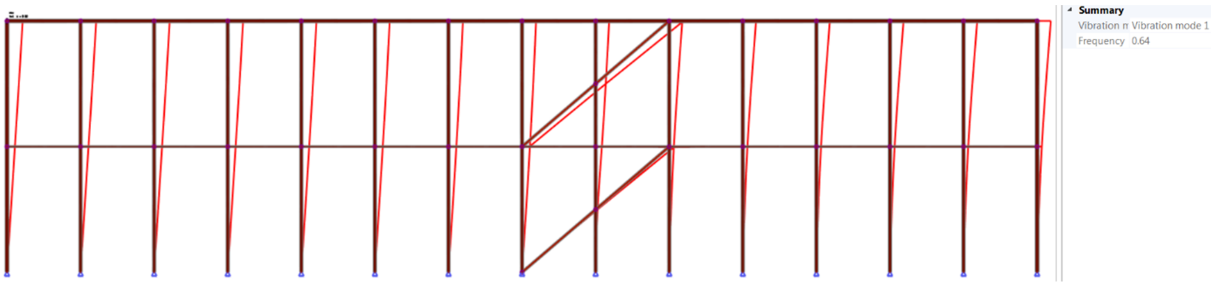


Figure 7.17. Results of Vibration analysis with reduced stiffness for Case 2.

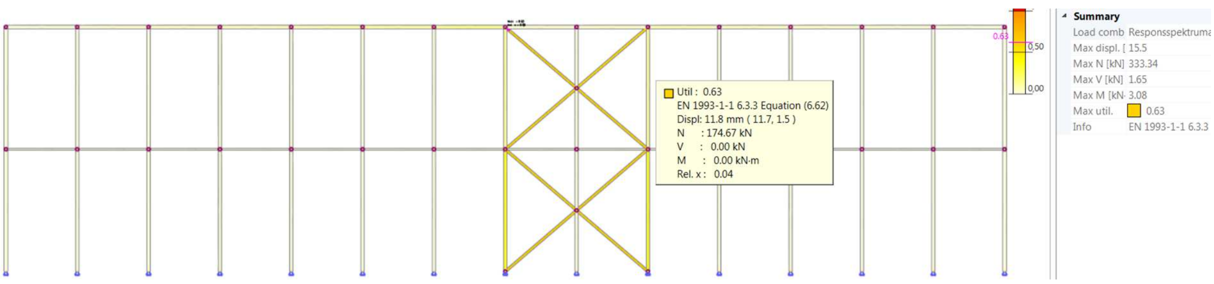


Figure 7.18. Results of Response spectrum analysis with full stiffness for Case 2.

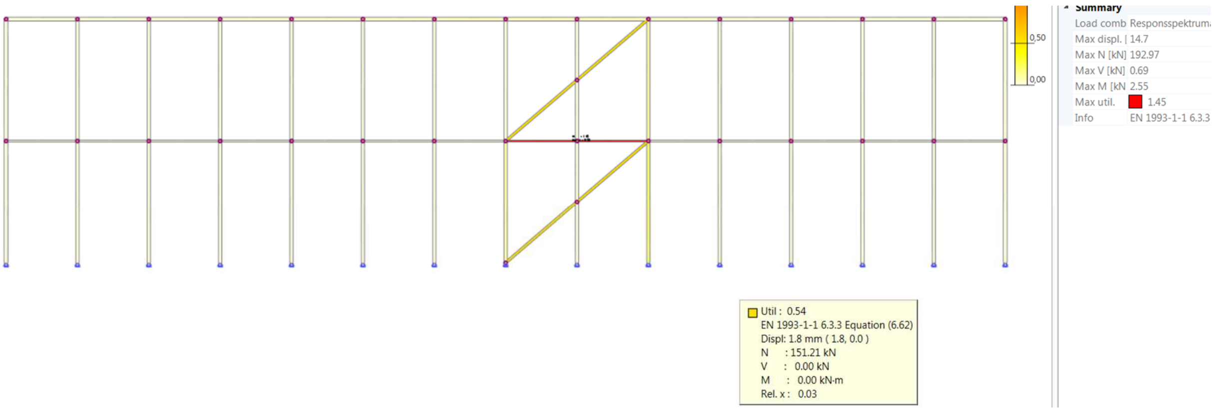


Figure 7.19. Results of Response spectrum analysis with reduced stiffness for Case 2.

Figures above show the vibration analyses and response spectrum analyses for Case 2 with default height, Metacon defined spectrum and 20% snow mass on roof considered for both default stiffness characteristics and reduced stiffness characteristics. Response spectrum analysis for every case is found in Appendix C.2.

Table 7.5. Summary of Response spectrum analyses for Case 2.

Response spectrum and height case		Default stiffness			Reduced stiffness		
		0% snow	20% snow	40% snow	0% snow	20% snow	40% snow
Metacon Default height	f [Hz]	1,14	0,97	0,86	0,75	0,64	0,57
	δ [mm]	15,50	15,50	15,50	14,80	14,70	14,70
	F [kN]	175,00	**174,67**	174,47	152,38	**151,21**	150,55
Thesis Default height	f [Hz]	1,14	0,97	0,86	0,75	0,64	0,57
	δ [mm]	27,20	31,90	36,00	39,20	41,30	41,20
	F [kN]	306,73	**359,87**	406,11	405,07	**424,42**	422,56
Metacon Reduced height	f [Hz]	1,60	1,36	1,20	0,95	0,80	0,71
	δ [mm]	16,10	17,40	17,40	15,40	15,40	15,40
	F [kN]	301,35	**324,10**	323,36	198,66	**197,86**	197,41
Thesis Reduced height	f [Hz]	1,60	1,36	1,20	0,95	0,80	0,71
	δ [mm]	21,90	25,70	29,00	32,70	38,40	43,20
	F [kN]	407,97	**478,82**	540,46	420,95	**493,50**	554,09

Table 7.5 shows a summary of the values for natural frequency, displacement and maximum axial force in the wind-bracing elements from the response spectrum analyses performed in Focus, for Case 2. The presented values of force is the maximum axial force in one of the elements (i.e. to find the total force in the system, the force presented in the results needs to be multiplied by the amount of elements in the system). A complete table of results for all cases is found in Appendix C.3. Results will be further discussed in chapter 8.

7.2.4 Case 3 · Two single cross wind-bracings

For the third case of analyses of the structure, the heights, lengths and dimensions of the elements are set as presented in the TEKLA model shown in Figure 7.1. However, some modifications to the wind-bracing system are done.

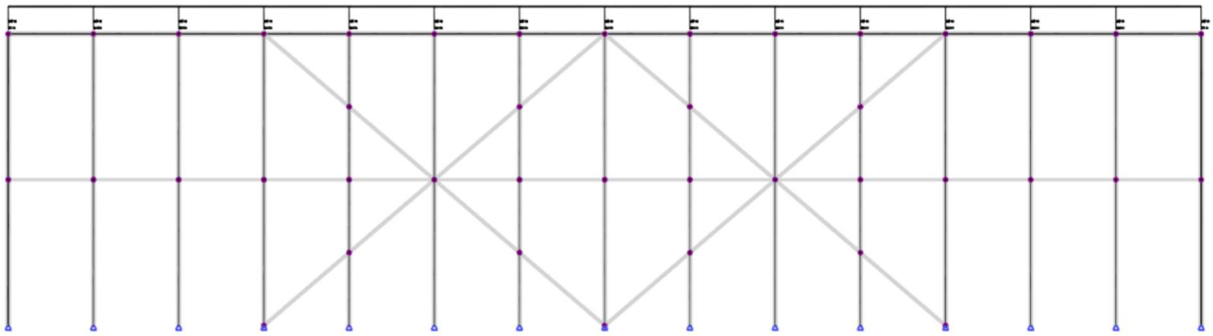


Figure 7.20. Focus model of Case 3, default stiffness.

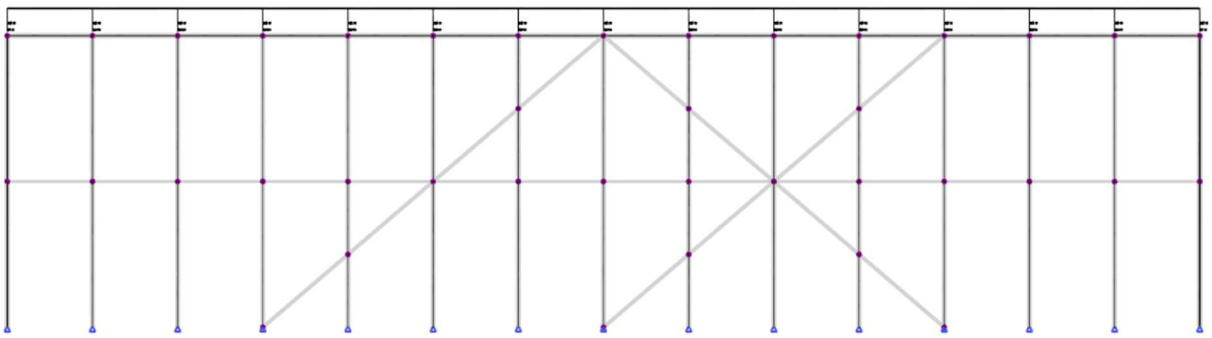


Figure 7.21. Focus model of Case 3, reduced stiffness.

For the third case, the wind-bracing system is designed as two single crosses, instead of a single cross as presented in the first case. As a modification to the systems stiffness characteristics, there is one case where the height of the structure is reduced by 10 meters and one case where one of the wind-bracing elements is removed, as shown in Figure 7.21.

7.2.4.1 Response spectrum analysis · Case 3

In this chapter, the procedure of analysis is only shown for one height scenario of the double cross wind-bracing system with the Metacon defined response spectrum. Only one mass combination (20% snow mass on roof) is shown, since this is the mass combination recommended in Eurocode 8 [5]. However, analysis of both the default design of the wind-bracing system and the design with reduced stiffness by removing members in the wind-bracing system are shown as a comparison.

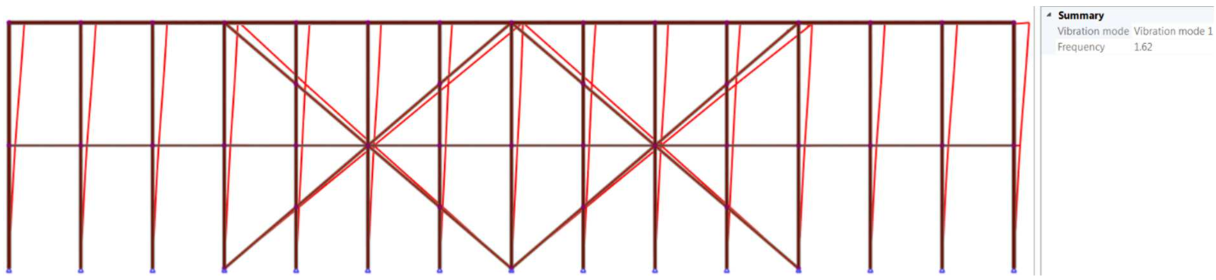


Figure 7.22. Results of Vibration analysis with full stiffness for Case 3.

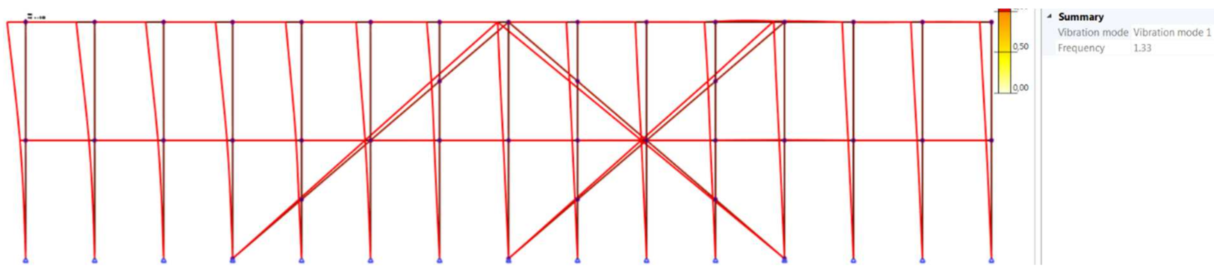


Figure 7.23. Results of Vibration analysis with reduced stiffness for Case 3.

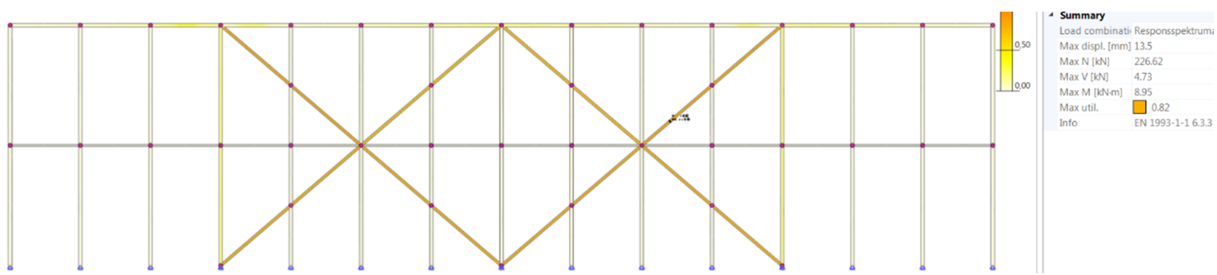


Figure 7.24. Results of Response spectrum analysis with full stiffness for Case 3.

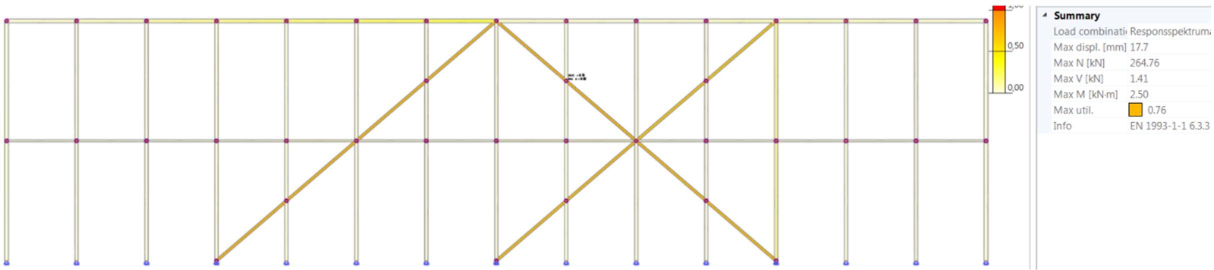


Figure 7.25. Results of Response spectrum analysis with reduced stiffness for Case 3.

Figures above shows the vibration analyses and response spectrum analyses for Case 3 with default height, Metacon defined spectrum and 20% snow mass on roof considered for both default stiffness characteristics and reduced stiffness characteristics. Response spectrum analyses for every case is found in Appendix C.2.

Table 7.6. Summary of Response spectrum analysis for Case 3.

Response spectrum and height case		Default stiffness			Reduced stiffness		
		0% snow	20% snow	40% snow	0% snow	20% snow	40% snow
Metacon Default height	f [Hz]	1,90	1,62	1,43	1,56	1,33	1,17
	δ [mm]	9,80	13,50	14,90	17,30	17,70	17,70
	F [kN]	164,92	**226,62**	249,79	257,64	**264,76**	264,87
Thesis Default height	f [Hz]	1,90	1,62	1,43	1,56	1,33	1,17
	δ [mm]	15,80	18,50	20,80	22,80	26,70	30,20
	F [kN]	265,80	**310,88**	349,23	340,22	**400,22**	452,49
Metacon Reduced height	f [Hz]	2,55	2,16	1,91	2,02	1,72	1,52
	δ [mm]	5,60	7,70	9,90	10,70	14,80	18,50
	F [kN]	145,02	**200,38**	255,73	167,04	**230,00**	285,57
Thesis Reduced height	f [Hz]	2,55	2,16	1,91	2,02	1,72	1,52
	δ [mm]	12,00	14,10	16,00	18,40	21,60	24,40
	F [kN]	312,68	**367,09**	414,39	286,16	**334,63**	376,93

Table 7.6 shows a summary of the values for natural frequency, displacement and maximum axial force in the wind-bracing elements from the response spectrum analyses performed in Focus, for Case 3. The presented values of force is the maximum axial force in one of the elements (i.e. to find the total force in the system, the force presented in the results needs to be multiplied by the amount of elements in the system). A complete table of results for all cases is found in Appendix C.3. Results will be further discussed in chapter 8.

8 Concluding remarks

This chapter covers discussions and conclusions to analyses and results presented in this thesis. Introducing the chapter with discussion of results from analyses presented in chapter 7.2 with proposed reason to why the resulting values of forces due to seismic actions change as the analyses show (Appendix C.3) with respect to the modifications done. After the results from analyses are discusses, a discussion of suitability for the analyzed practical example is presented, followed by a general conclusion. At the end of the chapter, recommendations for further work are suggested.

8.1 Discussion of results

First notice worth mentioning is that for some of the cases, the resulting maximum axial force in the wind-bracing element did not vary much by adding additional snow mass on the roof. Looking at the summary of results (Appendix C.3), the resulting force in the wind-bracing system increases for some cases, while decreasing for other cases, when adding additional snow mass on the roof. This chapter covers a discussion of the results from the analyses with suggested justification.

Summary of analysis results		0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø
MetaCon H1	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66	1,14	0,97	0,86	0,75	0,64	0,57	1,90	1,62	1,43	1,56	1,33	1,17
	δ [mm]	16,70	16,70	16,60	15,70	15,60	15,60	15,50	15,50	15,50	14,80	14,70	14,70	9,80	13,50	14,90	17,30	17,70	17,70
	F [kN]	229,38	228,71	228,32	201,55	199,93	199,02	175,00	174,67	174,47	152,38	151,21	150,55	164,92	226,62	249,79	257,64	264,76	264,87

Figure 8.1. Results with Metacon spectrum and full height.

Figure 8.1 shows the results form response spectrum analyses for the case where the Metacon defined response spectrum is used with the default height of project 14-112. The results are split into three different cases, marked with the colors red, green and blue.

As mentioned in the introduction to this chapter, for some cases of the structure, the maximum axial force in the wind-bracing element does not change much when increasing the amount of snow mass on roof considered. Results for these type of cases are marked with the color red in Figure 8.1.

For the case where the wind-bracing system is designed as two single crosses (chapter 7.2.3), there is an interesting spike in increase of the maximum force when considering 20% snow mass, compared to 0% snow mass. However, when comparing the difference between 20% snow mass and 40% snow mass, the difference in the resulting force is noticeably lower. Results for this case are marked with the color green in Figure 8.1.

Last marked result case represents the case where the wind-bracing system is design as two single crosses with reduced stiffness characteristics by removing one of the elements. Here it is noticed that the difference in resulting maximum force changes when comparing between 0% snow mass- and 20% snow mass considered. However, the difference in the resulting force, when comparing 20%- and 40% snow mass, is as low as for the first result cases marked with red. Results for this case are marked with the color blue in Figure 8.1.

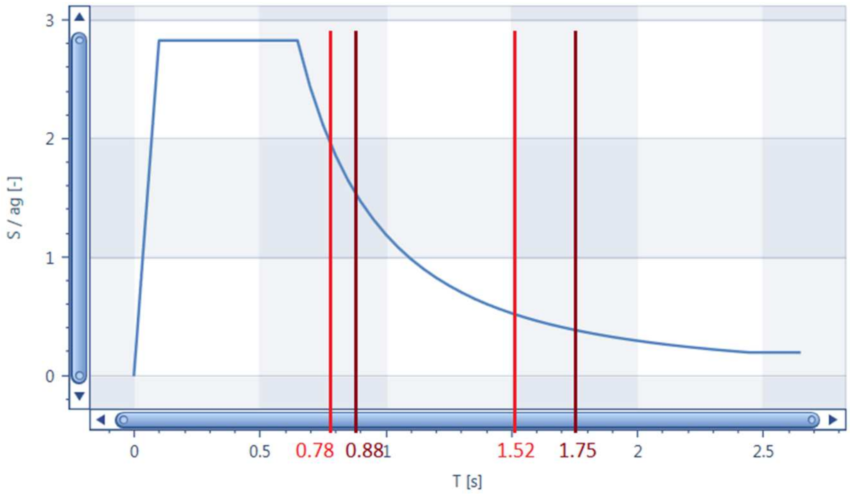


Figure 8.2. Spectrum area for results marked with red in Figure 8.1.

Figure 8.2 shows the Metacon defined spectrum with four highlighted values of the period. The light red highlighted periods are the highest and lowest natural periods of the cases with single cross wind-bracings (chapter 7.2.2), where the natural period is varying between $1{,}28Hz = 0{,}78s$ and $0{,}66Hz = 1.52s$ by modifying stiffness characteristics and amount of mass considered. The dark red highlighted periods are the highest and lowest natural periods of the case with double cross wind-bracings (chapter 7.2.2), where the natural periods is varying between $1{,}14Hz = 0{,}88s$ and $0{,}57Hz = 1.75s$ by modifying stiffness characteristics and amount of mass considered.

As discussed in chapter 3.3.3, the base force F_{bk}, acting in the direction of application of the seismic action, may be expressed with the equation $F_{bk} = S_d(T_k)m_k$. In other words, the form of the response spectrum and swinging mass are directly correspondent to the resulting force. Considering highlighted values in Figure 8.2, it is noticed that with full stiffness and 0% snow mass considered, the value of $S/_{a_g}$ [7] is at its highest value (1.97). By reducing the stiffness of the system and considering 40% snow mass, the value of $S/_{a_g}$ is at its lowest value (0.40). Thus, an increase of mass, which directly results in an increase of the base force F_b also decreases the value of S (or $S_d(T)$), which simultaneously decreases the base force F_b.

For this specific case (default height and Metacon defined spectrum), the force increased by considering additional snow mass on the roof is slightly less than the rate of

[7] a_g is a constant based on the ground acceleration and design factor γ [5].

reduction of S due to changes in natural frequency. As an example, the resulting forces for the case with single cross wind-bracings is calculated and assessed. When comparing the forces for cases where 0%- and 20% snow mass are considered, the difference in force is $\Delta F = -0{,}67kN$. Doing the same for cases where 20%- and 40% snow mass are considered, the difference in force is only $\Delta F = -0{,}39kN$. These observations match the suggested justification for the small changes in resulting force discussed above. Case with 0% snow mass considered has the lowest period, thus positioned furthest to the left on the response spectrum graph shown in Figure 8.2 (highlighted as 0.78s). For the two other cases, the period is increased by increasing snow mass. Higher periods are positioned further to the right in the response spectrum graph. The graph is steeper between 0%- and 20% snow mass, compared to graph between 20%- and 40% snow mass. Steeper graph results in more reduction of the force. As seen from the presented ΔF above, it is seen that the force has decreased more between 0%- and 20% snow mass, compared to the difference between 20%- and 40% snow mass.

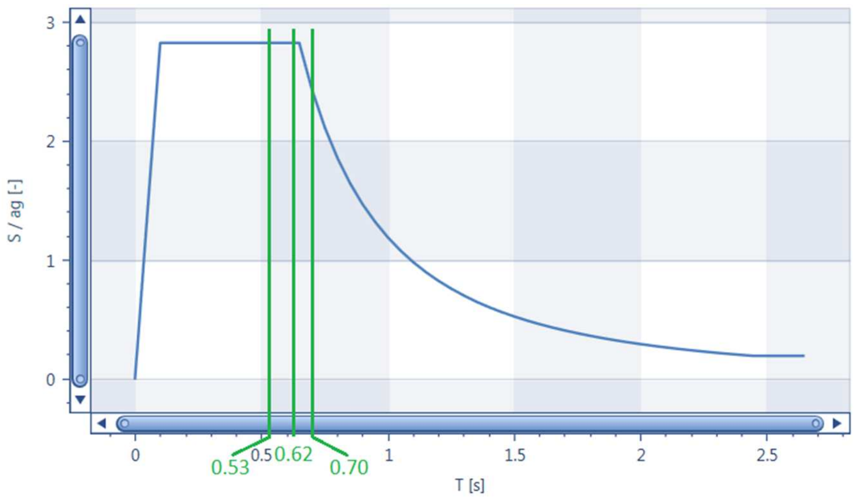


Figure 8.3. Spectrum area for results marked with green in Figure 8.1.

Figure 8.3 shows the Metacon defined spectrum with three highlighted values of the period. The green highlighted periods are the natural periods for the three different cases of snow mass on roof considered, for the cases with two single cross wind-bracings (chapter 7.2.3). First highlighted period (0.53s) is for the case where 0% snow mass is considered, second highlighted period (0.62s) is for 20% snow mass considered and the third highlighted period (0.70s) is for 40% snow mass considered.

This case interesting because two of the mass cases have natural periods positioned on the flat top of the response spectrum graph, and the last one positioned in the area where the graph is at its steepest. If the justification or reasoning for resulting forces discussed in the previous set of cases is correct, there should be a big spike in force increase when comparing cases with 0%- and 20% snow mass on the roof. When comparing cases with 20%- and 40% snow mass, the difference in force should either decrease or increase less than it did for the previous case.

As seen in Figure 8.3, when 0% snow mass is considered, the resulting maximum force in the wind-bracing elements is 164,92kN. When 20% snow mass is considered, the resulting force is 226,62kN. This means that the difference in force ($\Delta F = 61{,}7kN$) is increasing significantly, as assumed.

For the next case, the difference in force when considering 20%- and 40% snow mass on the roof is assessed. As mentioned above, for 20% snow mass the resulting force is 226,62kN. Considering 40% snow mass, the resulting force is 249,79kN. This means that the difference in force is $\Delta F = 23{,}17kN$. It was assumed that it would either decrease or increase less than it did for the previous case. As shown, it did not decrease, but the amount increased is 62% less compared to the previous case. Reason for this is because the natural period of the 40% snow mass case is positioned in the steep area of the response spectrum graph shown in Figure 8.3, resulting in higher level of reduction of the force due to reduced value of S.

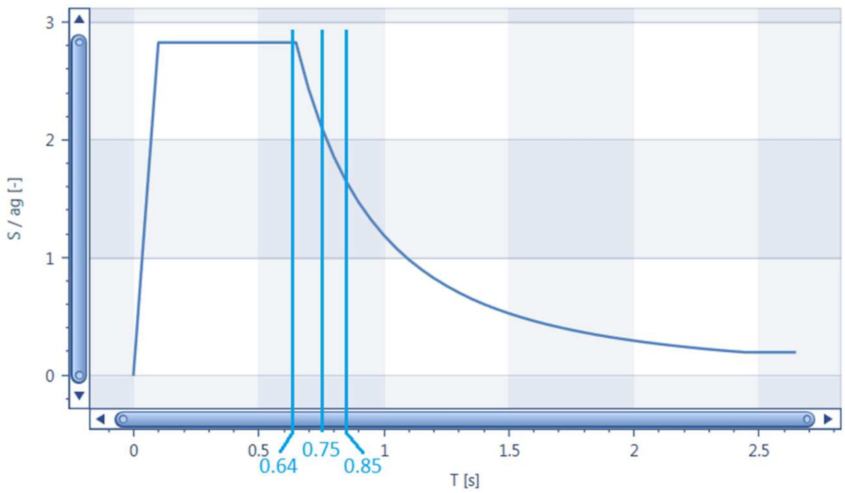


Figure 8.4. Spectrum area for results marked with blue in Figure 8.1.

Figure 8.4 shows the Metacon defined spectrum with three highlighted values of the period. The blue highlighted periods are the natural periods for the three different cases of snow mass on roof considered, for the cases with modified (stiffness reduced) two single cross wind-bracings (chapter 7.2.3). First highlighted period (0.64s) is for the case where 0% snow mass is considered, second highlighted period (0.75s) is for 20% snow mass considered and the third highlighted period (0.85s) is for 40% snow mass considered.

This case is interesting because two of the mass cases have natural periods positioned on the steep area of the response spectrum graph, while one is positioned on the flat area on the top of the graph. If the justification or reasoning for resulting forces discussed in the previous set of cases is correct, it should be seen that the resulting forces do not increase much due to the position of periods on the graph. An assumption for this case is that bigger differences in force should be present when 0%- to 20% snow mass is considered, compared to when 20%- to 40% snow mass is considered, since the natural period of the case with 0% snow mass is positioned on the flat part of the graph.

As seen in Figure 8.4, when 0% snow mass on the roof is considered, the resulting maximum force in the wind-bracing elements is 257,64kN. When 20% snow mass is considered, the resulting force is 264,76kN. The difference in force is thus, $\Delta F = 7{,}12kN$. The force still does increase, but noticeably less compared to the previous case with same mass combinations considered. Since this case has one of the periods positioned on the flat area of the graph, it may be assumed that the second case will have higher level of reduction in force.

For the next case, the difference in force when considering 20%- and 40% snow mass on the roof is assessed. As mentioned above, for 20% snow mass the resulting force is 264,76kN. Considering 40% snow mass, the resulting force is 264,87kN. This means that the difference in force is only $\Delta F = 0{,}11kN$. The additional reduction of force due to reduced value of S is not enough to decrease the force, but as shown, the reduction is higher compared to the previous case, as assumed.

8.2 Discussion of suitability for practical example

This chapter covers an example of calculations and suggested checks to determine whether or not; the method of installing smart elements to the practical example is suitable for project 14-112 with default wind-bracing design.

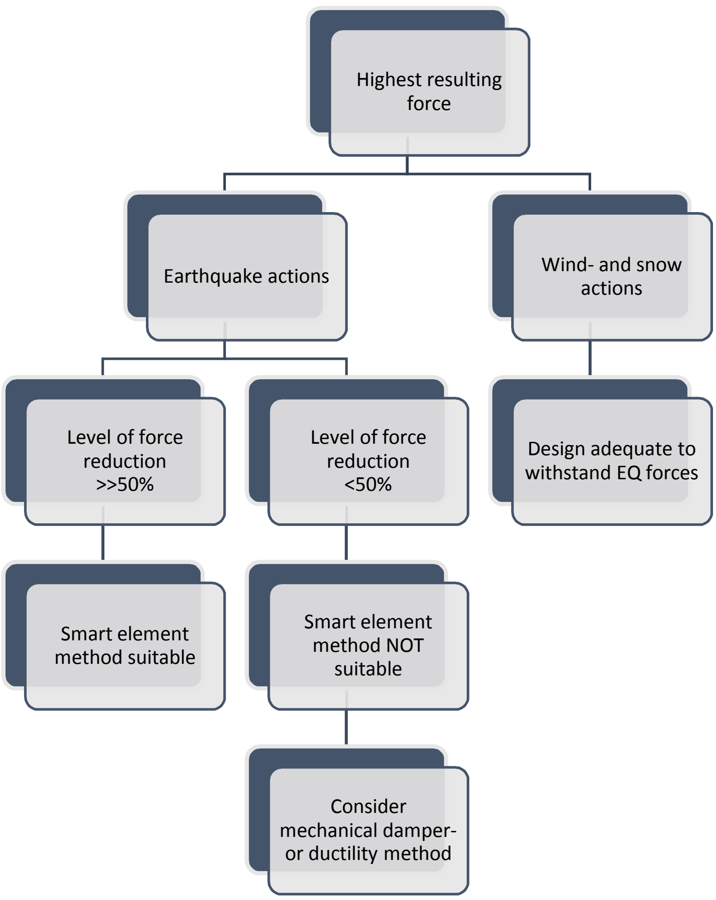
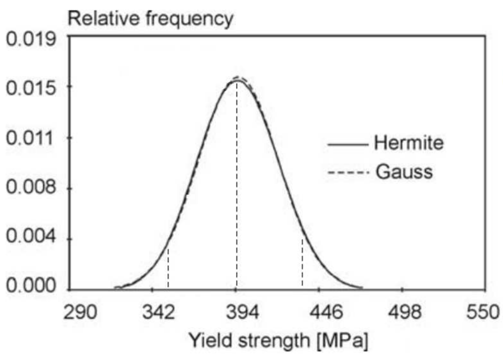


Figure 8.5. Hierarchy of suggested suitability check method.

Figure 8.5 shows a hierarchy table of a suggested procedure to check the suitability of smart element method (discussed in chapter 6.2.1) for an arbitrary structure. First step is to determine what type of force sets the design criterion. In case the resulting forces from often occurring environmental actions, such as wind- and snow actions, set the design criterion, the structure will be designed to withstand often occurring forces with adequate resistance to also withstand forces due to earthquake actions. In case the resulting forces from earthquake actions set the design criterion, the structure would be designed to withstand these forces, resulting in oversizing the dimensions of the elements when forces from often occurring actions are considered. Designing a structure to withstand a force with an assumed return period of 475 years [5] may be avoided by applying the discussed methods of modifying dynamic characteristics, as discussed in chapter 6.

Thus, if the often occurring actions set the design basis, it is not required to give the structure adaptive characteristics. In case the earthquake actions set the design basis, next step of suitability check is to determine the level of force reduction after the modifications are initiated. In the hierarchy shown in Figure 8.5, the boundary condition is set to greater- or less than 50% force reduction. However, this boundary condition relies on material type and its uncertainties of yield strength as discussed in chapter 6.1.



Statistical analysis of yield strength

Mean value	Standard deviation	Min value	Max value
395.68 MPa	25.126 MPa	355.00 MPa	436.36 MPa

Figure 8.6. Relative frequency of yield strength [30].

Figure 8.6 shows a normal distribution of values for yield strength from a tensile test of an s355 stainless steel. Values on the graph is showing a mean value of the steels yield strength at 395.68MPa. 355MPa is assumed to be the lowest value- and 436.36MPa is assumed to be the highest value of yield strength. These values will be used in calculation phase of the suitability check.

8.2.1 Default height

This chapter covers a suitability check for use of the smart element method on project 14-112 with *default* height. As the suggested procedure presented in Figure 8.5, starting by determining what type of force sets the design criteria for the dimensions of the wind-bracing elements.

Summary of analysis results		0% snø	20% snø	40% snø	0% snø	20% snø	40% snø
MetaCon H1	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66
	δ [mm]	16,70	16,70	16,60	15,70	15,60	15,60
	F [kN]	229,38	228,71	228,32	201,55	199,93	199,02
Thesis H1	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66
	δ [mm]	26,20	30,70	34,60	35,90	42,10	43,80
	F [kN]	358,67	420,37	474,09	461,37	538,03	558,62

Figure 8.7. Summary of results for case with single cross and default height.

Figure 8.7 shows the results from analyses of the single cross wind bracing design (chapter 7.2.2) where default height is considered. The highlighted values present the resulting forces from response spectrum analysis where 20% snow mass and Metacon defined response spectrum is considered for both the case with default- and reduced stiffness.

Resulting forces from often occurring actions (wind and snow) are defined by an ULS analysis as $F_{Ed} = 640.57kN$ for each element (chapter 7.2.2.1). Figure 8.7 shows results for both analyses with Metacon spectrum and Thesis spectrum (response spectrums defined in chapter 7.1.1). Even though the resulting forces are higher when considering Thesis spectrum, the maximum resulting force from earthquake actions is $F_{EQ} = 538.03kN$, which is lower than resulting force from often occurring environmental actions ($F_{Ed} > F_{EQ}$). Thus, the conclusion for this case is that use of the smart element method is not suitable.

Since $F_{Ed} > F_{EQ}$, the often occurring environmental loads set the design criteria for element dimensions and a modification of the structures dynamic characteristics is therefore not required.

8.2.2 Reduced height

This chapter covers a suitability check for use of the smart element method on project 14-112 with *reduced* height. As the suggested procedure presented in Figure 8.5, starting by determining what type of force sets the design criteria for the dimensions of the wind-bracing elements.

Summary of analysis results		0% snø	20% snø	40% snø	0% snø	20% snø	40% snø
MetaCon H2	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83
	δ [mm]	16,10	18,30	18,30	16,60	16,60	16,50
	F [kN]	275,97	313,63	313,02	258,64	257,53	265,90
Thesis H2	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83
	δ [mm]	22,40	26,40	29,80	30,10	35,40	40,00
	F [kN]	384,05	450,76	508,79	469,74	550,74	621,28

Figure 8.8. Summary of results for case with single cross and reduced height.

Figure 8.8 shows the results from analyses of the single cross wind-bracing design (chapter 7.2.2) where reduced height is considered. The highlighted values present the resulting forces from response spectrum analysis where 20% snow mass and Metacon defined response spectrum is considered for both the case with default- and reduced stiffness.

Compared to the previous case with default height, the resulting forces from often occurring actions (wind and snow) for this case are lower due to less surface exposed to wind. The force is determined by an ULS analysis as shown in chapter 7.2.2.1, defined as $F_{Ed} = 301.81kN$. For both type of response spectrums considered, the resulting force from earthquake actions is greater than the resulting force from often occurring environmental actions ($F_{EQ} > F_{Ed}$). Thus, evaluation of modifying the structures dynamic characteristics may be considered.

8.2.2.1 Metacon spectrum

When Metacon defined response spectrum is considered, the resulting forces from earthquake actions are $F_{EQ} = 313{,}63kN$ for the case with default stiffness, and reduced to $F_{EQ} = 257{,}53kN$ for the case with reduced stiffness. Here it is important to notice that the presented values are forces for each element in the wind-bracing system, i.e. total force in the wind-bracing system from often occurring environmental actions is $F_{Ed.tot} = 603.62kN$. Likewise, the total force in the wind-bracing system from earthquake actions for the case with default stiffness is $F_{EQ.tot.def} = 627.26kN$. For the case with reduced stiffness however, the total force in the wind-bracing system from earthquake actions is as shown in Figure 8.8, $F_{EQ.tot.red} = 257{,}53kN$ since only one element is active after the modification is initiated to the wind-bracing system. Reduction of force due to modifications to the systems stiffness characteristics is thus, $1 - \frac{257.53kN}{627.26kN} = 58.95\%$.

Next step is to check possibility to design sacrificial elements to acts as intended and initiate the modification of dynamic characteristics when needed. To follow the example shown in Figure 6.2, bolts are considered for this example.

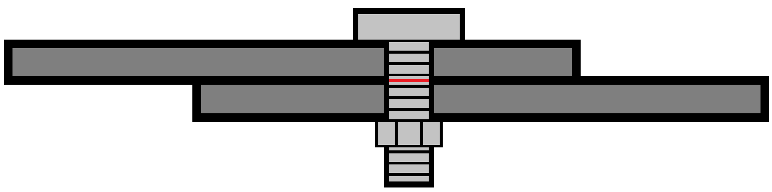


Figure 8.9. Simple illustration of bolt connecting two plates.

Figure 8.9 shows an arbitrary bolt connecting two plates. As shown with the red line in the figure, the bolt is exposed to a shear (cut) force at a single location of the bolt. Thus, total resistance to shear is determined based on the characteristic value of shear resistance per bolt, multiplied by the amount of bolts in the connection.

Table 8.1. Shear resistance of grade 10.9 Bolts, based on [31].

Resistance	Type of bolt									
	M12	M14	M16	M18	M20	M22	M24	M27	M30	M33
$F_{d,v}$	33,6	46,0	62,8	76,8	98,0	121,2	141,2	183,6	224,4	277,6

Table 8.1 shows a table of characteristic values for shear resistance per bolt. If a bolt group is designed as four M18 bolts (marked with light blue color in the table), the total capacity of the bolt group is $F_{d,v.tot} = 76{,}8 * 4 = 307.2kN$. Thus, there is adequate resistance to withstand resulting forces from often occurring environmental actions ($F_{d,v.tot} > F_{Ed} = 301.81kN$), yet the resulting forces from earthquake actions is high enough to break the bolts initiating the intended modification ($F_{d,v.tot} < F_{EQ} = 313.63kN$).

However, with a force difference by roughly 6kN when comparing bolts minimum resistance capacity to the maximum force from resulting earthquake actions the scenario has a low margin to successfully act in practice as theoretically calculated and assumed. Due to the low difference of $F_{EQ} = 313.63kN$ and $F_{Ed} = 301.81kN$, the resulting dimensions of the elements would most likely be the same even if F_{EQ} sets the design criterion.

Thus, as a conclusion, the smart element method is possible for this specific case and scenario, but not efficient or suggested.

8.2.2.2 Thesis spectrum

When the Thesis response spectrum is considered, the resulting total forces in the wind-bracing system from earthquake actions are $F_{EQ.tot.def} = 901{,}52kN$ for the case with default stiffness, and reduced to $F_{EQ.tot.red} = 550{,}74kN$ for the case with reduced stiffness. The reduction of force due to stiffness modifications is $1 - \frac{550{,}74kN}{901{,}52kN} = 38{,}91\%$. Compared to the total resulting force from often occurring environmental actions $F_{Ed.tot} = 603{,}62kN$, there is a relatively big gap between the resulting forces. However, since the reduction of force is <50%, the smart element method becomes unsuitable since the remaining elements in the wind-bracing system does not have adequate resistance capacity to withstand the reduced forces after the modifications.

For this case, methods where mechanical dampers or ductile elements should be considered, as discussed in chapter 6.2.2 and 6.2.3. These methods are more suitable for cases where reduction of force is <50%, since these methods are not based on total removal of elements and thus, result in a system post modification with higher resistance capacity to the reduced forces.

8.3 Conclusions

The level of robustness does not have a consistent definition and a quantitative measure. However, there are several generic proposed measure of system damage tolerance (reliability-based assessment) with correlations to metrics for robustness (risk-based assessment). Based on the practical robustness-increasing methods discussed in this thesis, the resulting design of the structure may be considered an extra safety feature, which draws a correlation between level of robustness and safety level of a structure.

For structures where earthquake actions set the design criteria based on structural codes, are not determined to be designed in a matter to withstand the resulting earthquake forces statically. As assumed and discussed in this thesis, there is a direct correlation between a structures stiffness characteristics and the resulting force from earthquake actions. Approach of design methods, which gives the structure features of adapting to various load scenarios are discussed, presented and analyzed in the thesis. Based on the results of the analyses, this approach of design should be considered for structures where the probability of earthquake events are relatively low, yet sets the design criteria.

For areas where probability of earthquake events are relatively low, e.g. Norway with a assumed return period of 475 years, designing a structure based on design criteria set by earthquake events may be considered a high additional cost factor to ensure safety against an unusual load scenario. Cost-beneficial analyses (CBA) are not performed in this thesis, but for specific building cases, it is assumed that the discussed design approach may reduce the cost factor, without compensating on the structures safety level. In other words, the safety against earthquake actions is not fully paid in advance.

The suggested design procedure is applied to the practical example of a structure (project 14-112 by Metacon) analyzed in this thesis. The structure originally (with default height of 21.5m) has often-occurring environmental loads as design criteria. To make the structure more suitable for the design approach, the structures height is reduced by 10m, causing a large decrease in wind loads, while increasing the resulting earthquake loads.

Results from suitability check calculations, vibration- and response spectrum analyses, show that the design procedure is both suitable and desirable for the practical example with reduced height. However, the low margin for error for the specific case assessed might be considered unacceptable. As a general limitation considering the suitability of the design approach, the earthquake load has to be much larger than the resulting loads from often-occurring environmental loads ($F_{EQ} \gg F_{Ed}$).

8.4 Recommendations for further work

Methods based on installing mechanical- and material dampers are mentioned and discussed in this thesis, but the resulting effects should be analyzed and assessed further, and compared to the other methods discussed.

Costs involved in installation of various dampers are mentioned, and the suggested design procedure is based on various practical robustness-increasing methods where cost might be a major design factor. Cost-beneficial analyses should be performed for various design methods discussed in the thesis and compared to traditional design methods.

Methods of structural robustness assessment are discussed, but not performed for the practical example assessed in this thesis. In addition, a procedure for required maintenance work for the discussed robustness-increasing methods should be suggested.

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Appendices

Appendix A · Calculation examples & derivations

A.1 Derivation of equation (2.14)

The derivation of equation (2.14) is done with Mathcad Prime 3.0 [7], by defining the equation to be solved, and using the solve function of the software.

$$x(t) := e^{-\zeta \cdot w_n \cdot t} \cdot \left(C_1 \cdot cos\left(\sqrt{1-\zeta^2} \cdot w_n \cdot t\right) + C_2 \cdot sin\left(\sqrt{1-\zeta^2} \cdot w_n \cdot t\right)\right)$$

$$\frac{d}{dt} x(t) \rightarrow e^{-(t \cdot \zeta \cdot w_n)} \cdot \left(C_2 \cdot w_n \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2} - C_1 \cdot w_n \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2}\right) - \zeta \cdot w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \left(C_1 \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) + C_2 \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right)\right)$$

$$x' = e^{-(t \cdot \zeta \cdot w_n)} \cdot \left(C_2 \cdot w_n \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2} - C_1 \cdot w_n \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2}\right) - \zeta \cdot w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \left(C_1 \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) + C_2 \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right)\right)$$

$$\xrightarrow{solve, C_2} \frac{x' + \zeta \cdot C_1 \cdot w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) + C_1 \cdot w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2}}{w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \cos\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right) \cdot \sqrt{1-\zeta^2} - \zeta \cdot w_n \cdot e^{-(t \cdot \zeta \cdot w_n)} \cdot \sin\left(t \cdot w_n \cdot \sqrt{1-\zeta^2}\right)}$$

By setting $t = 0$ and $C_1 = x_0$, the equation may be simplified to

$$\rightarrow \frac{x' + \zeta \cdot x_0 \cdot w_n}{w_n \cdot \sqrt{1-\zeta^2}}$$

$$C_2 = \frac{x' + \zeta\, x_0\, \omega_n}{\omega_d}$$

A.2 Force vs. Stiffness · Column

Calculations are done on the base guidelines given in Eurocode 8 [5]. Mathcad Prime [7] is used as a calculation tool to set up the equations to calculate the representative force from the assumed earthquake. A reduction factor φ is added, to check the resulting forces for each case of stiffness when reduction of stiffness is considered.

The element analyzed is a 10m long rigid column of the type SHS100x8. Self-mass of the element is concentrated at the top as a point load, assuming $\frac{1}{4}$ of the self-mass is active during the vibration, as discussed in chapter 2.2.2.

Element data

$M := 229\ \mathbf{kg}$

$I := 4.13 \cdot 10^{6}\ \mathbf{mm}^{4}$

$L := 10\ \mathbf{m}$

$E := 2.1 \cdot 10^{5}\ \dfrac{\mathbf{N}}{\mathbf{mm}^{2}}$

Reduction factor

$\varphi := 0.1, 0.11 .. 1$

Stiffness factor

$$k(\varphi) := \frac{3\ E \cdot I}{L^{3}} \cdot \varphi \qquad k(\varphi) = \begin{bmatrix} 260.19 \\ 286.209 \\ 312.228 \\ \vdots \end{bmatrix} \frac{\mathbf{kg}}{\mathbf{s}^{2}}$$

Eigenfrequency

$$\omega(\varphi) := \sqrt{\frac{k(\varphi)}{\left(\frac{M}{4}\right)}} \qquad \omega(\varphi) = \begin{bmatrix} 2.132 \\ 2.236 \\ 2.335 \\ \vdots \end{bmatrix} \frac{\mathbf{rad}}{\mathbf{s}} \qquad \omega_{Hz}(\varphi) := \frac{\omega(\varphi)}{2\ \pi}$$

$$\omega_{Hz}(\varphi) = \begin{bmatrix} 0.34 \\ 0.36 \\ \vdots \end{bmatrix} \mathbf{Hz}$$

Characteristic values for reponse spectrum (called Thesis)
Ground type E · Seismic class IV

$S := 1.55 \qquad T_B := 0.15\ \mathbf{s} \qquad T_C := 0.40\ \mathbf{s} \qquad T_D := 1.60\ \mathbf{s} \qquad \gamma_1 := 1.0$

$a_{g40Hz} := 0.80\ \dfrac{\mathbf{m}}{\mathbf{s}^{2}} \qquad a_{gR} := 0.8 \cdot a_{g40Hz} = 0.64\ \dfrac{\mathbf{m}}{\mathbf{s}^{2}} \qquad a_g := a_{gR} \cdot \gamma_1 = 0.64\ \dfrac{\mathbf{m}}{\mathbf{s}^{2}}$

$\beta := 0.2 \qquad q := 1.5$

Calculation of design spectrum Sd(T)

$$S_d(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\dfrac{2}{3} + \dfrac{T(\varphi)}{T_B} \cdot \left(\dfrac{2.5}{q} - \dfrac{2}{3}\right)\right) & \text{if } 0 \le T(\varphi) \le T_B \\ a_g \cdot S \cdot \dfrac{2.5}{q} & \text{else if } T_B \le T(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \dfrac{T_C}{T(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \dfrac{T_C \cdot T_D}{T(\varphi)^{2}}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$$S_d(\varphi) = \begin{bmatrix} 0.128 \\ 0.134 \\ 0.146 \\ \vdots \end{bmatrix} \frac{\mathbf{m}}{\mathbf{s}^{2}} \qquad \frac{S_d(\varphi)}{a_g} = \begin{bmatrix} 0.2 \\ 0.2094 \\ 0.2284 \\ \vdots \end{bmatrix}$$

Max displacement on top of column

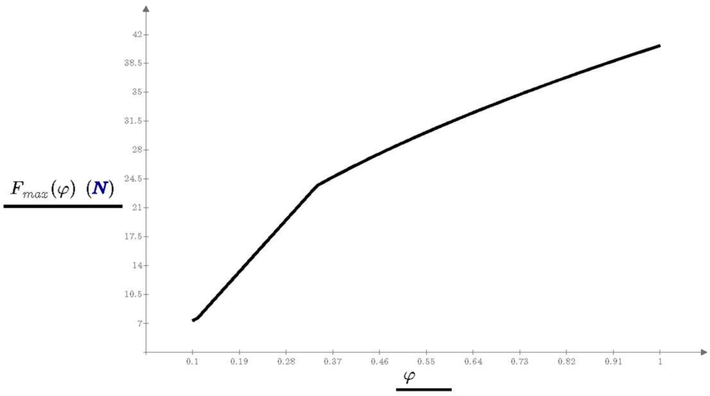
$$\delta_{max}(\varphi) := \frac{S_d(\varphi)}{\omega(\varphi)^{2}} \qquad \delta_{max}(\varphi) = \begin{bmatrix} 28.164 \\ 26.803 \\ 26.803 \\ \vdots \end{bmatrix} \mathbf{mm}$$

Max shearforce on foundation, assuming 2 or less storey building

$$F_{max}(\varphi) := S_d(\varphi) \cdot \frac{M}{4}$$

$$F_{max}(\varphi) = \begin{bmatrix} 7.328 \\ 7.671 \\ 8.369 \\ \vdots \end{bmatrix} \boldsymbol{N}$$

Plot of results



A.3 Force vs. Stiffness · Frame

Calculations are done on the base guidelines given in Eurocode 8 [5]. Mathcad Prime [7] is used as a calculation tool to set up the equations to calculate the representative force from the assumed earthquake. A reduction factor φ is added, to check the resulting forces for each case of stiffness when reduction of stiffness is considered.

Four frames with different heights and stiffness base stiffness characteristics are analyzed. The wind-bracing element is of the type SHS140x8. Stiffness characteristics of the frame are simplified by considering single degree of freedom and only take into account the contribution to stiffness from the wind-bracing elements.

Element cross-section
$A := 40.04 \cdot 10^2\ mm^2$

Young's modulus
$E := 200\ GPa$

Reduction factor
$\varphi := 0.05, 0.06 .. 1$

Default height
$H_1 := 20.5\ m$

Reduced height
$H_2 := 10.5\ m$

Width of wind-bracing system
$B := 24\ m$

Length of wind-bracing elements (default height)
$L_1 := \sqrt{{H_1}^2 + B^2} = 31.56\ m$

Length of wind-bracing elements (reduced height)
$L_2 := \sqrt{{H_2}^2 + B^2} = 26.2\ m$

Various mass cases

$M_{snow} := 144109\ kg$ $\quad M_{int.column} := 3008\ kg$ $\quad M_{truss} := 54860\ kg$ $\quad M_{beam} := 4114\ kg$

$M_{roof} := 299965\ kg$ $\quad M_{column} := 34440\ kg$ $\quad M_{add.truss} := 3591\ kg$ $\quad M_{windb} := 1982\ kg$

Total mass

$$M := M_{snow} + M_{roof} + M_{int.column} + M_{column} + M_{truss} + M_{add.truss} + M_{beam} + M_{windb} = 546069\ kg$$

Calculation of angle (default height)
from ground to wb-element
$\alpha_1 := \operatorname{atan}\left(\frac{H_1}{B}\right) = 0.707$

Calculation of angle (reduced height)
from ground to wb-element
$\alpha_2 := \operatorname{atan}\left(\frac{H_2}{B}\right) = 0.412$

Stiffness char (def stiff, def height)
simplfied SDOF considered
$K_1(\varphi) := 2 \cdot \frac{A \cdot E}{L_1} \cdot \cos(\alpha_1) \cdot \varphi$

Stiffness char (def stiff, red height)
simplfied SDOF considered
$K_2(\varphi) := 2 \cdot \frac{A \cdot E}{L_2} \cdot \cos(\alpha_2) \cdot \varphi$

Stiffness char (red stiff, def height)
simplfied SDOF considered
$K_3(\varphi) := \frac{A \cdot E}{L_1} \cdot \cos(\alpha_1) \cdot \varphi$

Stiffness char (red stiff, red height)
simplfied SDOF considered
$K_4(\varphi) := \frac{A \cdot E}{L_2} \cdot \cos(\alpha_2) \cdot \varphi$

Period & freq calc (def stiff, def height)
$T_{e1}(\varphi) := 2\ \pi \cdot \sqrt{\frac{M}{K_1(\varphi)}}$ $\quad f_{e1}(\varphi) := \frac{1}{T_{e1}(\varphi)}$

Period & freq calc (def stiff, red height)
$T_{e2}(\varphi) := 2\ \pi \cdot \sqrt{\frac{M}{K_2(\varphi)}}$ $\quad f_{e2}(\varphi) := \frac{1}{T_{e2}(\varphi)}$

Period & freq calc (red stiff, def height)
$T_{e3}(\varphi) := 2\ \pi \cdot \sqrt{\frac{M}{K_3(\varphi)}}$ $\quad f_{e3}(\varphi) := \frac{1}{T_{e3}(\varphi)}$

Period & freq calc (red stiff, red height)
$T_{e4}(\varphi) := 2\ \pi \cdot \sqrt{\frac{M}{K_4(\varphi)}}$ $\quad f_{e4}(\varphi) := \frac{1}{T_{e4}(\varphi)}$

Values of period (def stiff, def height)
$T_{e1}(\varphi) = \begin{bmatrix} 3.343 \\ 3.052 \\ \vdots \end{bmatrix} s$

Values of period (def stiff, red height)
$T_{e2}(\varphi) = \begin{bmatrix} 2.774 \\ 2.533 \\ \vdots \end{bmatrix} s$

Values of period (red stiff, def height)
$T_{e3}(\varphi) = \begin{bmatrix} 4.728 \\ 4.316 \\ \vdots \end{bmatrix} s$

Values of period (red stiff, red height)
$T_{e4}(\varphi) = \begin{bmatrix} 3.924 \\ 3.582 \\ \vdots \end{bmatrix} s$

Characteristic values for response spectrum (called Thesis)
Ground type E · Seismic class IV

$S := 1.55$ $\quad T_B := 0.15\ s$ $\quad T_C := 0.40\ s$ $\quad T_D := 1.60\ s$ $\quad \gamma_1 := 1.0$

$a_{g40Hz} := 0.80\ \frac{m}{s^2}$ $\quad a_{gR} := 0.8 \cdot a_{g40Hz} = 0.64\ \frac{m}{s^2}$ $\quad a_g := a_{gR} \cdot \gamma_1 = 0.64\ \frac{m}{s^2}$

$\beta := 0.2$ $\quad q := 1.5$

Calculation of design spectrum Sd(T)
(def stiff, def height)

$$S_{d1}(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{e1}(\varphi)}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T_{e1}(\varphi) \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T_{e1}(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T_{e1}(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T_{e1}(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T_{e1}(\varphi)^2}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$$S_{d1}(\varphi) = \begin{bmatrix} 0.128 \\ 0.128 \\ 0.133 \\ \vdots \end{bmatrix} \frac{m}{s^2}$$

$$\frac{S_{d1}(\varphi)}{a_g} = \begin{bmatrix} 0.2 \\ 0.2 \\ 0.2071 \\ \vdots \end{bmatrix}$$

Calculation of design spectrum Sd(T)
(def stiff, red height)

$$S_{d2}(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{e2}(\varphi)}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T_{e2}(\varphi) \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T_{e2}(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T_{e2}(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T_{e2}(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T_{e2}(\varphi)^2}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$$S_{d2}(\varphi) = \begin{bmatrix} 0.137 \\ 0.165 \\ 0.192 \\ \vdots \end{bmatrix} \frac{m}{s^2}$$

$$\frac{S_{d2}(\varphi)}{a_g} = \begin{bmatrix} 0.2148 \\ 0.2577 \\ 0.3007 \\ \vdots \end{bmatrix}$$

Calculation of design spectrum Sd(T)
(red stiff, def height)

$$S_{d3}(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{e3}(\varphi)}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T_{e3}(\varphi) \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T_{e3}(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T_{e3}(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T_{e3}(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T_{e3}(\varphi)^2}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$$S_{d3}(\varphi) = \begin{bmatrix} 0.128 \\ 0.128 \\ 0.128 \\ \vdots \end{bmatrix} \frac{m}{s^2}$$

$$\frac{S_{d3}(\varphi)}{a_g} = \begin{bmatrix} 0.2 \\ 0.2 \\ 0.2 \\ \vdots \end{bmatrix}$$

Calculation of design spectrum Sd(T)
(red stiff, red height)

$$S_{d4}(\varphi) := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_{e4}(\varphi)}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T_{e4}(\varphi) \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T_{e4}(\varphi) \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C}{T_{e4}(\varphi)}, \beta \cdot a_g\right) & \text{else if } T_C \le T_{e4}(\varphi) \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \frac{T_C \cdot T_D}{T_{e4}(\varphi)^2}, \beta \cdot a_g\right) & \text{else} \end{cases}$$

$$S_{d4}(\varphi) = \begin{bmatrix} 0.128 \\ 0.128 \\ 0.128 \\ \vdots \end{bmatrix} \frac{m}{s^2}$$

$$\frac{S_{d4}(\varphi)}{a_g} = \begin{bmatrix} 0.2 \\ 0.2 \\ 0.2 \\ \vdots \end{bmatrix}$$

Calculation of base force F (def stiff, def height)

$F_1(\varphi) := S_{d1}(\varphi) \cdot M$

$$F_1(\varphi) = \begin{bmatrix} 69.897 \\ 69.897 \\ 72.39 \\ \vdots \end{bmatrix} kN$$

Calculation of base force F (def stiff, red height)

$F_2(\varphi) := S_{d2}(\varphi) \cdot M$

$$F_2(\varphi) = \begin{bmatrix} 75.064 \\ 90.077 \\ 105.09 \\ \vdots \end{bmatrix} kN$$

Calculation of base force F (red stiff, def height)

$F_3(\varphi) := S_{d3}(\varphi) \cdot M$

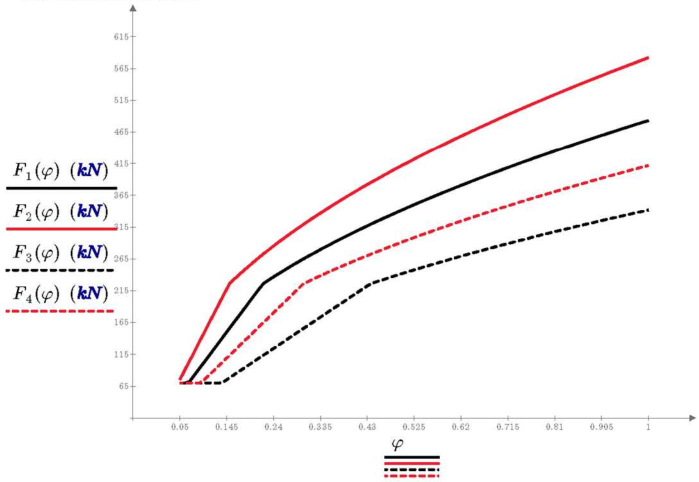
$$F_3(\varphi) = \begin{bmatrix} 69.897 \\ 69.897 \\ 69.897 \\ \vdots \end{bmatrix} kN$$

Calculation of base force F (red stiff, red height)

$F_4(\varphi) := S_{d4}(\varphi) \cdot M$

$$F_4(\varphi) = \begin{bmatrix} 69.897 \\ 69.897 \\ 69.897 \\ \vdots \end{bmatrix} kN$$

Plot of the results
Force vs Stiffness reduction



Appendix B · Practical example of existing structure

B.1 Response spectrum for S_2 ground type

Multiconsult

Notat RIG 02

Oppdrag:	**Hansa Borg bryggerier, Sarpsborg**	Dato:	**3. desember 2013**
Emne:	**Grunntype og responsspekter**	Oppdr.nr.:	**511944**
Til:	**Veidekke Entreprenør AS**		**Tommy Hansen**
Kopi:			
Utarbeidet av:	**Espen Fiskum**	Sign.:	Espen Fiskum
Kontrollert av:	**Roy Michel Nalbant**	Sign.:	Roy M. Nalbant
Godkjent av:	**Dag Erik Julsheim**	Sign.:	Dag Erik Julsheim

Sammendrag:

Det foreliggende notat omhandler grunnresponsanalyser og vurdering av responsspekter som RIB kan bruke for jordskjelvanalyser.

EK8-1 (NS-EN 1998-1:2004+NA:2008) gir verdier for forsterkningsfaktor (S) og responsspekter for grunntype A til E. I dette tilfellet vurderes grunntype S_2 da sensitive leirer (og spesielt kvikkleirer) havner i denne grunntypen hvor det kreves spesielle undersøkelser for å fastslå den seismiske påvirkningen.

Vurderingene er gjort basert på grunnresponsanalyser og med utforming av anbefalt elastisk responsspekter med samme form som i EK8-1 pkt. 3.2.2.2. Verdier for parameterne som beskriver anbefalt elastisk responsspekter er vist i tabellen under.

Grunntype	S (-)	T_B (s)	T_C (s)	T_D (s)
S_2	1,7	0,10	0,65	0,65

Innholdsfortegnelse

1. Innledning

Veidekke Entreprenør AS skal på oppdrag av Hansa Borg Bryggerier se på muligheter for oppføring av nytt lagerbygg ved Hansa Borg Bryggerier i Sarpsborg. Det nye lagerbygget er planlagt vest for eksisterende bygg

Multiconsult AS er engasjert for å utføre supplerende grunnundersøkelser og som rådgivende ingeniør i geoteknikk.

Foreliggende notat omhandler grunnresponsanalyser og vurdering av responsspekter som RIB kan benytte for jordskjelvanalyser.

2. Beregningsforutsetninger

2.1 Generelt

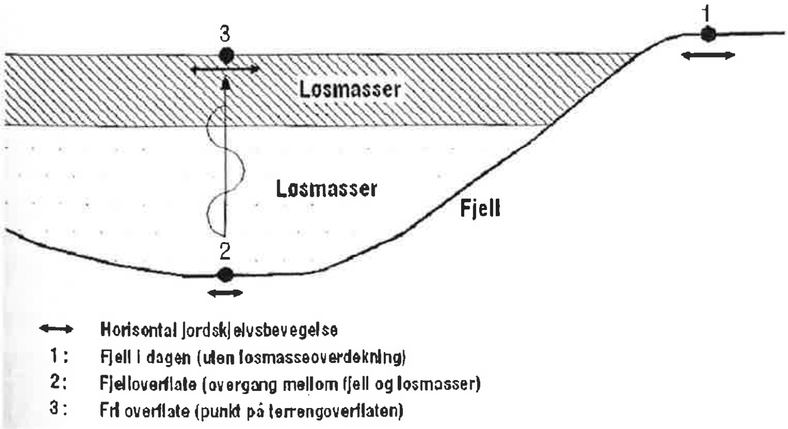
For grunnundersøkelser vises det til rapport nr. 511944-2 datert 14.11.2013 [1].

For vurdering av seismisk påkjenning av bygg gjelder spesielt EK8-1 (NS-EN 1998-1:2004+NA:2008 [2]).

EK8-1 gir verdier for forsterkningsfaktor (S) og responsspekter for grunntype A til E. I dette tilfellet vurderes grunntype S_2 da sensitive leirer (og spesielt kvikkleirer) havner i denne grunntypen. Med referanse til pkt. 3.1.2(4)P i EK8-1 kreves spesielle undersøkelser for å fastslå den seismiske påvirkningen for grunntype S_2.

2.2 Analysemetode

For å vurdere den seismiske påkjenningen av bygget er det utført grunnresponsanalyser ("ground response analysis") ved bruk av beregningsprogrammet EERA [3]. Figur 1 viser skjematisk presentasjon av en grunnresponsanalyse.



Figur 1 Skjematisk presentasjon av grunnresponsanalyse [4].

Prinsippet ved en grunnresponsanalyse er å benytte akselerasjonstidshistorier tilpasset aktuelt geografisk område ved berg (pkt. 2 i Figur 1) hvor bølger forplantes som skjærbølger gjennom løsmasser opp til terreng (pkt. 3 i Figur 1). Responsen ved terreng er såkalt "free field ground response", se for eksempel Kramer (1996) [5].

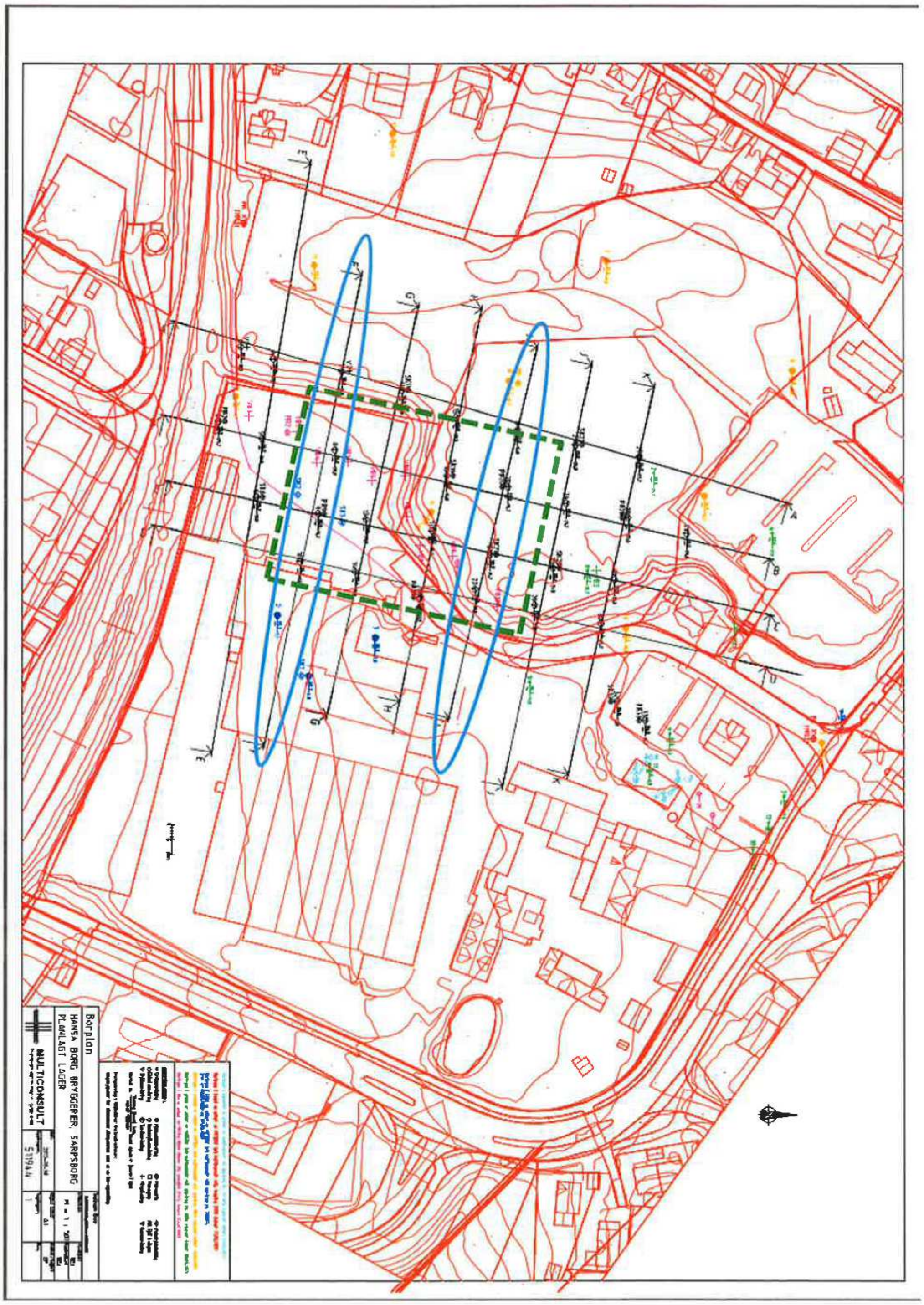
Nøkkelinput i beregningene er representative akselerasjonstidshistorier ved berg under løsmassene. Videre representeres jordens ikke-lineære oppførsel ved å modellere skjærstivhet ved lav tøyning (G_{max}) med ikke-lineær nedgradering av denne (G/G_{max}) og dempning som funksjon av skjærtøyning (γ).

2.3 Topografi og grunnforhold

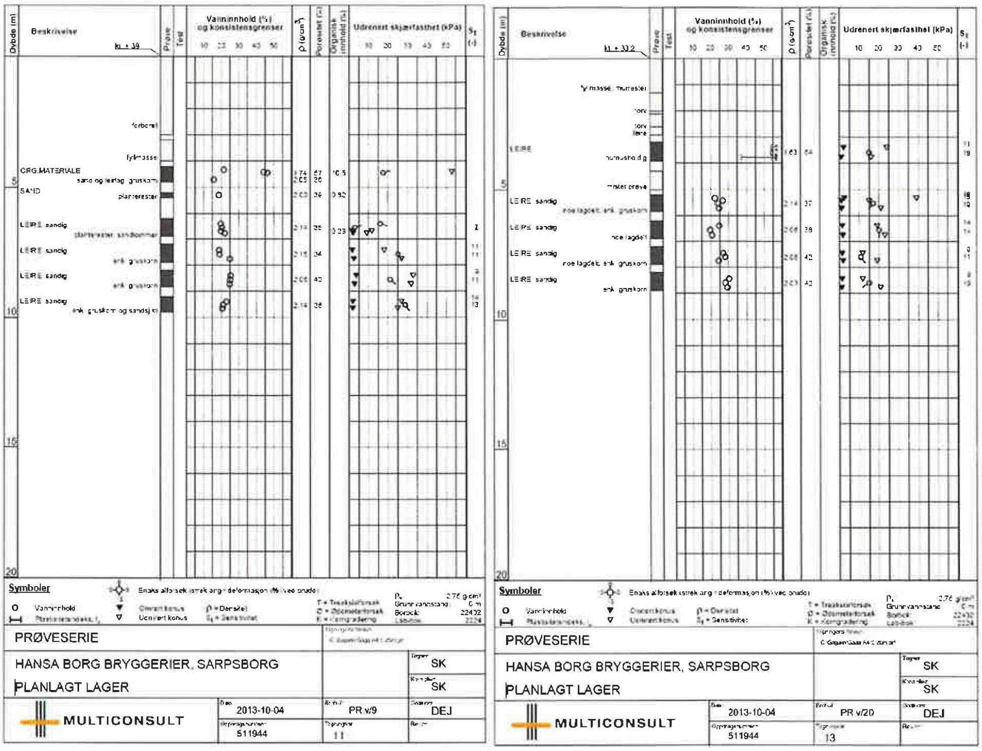
Grunnforholdene i området er beskrevet i vår rapport nr. 511944-2, datert 14.11.2013 [1]. Nedenfor følger sammendrag av grunnforholdene fra nevnte rapport:

> *På store deler av området er det oppfylte masser øverst. I sydøst er tykkelsen rundt 4 – 5 m (området med gasstank og lagring) mens den i myrområdet er rundt 1 m (fra rundt null til muligens over 3 m på det meste). Fyllmassene ser ut til å være "ymse" fyllmasser med leire, sand, grus og stein med innhold av humus.*
>
> *Under fyllinga i sydøst er det bløt til middels fast leire med varierende innhold av silt og sand samt enkelte gruskorn. Massene her inneholder ikke kvikkleire eller sprøbruddsmateriale.*
>
> *Mot vest og nord er det bløt til middels kvikkleire eller leire med sprøbruddsegenskaper under topplaget. I syd er topplaget noe fylling og derunder silt. Lenger nord er det torv under fyllinga og laget under torva, kvikkleire, inneholder øverst mye organisk materiale.*
>
> *Innblandingsforsøk med kalk og sement i den noe leirige og siltige torven, viser neglisjerbar effekt med innblanding ved 100 og 120 kg/m^3. Tilsvarende innblandingsforsøk i leira/kvikkleira med en del humus, viser relativ liten styrkeøkning med innblanding med 80 kg/m^3, og en del bedre med innblanding med 100 kg/m^3.*
>
> *Generelt viser boringene at det i syd er middels fast til fast lagrede masser fra rundt kote 25, ca. 10 – 15 m dybde. Dybden til dette laget øker mot nord, og er på rundt kote 15 nord på området, tilsvarende ca. 15 – 18 m dybde.*

Figur 2 viser borplanen med planlagt nybygg samt markering av to representative profiler for området. Videre er det vist to representative prøveserier på Figur 3.



Figur 2: Borplan grunnundersøkelser med fotavtrykk av planlagte nybygg i grønt, de 2 vurderte profilene er også markert med blått. [1]



Figur 3:Prøveserie PR. v/9 og PR.v/20 [1].

3. Valgte inngangsparametere for grunnresponsanalyse

3.1 Beregningsprofil og jordparametere

Den udrenerte direkte skjærfastheten, su_D, forutsettes ikke lavere enn 0,2*p_0'. Merk at oppgitte styrkeparametere er benyttet for å estimere skjærstivhet ved lave tøyninger, og må ikke benyttes til øvrige geotekniske problemstillinger uten en nærmere vurdering.

Grunnvannstanden er forutsatt i 2,5 m dybde for profil F-F og 0,5 m dybde for profil I-I.

Tabell 1 Jordprofil 1 benyttet for grunnresponsanalyser (18 m dybde til berg, snitt F-F)

Lag	Dybde (m)	Jordvekt (kN/m^3)	φ (^{o})	s_{uD} (kPa)	I_p (%)	k_{2max} (-)	v_s (m/s)
Fylling/tørrskorpe	0-3	18,0	35	-	-	50	119-178
Organisk materiale	3-4	17,0	-	-	-	-	50
Sandig leire	4-11	21	-	12-26	15	-	97-140
Morene	11-18	21	38	-	-	70	255-281
Fjell	-	26,0	-	-	-	-	1200

Tabell 2 Jordprofil 2 benyttet for grunnresponsanalyser (20 m dybde til berg, snitt I-I)

Lag	Dybde (m)	Jordvekt (kN/m³)	φ (°)	s_{uD} (kPa)	I_p (%)	k_{2max} (-)	v_s (m/s)
Sprengstein	0-3	19,0	42	-	-	70	135-176
Torv	3-5	16	-	-	-	-	50
Sandig leire (St<20)	5-9	20	-	12-16	15	-	98-113
Morene	9-20	20	38	-	-	70	234-282
Fjell	-	26,0	-	-	-	-	1200

Følgende korrelasjoner er benyttet for beregning av skjærstivhet ved lav tøyning (G_{max}):
For tørrskorpen/fyllmasser, sprengstein og morene (Seed et al. [6]):
$G_{max} = 220 * k_{2max} * \sqrt{\sigma_m'}$ (omskrevet til SI-enheter (kPa))

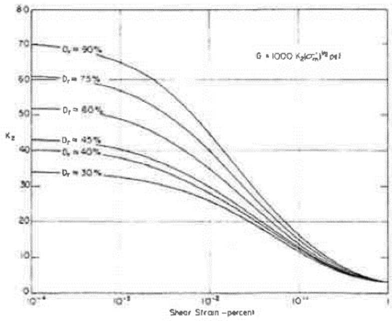


FIG. 4 SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES

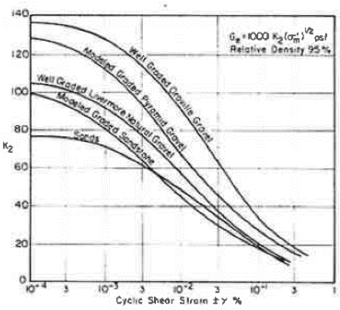


FIG. 16 COMPARISON OF SHEAR MODULI FOR GRAVELLY SOILS AND SANDS AT D_r = 95%

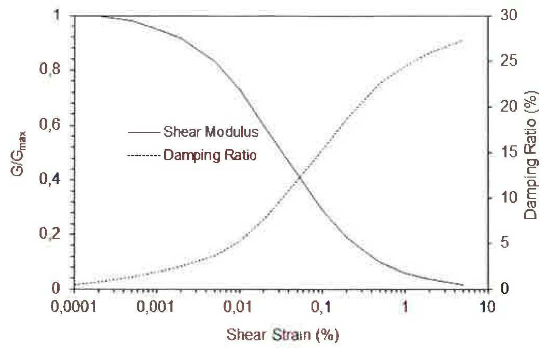
Figur 4 Skjærmodul for sand og friksjonsmateriale med ulik lagringstetthet [6]

For G_{max} i leire (Larsson & Mulabdic [7]):

$\frac{G_{max}}{s_{uD}} = \frac{208}{I_p} + 250$ (I_p > 10 %)

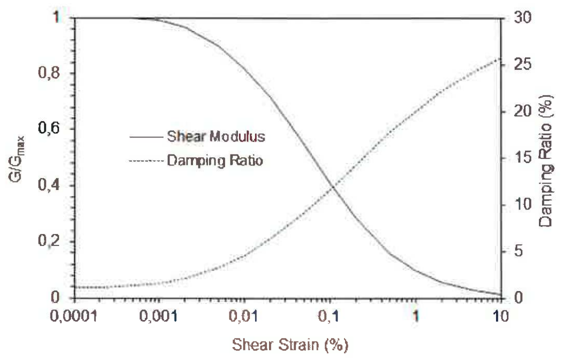
For degradering av skjærstivhet (G/G_{max}) og dempning som funksjon av skjærtøyning (γ) er følgende benyttet.

For tørrskorpeleire/fyllmasse, sprengstein og morene:



Figur 5 Kurver for degradering av skjærstivhet og dempningsforhold benyttet for friksjonsmateriale fylling/tørrskorpe, sprengstein og morene (Seed et. al. [6])

For leire og kvikkleire/meget sensitiv leire:



Figur 6 Kurver for degradering av skjærstivhet og dempningsforhold benyttet for leire med I_p= 15 % (Vucetic & Dobry [8])

For torva har vi ikke kunne finne noen erfaringsverdier for vurdering av skjærstivhet og dempningsforhold. Det er i forbindelse med beregningene utført parameterstudie der vi har vurdert ulike dempingsfaktorer for torv og også sett på effekten av ulike skjærbølge hastigheter (v_s). Vs verdier på 30 m/s. ga urealistisk lave spektralakselerasjoner. For Vs 50 m/s og høyere ga ikke utslagene betydelige variasjoner og av tilnærmet samme karakter. Det er derfor i beregningene benyttet en Vs på 50 m/s. Som dempingsfaktor vil det være naturlig å anta at torva har høyere dempning enn omkringliggende jord. Vi har i beregningene benyttet en dempingsfaktor på 10 % for torva, omkringliggende jord har dempning på opp mot 5 %. Det er også utført beregninger for en dempingsfaktor på 20 % for å kunne vurdere hvor stort utslag dette ville gi på akselerasjonene.

3.2 Akselerasjonstidshistorier

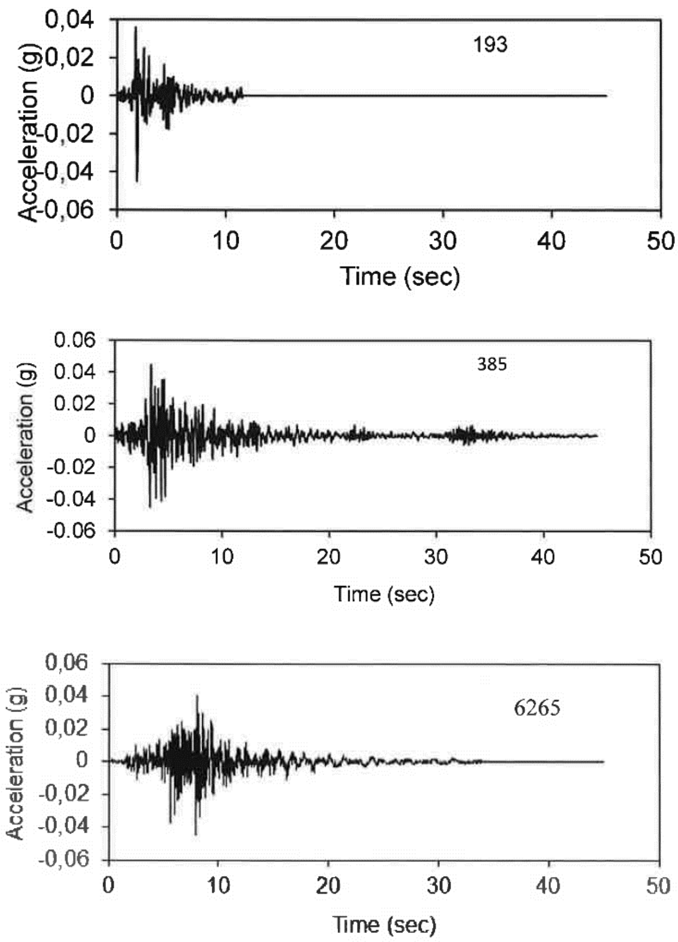
Det er benyttet såkalt "target spektrum" kompatible akselerasjonstidshistorier ved berg som input til grunnresponsanalysene. Spekteret for disse akselerasjonshistoriene er tilpasset norske forhold ved å justere de ved bruk av anbefalt spekter for grunntype A (ved berg) og skalert med referansespissverdi for berggrunnens akselerasjon for grunntype A (a_{gR}) iht. EK8-1 [2]. For Sarpsborg gjelder $a_{g40Hz} = 0,55$ m/s^2 => $a_{gR} = 0,8 * a_{g40Hz} = 0,8*0,55$ m/s^2 = 0,045g.

Det er benyttet 3 akselerasjonstidshistorier som er nærmere beskrevet i Tabell 3.

Tabell 3 Informasjon om benyttede akselerasjonstidshistorier

Waveform ID	Earthquake ID	Station ID	Earthquake Name	Date	Mw	Fault Mechanism	Epicentral Distance (km)	Site class
193	91	ST64	Montenegro	09.04.1979	5,4	Thrust	15	A
385	176	ST155	Lazio Abruzzo (aftershock)	11.05.1984	5,5	Normal	15	A
6265	1635	ST2494	South Iceland	17.06.2000	6,5	Strike slip	29	A

Figur 7 viser benyttede akselerasjonstidshistorier ved berg.



Figur 7 Benyttede akselerasjonstidshistorier ved berg

4. Resultater

4.1 Elastisk responsspekter

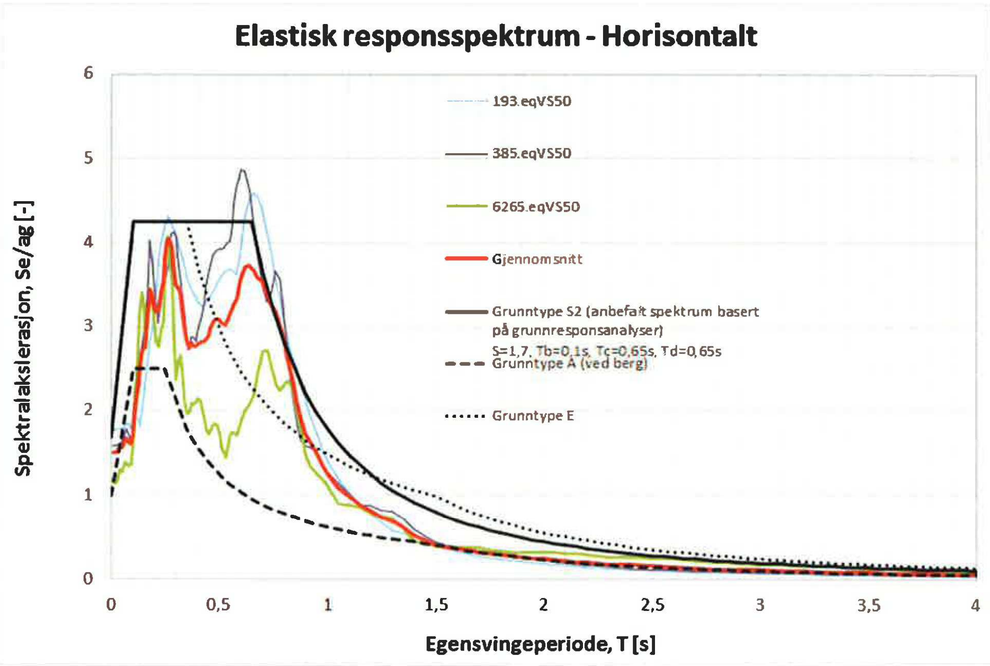
For jordprofil 1 (representativ for F-F) beregnes gjennomsnittlig skjærbølgehastighet for løsmasser over fjell for profilet til:

$$v_{s,18} = \frac{\sum h_i}{\sum_{i=1}^{N} \frac{h_i}{v_i}} = \frac{18}{\frac{3}{151} + \frac{1}{50} + \frac{7}{119} + \frac{7}{268}} \; 144 \; {}^{m}/{}_{s}$$

Grunntypen defineres som E iht. EK8-1 Tabell NA.3.1, hvor det er ca. 5-20 m til fjell hvor løsmassene over fjell har skjærbølgehastighet av type C eller D, dvs. 130-360 m/s.

For jordprofil 2 (representativ for I-I) defineres grunntype S2 da leiren vil defineres å ha sprøbruddegenskaper, dvs. sensitivitet > 15 og omrørt skjærfasthet < 2 kPa. For jordprofil 2 utføres dermed grunnresponsanalyser («free-field ground response analyses») hvor den lokale responsen ved terrengoverflaten beregnes, dvs. ved pkt. 3 i Figur 1.

Figur 8 viser elastisk responsspektrum ved terreng for analyser med de 3 tilfellene av akselerasjonstidshistorier for jordprofil 2. Utforming av anbefalt elastisk responsspekter er vurdert med utgangspunkt i begge jordprofilene og med samme form som i EK8-1 pkt. 3.2.2.2. Verdier for parameterne som beskriver anbefalt elastisk responsspekter er vist i Tabell 4.



Figur 8: Elastiske responsspektrum for tomta sammen med anbefalt spektrum.

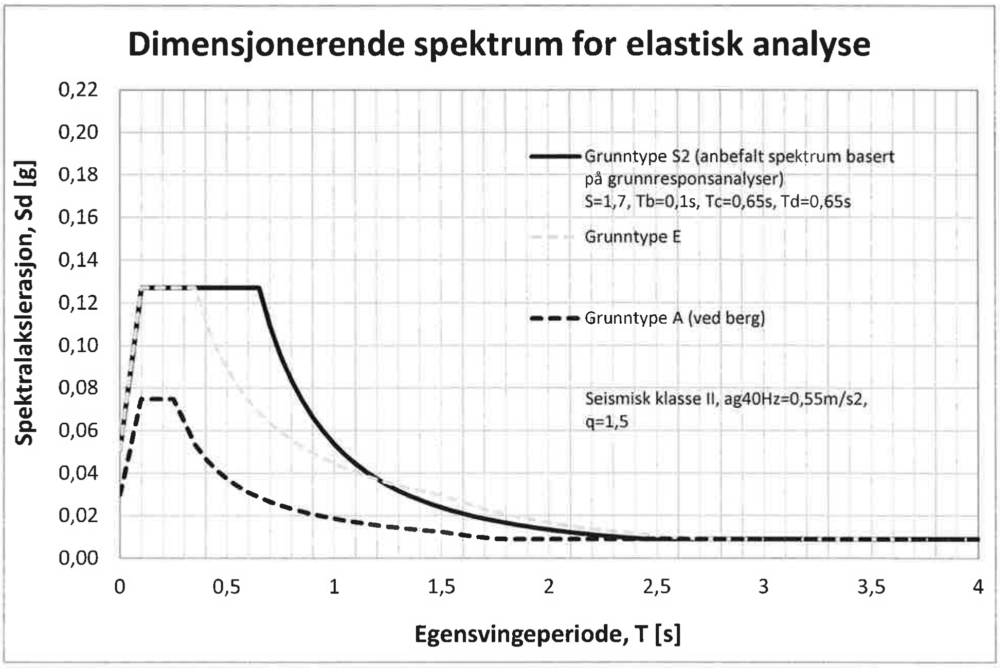
Tabell 4: Verdier for parametere som beskriver anbefalt elastisk responsspekter.

Grunntype	S (-)	T_B (s)	T_C (s)	T_D (s)
S_2	1,7	0,10	0,65	0,65

4.2 Dimensjonerende spektrum for elastisk analyse

For Sarpsborg gjelder $a_{g40Hz} = 0,55\ m/s^2$. Figur 9 viser dimensjonerende responsspektrum iht. pkt. 3.2.2.5 i EK8-1 som kan benyttes av RIB. Merk at dette tar utgangpunkt i konstruksjonsfaktor q = 1,5 og seismisk klasse II.

Iht. Tabell NA.4(902) i EK8-1 velges normalt industribygg i seismisk klasse II. Konstruksjonsfaktor og seismisk klasse må RIB vurdere og fastsette.



Figur 9: Dimensjonerende spektrum for elastisk analyse (forutsatt q=1,5 og seismisk klasse II)

Referanser

[1] Multiconsult AS, «Rapport 511944-2,» 14.11.2013.

[2] Eurokode 8, del 1. Prosjektering av konstruksjoner for seismisk påvirkning. Del 1:Allmenne regler, seismiske laster og regler for bygninger. NS-EN 1998-1:2004+NA:2008..

[3] J. e. A. Bardet, «EERA, A computer Program for Equivalent-linear Earthquake site Response Analyses of Layered Soil Deposits,» University of Southern California, Department of Civil Engineering, August 2000.

[4] A. M. Kaynia, Forfatter, *Geoteknsike aspekter vedjordskjelvmodellering.* [Performance]. Presentasjon holdt på Geoteknikkdagen 24. november 2006, Oslo.

[5] S. L. Kramer, Geotechnical Earthquake Engineering, University of Washington: Prentice Hall International Series in Civil Engineering and Engineering Mechanics, 1996.

[6] H. Seed, R. Wong, I. Idriss og T. Tokimatsu, «Moduli and Damping Factors for Dynamic Response Analyses. Report EERC 84-14», Earthquake Research Center, University of California, Berkeley., 1984.

[7] R. Larsson og M. Mulabdic, «Shear Moduli in Scandinavian Clays, Measurement of Initial Shear Modulus with Seismic Cones, Empirical Correlaations for the Initial Shear Modulus in Clay,» SGI rapport No 40. Stetens Geotekniska Institut, Linköping, 1991.

[8] M. &. D. R. Vucetic, «Effect of Soil Plasticity on Cyclic response.,» *Journal of Geotechbical Engineering, ASCE, Vol. 117 (1),* pp. 89-107.

[9] «RIF-veileder (2010). Dimensjonering for jordskjelv. Veileder til bruk av NS-EN 1998-1:2004+NA:2008. September 2010».

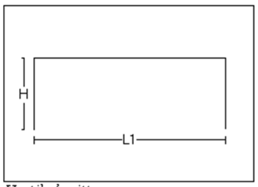
B.2 Wind loads

Master thesis

Tittel			Side
Wind loads for ULS analysis			1
Prosjekt	Ordre	Sign	Dato
Project 14-112		SB	13-05-2015

Dataprogram: LastBeregning versjon 6.2.1 Laget av Sletten Byggdata AS
Standard NS-EN 1991-1-4: Vindlaster
Data er lagret på fil:

1. Geometri



Vertikalsnitt

H	12000	mm
L1	64900	mm

Byggets lengde, L2: 84000 mm
Takvinkel : 0,00 (grader)

Parapet: hp/h=0,025

2. Vindhastighet

Fylke: Østfold Kommune: Sarpsborg Referansevindhastighet: 24 m/s
Byggested, høyde over havet (m): 36,5 Calt: 1
Returperiode (år):50 Cprob: 1
Årstidsfaktoren, Cseason: 1 hele året
Vindretning (region):Bruker retningsfaktoren C-ret: 1
Basisvindhastighet: 24 m/s
Høyde Z over grunnivået: 22 m

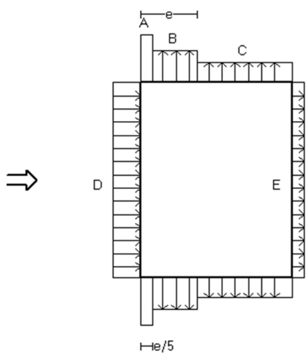
BYGGESTEDETS TERRENGDATA
Terrengruhetskategori III: Sammenhengende småhusbebyggelse industriområder eller skogsområder.
Terrengruhetsfaktoren Kt: 0,22 Ruhetslengden Zo (m): 0,3 Zmin (m): 8 Vm (m/s): 22,68 Cr: 0,94

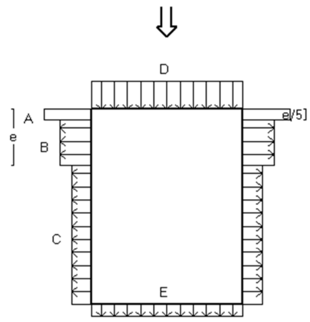
TOPOGRAFI: Ingen topografisk påvirkning.
Terrengformfaktor Co(z): 1 Turbulensfaktor Ki: 1

Vkast: 36,78 m/s
Qkast: 0,845 kN/m2

3. Yttervegger

3.1 Utvendig vindlast





Vindretning 0 grader. e=24000 mm *Vindretning 90 grader. e=24000 mm*

Vindinnfallsretning på 0 grader.

	A	B	C	D	E
Formfaktor Cpe,10	-1,20	-0,80	-0,50	0,70	-0,30
Utvendig last (kN/m2)	-1,01	-0,68	-0,42	0,59	-0,25
Formfaktor Cpe,1	-1,40	-1,10	-0,50	1,00	-0,30
Utvendig last (kN/m2)	-1,18	-0,93	-0,42	0,85	-0,25
Utstrekning (mm)	4800	19200	40900	84000	84000

Vindinnfallsretning på 90 grader.

	A	B	C	D	E
Formfaktor Cpe,10	-1,20	-0,80	-0,50	0,70	-0,30
Utvendig last (kN/m2)	-1,01	-0,68	-0,42	0,59	-0,25
Formfaktor Cpe,1	-1,40	-1,10	-0,50	1,00	-0,30
Utvendig last (kN/m2)	-1,18	-0,93	-0,42	0,85	-0,25
Utstrekning (mm)	4800	19200	60000	64900	64900

Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.

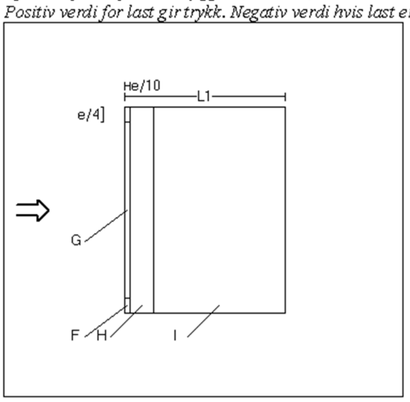
3.2 Innvendig vindlast

Bygning uten dominerende vindfasade
Beregn innvendig vindlast for u=0.2 overtrykk og u=-0.3 (undertrykk)

	Undertrykk	Overtrykk
Formfaktor	-0,30	0,20
Innvendig last (kN/m2)	-0,25	0,17

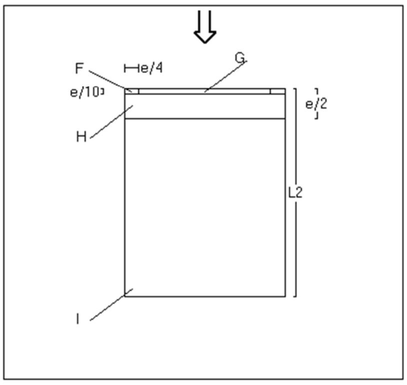
4 Overside av tak

Taktype: Flatt tak
L1=64900 mm   L2=84000 mm
Cpe,10 Gjelder for hele bygget. (> =10m2)
Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.



Utstrekning (mm)
e=24000
e/4=6000
e/10=2400

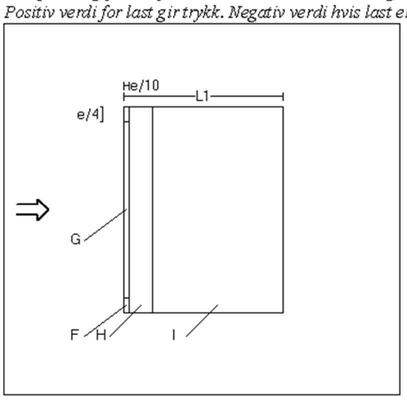
	Cpe,10	Last (kN/m2)	Hor.projeksjon (mm)
F	-1,60	-1,35	6000x2400
G	-1,10	-0,93	72000x2400
H	-0,70	-0,59	84000x9600
I	+/-0,20	+/-0,17	84000x52900



Utstrekning (mm)
e=24000
e/4=6000
e/10=2400

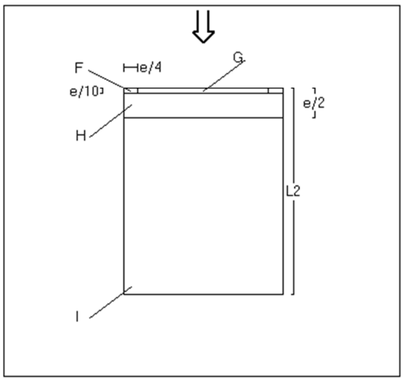
	Cpe,10	Last (kN/m2)	Hor.projeksjon (mm)
F	-1,60	-1,35	6000x2400
G	-1,10	-0,93	52900x2400
H	-0,70	-0,59	64900x9600
I	+/-0,20	+/-0,17	64900x72000

Taktype: Flatt tak
L1=64900 mm   L2=84000 mm
Cpe,1 Gjelder for en lokal flate på 1m2. Benyttes ved dimensjonering av limfuger, spikring, båndstål o.l.
Interpoleringsformel for belastet areal A mellom 1 og 10 m2 : $Cpe = Cpe,1 + (Cpe,10 - Cpe,1) * log_{10}A$
Positiv verdi for last gir trykk. Negativ verdi hvis last er sug.



Utstrekning (mm)
e=24000
e/4=6000
e/10=2400

	Cpe,1	Last (kN/m2)	Hor.projeksjon(mm)
F	-2,20	-1,86	6000x2400
G	-1,80	-1,52	72000x2400
H	-1,20	-1,01	84000x9600
I	+/-0,20	+/-0,17	84000x52900



Utstrekning (mm)
e=24000
e/4=6000
e/10=2400

	Cpe,1	Last (kN/m2)	Hor.projeksjon(mm)
F	-2,20	-1,86	6000x2400
G	-1,80	-1,52	52900x2400
H	-1,20	-1,01	64900x9600
I	+/-0,20	+/-0,17	64900x72000

B.3 Check for neglect criteria

Eurocode 8 [5] has set criteria allowing to neglect the requirements for seismic assessment for specified cases. The criteria is localized in point 3.2.1(5) in the Norwegian national annex, saying: "*It is usually not required to detect adequate safety for seismic actions according to Eurocode 8 for structures within seismic class 1, light wooden structures, in cases where* $a_g S < 0{,}05g = 0{,}49\,{}^{m}/_{s^2}$*, or in cases where* $S < 0{,}05g = 0{,}49\,{}^{m}/_{s^2}$ *calculated with construction factor q* $\leq 1{,}5$*. For Bridges within seismic class 4, it is always required to detect adequate safety for seismic actions according to Eurocode 8*".

A calculation tool is made in MathCad Prime 3.0 to check if the specific structure meets the criteria to neglect the detection of adequate safety or not.

Seismiske beregninger - Kontroll mot utelatelseskriterier

Prosjekt nr.: 14-112
Prosjekt navn: Hansa Borg

1. Beregning av dimensjonerende spektret for periode T1

1.1. Parametere for elastisk responsspektrum:

$S := 1.7$ $\quad T_B := 0.10\ s$

$T_C := 0.65\ s$ NA.3.2.2.2

$T_D := 0.65\ s$

1.2. Dimensjonerende grunnakselerasjonen for grunntype - EC 8 Figur NA.3(901):

$a_g := 0.8 \cdot 0.55\ \frac{m}{s^2} = 0.44\ \frac{m}{s^2}$ Figur NA.3(901) og NA3.2.1(4)

1.3. Konstruksjonensfaktor:

$q := 1.5$ Tabell 6.1

1.4. Faktor for den nedre grenseverdien for det horisontale dimensjonerende spektret:

$\beta := 0.2$

NA.3.2.2.5(4)

1.5. Bygningens første egensvingeperiode for sidebevegelse i retningen som vurderes:

$T_1 := 0.917\ s$ hentet fra Etabs - vibrasjonsanalyse (material og geometrisk stivhet medtatt)
Konstruksjonen er modelert i Etabs, får resulterende svingetid T1

1.6. Dimensjonerende spektrum - EC 8 p.3.2.2.5 4(P):

$$S_d := \begin{cases} a_g \cdot S \cdot \left(\frac{2}{3} + \frac{T_1}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3}\right)\right) & \text{if } 0 \le T_1 \le T_B \\ a_g \cdot S \cdot \frac{2.5}{q} & \text{else if } T_B \le T_1 \le T_C \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C}{T_1}\right), \beta \cdot a_g\right) & \text{else if } T_C \le T_1 \le T_D \\ \max\left(a_g \cdot S \cdot \frac{2.5}{q} \cdot \left(\frac{T_C \cdot T_D}{T_1^2}\right), \beta \cdot a_g\right) & \text{else} \end{cases} = 0.626\ \frac{m}{s^2}$$

(EC 8 f.3.13-3.16)

1.7. Kontrol mot utelatelseskriterier. - NA.3.2.1(5)P

1.7.1. Seismisk klasse (U1)

$SK := 2$

$$U1 := \text{if } SK = 1 \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“ikke OK”} \quad = \text{“ikke OK”}$$

1.7.2. Betingelse agS<0,05g=0,49m/s2 (U2)

$$U2 := \text{if } a_g \cdot S < 0.49 \frac{m}{s^2} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“ikke OK”} \quad = \text{“ikke OK”}$$

1.7.3. Betingelse Sd<0,05g=0,49m/s2 (U3)

$$U3 := \text{if } S_d < 0.49 \frac{m}{s^2} \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“ikke OK”} \quad = \text{“ikke OK”}$$

1.7.4. Lette trekonstruksjoner (U4)

$LT := 2$ **LT=1 hvis det er lette trekonstruksjoner, ellers LT=2**

$$U4 := \text{if } LT = 1 \;\; \| \text{“OK”} \;\; \text{else} \;\; \| \text{“ikke OK”} \quad = \text{“ikke OK”}$$

1.7.4. Oppsummering utelatelseskriterier

$U1 = \text{“ikke OK”}$
$U2 = \text{“ikke OK”}$
$U3 = \text{“ikke OK”}$
$U4 = \text{“ikke OK”}$

OBS! Hvis ett av kriterier er oppfylt, kreves det ikke påvisning av tilstrekkelig sikkerhet etter NS-EN 1998.

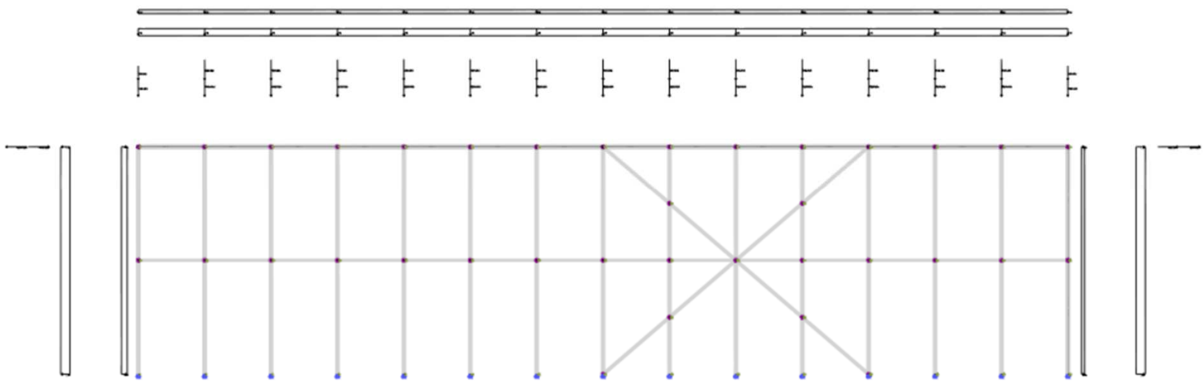
Appendix C · Analyses

C.1 ULS analysis of Project 14-112

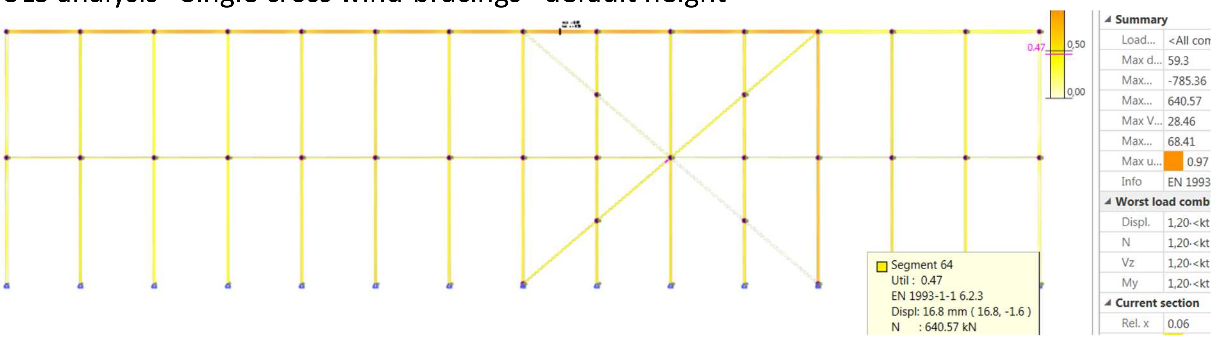
Before the response spectrum analyses, some ULS analyses of the structure are performed to determine the resulting forces from often occurring environmental actions, such as wind- and snow actions. In chapter 8.1, these loads are compared to resulting loads from earthquake actions defined by the response spectrum analyses (Appendix C.2).

C.1.1 ULS analysis · Case 1 · default height

Focus model · Single cross wind-bracings · default height

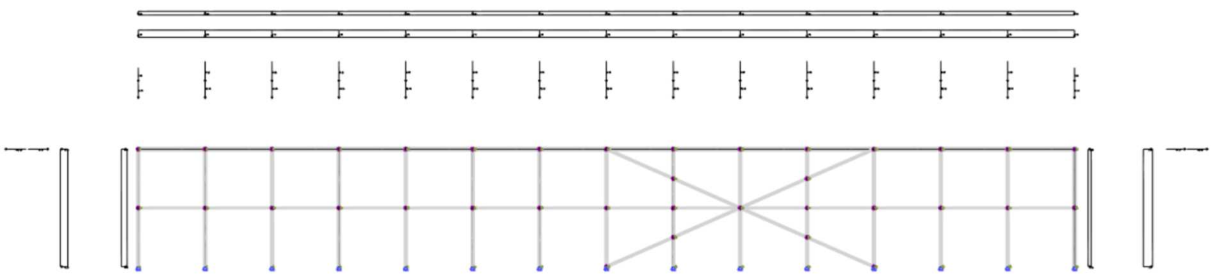


ULS analysis · Single cross wind-bracings · default height

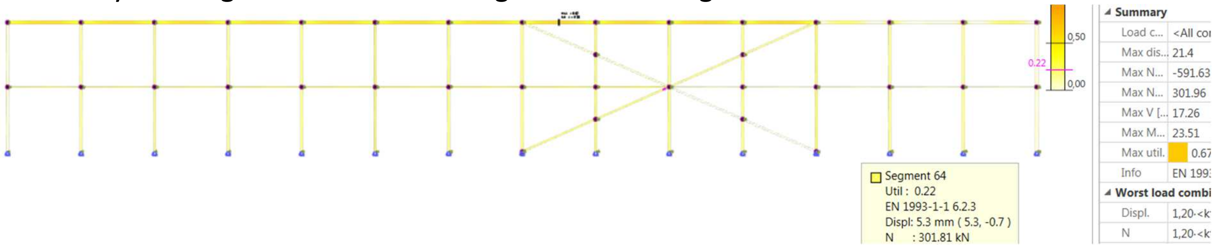


C.1.2 ULS analysis · Case 1 · reduced height

Focus model · Single cross wind-bracings · reduced height



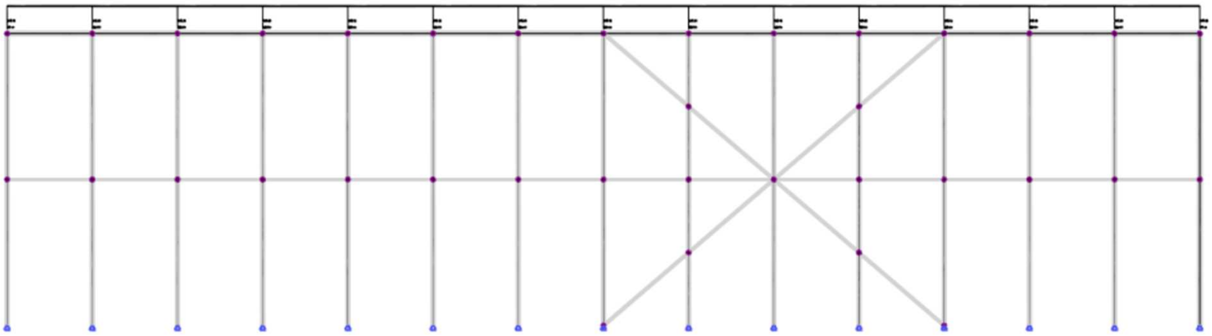
ULS analysis · Single cross wind-bracings · reduced height



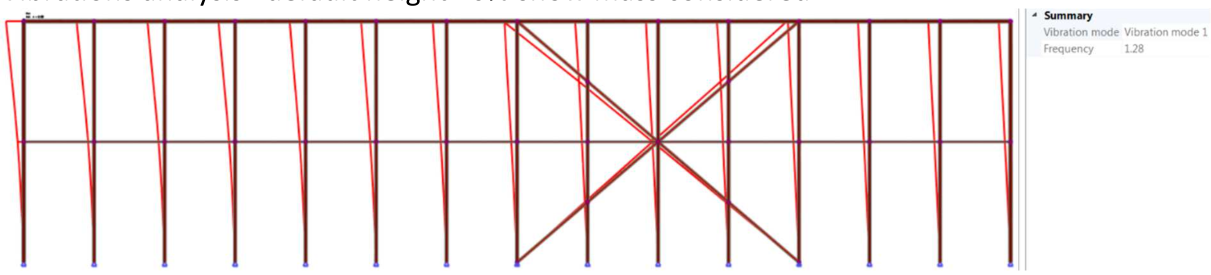
C.2 Response spectrum analysis of Project 14-112

C.2.1 Case 1 · Single cross wind-bracings (default height)

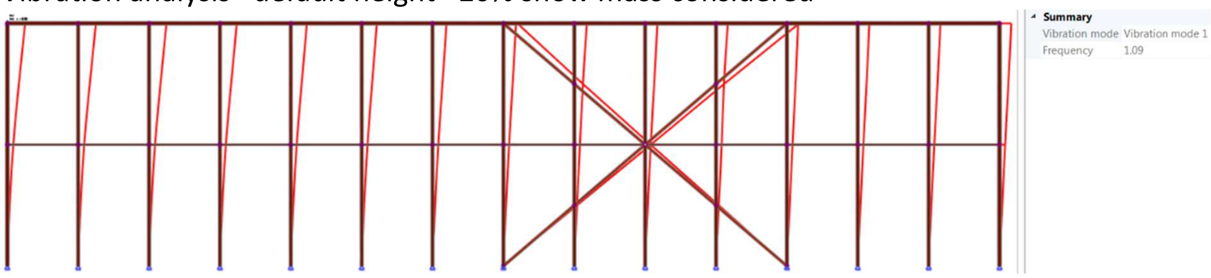
Focus model · Single cross wind-bracings · default height



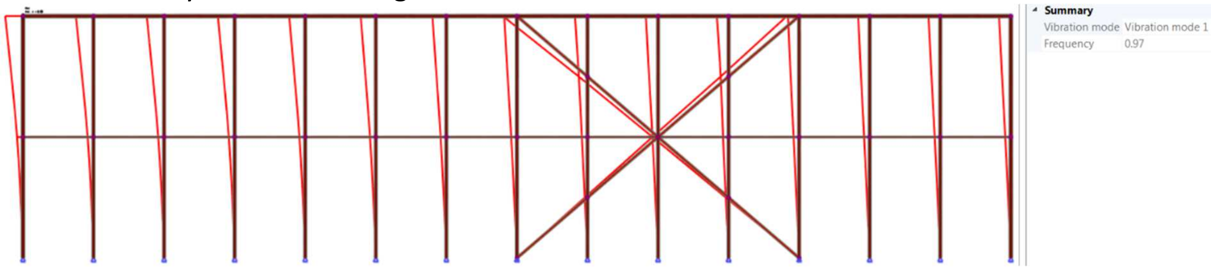
Vibrations analysis · default height · 0% snow mass considered



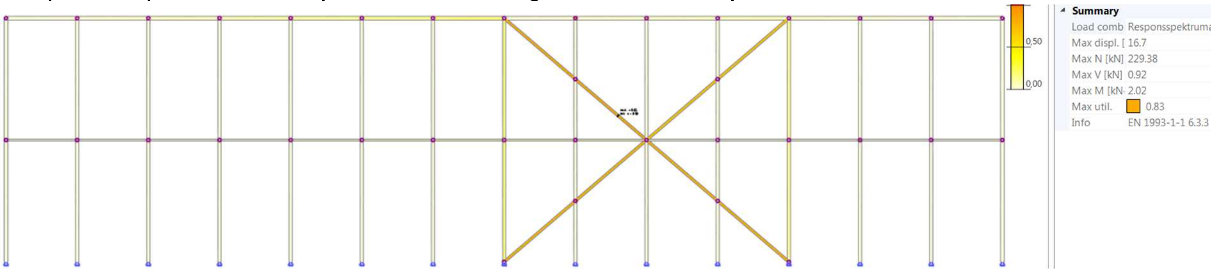
Vibration analysis · default height · 20% snow mass considered



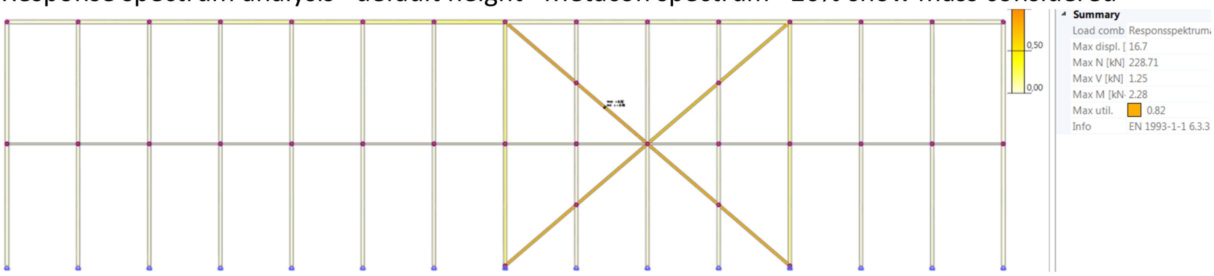
Vibration analysis · default height · 40% snow mass considered



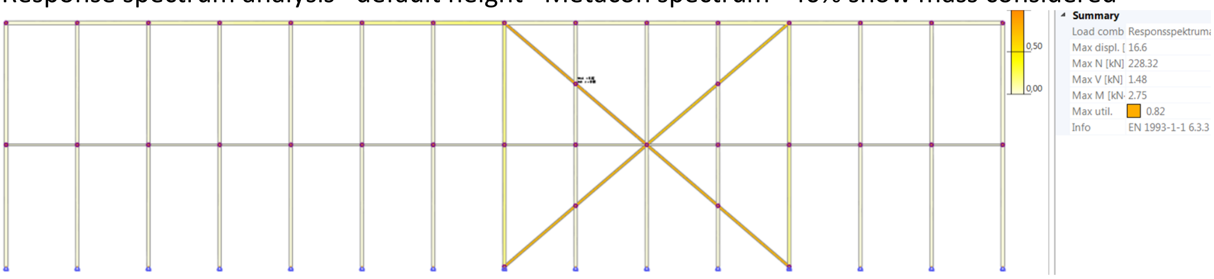
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



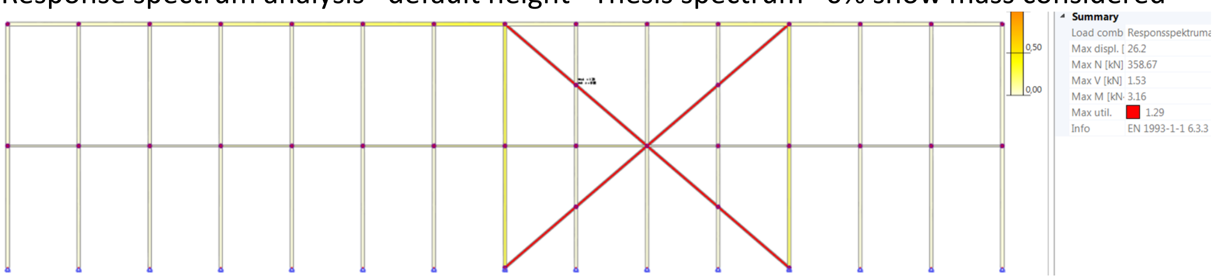
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



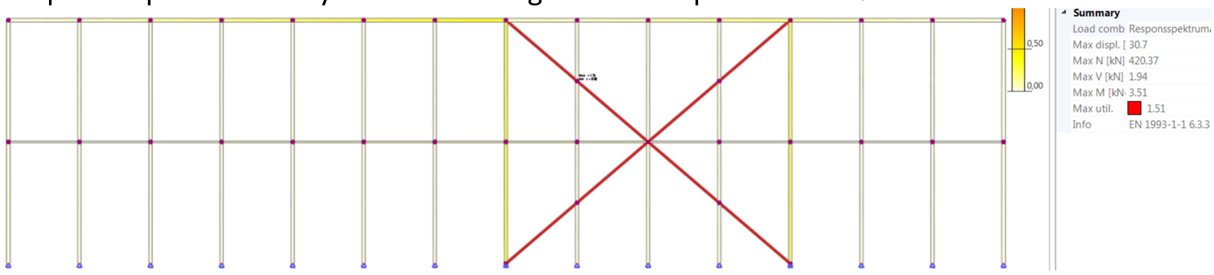
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



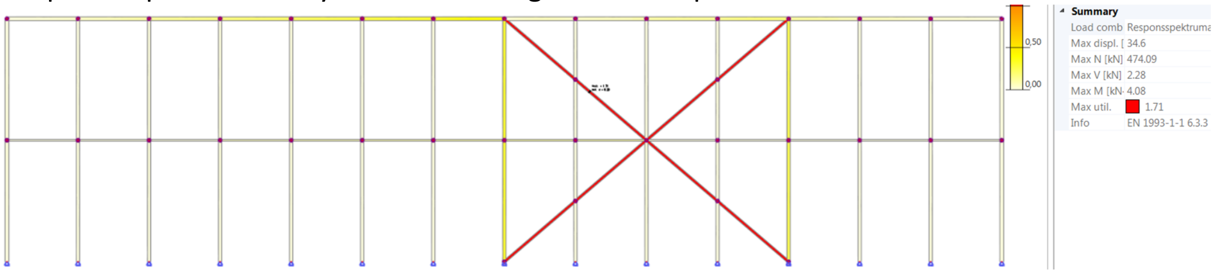
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

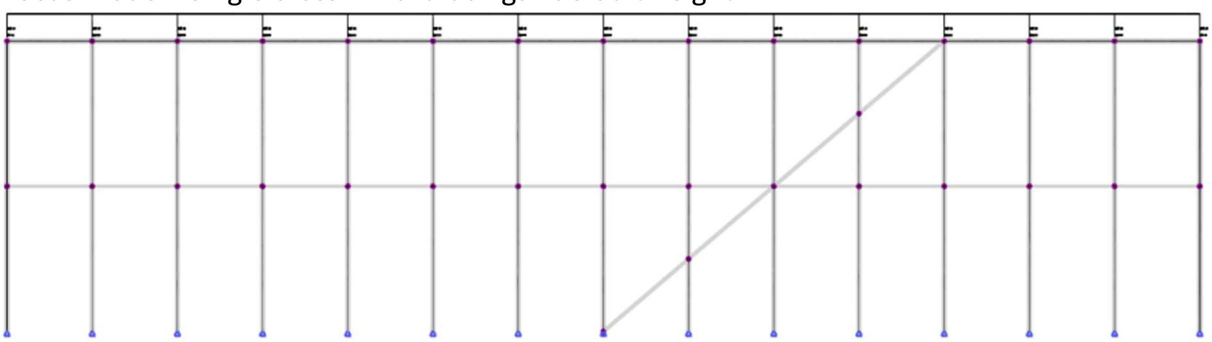


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

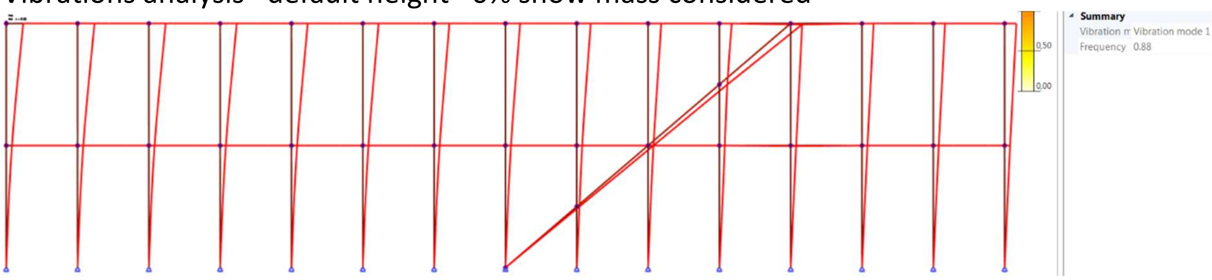


C.2.2 Case 1 · Single cross wind-bracings (default height) modified

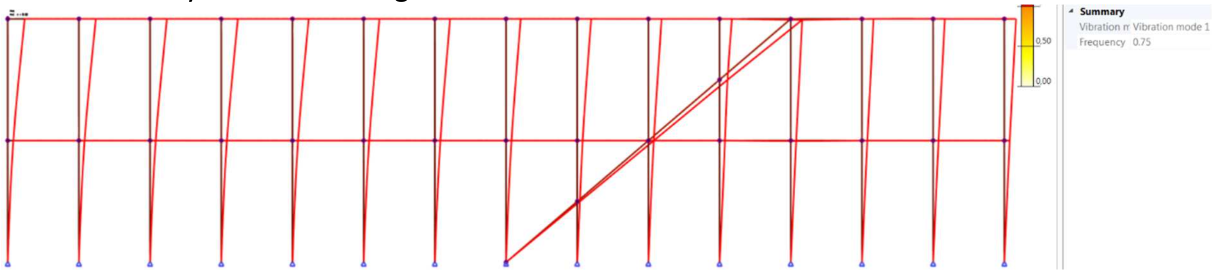
Focus model · Single cross wind-bracings · default height



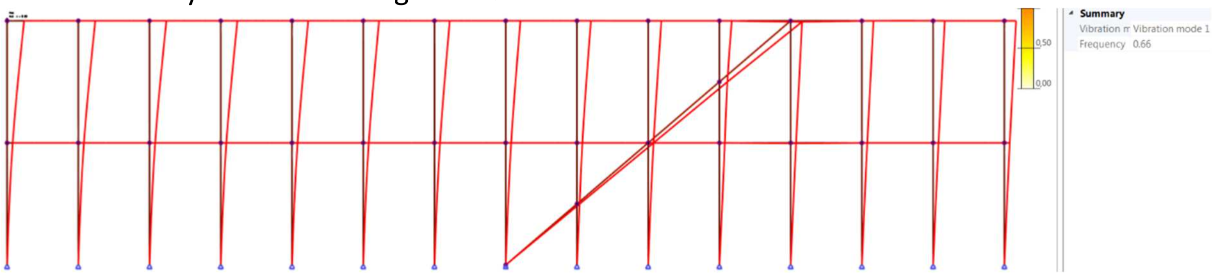
Vibrations analysis · default height · 0% snow mass considered



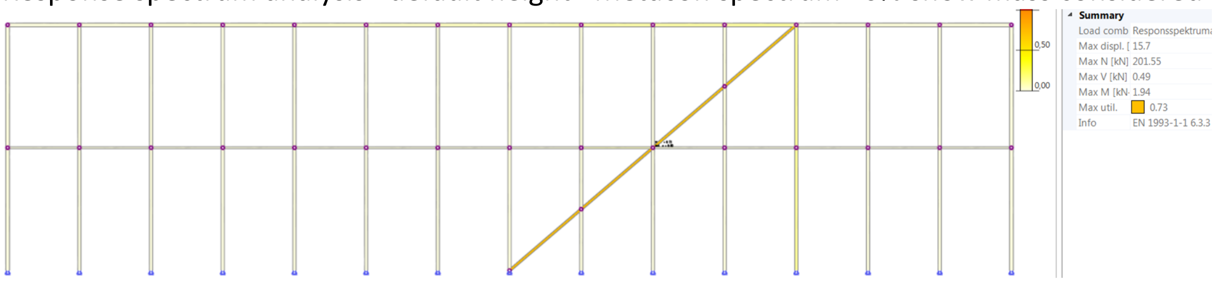
Vibration analysis · default height · 20% snow mass considered



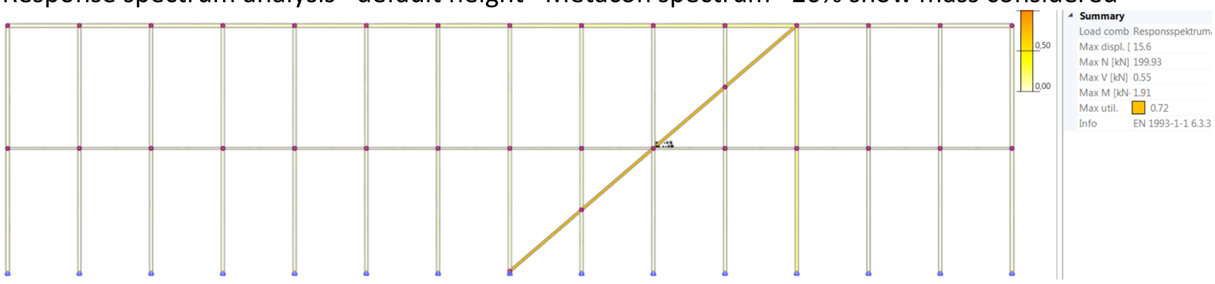
Vibration analysis · default height · 40% snow mass considered



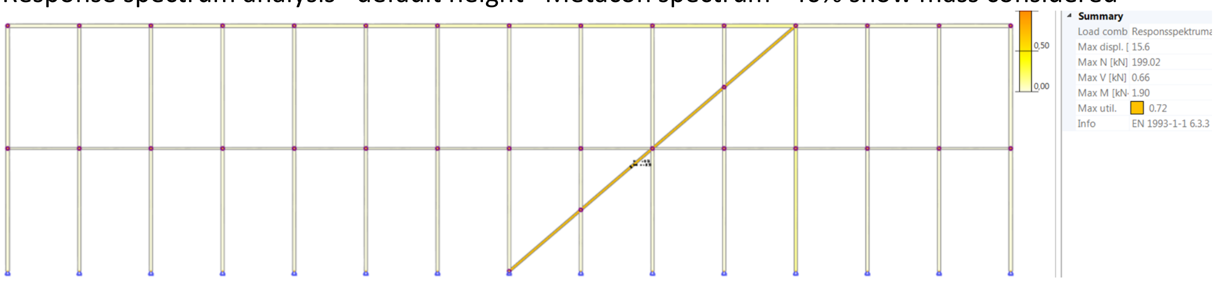
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



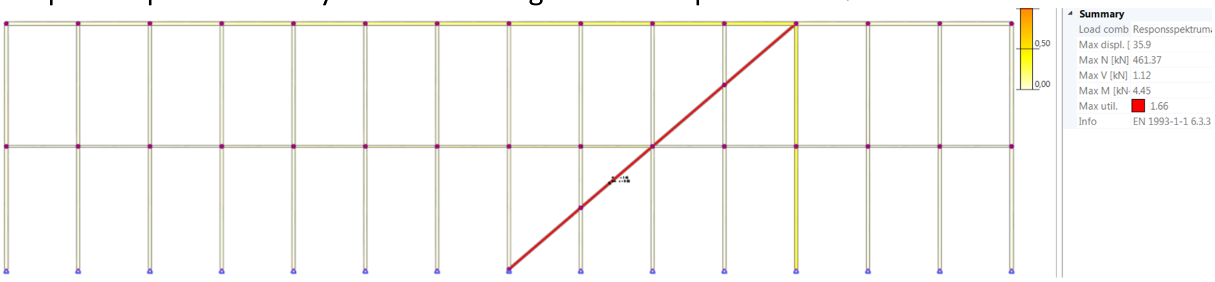
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



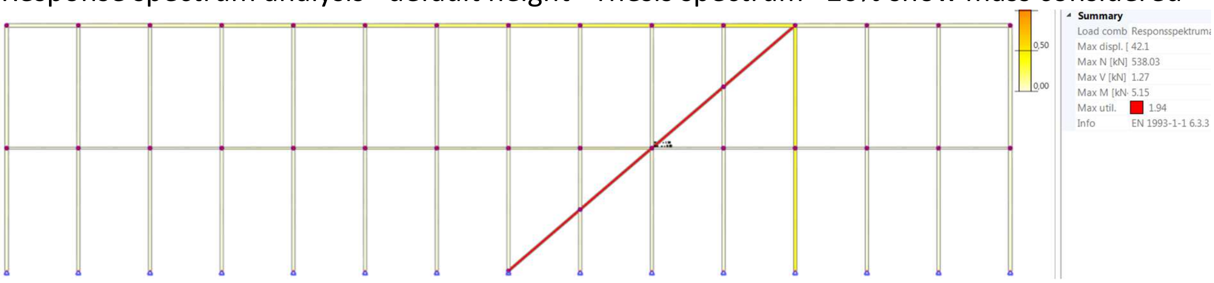
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



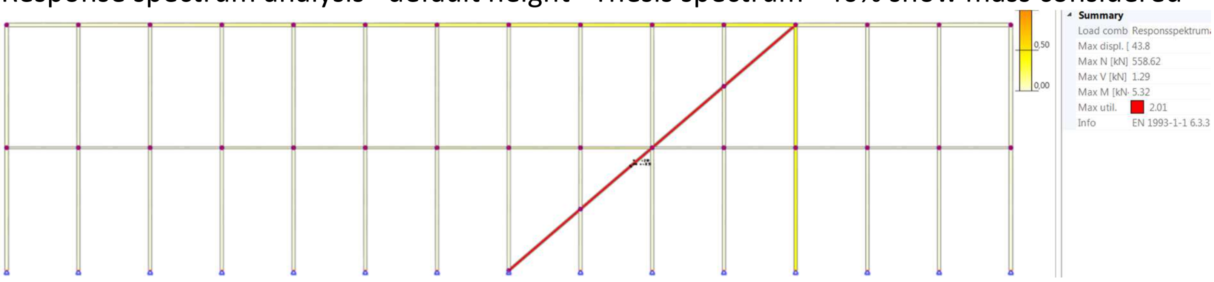
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

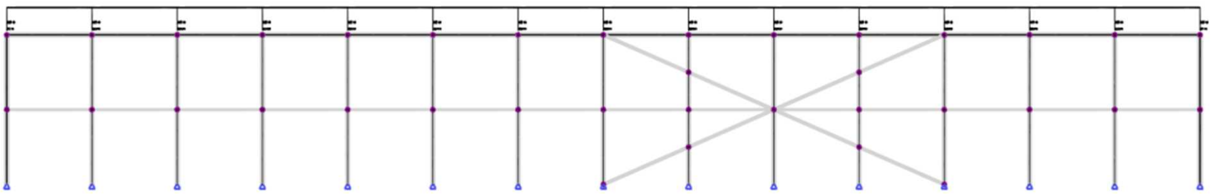


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

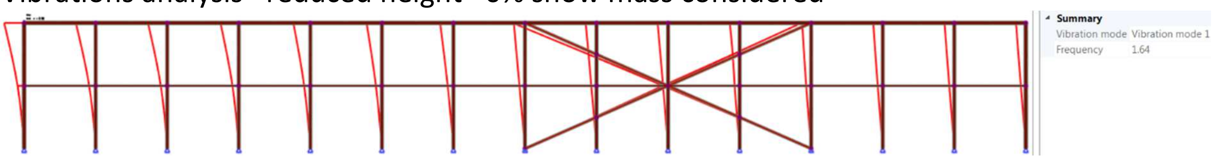


C.2.3 Case 1 · Single cross wind-bracings (reduced height)

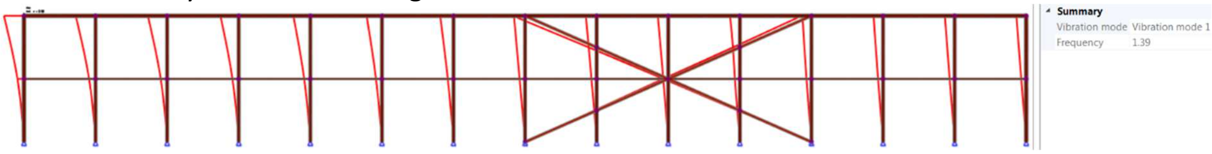
Focus model · Single cross wind-bracings · reduced height



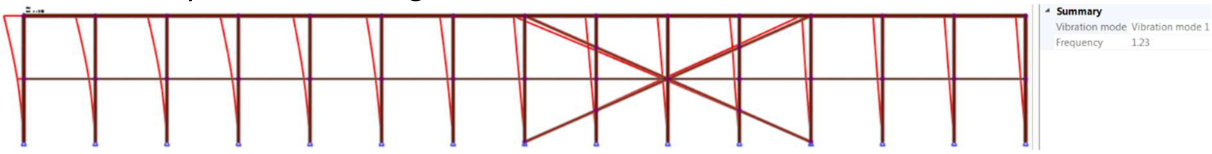
Vibrations analysis · reduced height · 0% snow mass considered



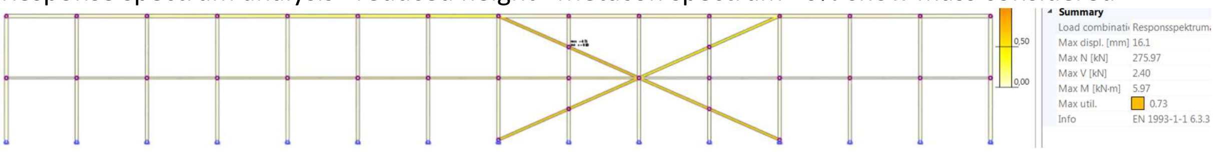
Vibration analysis · reduced height · 20% snow mass considered



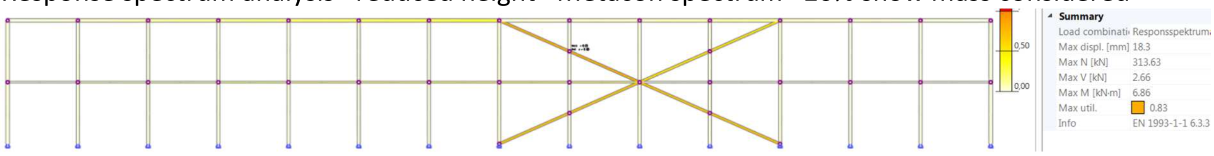
Vibration analysis · reduced height · 40% snow mass considered



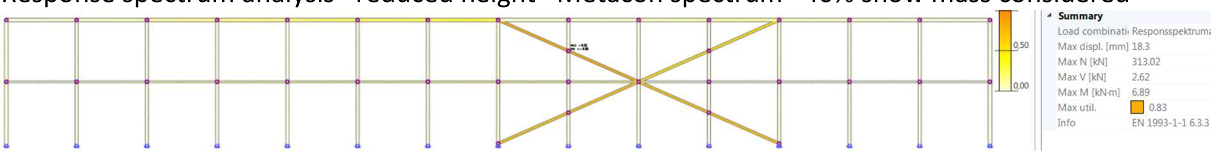
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered

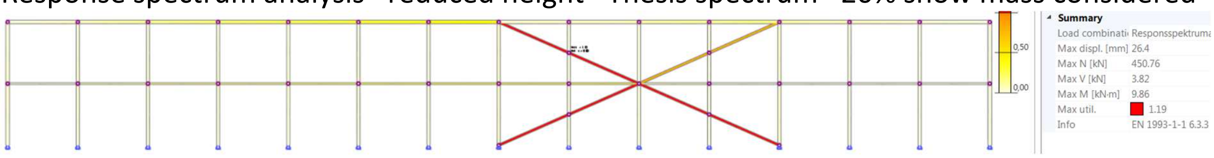


Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered

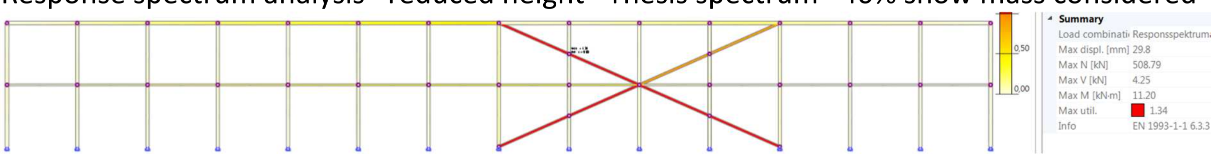


Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered

Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered

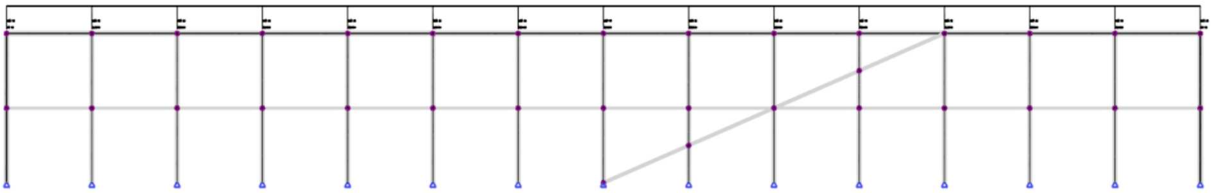


Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered

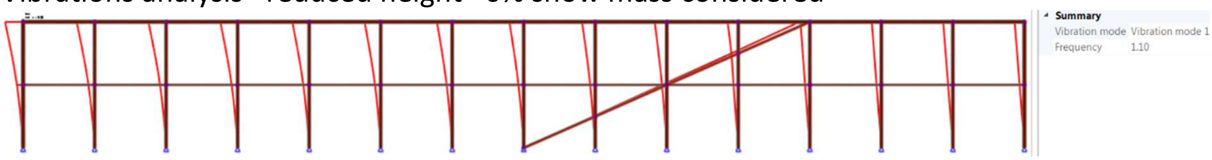


C.2.4 *Case 1 · Single cross wind-bracings (reduced height) modified*

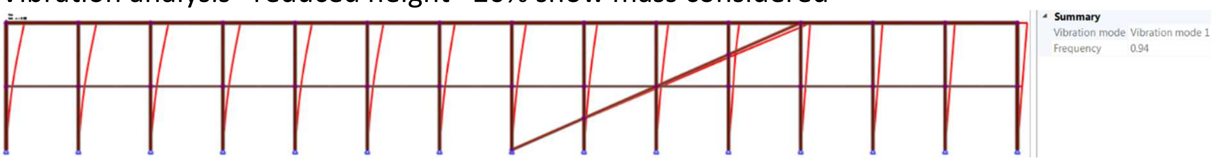
Focus model · Single cross wind-bracings · reduced height



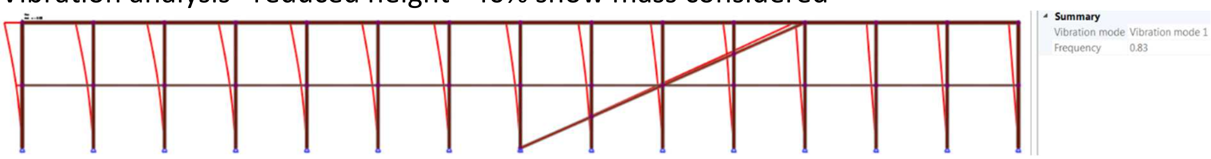
Vibrations analysis · reduced height · 0% snow mass considered



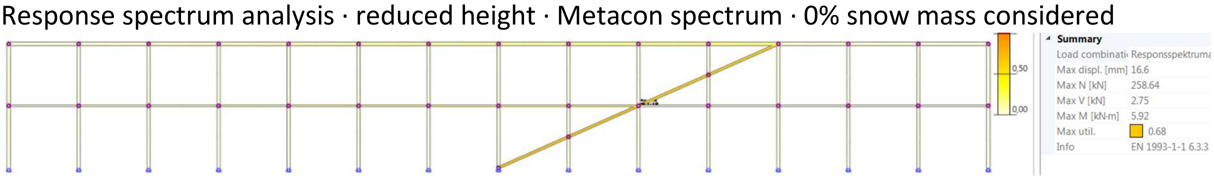
Vibration analysis · reduced height · 20% snow mass considered



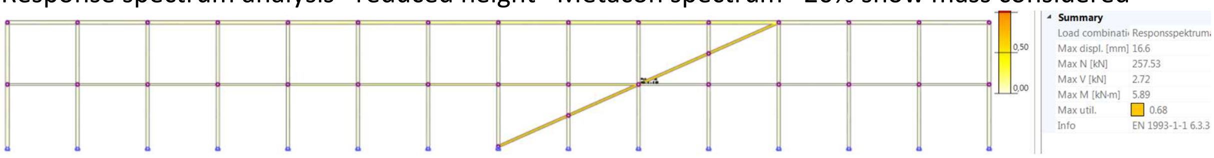
Vibration analysis · reduced height · 40% snow mass considered



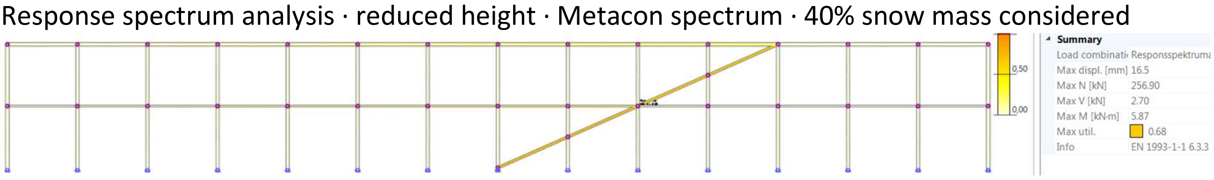
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



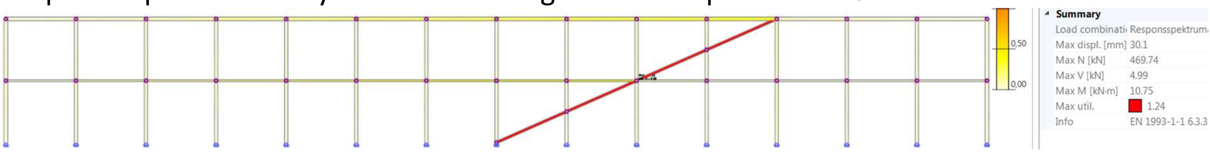
Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered



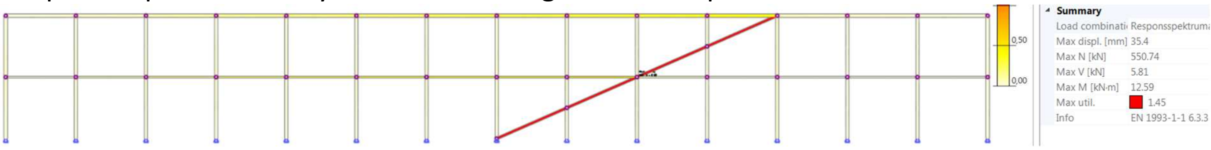
Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered



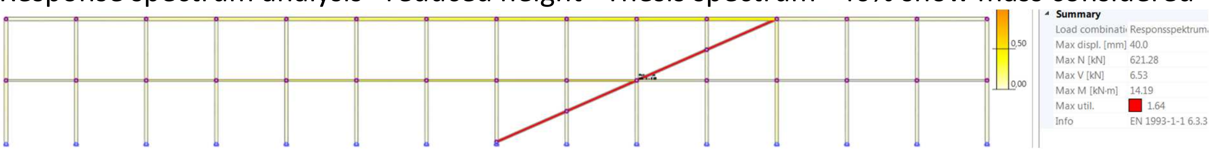
Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered

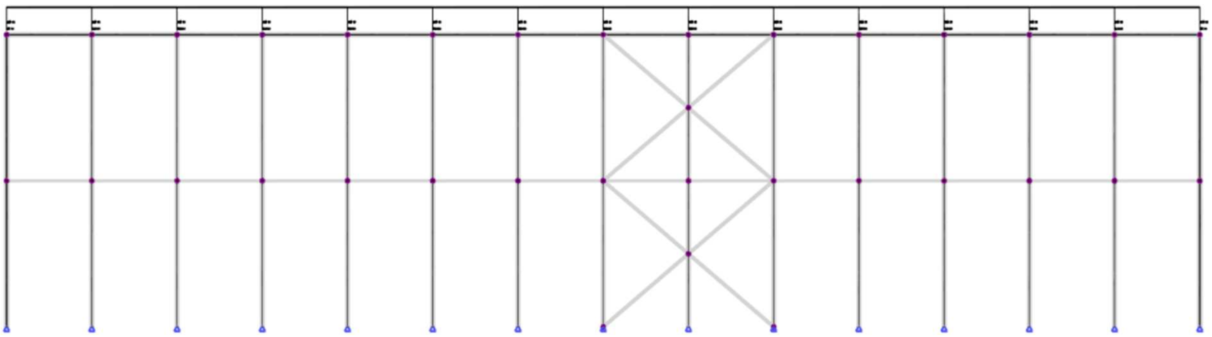


Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered

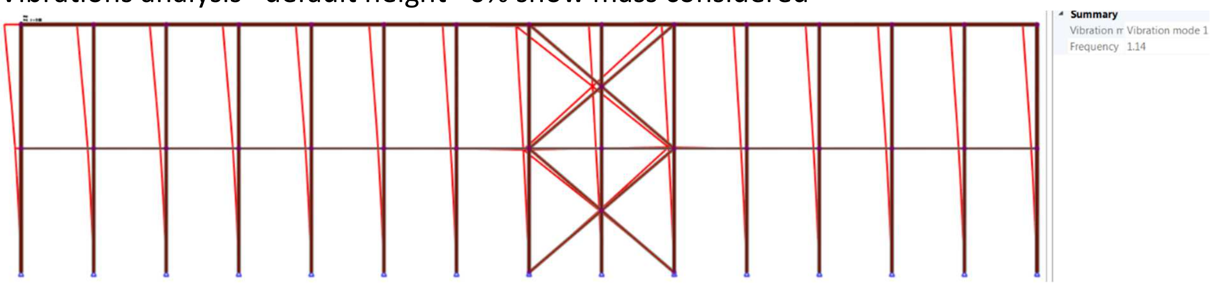


C.2.5 Case 2 · Double cross wind-bracings (default height)

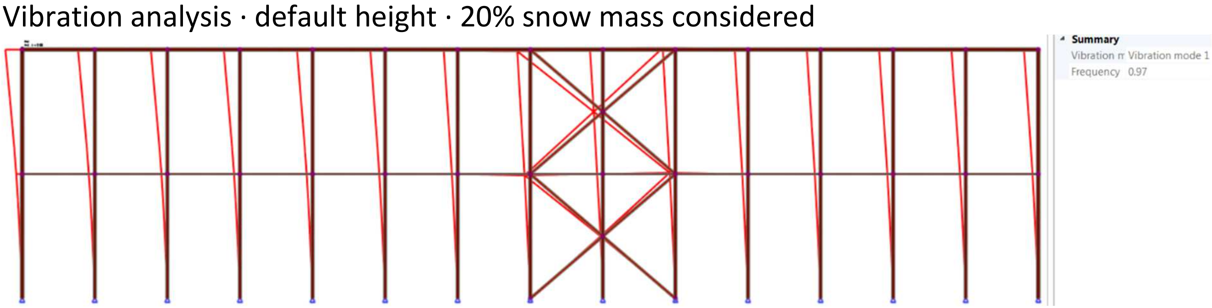
Focus model · Double cross wind-bracings · default height



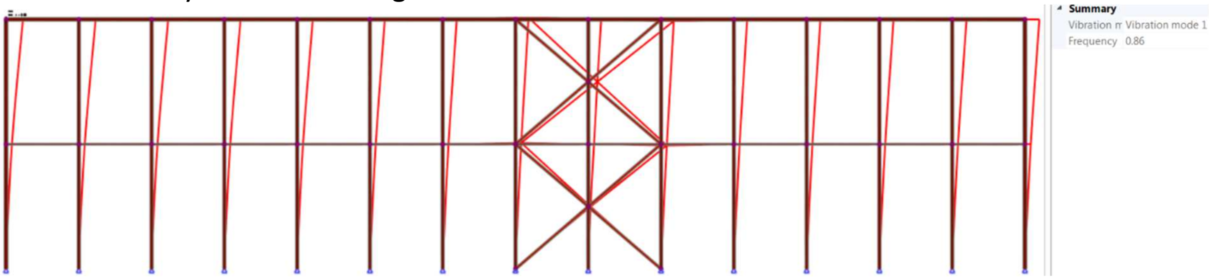
Vibrations analysis · default height · 0% snow mass considered



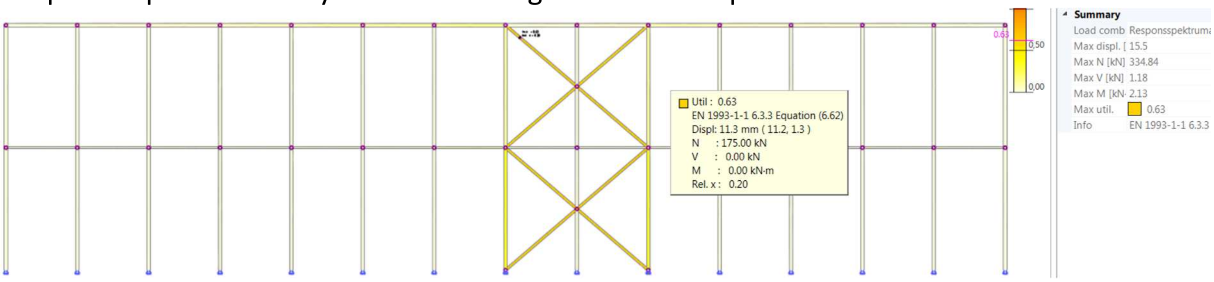
Vibration analysis · default height · 20% snow mass considered



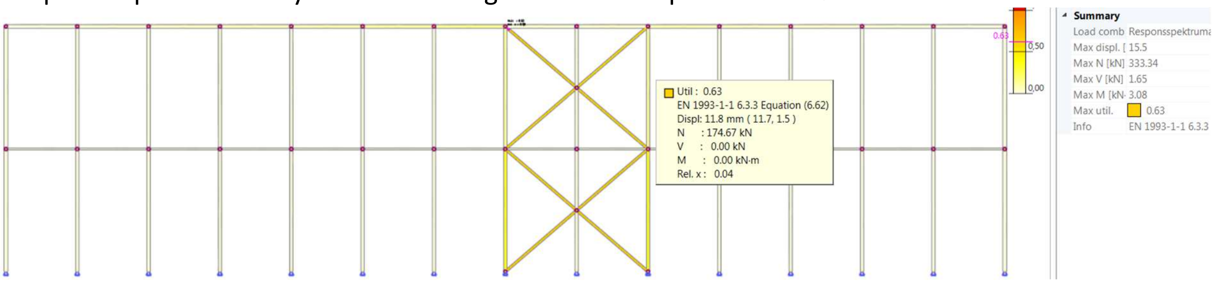
Vibration analysis · default height · 40% snow mass considered



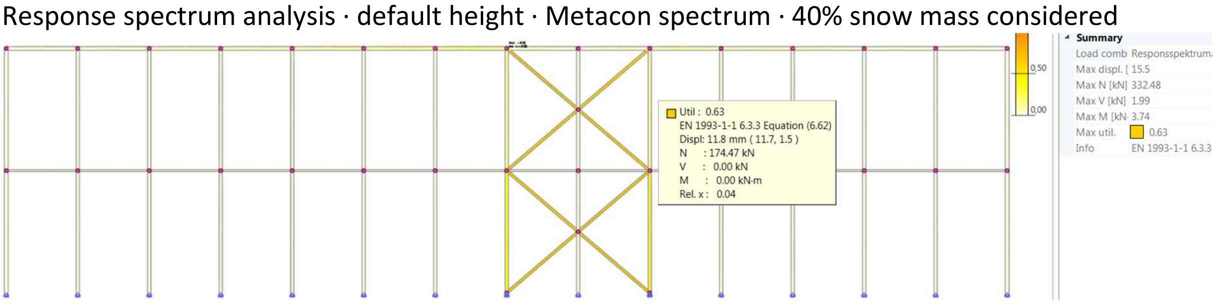
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



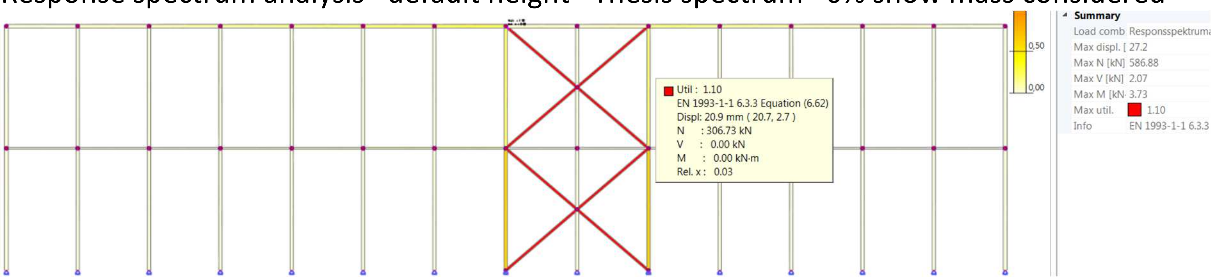
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



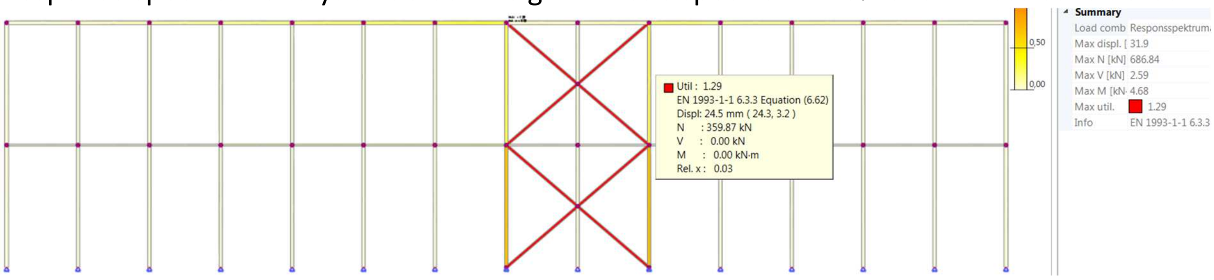
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



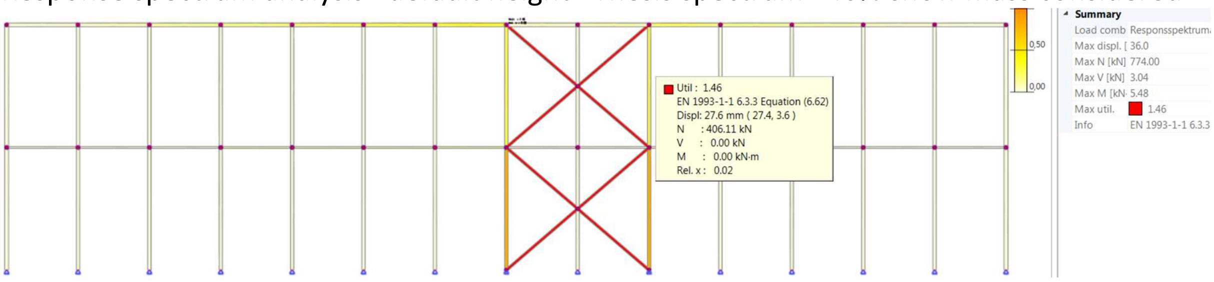
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

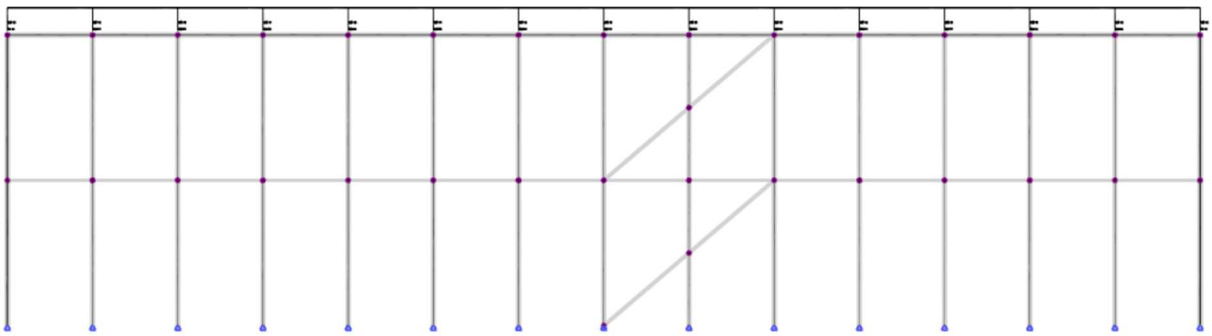


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

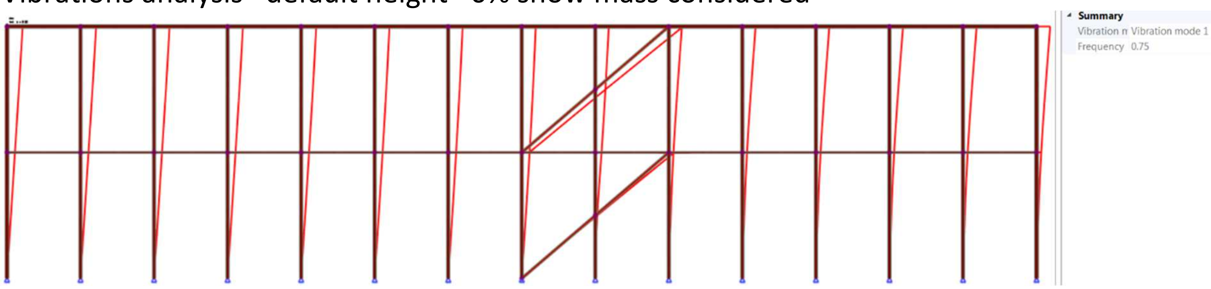


C.2.6 Case 2 · Double cross wind-bracings (default height) modified

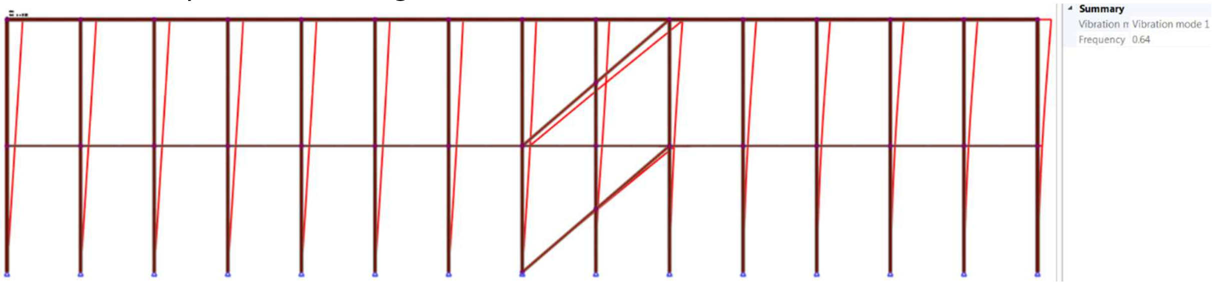
Focus model · Double cross wind-bracings · default height



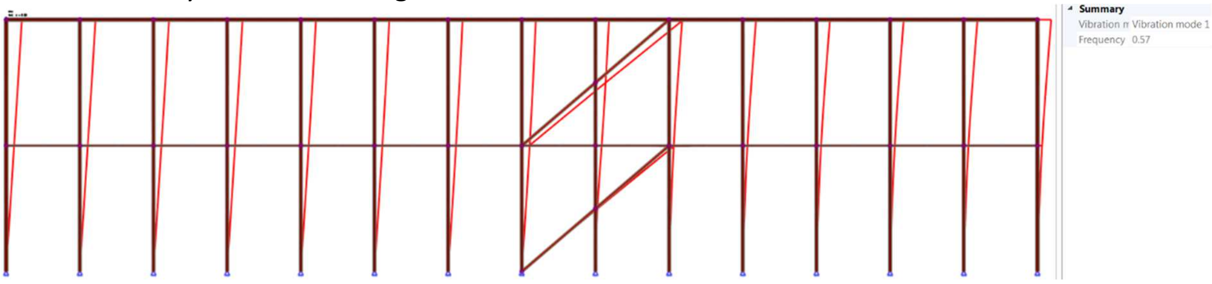
Vibrations analysis · default height · 0% snow mass considered



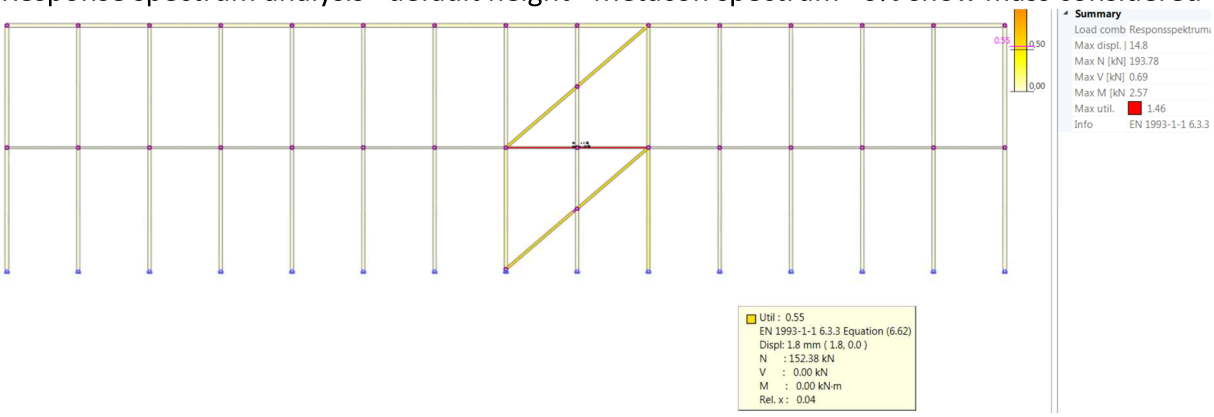
Vibration analysis · default height · 20% snow mass considered



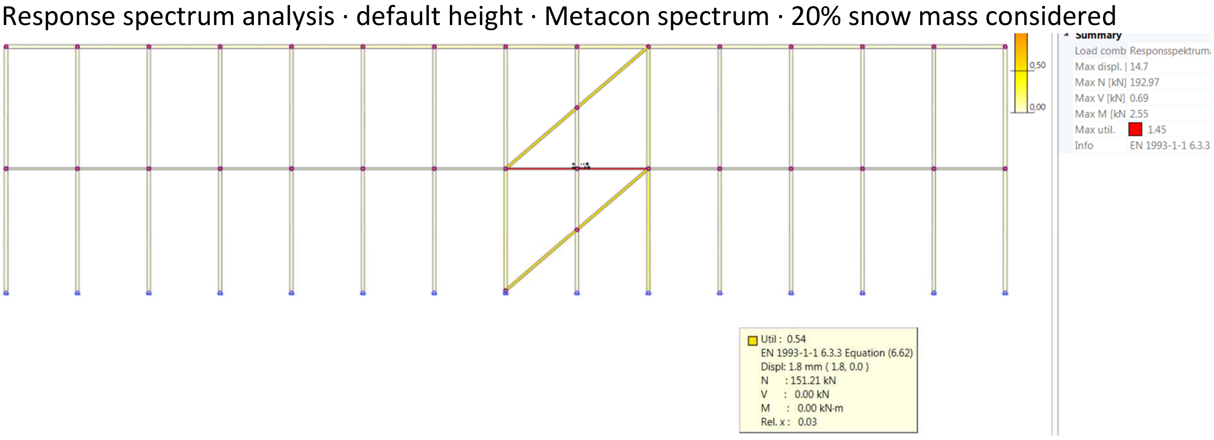
Vibration analysis · default height · 40% snow mass considered



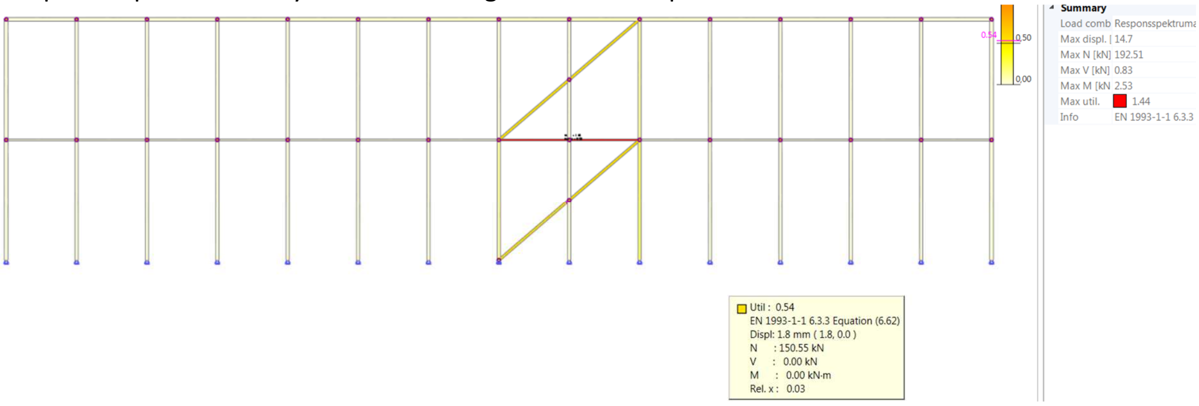
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



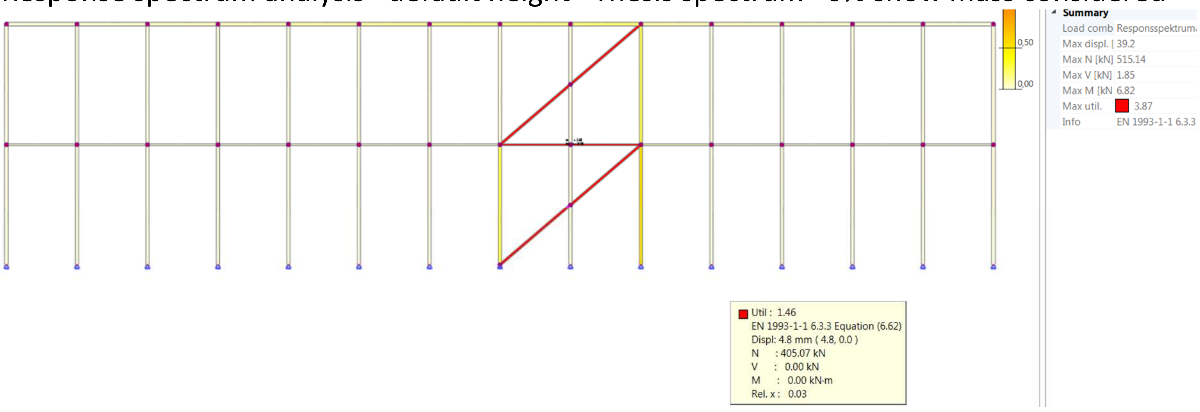
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



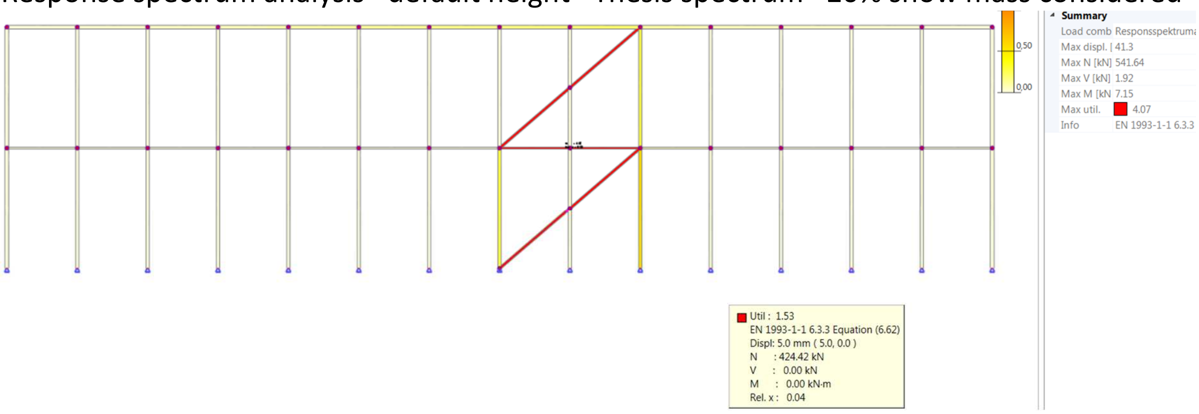
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



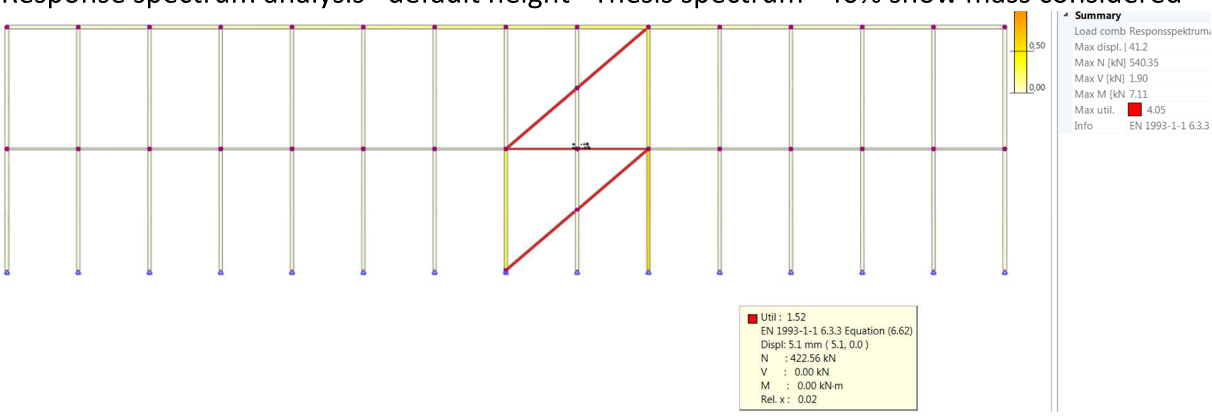
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

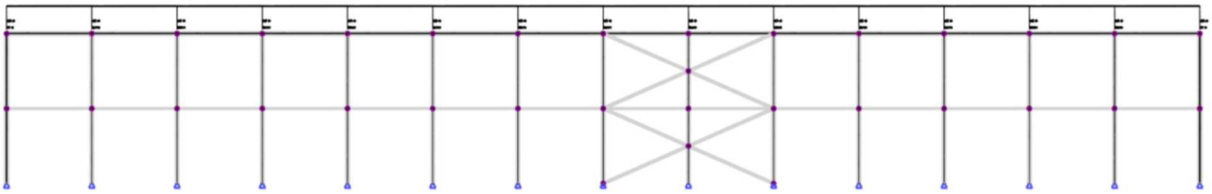


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

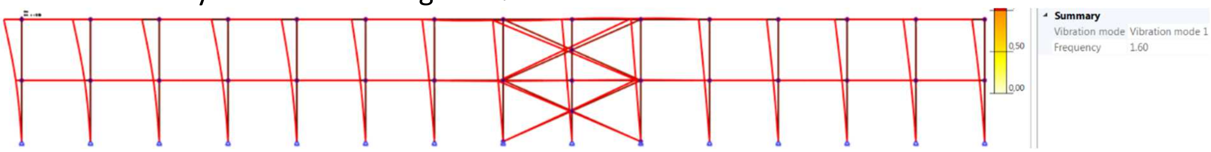


C.2.7 Case 2 · Double cross wind-bracings (reduced height)

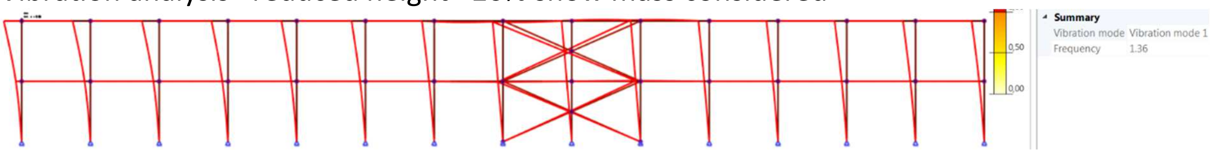
Focus model · Double cross wind-bracings · reduced height



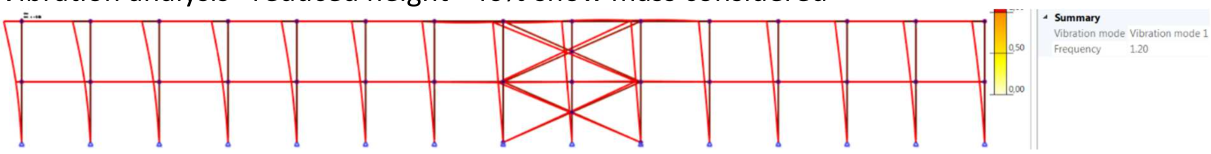
Vibrations analysis · reduced height · 0% snow mass considered



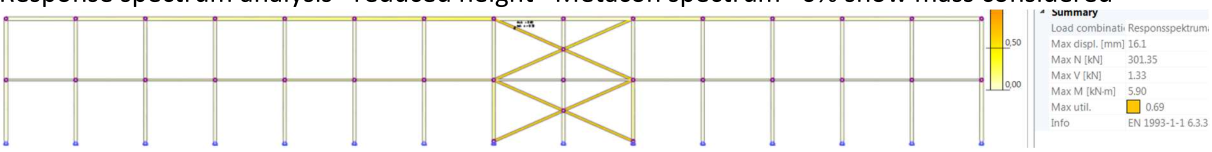
Vibration analysis · reduced height · 20% snow mass considered



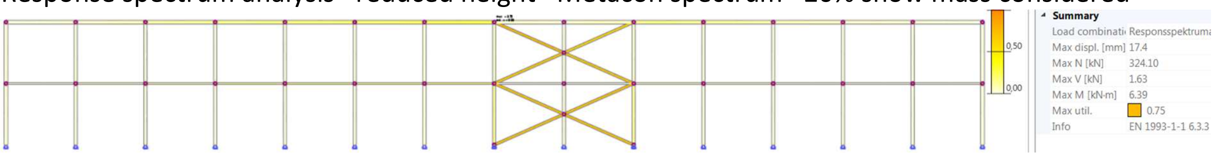
Vibration analysis · reduced height · 40% snow mass considered



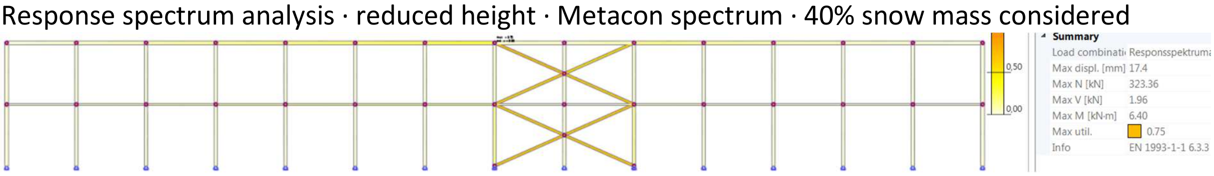
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



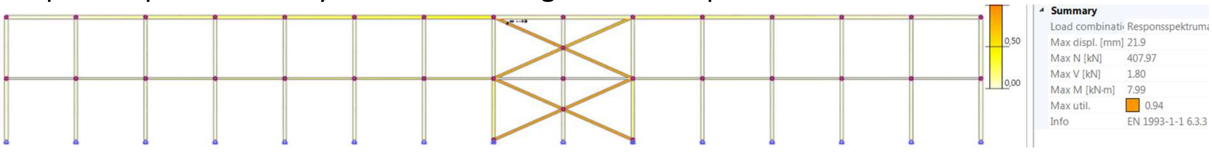
Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered



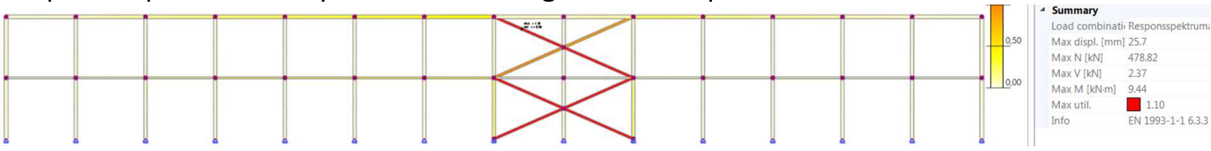
Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered



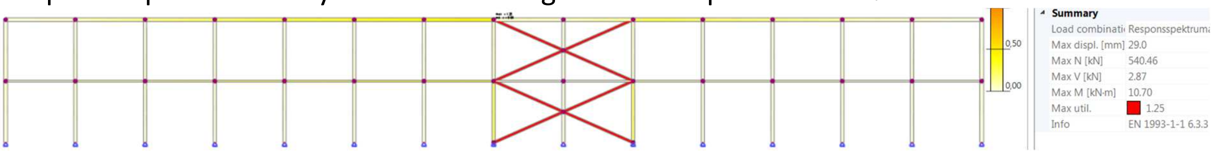
Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered

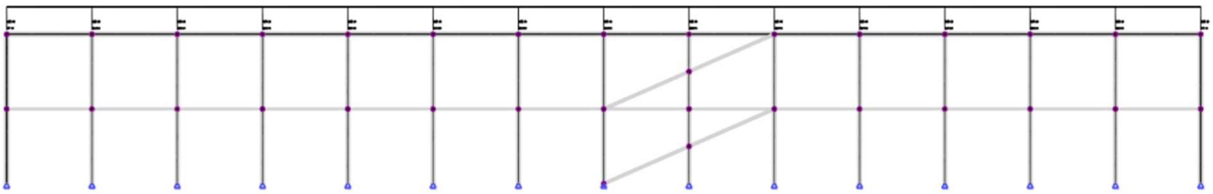


Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered

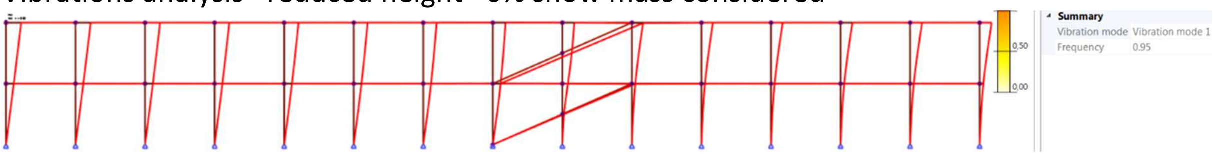


C.2.8 Case 2 · Double cross wind-bracings (reduced height) modified

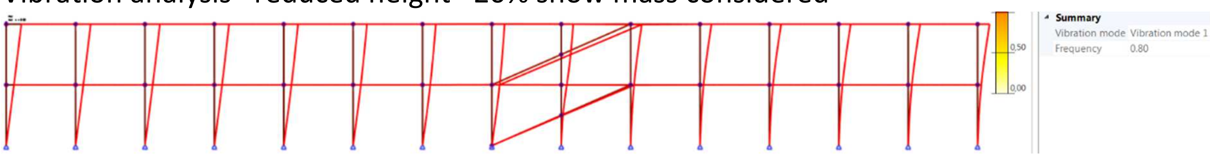
Focus model · Double cross wind-bracings · reduced height



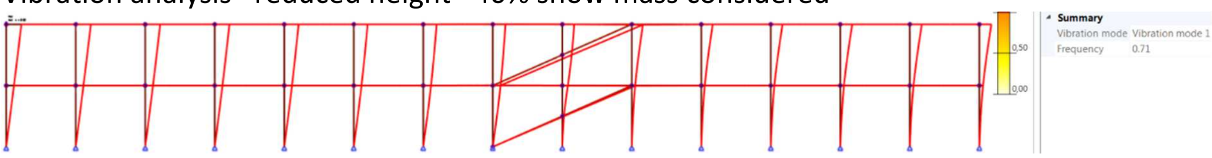
Vibrations analysis · reduced height · 0% snow mass considered



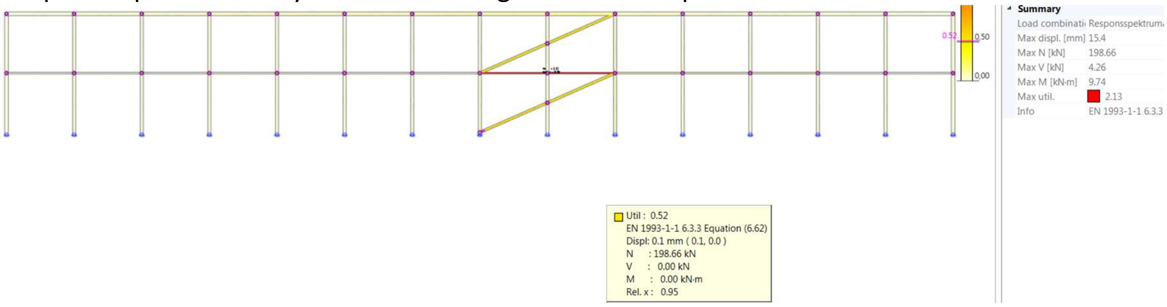
Vibration analysis · reduced height · 20% snow mass considered



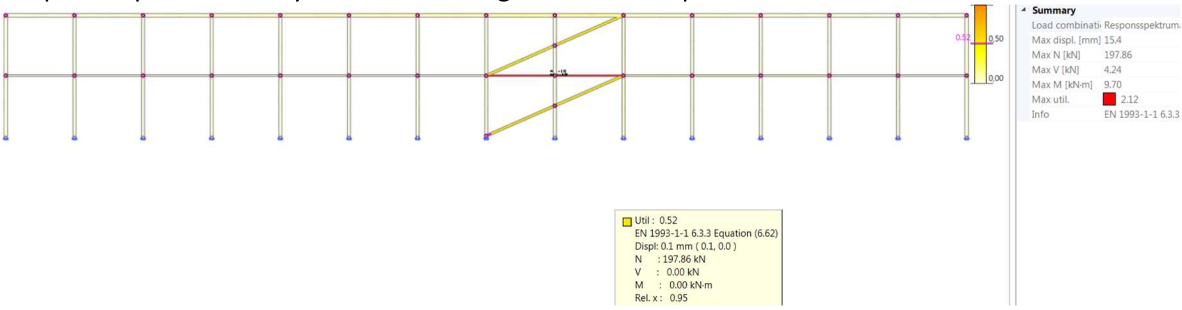
Vibration analysis · reduced height · 40% snow mass considered



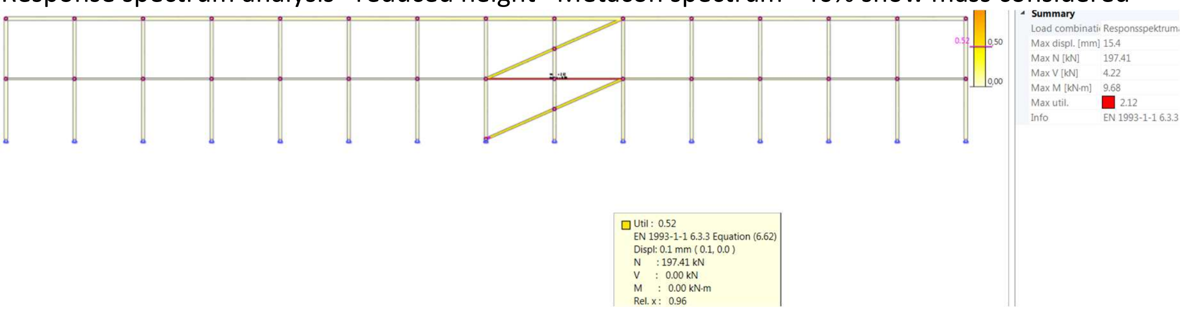
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



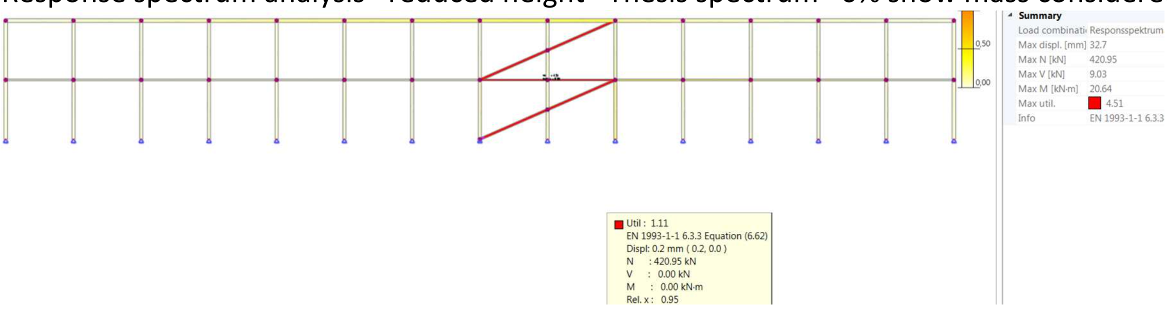
Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered



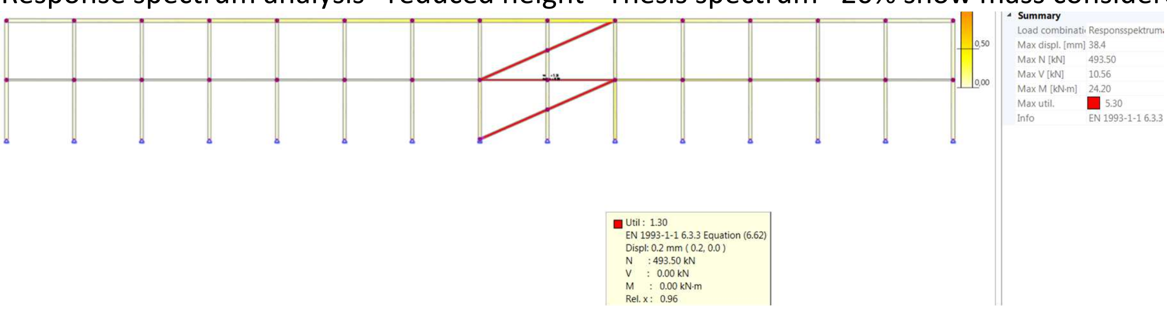
Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered



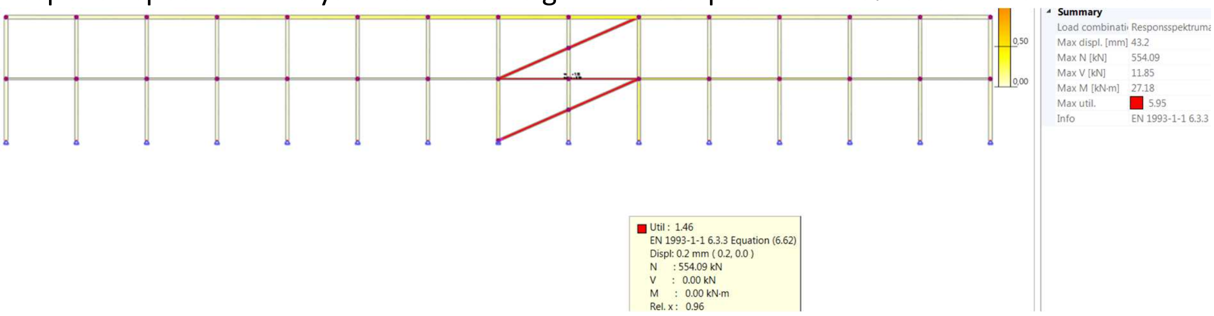
Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered

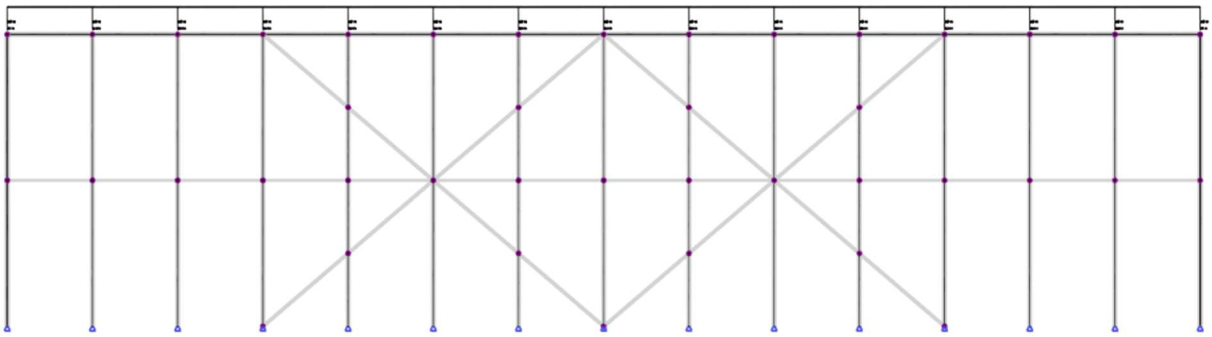


Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered

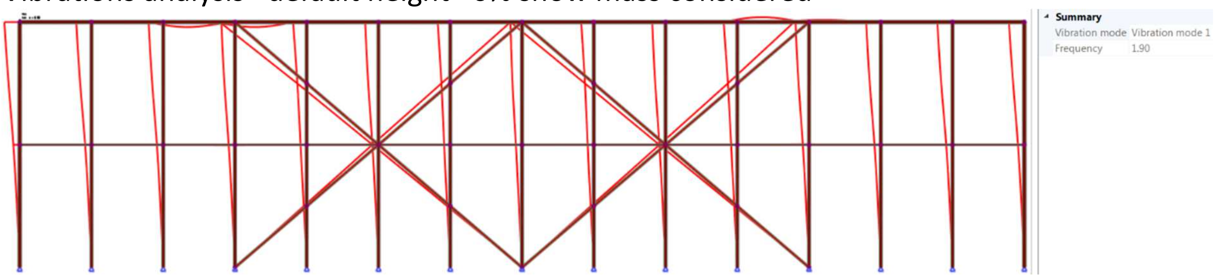


C.2.9 Case 3 · Two single cross wind-bracings (default height)

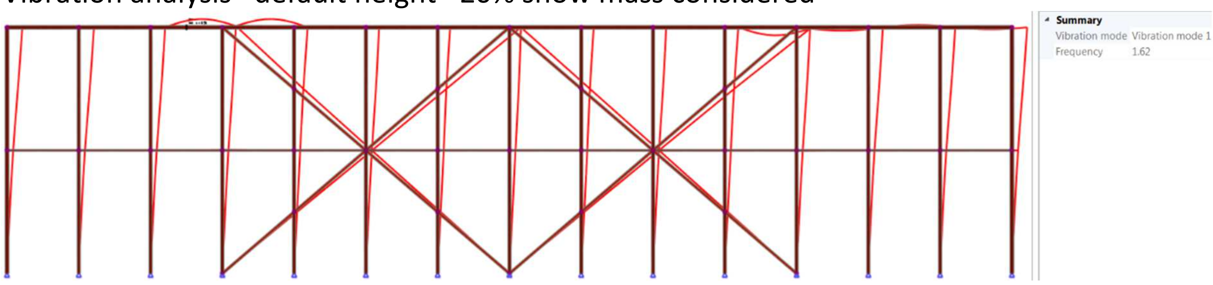
Focus model · Two single cross wind-bracings · default height



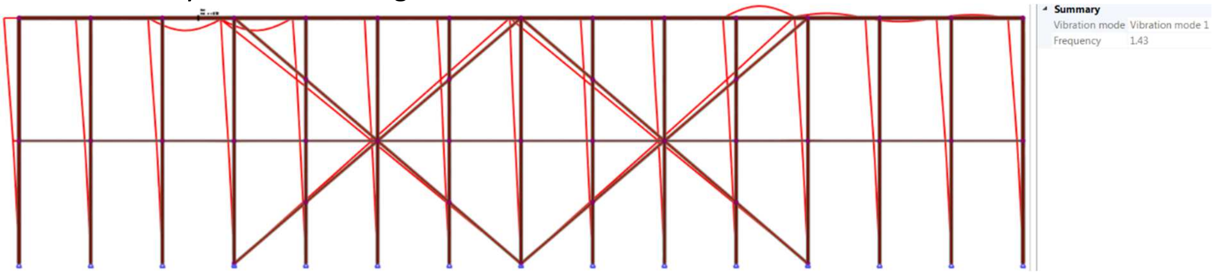
Vibrations analysis · default height · 0% snow mass considered



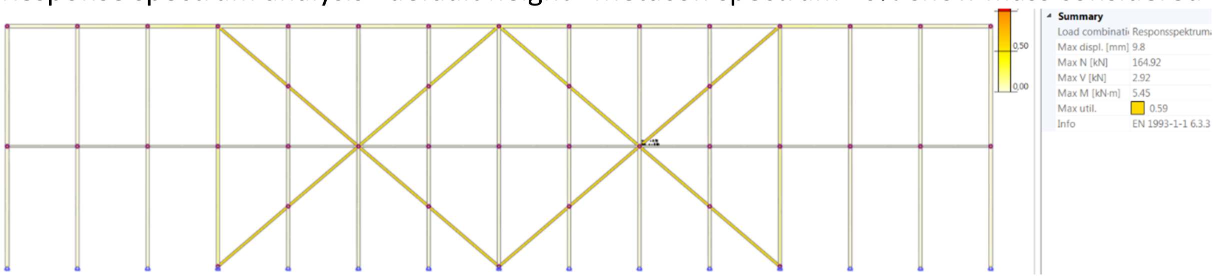
Vibration analysis · default height · 20% snow mass considered



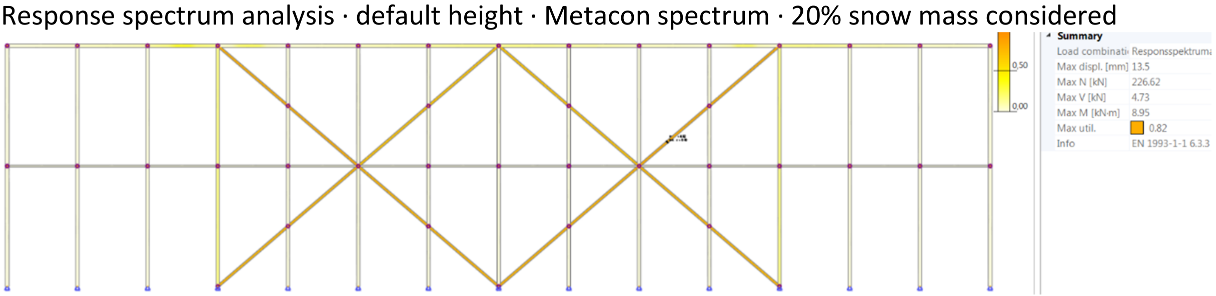
Vibration analysis · default height · 40% snow mass considered



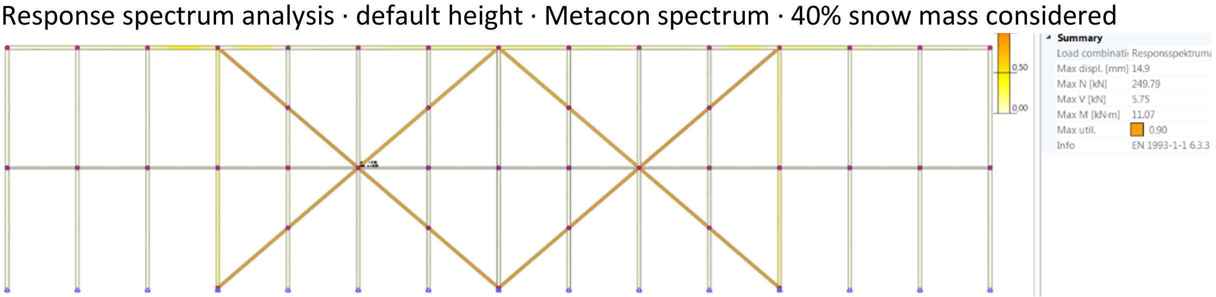
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



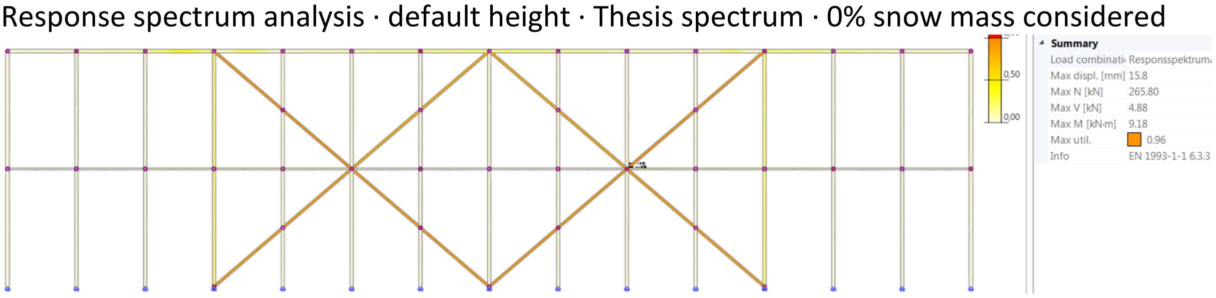
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



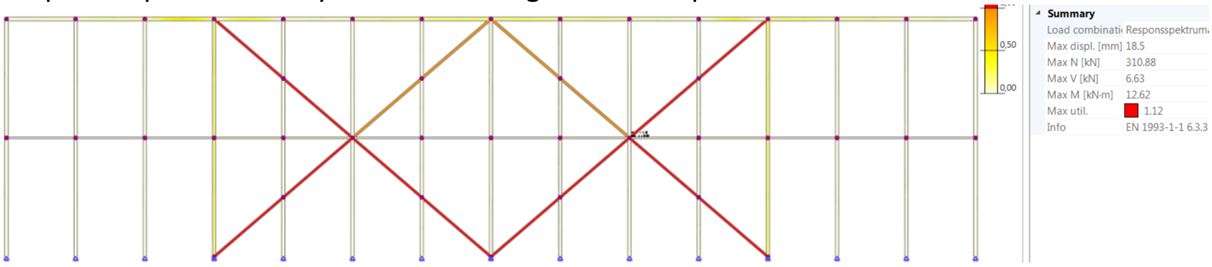
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



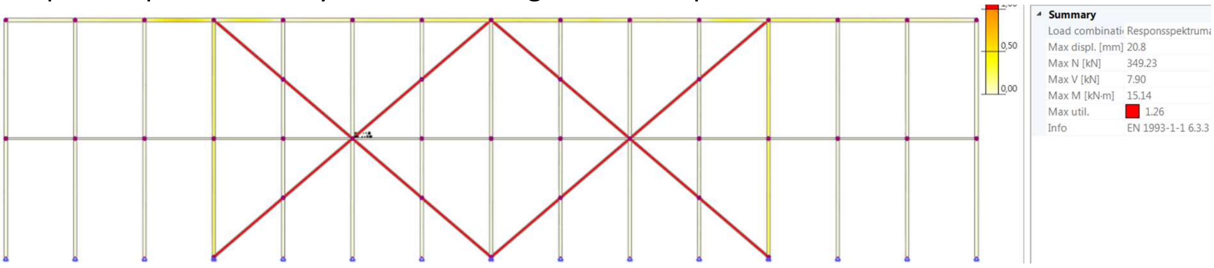
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

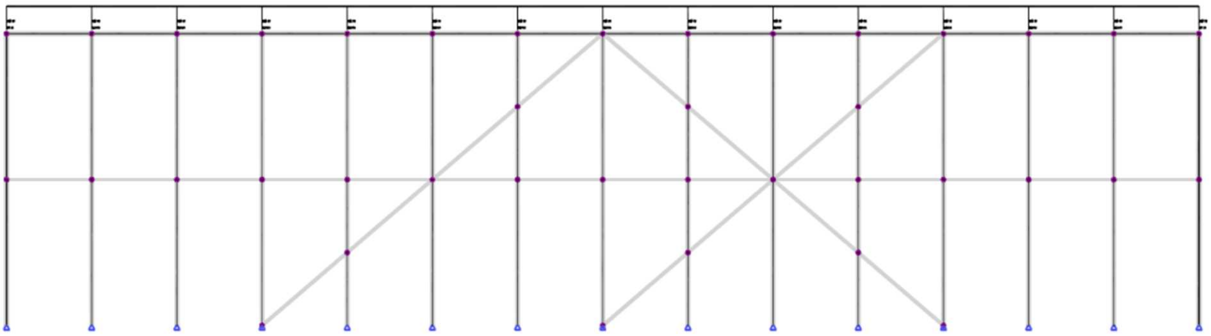


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

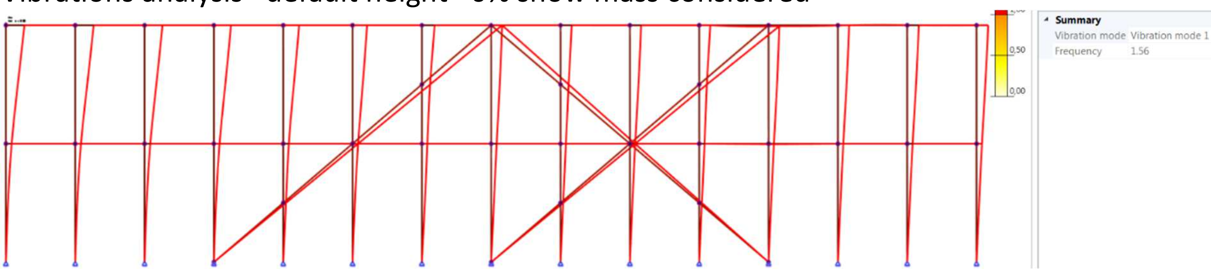


C.2.10 Case 3 · Two single cross wind-bracings (default height) modified

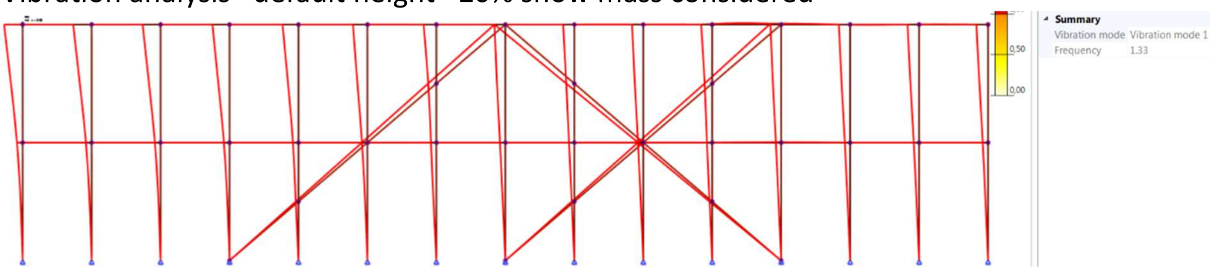
Focus model · Two single cross wind-bracings · default height



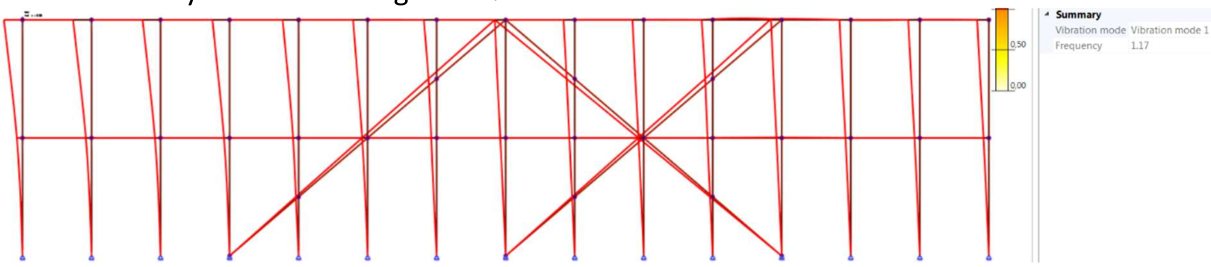
Vibrations analysis · default height · 0% snow mass considered



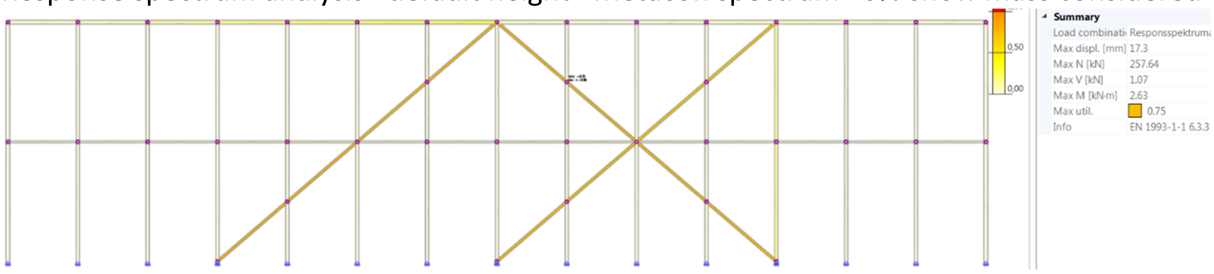
Vibration analysis · default height · 20% snow mass considered



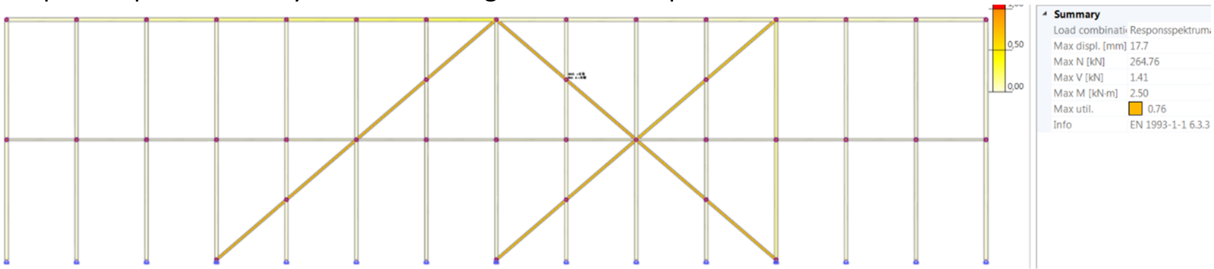
Vibration analysis · default height · 40% snow mass considered



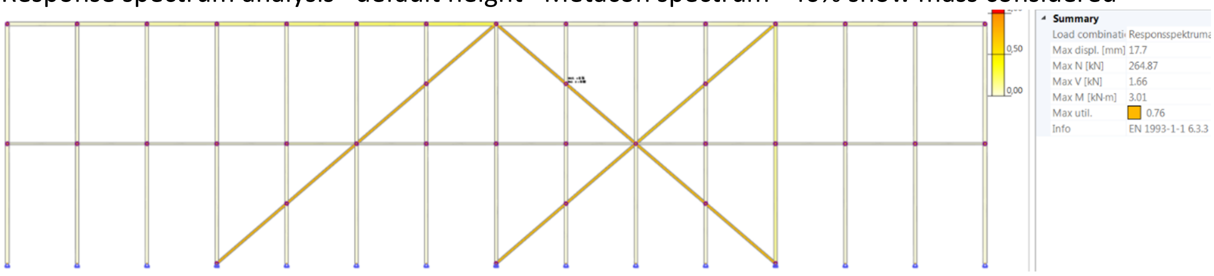
Response spectrum analysis · default height · Metacon spectrum · 0% snow mass considered



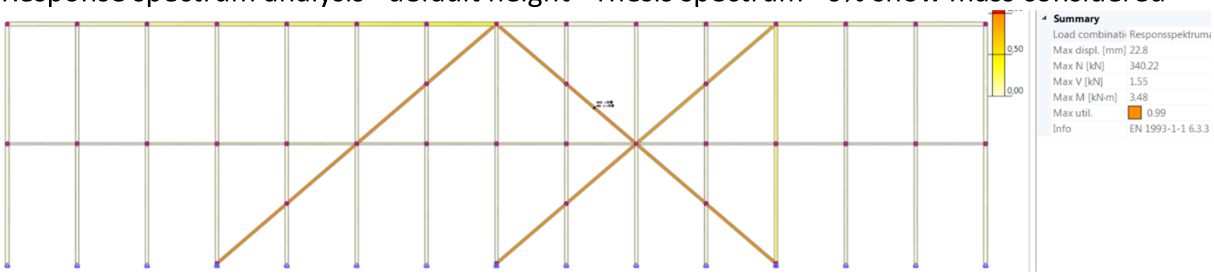
Response spectrum analysis · default height · Metacon spectrum · 20% snow mass considered



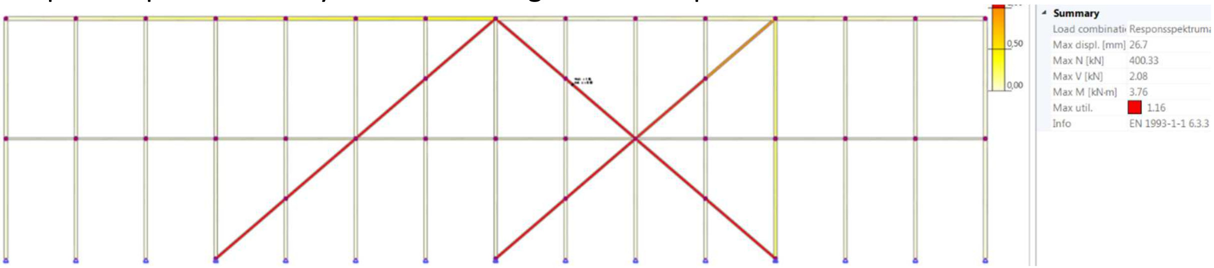
Response spectrum analysis · default height · Metacon spectrum · 40% snow mass considered



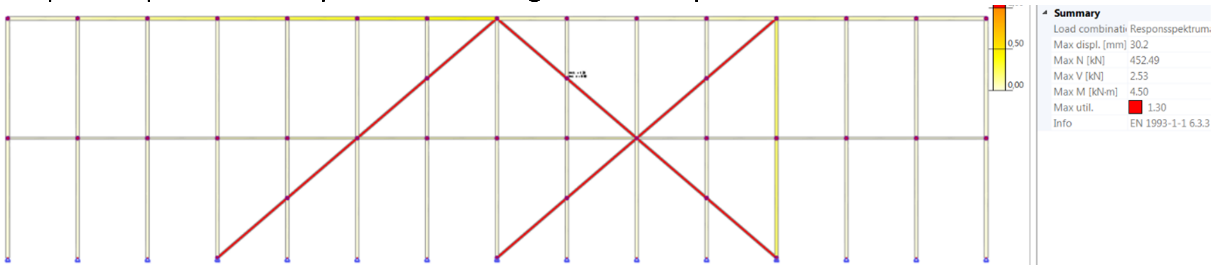
Response spectrum analysis · default height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · default height · Thesis spectrum · 20% snow mass considered

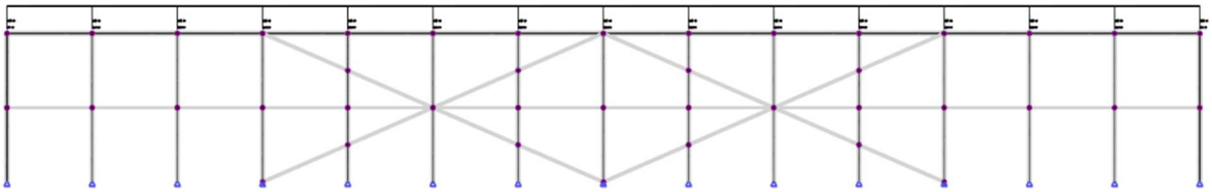


Response spectrum analysis · default height · Thesis spectrum · 40% snow mass considered

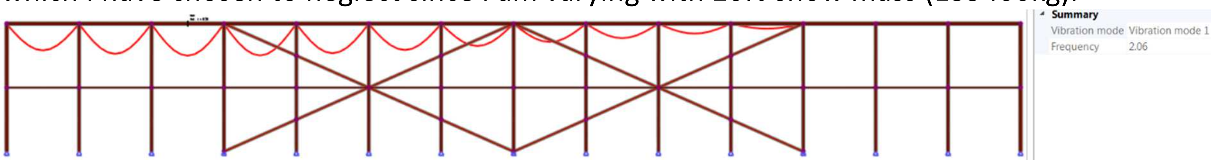


C.2.11 Case 3 · Two single cross wind-bracings (reduced height)

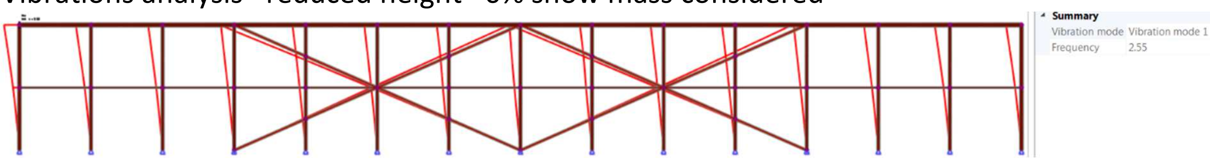
Focus model · Two single cross wind-bracings · reduced height



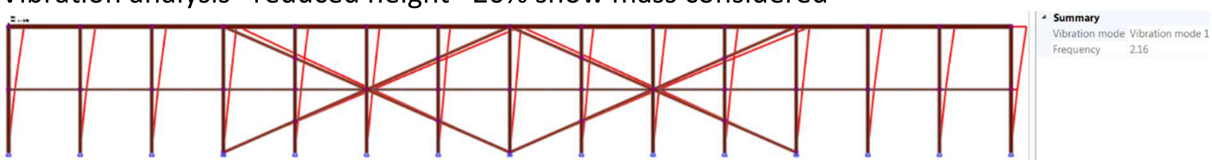
Vibration analysis giving wrong form. To change the swing form, the dimensions of the beams is increased from IPE270 to IPE300. The weight increase was only about 1000kg, which I have chosen to neglect since I am varying with 20% snow mass (135400kg).



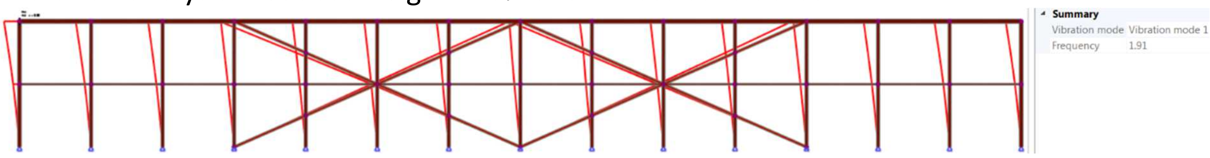
Vibrations analysis · reduced height · 0% snow mass considered



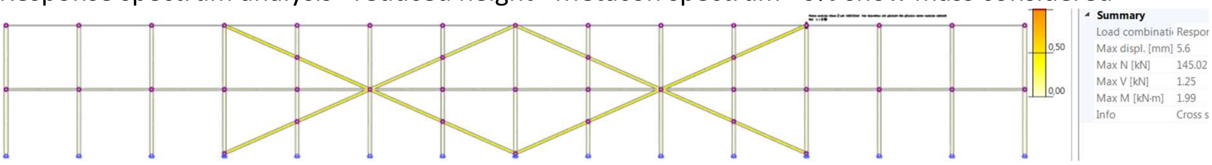
Vibration analysis · reduced height · 20% snow mass considered



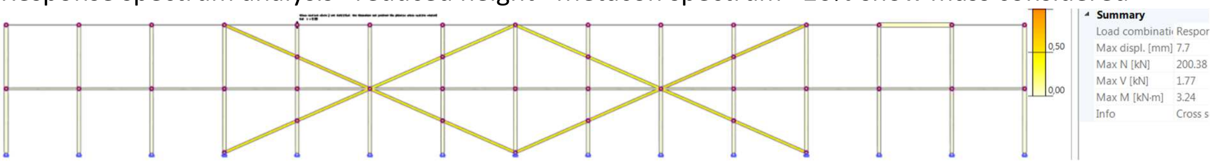
Vibration analysis · reduced height · 40% snow mass considered



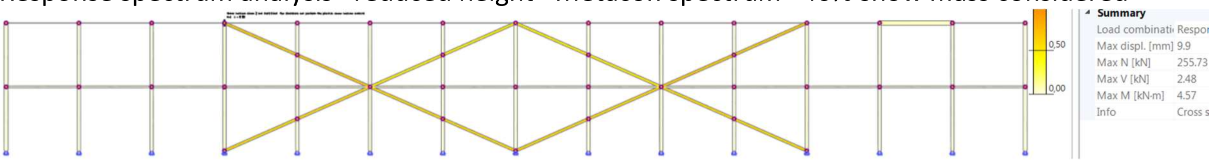
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



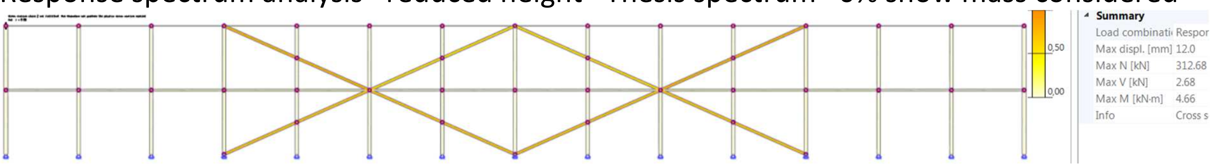
Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered



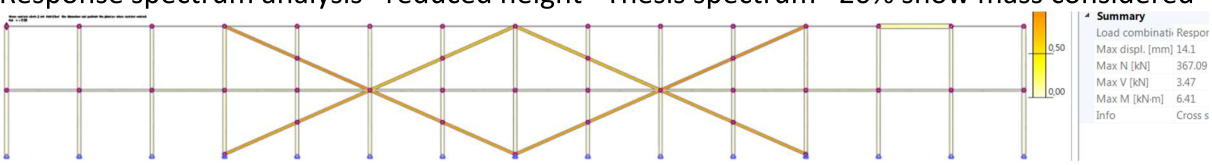
Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered



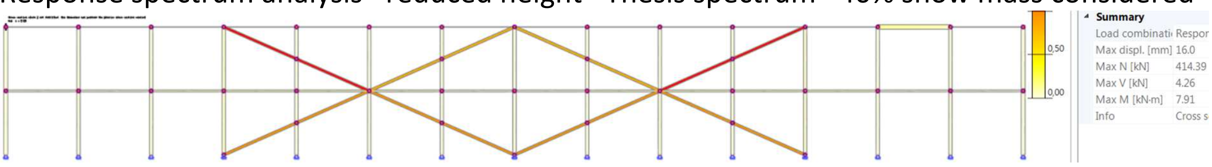
Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered



Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered

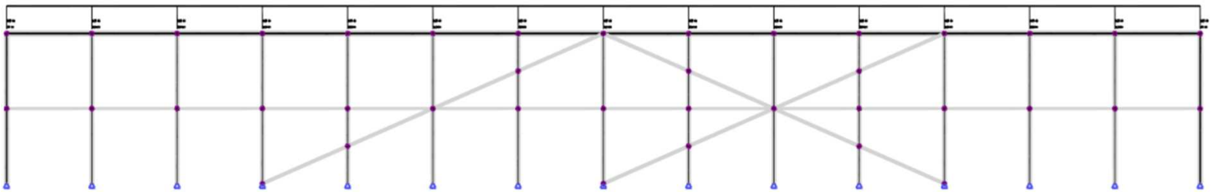


Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered

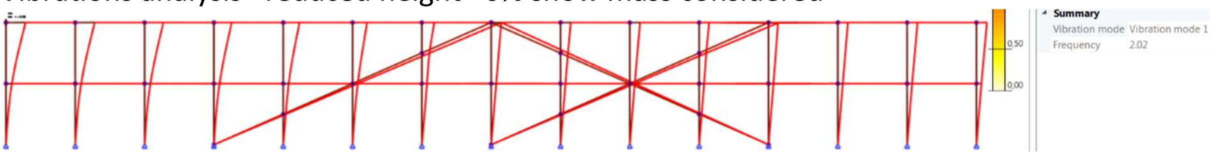


C.2.12 Case 3 · Two single cross wind-bracings (reduced height) modified

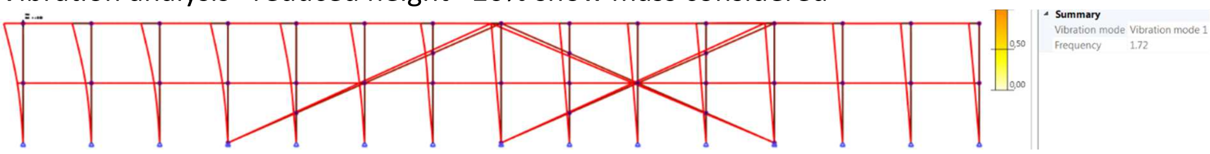
Focus model · Two single cross wind-bracings · reduced height



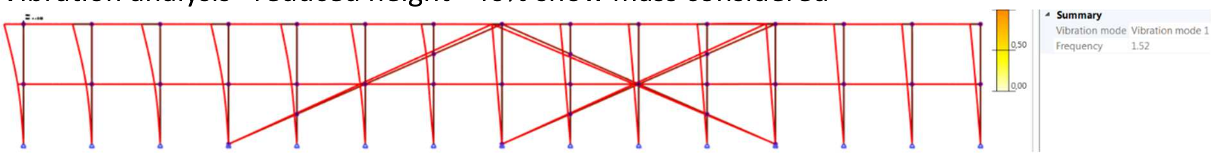
Vibrations analysis · reduced height · 0% snow mass considered



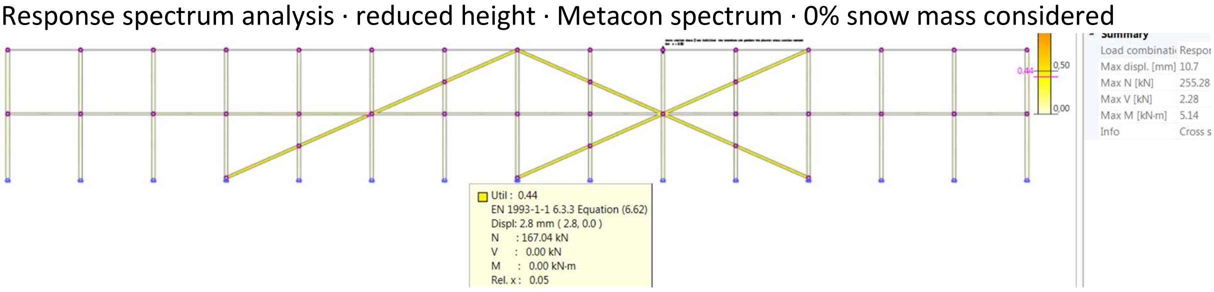
Vibration analysis · reduced height · 20% snow mass considered



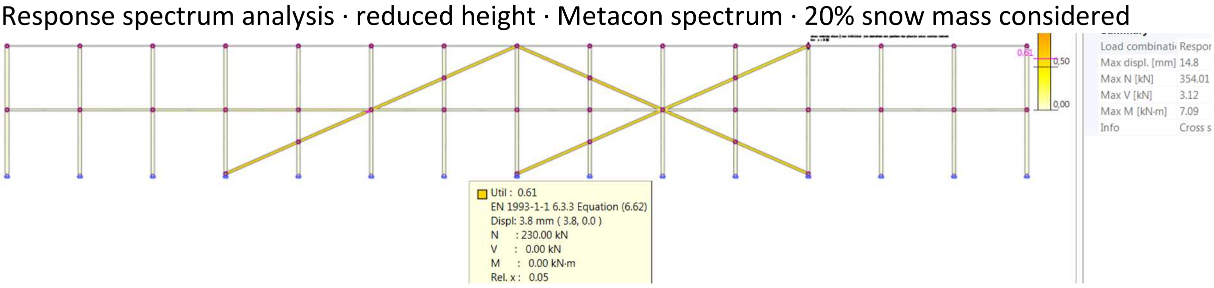
Vibration analysis · reduced height · 40% snow mass considered



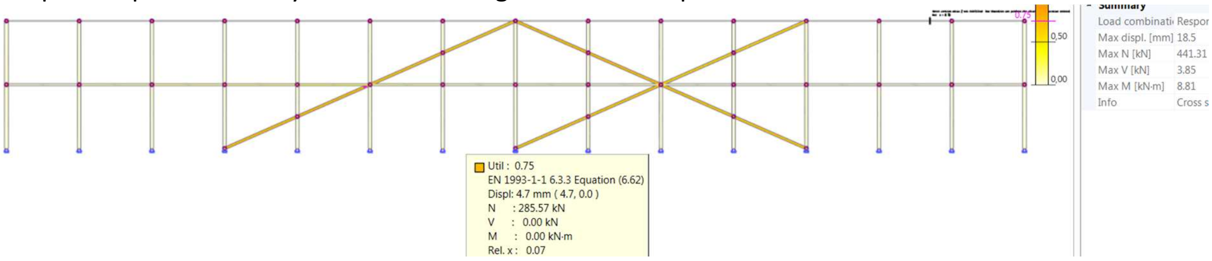
Response spectrum analysis · reduced height · Metacon spectrum · 0% snow mass considered



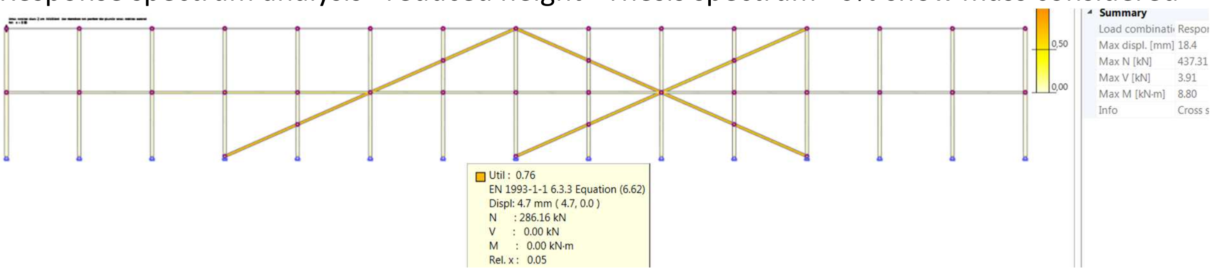
Response spectrum analysis · reduced height · Metacon spectrum · 20% snow mass considered



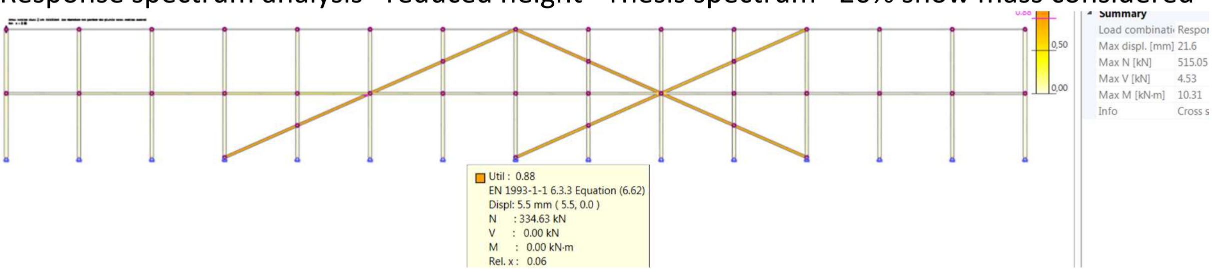
Response spectrum analysis · reduced height · Metacon spectrum · 40% snow mass considered



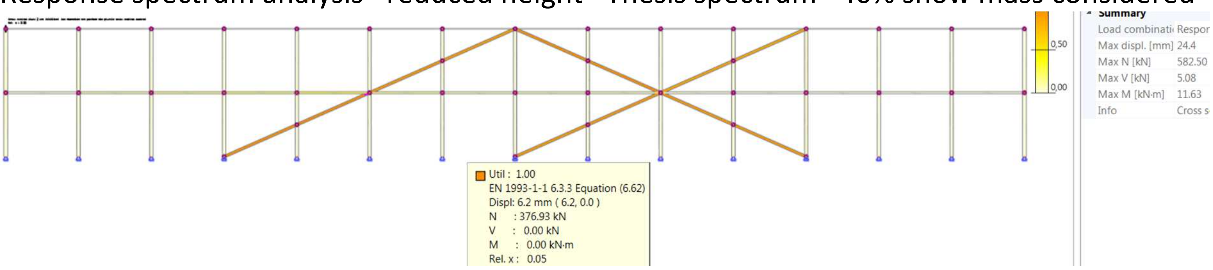
Response spectrum analysis · reduced height · Thesis spectrum · 0% snow mass considered



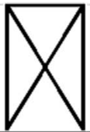
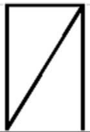
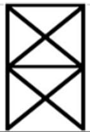
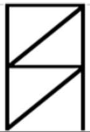
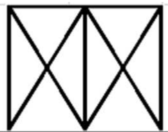
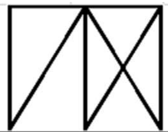
Response spectrum analysis · reduced height · Thesis spectrum · 20% snow mass considered



Response spectrum analysis · reduced height · Thesis spectrum · 40% snow mass considered



C.3 Summary of results for all cases

Summary of analysis results																			
		0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø	0% snø	20% snø	40% snø
Metacon H1	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66	1,14	0,97	0,86	0,75	0,64	0,57	1,90	1,62	1,43	1,56	1,33	1,17
	δ [mm]	16,70	16,70	16,60	15,70	15,60	15,60	15,50	15,50	15,50	14,80	14,70	14,70	9,80	13,50	14,90	17,30	17,70	17,70
	F [kN]	229,38	228,71	228,32	201,55	199,93	199,02	175,00	174,67	174,47	152,38	151,21	150,55	164,92	226,62	249,79	257,64	264,76	264,87
	UF	0,83	0,82	0,82	0,73	0,72	0,72	0,63	0,63	0,63	0,55	0,54	0,54	0,59	0,82	0,90	0,75	0,76	0,76
Thesis H1	f [Hz]	1,28	1,09	0,97	0,88	0,75	0,66	1,14	0,97	0,86	0,75	0,64	0,57	1,90	1,62	1,43	1,56	1,33	1,17
	δ [mm]	26,20	30,70	34,60	35,90	42,10	43,80	27,20	31,90	36,00	39,20	41,30	41,20	15,80	18,50	20,80	22,80	26,70	30,20
	F [kN]	358,67	420,37	474,09	461,37	538,03	558,62	306,73	359,87	406,11	405,07	424,42	422,56	265,80	310,88	349,23	340,22	400,33	452,49
	UF	1,29	1,51	1,71	1,66	1,94	2,01	1,10	1,29	1,46	1,46	1,53	1,52	0,96	1,12	1,26	0,99	1,16	1,30
Metacon H2	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83	1,60	1,36	1,20	0,95	0,80	0,71	2,55	2,16	1,91	2,02	1,72	1,52
	δ [mm]	16,10	18,30	18,30	16,60	16,60	16,50	16,10	17,40	17,40	15,40	15,40	15,40	5,60	7,70	9,90	10,70	14,80	18,50
	F [kN]	275,97	313,63	313,02	258,64	257,53	265,90	301,35	324,10	323,36	198,66	197,86	197,41	145,02	200,38	255,73	167,04	230,00	285,57
	UF	0,73	0,83	0,83	0,68	0,68	0,68	0,69	0,75	0,75	0,52	0,52	0,52	0,38	0,53	0,68	0,44	0,61	0,75
Thesis H2	f [Hz]	1,64	1,39	1,23	1,10	0,94	0,83	1,60	1,36	1,20	0,95	0,80	0,71	2,55	2,16	1,91	2,02	1,72	1,52
	δ [mm]	22,40	26,40	29,80	30,10	35,40	40,00	21,90	25,70	29,00	32,70	38,40	43,20	12,00	14,10	16,00	18,40	21,60	24,40
	F [kN]	384,05	450,76	508,79	469,74	550,74	621,28	407,97	478,82	540,46	420,95	493,50	554,09	312,68	367,09	414,39	286,16	334,63	376,93
	UF	1,01	1,19	1,34	1,24	1,45	1,64	0,94	1,10	1,25	1,11	1,30	1,46	0,83	0,97	1,09	0,76	0,88	1,00