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MASTER THESIS

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Seismic analysis of an RC building with newly constructed additional stories, Kristina kvartalet, Tønsberg	May 25, 2022
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SUMMARY

This thesis will handle a seismic analysis of a RC structure, which has been increased from 4 to 7 stories, located in Tønsberg, Norway. The structure has been omitted for seismic dimensioning after criterias approved by Eurocode, but this thesis will perform it and discuss around the process, omission and potential risks involved with both the soil at this location and generally in omitting a structure for seismicity.

The analysis compares the new agains the original structure, and finds that the base shear and torsion loads have been tremendously increased in both horizontal axis, and that an evaluation of the foundation – a separate proper analysis of the structure with the foundation and piles is recommended. The omission of the structure is considered somewhat safe, but the foundations should not be ignored as it has been.

3 KEYWORDS
Seismic analysis of concrete structure
Comparisson of structure expanded as built, by hollow core slabs + steel columns and original structure.
Evaluation of vulnerabilities with use of omission criterias

Abstract

This master thesis will be based on a real-world project where the author's employer – Projekt Planung AS were responsible for the structural engineering. The building is the Kristinakvartalet / Kristina Quarter in Tønsberg, an angled, partially rectangular concrete structure which will be expanded by three stories in height. In addition, there will be modifications of certain structural parts to streamline the projects workflow and reduce costs.

With the expansion of three additional stories, the building have increased load and dynamic properties, and will require to be analysed for both static and new dynamic characteristics, if not assessed for omission criterias in regard to seismic dimensioning.

This thesis will focus on analysing the structure before and after expansion for dynamic properties, both with various materials used in the expanded stories, using FEM software and hand calculations based on the proceedings described in the Eurocode standard. Further, these results will be used to evaluate the structure, as well as the process and difficulties around the omission criterias with its strengths and weaknesses.

During the process for the thesis, I have learned a tremendous amount about how an analysis is performed with all the steps and data, but also much of the theory behind it. This comes from both the standards, guidelines and the academic theory found in scientific journals that have been used in the thesis. This process have truly made me very interested in working with seismic analysis of structures in the future.

I would like to thank my supervisors during the analysis and writing of this thesis – my colleague Jacob C Emesum, M.Sc at Projekt Planung, and Associate Professor Mahdi Kioumarsi, P.hD at Oslo Metropolitan University.

Oslo, May 25th 2022

Bjørnar Bjerkestrand

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1 Introduction

This thesis is based on the Kristina kvartalet, an office building in downtown Tønsberg, Norway. The structure is originally a four-story + two basement levels RC structure, and has been increase by three stories in height to house more office space and refurbishing the building. During the engineering process, it was decided to not validate the structure for seismic loads, and only consider static loads and wind.

Not validating or omitting the structure for seismic loads is allowed if the structure fulfils one of four criterias for omission, based on Eurocode 8 standard for seismic loads on structures. However, for this structure standing on clays in risk of liquefaction, risks are involved in not assessing the structure and atleast the foundation and piles for loads during an earthquake.

This work will prove important for own and fellow engineers use to be aware of the difficulties and potential risks introduced with omission criterias.

1.1 Objectives

This thesis explains a real world engineering project where a structure has been increased by several floors, while it has not been assessed for seismic loads. The main objectives in this report are:

1. Study the seismic analysis of the new structure, compare it with an alternative in hollow core slabs and the original structure. The analysis will be performed within Autodesk Robot based on Eurocode 8 in regard to parameters and procedures.

2. Analysis of potential weaknesses and problems regarding the omission of seismic dimensioning for structures.

1.2 Scope and limitations

With the background of the engineering process for Kristina kvartalet in Tønsberg, Norway, where omission criterias were exerted to avoid seismic analysis and dimensioning and thereby only evaluated for static loads. This thesis will analyse the structure for dynamic loads – earthquake and perform a seismic analysis of the structure based on Eurocode settings.

A total of three variations of the structure is analysed. This is done as a step towards discussion of the results, assumptions and procedure faced in such a task. The analysis will also give the design loads for seismic dimensioning of the structure. A major part of this report is the use of various assumptions that have major influences on the results, these are thoroughly discussed based on dynamic properties during an earthquake, and represent an important part of the report.

The report is limited to the modal and seismic analysis of the structure above ground level, where the basement and foundation is ignored in the analysis. Analysis of stress, foundation, Soil-structure interaction and liquefaction of soil is not included as it would require far more time and detailed information about the soil and layers, which is not known at this stage.

1.3 Outline of thesis

Structure of the thesis

Chapter 2 – Theory provides the basic background for the theoretical parts relevant to the thesis, starting with earthquakes and the basic mechanics behind it, before basics of the dynamic behaviour and parameters of a structure is described. The basics of the dynamics is not explored further than the expressions in this chapter. Next, piles have a subchapter to describe the phenomenons they encounter during accelerations. Eurocode standard is a chapter to provide the design parameters and relevant simplified equations used during an analysis. Last, a chapter to show the modifications made of the structure during the construction phase is written to show what is done, the assumptions and decisions made, as well as describing all the omission criterias.

Chapter 3 – Method describes the process of performing the analysis within Autodesk Robot, starting with showing and describing the structure in the BIM model, following up with the steps and inputs such as constraints and loads made to perform the analysis. A separate subchapters for the figures is written to separate the text from figures.

Chapter 4 – Results shows all the results from the analysis within Robot, in addition a basic hand calculation is performed with the method from Eurocode 8 to be able to compare the results with the numerical model.

Chapter 5 – Discussion to discuss results, weaknesses and assumptions. The analysis is performed under several assumptions that influences the results and ignores for example soil-structure interactions by using fixed support at base. Validity, relevance and omission criterias is also discussed.

Chapter 6 – Conclusion is written based on the discussion and a final verdict is made.

Chapter 7 – Further work is a recommendation chapter of futher work to complete the limtations of this thesis.

Appendix – Report attachments relevant for the thesis that is not published.

2 Theory

2.1 Earthquakes

2.1.1 Seismology

The earth's crust consists of multiple continental shelves which moves slightly every year, and during movement they release large amounts of energy through seismic waves and makes an earthquake [1]. The seismic waves is divided into two categories – P-waves and S-waves. P-waves (pressure waves) travels through the surface with compressions and does not excite in the vertical axis, they can travel through solid material, liquids and gaseous substances [1], [2]. See Figure 1 for P-waves [3] . S-waves (shear waves) travels through the surface as vertical oscillations, and exictes the surface up and down [1], [2]. S-waves travels slower than P-waves and cannot travel through liquids nor gaseous substances. See Figure 2 for S-waves [4].

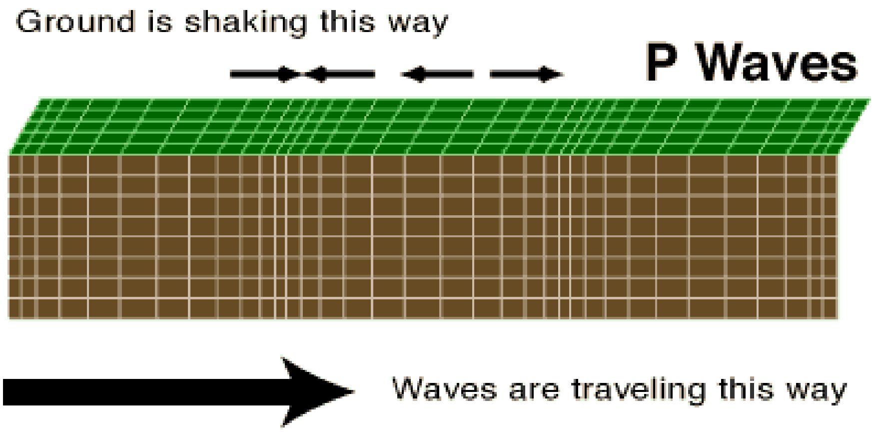


Figure 1: Illustration of P-wave [3]

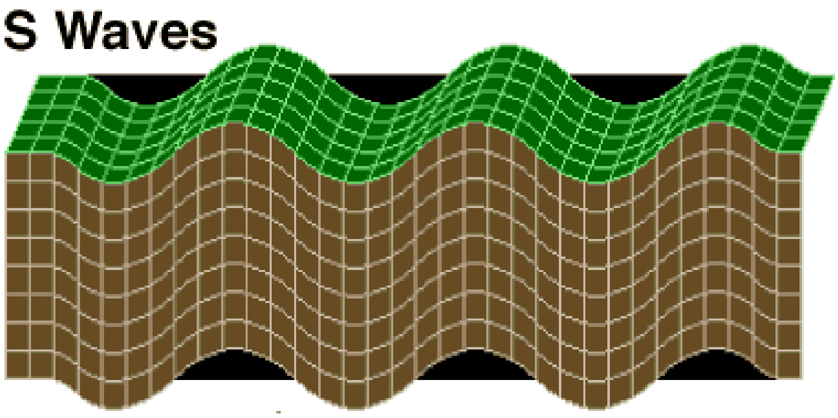


Figure 2: Illustration of S-wave [4]

2.1.2 Magnitude of earthquakes

The size of an earthquake can be described in different ways. Measuring the intensity by describing the effects of the earthquake at a particular location, as evidence by observed damage and human reaction has been replaced by measuring the magnitude.

Seismic instruments allow an objective, quantitative measurement of earth shaking to determine the earthquake magnitude [2].

Surface Wave Magnitude

Surface wave magnitude, unlike other magnitude scales does not distinguish between different types of waves, and is a worldwide magnitude scale and is based on the amplitude of Rayleigh waves with a period of about 20 seconds (eq. 2.1.1) [2].

$$M_s = \log A + 1{,}66 \log \Delta + 2{,}0 \qquad (eq. 2.1.1)$$

where:

- A is the maximum ground displacement in micrometer
- Δ is the epicentral distance of the seismometer measured in degrees

Magnitude and acceleration to structures

The magnitude and distance of the earthquake descides the dimensioning acceleration subjected to the structure, where importance class and soil factors also play a role. See Figure 3 for graph vizualisation of equation 2.1.2 and 2.1.3 [5].

$$a_g = \gamma_1 \cdot a_{gR} \qquad (eq. 2.1.2)$$

$$\log a_g = -1{,}48 + 0{,}27 \cdot M - 0{,}92 \log R \qquad (eq. 2.1.3)$$

where:

- γ_1 is importance factor for seismic class, see Figure 5 in chapter 2.4.4
- M is magnitude
- R is the epicentral distance
- Valid for 4 < M < 7,3 and 3km < R < 200km

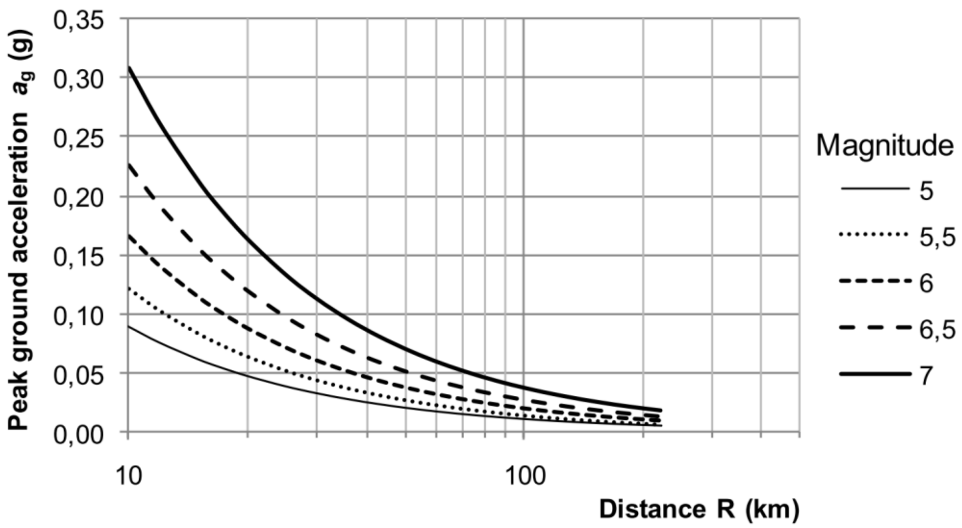


Figure 3: Magnitude to peak ground acceleration based on eq. 2.1.3 [5]

2.1.3 Earthquakes in Norway

In general, the seismicity in Fennoscandia (Norway, Sweden, Finland and the Russian areas of Kola-peninsula and Karelen) is characterized as low to intermediate intensity, but for an area with no close continental shelves, the intensity is higher than normal [6]. The magnitude of earthquakes is usually below 5,5 on the Richter scale [6]. The highest recorded earthquake in Norway is M_S (magnitude surface wave, richters scale) 5,8 in 1819 in Nordland, while the highes in the Oslo-fjord area is M_S 5,4 in 1904 [6]. The Norwegian western coast has far higher concentrations of earthquakes than the rest of the country, and even higher than the rest of Fennoscandia [6], [7] . See Figure 4 for registered earthquakes in Fennoscandia.

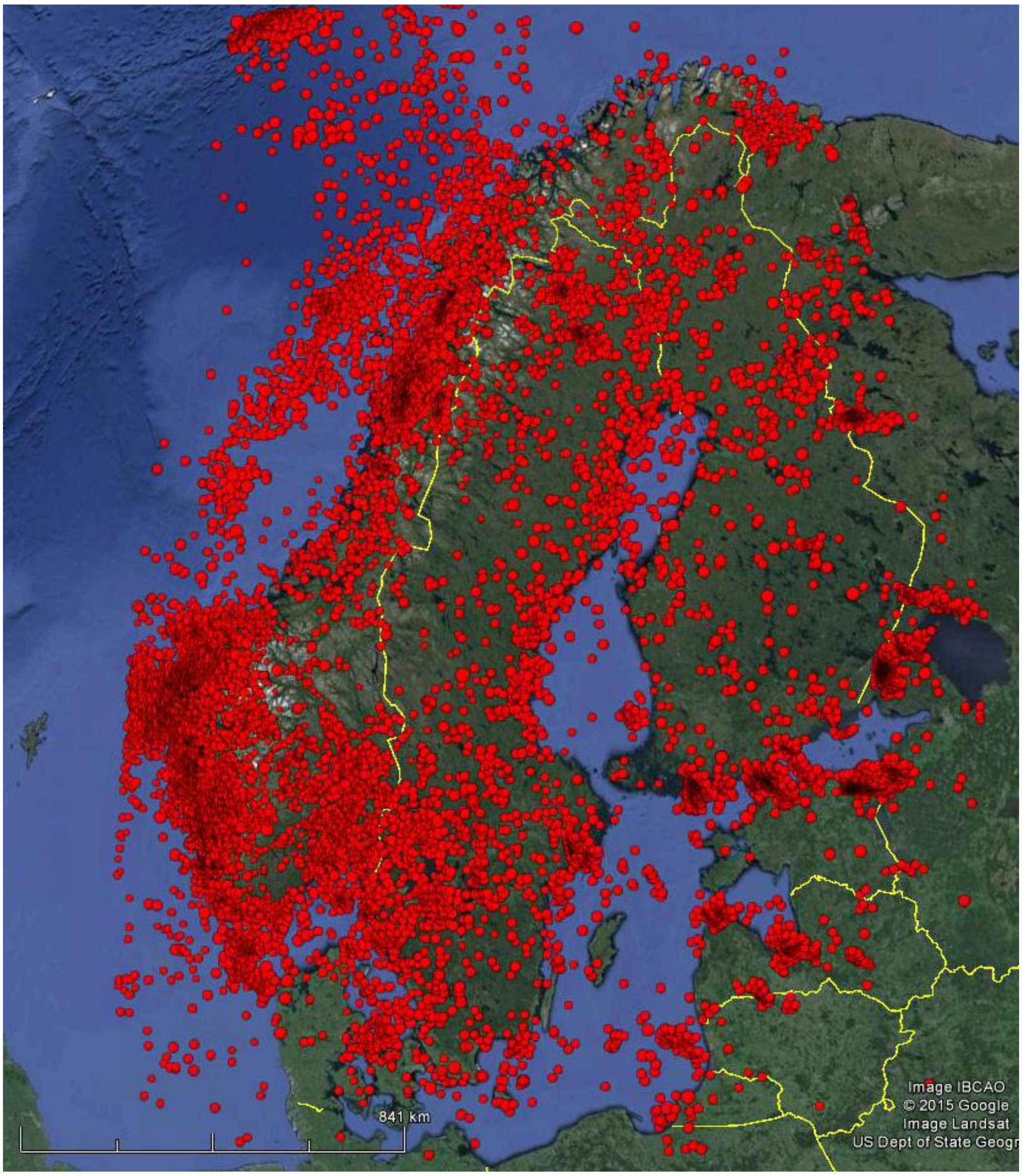


Figure 4: Seismicity in Fennoscandia, registered earthquakes from 1497 to January 1. 2015 [6]

2.2 Dynamic response

The general movement equation for a MDOF system, damped structure which expresses the position P of a point in a damped structure at time t whenever subjected to acceleration, moving at a velocity or is displaced. See Figure 5 below for a standard design spectrum which gives a general idea of the stages during shaking based on the dynamics in the equation of motion (eq. 2.2.4) when acceleration happens first. The figures are set with damping of 5% [8].

$$M\ddot{u} + C\dot{u} + Ku = P(t) \qquad (eq. 2.2.4)$$

where:

- P(t) is the position of a point at the structure at time t
- M is mass of the structure in matrix form
- $\ddot{u}$ is acceleration vector
- C is the damping of the structure in matrix form
- $\dot{u}$ is velocity vector
- K is the stiffness of the structure in matrix form
- u is displacement vector

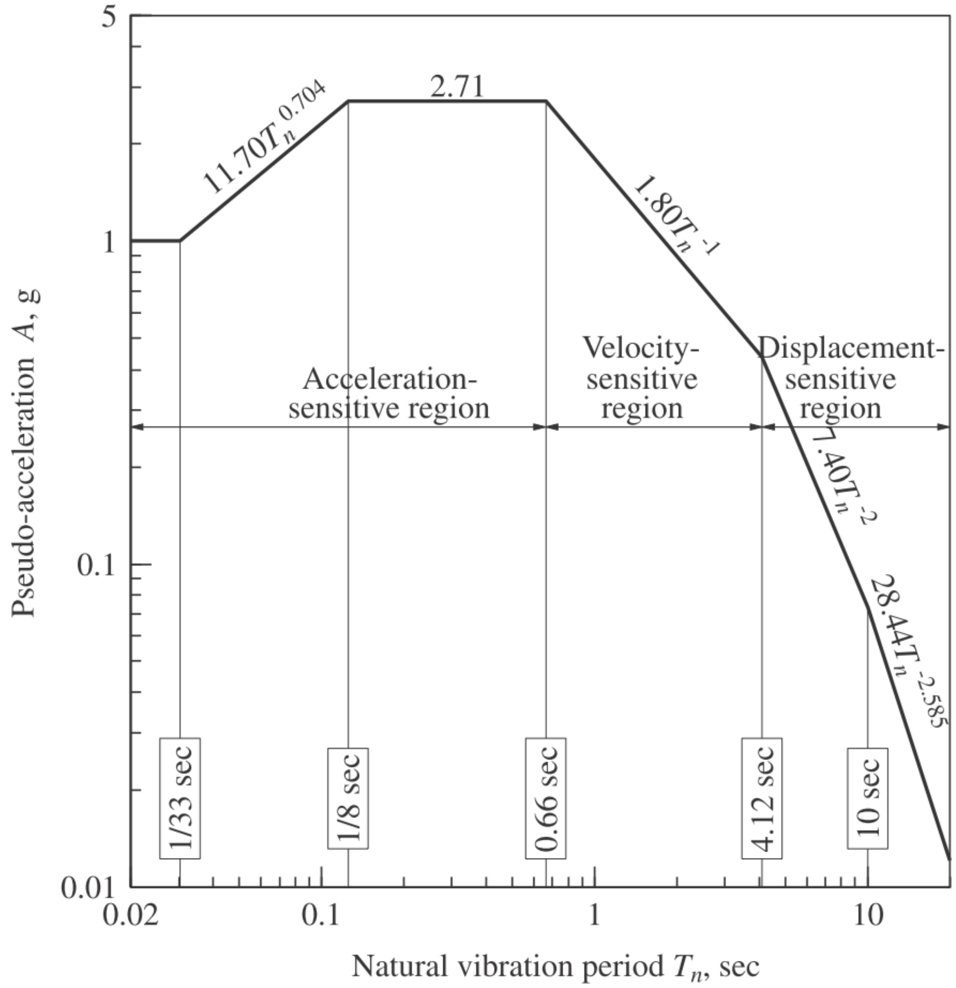


Figure 5: Design spectrum [8]

2.2.1 Damping

The damping of the structure decides how fast the structure will come to rest after being subjected to either acceleration, velocity of displacement. Higher damping means it comes to rest faster than with a low damping ratio. The damping of the structure is very hard to find by precisely by hand, while a number of approximations can be made based on the structures geometry and properties [8]. [8] provides several different ways to estimate the damping of a structure, see equations 2.2.5 – 2.2.10 below.

Mass proportional damping for the n'th mode with its modal damping ratio:

$$C_n = a_0 M_n \qquad (eq. 2.2.5)$$

$$\zeta_n = \frac{a_0}{2\omega_n} \qquad (eq. 2.2.6)$$

Stiffness proportional damping for the n'th mode with its modal damping ratio:

$$C_n = a_1 \omega_n^2 M_n \qquad (eq. 2.2.7)$$

$$\zeta_n = \frac{a_0 \omega_n}{2} \qquad (eq. 2.2.8)$$

Rayleigh damping combining the mass proportional and stiffness proportional:

$$\zeta_n = \frac{a_0}{2\omega_n} + \frac{a_1 \omega_n}{2} \qquad (eq. 2.2.9)$$

Estimation of modal damping ratios for concrete buildings:

$$\zeta_1 = 3{,}01 + 3{,}45 e^{-0{,}019H} \qquad (eq. 2.2.10)$$

where:

- a_0 is coefficient for damping
- M_n is mass at mode n
- ζ_n is damping at mode n
- ω_n is natural frequency at mode n
- a_1 is coefficient for damping
- H is height of the structure

Recommended standard values for damping varies from material and stress level the structure is operating under. See table 1, where it is clear that a stiffer structure has a lower damping, and a relatively flexible structure has high damping ratio [8]. For Kristina kvartalet, standard damping ratio 5% is used.

Table 1: Recommended damping values for structures based on materials, type and stress level [8]

Stress Level	Type and Condition of Structure	Damping Ratio (%)
Working stress, no more than about $\frac{1}{2}$ yield point	Welded steel, prestressed concrete, well-reinforced concrete (only slight cracking)	2–3
	Reinforced concrete with considerable cracking	3–5
	Bolted and/or riveted steel, wood structures with nailed or bolted joints	5–7
At or just below yield point	Welded steel, prestressed concrete (without complete loss in prestress)	5–7
	Prestressed concrete with no prestress left	7–10
	Reinforced concrete	7–10
	Bolted and/or riveted steel, wood structures with bolted joints	10–15
	Wood structures with nailed joints	15–20

2.2.2 Acceleration

Accleration excited into the structure in the instance of an earthquake, causing the building to be shaken and therefore affected by large shear and torsion forces at each floor, with the largest forces at the base [8]. The basic equation for a single-degree freedom system subjected to ground acceleration $-\ddot{u_g}(t)$ [8] in equation 2.2.11 is:

$$\ddot{u} + 2\zeta\omega_n\dot{u} + \omega^2 u = -\ddot{u}_g(t) \qquad (eq. 2.2.11)$$

where:

- $\ddot{u}$ is acceleration vector
- ζ is damping
- ω_n is natural frequency at mode n
- $\dot{u}$ is velocity vector
- u is displacement vector

2.3 Piles

The piles of a structure have an important role when a structure is excited to an earthquake, in addition to the usual static loads, they must withstand high loads in all axis in addition to dynamic soil behavior, where the type and characteristics of the soil is very important. During and after an earthquake, the soil may experience increase in pore pressure within the soil, time-dependent vertical and lateral ground movements [9]. Designing piles during static loads is simpler, where buckling is the main design load [9], given in equations 2.3.1 and 2.3.2 below:

$$P_E = \frac{\pi^2 EI}{L_e^2} \qquad (eq. 2.3.1)$$

$$SR = \frac{L}{\sqrt{IA}} \qquad (eq. 2.3.2)$$

where:

- P_E is buckling load
- E is modulus of elasticity
- I is minimum moment of inertia
- L_e is equivalent length of pile
- A is area of the pile cross section
- SR is slenderness ratio, which should be equal or less than 50 to avoid buckling instability

2.3.1 Soil-structure interaction (SSI)

Soil-structure interaction – the relationship between the soil and the structure is highly relevant during vibrations such as an earthquake, as the soil will change its characteristics.

When the structure is exposed to incident waves, the phenomena kinematic interaction occurs [10]. Further, the response of the base and foundation of the structure is further modified due to the response of the stuperstructure and inertial loads transferred back to the structure [10]. This – inertial interaction has been found to affect the foundation motion only in a limited frequency range around the fundamental frequency of the structure [10].

Kinematic and intertial ineraction are the two components of SSI which are treated as successive steps during analysis, although they occur at the same time [10].

2.3.2 Damping in foundations

The damping of the foundation has been researched over time, and several models from various researchers can be used. The paper "Simplified discrete systems for dynamic analysis of structures on footings and piles" [11] reviews two former methods for determining the natural period of the soil-structure sytems, as well as introducing a new method and equations.

Damping of Surface rigid foundation [11] in equations 2.3.4 and 2.3.5:

$$\tilde{\zeta} = S\left[\frac{\zeta_x}{\omega_\chi^2(1+4\zeta_\chi^2)} + \frac{\zeta_\theta}{\omega_\theta^2(1+4\zeta_\theta^2)} + \frac{\zeta}{\omega_c^2(1+4\zeta^2)}\right] \qquad (eq. 2.3.4)$$

$$S = \left[\frac{1}{\omega_\chi^2(1+4\zeta_\chi^2)} + \frac{1}{\omega_\theta^2(1+4\zeta_\theta^2)} + \frac{1}{\omega_c^2(1+4\zeta^2)}\right]^{-1} \qquad (eq. 2.3.5)$$

where:

- $\tilde{\zeta}$ is damping of soil-structure system
- ζ_x is damping under swaying conditions
- ζ_θ is damping under rocking oscillations
- ζ_c is damping of the structure in its fixed base position
- ζ is damping of the structure under fixed base position
- ω_χ is fictional uncoupled circular natural frequency under swaying iscolations
- ω_θ is fictional uncoupled circular natural frequency under rocking oscillations
- ω_c is circular natural frequency of structure in its fixed base position

Further, the paper "Some cornerstones of dynamic soil-structure interaction" [12] uses another variation, equations 2.3.6 and 2.3.7:

$$\tilde{\zeta} = \left(\frac{\tilde{\omega}}{\omega_c}\right)^2 \zeta + \left[1 - \left(\frac{\tilde{\omega}}{\omega_c}\right)^2\right]\zeta_c + \left(\frac{\tilde{\omega}}{\omega_\chi}\right)^2 \zeta_\chi + \left(\frac{\tilde{\omega}}{\omega_\theta}\right)^2 \zeta_\theta \qquad (eq. 2.3.6)$$

$$\omega_c = \sqrt{\frac{k}{m}}, \qquad \omega_\chi = \sqrt{\frac{K_\chi}{m}}, \qquad \omega_\theta = \sqrt{\frac{K_\theta r^2}{mh^2}} \qquad (eq. 2.3.7)$$

where:

- $\tilde{\zeta}$ is damping of soil-structure system
- $\tilde{\omega}$ is circular natural frequency of soil-structure system
- ω_c is circular natural frequency of structure in its fixed base position
- ω_χ is fictional uncoupled circular natural frequency under swaying oscillations
- ω_θ is fictional uncoupled circular natural frequency under rocking oscillations
- ζ is damping of the structure
- ζ_c is damping of the structure in its fixed base position
- ζ_χ is damping under swaying oscillations
- ζ_θ is damping under rocking oscillations

Last, the paper "Lateral and Rocking Vibration of Footings" [13] have their proceeding, equations 2.3.8 and 2.3.9:

$$\tilde{\zeta} = \tilde{\zeta}_0 + \left(\frac{\tilde{T}}{T}\right)^{-3} \zeta \qquad (eq. 2.3.8)$$

$$\tilde{T} = T\sqrt{1 + \frac{k}{K_\chi}\left(1 + \frac{K_\chi h^2}{K_\theta}\right)} \qquad (eq. 2.3.9)$$

where:

- $\tilde{\zeta}_0$ is the contribution to the overall damping of the radiation damping of the footing
- $\tilde{T}$ is the natural period of the soil-structure system
- T is the natural period of the structure in fixed-position condition
- K_χ is modulus of distributed Winkler springs
- h is the height of the structure

2.3.3 Liquification of soil

During earthquakes, the soil may start to liquify as a result of vibrations and seismic energy passing through it. If liquification occur, the soil will result in an almost complete loss of strength and stiffness in the soil, and large lateral ground movements, increasing the risk of collapse [9]. The potential (I_L) and risks (I_R) for liquefacation can be calculated by equations 2.3.10 – 2.3.15 [14]:

$$I_L = \sum 20F_1 \cdot W(z)dz \qquad (eq. 2.3.10)$$

$$F_1 = 1 - F_s \qquad (eq. 2.3.11)$$

$$I_R = \sum 20P_L \cdot W(z)dz \qquad (eq. 2.3.12)$$

$$P_L = 11 + \frac{F_s}{0{,}96}^{4{,}5} \qquad (eq. 2.3.13)$$

$$F_s = \frac{f_s}{q_c - \sigma_v} \cdot 100\% \qquad (eq. 2.3.14)$$

$$W(z) = 10 - 0{,}5z \qquad (eq. 2.3.15)$$

where:

- P_L is probability of liquefacation
- w(z) is the weight function
- f_s is sleeve friction
- q_c is the cone tip resistance of soil
- σ_v is total overburden stress
- z is the depth from the ground surface in meters

During liquefaction, large ground displacements can take place on sloping ground or towards and open face, river bank or sea [15]. Displacements of over 10 meters have been recorded during a major earthquake in Niigata in 1964 [15]. The following points must be considered in dimensioning of piles [15]:

1. Temporary loss of axial load capacity
2. Possibility of axial buckling of the pile
3. Bending moment and shears developed in the pile to inertial and kinematic loadings

There are several methods for calculating adjusted forces and torsions under various circumstances:

Peak bending moment

Approximation for peak bending moment M_{pk} during the transient phase of seismic excitation [9] in equations 2.3.16 – 2.3.18:

$$M_{pk} = \eta M_{res} \qquad (eq. 2.3.16)$$

$$M_{res} = 0{,}048\tau_c d^3 \left(\frac{L}{d}\right)^{0,3} \left(\frac{E_p}{E_1}\right)^{0,65} \left(\frac{v_{s2}}{v_{s1}}\right)^{0,5} \qquad (eq. 2.3.17)$$

$$\tau_c = a_s \rho_1 h_1 \qquad (eq. 2.3.18)$$

where:

- M_{res} is bending moment developed under resonant conditions
- η is reduction factor to allow for non-resonant conditions
- d is pile diameter
- L is pile length
- E_p is Young's modulus of pile
- E_1 is Young's modulus of upper layer
- v_{s1} is average shear wave velocity in upper layer
- v_{s2} is average shear wave velocity in lower layer
- a_s is peak ground surface acceleration
- ρ_1 is mass density of upper layer
- h_1 is thickness of upper layer

Upper limit kinematic bending moment

Upper limit to kinematic bending moment – if the layer has completely liquified and flows past the pile, the limiting kinematic bending moment [9] is given by eq. 2.3.19:

$$M_{klim} = n s_{u-Liq} d h_1 (h_1 + dh) \qquad (eq. 2.3.19)$$

where:

- n is undrained shear strength mulitiplier for limiting lateral pile-soil factor
- $s_{u\text{-}Liq}$ is shear strength of liquefied soil
- d is pile diameter
- h_1 is thickness of liquefied layer
- dh is additional distance within underlying layer at which the maximum moment occurs, expected to be 0,5d to 1d

Estimation of intertial bending moment in pile

Estimation of inertial bending moment in pile, in case of complete liquefaction of the upper layers occur, the intertial force [9] can be estimated by eq. 2.3.20:

$$H_i = a_s \cdot P \qquad (eq. 2.3.20)$$

where:

- a_s is peak ground acceleration
- P is vertical load acting on pile

Maximum moment due to inertial loading

Assuming that an elastic analysis of the pile in an liquefied layer can be performed and a constant Young's modulus can be used for the layer, the maximum moment due to inertial loading, M_{imax} [9]can be estimated from eq. 2.3.21 – 2.3.23:

Free-head pile:

$$M_{imax} = 0{,}1H_i(L_c + h_1) \qquad (eq. 2.3.21)$$

Fixed head pile, fixed moment at pile head:

Similar to version with free-head pile, while in this instance the pile is fixed ad the head [9].

$$M_{imax} = -0{,}1875H_i(L_c + h_1) \qquad (eq. 2.3.22)$$

$$L_c = d\left(\frac{E_2}{G_{red}}\right)^{\frac{2}{7}} \qquad (eq. 2.3.23)$$

where:

- H_i is inertial force on pile (from eq. 2.3.14)
- h_1 is depth of liquefied soil
- L_c is critical pile length in the non-liquefied soil
- E_2 is young's modulus of the non-liquefied layer
- G_{red} is shear modulus of the non-liquefied layer

2.4 Eurocode standard

Eurocode is a series of standards for projecting and engineering of structures in Europe. The code is divided into 10 subchapters, expressing rules on how to consider the structures and how to engineer them in a safe and standardized way. This also includes situations with non-ideal settings and circumstances that can increase loads, and the general solution is to have higher load- and material factors to increase the safety in regards to collapse. There are also some national additions to the standards, where specific conditions for that country applies.

2.4.1 Damping

Eurocode does not provide a specific damping ratio to use in certain situations, but 5% damping is considered a standard value that can be used in most cases. For special structures and buildings with high seismic importance class, a precise estimate for the damping in the structure is often calculated [16].

2.4.2 Modal analysis

Number of modes that must be included in modal analysis is decided by either point a og b, equation 2.4.1, Eurocode 8 – 4.3.3.3.1 [16]:

a) Relative mass > 90 % and
All modes with relative mass > 5% is included in the analysis

b)

$$k \geq 3\sqrt{n} \; and$$
$$T_k \leq 0{,}20\; s \qquad (eq. 2.4.1)$$

where:

- k is number of modes used
- n is number of floors of the structure
- T_k is natural period of mode k

2.4.3 Base shear at foundation level or top of stiff basement

The total seismic load F_b is determined by eq. 2.4.2 – 2.4.7, Eurocode 8 [16]:

$$F_b = S_d(T_1) m\lambda \qquad (eq. 2.4.2)$$

$$T_1 = C_t \cdot H^{\frac{3}{4}} \qquad (eq. 2.4.3)$$

$$for\ 0 \leq T \leq T_B: \quad S_d(T) = a_g S\left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \qquad (eq. 2.4.4)$$

$$for\ T_B \leq T \leq T_C: \quad S_d(T) = a_g S\frac{2{,}5}{q} \qquad (eq. 2.4.5)$$

$$for\ T_C \leq T \leq T_D: \quad S_d(T) = a_g S\frac{2{,}5}{q}\left[\frac{T_C}{T}\right] \geq \beta a_g \qquad (eq. 2.4.6)$$

$$for\ T_D \leq T: \quad S_d(T) = a_g S\frac{2{,}5}{q}\left[\frac{T_C T_D}{T^2}\right] \geq \beta a_g \qquad (eq. 2.4.7)$$

where:

- $S_d(T_1)$ is the ordant of dimensioning spectre for period T_1
- T_1 is first natural period of the structure
- m is total mass of the structure
- λ is correction factor where $\lambda = 0{,}85$ if $T_1 \leq 2T_c$ and the building has more than two floors. If not: $\lambda = 1{,}0$
- C_t is 0,075 for moment stiff spacious concrete frames
- H is height of the building
- Factors T_B, T_C and T_D is properties form the soil, see tables 3 and 4
- q is structural factor
- β is factor for lower limit value for horizontal dimension spectre
- S is soil enhancement factor

2.4.4 Seismic importance and soil factors

Eurocode provides several tables to use in calculations, where factors for the soil is important as one type may reduce and another type could amplify the acclerations to the structure. See table 2 for importance class / seismic class and tables 3 and 4 for factors regarding soil type [5], [16].

Table 2: Importance factor for buildings [5]

Importance class	Buildings	Importance factor γ_I (recommended value)
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0,8
II	Ordinary buildings, not belonging in the other categories.	1,0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,4

Table 3: Ground types with parameters [5]

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

Table 4: Extended parameters for soil types [15]

Grunntype	S	T_B (s)	T_C (s)	T_D (s)
A	1,0	0,10	0,20	1,7
B	1,3	0,10	0,25	1,5
C	1,4	0,10	0,30	1,5
D	1,55	0,15	0,40	1,6
E	1,65	0,10	0,30	1,4

2.4.5 Seismic zones

Eurocode 8 provides a general map for the seismic zones in Norway amongst others to use as basis acceleration. See Figure 6 where Tønsberg is marked with a red dot. This map indicate to use a_{g40Hz} = 0,5 m/s^2 as direct input in Autodesk Robot.

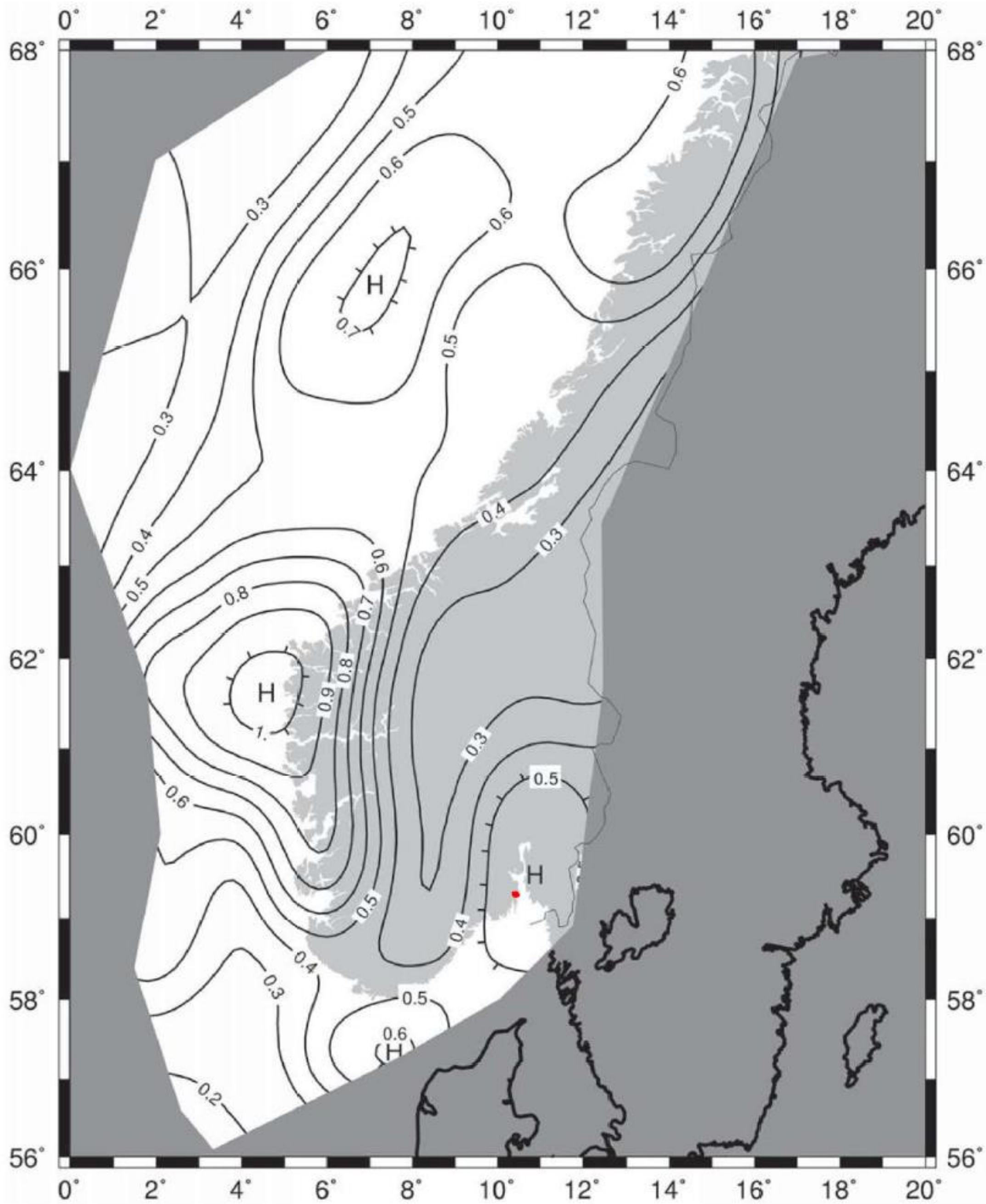


Figure 6: Current seismic map of Norway [16]

2.4.6 Soil-structure interaction

Eurocode 8 – NS-EN 1998-5 (6) [17] states that the effects of soil-structure interactions shall be considered when one of the following criterias is met:

a) Structures where P-δ (2^{nd} order) effects play a significant role (amplifying effect for combination of static and dynamic loads, primarily in inelastic structures)
b) Structures with massive or deep-seated foundations, such as bridge piers, offshore installations and silos
c) Slender tall structures such as towers and chimneys
d) Structures supported on very soft soils, with average wave velocity less than 100 m/s, such as soil type S_1

Kristina kvartalet meets both criteria a) and d) and is therefore recommended to be analysed for soil-structure interactions.

2.5 Modifications and engineering decisions, evaluations

2.5.1 Modification of roof of 4th Floor to 5th floor

The roof of 4th floor was previously smaller than its floor, with an angled façade. See Figures 7 and 8. The roof is expanded to function as 5th floor. The slab for 6th and 7th floor will be identical to 5th floor, while the roof of 7th floor will be identical to the previous roof of 4th floor. The new floor of 5th is now considered as good as a normal, existing slab with the same geometry in the calculations. The new roof will have the same geometry as the previous roof of 4th floor. This is the only major structural change second to the expansion of the entire structure. See Figures 7 and 8.

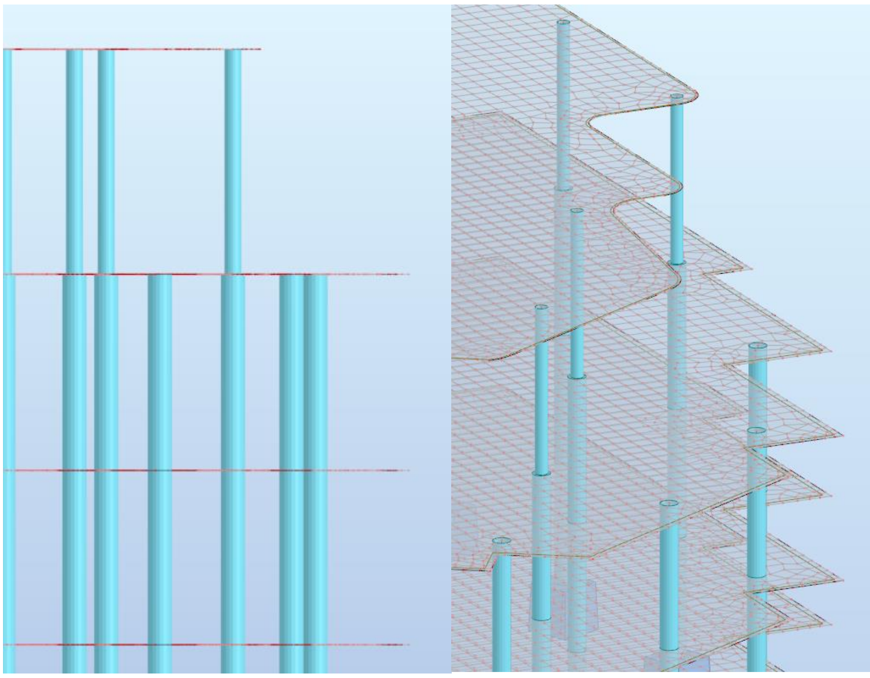


Figure 7: Existing difference between 4th floor and roof

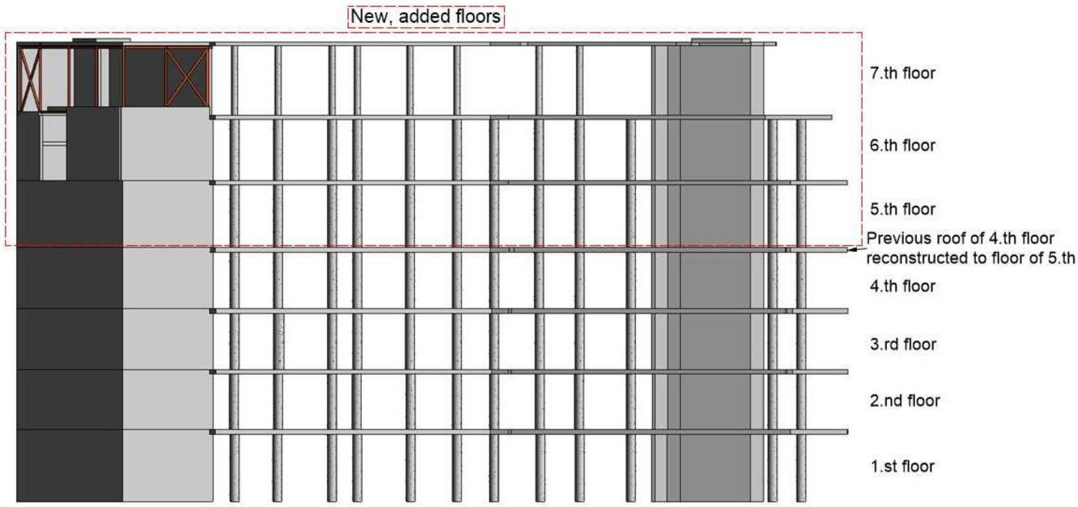


Figure 8: Illustration floors

The concrete slab is expanded from the red line to the green line on the outer side with circular edges, marking of what reinforcement to use in the various parts, which is not the same all over. This is done by jack hammering into the slab to unveil the reinforcement which the expansion part of the floor with its reinforcement is connected to. See Figure 9 below.

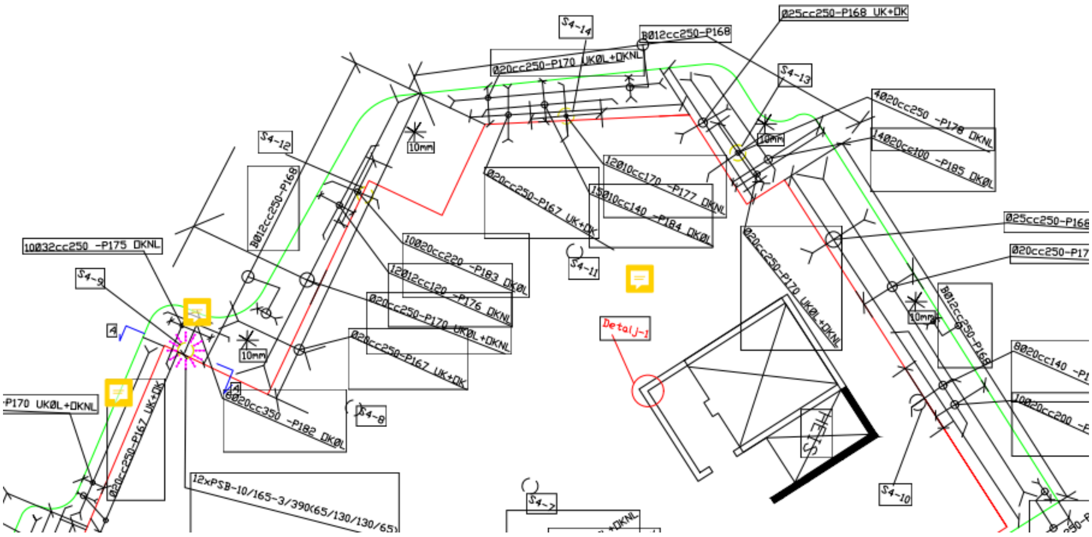


Figure 9: Expansion of concrete slab with reinforcement

2.5.2 Assumptions for engineering and calculations

For a potential seismic analysis, the basement structure is so stiff that the structure can be considered fixed in place by the columns and walls in 1st floor. (EC8 NA. 3.2.1(5)P) [16]states that fixed support at base level is a sub-criterias to qualify for omission criteria #3 – dimensioning spectre (chapter 2.5.4). Further, the assumption of rigid basement and fixed support is seemingly a normative assumption used in many situations for these types of analysis.

The new floors and the expanded previous roof of 4th floor into the cover of 5th floor is modified in such a way that the stiffness and weight is identical to the existing floors. This is very close to reality and makes the calculations and analysis easier when all floors are the same.

Neighbouring structures is ignored in this analysis. This is because the contact between them is not fixed, and will at best work as a pinned line support in one direction from the specific structure. This is done as the effect they have on this analysis is considered small, and a precise value for a partially flexible support is hard to determine.

2.5.3 Engineering decisions and evaluations during the process

The project has previously been analysed for static loads only, considering the new floors on top of the existing structure. The slabs, columns and walls were considered strong enough, and concluded not to be strengthened for increased loads. During the process, several smaller considerations, changes and decisions has been made.

2.5.4 Seismic dimensioning, omission criteria / utelatelseskriterie

NORSAR – a Norwegian research institute specializing in seismology and applied geophysics have updated their database of seismic zones over Norway [1], [18], where most places now have a lower dimensioning acceleration. These updated maps must be bought for each single project they shall be used for. An independent geotechnical engineering firm – Grunnteknikk AS was involved to decide whether seismic dimensioning could be omitted for this project.

Since Norway is classified as an area with low seismic activity, Eurocode 8 gives to opportunity to omit seismic dimensioning to be conducted, if one of certain criterias is met. These are:

Criteria 1: Structural type / konstruksjonstype – Structures in seismic class 1, which is primarily structures with low significance for public health, see table 5 (same as table 2) below.

Table 5: Seismic classes / importance factor [16]

Importance class	Buildings	Importance factor γ_I (recommended value)
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0,8
II	Ordinary buildings, not belonging in the other categories.	1,0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1,2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1,4

Criteria 2: Very low seismic activity – Satisfactory low activity so that dimensioning for seismicity can be neglected – defined in EC8 NA 3.2.1(5) [16] as equation 2.5.1.

$$a_g \cdot S = \gamma_1 \left(0{,}8 \cdot a_{g40Hz}\right) S < 0{,}49 \frac{m}{s^2} \qquad (eq. 2.5.1)$$

where:

- γ_1 is factor for seismic class
- a_{g40Hz} is peak value for bedrock acceleration
- S is factor for the soil type

Criteria 3: Dimensioning spectrum – Response spectrum, a descriptive representation of the influence on the structure. Typically a graph which shows displacement response over time during the earthquake. The following subcriterias must be met [16]:

- $S_d(T) < 0{,}49 \frac{m}{s^2}$
- Structural factor $q \leq 1{,}5\ DCL$
- No reduction in striffness properties after EC8 4.3.1 (7)
- The structure is considered fixed support

Criteria 4: Size of forces – For structures not in seismic class 4, it is not necessary to know sufficient resistance to seismic loads if the shear force at ground level or right above basement level is less than from other relevant load combinations, equation 2.5.2 [16].

$$F_b < (1{,}5 wind + 1{,}05\ skew) \left(\frac{\gamma_{c\ ULS}}{\gamma_{cDCL}}\right) \qquad (eq. 2.5.2)$$

where:

- F_b is horizontal base shear force due to earthquake
- Wind is wind load
- Skew is loads due to skewed position
- $\gamma_{cULS}/\gamma_{cDCL}$ is relationship between material factors in ordinary ultimate state limit and seismic loading

The geotechnical engineers from Grunnteknikk AS have used NORSAR's updated seismic data in their evaluation to implement the omission criterias [19]. They conclude that the structure can be omitted for seismic dimensioning with criteria 2, as the seismic acceleration is low enough, see equation 2.5.3 [19]. Dimensioning acceleration in soil a_g is 0,2787 m/s^2 and enhancement factor S is conservatively set to 1,7 [19]. See attachement report produced by Grunnteknikk AS.

$$a_g \cdot S = 0{,}474 \frac{m}{s^2} < 0{,}49 \frac{m}{s^2} \qquad (eq. 2.5.3)$$

3 Method

3.1 Structure information for Kristina kvartalet

The structure used in this thesis is the Kristina kvarteret / Kristina quarter, address Farmannsveien 3, Tønsberg, Norway. This building is used as an office building will be expanded by three stories, and an expanded unit for apartments in addition to more office space. There is also two stories with parking beneath the building. The structure is built in 1983 in all concrete, and uses both columns and walls as load carrying system. The outer façade are not load carrying. Part of the quarter is considered not connected to the structure, and is therefore not included, this can bee seen in the difference between the view from rear side and the 3D model used in the project, see Figures 18 – 20. There is no good model of the structure before the construction, but it is similar to the one in figures with some angled roofing. The new height of the structure above ground level is 25,15 meters with floor height. In the analysis, the neighbouring parts is considered not attached to the structure, as they are not expanded. All non-load bearing parts of the structure is removed in the model. In this report, 3 variations (see Figures 14 – 18) of the structure will be presented and analysed. These are:

1: The final structure expanded with concrete casted on site

2: The final structure where the expanded floors are hollow core concrete slabs and the columns are H steel beams.

3: The original structure with 4 floors + roof.

Load bearing members:

The load bearing structure is all reinforced concrete, mainly supported by columns. Walls for stairwell and elevators and dividing wall on the southern part of the structure. Se tables 6 – 8 for data of materials, Figures 10 – 12 for vizualisation of settings in Robot.

Table 6: Slabs thickness and quality

Floor level	Floor Thickness [mm]	Concrete quality	Floor height [m]
1	230	C35	3,95
2	230	C35	3,28
3	230	C35	3,32
4	230	C35	3,35
5	230	C35	3,65
6	230	C35	3,6
7	230	C35	4
Roof	230	C35	-

Table 7: Columns diameter and quality

Floor level	Column Diameter [mm]	Concrete quality
1	500	C45
2	500	C45
3	500	C45
4	500	C45
5	500	C45
6	500	C45
7	350	C45

Table 8: Walls thickness and quality

Walls placement	Wall Thickness [mm]	Concrete quality
Basement main part	200 mm	B25
Basement east outer wall	250 mm	B25
Stairwell and elevator shaft	180 mm	B25

Settings for load bearing members within autodesk robot

Settings for slabs in regard to material and thickness in robot, Figure 10.

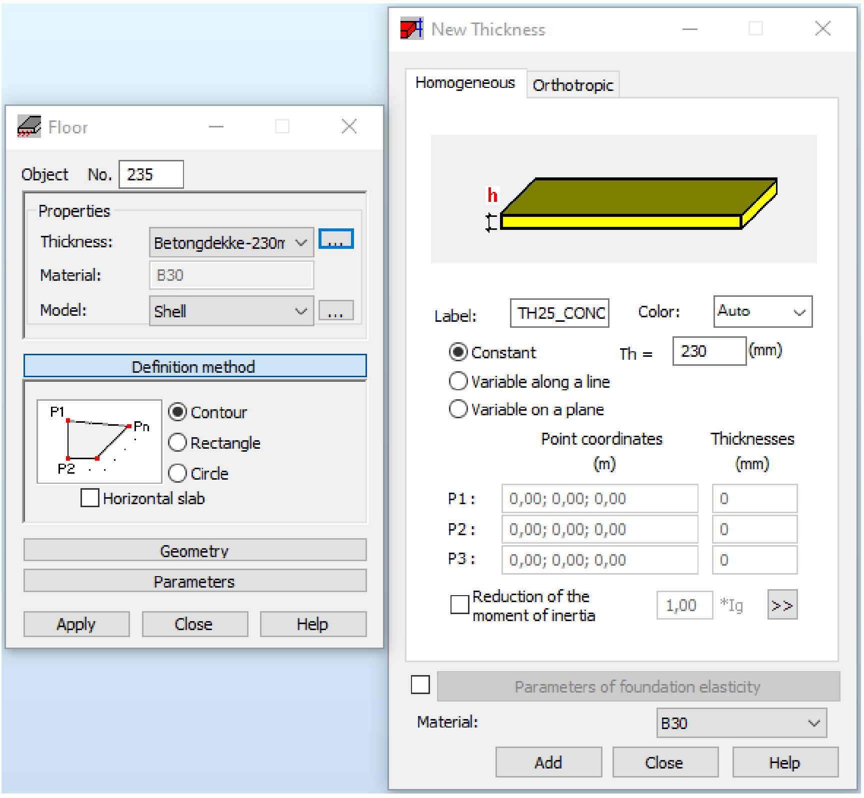


Figure 10: Settings for slabs within Autodesk Robot

Settings for columns in regard to material and diameter in robot, Figure 11.

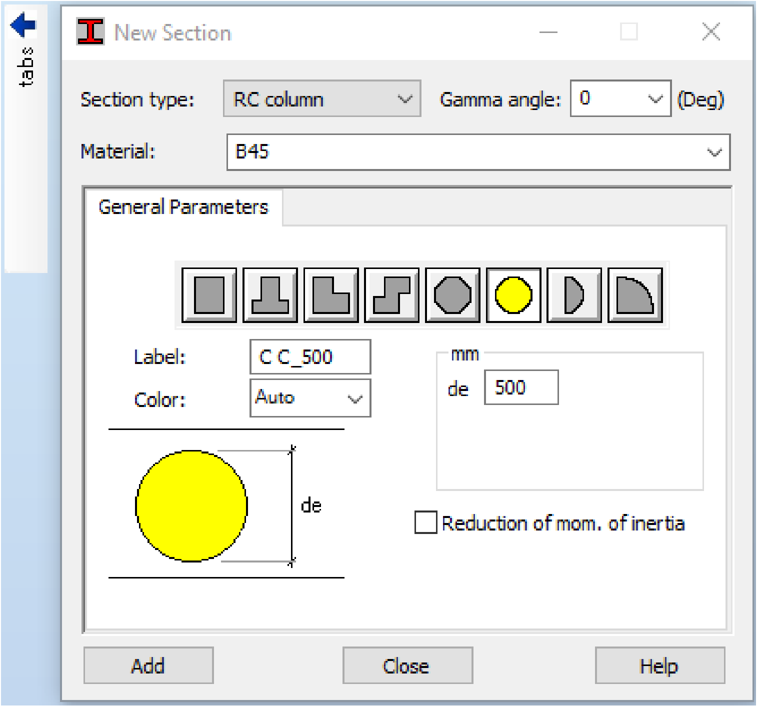


Figure 11: Settings for columns within Autodesk Robot

Settings for the internal walls around the stairwells in regard to thickness in robot, Figure 12.

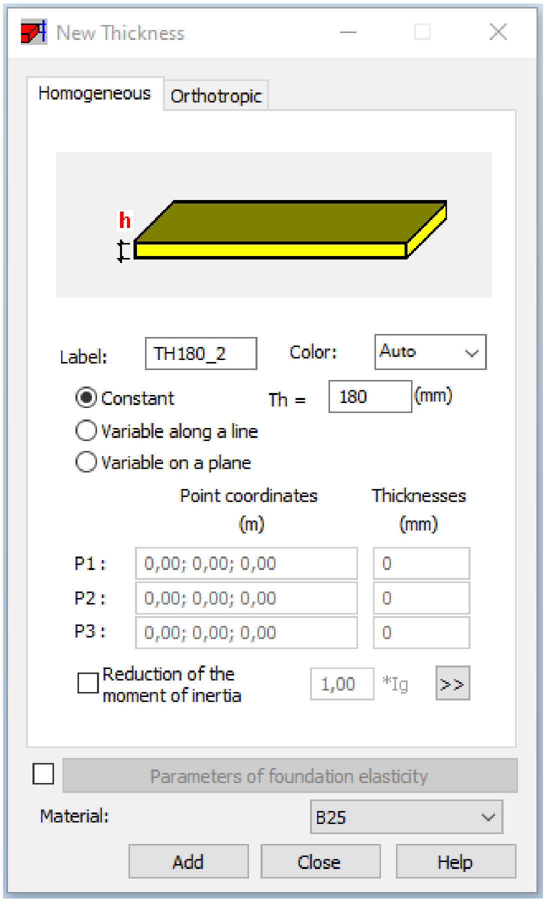


Figure 12: Settings for walls within Autodesk Robot

Reinforcement

Slabs: Reinforcement of the slabs varies according to distance between columns resulting in different loads, but the typical choise is Ø10 C/C 200 mm and Ø12 C/C 200 mm both overside and underside in length and Ø16 C/C 200 mm cross. Around joints with columns and walls, coarser and more dense reinforcement is used. Cover thickness overside and underside 1,5 cm.

Columns: Typical reinforcement of the columns is 10Ø25 in length and Ø10 C/C 300 mm hoops. Some columns have variations. Cover thickness is 3,5 cm. See Figures 13 and 14.

Walls: Typical reinforcement of the internal walls is Ø10 C/C 200 mm on both sides, with coarser and denser at the ends of the walls, and cross reinforcement Ø12 C/C 150 mm. Variations do accur. See Figure 15 and 16.

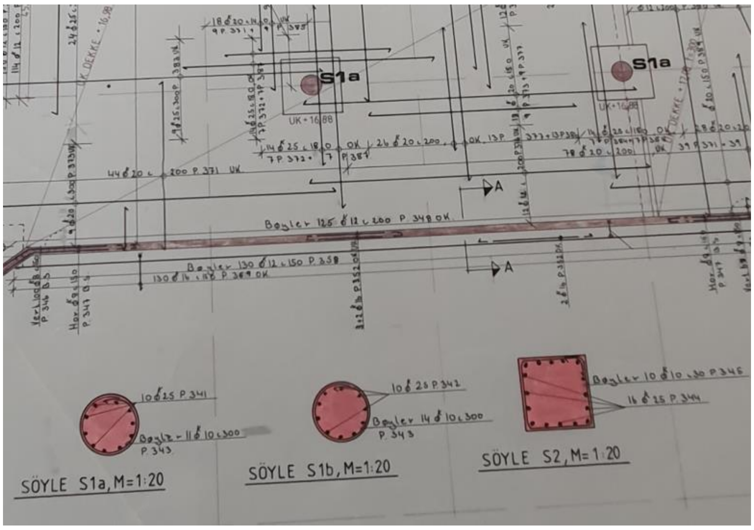
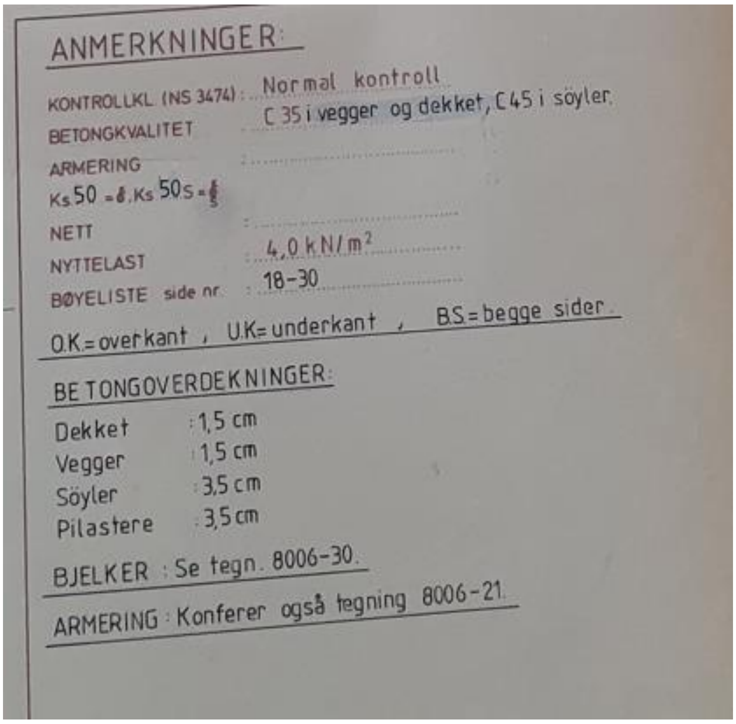


Figure 13: Reinforcement of columns



ANMERKNINGER:

KONTROLLKL (NS 3474): Normal kontroll
BETONGKVALITET: C 35 i vegger og dekket, C45 i söyler.
ARMERING
Ks 50 = ∅, Ks 50s = ∮
NETT
NYTTELAST: 4,0 kN/m²
BØYELISTE side nr.: 18-30
O.K.=overkant, U.K.=underkant, B.S.=begge sider.

BETONGOVERDEKNINGER:

Dekket	1,5 cm
Vegger	1,5 cm
Söyler	3,5 cm
Pilastere	3,5 cm

BJELKER : Se tegn. 8006-30.

ARMERING : Konferer også tegning 8006-21.

Figure 14: Cover thickness for reinforcement

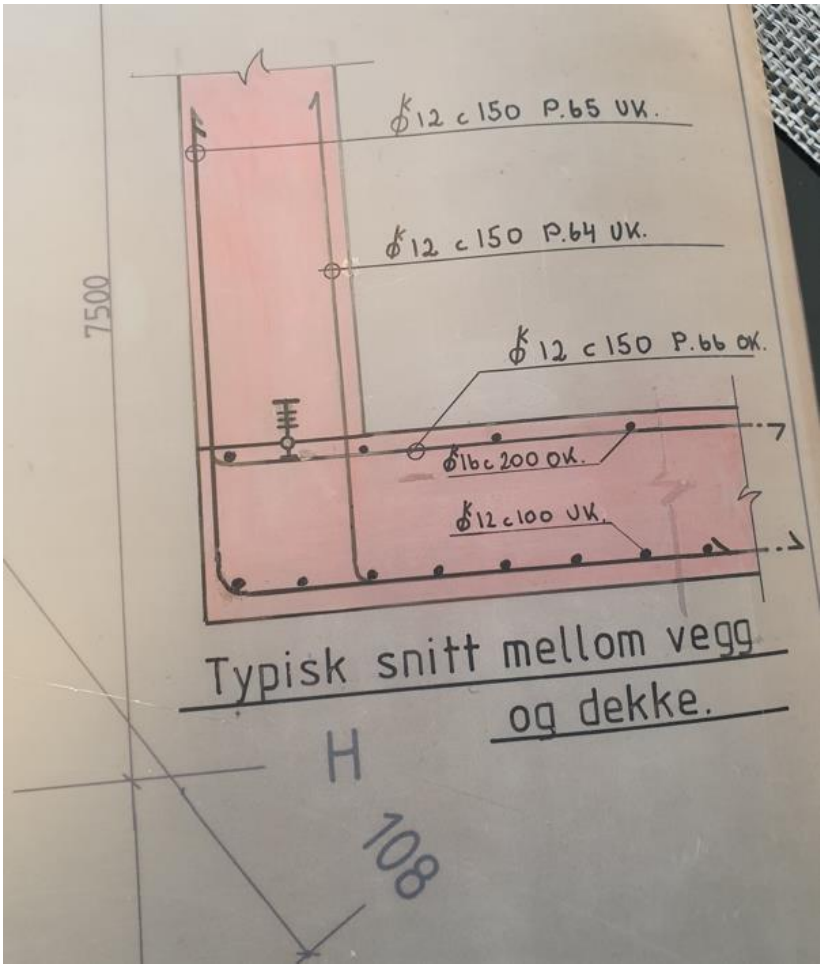


Figure 15: Typical reinforcement for transition wall and slab

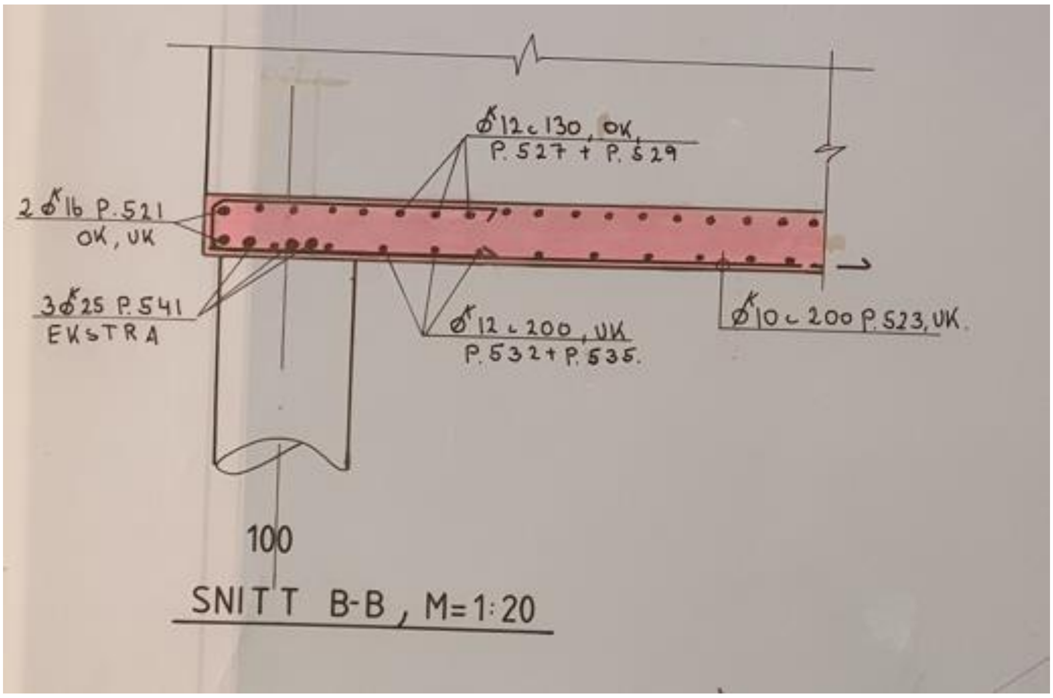


Figure 16: Typical reinfocement in transition slab to column

3.2 BIM model of the building

The main method for analysis is to use Autodesk Robot Structural Analysis Professional 2022 with an accurate 3D model, which will provide accurate results given that accurate data has been used. The model is extracted from a revit file into Robot Structural Analysis and cleaned for everything that is not load carrying, so it is only concrete slabs, walls and columns left. Figures 17 and 18 shows the structure as built after expansion, type 1. Figure 19 shows the axis net of the building with some of the neighbouring structures.

The figures to show the procedure of inputs into Robot is shown in a separate subchapter – chapter 3.4.

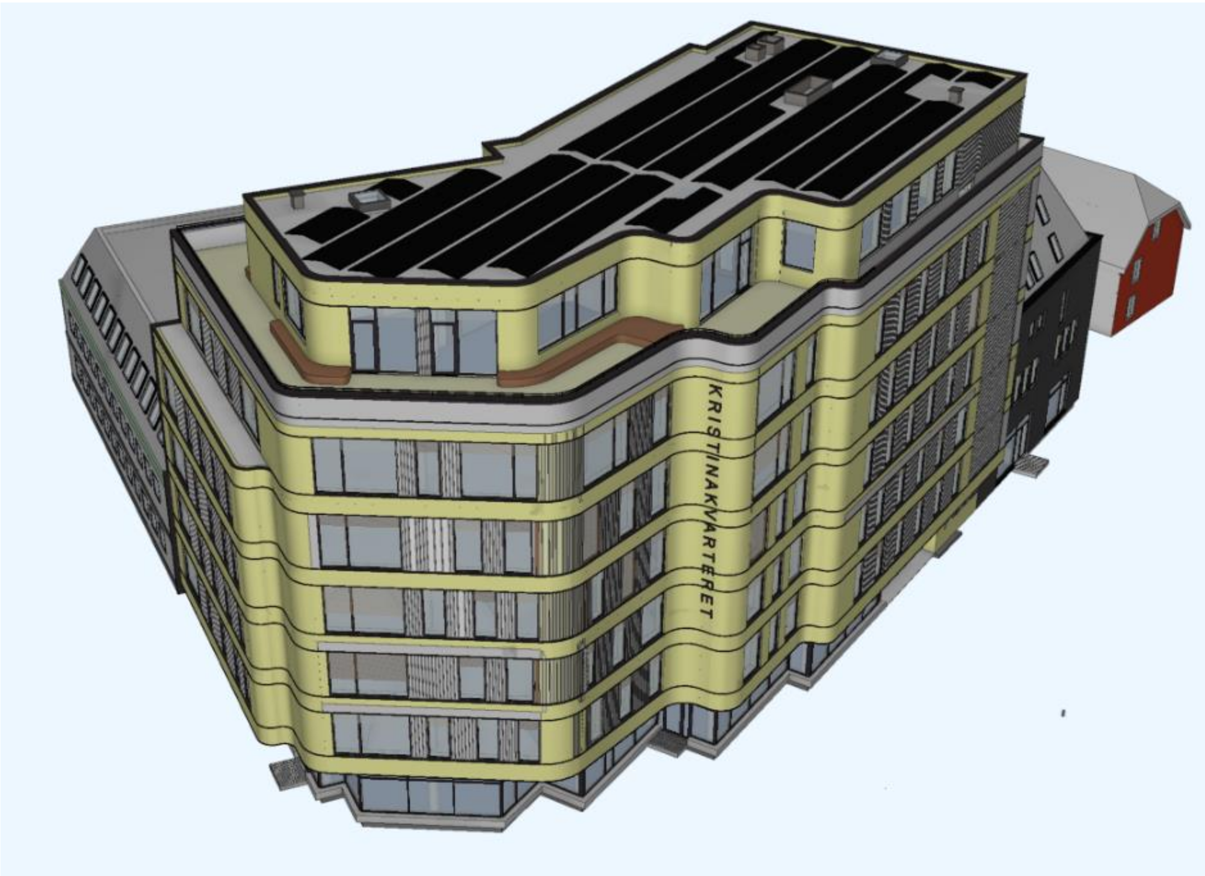


Figure 17: View of structure front after construction [20]

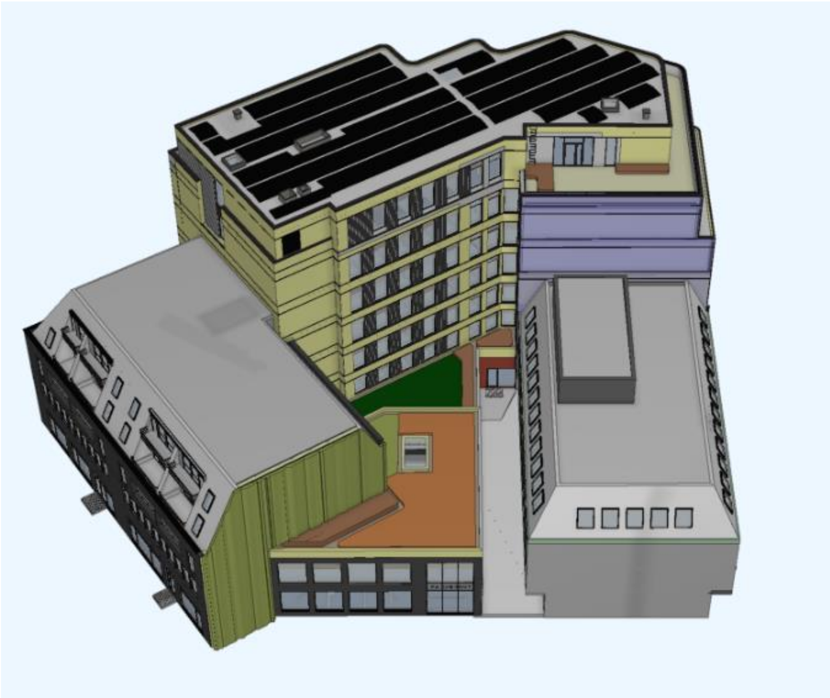


Figure 18: View of structure from rear after construction [20]

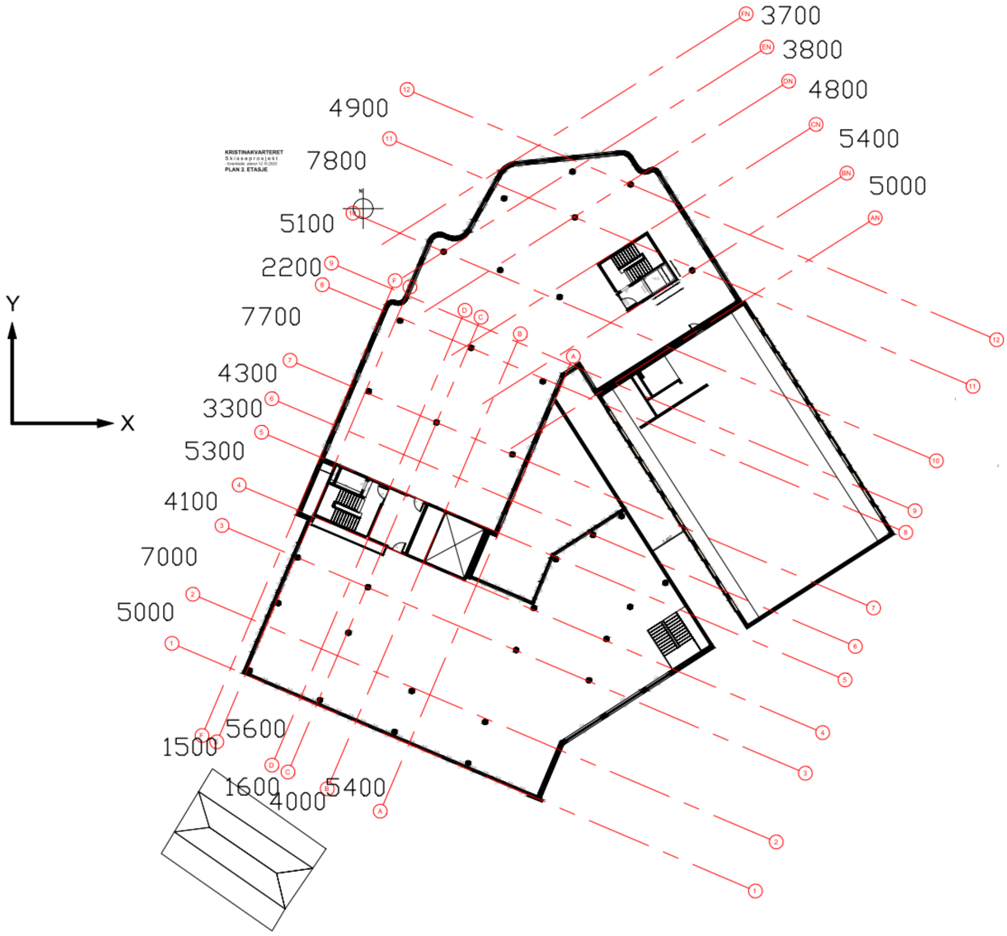


Figure 19: Axis net of the model, horizontal axis [20]

3.2.1 Structure variation 1 – As built

This type represents the actual structure as it will be built. The additional floors will be identical to the existing floors and column layout, solid cast-in-place reinforced concrete. Figures 20 and 21.

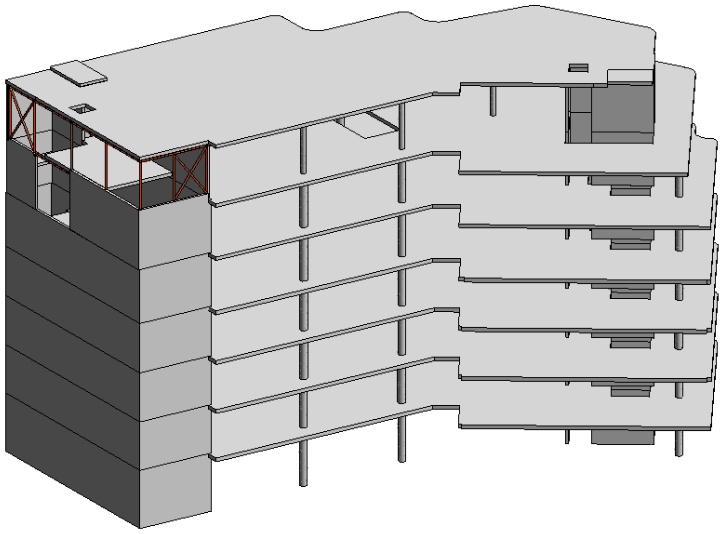


Figure 20: Model in Revit, stripped from non-bearing members, neighbouring buildings removed [20]

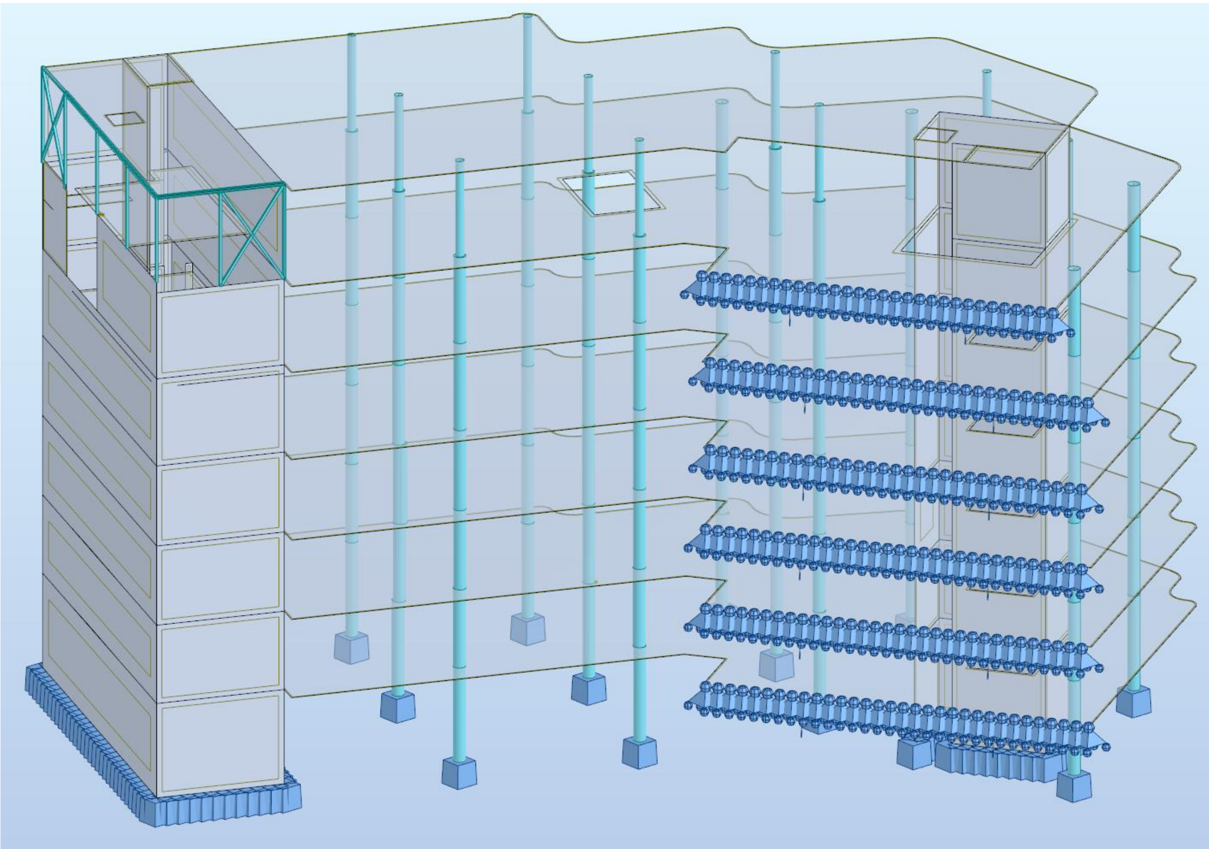


Figure 21: Robot 3D model of structure variation 1 [21]

3.2.2 Structure variation 2 – Hollow core concrete slabs and steel columns

With this variation, the layout will be identical to variation 1, while the added floors will be made by hollow core concrete slabs, with steel columns. This requires load carrying steel beams to be placed in between the columns which the slabs will rest on. The previous roof of 4th floor had to be extended, and the new floor of 5th level is therefore not a hollow core slab. For this model, the slab is entered into Autodesk Robot as a normal solid concrete slab with the same weight as a Hollow core 265mm, this corresponds to a reinforced solid concrete slab of 14,6 thickness. The beams carrying the hollow core slabs are calculated to be HE300B, S355J0, and the columns HE160B, S355J0 by a separate static analysis. See Figure 22 below for representation, as one can clearly see the difference in columns and the added beams. The walls are the same as in the as-built model – with thickness 180 mm. Figures 23 – 25 shows settings for slabs, beams and columns.

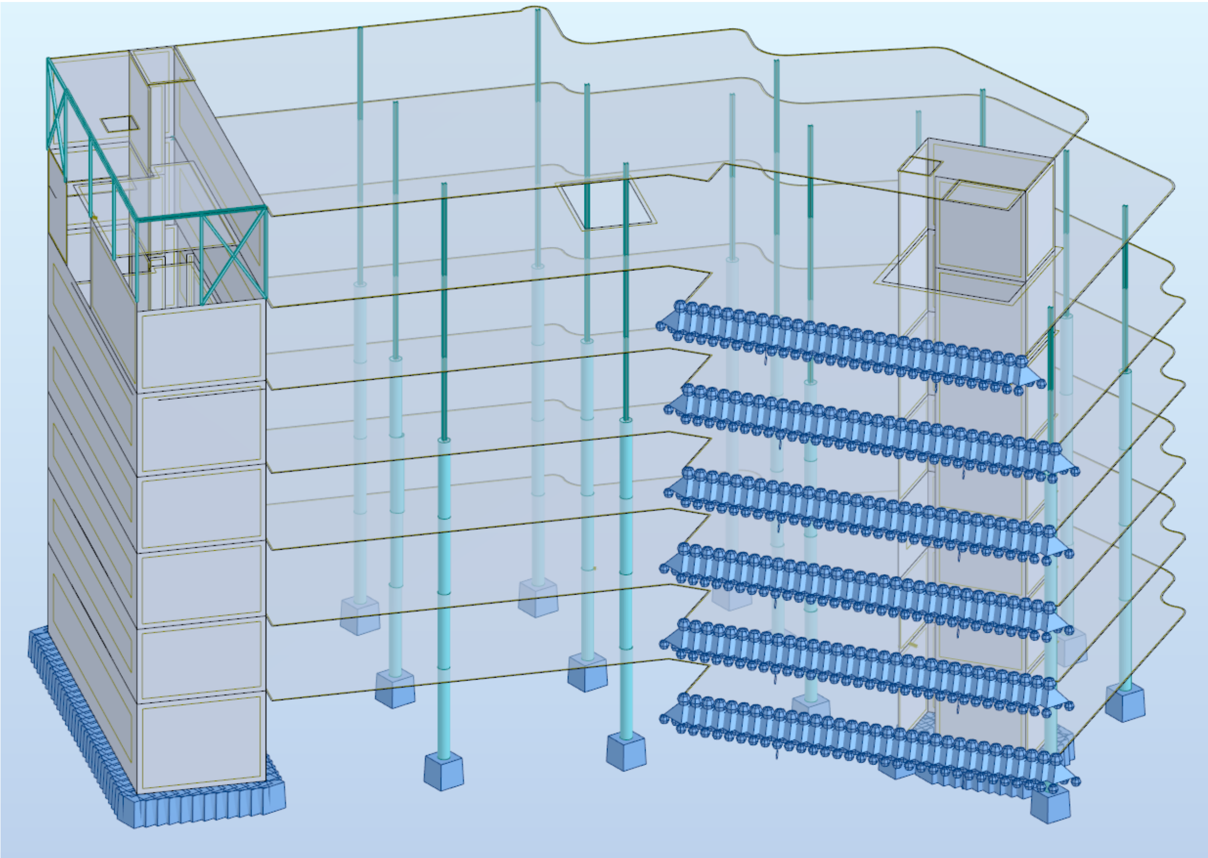


Figure 22: Robot 3D model of structure variation 2 [21]

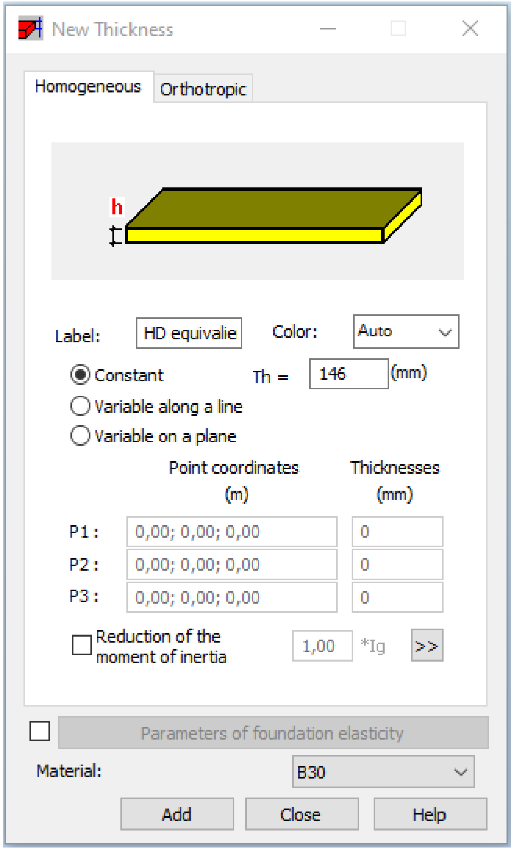


Figure 23: Equivalent slab thickness for Hollow core slab [21]

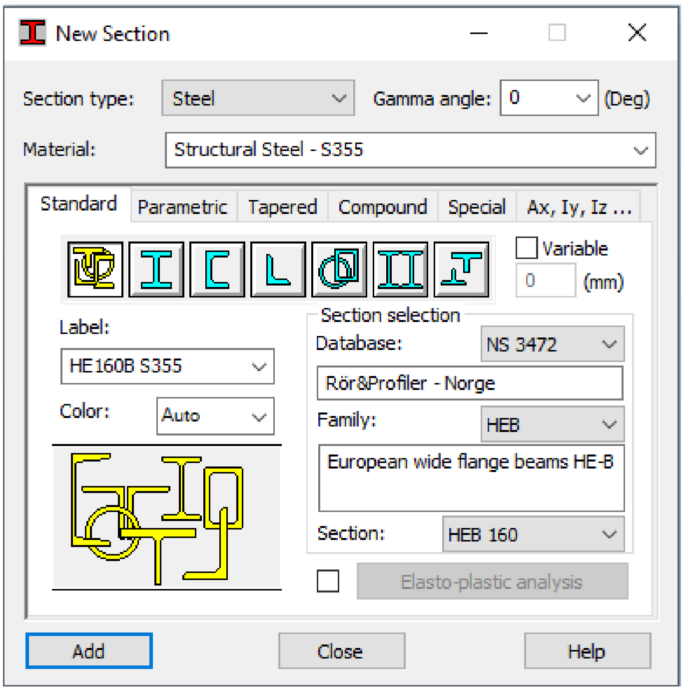


Figure 24: Columns for floor 6, 7 and roof [21]

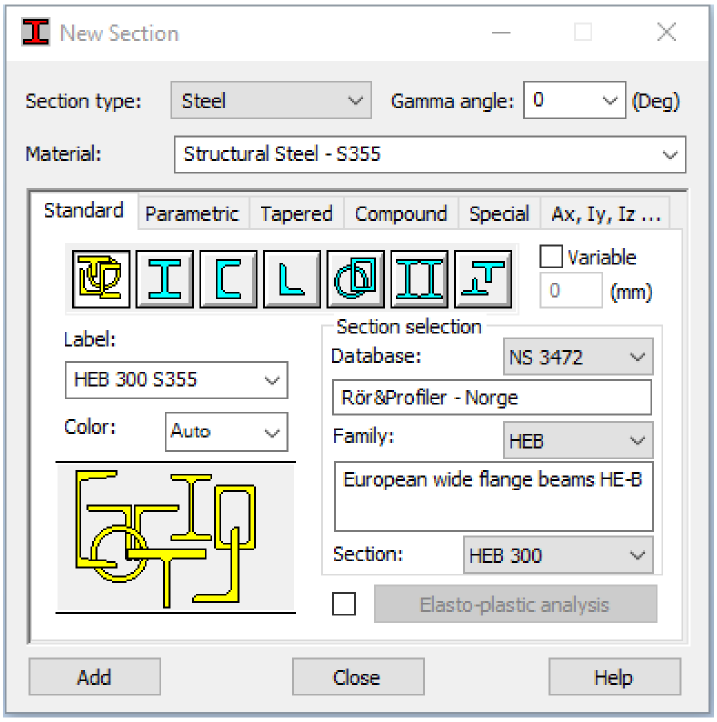


Figure 25: Beams for carrying the hollow core slabs [21]

3.2.3 Structure variation 3 – original building

This variation represents the structure as it were before the expansion, with 4 floors + roof. The slabs and columns settings are identical, to the as-built structure. See Figure 26.

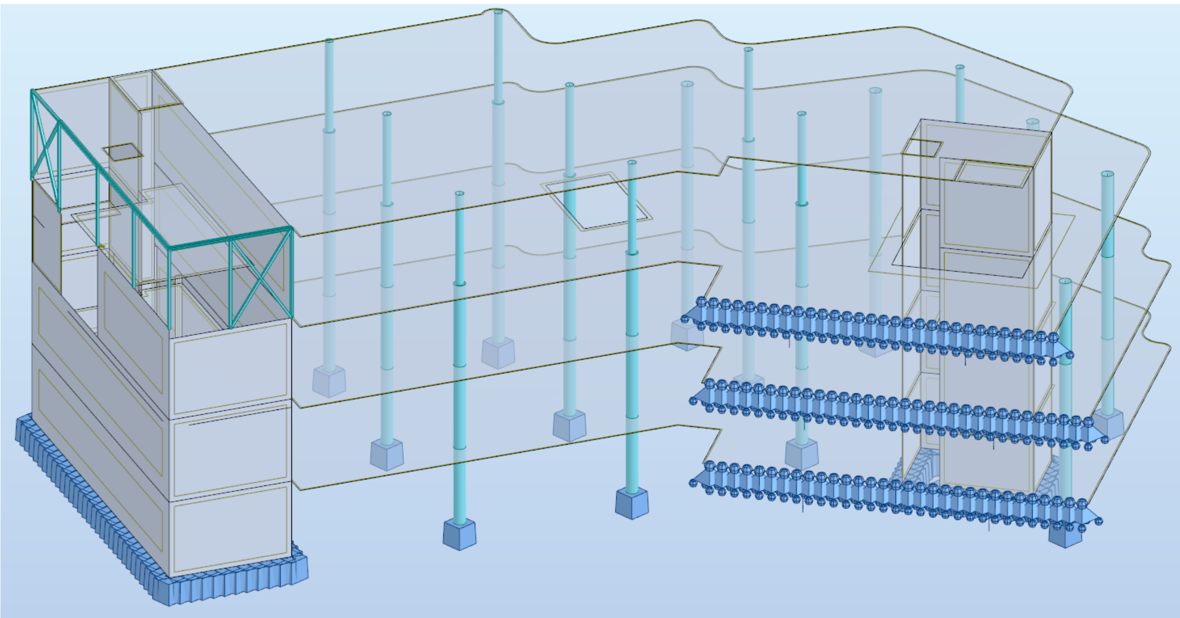


Figure 26: Robot 3D model of the original building [21]

3.2.4 Pile foundations

The entire structure is founded on reinforced concrete piles with height 3,1 to 12 meters and is connected to the bedrock, as the reinforcement is cast into the rock. Diameter is 900 mm, in concrete C25. Reinforcement is usually 13Ø20 in outer layer with clamp Ø8mm C/C 300mm, and 8Ø25 in inner layer of the pile, see Figure 27. The piles is placed in square grid with some variation in distances between grid lines. The distance is usually 5m to 7,5m between the grid lines. See Figure 28 for a snip of the plan.

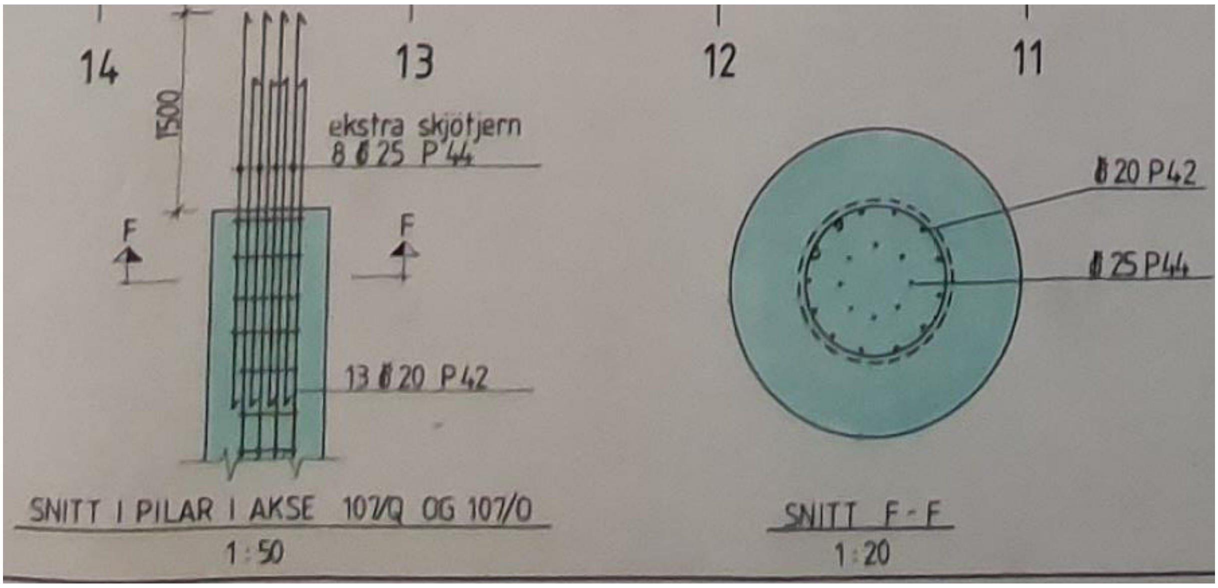


Figure 27: Reinforcement of piles

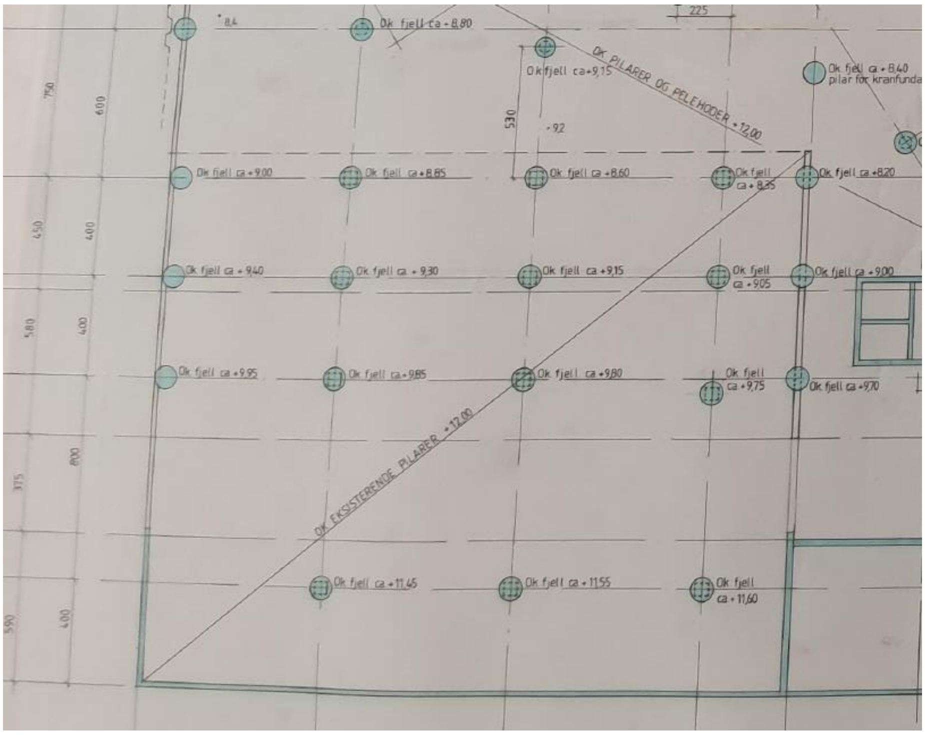


Figure 28: Grid line with placement of piles

3.3 Parameters for analysis of the structures

3.3.1 Constraints

In the model, the two stories of basement is removed to avoid unnessecarily many faults and details that is not important. The basement itself is considered so stiff that walls and columns at base floor level are fixed supports. A wall that is removed from the model is represented by pinned support in Z-direction at edges towards on neighbouring building. See Figures 29 and 30 in chapter 3.4 for visualization.

3.3.2 Loads

Loads are based on Eurocode 1 – NS-EN 1991-1-1 [22] and applied within the software on the representative members of the structure. Dead-, live- and snow loads are also added as they will be present during an earthquake.

Dead load

Dead loads / self load / permanent loads is the weight of the slabs, walls and columns. The floor dividers have set thicknesses (see chapter 3.1), and extra weight for floor covers is added according to the standard, assuned to be 1,0 kN/m^2 [23]. Added weight for walls is negligible, and is therefore not added. Reinfoced concrete weighs 25 kN/m^3, the dead loads for concrete slabs and walls are calulcated automatically in Autodesk Robot. See Figures 31, 32 and 33 in chapter 3.4 for visualization. While the loads are applied automatically, the general equations for dead loads of floors and walls are given in equations 3.3.1 – 3.3.3.

$$g_{k,230\ mm\ floor} = 5{,}75\frac{kN}{m^2} \qquad (eq. 3.3.1)$$

$$g_{k,180\ mm\ wall} = 4{,}5\frac{kN}{m^2} \qquad (eq. 3.3.2)$$

$$g_{k,added\ weight\ floors} = 1{,}0\frac{kN}{m^2} \qquad (eq. 3.3.3)$$

Live load

Live load is a varying load that typically represents people and furniture, displaced over the floors. Standard value for offices from NS-EN1991-1-1 Category B: 2 – 3 kN/m^2 [23]. Live load used for calculations during construction in 1983 were 2,5 kN/m^2. In this calculation, live load of 3 kN/m^2 is used, equation 3.3.4. Live load is not added to the roof. See Figures 34 in chapter 3.4 for visualization.

$$q_{k,live} = 3{,}0\frac{kN}{m^2} \qquad (eq. 3.3.4)$$

Snow load

For snow load, several parameters for conditions regarding roof angle, surrounding buildings with special circumstances is used to calculate the dimensioning snow load. This structure has a flat roof, and have no close tall buildings to influence the acting snow loads. For roof angles between 0 – 30 degrees have shape factor μ_1 = 0,8 to the nominal snow load [23].

$$q_{k,snow} = S_k \cdot 0,8 = (S_{k,0} + n\Delta S_k) \cdot 0,8 = 3,2 \frac{kN}{m^2} \qquad (eq. 3.3.5)$$

The factor n = $(H/H_g)/100$ is adjusting for altitude, and applies only if the altitude of the building H > H_g which is the nominal limit set for the specific county [23]. For Tønsberg, the limit is 150 meters above sea level. The structures altitude is approximately 18 meters above the sea level at ground level. Factor n is therefore set to 0. Final equation with calculation given in equation 3.3.5 above. See Figure 35 in chapter 3.4 for visualization.

Seismic load

The seismic load / earthquake load is the maximum acceleration expected. This is directly input acceleration, and can be found in Eurocode 8, maps for ground accelerations. Factors for the soil and seismic class is also relevant for the peak acceleration. The direct input acceleration is given in equation 3.3.6:

$$a_{g40Hz} = 0,5 \frac{m}{s^2} \qquad (eq. 3.3.6)$$

Total list of load case types

A totalt of 6 loads types for actual loads (Case 1, 2, 3, 4, 6 and 7), Modal analysis (Case 5) and Load combination (Case 8) is in the list of loads. See Figure 36 in chapter 4.3 for visualization.

3.3.3 Load combinations

The ultimate state limit load combinations during seismic analysis have noticable different values than in static analysis. In seismic analysis dead load has load factor 1,0 while live load has 0,3 for in offices. Snow load has factor 0,2 and wind load has factor 0 and is not considered at all. Earthquake has factor 1,0 [24]. See table 9 below and Figure 37 in chapter 3.4 for list of load combinations within Autodesk Robot. Equation 3.3.7 shows the general formula for load combination [24].

$$\sum_{j\geq i} G_{k,i} + \sum_{j\geq i} \psi_{2,i} Q_{k,i} \qquad (eq. 3.3.7)$$

where:

- $G_{k,i}$ is self / permanent load
- $Q_{k,i}$ is variable loads (live, snow, wind)
- $\psi_{2,i}$ is load factors

Table 9: Table of loads and factors in seismic load combination [24]

Type load	Dead load, ($G_{k,i}$)	Live load	Snow load	Wind load	Seismic load
Load, $Q_{k,i}$	1,0	3,0	3,2	-	Determined in analysis
Factor, $\psi_{2,i}$	1,0	0,3	0,2	0	1,0

3.3.4 Load to Mass Conversion

In the event of modal analysis and finding the responsive natural frequencies of the structures only includes dead loads. In the event of an earthquake, some of the live loads and snow loads will also be present, and must be converted to a corresponding dead load to be included. This is done under Analysis types, and Load to Mass Conversion tab. The same load factors is applied for live load and snow load (see table 9 for load combinations above), which is converted case number 3 (snow) and 4 (live) in Figure 38 in chapter 3.4.

3.3.5 Modal analysis

Modal analysis is performed to find the structures eigenvalues, which determines the response of the building when exposed to seismic loads. In the modal analysis, the number of modes nessecary is typically linked to how compicated the structure is, a square structure with few or symmetric columns / walls and elevator shafts will often require no more than 10 modes in the analysis. The reqired amount of modes is found by applying enough modes that will lead to relative mass movement in X- or Y direction above 90% relative mass according to EC 8. To be sure to reach above 90% relative mass, a conservative 12 modes is used. For criteria $k > 3\sqrt{n}$, 8 modes would be enough [16]. Mass direction in Z axis is unchecked, as we are only interrested in loads in X-Y plane. 5% damping ratio is chosen for this structure, values for soil – see Figure 39 in chapter 3.4 for visualization.

3.3.6 Seismic analysis

Analysing for seismic loads is added to the list by choosing Seismic analysis type in new load case definition, the newest NS-EN standard is chosen as basis. When choosing only by X-Y axis, two seismic analysis load cases appear, representing each direction. Seismic acceleration a_{g40Hz} is set to 0,5 m/s^2 as indicated by Eurocode 8 map over seismic zones of Norway. Ground type is set to S_2 as it is primarily clays and quick clay in the soil. This soil type is not standard within Autodesk Robot, and the parameters S, T_b, T_c and T_d must be entered separately by choosing Elastic spectrum. Table 2 from chapter 2.4.3 shows table for importance class / seismic class, which is set to 2. Tables 11 and 12, Figures 40 – 41 in chapter 3.4 for visualizations and tables. Relevant input factors based on factors for special soils from [25]:

Table 10: Input factors acceleration, seismic class and soil factors

Peak grounda acceleration		a_g40Hz =	0,5	
Seismic class		γ_1 =	1,0	
Soil factors	S	T_b(s)	T_c(s)	T_d(s)
Ground type S_2	1,7	0,1	0,4	1,4

3.4 Visualization of input into autodesk robot

3.4.1 Constraints, supports of the structure

The structure is considered fixed at the base of 1st floor, and thus have fixed support at all columns and walls at the base, see Figure 29. Figure 30 shows the pinned support in UZ direction, as there is a light wall there in reality that carries the outer line of the slabs.

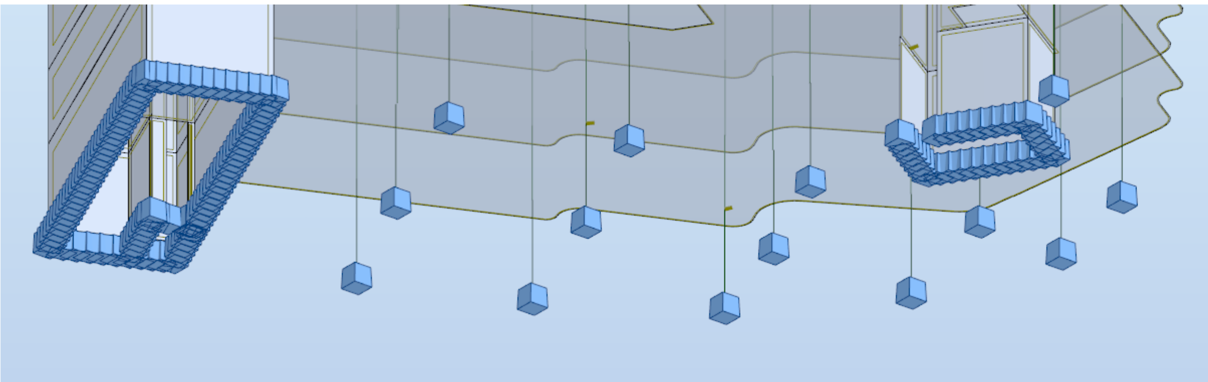


Figure 29: Visualization of added constraints, fixed supports[21]

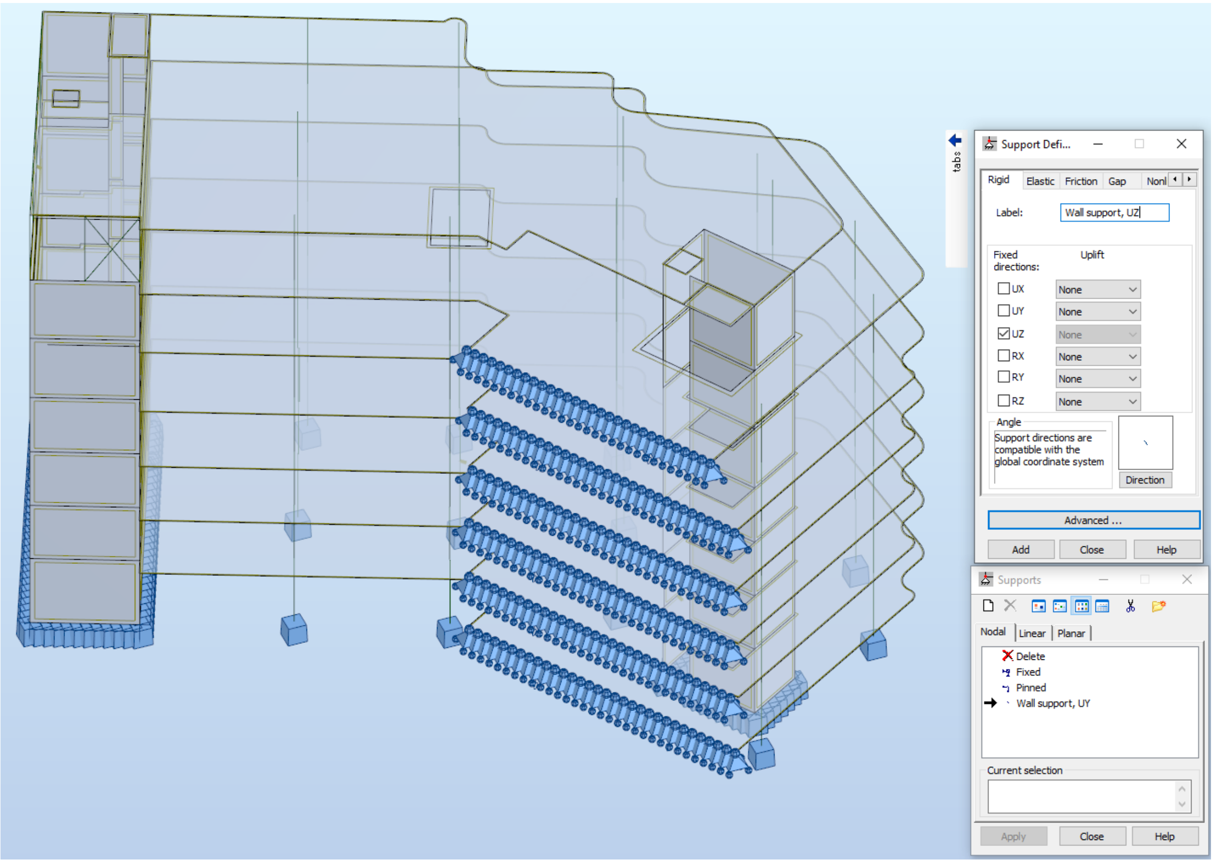


Figure 30: Visualization of pinned support at edge, to simulate wall[21]

3.4.2 Loads

Inputs of loads on the structure, figures 31 and 32 shows the settings for applying loads, and Figures 33 – 35 shows where the added self load, live load and snow load is applied on the specific floors.

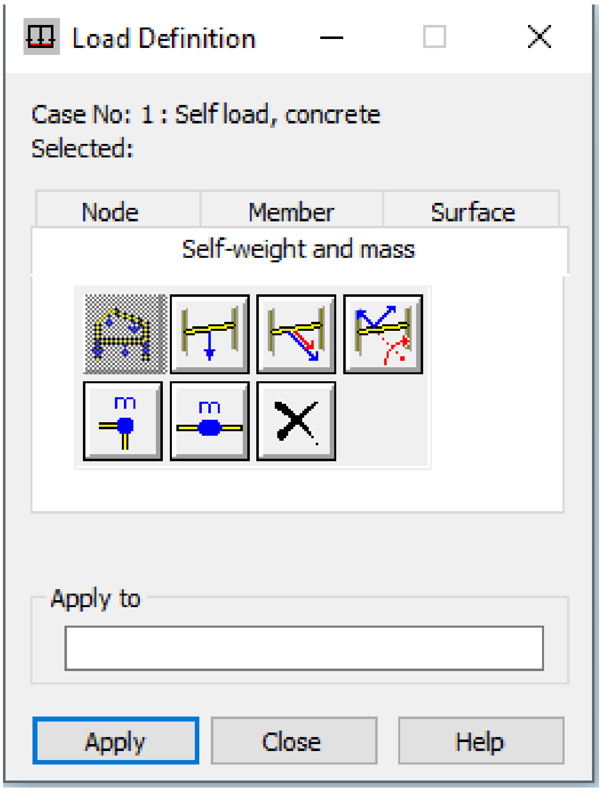


Figure 31: Self weight load has been added for the entire structure [21]

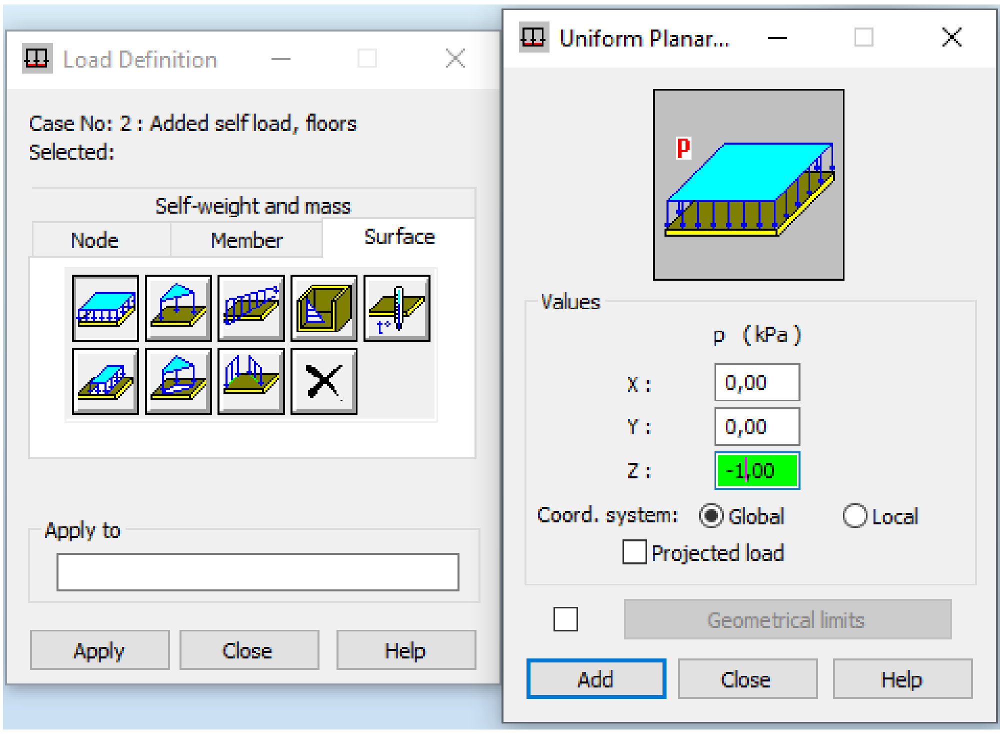


Figure 32: Defining loads applied to chosen floors, used for added self weight, live- and snow loads [21]

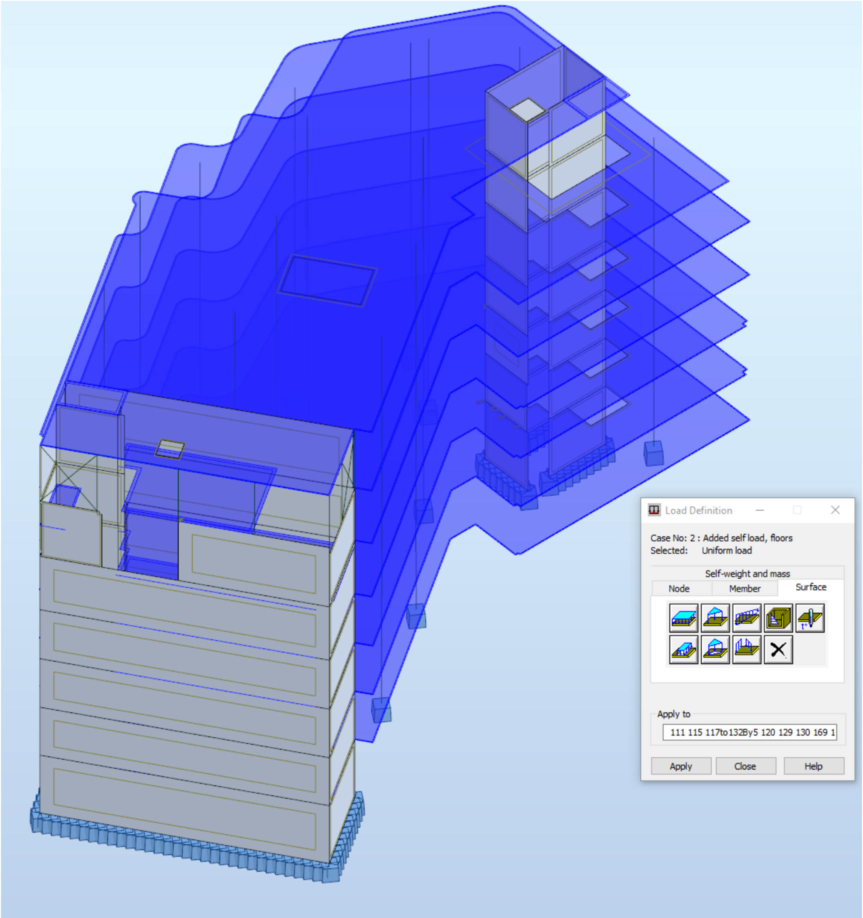


Figure 33: Added self load applied on all floor slabs [21]

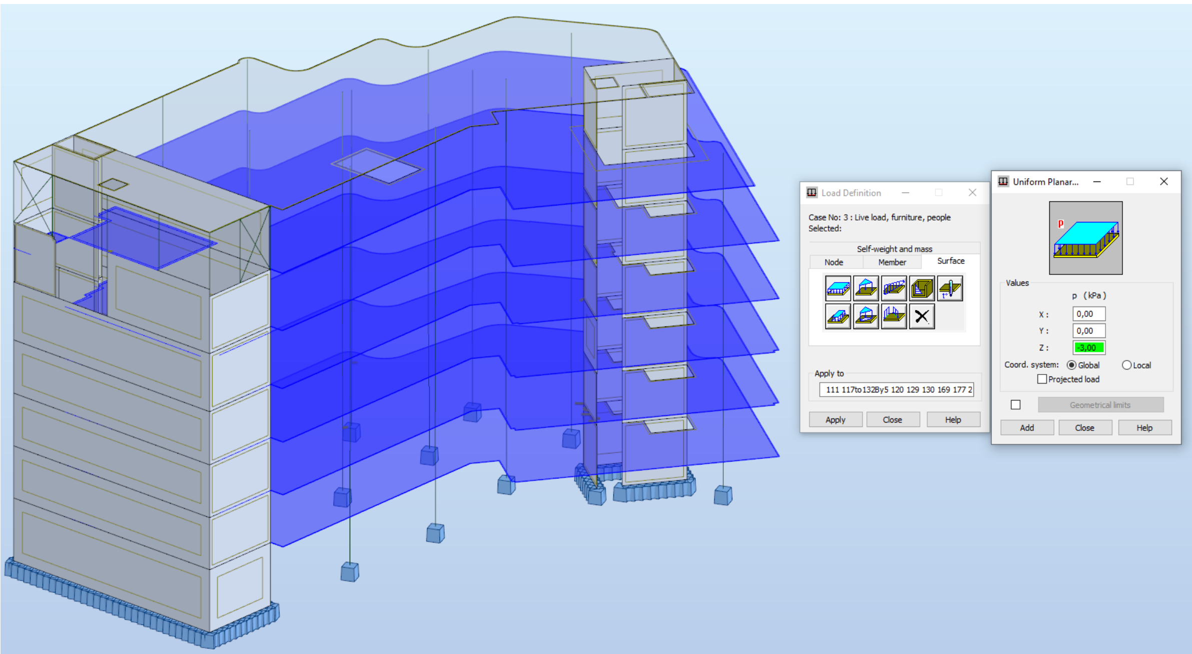


Figure 34: Live load applied to all floors except roof [21]

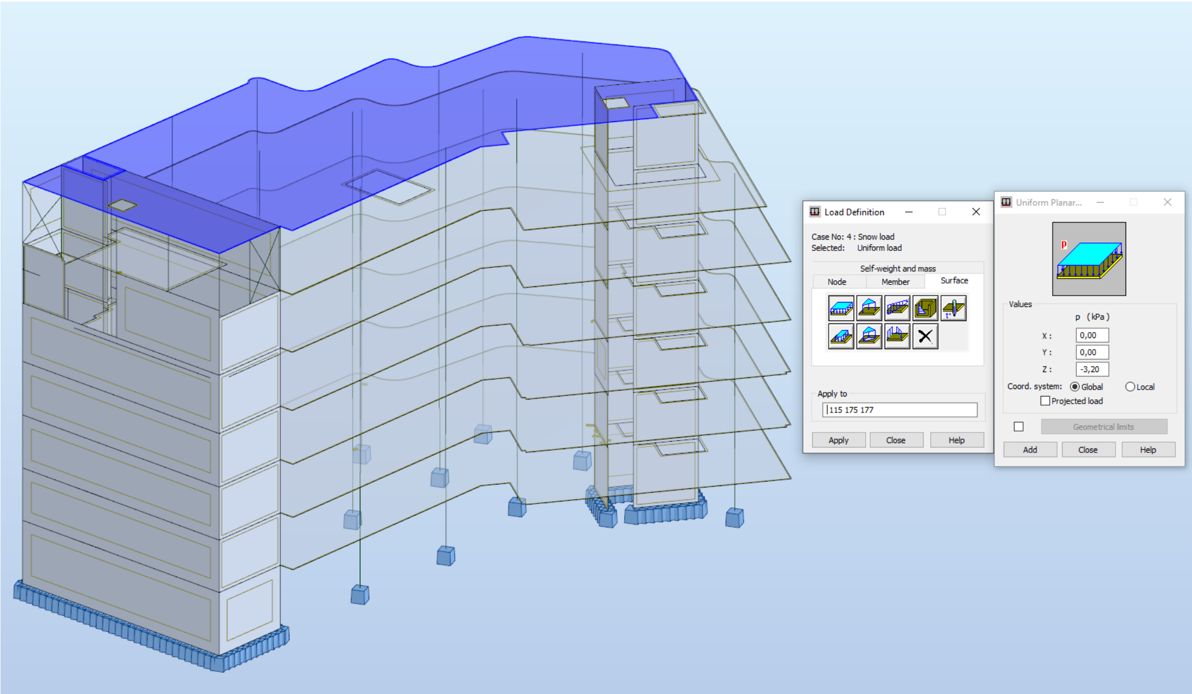


Figure 35: Snow load only applied to the roof [21]

3.4.3 Final list of load types

The final list of all loads in robot, included modal analysis and the load combination is shown in Figure 36 below.

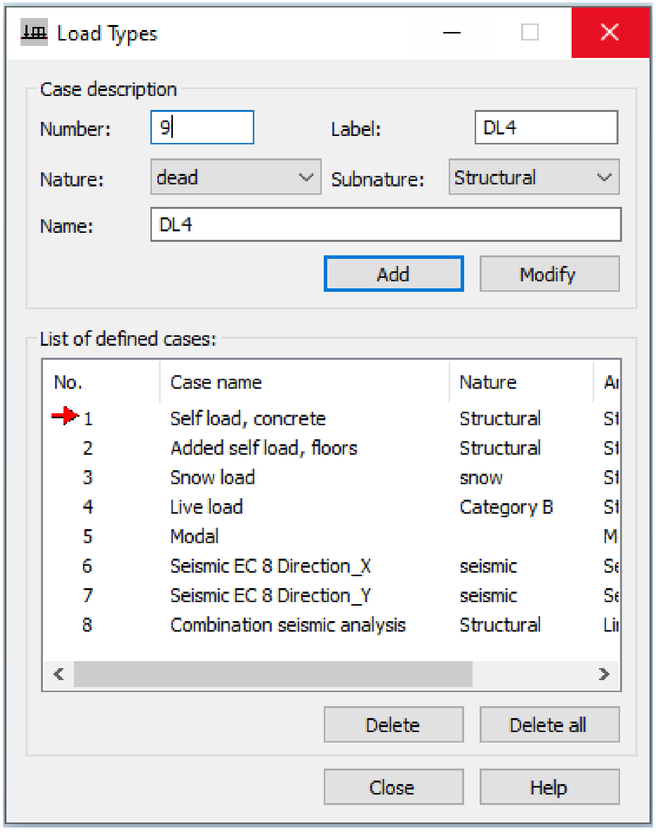


Figure 36: Final list of load types, can be seen under “Load Types” [21]

3.4.4 Load combinations

Load combinations relevant for the analysis, showing load types and load factors, Figure 37.

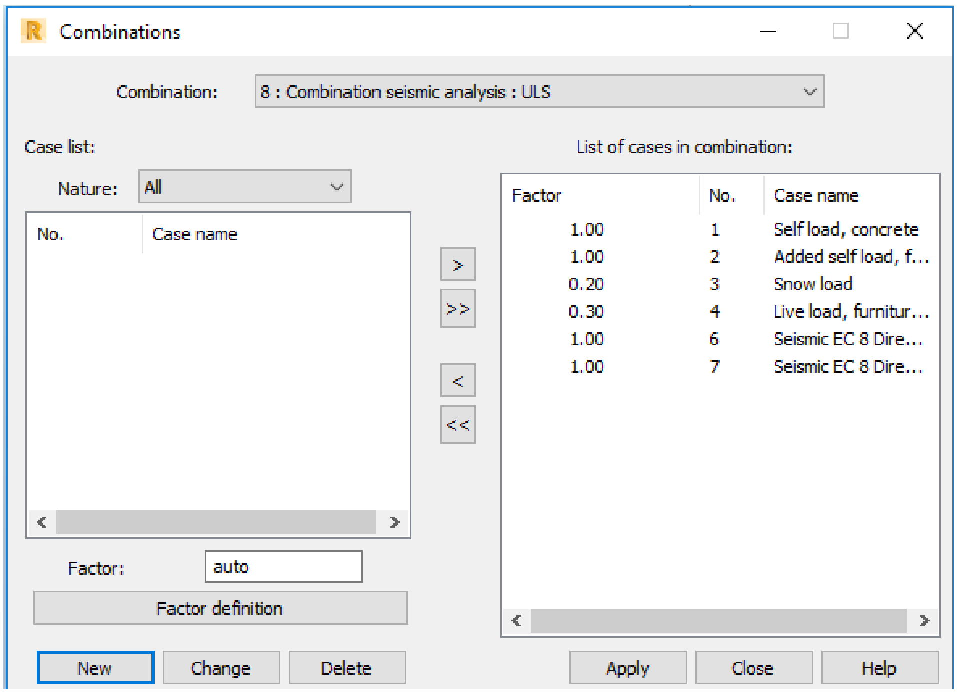


Figure 37: Input load combination for seismic analysis, seen under Loads – Load combinations [21]

3.4.5 Load to mass conversion

Since there will still be live load and snow load present during an earthquake, these loads must be included and converted to self load to be included in the analysis for and earthquake. Figure 38 for settings, the same factors as the load combination are used.

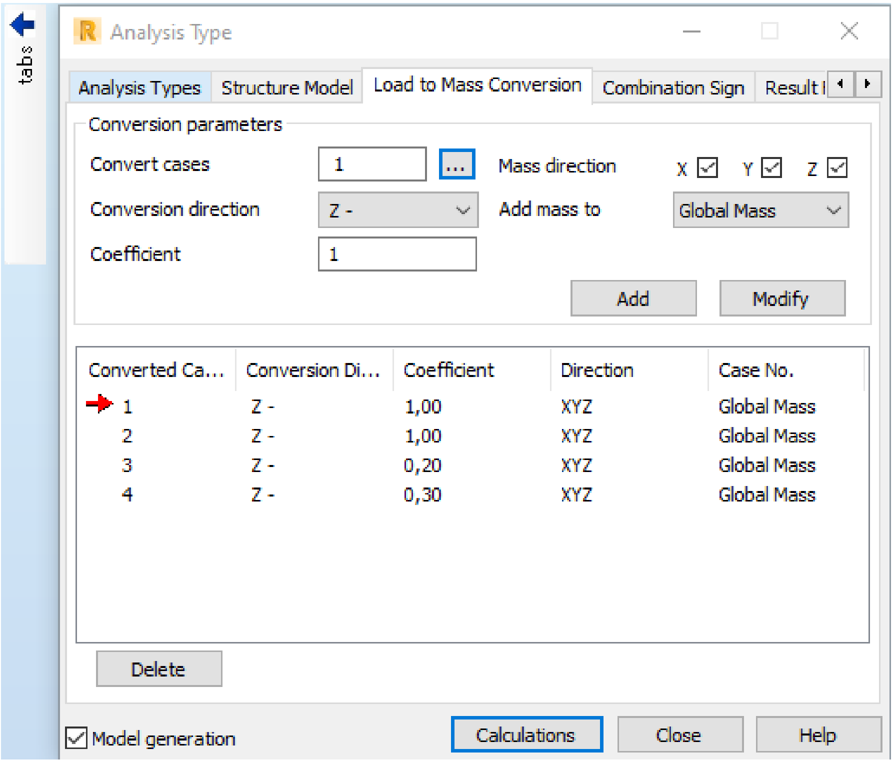


Figure 38: Load to Mass Conversion tab and options, can be found under Loads – Load to mass conversion [21]

3.4.6 Modal analysis

Settings for modal analysis in Figure 39, 12 modes is used, and only X and Y directions are chosen. The rest of the settings are basic settings only changed for special circumstances.

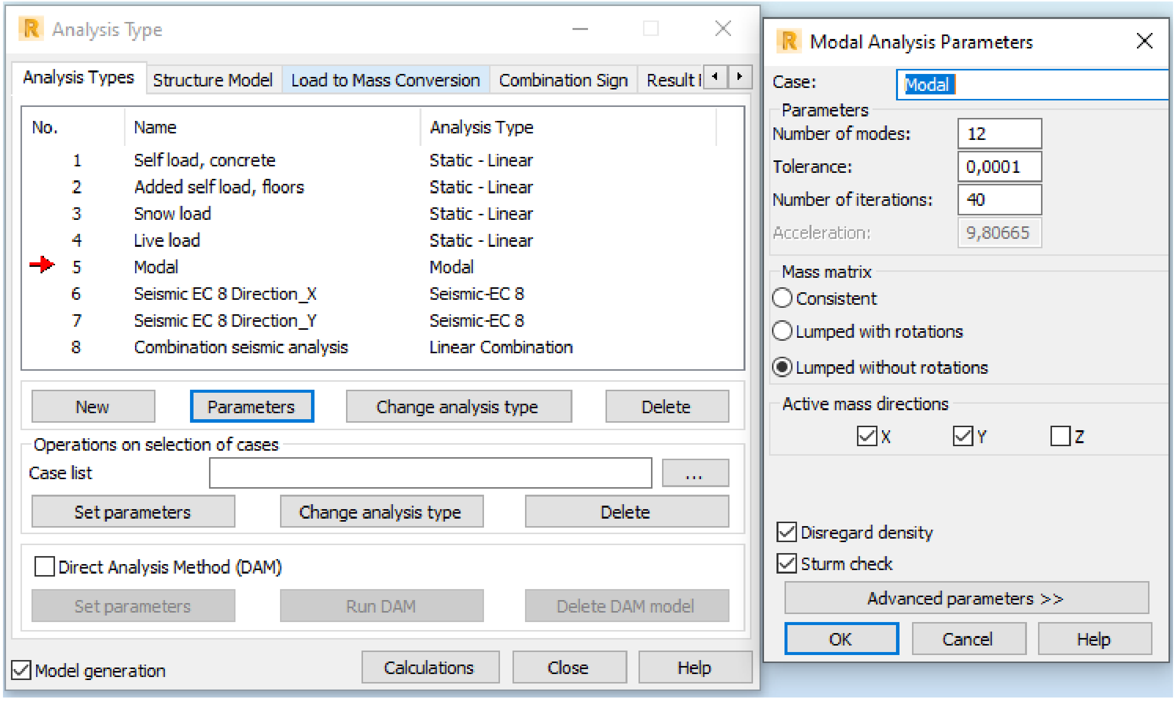


Figure 39: Adding modal analysis – Under Analysis Type – New – Choose Modal analysis [21]

3.4.7 Seismic analysis

Inputs for the seismic analysis in Tables 11 and 12 for soil factors, Figures 40 and 41 shows the input for the seismic analysis, where separate cases for direction UX and UY must be added. Analysis based on NS-EN 1998 is chosen which means based on Eurocode 8 procedure.

Table 11: Soil types / grunntyper, extended table for S1 and S2 [16]

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

Table 12: Soil parameters for soil type S2 [25]

Dybde til fjell	S	T_B (s)	T_C (s)	T_D (s)
D = 6 - 20 m	1,7	0,10	0,40	1,4
D = 20 - 35 m	1,6	0,15	0,50	1,5
D = 35 - 50 m	1,5	0,2	0,60	1,6

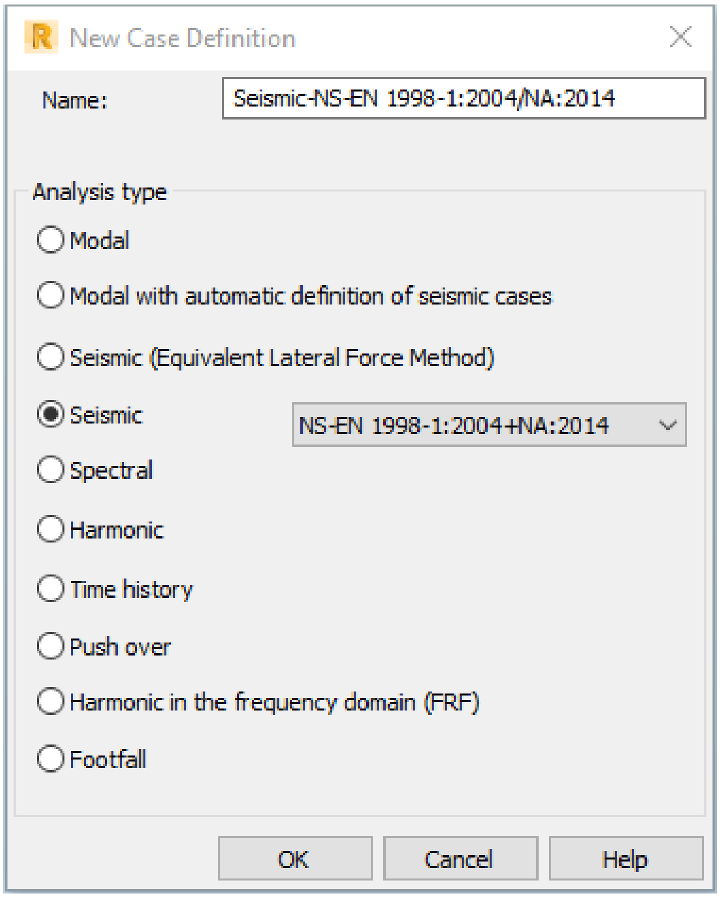


Figure 40: Seismic analysis window after clicking New on Analysis Types [21]

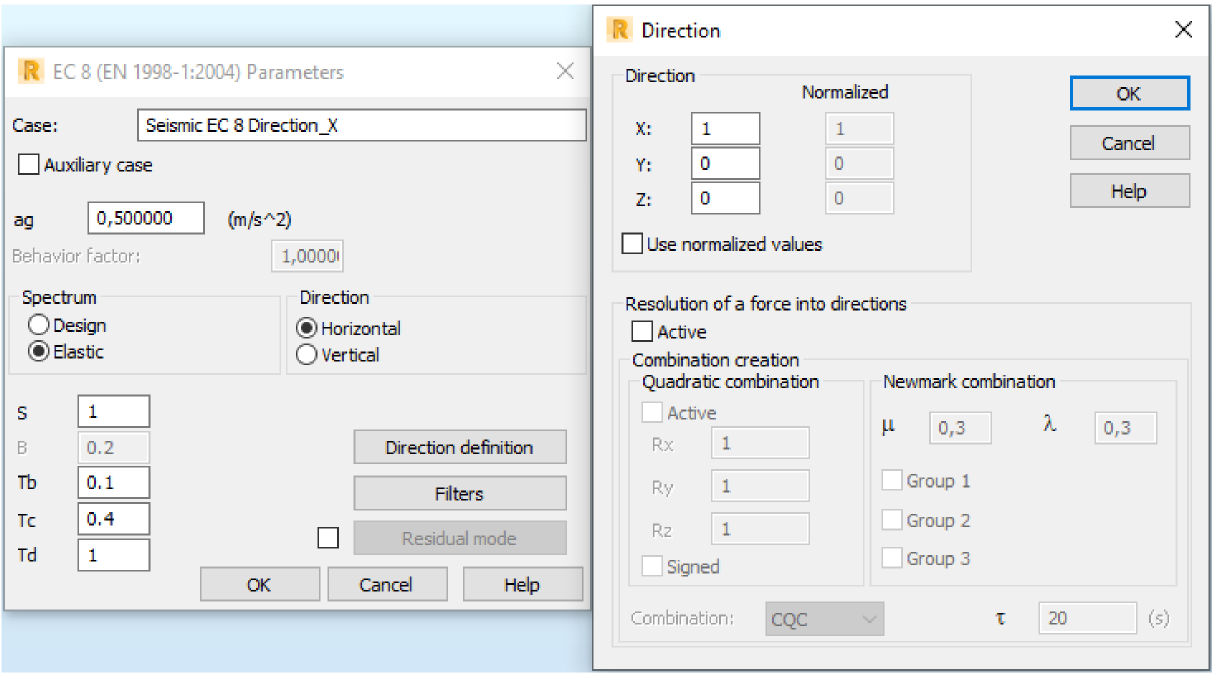


Figure 41: Seismic analysis, 2.nd step, X-direction - main settings [20]

3.4.8 Running the analysis

Click Calculations in the analysis type window. A screenshot of the calculation / analysis process is shown in Figure 42 below.

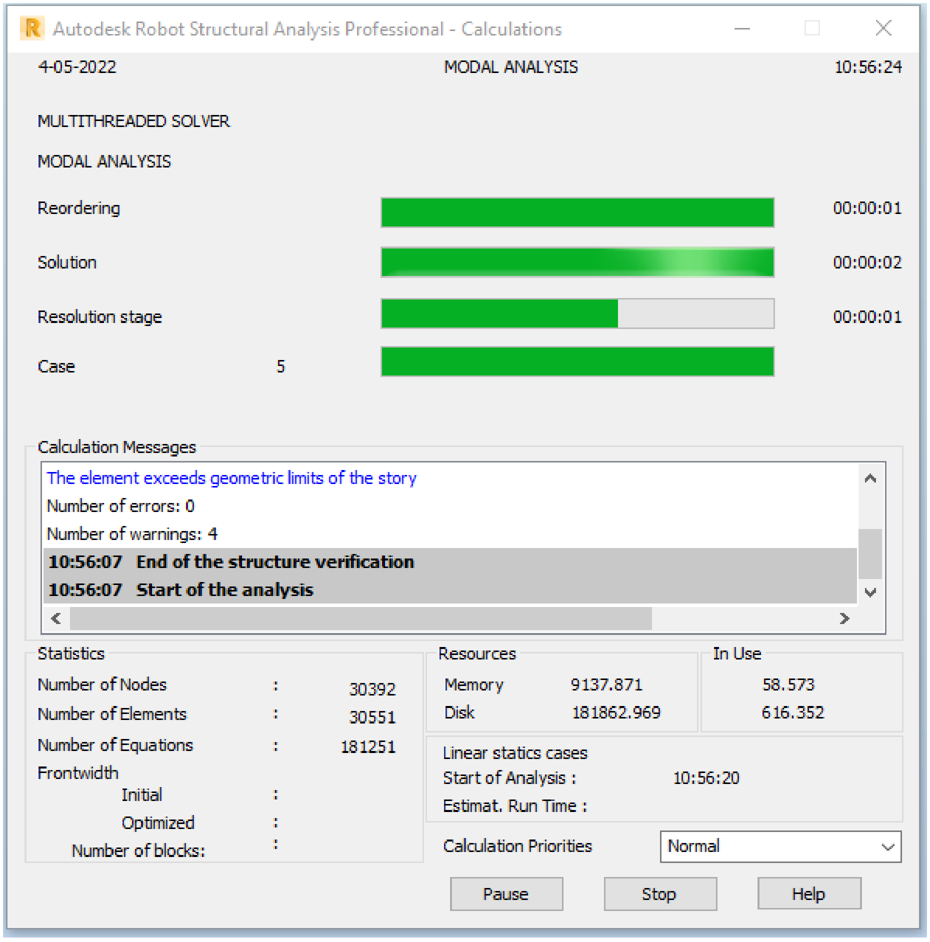


Figure 42: Screenshot of the process while analysing [21]

3.5 Foundation and piles

In the analysis of the structure, fixed support is applied in the transition between soil and structure – at ground level, and the basement levels and pile foundations is excluded from the model and analysis. This is acceptable as the basement levels are so stiff that fixed supports between structure and basement can be assumed, see chapter 2.5.2 – assumptions. An analysis of the foundation is not included in this report, and would require good knowledge of the soil where precise soil type, layers and depths must be known in addition to the foundation and structure geometry. This typically requires there to be drilled a core sample for analysis and measurements to map the entire area beneath the structure. The piles also have variable lengths, showing that the depth of the soil varies greatly.

4 Results

4.1 Modal analysis in autodesk robot

The main result is the resulting natural periods of the structure, which determines the response when subjected to acclerations, and is used further in the seismic analysis. The resulting natural periods is the main data presented in tables with extended information.

The result from the structure as built shows that Relative mass > 90% is met at mode 8 (Figure 43 below), which means only 8 modes would be required for this structure. The result tables for variant 2 and 3 meet 90% mass at mode 8 as well, and their figures is therefore not included.

Structure variation 1 – As built

The modal analysis gives T_1 = 0,58s and T_2 = 0,45s which is the primary frequencies (Figure 43), It is results for 12 modes, the majority is therefore not included in these tables. Weight of the structure is 4639,35 tonnes.

Table 13: Results from modal analysis of structure variant 1

Modal T_1	0,58	s
Modal T_2	0,45	s
Weight	4639,35	tonnes
90% relative mass met at mode	8	Direction UY

Case/Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)	Cur.mas.UX (%)	Cur.mas.UY (%)	Cur.mas.UZ (%)	Total mass UX (kg)	Total mass UY (kg)	Total mass UZ (kg)
5/ 1	1,74	0,58	22,05	32,84	0,0	22,05	32,84	0,0	4639347,40	4639347,40	0,0
5/ 2	2,24	0,45	56,11	65,51	0,0	34,06	32,67	0,0	4639347,40	4639347,40	0,0
5/ 3	5,67	0,18	59,04	73,31	0,0	2,93	7,79	0,0	4639347,40	4639347,40	0,0
5/ 4	6,69	0,15	78,75	77,20	0,0	19,71	3,90	0,0	4639347,40	4639347,40	0,0
5/ 5	7,25	0,14	84,73	87,91	0,0	5,98	10,71	0,0	4639347,40	4639347,40	0,0
5/ 6	8,13	0,12	84,76	87,92	0,0	0,03	0,01	0,0	4639347,40	4639347,40	0,0
5/ 7	9,32	0,11	85,79	88,02	0,0	1,03	0,10	0,0	4639347,40	4639347,40	0,0
5/ 8	10,29	0,10	85,79	91,09	0,0	0,00	3,07	0,0	4639347,40	4639347,40	0,0
5/ 9	12,27	0,08	89,34	91,44	0,0	3,55	0,35	0,0	4639347,40	4639347,40	0,0
5/ 10	12,55	0,08	89,40	91,44	0,0	0,06	0,00	0,0	4639347,40	4639347,40	0,0
5/ 11	13,32	0,08	89,53	91,85	0,0	0,13	0,41	0,0	4639347,40	4639347,40	0,0
5/ 12	13,56	0,07	91,43	95,21	0,0	1,90	3,36	0,0	4639347,40	4639347,40	0,0

Figure 43: Results of modal analysis [21]

Structure variation 2 – Expanded floors of hollow core slabs and steel columns

Modal analysis results from variation 2 in table 14.

Table 14: Results from modal analysis of structure variant 2

Modal T_1	0,53	s
Modal T_2	0,45	s
Weight	4169,6	tonnes
90% relative mass met at mode	8	Direction UY

Structure variation 3 – Original structure

Modal analysis results from variation 3 in table 15.

Table 15: Result from modal analysis of structure variant 3

Modal T_1	0,28	s
Modal T_2	0,22	s
Weight	2538,55	tonnes
90% relative mass met at mode	8	Direction UY

4.2 Seismic analysis in autodesk robot

The main result for this analysis is the resulting base shear and torsion forces at the floors and base, as a reaction for the seismic acceleration applied to the structure. In the results shown, the total shear force and torsion is displayed at each floor. The acting shear and torsion differs from the standard general equation at the specific floor, and is for the output results from robot is calculated by equation 4.2.1:

$$F_{shear/torsion\ floor\ n} = F_n - F_{n+1} \qquad (eq. 4.2.1)$$

The more general formula for shear load in each floor is equation 4.2.2 [16]:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \qquad (eq. 4.2.2)$$

Considering approximated mode shapes by horizontal displacement, equation 4.2.3 [16]:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \qquad (eq. 4.2.3)$$

where:

- F_i is the horizontal force acting on storey i
- F_b is the seismic base shear in accordance with equation 2.4.2
- s_i, s_j are the displacement of masses m_i, m_j in the foundation mode shape
- m_i, m_j are the storey masses
- z_i, z_j are the heights of masses m_i, m_j above level of application for seismic action (foundation or top of rigid basement)

Structure variation 1 – As built

Resulting disc effect – shear / torsion forces, see Figures 44 – 47 below. Tables 16 – 18 shows the base loads at each floor for all structure variations. The results from variant 2 and 3 will be similar, and figures for their results is therefore not included.

Table 16: Base shear and torsion forces, structure variation 1

Floor n	Shear X [kN]	Shear Y [kN]	Torsion X [kNm]	Torsion Y [kNm]
Roof	463,72	699,23	1622,29	2316,11
7.th	458,38	428,91	1512,1	475,79
6.th	878,09	1066,76	5427,88	5389,07
5.th	840,84	756,37	6899,06	5679,58
4.th	1200,12	1324,62	11714,54	11409,81
3.rd	1114,92	980,42	13730,64	12682,86
2.nd	1248,99	1283,84	19861,53	18847,99
Base	1248,99	1283,84	19861,53	18847,99

Figures 44 – 47 shows the results of loads at each floor in autodesk robot based on settings FX, FY, MX and MY, and load results for direction UX or UY must be chosen. To find shear load in UX direction, apply FX in UX direction which is done in figure 44. The results shows the total shear load at each floor, and the effective load is calculated by equation 4.2.1. To find the torsion loads, choose direction UY and MX to find torsion in Y direction, and direction UX with MY for torsion in Y direction, see Figure 48 for how the horizontal axis are for the structure. Tables 16 – 18 shows the final calculated shear and torsion loads in all directions for all three structure variations. See Figure 19 which shows how X and Y axis are placed for the structure at the full axis system for the structure.

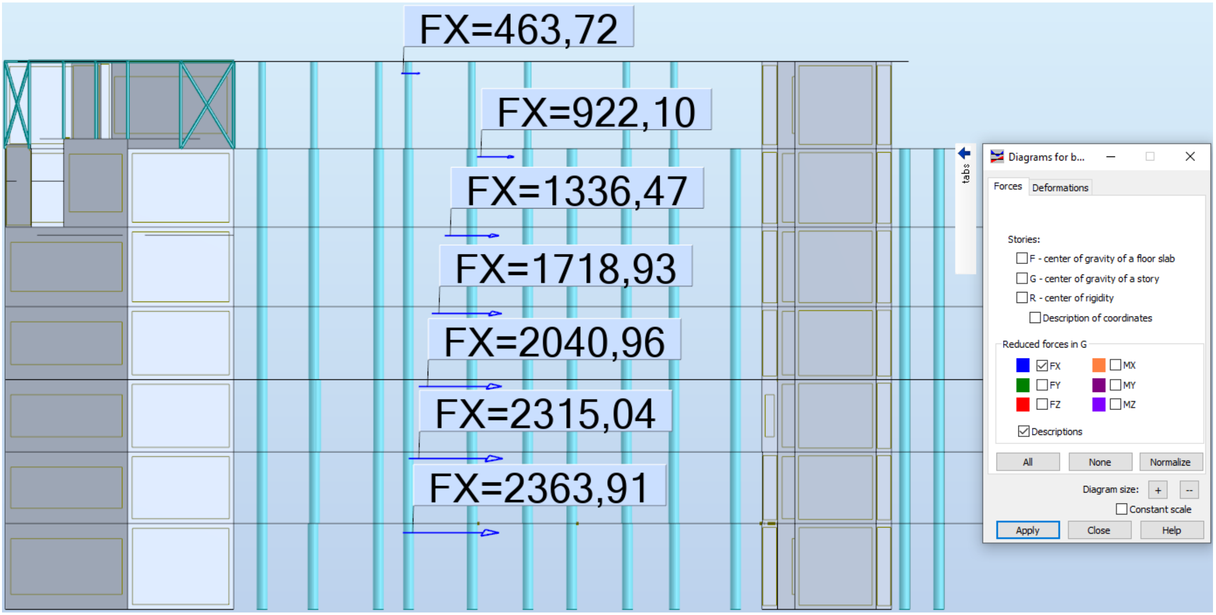


Figure 44: Measured shear in X direction for all floors [21]

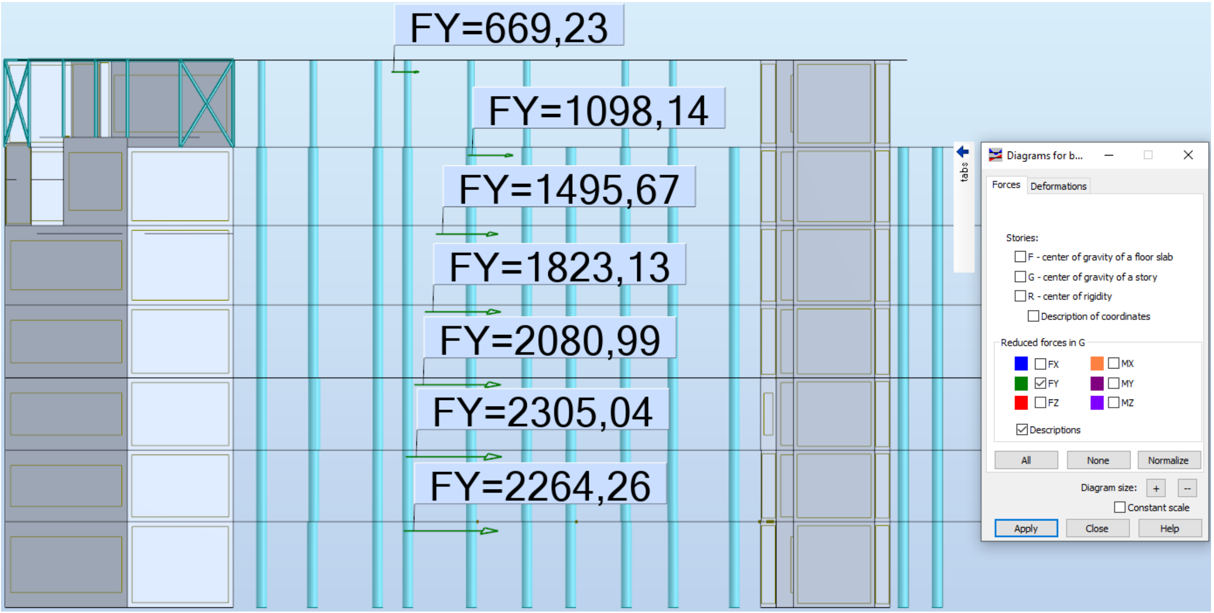


Figure 45: Measured shear in Y direction for all floors [21]

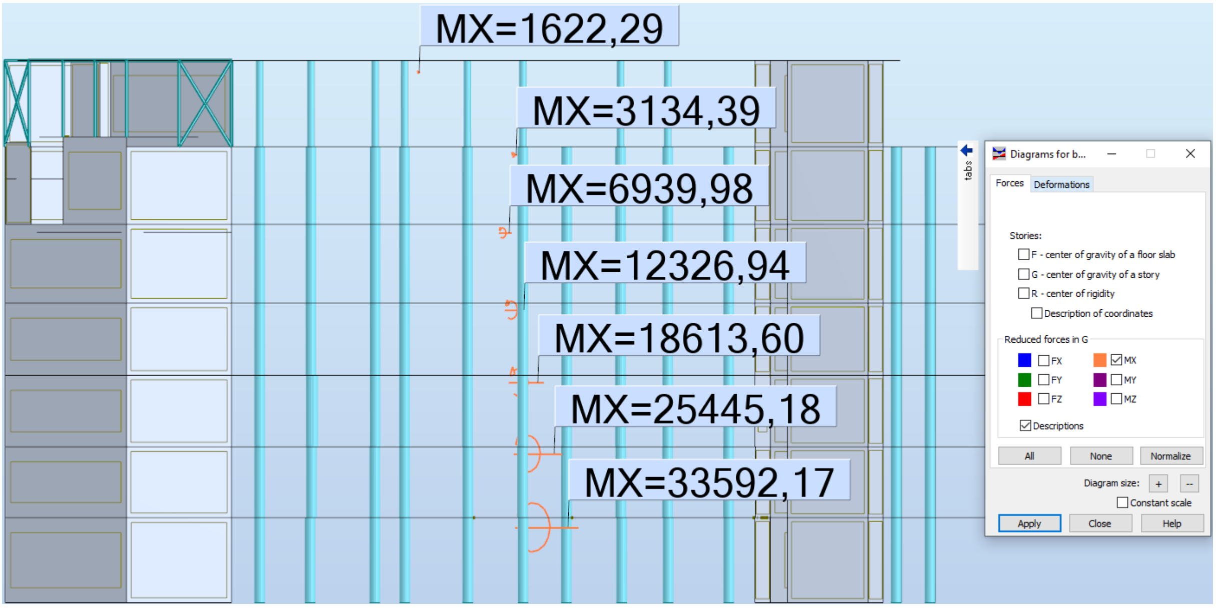


Figure 46: Measured moment MX for all floors [21]

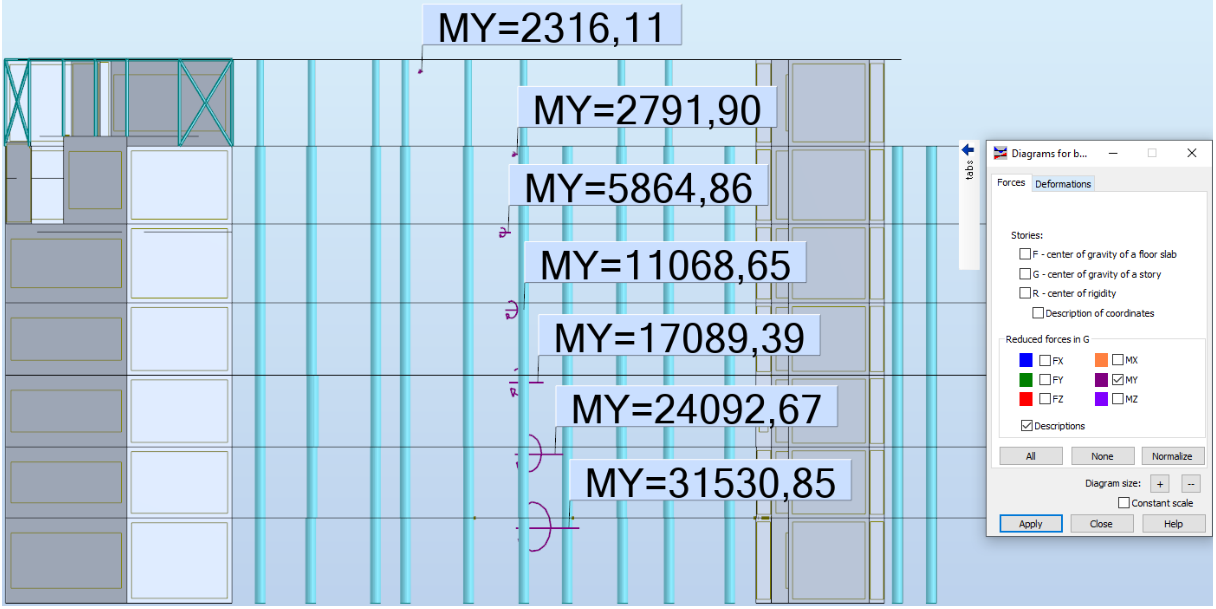


Figure 47: Measured moment MY for all floors [21]

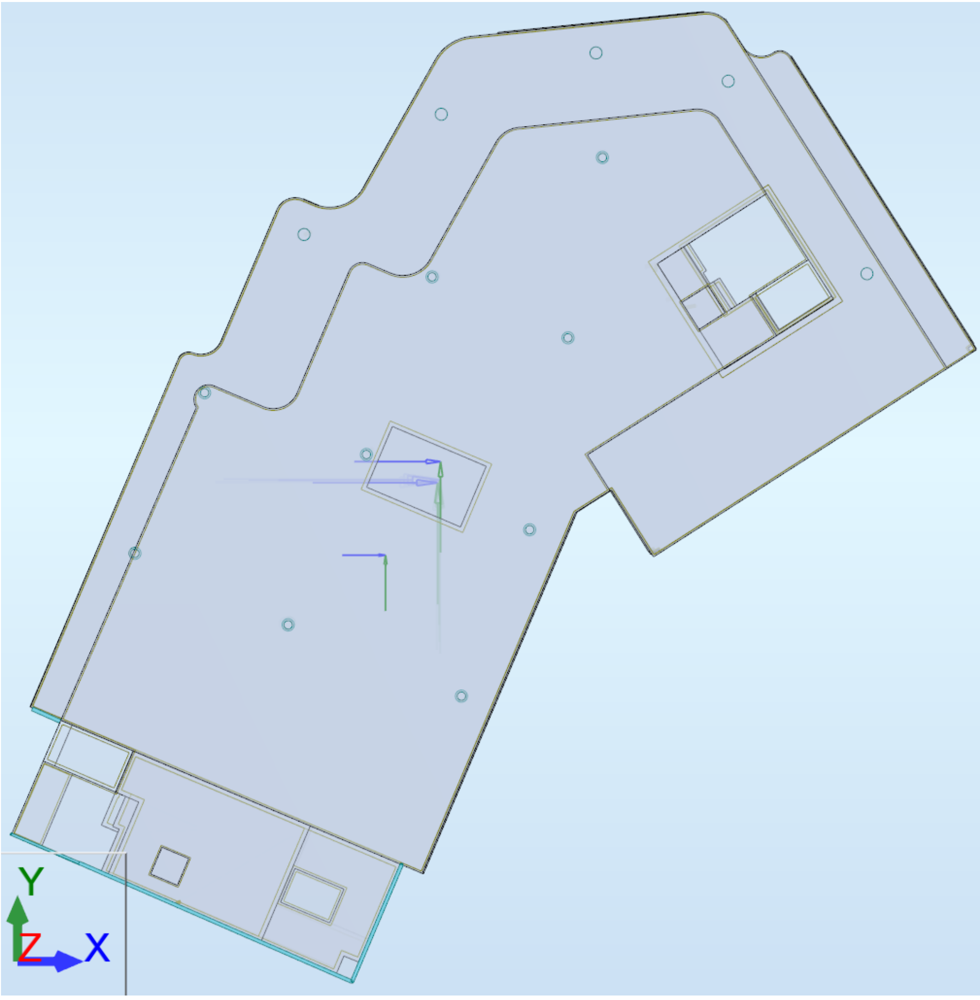


Figure 48: Axis system for the structure, blue and green arrows (weak in the middle, stronger coloured at bottom left corner) represents shear loads in each direction [21]

Structure variation 2 – Expanded floors of hollow core slabs and steel columns

Final calculated efficient shear and torsion loads for all floors in structure variation 2 in Table 17.

Table 17: Base shear and torsion forces for structure variation 2

Floor n	Shear X [kN]	Shear Y [kN]	Torsion X [kNm]	Torsion Y [kNm]
Roof	434,01	606,71	1444,01	1986,84
7.th	342,04	348,7	970,64	30,82
6.th	734,36	616,49	4973,95	5082,33
5.th	785,58	732,65	5427,77	4060,4
4.th	1078,97	1202,72	10568,49	10408,66
3.rd	1082,95	977,27	11724,43	10425,78
2.nd	1147,43	1182,8	18194,08	17297,77
Base	1147,43	1182,8	18194,08	17297,77

Structure variation 3 – Original structure

Final calculated efficient shear and torsion loads for all floors in structure variation 2 in Table 18.

Table 18: Base shear and torsion forces, structure variation 3

Floor n	Shear X [kN]	Shear Y [kN]	Torsion X [kNm]	Torsion Y [kNm]
Roof	478,91	697,15	1342,63	1378,31
4.th	459,63	420,59	1557,69	1164,17
3.rd	837,2	1006,59	5250,98	4755,55
2.nd	625,57	498,49	6798,75	5651,83
Base	625,57	498,49	6798,75	5651,83

4.3 Hand calculations

Hand calculations is based on Eurocode procedures and equations for the various parameters and parts. These results is only ment to compare the results from the numerical model. This process includes natural period, dimensioning spectre and base shear load.

4.3.1 Natural period

The natural period of the structure is used for caculating a structure's dynamic properties, and can be calculated in sevarl different ways based on the type and geometry of the structure. Eurocode 8 4.3.3.2.2 (4.6) [16] equation for 1.st natural period – equation 4.3.1 calculated from equation 2.4.3:

$$T_1 = C_t \cdot H^{\frac{3}{4}} = 0{,}74\ s \qquad (eq. 4.3.1)$$

where:

- C_t is geometric factor. For moment stiff spacious concrete frames C_t = 0,075
- H is height of the structure 25,15 m

4.3.2 Seismic load

The seismic load / earthquake load is the maximum acceleration expected, equation 4.3.2, calculated from equation 2.1.2 [16]. This is directly input acceleration.

$$a_g = 0{,}8 \cdot a_{gR} \cdot \gamma_1 = 0{,}4 \frac{m}{s^2} \qquad (eq. 4.3.2)$$

where:

- a_g is dimensioning gound acceleration in soiltype X
- a_{gR} is reference peak value for the soil acceleration for soil type X
- γ_1 is seismic factor – 1,0
- a_{g40Hz} is acceleration value found in Eurocode 8 map – 0,5 m/s^2

4.3.3 Dimensioning spectre

Based on EC8, 3.2.2.5 (3.13 – 3.16) [16], the horizontal dimensioning spectre $S_d(T)$ can be calculated based on previously calculated results from eq. 4.3.1, 4.3.2 and factors from [19] and [25]. Equation 4.3.3 calculates based on equation 2.4.6.

$$for\ T_C \leq T \leq T_D: \quad S_d(T) = a_g S \frac{2{,}5}{q} \left[\frac{T_C}{T}\right] = 0{,}6126 \frac{m}{s^2} \qquad (eq. 4.3.3)$$

where:

- q is structural factor, set at 1,5
- β is factor for lower limit value for horizontal dimension spectre
- S is soil enhancement factor – 1,7
- T_1 is first modal time, found in equation 4.3.1 – 0,74s
- $T_B(s) = 0{,}10$
- $T_C(s) = 0{,}40$
- $T_D(s) = 1{,}4$

4.3.4 Shear loads at base

Shear loads (F_b) at foundation level or top of stiff basement is based on EC8-1-1, 4.3.3.2.2(1) [16] calculated on equation 4.3.4 based on equation 2.4.2.

$$F_b = S_d(T_1) m \lambda = 2415{,}8\ kN \qquad (eq. 4.3.4)$$

where:

- $S_d(T_1)$ is dimensioning spectre for period T_1
- m is mass of the total structure above foundation level, input mass is the same as the result from the numerical model
- λ is correctional factor, $\lambda = 0{,}85$ if $T_1 < 2T_c$ and the structure has more than two stories, if not: $\lambda = 1{,}0$. In this instance, $T_1 < 2T_c$

Seeing how the shear load is dependent on period T_1 and where it is placed relative to T_b, T_c and T_d, as well as the factors for the soil, the following graph in Figure 49 can be drawn based on T_1 as the only variable, when all the other factors are set:

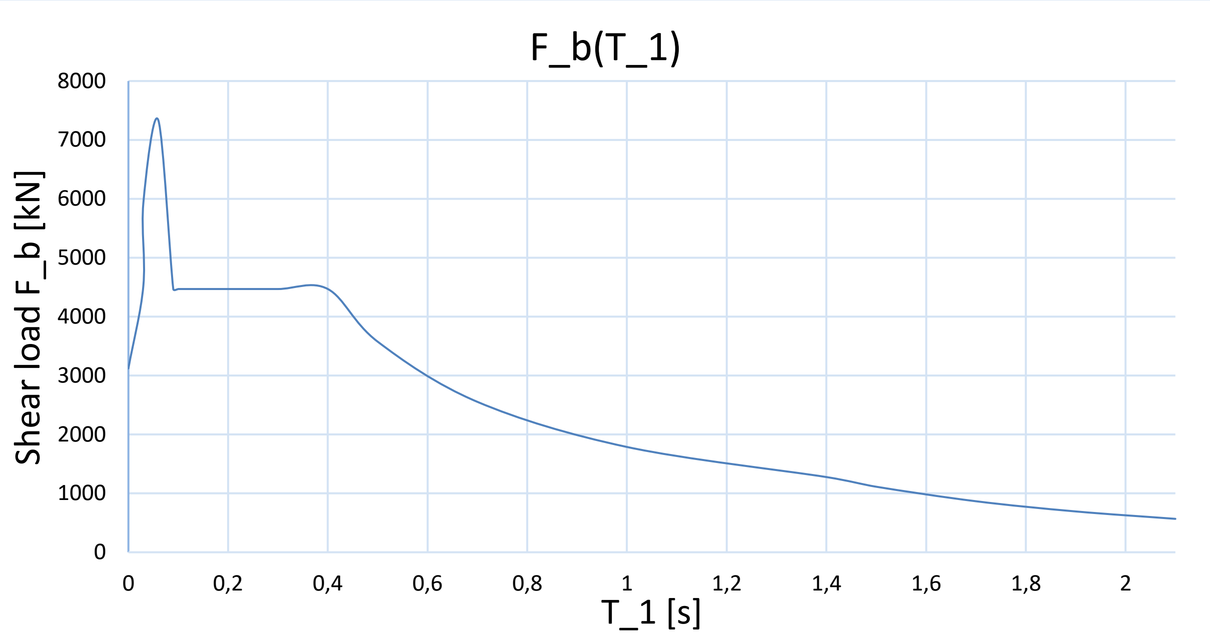


Figure 49: Graph F_b based on T_1

4.4 Summary of results

The final results used for comparison from both hand calculations and numerical calculation, as well as graphs drawn for the results from numerical model in Figures 50 and 51.

Results of hand calculations for structure variation 1 in table 19:

Table 19: Hand calculation main results

Natural period T_1 [s]	0,74 s
Base shear load at foundation F_b [kN]	2415,8

Results from numerical model in table 20:

Table 20: Summarized numerical model results

Modal:	V1 – Structure as built	V2 – Hollow core model	V3 – Original structure
T_1 [s]	0,58	0,53	0,28
T_2 [s]	0,45	0,40	0,22
Weight [tonnes]	4639,4	4169,6	2538,6
Base Shear / Torsion:			
Shear X [kN]	1260,2	1147,4	625,6
Shear Y [kN]	1288,1	1182,8	498,5
Torsion X [kNm]	19861,5	18194,1	6798,8
Torsion Y [kNm]	18842,9	17297,8	5651,8

Graphs of results from numerical model, Figures 50 and 51:

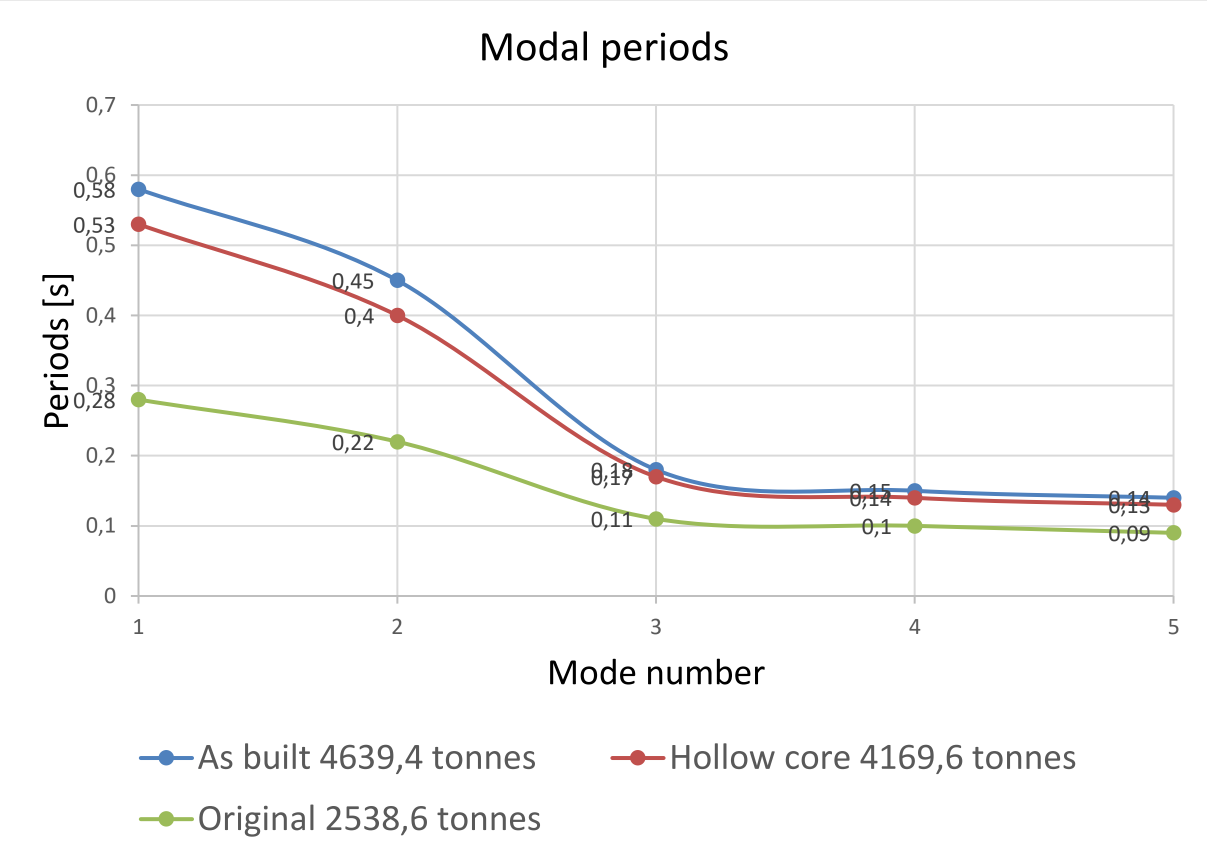


Figure 50: Summarized results of modal analyses

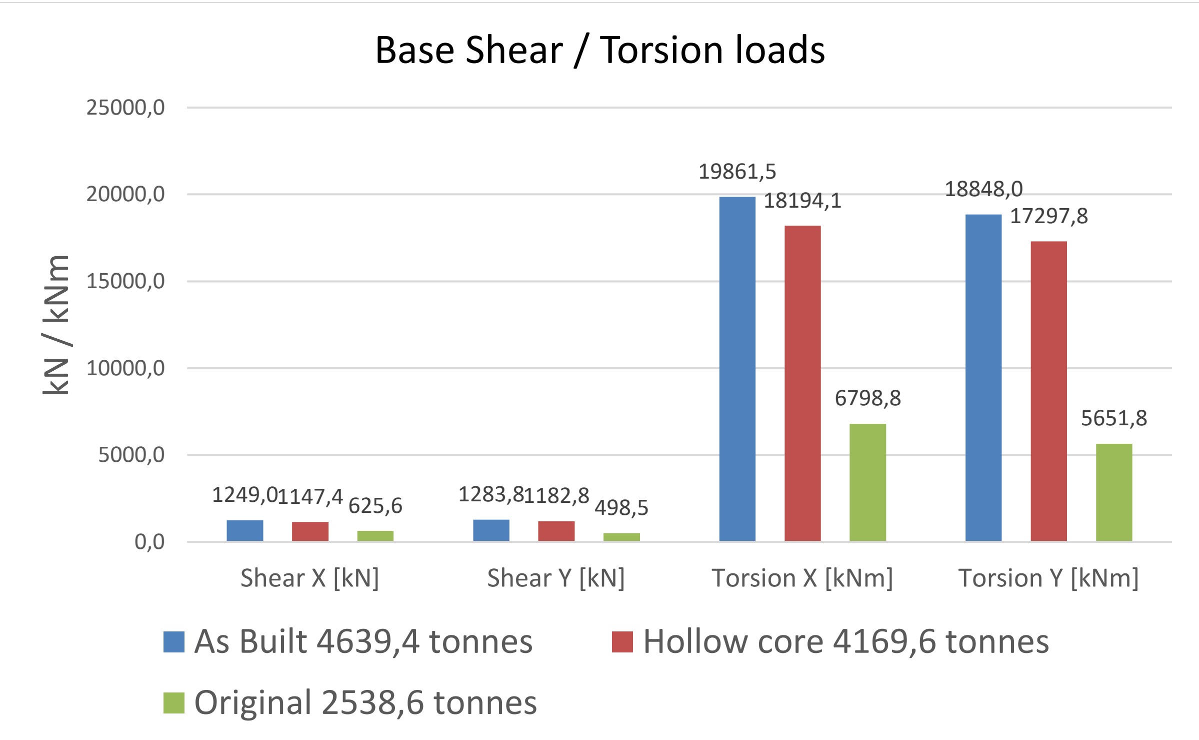


Figure 51: Loads at base of the structure

5 Discussion

5.1 Of results

The combined analysis of both modal and seismic analysis were performed on the three variations of the structure using autodesk robot software. The 3D model itself is obtained from a relative precise model from a team of architects and administration on in the project, and positions of columns, geometry and other parameters is considered very close to the exact reality. The model is rendered from a revit model to autodesk robot model, and cleaned up for analysis by removing all interior and non load carrying members, resulting in the raw structure made up of columns, slabs and stairwells with walls. Robot has a vast variety of different settings to accommodate various needs and situations. The options is based on standard values from Eurocode 8, and the rest is a standardized approach with normative settings. As there were no special circumstances such as nonoptimal geometries, loads or other with the structure. These options are displayed in the figures showing the approach on how to set the correct loads, load combinations, analysis types and more. The hand calculations is solely based on Eurocode 8's methods, and is far simpler than the numerical process.

Even though the seismic analysis in robot is based on the Eurocode 8 standard, the equations in Eurocode must be assumed to be somewhat simplified and conservative, both to make the process simpler but also so adjust for uncertainties, non optimal scenarios and similar. For both hand- and numerical calculations, the existing dimensioning acceleration $a_{g40Hz} = 0{,}5\,\frac{m}{s^2}$ is used as all other Eurocode parameters is used. Grunnteknikk AS concludes based on the new NORSAR map that $a_{g40Hz} = 0{,}2787\,\frac{m}{s^2}$ in the area of Tønsberg [19], but this is ignored in this report other than information and laying the basis for omission criterias.

Reinforcement in the slabs and columns is not included in the slabs and columns for the analysis as this has no influence on the modal periods nor base shear loads which is only based on weight and dimensions. The parameters for concrete weight is the standardized self weight with reinforcement. The real world placing of reinforcement is so complex and variable from part to part in the slabs that it would take a tremendous amount of time to complete. During a stress and deflection analysis, the reinforcement would be required, but this is not a part of the thesis.

Results of the modal analysis: Tables 13 – 15 shows that the structure as built, which is the heaviest has the highest natural periods, 9,4% higher T_1 period and 12,5% higher T_2 swing time than the 11,2% lighter alternative with the new floors built in hollow core slabs and steel columns. The original structure of 4 floors has significantly shorter periods and lower weight. The hand calculation results ends at 27,6% higher than the numerical model, which is significant allthough based on a simple equation with two variables (eq. 4.3.1). The modal analysis determines the natural periods of the structure and thereby the dynamic response duing variable loads such as an earthquake, and is a essential parameter with great influence in a seismic analysis.

Results from the seismic analysis: Tables 16 – 18 presents the primary results of base shear forces and torsion load at the base of the structure above ground level. The heavier variant 1 shows higher loads than variant 2 at 8,5 – 9,1% in both direction than variant 2 with new hollow core slabs and steel columns. Allthough relatively low %-values, it means a reduction of approximately 1 kN shear in both directions and 1,5 and 1,65 kNm torsion in each directions.

The results from the numerical model and hand calculations of variant 1 are surprisingly similar, where the hand calculation result only have one resulting shear force, and the numerical is divided into shear force in each axis as well as torsion load, combined shear force in UX and UY is almost the same as F_b with 5,5% difference.

In comparison with the original structure, variants 1 and 2 shows a significantly higher shear and torsion load, where the structure as built have approximately 100% increase in Shear UX, 157% increase in Shear UY, 192% increase in Torsion UX and 233% increase in Torsion UY loads.

With these massively increased base loads during an earthquake, even if the structure had been dimensioned for seimic loads when the original structure was built in 1983, these loads could bring the structure very close to if not beyond the ultimate state limit. This statement is made based on the tremendous increase in loads and is naturally marked without the knowledge of the actual loads this structure is dimensioned for and capable of handling.

These results are based on the assumption that the structure is fixed supported at the base. Though in reality, the structure with its basement and piles is altered by contact with the soil, where the real modal periods differs from the results in tables 13 – 15. The conference paper "Soil-structure interaction effects on modal parameters of office buildings with different number of stories" [26] reviews an example structure of 4, 8 and 12 stories, rectangular RC structure with base dimensions 28m x 42m, and finds the modal parameters with fixed support, fundamented in dense, stiff and soft soil.

It's [26] results shows that the structure will have notable lower frequencies (higher T-values) in dense soil than with fixed support, the 8 story structure (closest no. of floors to Kristina kvartalet) frequency were 10,6% lower in transverse direction and 13,5% lower in longitudal direction. This means the swing time is lower, increasing natural period time – the time between the oscillations, further altering the results from the seismic analysis.

Further, structure variation 2 with hollow core slabs and steel columns is an alternative where the new floors and columns is not continued with the identical set up as the previous floors. This alternative would require the optimization of placement of steel beams to support the slabs, as they primarily only carry the load in one direction. The special geometry of the previous RC slabs may prove difficult to copy with hollow core slabs, and non-optimal placement of both beams and columns is very likely. In this instance, new columns may be installed and placed on the floor of 5.th level with no column beneath, or require new columns on line in each floor under, causing further difficulties.

Furthermore, the difference in material properties in the layer where concrete slab changes to steel column instead of concrete, which behaves differently will increase the shear and torsional loads in that plate. This sudden increase between 5th and 4th floor is also shown in Table 17, especially for torsional loads which more than doubles.

Variation 2 has only slightly lower base loads at 1st floor than variation 1, and considering that if the structure were to be dimensioned for seismic loads, the difference between the two variations is so low that the cost would doubtly differ for the two alternatives. The only likely possibility that variation 2 could be chosen is if the total cost would be lower.

An alternative structure in all hollow core slabs and steel columns is based on these results and journal papers belived to have notable lower base loads, as all floors and columns would be far lighter and more flexible.

5.2 Assumptions for the model and analysis

Stiff basement, fixed support

The analysis performed in this thesis is based on the structure alone, disregarding the foundation and assuming fixed support at the base of 1st floor. This is a common assumption, and can be used when the structure has a stiff basement or foundation, as the interaction between the structure and foundation will be so stiff that swaying or displacement in this intersection can be neglected, see chapter 2.5.2. This assumption is made as it significantly lowers the difficulty of the analysis, not only with a simpler model where small faults, often trifles can cause trouble during the analysis. The parameters and properties of the soil and layers is not known, and performing an analysis with the foundation and soil-structure interaction is therefore not possible at this stage.

In reality a basement and or pile foundation will not be fixed in the soil, and displacement will occur in some extent during event such as an earthquake. The soil has its own stiffness mode based on type, and may even change characteristics itself during shaking. The soil beneath Kristina kvartalet – clay / quick clay is also vulnerable to liquefaction, further reducing the stiffness between structure and soil. A full analysis of the structure would require the piles aswell with soil-structure interaction.

There is also some assumptions and approximations within the 3D model, where the slabs and columns is without reinforcement, and the hollow core slabs is modelled as a solid concrete slab, and the thickness is adjusted so the total self weight is equal to the self weight of the hollow core slab. Here, a hollow core slab 265 transfers to a solid concrete slab of thickness 146 mm. This can be done for these types of analysis, where weight and stiffness is the only

Neighbouring structures

Kristina kvartalet have two neighbouring structures which it might not seem to be in connection with. In reality they are in contact, but only face to face which might have been resembled by a pinned support line that would not let the structure move through the neighbouring structure.

Even though they could have a slight impact on our structure, it is assumed that Kristina kvartalet is free to move – thereby removing the neighbouring structure.

5.3 Piles and Soil-structure interactions (SSI)

In this report, the piles have not been assessed nor analyzed for loads during an earthquake, therefore, soil-structure interaction analysis has neither. The analysed model is structure above ground level only, and the assumption to use fixed support at the base level is reasoned with a stiff basement, which enables this premise.

In reality, the structure is compised by the seven floors, two basement levels beneath 1.st floor and pile foundation from basement and down through clays till rock / berg is reached 3 – 12 meters below the bottom level of the lowes basement. The piles will therefore be exposed to the base loads at the bottom level of the basement, which is believed to be higher than the resulting base loads at the 1st floor, based on overall increased load the lower in the structure it is measured – which is naturally with increased weight.

The more realistic effective forces will differ from the results shown in this report due to a larger and heavier structure with different system damping and modal periods. The piles will also undergo the dynamic loading from the interaction between the piles and the soil. If the foundation and basements were included, the modal periods would be different from the analysis with no SSI, as the modal periods will be for the entire structure with soil contact [27], [28].

The journal paper "Effect of Embedded Basement Stories on Seismic Response of Low-Rise Building Frames Considering SSI via Small Shaking Table Tests" [29] have compared structures with two different basement depths with both fixed support at basement levels and non-fixed – founded in the soil. Both steel-frame and concrete structure versions of a 7-story building were used, basement levels of 3m and 6m, experiments took place at shaking tables with 1:50 building size ratio. Earthquake magnitude equal to 3 very large earthquakes of magnitude 6,7 to 7,6 were simulated. This paper did not consider or measure variations in base loads, but concluded the non-fixed structures had greatly amplified dynamic response in regard to lateral floor displacement, the version with 6m basement had lower amplification than the version with 3m basement. The deeper basement also ensured lower basement displacement than the other. They also encourage design engineers to consider SSI also for lower-rise buildings, although they achnowledges that the process is complex [29]. See figure 51 for illustration of fixed support compared to basement with piles, where the displacement will be larger and swing further and slower when considering basement and piles with Soil-structure interaction.

This paper simulates major earthquakes with large magnitude with a scaled model with a foundation of only walls and floors. It also says that their experimental and numerical results for their scaled model are in good agreement with numerical models of full size models. Kristina kvartalet has as this model two levels of basement floors beneath ground level, in addition to piles, where they will also contribute to reduce lateral displacement of the structure floors [29]. There is however no doubt that the structure will have larger lateral displacement in the real world than with the assumption of fixed support at the base floor [29].

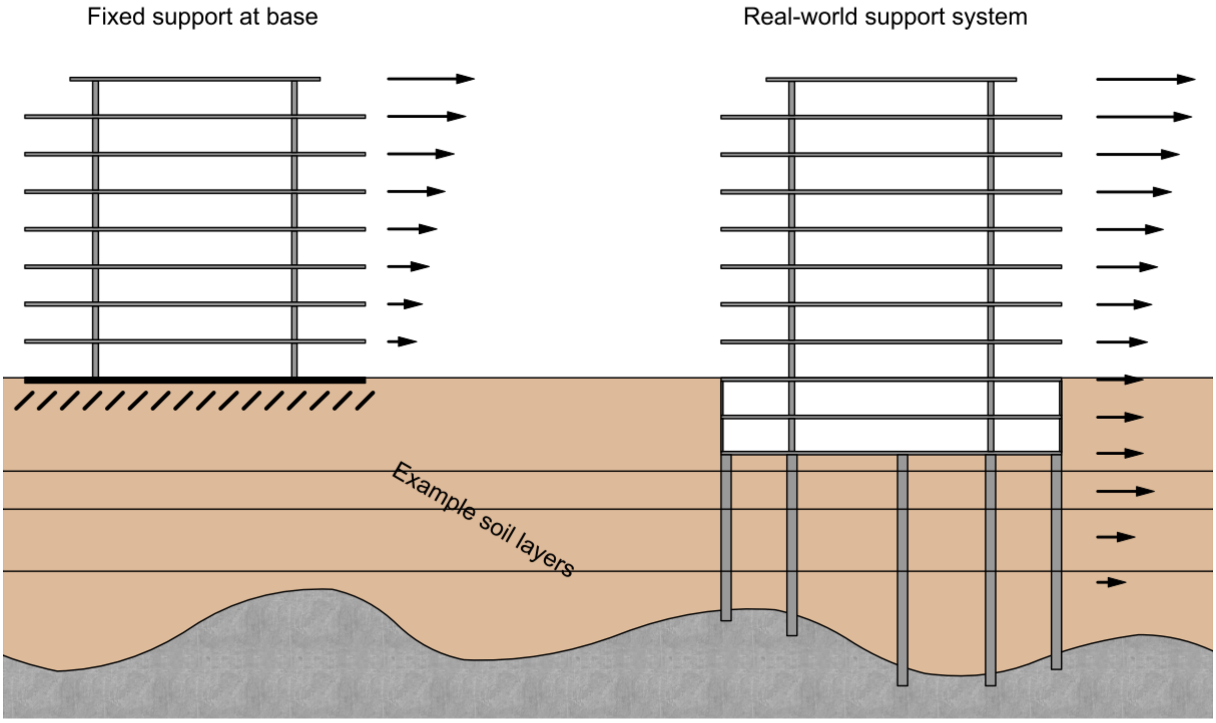


Figure 52: Illustration displacement with fixed support and real world constraints

Based on the Eurocode 8 equations (eq. 2.4.2 – 2.4.7) for calculating base shear loads and dimensioning spectre (see chapter 2.4.1 and 4.3.4 with Figure 49 for illustration of load based on T), a higher swing time T will reduce the horizontal spectre S_d and thus the base shear load F_b, given that the only variable parameter is the natural period T between the cases. As the structure based on the results from [29] will see a higher swing time if the support is based on piles and a deep basement is included, the real base loads at basement top will in reality be lower than the results in this report shows with fixed support. These equations could by some be argued to be conservative, but will still prepresent loads close to the real loads.

Moreover, Eurocode 8 states that structures in soft soil types such as S_1 should be assessed for SSI [17], further supporting the previous statements of the thesis.

5.4 Liquefaction of soil

Another factor for this case is the risk of liquefiable soil during and earthquake, as the soil is clays and quick clays. The potential and risk for liquefaction [14] for in this instance is not assessed due to lack of information. The paper – A fundamental omission in seismic pile design leading to collapse [30] reviews collapsing of piled foundations in liquefiable soils, as each case studied shows that lateral deflections caused by lateral loads are greatly amplified if the axial load approaches the staticly critical load [30]. The paper focuses on past collapses originated from Japan and Alaska, regions with far higher seismic activity than Norway. For reference, the seismic activity in Japan lies roughly between 0,08 to 1,5g, Alaska between 0,01 – 0,9g and Tønsberg between 0,03 – 0,05g in peak ground acceleration before adjustment for soil and seismic importance factor, see Figures 53 and 54 below based on a map for seismic hazard and expected ground acceleration [31].

While the seismic activity in Tønsberg is far lower, the principles and risks are still relevant. [30] concludes that during liquefied soil the initiation of buckling cannot be prevented, but the resulting displacement will dictate the location of a hinge by offering lateral resistance to the buckling pile. This means that the equivalent length of the column / pile will increase when the top displaces, dramatically reducing the buckling load (equation 2.3.1, chapter 2.3 Piles). [30] and [9] provides adjusted equations for calculating the increased loads on piles during events of liquefied soil. The equations clearly illustrates that there will be significantly higher loads during liquefaction and thereby increased need for evaluation of the piles in these events.

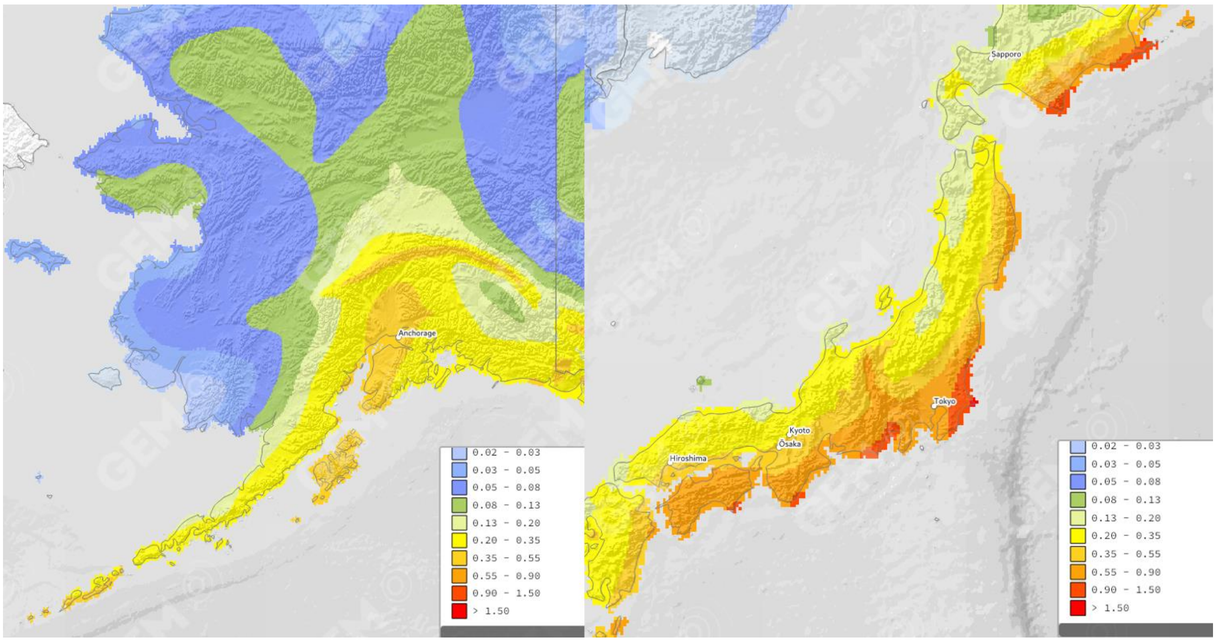


Figure 53: Seismic hazard map of Alaska, USA (left) and Japan (right) [31]

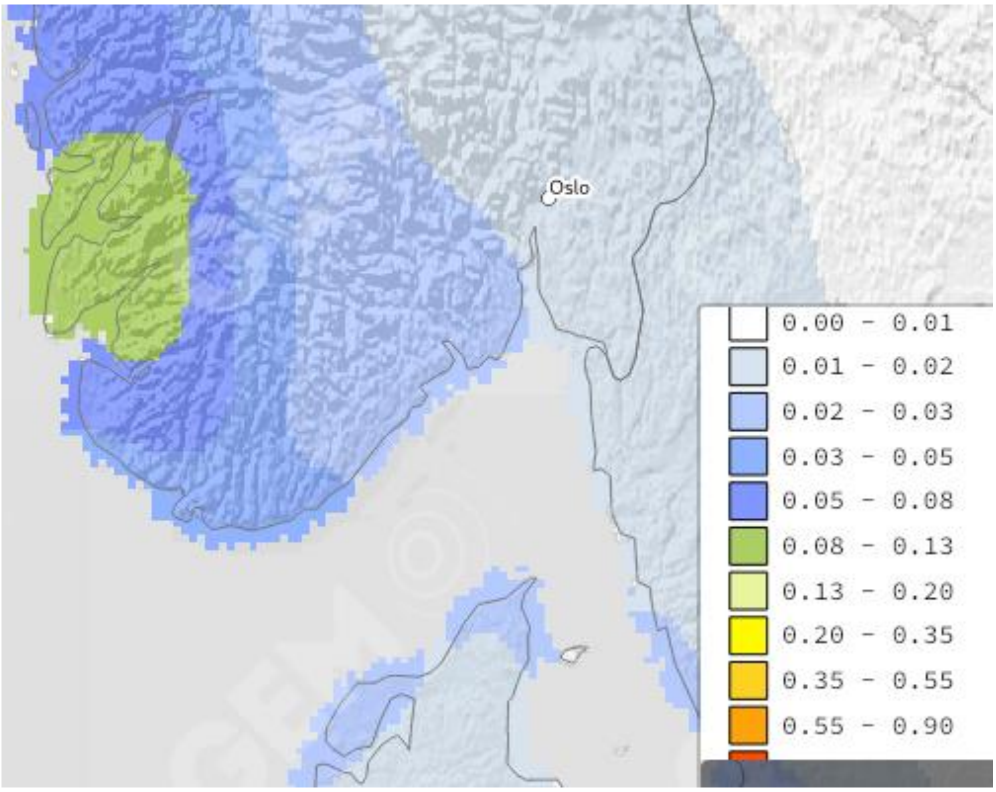


Figure 54: Seismic hazard map of southern Norway [31]

5.5 Validity and relevance of the results

As previously written, the structure is not analysed for SSI nor interaction between the structure and foundation. In accordance to the real structure, not including these these parameters automatically renders the results inaccurate. The real world natural periods and base loads will certainly differ from the results as shown [32], but in which magnitude is unknown. Based on Eurocode where one can safely assume fixed support at the base of the structure, the results can be considered somewhat accurate, granted that the 3D model and all inputs made to the model and analysis is dependable.

The discussion wether assuming fixed support at the base is acceptable, relies on how accurate the results need to be. For the design engineer, the resulting base loads is the primary objective to dimension the columns and piles it is close enough if only considering the structure above ground level. While for studies or projects where an accurate result is needed, this is not good enough. Not knowing the exact design loads, means the design engineer must make conservative estimates would be made, risking to add more concrete and reinforcement than necessary.

5.6 Omission criteria / utelatelseskriterie

In the previous static engineering process for adding the floors of Kristina kvartalet, seismic dimensioning had been deemed not needed, as omission criteria 2 – very low seismic activity $a_g \cdot S < 0{,}49 \frac{m}{s^2}$ was met. In this simple equation dimensioning seismic acceleration $a_g = 0{,}2787 \frac{m}{s^2}$ [19] which is very low. The enhancing factor for soil S is conservatively set to the highest at 1,7 [19] as the soil is primarily clay / quick clay and is at risk for liquefaction. This means that even while considering high-risk soil in comparison to liquefaction, the seismic load is still very low and seismic dimensioning could therefore comfortably be omitted in accordance with Eurocode, which corresponds to both the structure and foundation with piles.

Pro – use of omission criterias:

The prime reason for using omission criterias to avoid seismic dimensioning is to perserve resources. Not only the spared time and cost to engineer the project, but increasing the size of slabs, columns and piles unnecessarily will increase the cost of building or reinforcing the structure by far more.

In the Tønsberg area of Norway, the seismic activity is mid-tier for the country according to the map, adding that Norway is a land of low activity, a potential earthquake of low magnitude will do little damage as the most powerful recorded earthquake in Norwegian history is M_s 5,8 and more normal earthquakes are usually below 5,5 [6]. Seeing Figure 3 in chapter 2.1.4 that indicates the relationship between earthquake magnitiude and ground acceleration it is clear that a normal earthquake will result is a very low ground acceleration, far below 0,5 m/s^2 which is used in this analysis [5]. This relationship and graph further comforts an omission of the structure.

[18] is a report based on NORSAR's updated map for seismic zones, where the authors made an economic consequence estimate for a example hospital structure and an office building. The example hospital is relatively small with 8 floors with area 84x20 meters. Comparing costs with and without seismic dimensioning and reinforcing for seismic loads, the hypothetical hospital structure could save approximately 18% of the building cost, considering only the raw structure parts such as piles, basement, load carrying members and floors. (Price with seismic: 33,33 M NOK, price without: 39 M NOK, saved 6,33 M NOK.) The office structure with 8 floors at 54x18 meters and piled foundation would save approximately 700.000 NOK – 9% if omitted for seismic dimensioning.

Both of these calculations has been performed by Multiconsult in their ISY-Calcus software, based on a standardized price registery – Norsk Prisbok for august 2019. This can therefore not be considered 100% precise, but relatively close to the real cost and good enough for comparison [18].

Omission criterias will make seismic dimensioning unnecessary in the events where the risks for loss of life or injuries is so low that an analysis would be considered a waste of resources. The criterias is defined and verified by Eurocode, and is standardized and used extensively in Norway. Utilizing the omission criterias should therefore be considered safe if performed properly. Short summary of the omission criterias:

In criteria 1 - Buildings must be considered low significance (seismic importance class 1) to human health, such as agricultural structures and small buildings. Houses, apartment buildings, offices and similar buildings does not qualify for criteria 1 [16].

Criteria 2 – The structure must be in a zone of low seismic activity, peak ground acceleration adjusted for seismic importance class and soil factor must be lower than 0,49 m/s^2 [16].

Criteria 3 – The structure it self must not accelerate too much under an earthquake while having relatively stiff properties to meet this criteria [16].

Criteria 4 – As wind load and seismic load have the same type of dimensioning loads – axial loads on the base, an adjusted equation for wind load can make seismic dimensioning excessive. Checking this, would require to perform a basic seismic analysis to find bare shear forces, in addition to wind load. Structures in seismic class 4 – such as hospitals, fire stations, power plants, dams etc. cannot qualify for criteria 4 [16].

Con – use of omission criteras:

Exerting the omission criterias where only one criteria have to be met, other factors such as soil type, foundation type and properties does not have to be considered. I.e. Criteria 3 – dimensioning spectrum $S_d(T) < 0{,}5\,\frac{m}{s^2}$ does not account for the soil, and criteria 2 – very low seismic activity $a_g \cdot S < 0{,}49\,\frac{m}{s^2}$ does not consider the structure properties other than seismic importance class. Criteria 4 does not consider soil nor structure response at all, as long as the resulting base shear load of the wind is larger than the seismic load. As long as one of the criterias is met, all other factors in the other three criterias is not important, which might cause risk.

An important weakness of the omission criterias is that when seismic dimensioning can be omitted, it applies to both the structure and the foundation. In this particular case, seismic dimensioning were excluded due to low seismic activity, but it could might as well be excluded due to high wind, while the seismic activity were far higher than they are at this site. When the soil is clays vulnerable to liquefaction, introduces a risk in the event of an earthquake where the piles might collapse, further collapsing the structure, while the structure still qualifies for an omission criteria.

6 Conclusion

The analysis is based on Eurocode settings and parameters such as ground acceleration, soil factors, structural factor and load combinations. With theses results and the assumption of fixed support at the base of the structure, the base shear and torsion loads has been increased by 100% - 233% from the original structure to the new enlargened structure. Even if a lower ground acceleration and support on piles, the increase would still be far higher than original, thus raising concern about a possible problem towards ultimate state limit for the columns of the structure.

According to [29], a structure with basement in soil will have higher modal periods T than with fixed support. Based on the base shear loads formula from Eurocode (Eq. 2.4.2 – 2.4.7 in the thesis), a higher modal period T will lower the base shear load, given that the other parameters involved in the structure doesn't change. These shear loads is only applies to the structure in this case, but considering the foundation aswell, the base shear load will increase downwards toward the piles. It is clear that the assumption of fixed support will in addition to simplify the process, ensure higher swing speed – lower modal periods and thereby increase loads and hence make the analysis somewhat conservative [32].

To consider the basement and foundation for seismicity, a separate analysis must take place to analyse the influence of soil-structure interaction and a case of liquefaction of the soil which potentially dramatically increases the loads upon the piles. Since these analyses arent performed here, the state of foundation can not be concluded. Provided the magnified increase in shear loads regardless of fixed support, as well as the recommendation of [29], an analysis of the foundation is highly revised for this structure. This is also supported by Eurocode 8 due to soft soils [17].

For the omission criterias, the low seismic activity is the fundament for safe omission both because it has been used to qualify for omission, but also because with little to no acceleration, there will be very low to no seismic loads. With this low loads, spending resources on engineering as well as reinforcing the structure would be a waste of time and money. Yet, the special combination of no analysis of the fundament and a soil type in risk of liquefaction brings out some concerns in the case of a normal to unusually high earthquake for the region.

The final conclusion is therefore – Omission of seismic analysis for the structure is considered safe, but the foundation should be assessed for the impact of an earthquake and liquefaction.

7 Further work

As this thesis have covered the analysis of the structure itself, several significant assumptions and limitations has been made, and the parameters is based on values and procedures from Eurocode.

- In accordance with the conclusion, the next step is a proper seismic analysis of the entire structure – with the foundation, assessing for soil-structure interactions and the risk of liquefaction and its impact in the case it happens.

- A stress analysis should be performed for the structure, adding the reinforcement representative into the structure to assess wether the structure can handle the loads under both Eurocode acceleration value 0,5 m/s^2 and the NORSAR value of 0,2787 m/s^2. This will also apply for the foundation – basement and piles, as an important part of the structural integrity.

- Analyse the structure under the influence of neighbouring structures if they were connected

- Monitoring and or detailed analysis of parameters such as the real damping of the structure natural frequency, which could alter the results in the mentioned assessments.

These steps could naturally be a ordinary engineering task, but would also be extensive enough for a thorough new master thesis with this one as a base.

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Appendix, attachments

The following reports are attcheded as they are not published, and is essential to this thesis, as some of the discussions and background information is based on them. Both of these reports are in Norwegian.

Attachment 1 – Report from Grunnteknikk AS – Vurdering utelatelseskriterier Kristina kvartalet (8 pages)

Attachment 2 – Report from Multiconsult ASA – NORSAR Oppdaterte seismiske sonekart (28 pages)

TIL:	Kristina Kvarteret AS v/Even Landsem
Kopi:	
Fra:	GrunnTeknikk AS

Dato:	26.03.21
Dokumentnr:	115584n1
Prosjekt:	113059
Utarbeidet av:	Janne Reitbakk
Kontrollert av:	Jon Adsersen Gulbrandsen

Tønsberg. Kristinakvarteret
NORSAR - seismisk forhold

Sammendrag:
GrunnTeknikk AS er engasjert av Kristina Kvarteret AS til å utføre geoteknisk bistand i forbindelse med utvikling av deres eiendom i Tønsberg sentrum.

I foreliggende notat gis en oppsummering av våre seismiske vurderinger. Dette omfatter bestemmelse av grunntype og forsterkningsfaktor, bestemmelse av dimensjonerende bergakselerasjon ved bruk av NORSAR soneringskart, samt vurdering av utelatelseskriterium for seismisk prosjektering.

Vi mener prosjektet bør kunne plasseres i seismisk klasse 2.

Dimensjonerende grunnakselerasjon for berg a_g blir 0,2787 m/s^2 iht. NORSAR jordskjelv soneringskart for det aktuelle prosjektet.

Aktuell forsterkningsfaktor er konservativt bestemt til S=1,7.

Dette medfører at prosjektet faller under utelatelseskriterium for seismisk prosjektering iht. Eurokode 8 del 1 avsnitt NA.3.2.1(5)P (utelatelseskriterium a_g * S < 0.49 m/s^2).

Nærmere gjennomgang fremgår av notatet.

INNHOLDSFORTEGNELSE

VEDLEGG

1 Bergakselerasjon og responsspektrum for berg beregnet av NORSAR

REFERANSER

[1] GrunnTeknikk AS. Geoteknisk notat områdestabilitet 114971n1 av 18.08.2020
[2] NS-EN 1998-1:2004+A1:2013+NA:2014 (Eurokode 8, del 1)
[3] NORSAR. Webinar «The new Norwegian seismic zonation map for the Eurocode 8 design spectra and amplification factors.", avholdt 19.01.2021
[4] Rådgivende Ingeniørers Forening (RIF). Veileder "Dimensjonering for jordskjelv", revidert 2018

1 Innledning

GrunnTeknikk AS er engasjert av Kristina Kvarteret AS til å utføre geoteknisk bistand i forbindelse med utvikling av deres eiendom i Tønsberg sentrum. Eiendommen har adresse Farmannsveien 3, omtrentlig avgrensing er vist i figur 1.

Figur 1. Flyfoto (1881.no) med planområdet omtrentlig avgrenset med rødt.

I foreliggende notat gis en oppsummering av våre seismiske vurderinger. Dette omfatter bestemmelse av grunntype og forsterkningsfaktor, bestemmelse av dimensjonerende bergakselerasjon ved bruk av NORSAR soneringskart, samt vurdering av utelatelseskriterium for seismisk prosjektering.

2 Terreng og grunnforhold

Eiendommen brukes i dag til parkering, handel og kontordrift og er generelt flatt over området.

Det er utført grunnundersøkelser i området ved flere anledninger i nærområdet. Utførte grunnundersøkelser er omtalt og vurdert i notat for områdestabilitet [1]. Det er ikke utført egne undersøkelser for prosjektet i denne omgang.

Kort fortalt består løsmassene i nærområdet av siltig leire over fastere morenemasser til berg. Leirmassene kan være sensitive og kvikke.

3 Seismiske forhold

Det er utført seismiske vurderinger iht. Eurokode 8 del 1 (EK 8-1) [2] kombinert med dimensjonerende grunnakselerasjon for berg bestemt ut fra NORSAR soneringskart.

På webinar avholdt av NORSAR den 19.01.2021 [3] ble det opplyst at NORSAR soneringskart kan anvendes sammen med grunntyper og tilhørende responsspektra angitt i nasjonalt anneks til Eurokode 8 del 1. Dette er lagt til grunn for våre vurderinger.

3.1 Dimensjonerende bergakselerasjon bestemt av NORSAR

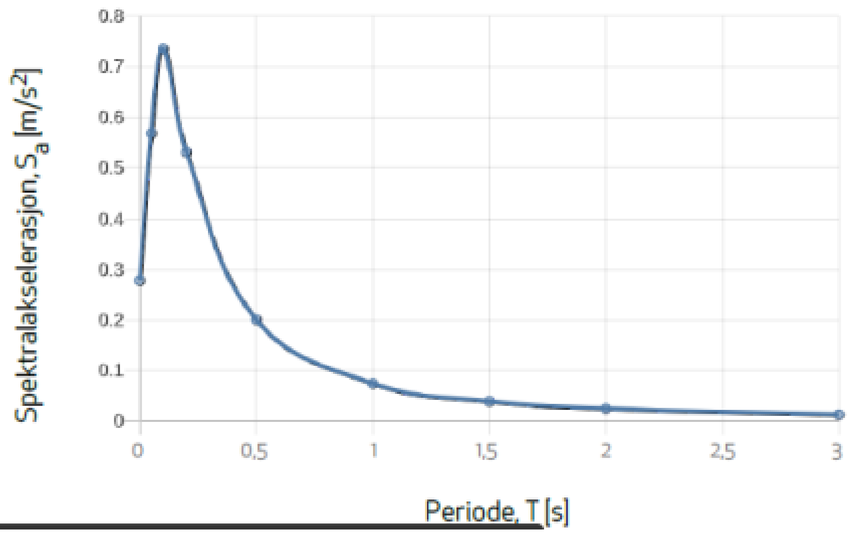
Dimensjonerende grunnakselerasjon for berg er iht. EK 8-1 definert som:

$$a_g = a_{gR} * \gamma_l \qquad (1)$$

Der, a_{gR} er «referansespissverdi for berggrunnens akselerasjon» og γ_l er seismisk faktor (bestemt avhengig av seismisk klasse). Vi mener prosjektet bør kunne plasseres i seismisk klasse 2, hvilket gir γ_l=1,0.

Figur 2 viser verdier for horisontal spektralakselerasjon (Sa), 5% dempet, for ulike svingeperioder T (der PGA er lik a_{gR}). Beregningene er utført av NORSAR. For nærmere detaljer vises til vedlegg 1.

T[s]	S_a[m/s^2]
PGA	**0.2787**
0.05	0.5679
0.1	0.7349
0.2	0.5304
0.5	0.2011
1.0	0.0738
1.5	0.0384
2.0	0.0243
3.0	0.0124



Figur 2. Bergakselerasjon og responsspektrum for berg beregnet av NORSAR (utsnitt av vedlegg 1).

For seismisk klasse 2 blir dimensjonerende grunnakselerasjon for berg da a_g=0,2787 m/s^2.

3.2 Grunntype og forsterkningsfaktor

Det er bestemt grunntype og forsterkningsfaktor iht. Eurokode 8 del 1 (EK 8-1) [2], gjeldende regelverk for seismisk påkjenning.

Grunntype velges iht. standardens tabell NA.3.1, vist på figur 3:

Tabell NA.3.1 – Grunntyper 1)

Grunn-type	Beskrivelse av stratigrafisk profil	Parametere 2) 3)		
		$v_{s,30}$ (m/s)	N_{SPT} (slag/30cm)	c_u (kPa)
A	Fjell eller fjell-liknende geologisk formasjon, medregnet høyst 5 m svakere materiale på overflaten.	> 800	–	–
B	Avleiringer av svært fast sand eller grus eller svært stiv leire, med en tykkelse på flere titalls meter, kjennetegnet ved en gradvis økning av mekaniske egenskaper med dybden.	360 – 800	> 50	> 250
C	Dype avleiringer av fast eller middels fast sand eller grus eller stiv leire med en tykkelse fra et titalls meter til flere hundre meter.	180 – 360	15 - 50	70 - 250
D	Avleiringer av løs til middels fast kohesjonsløs jord (med eller uten enkelte myke kohesjonslag) eller av hovedsakelig myk til fast kohesjonsjord.	120 – 180	10 – 15	30 – 70
E	Et grunnprofil som består av et alluviumlag i overflaten med v_s-verdier av type C eller D og en tykkelse som varierer mellom ca. 5 m og 20 m, over et stivere materiale med v_s > 800 m/s.			
S_1	Avleiringer som består av eller inneholder et lag med en tykkelse på minst 10 m av bløt leire/silt med høy plastisitetsindeks (PI > 40) og høyt vanninnhold	< 100 (antydet)	–	10 - 20
S_2	Avleiringer av jord som kan gå over i flytefase (liquefaction), sensitive leirer eller annen grunnprofil som ikke er med i typene A – E eller S_1.			

1) Hvis minst 75 % av konstruksjonen står på fjell og resten på løsmasser, og konstruksjonen står på ett kontinuerlig fundament (platefundament), kan grunntype A benyttes.

2) Valget av grunntype kan være basert på enten $v_{s,30}$, N_{SPT} eller c_u. $v_{s,30}$ anses som den mest aktuelle parameteren å benytte.

3) Der det er tvil om hvilken jordtype som skal velges, velges den mest ugunstige.

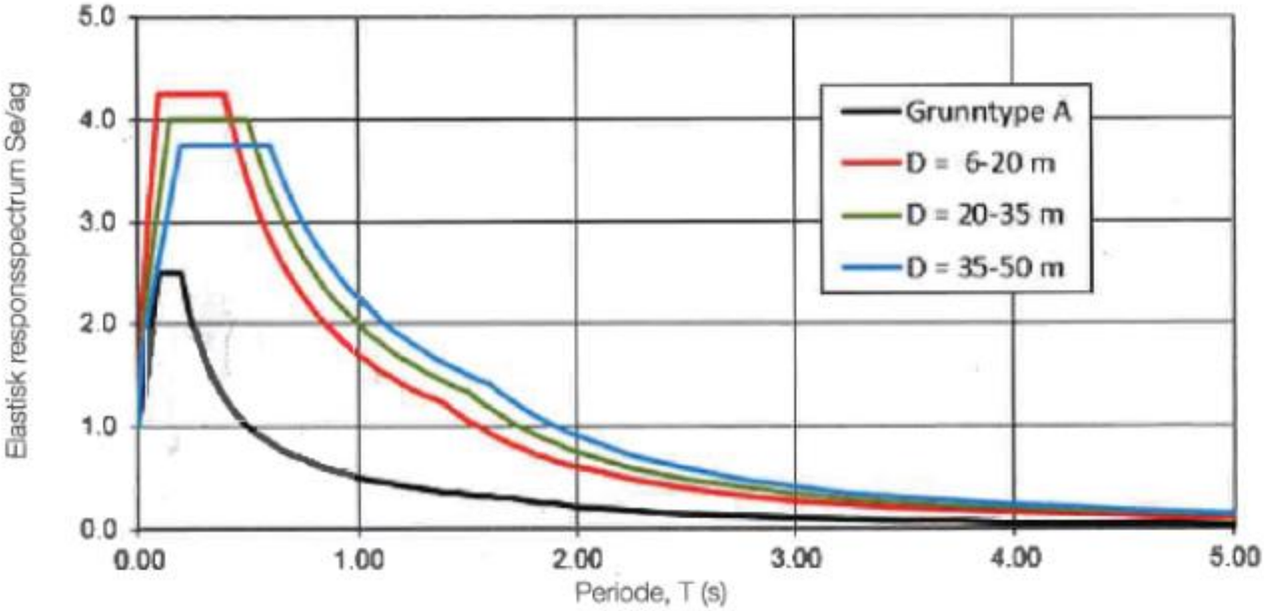
Figur 3. Tabell NA.3.1 – Grunntype fra nasjonalt anneks i Eurokode 8 del 1 [2].

Grunntypen er konservativt bestemt som S2. Dette da det ikke er utført grunnundersøkelser på tomta og dermed ikke kan utelukkes at det finnes sprøbruddmaterialer/kvikkleire i grunnen.

For grunntypene S1 og S2 anbefales det i nasjonalt tillegg til EK 8-1 at det utføres en grunnresponsanalyse for bestemmelse av elastisk responsspektrum.

RIF veilederen «Dimensjonering for jordskjelv» [4] har basert på slike analyser anbefalt elastiske responsspektra for norske bløte leire og kvikkleirer, vist på figur 4 og 5.

Med utgangspunkt i nærliggende grunnforhold og mest konservative antagelse, vurderes forsterkningsfaktor S=1,7 å kunne anvendes.



Figur 4. Elastisk responsspektrum for norske bløte leirer og kvikkleire, RIF veileder for jordskjelv [5].

Dybde til fjell	S	T_B (s)	T_C (s)	T_D (s)
D = 6 - 20 m	1,7	0,10	0,40	1,4
D = 20 - 35 m	1,6	0,15	0,50	1,5
D = 35 - 50 m	1,5	0,2	0,60	1,6

Figur 5. Verdier for parametere som beskriver de anbefalte elastiske responsspektra i figur 3, RIF veileder for jordskjelv [4].

3.3 Vurdering av utelatelseskriterium

Iht. EK 8-1 avsnitt NA.3.2.1(5)P faller prosjektet under utelatelseskriterium dersom følgende er oppfylt:

$$a_g * S < 0{,}49 \text{ m}/s^2 \quad (2)$$

For prosjektet fås da a_g*S=0,2787*1,7=0,47 m/s^2 og prosjektet faller da inn under utelatelseskriterium for seismisk prosjektering.

Kontrollside

Dokument	
Dokumenttittel: Tønsberg. Kristinakvarteret, NORSAR - seismisk forhold	Dokument nr: 115584n1
Oppdragsgiver: Kristina Kvarteret AS	Dato: 26.03.21
Emne/Tema: Seismiske forhold	

Sted		
Land og fylke: Norge, Vestfold	Kommune:	
Sted:		
UTM sone:	Nord:	Øst:

Kvalitetssikring/dokumentkontroll					
		Egenkontroll av		Sidemannskontrav	
Rev	Kontroll	dato	sign	dato	sign
	Oppsett av dokument/maler	26.03.21	jr	26.03.21	JAG
	Korrekt oppdragsnavn og emne	26.03.21	jr	26.03.21	JAG
	Korrekt oppdragsinformasjon	26.03.21	jr	26.03.21	JAG
	Distribusjon av dokument	26.03.21	jr	26.03.21	JAG
	Laget av, kontrollert av og dato	26.03.21	jr	26.03.21	JAG
	Faglig innhold	26.03.21	jr	26.03.21	JAG

Godkjenning for utsendelse	
Dato: 29.03.21	Sign.: Janne Reitbakk

NORSAR

Seismiske laster er generert fra jordskjelv soneringskart v.1.0.2019*

* Seismic Zonation and Earthquake loading for Norway and Svalbard; Load estimates based for Eurocode 8 applications

Dato:	2021-03-26
Klokkeslett:	15:06:07
Bruker-id:	Janne Reitbakk
Rapport sendes til:	janne@grunnteknikk.no
Data er generert for geografisk lokasjon:	Farmannsveien 3, 3111 Tønsberg, Norge 59.2709° N; 10.4083° E
Seismisk grunnakselerasjon er generert for:	Berg, v_s = 1200 m/s
Prosjektnavn / Utbygger:	Tønsberg. Kristinakvarteret / kRISTINAKVARTERET as
Verdiene er gyldig innenfor 500 m radius rundt geografisk lokasjon. For utvidet område eller lavere sannsynligheter, kontakt:	soneringskart@norsar.no
Bekrefter bruk av data kun på angitt lokasjon / prosjekt:	Ja

Seismisk grunnakselerasjon, Berg, 5 % dempet

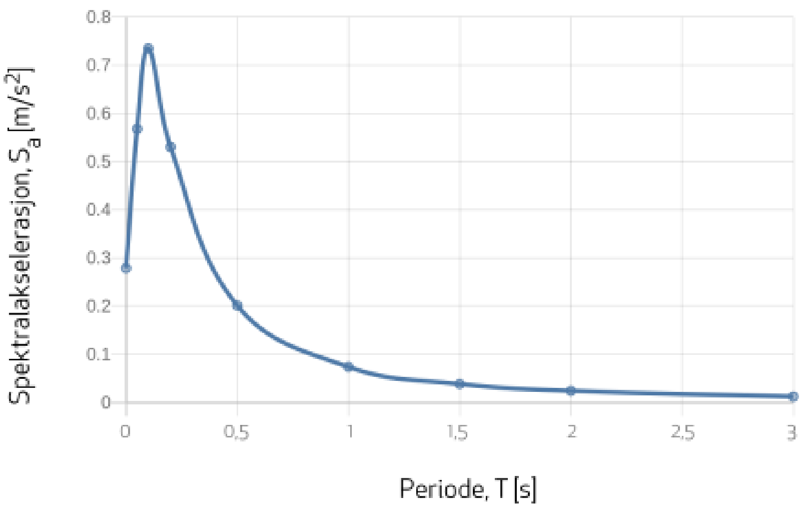
Dimensjonerende grunnakselerasjon er definert som:

Ag = seismisk faktor * a_{gR}= seismisk faktor * 0.8*a_{g40Hz}

Beregnet verdi for seismisk grunnakselerasjon a_{gR}: 0.2787 m/s^2

Verdiene for horisontal seismisk akselerasjon (S_a), 5% dempet, er vist som funksjon av perioden T i tabellen og grafen (seismisk responsspektrum). Eurokode 8 spektrum kan beregnes ut fra $\mathbf{a_{gR}}$. Seismisk grunnakselerasjon er basert på berggrunn med v_s >800 m/s (v_s=1200 m/s) og beregnet for returperiode av 475 år (overskridelsessannsynlighet på 10% over 50 år).

T[s]	S_a[m/s^2]
PGA	**0.2787**
0.05	0.5679
0.1	0.7349
0.2	0.5304
0.5	0.2011
1.0	0.0738
1.5	0.0384
2.0	0.0243
3.0	0.0124



Seismiske laster generert for oppgitt geografisk lokasjon er basert på siste versjon av jordskjelv soneringskart (v.1.0.2019). Tabellen over angir berggrunnens akselerasjon som forventes å bli overskredet over en tidsperiode på 475 år (overskridelsessannsynlighet på 10% over 50 år).

NORSARs tjenester og produkter for seismisk fare har blitt utviklet innenfor et probabilistisk rammeverk, jfr. disclaimer i vedlagte Executive Summary. Bruker av data må gjøre seg kjent med disclaimer.

RAPPORT

NORSAR Oppdaterte seismiske sonekart

OPPDRAGSGIVER

NORSAR

EMNE

Konsekvenser av endring i seismiske data for Norge

DATO / REVISJON: 02. mars 2020 / 02

DOKUMENTKODE: 10216470-RIB-RAP-001

Multiconsult

Multiconsult

RAPPORT

OPPDRAG	**Oppdaterte seismiske sonekart**	DOKUMENTKODE	10216470-RIB-RAP-001
EMNE	Konsekvenser av endringer i seismiske data for Norge	TILGJENGELIGHET	Åpen
OPPDRAGSGIVER	**NORSAR**	OPPDRAGSLEDER	Åshild Huseby
KONTAKTPERSON	Anne S. Lycke	UTARBEIDET AV	Nils E. Forsén, Åshild Huseby, Terje Kvarme, AS Bygganalyse
KOPI		ANSVARLIG ENHET	Multiconsult ASA

SAMMENDRAG

NORSAR har etablert en oppdatert database for seismiske data for Norge. Ved hjelp av databasen kan grunnakselerasjoner fastsettes for et geografisk sted, ut fra koordinater. Endringene i karakteristiske akselerasjoner i seismisk situasjon tenderer mot lavere verdier enn de någjeldende iht. NS-EN 1998-1:2004+A1:2013+NA:2014 (Eurokode 8), og kan således føre til et gunstigere lastbilde ved dimensjonering mot jordskjelv. Av særlig betydning er tilfelle der en kommer under utelatelseskriteriet iht. NS-EN 1998, der en per i dag er over. Endringene har også vesentlig betydning for byggverk i seismisk klasse IV, som sykehus og brannstasjoner med hensyn til konseptuelle løsninger og kostnader.

Rapporten sikter mot å illustrere de økonomiske konsekvensene av dette og hvordan de nye dataene praktisk kan innlemmes i regelverket.

Rent prinsipielt, og ut fra rapportens konklusjoner om økonomiske besparelser, er det hevet over tvil at bransjen må få tilgang til og anvende de oppdaterte 475årsverdiene som NORSAR har utarbeidet.

Formelt kan dette i det enkelte prosjekt forankres i Byggteknisk forskrift (TEK 17) §2-1(3). I denne paragrafen gis det adgang til alternativt til Norsk Standard å anvende annen, likeverdig standard. Det finnes i realiteten ikke likeverdig standard til Eurokode 8 (EK 8), men inntil endringsblad eller revisjon av det nasjonale tillegget foreligger, bør en kunne anvende verdier for spissakselerasjoner som det kompetente fagorganet NORSAR har utarbeidet. Direktoratet for byggkvalitet (DiBK) kan anmodes om å utarbeide et rundskriv om dette, eventuelt via Kommunal- og moderniseringsdepartementet (KMD). Anmodningen kan komme fra Rådgivende Ingeniørers Forening (RIF), som blant annet representerer foretak som erklærer offentligrettslig ansvar for konstruksjonssikkerhet i byggetiltak.

02	30.03.2020	Endringer pga. opphavsrett SN	ASHH	NEF	NEF
01	02.03.2020	Rapport	NEF/ASHH/TK	NEF	NEF
REV.	DATO	BESKRIVELSE	UTARBEIDET AV	KONTROLLERT AV	GODKJENT AV

INNHOLDSFORTEGNELSE

1 Bakgrunn

NORSAR har foretatt en oppdatering av seismiske sonedata for Norge og ønsker konsekvensene av oppdateringen belyst med tanke på byggekostnader. Multiconsult er engasjert for å utarbeide en rapport om dette basert på erfaringer og egnet eksemplifisering. Rapporten skal også belyse betydningen for regelverket Byggteknisk forskift (TEK) /Byggesaksforskriften (SAK) / Norsk Standard (NS).
Bygganalyse er engasjert for å bidra med vurderinger av kostnadsbildet på et overordnet nivå.

2 Regelverk

Bestemmelser om å ta hensyn til jordskjelv ved prosjektering av byggverk og deres konstruksjonssikkerhet ble innført i Norge i 2004. I dag gjelder Eurokode 8 med nasjonalt tillegg som regelverk. Eurokodene angir generelt nasjonale tillegg som informative. I forordene til det norske nasjonale tillegget for Eurokode 8 angis imidlertid: «Dette nasjonale tillegget fastsetter de bestemmelser som skal anvendes ved bruken av NS-EN 1998-1:2004 for prosjektering av konstruksjoner i Norge» *(Red.anm.: Med endringsblad*). På bakgrunn av dette har reglene i Eurokode 8 gyldighet som ytelseskrav iht. TEK.

3 Dimensjonerende seismisk situasjon

Grunnlag for prosjektering av konstruksjonssikkerhet er gitt i NS-EN 1990 som gir krav til påvisning av motstand mot *dimensjonerende seismisk situasjon*, med henvisning til Eurokode 8.
I punkt NA.2.1(1)P angir Eurokode 8 at krav ved påvisning av motstand mot sammenbrudd settes referansesannsynlighet for overskridelse, P_{NCR} og referansereturperiode for den seismiske referanselasten, T_{NCR} lik hhv. 10% og 475 år. Spissverdi for berggrunnens akselerasjon, a_{g40Hz} som benyttes i bestemmelse av seismisk referanselast, finnes i dag for ulike deler av landet ut fra sonekart gitt i Eurokode 8 punkt NA.3.2.1(1), Figur NA.3(901) og figur NA.3(902). NORSARs virksomhet og kompetanse omfatter blant annet både løpende registrering av jordskjelv, samt anvendelse av statistiske metoder for å bestemme grunnlaget for det som kan benevnes som *karakteristisk seismisk situasjon* (475-årshendelse). I Norge er det derfor NORSAR som i realiteten har ansvaret for å bestemme spissverdier for berggrunnens akselerasjon.
Når nye måledata stadig registreres, danner disse et grunnlag for å kunne oppdatere det statistiske grunnlaget for å bestemme 475-års spissverdier for berggrunnens akselerasjon. Slik oppdatering er tidvis også aktuell for andre naturlaster, der statistikkgrunnlaget tilføres nye data, som for vind og snø.

4 Terskelsituasjoner i NS EN 1998-1-1

Følgende terskelsituasjoner er særlig relevante:

- Hvorvidt byggverket kommer under utelatelseskriteriet (se kapittel 6)
- Hvorvidt byggverkets konstruksjonskonsept i vesentlig grad må tilpasses påvirkning i seismisk situasjon

Det sistnevnte punktet kan gjelde for eksempel sykehus og viktig infrastruktur, der det skal dimensjoneres med sikte på særlig høy sikkerhet. Da faktoriseres 475-årsverdiene opp, slik at returperioden øker vesentlig.

Her kan det bli aktuelt å bruke beregning med DCM-metoden (middels duktilitet) i stedet for DCL-metoden (lav duktilitet), det vil si at en tar hensyn til konstruksjonens evne til å absorbere energi. Dette bevirker økt bruk av rammevirkning som igjen kan påvirke bjelkehøyder mv. Slik kan tilgjengelige volumer for tekniske føringer mv bli påvirket, noe som igjen kan ha stor tverrfaglig betydning (arkitektur/tekniske føringer).

For øvrig er de viktigste utgangspunktene:

- Geografisk sted, koordinater
- Grunnforhold og fundamenteringsmåte

Dette eksemplifiseres i det følgende.

5 Overordnet økonomisk vurdering

Det kan forventes at de økonomiske følgene av endringer i grunnlaget for dimensjonering mot jordskjelv ikke er veldig store med hensyn til byggekostnader sett under ett i Norge. Andel av byggekostnader som er knyttet til jordskjelvsikring er begrenset i forhold til de totale byggekostnadene i et prosjekt. Videre er Norge er et lavseismisk land og omfanget av byggeprosjekter der seismikk har stor betydning, er relativt begrenset. Imidlertid, for det enkelte prosjekt, kan kostnadseffekten være signifikant for

- Grunn og fundamenter (Kapittel 21 i Bygningsdelstabellen – NS 3451)
- Bæresystem (Kapittel 22 i Bygningsdelstabellen- NS 3451)
- Bærende vegger (Kapittel 231 i Bygningsdelstabellen- NS 3451)
- Bærende yttervegger (Kapittel 241 i Bygningsdelstabellen- NS 3451)
- Dekker og påstøper (Kapittel 251 & 253 i Bygningsdelstabellen- NS 3451)
- Prosjektering (Under post 8 i Bygningsdelstabellen- NS 3451)

Generelt og prinsipielt er det derfor av stor betydning å oppdatere dimensjoneringsgrunnlaget når data for dette foreligger. Som det fremgår av denne rapporten kan en for enkelteksempler finne signifikante besparelser ved å redusere dimensjonerende verdier for seismisk situasjon.

6 Utelatelseskriterier

EK 8 gir anledning til å utelate påvisning av tilstrekkelig sikkerhet for seismisk påkjenning fordi Norge er ett lavseismisk område. Det er i alt 4 kriterier. Hvis ett av dem tilfredsstilles, kan dimensjonering for seismisk påkjenning utelates.

6.1 Konstruksjonstype

EK 8 punkt NA.3.2.1(5)P angir at det ikke kreves påvisning for tilstrekkelig sikkerhet etter EK 8 for:

1) Konstruksjoner i seismisk klasse 1

Veiledende tabell for valg av seismisk klasse for byggverk er gitt i EK 8 tabell NA.4(902).

6.2 Svært lav seismisitet

I de tilfeller med svært lav seismisitet er det ikke nødvending å påvise tilstrekkelig sikkerhet for seismisk påkjenning. Dette kriterium er gitt i EK 8 punkt NA.3.2.1 (5) P. Svært lav seismisitet er definert som:

$$2)\ a_g \cdot S = \gamma_I \cdot (0{,}8 \cdot a_{g40Hz}) \cdot S < 0{,}49 m/s^2$$

γ_I er faktor for seismisk klasse.
a_{g40Hz} er spissverdi for berggrunnens akselerasjon.
S er forsterkningsfaktor for grunnforholdene.

6.3 Dimensjonerende spektrum

Dersom dimensjonerende spektrum $S_d(T)$ beregnet med følgende betingelser

- Konstruksjonsfaktor q≤1,5 DCL
- Ingen reduksjon av stivhetsegenskaper etter EK 8 punkt 4.3.1 (7)
- Konstruksjonen regnet som fast innspent i grunnen

Oppfylles følgende krav er ikke påvisning av tilstrekkelig sikkerhet for seismisk påkjenning nødvendig.

$$3)\ S_d(T) < 0{,}49 m/s^2$$

6.4 Størrelse på krefter

EK 8 punkt 4.4.1 (2) angir at for bygninger i andre seismiske klasser enn IV er det ikke nødvendig å påvise tilstrekkelig sikkerhet for seismisk påkjenning hvis skjærkraften på grunnivå eller ved overkant stiv kjeller er mindre enn fra andre relevante lastkombinasjoner.

$$4)\ 1{,}0 \cdot F_b < (1.5 \cdot \text{Vind} + 1{,}05 \cdot \text{Skjev}) \cdot (\gamma_{cburuddgrense} / \gamma_{cDCL})$$

F_b er horisontalkraft på grunn av jordskjelv.
Vind er kraft fra vindpåkjenning.
Skjev er skjevstillingslaster.
$\gamma_{cburuddgrense}/\gamma_{cDCL}$ er forholdet mellom materialfaktorer i ordinær bruddgrensetilstand og seismisk dimensjonering.

Følgende krav må også være oppfylt.

- Kraften må være basert på lineær elastisk analyse av bygget.
- Konstruksjonsfaktor q≤1,5 DCL
- Bygget må tilfredsstille krav til enkelhet i plan og oppriss.
- Den lineær elastiske analysemodellen skal være representativ for bygget.

6.5 Vurdering av kriteriene mot lavere grunnakselerasjon.

6.5.1 Innledning

Kriteriene 3) dimensjonerende spektrum og 4) størrelse på krefter tar hensyn til byggets utforming og oppførsel under seismisk belastning. Det er derfor vanskelig å si noe hvilke type bygg som kan tilfredsstille utelatelseskriteriene ved en lavere dimensjonerende grunnakselerasjon annet enn at det er større sannsynlighet for at de kan tilfredsstilles. Kriterium 1 er basert på hvilket type bygg en har og konsekvenser av sammenbrudd ved en seismisk hendelse. Kriterium 1 er således uavhengig av dimensjonerende grunnakselerasjon. Kriterium 2 svært lav seismisitet er påvirket av grunnakselerasjon, grunntype og seismisk klasse. Det er gjort en sammenligning for byggverk i Oslo-, Bergen-, Trondheim- og Tromsø-området. Her ser en på hvilke seismisk klasser og grunntyper som tilfredsstiller utelatelseskriteriet ved å benytte seismisk sonekart i EK 8 og ved reduserte verdier iht. NORSAR av grunnakselerasjonen i de oppgitte områder.

6.5.2 Dimensjonerende grunnakselerasjon

Spissverdi for berggrunnens akselerasjon a_{g40Hz} er i EK 8 gitt i punkt NA.3.2.1 med en returperiode på 475år. Referanseverdien for berggrunnens akselerasjon a_{gR} settes lik $0{,}8 \cdot a_{g40Hz}$.

For maksimumsområder benyttes det ett konstant tillegg på $0{,}05 m/s^2$ på a_{g40Hz}. Mellom kurvene vist i Figur NA.3(901) og Figur NA.3(901) i EK 8, kan det interpoleres. Tabellen nedenfor viser referanseverdien for berggrunnens akselerasjon etter sonekart i EK 8 og en redusert verdi for en punktmåling i samme område gitt av NORSAR.

Tabell 1 Sammenligning av referanseverdi for grunnakselerasjon etter sonekart i EK 8 og redusert verdi for en punktmåling i samme område iht. NORSAR.

Område	EK 8 a_{gR} $[m/s^2]$	Redusert verdi a_{gR} $[m/s^2]$	Endring
Oslo	0,44	0,29	-35 %
Bergen	0,72	0,50	-31 %
Trondheim	0,30	0,07	-76 %
Tromsø	0,28	0,10	-64 %

Dimensjonerende grunnakselerasjon for grunntype A er gitt som: $a_g = \gamma_I \cdot (0{,}8 \cdot a_{g40Hz})$ hvor faktor for seismisk klasse γ_I er gitt i tabell EK 8 NA.4(901). Tabell 2 og 3 viser dimensjonerende grunnakselerasjon for grunntype A etter henholdsvis EK 8 og reduserte verdier iht. NORSAR for ulike seismiske klasser for omtalte steder.

EK 8

Tabell 2 Dimensjonerende grunnakselerasjon for grunntype A etter EK 8 for ulike seismiske klasser for de omtalte stedene

		Oslo	Bergen	Trondheim	Tromsø
Seismisk klasse	γ_I	a_g [m/s²]	a_g [m/s²]	a_g [m/s²]	a_g [m/s²]
I	0,70	0,31	0,50	0,21	0,20
II	1,00	0,44	0,72	0,30	0,28
III	1,40	0,62	1,01	0,42	0,39
IV	2,00	0,88	1,44	0,60	0,56

Redusert verdi iht. NORSAR

Tabell 3 Dimensjonerende grunnakselerasjon for grunntype A etter reduserte verdier iht. NORSAR for ulike seismiske klasser for et målepunkt i de omtalte områdene

		Oslo	Bergen	Trondheim	Tromsø
Seismisk klasse	γ_I	a_g [m/s²]	a_g [m/s²]	a_g [m/s²]	a_g [m/s²]
I	0,70	0,20	0,35	0,05	0,07
II	1,00	0,29	0,50	0,07	0,10
III	1,40	0,41	0,70	0,10	0,14
IV	2,00	0,58	1,00	0,14	0,20

6.5.3 Forsterkningsfaktor for grunnforhold

Forsterkningsfaktor S er gitt i EK 8 tabell NA.3.3 for grunntype A-E. Forsterkningsfaktor for andre grunntyper krever geotekniske vurdering og vurderes ikke her.

6.5.4 Svært lav seismisitet etter EK 8

Tabell 4 - 7 viser verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser basert på sonekart i EK 8. For Oslo er det kun bygg i seismisk klasse II fundamentert på grunntype A som tilfredsstiller utelatelseskriterium 2. For Bergen så tilfredsstiller ingen bygg kriteriet, mens det for Trondheim er tilfredsstilt for grunntype A-D i seismisk klasse II og ved grunntype A i seismisk klasse III. For Tromsø så er kriteriet tilfredsstilt for alle grunntyper i seismisk klasse II og grunntype A i seismisk klasse III. Ingen bygg og grunntyper tilfredsstiller kriteriet i seismisk klasse IV for noen av stedene.

Oslo

Tabell 4 Verdier av produktet a_g·S for ulike grunntyper og seismiske klasser i Oslo basert på sonekart i EK 8. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	Seismisk klasse II a_gS [m/s²]	Seismisk klasse III a_gS [m/s²]	Seismisk klasse IV a_gS [m/s²]
A	0,44	0,62	0,88
B	0,57	0,80	1,14
C	0,62	0,86	1,23
D	0,68	0,95	1,36
E	0,73	1,02	1,45

Bergen

Tabell 5 Verdier av produktet a_g·S for ulike grunntyper og seismiske klasser i Bergen basert på sonekart i EK 8. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	Seismisk klasse II a_gS [m/s²]	Seismisk klasse III a_gS [m/s²]	Seismisk klasse IV a_gS [m/s²]
A	0,72	1,01	1,44
B	0,94	1,31	1,87
C	1,01	1,41	2,02
D	1,12	1,57	2,23
E	1,19	1,67	2,38

Trondheim

Tabell 6 Verdier av produktet a_g·S for ulike grunntyper og seismiske klasser i Trondheim basert på sonekart i EK 8. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	Seismisk klasse II a_gS [m/s²]	Seismisk klasse III a_gS [m/s²]	Seismisk klasse IV a_gS [m/s²]
A	0,30	0,42	0,60
B	0,39	0,55	0,78
C	0,42	0,59	0,84
D	0,47	0,65	0,93
E	0,50	0,69	0,99

Tromsø

Tabell 7 Verdier av produktet a_g·S for ulike grunntyper og seismiske klasser i Tromsø basert på sonekart i EK 8. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	Seismisk klasse II a_gS [m/s²]	Seismisk klasse III a_gS [m/s²]	Seismisk klasse IV a_gS [m/s²]
A	0,28	0,39	0,56
B	0,36	0,51	0,73
C	0,39	0,55	0,78
D	0,43	0,60	0,87
E	0,46	0,64	0,92

6.5.5 Konsekvenser av reduserte verdier av spissverdi for berggrunnens akselerasjon iht. NORSAR

Tabell 8-11 viser verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser basert på oppdaterte NORSAR-verdier for berggrunnens spissakselerasjon.

Målepunkt i Oslo-området

Tabell 8 Verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser for et målepunkt i Oslo-området med reduserte grunnakselerasjoner. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	**Seismisk klasse II** a_gS [m/s^2]	**Seismisk klasse III** a_gS [m/s^2]	**Seismisk klasse IV** a_gS [m/s^2]
A	0,29	0,41	0,58
B	0,38	0,53	0,75
C	0,41	0,57	0,81
D	0,45	0,64	0,90
E	0,48	0,68	0,96

Målepunkt i Bergensområdet

Tabell 9 Verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser for et målepunkt i Bergensområdet med reduserte grunnakselerasjoner. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	**Seismisk klasse II** a_gS [m/s^2]	**Seismisk klasse III** a_gS [m/s^2]	**Seismisk klasse IV** a_gS [m/s^2]
A	0,50	0,70	1,00
B	0,65	0,91	1,30
C	0,70	0,98	1,40
D	0,78	1,09	1,55
E	0,83	1,16	1,65

Målepunkt for Trondheimsområdet

Tabell 10 Verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser for et målepunkt i Trondheimsområdet med reduserte grunnakselerasjoner. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	**Seismisk klasse II** a_gS [m/s^2]	**Seismisk klasse III** a_gS [m/s^2]	**Seismisk klasse IV** a_gS [m/s^2]
A	0,07	0,10	0,14
B	0,09	0,13	0,18
C	0,10	0,14	0,20
D	0,11	0,16	0,22
E	0,12	0,17	0,23

Målepunkt for Tromsø-området

Tabell 11 Verdier av produktet $a_g \cdot S$ for ulike grunntyper og seismiske klasser for et målepunkt i Tromsø-området med reduserte grunnakselerasjoner. Rød verdi viser hvor utelatelseskriterium 2 ikke tilfredsstilles.

Grunntype	Seismisk klasse II a_gS [m/s²]	Seismisk klasse III a_gS [m/s²]	Seismisk klasse IV a_gS [m/s²]
A	0,10	0,14	0,20
B	0,13	0,18	0,26
C	0,14	0,20	0,28
D	0,16	0,22	0,31
E	0,17	0,23	0,33

Sammenlignes tabell 4-7 (der EK 8 er benyttet) henholdsvis med tabell 8-11 (der oppdaterte NORSAR verdier er benyttet), ser en at utslagene er betydelige. Vesentlig flere byggverk vil tilfredsstille utelatelseskriterium og vil dermed ikke ha krav om å dokumentere tilstrekkelig sikkerhet mot seismisk belastning.

For målepunktet i Trondheims- og Tromsø-området vil det ikke være krav om å dokumentere tilstrekkelig sikkerhet mot seismisk belastning for noen byggverk fundamentert på grunntypene A til E. For målepunktet i Oslo-området slipper alle byggverk i seismisk klasse II og byggverk på grunntype A i seismisk klasse III, mens det for målepunktet i Bergensområdet ikke vil være noe endring i forhold til utelatelseskriteriet svært lav seismisitet ved reduserte verdier av akselerasjon.

Forskjellene for seismisk klasse II-IV kan oppsummeres slik:

Tabell 12 Oppsummering av hvor dimensjonering for seismisk påkjenning kan utelates ved bruk av referanseverdi for grunnakselerasjon fra EK 8 og oppdaterte verdier fra NORSAR

	Gjeldende EK 8/NA	Reduserte verdier NORSAR
Oslo-området	Utelatelse kun for seismisk klasse II /Grunntype A	**Alle i seismisk klasse II får utelatelse samt seismisk klasse III/Grunntype A**
Bergen-området	Ingen utelatelse	Ingen utelatelse
Trondheim-området	Utelatelse for 5 tilfeller iht. tabell 6	**Utelatelse for samtlige tilfeller**
Tromsø-området	Utelatelse for 6 tilfeller iht. tabell 7	**Utelatelse for samtlige tilfeller**

7 Eksemplifisering

7.1 Illustrasjonseksempler nybygg

7.1.1 Eksempelbygg - Sykehus

Sykehus med 8 etasjer over terreng. Fundamentert på borede peler. Opptak av horisontalkrefter med passivt jordtrykk mot kjeller og fundamenter. Samvirke mellom jord og konstruksjon er hensyntatt i den seismiske analysen.

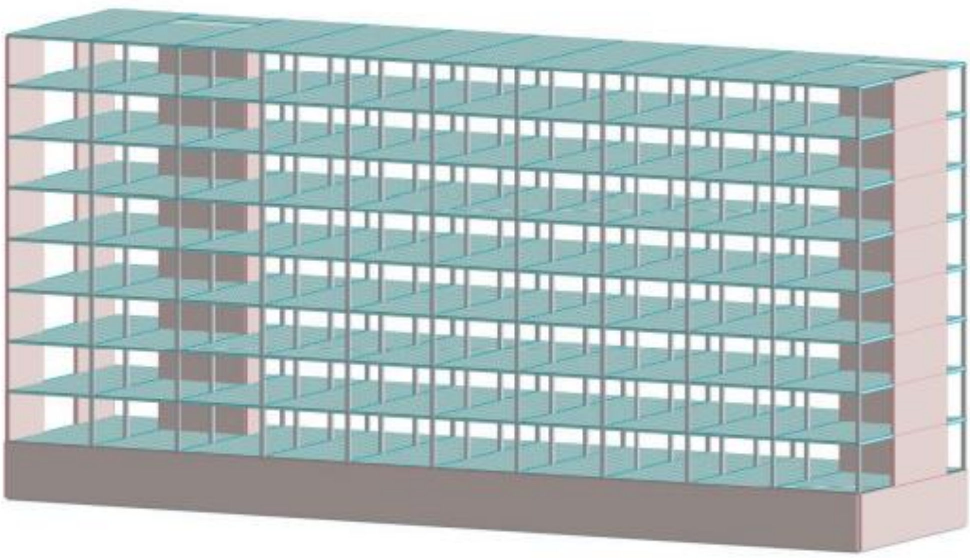
Etasjeskiller i prefabrikkerte hulldekker opplagt på opplagt på hattebjelker i stål. Vertikalt avstivende skiver i plasstøpt betong. Grunnflate BxL = 84,0mx20,4m

Seismiske hovedparametere

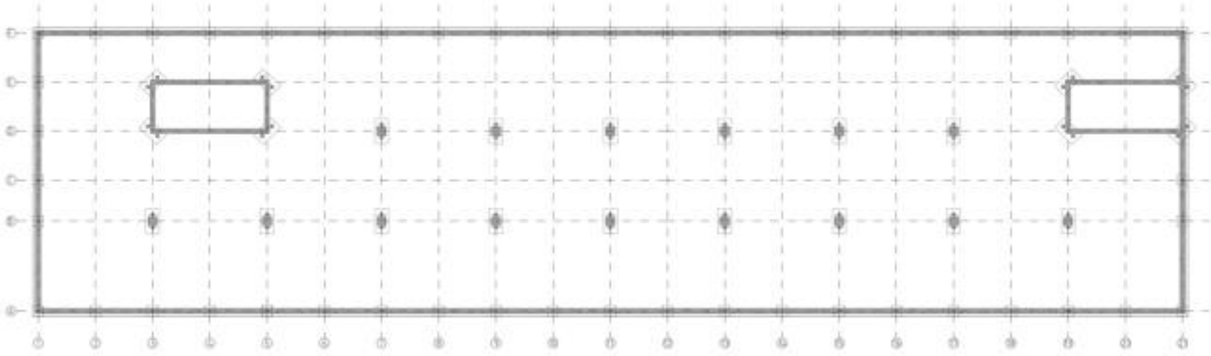
Lav duktilitet DCL				q=1,5
Grunntype E	S=1,65	T_B = 0,10s	T_C = 0,30s	T_D = 1,40s
Seismisk klasse IV				γ_I = 2,0
Beliggende på Vestlandet			a_{g40Hz} = 0,90m/s^2	a_{gR} = 0,72m/s^2
Dimensjonerende grunnakselerasjon etter NS-EN 1998				a_g=1,44/s^2
Dimensjonerende grunnakselerasjon redusert 30%				a_g=1,00m/s^2

Byggets utforming

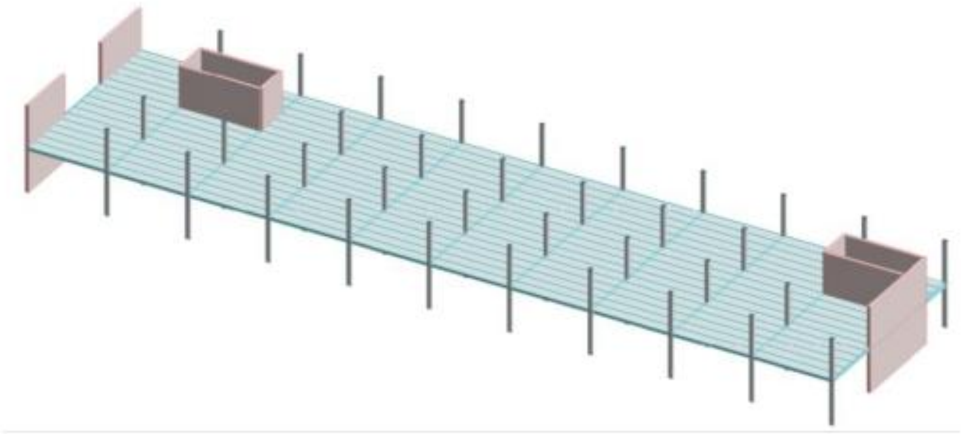
De følgende figurer viser byggets geometri for hovedbæresystemet.



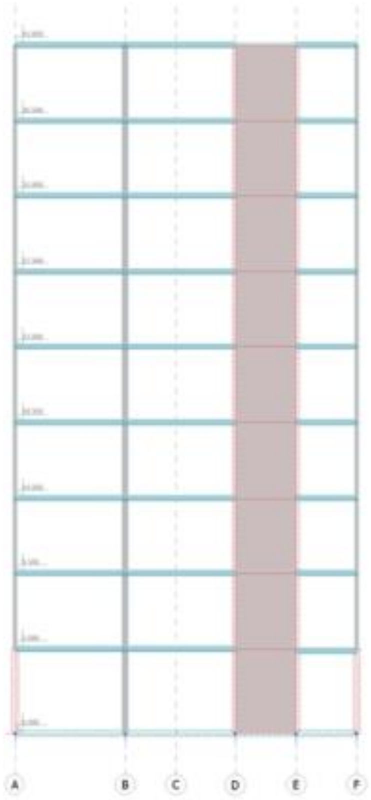
Figur 1 Modell av hele bygget i FEM-Design



Figur 2 Fundamentplan for sykehus-eksempel



Figur 3 Plansnitt for sykehus-eksempel



Figur 4 Snitt akse 5 i sykehuseksempel

Sjekk mot utelatelseskriterier

Kontroll om bygget tilfredsstiller utelatelseskriterier for begge akselerasjoner.

- *Konstruksjonstype*

Bygget er i seismisk klasse IV dermed er ikke kriterium tilfredsstilt.

- *Svært lav seismisitet*

Krav:

$$a_g \cdot S = \gamma_I.(0.8 \cdot a_{g40Hz}) \cdot S < 0.49 m/s^2$$

$a_g = 1,44$ m/s^2 gir $a_g \cdot S = 2,38$ m/s^2
$a_g = 1,00$ m/s^2 gir $a_g \cdot S = 1,65$ m/s^2

Kriterium er ikke tilfredsstilt.

- *Dimensjonerende responsspektrum*

Krav $S_d(T) < 0,49 m/s^2$. Egenperioder beregnet med konstruksjonen fast innspent mot grunnen.

Tabell 13 Dimensjonerende responsspektrum, $S_d(T)$, gitt av EK 8 punkt 3.2.2.5 (4)P med egenperioder funnet i FEM-Design

		a_g = 1,44m/s^2	**a_g = 1,00m/s^2**
Mode nr	T [s]	S_d(T) [m/s^2]	S_d(T) [m/s^2]
1	1,318	1,32	1,14
2	1,120	1,93	1,34
3	0,699	3,09	2,15
4	0,371	4,80	3,33
6	0,231	4,80	3,33

Tabell viser at kriterium ikke er tilfredsstilt for noen av periodene.

- *Størrelse på krefter*

Bygget er i seismisk klasse IV og dette kriterium gjelder ikke for bygg i seismisk klasse IV.

Seismisk analyse

Svingeformer og tilhørende svingende masse

Tabell 14 Effektiv modal masse i x- og y-retning

Shape no.	**T**	**mx'**	**my'**
[-]	[s]	[%]	[%]
1	1,931	1,70	**60,00**
2	1,685	**56,00**	1,40
4	0,412	0,00	**14,10**
5	0,277	**17,20**	0,00
14	0,148	**0,90**	**4,70**
15	0,142	**9,00**	0,40
	Σ =	**82,20**	**78,80**

Baseskjær og krefter etasjevis

Tabell 15 Ekvivalente horisontalkrefter fra seismisk analyse i byggets etasjer

	$a_g = 1{,}44 m/s^2$			$a_g = 1{,}00 m/s^2$		
	Fx'	Fy'	Fr	Fx'	Fy'	Fr
	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]
Base	29967	23652	38176	18388	14311	23301
Terreng	5128	3568	6247	3203	1313	3461
Etasje 1	7421	4754	8813	4547	2896	5391
Etasje 2	7286	6701	9899	4388	3973	5920
Etasje 3	6858	6696	9584	4225	4078	5872
Etasje 4	6669	5587	8700	4214	3338	5376
Etasje 5	5456	3348	6401	2684	1704	3179
Etasje 6	3630	2616	4474	1982	1561	2523
Etasje 7	5874	5411	7986	3110	2796	4182
Etasje 8	6518	5339	8426	4119	3429	5360

Konsekvenser for hovedbæresystem

Det er gjort en vurdering av hvilke konsekvenser en reduksjon på 30% av dimensjonerende grunnakselerasjon har på byggets hovedbæresystem som fundamenter, vertikale skiver og horisontale skiver.

Fundamentering og kjeller

Opptak av horisontalkrefter fra seismisk belastning gjøres med passivt jordtrykk mot kjellervegger og pelehoder. Det er antatt at deformasjonene er så små at den generelle pelekapasiteten ikke er reduseres betydelig. Det er det vurdert at det er tilstrekkelig kapasitet for opptak av horisontalkrefter og slik at hovedforskjellen mellom de ulike akselerasjoner er det passive jordtrykkets størrelse på kjellervegger og lasten i peler under vertikale skiver.

Det er gjort en vurdering av hva de ulike akselerasjonene har si for dimensjoner og armeringsmengder til yttervegger i kjeller og for dimensjoner av peler.

Nødvendig tykkelse på kjellervegger:
Dimensjonerende grunnakselerasjon $a_g = 1{,}44 m/s^2$ t=600mm
Dimensjonerende grunnakselerasjon $a_g = 1{,}00 m/s^2$ t=400mm

Armeringsmengde:
Nødvendig beregningsmessig armeringsintensitet (kg armering pr kubikk betong) for kjellervegger:
Dimensjonerende grunnakselerasjon $a_g = 1{,}44 m/s^2$ $\rho = 225 kg/m^3$
Dimensjonerende grunnakselerasjon $a_g = 1{,}00 m/s^2$ $\rho = 195 kg/m^3$

Peler:
Behovet for strekkpeler øker betydelig ved den største grunnakselerasjonen, tverrsnittet må i tillegg økes på en del av pelene. Fordeling av peler er vist i nedenstående tabell.

Tabell 16 Estimat av antall og type peler som er nødvendig for sykehus-eksempelbygget ved bruk av grunnakselerasjon fra EK 8 og 30 % redusert verdi.

		Antall	
Dimensjon	Type	a_g = 1,44m/s^2	a_g = 1,00m/s^2
Ø180	Trykk	40	30
Ø180	Strekk	16	0
Ø150	Trykk	38	62
Ø150	Strekk	0	2

<u>Vertikale skiver</u>

Vertikale skiver er plasstøpte betongvegger fra kjeller til topp. Det er gjort en beregning av nødvendig armeringsmengder og dimensjoner for avstivende vegger fra kjeller og opp for de to akselerasjoner.

Nødvendig tykkelse på vegger:
Vises i nedforstående tabell for de to akselerasjoner.

Tabell 17 Estimat av nødvendig veggtykkelse for sykehus-eksempelbygg ved bruk av grunnakselerasjon fra EK 8 og 30 % redusert verdi. Oransje farge markerer en økning i veggtykkelse

	a_g = 1,44m/s^2	a_g = 1,00m/s^2
Kjeller	400 mm	300 mm
Etasje 1	400 mm	300 mm
Etasje 2	350 mm	300 mm
Etasje 3	350 mm	300 mm
Etasje 4	350 mm	300 mm
Etasje 5	350 mm	300 mm
Etasje 6	350 mm	300 mm
Etasje 7	350 mm	300 mm
Etasje 8	350 mm	300 mm

Armeringsmengde:

Nødvendig beregningsmessig armeringsintensitet (kg armering pr kubikk betong) for vertikale skiver:
Dimensjonerende grunnakselerasjon a_g = 1,44m/s^2 ρ=155kg/m^3
Dimensjonerende grunnakselerasjon a_g = 1,00m/s^2 ρ=115kg/m^3

Horisontale skiver

De horisontale skiver er dekkene består prefabrikkerte hulldekker. NS-EN 1992 -1-1:2004+NA:2008 punkt 10.9.3 (12) gir en begrensing på tillatt skjærspenning for utstøpte fuger mellom prefabrikkert elementer. Grensen er satt til 0,15MPa, men ved å ta hensyn til forhold mellom materialfaktorer i seismisk og bruddgrense kan grensen økes til 0,19MPa.
Ved en overskridelse av denne grensen over større områder benyttes en konstruktiv påstøp for å oppta disse krefter. Konstruktiv påstøp gir større vertikale laster som igjen har en påvirkning på bæresystemet. Det er gjort en vurdering om kapasitet for skjærspenning på 0,19MPa overskrides ved seismisk belastning ved de ulike akselerasjoner. Ved overskridelse benyttes en konstruktiv påstøp på 120mm hvis ikke så benyttes en oppbygning over etasjeskiller av trykkfast ubrennbar isolasjon og tynnavrettning. Tabell viser hvilken oppbygning som er nødvendig for hvert plan for de to akselerasjoner.

Tabell 18 Oppbygning på prefabrikkerte dekke for sykehus-eksempelbygg ved bruk av grunnakselerasjon fra EK 8 og 30 % redusert verdi. Oransje farge markerer en hvor konstruktiv påstøp må benyttes.

	a_g = 1,44m/s^2	a_g = 1,00m/s^2
Terreng	Konstruktiv påstøp	Konstruktiv påstøp
Etasje 1	Konstruktiv påstøp	Konstruktiv påstøp
Etasje 2	Konstruktiv påstøp	Konstruktiv påstøp
Etasje 3	Konstruktiv påstøp	Konstruktiv påstøp
Etasje 4	Konstruktiv påstøp	Konstruktiv påstøp
Etasje 5	Konstruktiv påstøp	Tynnavrettning og isolasjon
Etasje 6	Konstruktiv påstøp	Tynnavrettning og isolasjon
Etasje 7	Konstruktiv påstøp	Tynnavrettning og isolasjon
Etasje 8	Konstruktiv påstøp	Konstruktiv påstøp

Kostnader

Det er gjort en vurdering av hvilke kostnader de to grunnakselerasjoner har av betydning for hovedbæresystem for de elementer som angitt ovenfor. Det er forutsatt stålkjernepeler med en snittlengde på 22m og 24m for hhv. trykk- og strekkpeler.

Tabell 19 Estimat av kostnadsbesparelse ved bruk av redusert grunnakselerasjon mot grunnakselerasjon gitt av EK 8

	Dagens grunnakselerasjon:		Ny grunnakselerasjon:		Besparelse:	
	Sum (kr)	kr/m2	Sum (kr)	kr/m2	Sum (kr)	kr/m2
Pelefundamentering	10 284 828	667	9 016 315	585	1 268 513	82
Yttervegger under mark	5 993 057	389	4 239 446	275	1 753 611	114
Øvrige bærende vegger	10 167 636	659	7 907 355	513	2 260 281	147
Oppbygning på prefabrikerte dekker	8 889 219	576	7 838 740	508	1 050 479	68
Sum konto 02 bygning	35 334 740	2 291	29 001 856	1 881	6 332 884	411

Tabell viser kostnader med dagens grunnakselerasjon og ved ny grunnakselerasjon for eksempelprosjektet. Besparelse for prosjektet er beregnet å utgjøre ca. 6,3MNOK (411kr/m^2 bruttoareal). Kalkulasjon er utført i ISY-Calcus med prisregister (Norsk Prisbok) for august 2019.

7.1.2 Eksempelbygg – Kontorbygg

Kontorbygg med 8 etasjer uten kjeller. Fundamenter på peler. Forutsetter at arbeidsdekket har tilstrekkelig tykkelse og at forbindelse til pelehode utformes slik at opptak av horisontalkrefter kan gjøres ved friksjon mellom arbeidsdekket og grunn.

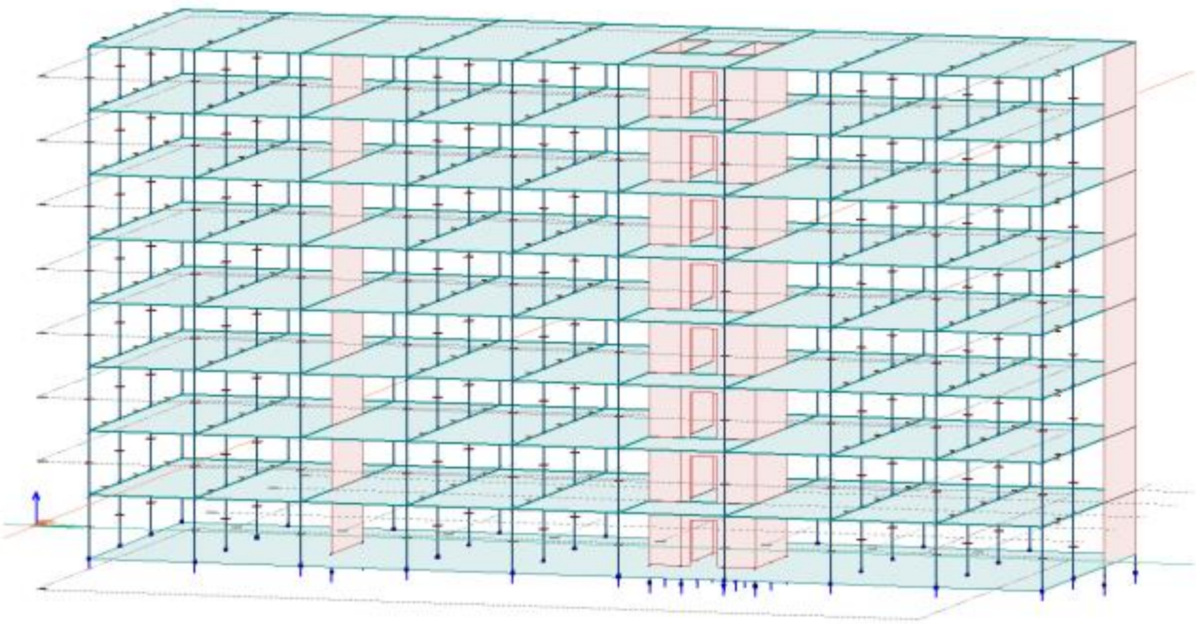
Etasjeskiller i prefabrikkerte hulldekker opplagt på hattebjelker i stål. Vertikalt avstivende skiver i plasstøpt betong. Grunnflate BxL = 54,0mx18m

Seismiske hovedparametere

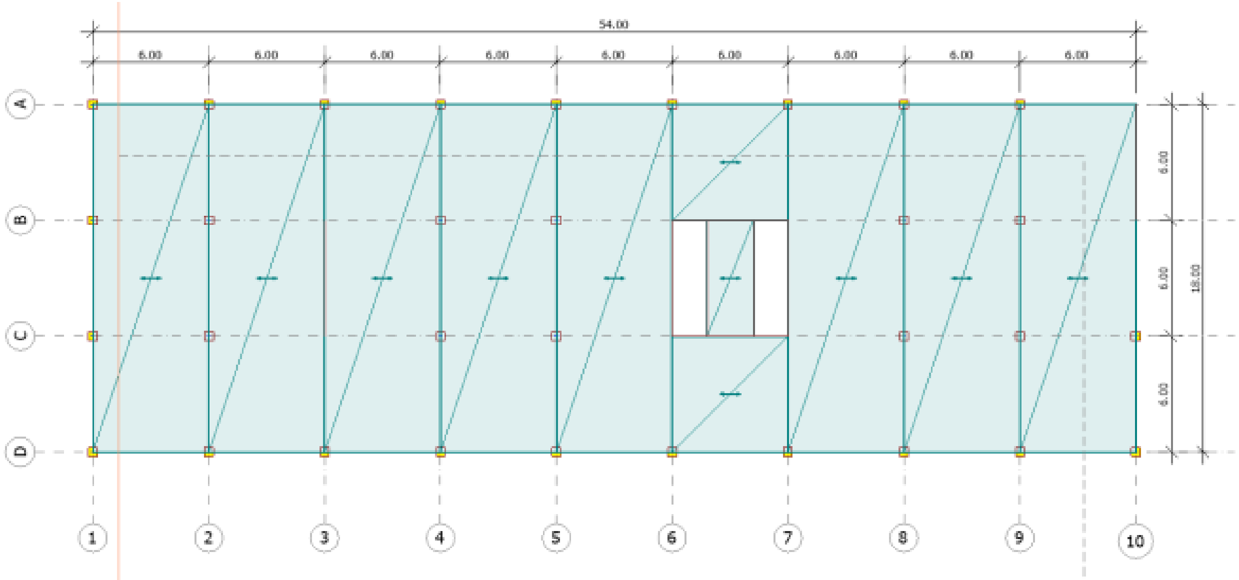
Lav duktilitet DCL				q=1,5
Grunntype C	S=1,4	T_B = 0,10s	T_C = 0,30s	T_D = 1,5s
Seismisk klasse 2				γ_I = 1,0
Beliggende i Oslo-området		a_{g40Hz} = 0,55m/s^2		a_{gR} = 0,44m/s^2
Dimensjonerende grunnakselerasjon etter NS-EN 1998				a_g=0,44m/s^2
Dimensjonerende grunnakselerasjon redusert 35%				a_g=0,29m/s^2

Byggets utforming

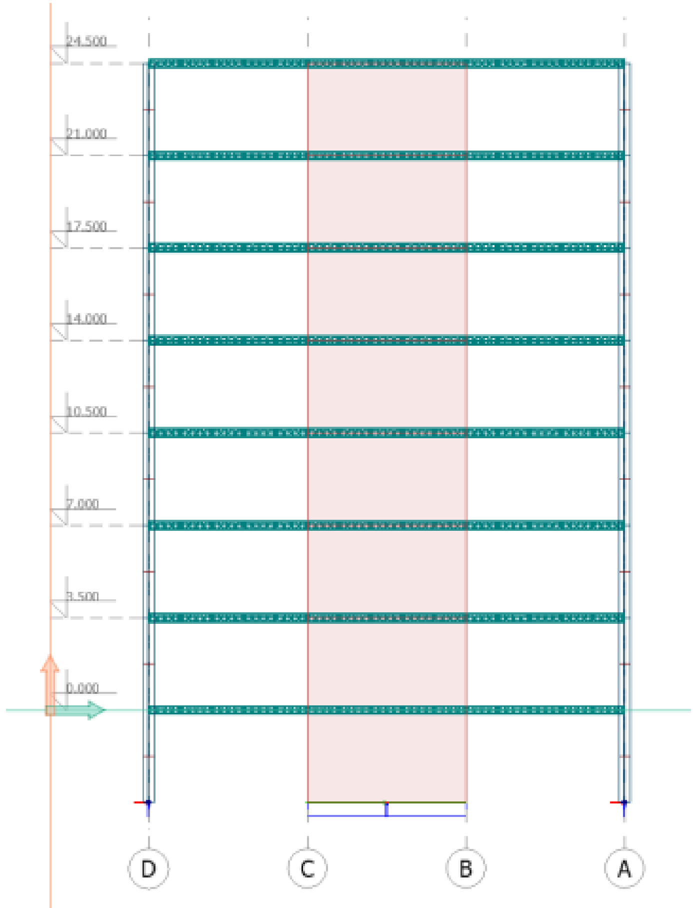
De følgende figurene viser byggets geometri for hovedbæresystemet.



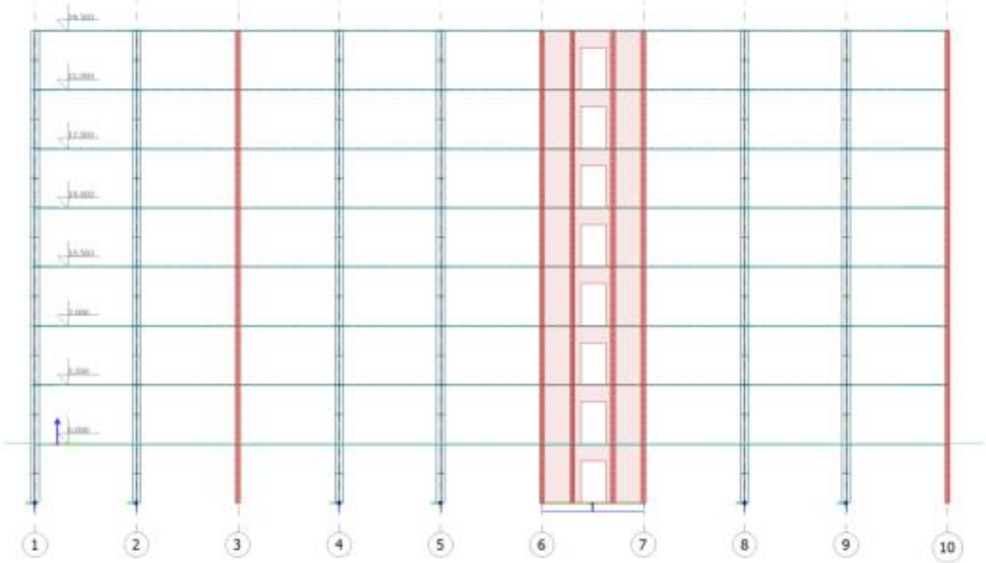
Figur 5 Modell av hele bygget i FEM-Design



Figur 6 Plansnitt for kontorbygg-eksempel



Figur 7 Snitt akse 3 av kontorbygg-eksempel



Figur 8 Snitt akse B av kontorbygg- eksempel

Seismisk analyse

Svingeformer og tilhørende svingende masse

Tabell 20 Effektiv modal masse i x- og y-retning

Shape no.	T	mx'	my'
[-]	[s]	[%]	[%]
1	1,142	0	**24,7**
2	1,084	**74,9**	0
3	0,744	0,1	**47**
4	0,257	**17**	0
5	0,248	0	**5,2**
6	0,146	0	**14,1**
7	0,133	**4,1**	0
8	0,121	0	**1,5**
∑=		**96,1**	**92,5**

Baseskjær og etasjevis krefter

Tabell 21 Ekvivalente horisontalkrefter fra seismisk analyse i byggets etasjer

		a_g = 0,44m/s²	
	Fx'	Fy'	Fr
	[kN]	[kN]	[kN]
Base	1642	1531	2244
Etasje 1	249	160	296
Etasje 2	413	284	501
Etasje 3	451	354	573
Etasje 4	421	354	550
Etasje 5	364	294	468
Etasje 6	277	234	363
Etasje 7	342	310	462
Etasje 8	499	428	658

Sjekk mot utelatelseskriterier

Kontroll om bygget tilfredsstiller utelatelseskriterier for begge akselerasjoner.

- *Konstruksjonstype*

Bygget er i seismisk klasse 2 dermed er ikke kriterium tilfredsstilt.

- *Svært lav seismisitet*

Krav:

$$a_g \cdot S = \gamma_l.(0.8 \cdot a_{g40Hz}) \cdot S < 0.49 m/s^2$$

a_g = 0,44 m/s^2 gir $a_g * S = 0{,}44 \frac{m}{s^2} * \gamma_l * 1{,}4 = 0{,}62 \frac{m}{s^2} > 0{,}49 \frac{m}{s^2}$
a_g = 0,29 m/s^2 gir $a_g * S = 0{,}29 \frac{m}{s^2} * \gamma_l * 1{,}4 = 0{,}40 \frac{m}{s^2} < 0{,}49 \frac{m}{s^2}$

Kriterium er ikke tilfredsstilt for grunnakselerasjon etter NS-EN 1998, men tilfredsstilt for ny redusert grunnakselerasjon. Vurderer grunnakselerasjon fra EK8 videre.

- *Dimensjonerende spektrum*

Krav $S_d(T) < 0{,}49 m/s^2$

Tabell 22 Dimensjonerende responsspektrum, Sd(T), gitt av EK 8 punkt 3.2.2.5 (4)P med egenperioder funnet i FEM-Design

Mode nr.	T [s]	$S_d(T)$ [m/s^2]
1	1,142	0,27
2	1,084	0,28
3	0,744	0,41
4	0,257	1,03
5	0,248	1,03
6	0,146	1,03

Tabellen viser at kriterium er ikke tilfredsstilt for 3 av periodene ved bruk av grunnakselerasjon etter NS-EN 1998.

- *Størrelse på krefter*

Krav i begge retninger:

$$1{,}0 \cdot F_b < (1.5 \cdot \text{Vind} + 1{,}05 \cdot \text{Skjev}) \cdot (\gamma_{cburuddgrense}/\gamma_{cDCL})$$

X-retning:

$$1642\ kN > (1{,}5 * 243kN + 1{,}05 * 172kN) * \frac{1{,}5}{1{,}2} = 682kN$$

Y-retning

$$1531\ kN < (1{,}5 * 817kN + 1{,}05 * 172kN) * \frac{1{,}5}{1{,}2} = 1757kN$$

Kriterium er ikke tilfredsstilt i x-retning for grunnakselerasjon etter NS-EN 1998, og det må påvises tilstrekkelig sikkerhet for seismisk påkjenning.

Konsekvenser for hovedbæresystem

Det er gjort en vurdering av hvilke konsekvenser det vil ha for hovedbæringssystemet ved å benytte grunnakselerasjon gitt av Eurokode 8 hvor det det må påvises tilstrekkelig sikkerhet for seismisk påkjenning, sammenlignet med redusert grunnakselerasjon hvor utelatelseskriteriet inntreffer.

Fundamentering
Opptak av horisontale krefter gjøres med friksjon under arbeidsdekket. Det er vurdert tilstrekkelig kapasitet for opptak av horisontalkrefter for begge tilfellene av grunnakselerasjon og at deformasjonen er så små at den generelle pelekapasiteten ikke er reduseres betydelig. Det antas dermed at hovedforskjellene mellom tilfellene er peler under vertikale skiver.

Peler
Ved å benytte grunnakselerasjon fra NS-EN 1998 må 12 stk. strekkpeler benyttes, i tillegg må dimensjonen av enkelte peler under avstivende elementer økes. Fordelingen av peler er vist tabellen nedenfor.

Tabell 23 Estimat av antall og type peler som er nødvendig for kontorbygg-eksempel ved bruk av grunnakselerasjon fra EK 8 og 35 % redusert verdi.

Dimensjon	Type	Antall	
		$a_g = 0{,}44 m/s^2$	$a_g = 0{,}29 m/s^2$
Ø180	Trykk	14	14
	Strekk	0	0
Ø130	Trykk	6	10
	Strekk	9	0
Ø100	Trykk	17	25
	Strekk	3	0

Vertikale skiver
Vertikale skiver er av plasstøpt betong. Det er gjort beregning av nødvendige armeringsmengder og dimensjoner for avstivende vegger.

Nødvendig tykkelse
Vises i nedforstående tabell for de to akselerasjoner.

Tabell 24 Estimat av nødvendig veggtykkelse for kontorbygg-eksempel ved bruk av grunnakselerasjon fra EK 8 og 30 % redusert verdi. Oransje farge markerer en økning i veggtykkelse

	$a_g = 0{,}44 m/s^2$	$a_g = 0{,}29 m/s^2$
Etasje 1	300 mm	200 mm
Etasje 2	200 mm	200 mm
Etasje 3	200 mm	200 mm
Etasje 4	200 mm	200 mm
Etasje 5	200 mm	200 mm
Etasje 6	200 mm	200 mm
Etasje 7	200 mm	200 mm
Etasje 8	200 mm	200 mm

Armeringsmengde:
Følgende er nødvendig armeringsintensitet (kg armering pr kubikk betong) for vertikal skiver for de to tilfellene:

Dimensjonerende grunnakselerasjon $a_g = 0{,}44 m/s^2$ ρ=117 kg/m^3
Dimensjonerende grunnakselerasjon $a_g = 0{,}29 m/s^2$ ρ=84 kg/m^3

Horisontale skiver

Dekkene består av prefabrikkerte hulldekker. Det er gjort en vurdering og funnet at det ikke er nødvendig med konstruktiv påstøp for de to alternative grunnakselerasjonene. Største tillatt skjærspenning for utstøpte fuger mellom prefabrikkerte elementer er satt til 0,19 MPa. ref. NS-EN 1992 -1-1:2004+NA:2008 punkt 10.9.3 (12). Det er da tatt hensyn til forhold mellom materialfaktorer i seismisk og bruddgrense.
Det benyttes derfor tynnavretting som oppbygning over etasjeskiller for begge grunnakselerasjoner.

Kostnader

Det er gjort en vurdering av hvilke kostnader de to grunnakselerasjoner har av betydning for hovedbæresystem for de elementer som angitt ovenfor. Det er forutsatt stålkjernepeler med en snittlengde på 22m og 24m for hhv. trykk- og strekkpeler.

Tabell 25 Estimat av kostnadsbesparelse ved bruk av redusert grunnakselerasjon mot grunnakselerasjon gitt av EK 8

	Dagens grunnakselerasjon:		Ny grunnakselerasjon:		Besparelse:	
	Sum (kr)	kr/m2	Sum (kr)	kr/m2	Sum (kr)	kr/m2
Pelefundamentering	4 544 587	599	4 101 650	541	442 937	58
Yttervegger over mark	413 704	55	380 727	50	32 977	4
Øvrige bærende vegger	2 727 637	360	2 509 843	331	217 794	29
Sum konto 02 bygning	7 685 928	1 013	6 992 220	922	693 708	92

Tabell viser kostnader med dagens grunn akselerasjon og ved ny grunnakselerasjon for eksempelprosjektet. Besparelse for prosjektet er beregnet å utgjøre ca. 700 000 kr (92kr/m2 bruttoareal). Kalkulasjon er utført i ISY-Calcus med prisregister (Norsk Prisbok) for august 2019.

7.2 Eksisterende bygg

PBL § 31.2 Tiltak på eksisterende byggverk angir at i utgangspunktet skal gjeldende regler anvendes for dokumentasjon av tiltakene. Dette betyr at TEK og eurokodene i utgangspunktet har gyldighet, med mulighet for visse unntak. Unntak med tanke på seismisk situasjon kan ikke generelt påberopes, en dokumentasjon opp mot Eurokode 8 må påregnes som krav. For eksempel eldre teglbygg, der en ønsker bruksendring og ombyggingstiltak i forbindelse med dette, vil her kunne gi utfordringer. En spesiell del av Eurokode 8 for eksisterende bygg er utgitt med tilpassede metoder, men fortsatt kan slike prosjekter få signifikante kostnadsutslag med tanke på prosjektering og fysiske løsninger. Her vil lavere spissverdier for berggrunnens akselerasjon i mange tilfelle få stor betydning ved at en vil tilfredsstille utelatelseskriteriet.

8 Analyse av økonomiske konsekvenser

8.1 Fundamentering

Andel av byggekostnader som fundamenteringsarbeider utgjør vil variere svært mye basert på eksempelvis grunnforhold og antall etasjer som disse kostnadene fordeles på.

Det er en viss korrelasjon mellom grunntyper/forsterkningsfaktorer (EK 8 tabell NA.3.3) og kostnader for fundamentering av ett bygg. Normalt vil kostnader per fundamenteringspunkt øke jo høyere forsterkningsfaktoren er. Dette vil si at det er nybygg som oppføres på lokaliteter med dårlige grunnforhold som vil ha størst besparelsespotensial ved å redusere spissverdi for berggrunnens akselerasjon.

Eksempel:

- Et bygg på en lokalitet med grunntype A/B vil typisk kunne fundamenteres til fjell eller på faste masser på en kostnadseffektiv måte. Da forsterkningsfaktoren er lavere, og kostnad for tilleggsfundamentering vil være beskjeden kan det antas at merkostnad grunnet jordskjelv er liten.
- Et bygg på en lokalitet med grunntype S_1/S_2 vil typisk måtte pelefundamenteres og ha betydelig høyere fundamenteringskostnad per fundamentpunkt. Da forsterkningsfaktoren er høyere, og kostnad for tilleggsfundamentering vil være høy kan det antas at merkostnad grunnet jordskjelv vil være betydelig.

8.2 Avstivende konstruksjoner og dekker

Som eksempelprosjektet viser, kan reduksjon i spissverdi for berggrunnens akselerasjon medføre reduksjon av dimensjon på og/eller omfanget av avstivende konstruksjonene. Reduksjon av dimensjon/omfang av avstivende skiver kan gi signifikante kostnadsbesparelser i enkelte prosjekter.

Overføring av horisontalkrefter i dekkekonstruksjoner er hovedsakelig en utfordring i prefabrikkerte dekker av typen referert til i eksempelprosjektet. Merkostnad for konstruktiv påstøp på hulldekkeelement vil avhengig av tykkelse utgjøre 200 - 400 kr/kvm. Alternativ oppbygning over hulldekker med eksempelvis behov for trinnlydsisolasjon (medtatt i eksempelprosjekt) vil dog gjøre besparelsen noe lavere enn dette.

Det kan anmerkes at kravene til dimensjonering mot jordskjelv i en del tilfelle har ført til økt robusthet. Robusthet er en egenskap som uansett må vurderes særskilt i det enkelte tiltak. Når det lempes på kravene mht. jordskjelv, kan det sies at robusthet bør få økt eksplisitt fokus.

8.3 Prosjektering

Prosjektering for motstand mot påvirkning fra jordskjelv er en del av den konstruksjonstekniske prosjekteringen i et byggetiltak. Oppgaven ligger under rådgivende ingeniør i byggeteknikk (RIB). Da reglene ble innført i 2004 måtte bransjen ruste seg for å sikre kompetanse innen relevant metodikk for å møte kravene. Etter hvert har hele bransjen løftet seg på dette området, og kurs har vært avholdt, ikke minst gjennom Rådgivende Ingeniørers Forening (RIF). En tydelig utvikling har gått mot etablering av digitale modeller, uansett hvilke laster som måtte gjelde. Dette betyr at en del av prosjekteringsarbeidet med hensyn til jordskjelv ligger innenfor det arbeidet som uansett ligger i å etablere en modell av det aktuelle byggverket. Av denne grunn kan en si at det prosjekteringsmessige arbeidet med håndtering av jordskjelv etter hvert utgjør en forholdsvis mindre del av det totalen innen konstruksjonsteknisk prosjektering. Et unntak er tilfelle der en må gjøre helt grunnleggende konseptuelle studier (f.eks. sykehus på dårlig grunn) der prosjektering med hensyn til jordskjelv kan utgjøre millionbeløp.

Rådgivende ingeniør innen geoteknikk (RIG) er en viktig aktør når det gjelder prosjektering for motstand mot påvirkning fra jordskjelv. Dette gjelder særlig for tilfelle med dårlige grunnforhold og der det kan være behov for flere runder mellom RIB og RIG for å finne frem til en riktig modell for interaksjonen jord (grunn) og konstruksjon.

For et byggetiltak i seismisk klasse II med grunntype E (vanlig kontor/bolig) kan det være snakk om besparelser i størrelsesorden noen hundre tusen kroner i prosjekteringskostnader, dersom en ser på konsekvensen av å komme inn under utelatelseskriteriet. Besparelsen er med andre ord signifikant, men utgjør ikke en betydelig del av byggekostnadene.

9 Mulig implementering i regelverket

Rent prinsipielt, og ut fra rapportens konklusjoner om økonomiske besparelser, er det hevet over tvil at bransjen må få tilgang til og anvende de oppdaterte 475årsverdiene som NORSAR har utarbeidet.

Formelt kan dette i det enkelte prosjekt forankres i Byggteknisk forskrift TEK 17 §2-1(3).

I denne paragrafen gis det adgang til alternativt til Norsk Standard å anvende annen, likeverdig standard. Det finnes i realiteten ikke likeverdig standard til Eurokode 8, men inntil endringsblad eller revisjon av det nasjonale tillegget foreligger, bør en kunne anvende verdier for spissakselerasjoner som det kompetente fagorganet NORSAR har utarbeidet. Direktoratet for byggkvalitet (DiBK) kan anmodes om å utarbeide et rundskriv om dette, eventuelt via KMD. Anmodningen kan komme fra RIF, som blant annet representerer foretak som erklærer offentligrettslig ansvar for konstruksjonssikkerhet i byggetiltak.