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Department of Mathematical Sciences and Technology

Behavior of concrete buildings under earthquake loading: precast vs on-site casted.

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Preface

This master thesis has been written at the Norwegian University of Life Sciences (NMBU), Department of Mathematical Sciences and Technology. The work has been conducted in the spring semester of 2016.

The idea for my thesis came from my father, Helge Moldskred, who works as an engineer in Moldskred AS. There is a lack of knowledge, on how much difference exists in the use of prefabricated concrete elements compared to on-site casted concrete walls, when we design a building under earthquake loading.

I have learned a lot writing this master thesis, related to earthquake as a physical phenomenon and its interaction to reinforced concrete structures. It has been genuinely interesting to learn more about how buildings are designed for earthquake loads, and how to construct models that simulate this problem using a finite element program.

Acknowledgements

First of all, I would like to thank the employees of Moldskred AS for great conversations about relevant topics, tips and help with the simulations using FEM-Design software. I also want to thank my supervisor at NMBU, Associate Professor Themistoklis Tsalkatidis, for his help and motivation. I must also thank a fellow good student, Eirunn Dvergsnes, who has often come with motivation during coffee-brake conversations through this semester. I want to thank my very good friend, Daniel Nøkland, for his precious advice. Last but not least, I have to thank my friends and my family who have supported me during this period.

Abstract

Seismic design of buildings is a relatively new field in Norway. It has not been long since the requirements that buildings must be designed for seismic loads became appliable. This thesis will compare two different models using a finite element program according to seismic influences. The first one is a prefabricated element model, and the second is an on-site cast model. The thesis consists of a concise theory part, which will address important topics according to earthquake analysis.

This thesis is utilising a new building, which is under construction in Fosnavaagen city, and used this as a starting point for the two models. It is desirable to see how big a difference it would have been if the building was casted on-site in relation to the prefabricated element building.

The analysis of the models made using static lines form, and are performed by FEM-Design. The seismic parameters used in the task are taken from the Norwegian appendix to the Eurocode 8.

The results of the analysis shows that there is not significant difference between the two models under the seismic conditions on the site.

Keywords: Earthquake Analysis, FEM Modelling, Eurocode 8.

Sammendrag

Jordskjelvs dimensjonering er et relativt nytt fagfelt i Norge. Det er ikke lenge siden kravene om at bygninger skal dimensjoneres for seismiske laster kom. Denne masteroppgaven vil sammenligne to ulike modeller ved bruk av finite element program i forhold til seismiske påvirkninger. Den ene modellen er en prefabrikkert element modell, og den andre er en stedstøpt modell. Oppgaven er bygd opp av en kortfattet teoridel, som vil ta for seg viktige temaer ifølge jordskjelvs dimensjonering.

Oppgaven tar for seg et eksisterende bygg, som er under bygging i Fosnavågen sentrum, og bruker det som utgangspunkt for de to modellene. Det er ønskelig å se hvor stor forskjell det hadde vært om bygget ble støpt på stedet, i forhold til det prefabrikkerte element bygget.

Analysen av modellene gjøres ved bruk av statisk linjer form, som utføres av FEM-Design. De seismiske parameterne brukt i oppgaven er hentet fra det norske tillegget til Eurokode 8.

Resultatet av analysen viser at det ikke er så stor forskjell på de to modellene under de seismiske forholdene på stedet.

Nøkkelord: Jordskjelvsanalyse, FEM Modellering, Eurokode 8.

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Terminology

Roman upper case letters

BIM	Building information modeling
CAD	computer-aided design
Co	terrain form factor
Ce(z)	exposure coefficient
Cpe,10	form factor for wind pressure
EC1	Eurocode 1
EC8	Eurocode 8
HVAC	heating, ventilation and air-condition
S	amplification factor of soil conditions
T	natural own swinging period
Tb(s)	Lower limit for the area with constant spectral acceleration
Tc(s)	upper limit of the range of constant spectral acceleration
Td(s)	value that defines the beginning of the spectre of constant displacement

Roman lower case letters

a_{g40Hz}	Bedrock acceleration
a_{gR}	reference value on bedrock acceleration
a_g	dimensioning bedrock acceleration
c	damping factor
f_I	terrain force
f_D	resistance due to energy attenuation in frame construction
X	

f_S	resistance because of lateral rigidity
k	stiffness factor
m	construction mass
p_{eff}	effective seismic load
q_k	payload
qp(z)	ground value for velocity pressure from wind
s	snow load
sk	characteristic snow load
vb,0	wind load, reference value
u(t)	relative displacements
$u^t(t)$	total displacements
$u_g(t)$	displacements in the ground
$\dot{u}$	frames speed
$\ddot{u}$	frames acceleration
$\ddot{u}_g$	ground acceleration
$\ddot{u}^t$	acceleration

Greek lower case letters

γ	Seismic factor
μ	form factor for snow load
ψ_2	combination factor
ω	vibration frequency

1 Introduction

1.1 Background

In 2004 came the first requirements for earthquake dimensioning in Norway (NS 3491-12) and they lasted until 2010 when it was replaced with a more precise standard, Eurocode 8, with the national appendix, NS-EN 1998-1:2004+NA:2008. This has led to an increasing focus on earthquake design in Norway, in comparison to the past, and requires structural engineers with expertise in the field of earthquake resistance design of buildings.

When comparing on site casted concrete and precast concrete elements, there is a significant difference in the way they carry and distribute the loads. Precast concrete elements have weaker connections than the on-site casted ones. The latter ones have monolithic connections with increased strength and stiffness.

In Norway, the use of precast concrete elements has increased in recent years. It is intriguing to see the difference in relation to the on-site casted concrete, which were mostly used in the past. As an example, a building (under construction) south of Aalesund has been investigated.

1.1.1 Earthquake history in Aalesund

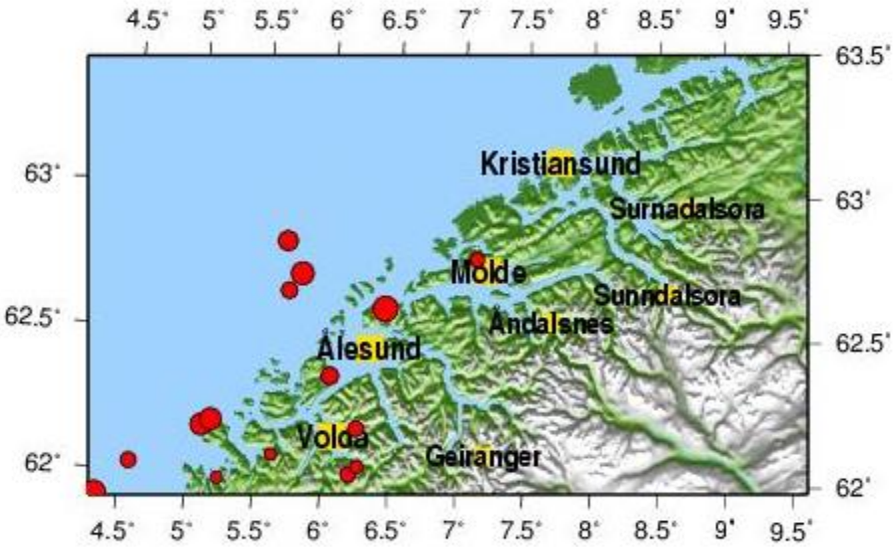


Figure 1-1: Map of Moere og Romsdal county, with dots where there has been noticeable earthquakes since 2000 until today. (NORSAR 2016)

The building is placed in Fosnavaagen, a town southwest of Aalesund. The building is located in a more earthquake prone area compared to Aalesund, as it shows in the Figure A - 3 in

Appendix A: Hand calculations. In Figure 1-1 the earthquakes that have been noticeable in Moere og Romsdal since year 2000, are shown. The largest earthquake, during the last 16 years, was in 2007, a 3.4 magnitude earthquake on the Richter's scale, located north-northeast of Aalesund. All the seismic events since year 2000 until today are depicted in Figure 1-2.

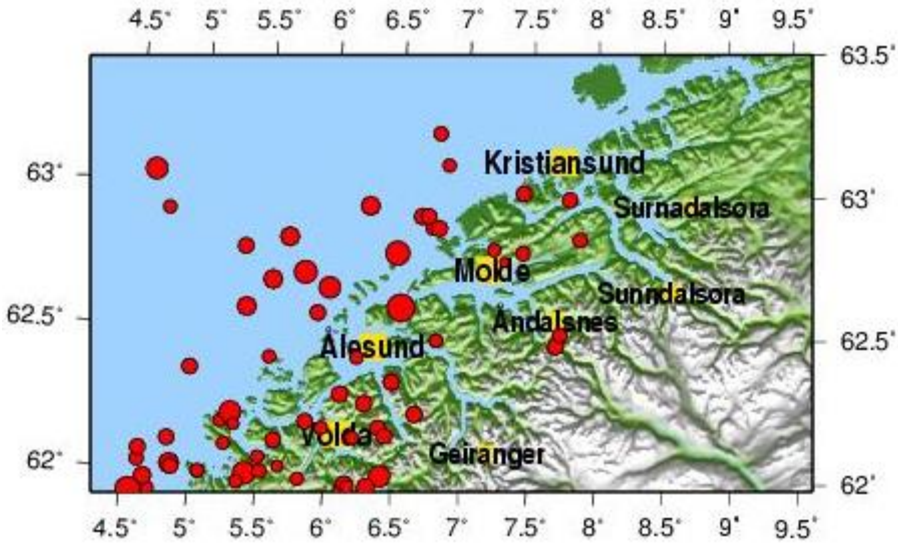


Figure 1-2: Map of Moere og Romsdal county, with dots for all seismic events since 2000 until today.

1.2 Objective of the thesis

The main aim of this thesis is to investigate the seismic behaviour of reinforced concrete buildings, which have been constructed either with precast structural elements or with on-site moulded ones. The case study of a residential building in Fosnavaagen south-west of Aalesund is presented. The structural analysis has been conducted using both national and European standards.

2 Theory

2.1 General information on earthquakes

According to NORSAR (Norwegian Seismic Array) an earthquake is defined as a sudden rupture in the crustal due to natural causes. The point of a faulting where the rupture starts is called focus or hypocentre, Figure 2-1, while the point on the surface that is across called epicentre see Figure 2-1.

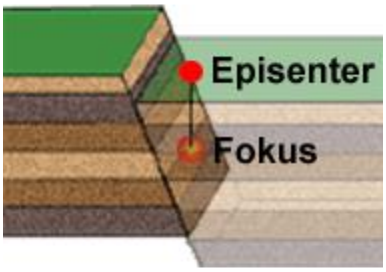


Figure 2-1: Focus and epicentre of an earthquake, (NORSAR 2016)

When this occurs, it sends out energy waves in forms of vibrations. These vibrations are variable in size, from unnoticeable to very noticeable. The fractions in the crustal are due to the continental plates that are in constant motions. These plates are called tectonic plates. It is common to divide earthquake and movement on an escarpment into three categories, shown in Figure 2-2.

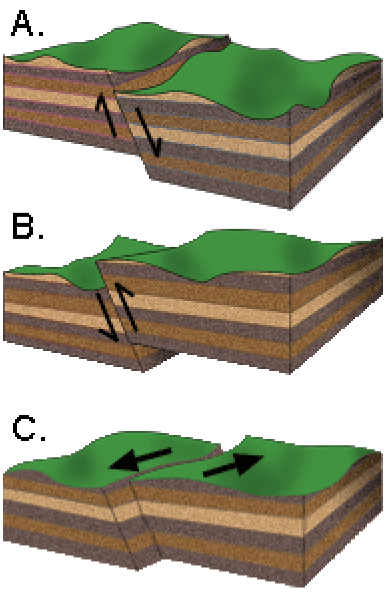


Figure 2-2: A: Normal faults, B: Reverse faults, C: sideways faults.(NORSAR 2016)

In cases where the crust is getting longer by the movement, it is called fault normal (A). This will occur in areas where the crust is unstressed. If the crust shortens by a movement on an escarpment, it is called a reverse fault (B). When the earth's crust is being compressed in an area, it will typically give rise to reverse mechanisms. Sideways faults (C), is movement going sideways along the crust. Sideways faults can be further divided into left-lateral and right-lateral, in which way the opposite block appear to move. (NORSAR 2016)

Although Norway is a safe distance from the rim of the continental plates, it is one of the most earthquake-prone areas in Northern Europe. According to EC8, Norway is considered a low to medium seismic country, since the earthquake activity is moderate. (NORSAR 2016)

When calculating a building, you must think of the strength of the structure. When calculating wind force you take into account a certain amount of stiffness, but for earthquake forces, this stiffness can become difficult to calculate. Earthquake's impact on structures must be investigated in both directions (x and y), whereas the wind load often has one dominant direction. (Løset et al. 2011)

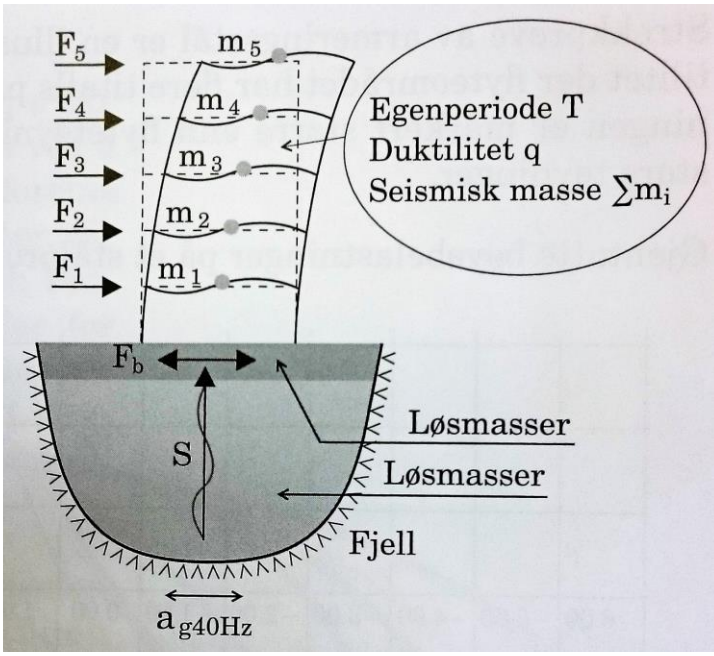


Figure 2-3: Ideal representation of EC8 processing of earthquake dimensioning. (Løset et al. 2011)

As shown in Figure 2-3. The horizontal ground acceleration a_{g40Hz}, that shows the largest bedrock acceleration values. The horizontal vibrations, propagate through the soil and can change character, these vibrations are expressed with the foundation factor S. Fluctuation can occur in the building's main construction by these vibrations. The vibrations depends on the difference between period on the earth's fluctuations and the buildings own fluctuation period T. If the earth's fluctuation period is close to the own fluctuation period (T) of the building, the added force from the earthquake will become larger with dynamic resonance effect. The forces in the building will become dependent of the construction's ability to absorb and distribute earthquake energy, expressed with the construction factor q. Together these gives us parameters horizontal shear force caused by seismic impact(F_b) on top of a stiff basement or on ground level. (Løset et al. 2011)

2.2 About the building site

The building is located in Fosnavaagen, just south of Aalesund, in the north-western part of Norway. It is in close proximity to the sea, as shown in Figure 2-4.

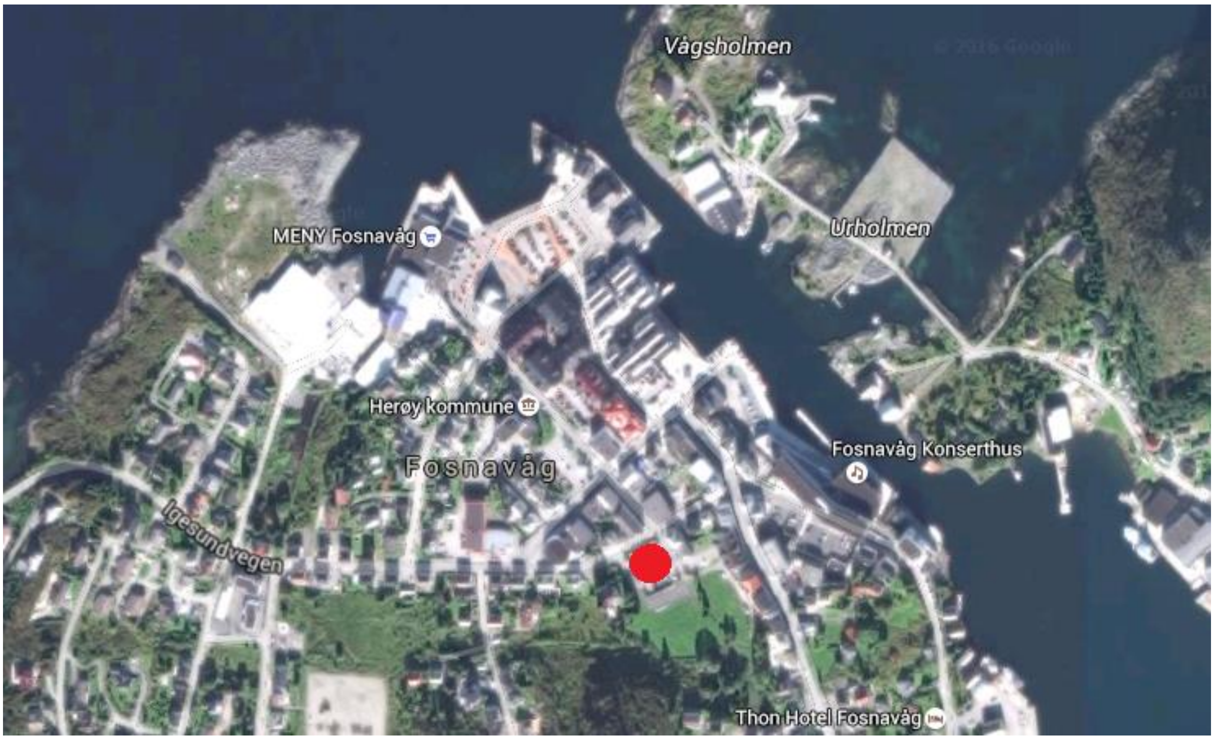


Figure 2-4: Picture of Fosnavaagen, the red dot marks the building site.

The terrain is mostly hard rock. The site has been blasted out, so there is a minimum of 0,5m down to the hard rock. The site is than filled with the rock fill, and finished off with a layer of crushed stone. The site plan is shown in Figure 2-5.

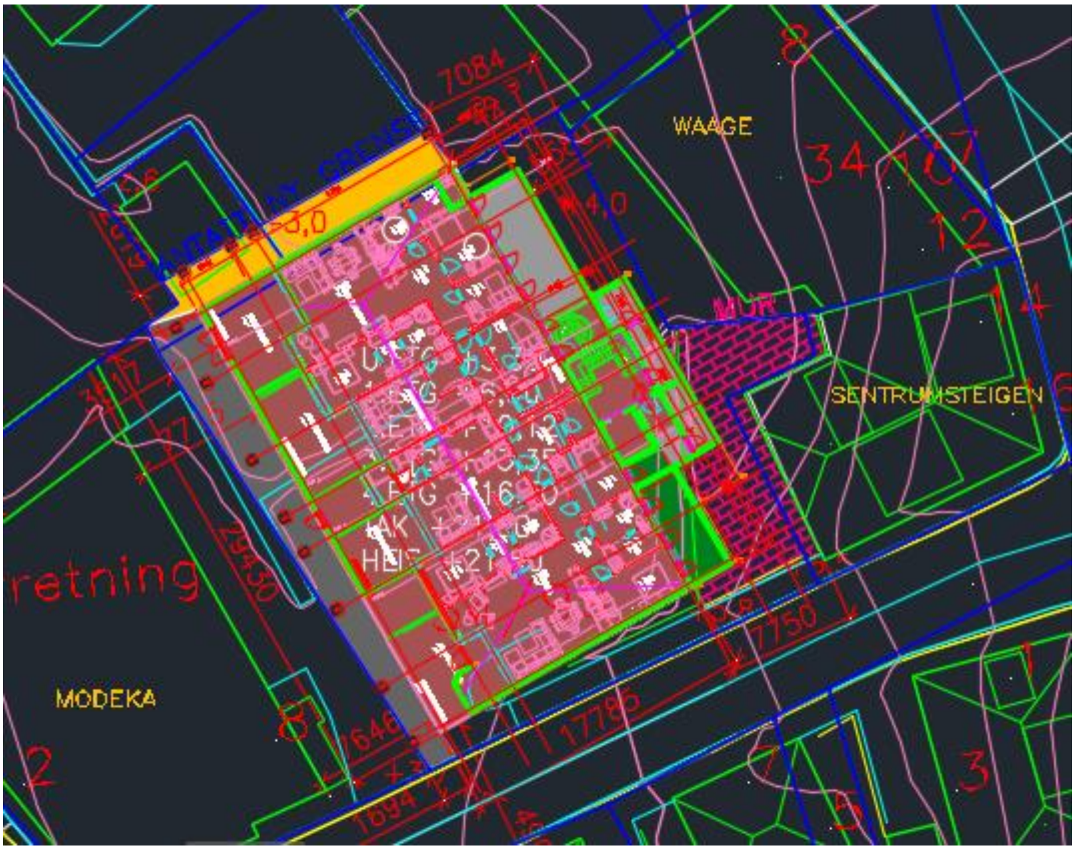


Figure 2-5: Site plan from Moldskred AS

2.3 Description of the building

The building has four-storeys and a basement. The basement is built of on-site casted concrete, and is used as a garage for the apartments in the upper floors and as storage space. See Appendix C for the AutoCAD drawings of the plan solution. In the rest of the building the outer walls are constructed using precast concrete elements. The floor is built with hollow core slabs with a thickness 400mm. The building is built with a light weight roof as shown in Figure 2-6

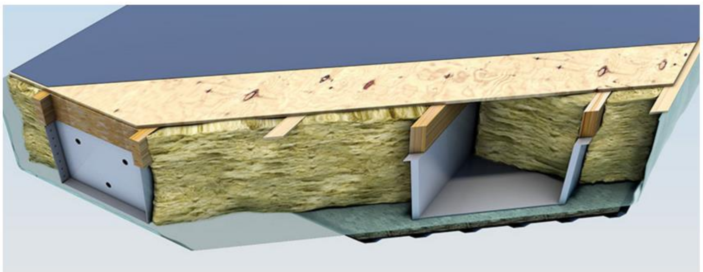


Figure 2-6: The roofing system, (LETT-TAK SYSTEMER AS 2016)

The building has a solid foundation system. There are two ground foundations in each axis in order to transfer the loads from the building to the ground. The axis system is shown in Figure 2-5 and Figure C - 1.

Figure 2-7: The building, (Moldskred 2016)

The building is under construction, in the spring of 2016. See Appendix B: Pictures from the site, showing how far the building process have come by May 2016. The ground work started in September 2015, and the Concrete work started in December 2015. Completion of the project is expected approximately December 2016, depending on the number of apartments that has been sold. Figure 2-7 shows the model from the drawing program, as it is going to look when completed. The involved companies is Kvadratbygg AS, Moldskred AS, Stryn Betongelement, Fosnavåg Rør and Elek 24 AS.

2.4 Mercalli intensity scale

The seismic scale used to measure the intensity of an earthquake is called Mercalli intensity scale, see Table 2-1. It is used to classify the effects of the earthquake and not the magnitude.

2.5 Richter magnitude scale

The strength of the earthquake is considered by measuring the energy that is released during the earthquake. To avoid the use of large numbers, the magnitude scale is a logarithmic scale. Richter-magnitude scale is the most common way to express the magnitude of an earthquake, see Table 2-1. However, in earthquake design the value of the ground acceleration is used instead of the size of magnitude, according to EC8.

Table 2-1: Richter magnitude scale and the Mercalli intensity scale

Richter Magnitude	Average earthquake effects	Mercalli intensity	Frequency of occurrence, per year
<2,0	Microearthquakes, not felt, or felt rarely.	<II	>1 million
2,0-2,9	Felt slightly by some people. No damage to buildings	I-III	>300 000
3,0-3,9	Often felt by people, but very rarely causes damage	III-V	49 000
4,0-4,9	Felt by most people in affected area. Noticeable shaking of indoor objects and ratting noises.	IV-VI	6200
5,0-5,9	Felt by everyone. Can cause damage of varying severity to poorly constructed buildings.	VI-VIII	800
6,0-6,9	Felt in wider areas; up to hundreds of miles/kilometres from the epicentre. Strong violent shaking in epicentral area. Damage to a moderate number of well-built structures in populated areas.	VII-IX	108

7,0-7,9	Felt across grate distances with major damage mostly limited to 250 km from epicenter. Causes damage to most buildings, some to partially or completely collapse or receive severe damage.	IX-XI	18
≥ 8,0	Major damage to buildings, structures likely to be destroyed. Damaging in large areas. Felt in extremely large regions.	X-XII	1-1,5

2.6 Construction Dynamics

2.6.1 The movement equation

According to (Chopra 2013) a way to describe the earthquake impact on a building is by using an idealized frame structure. This involves a frame structure with the entire construction mass is centred in the frames beam. The structure has a certain lateral rigidity and being composed of one floor with the stiffness k. The frame system is consist of one degrees of freedom system, with only one displacement in the system.

The movement of the idealized one-storey frame due to earthquake is produced by a single motion equation using dynamic equilibrium. During earthquakes displacements in the ground will be referred to as $u_g(t)$ and the total displacement $u^t(t)$. The relative displacements are referred as u(t). These displacements are time-related and can be represented as:

$$u^t(t) = u(t) + u_g(t) \qquad 1$$

The equation of motion related to the earthquake can be prepared by using dynamic equilibrium. Another form to describe it could be by using Newton's second law (Σ F = m x a), which will also provide the same equation as the dynamic equilibrium provides. In earthquake engineering, it is common to rely on dynamic equilibrium. Dynamic equilibrium framework then provides:

$$f_I + f_D + f_S = 0 \qquad 2$$

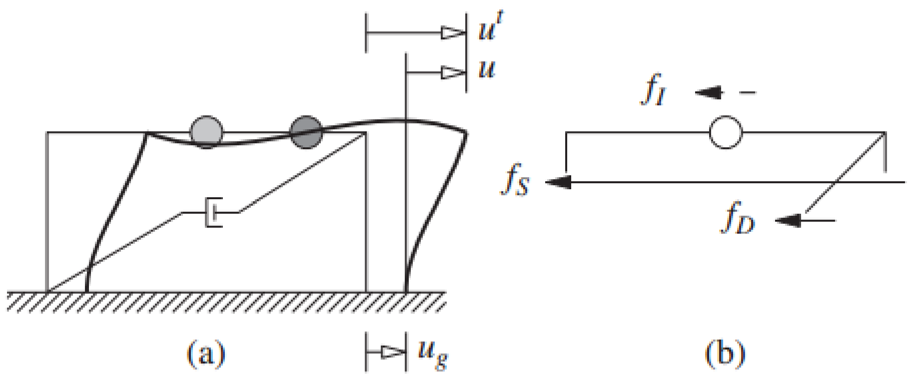


Figure 2-8: Dynamic equilibrium, (Chopra 2013)

f_I is the inertia force of the system counteracting earthquake movement, and consists of structural mass m subjected to an acceleration $\ddot{u}^t$.

Resistance force caused by dampening, f_D, occurs due to the mechanisms of the system that allows the energy in the system is damped by performing a work. In the structure that is under influence of vibration, these mechanisms can for example, be friction in steel connections, opening and closing movements of micro-cracks in concrete. Other forms of damping may also be friction between the main grid and non-structural elements. It is difficult to identify these mechanisms, therefore an idealized damping factor is used. (Chopra 2013)

According to damping in construction, resistance is caused by lateral rigidity, f_S. Lateral rigidity is related to the element mechanical properties such as stiffeners and axial, which prevents the structure dislocations. Expressions below shows what the equation of motion consists of. (Chopra 2013)

$$f_I = m\ddot{u}^t \qquad 3$$

Insertion of equation (1) into equation (3), $f_{I,}$ can be expressed as:

$$f_I = m(\ddot{u}_g + \ddot{u}) \qquad 4$$

$$f_D = c\dot{u} \qquad 5$$

$$f_S = ku \qquad 6$$

Put equation (4), (5) and (6) into equation (2), can be expressed as the equation of motion (7) as the basis for the theory behind earthquake engineering.

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g(t) = p_{eff} \qquad 7$$

p_{eff} is the effective seismic load of the ground movement

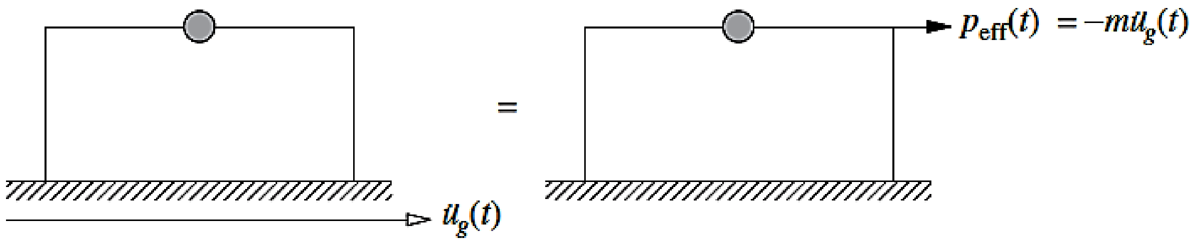


Figure 2-9: The effective seismic load p_{eff} is horizontal because acceleration $\ddot{u}_g(t)$.(Chopra 2013)

2.6.2 Free vibration

An object is under free vibrations when it is disturbed from its equilibrium position, and allowed to vibrate without external dynamic influences. (Chopra 2013)

By considering the idealized frame system which was presented in the previous section without damping and external dynamic strain, we can study on a simplified system. Where c = 0 and P_{eff} = 0. And if the mass is disturbed from its equilibrium position, i.e. with a displacement (0) and / or rate of $\dot{u}(0)$ in time 0, the system will pivot about its static equilibrium position shown in Figure 2-10. The figure is a graphical representation of the homogeneous equation, and can be derived from the equation of motion (7).

$$u(t) = \frac{\dot{u}(0)}{\omega}\sin\omega t + u(0)\cos\omega t \qquad 8$$

ω is the vibration frequency:

$$\omega = \sqrt{\frac{k}{m}} \qquad 9$$

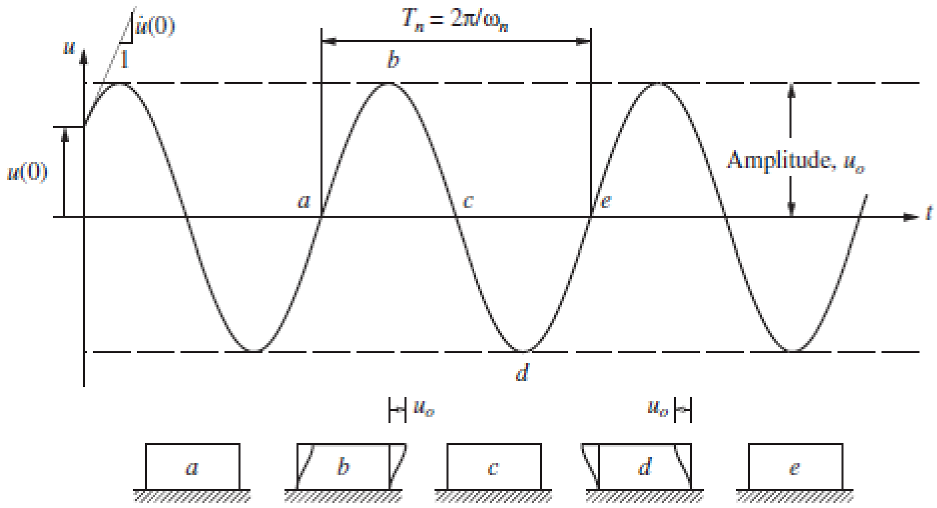


Figure 2-10: Free vibration of an system without damping, (Chopra 2013)

In Figure 2-10 the graph shows equation (8) which describes a feature of harmonic motion. The graph turns on the static equilibrium position, which in practice means that the frame structure also turns on its original equilibrium position. This movement is illustrated at the bottom in Figure 2-10 in that, for example in position a, the frame in static equilibrium. However, when the structure moves to position b, the swing frame moves toward the right side, and frame is no longer in equilibrium. The frame will continue to move about its equilibrium position without stopping. In reality, the frame go to rest because of different damping mechanism in the system and provides similar waveforms as shown in Figure 2-11 (Chopra 2013). For the sake of understanding it will be sufficient with an undamped system.

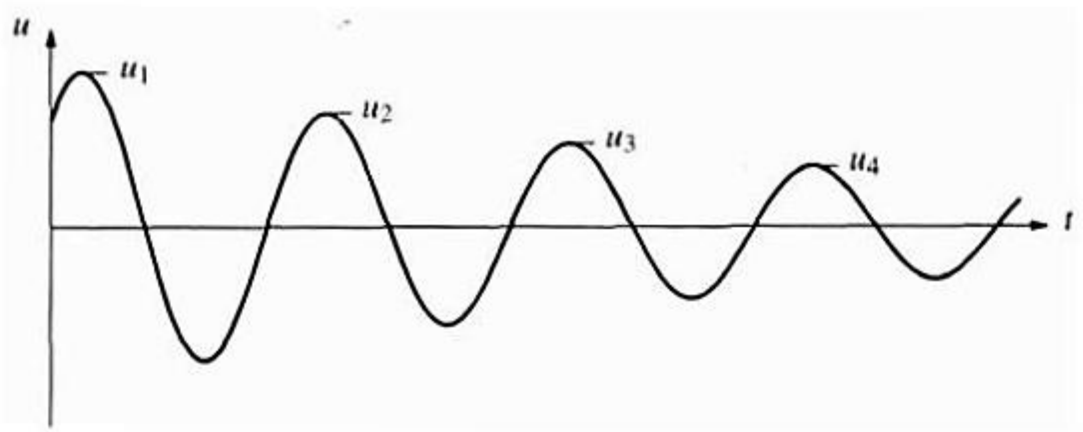


Figure 2-11: Free vibration of a damped system, (Chopra 2013)

Time required for a undamped system to complete a cycle of the free vibration is the natural oscillatory period or the resonance period, T expressed as:

$$T = \frac{2\pi}{\omega} \qquad 10$$

T is own turn period, see Figure 2-10

All building structures will have one or more waveforms, depending on the number of degrees of freedom. These fluctuations provide base for swing form, which is an important factor in earthquake engineering. A swing form has its own turn period, which also helps to determine the structure's response.

2.7 Response spectrum

Response spectrum is a useful way to illustrate and determine how their swing period of vibration and damping of a structure affects the response of the building exposed to a given earthquake motion. (Charleson 2009)

Modal response spectrum is a linear dynamic static method that determines contribution of each natural oscillatory of vibration for a certain attenuation, to indicate the maximum seismic response of a resilient construction.

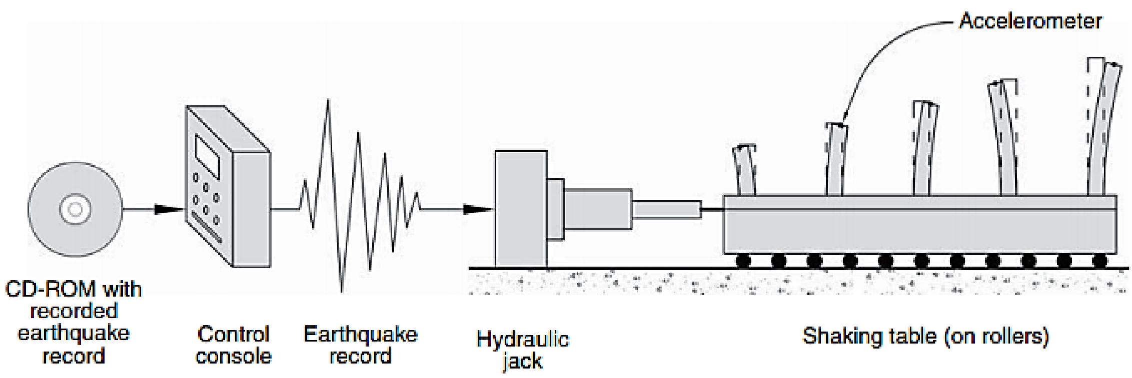


Figure 2-12: Generating a response spectrum from an earthquake record using a shaking table. (Charleson 2009)

Figure 2-12 shows an example of model structures on a shaking table with different shape and vibration periods, rising period from left to right. The structures have identical attenuation each with its accelerometer attached to the roof to measure its maximum horizontal acceleration. The models are subjected to a specific earthquake, and their maximum accelerations are measured and plotted in a graph, Figure 2-13.(Charleson 2009)

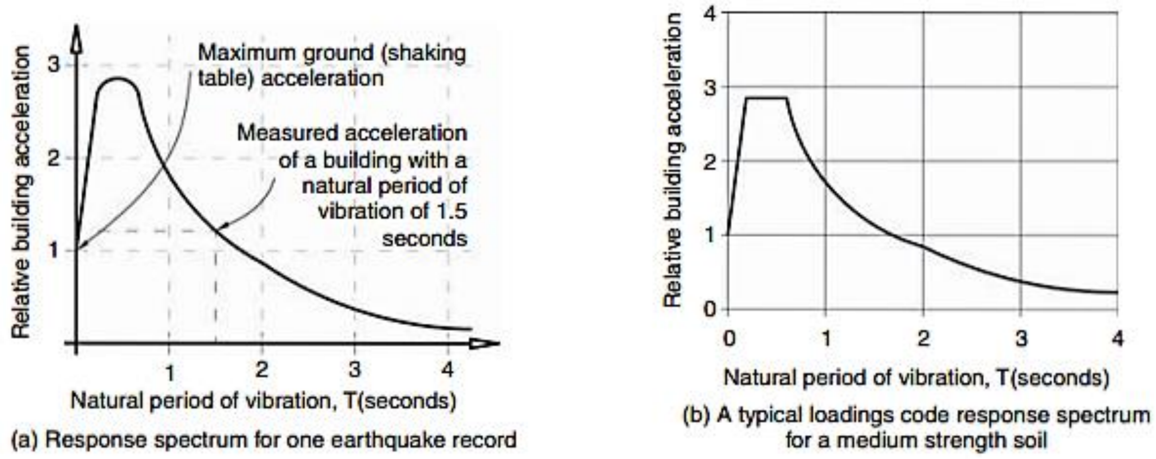


Figure 2-13: A typical response spectrum (a) and its expression in an earthquake loadings, (Charleson 2009)

The shape of the response spectrum shows how separate period and the vibration of a system has a major effect on the horizontal acceleration as it is exposed to, and that in turn affects the size of the internal forces that structure shall be designed. After a certain point, the longer their period, the smaller the maximum acceleration and the seismic design forces. This depends a lot on how the flexibility of a system is (Charleson 2009).

2.8 Ductility

Ductility is defined as the material's ability to deform out beyond the elastic limit, without losing much of its strength or characteristics. In the design, we mainly think about the construction's ability to absorb earthquake energy in plasticity areas, and to distribute the energy in all the construction parts that is assumed to be plasticised under the earthquake. The construction factor q is an expression on how ductile the main contraction is. Demands to maintain the ductility in the concrete and reinforcement are given in the Eurocode 8. (Løset et al. 2011)

2.8.1 DCL – Low ductility

A construction with low ductility has q ≤ 1.5. The ductility level is related to the earthquake structural design. The benefits of choosing a low ductility factor is to dimensioning, according to the most regular standards for capacity. EC8 considers that this standard provides sufficient capacity but limited energy absorption q ≤ 1.5, and is used only to reduce the earthquake loads of the building. (Løset et al. 2011)

2.8.2 DCM – Medium ductility

Construction factor q, with medium ductility has values ranging from 1.5 to 4. Earthquake loads are reduced. Meanwhile, it is assumed that when a ductile deformation of the supporting structure is identified, the detailed requirements of local compounds and materials given in EC8 for ductile prefabricated concrete structures, are satisfied. (Løset et al. 2011)

2.9 On-site casted concrete

Concrete consists of cement, water, sand, stone and additives. The most commonly used compressive strength class for concrete is C25/30. For the strength classes see Table 2-2.

Table 2-2: Strength classes for concrete

Compressive strength class	Characteristic cylinder strength $f_{ck,cyl}$ N/mm²	Characteristic cube strength $f_{ck,cube}$ N/mm²
C8/10	8	10
C12/15	12	15
C16/20	16	20
C20/25	20	25
C25/30	25	30
C30/37	30	37
C35/45	35	45
C40/50	40	50
C45/55	45	55
C50/60	50	60

2.10 Precast concrete

Precast concrete is concrete that is cast in a different location, and transported to the site, and assembled there. The hollow core slabs are built up from plastic pipes, which is oval or circular. The slabs are not casted in a formwork, but by using a less watery concrete it is casted with a machine, that runs continuously leaving the concrete with holes, shown in Figure 2-14. The hollow core elements has a width on 1200mm, and can be made in several different lengths. The underneath part of the hollow core slabs is smooth, while the top and side edges have a more rough embodiments that provides good cohesion for joint casting, levelling and screeds.

Figure 2-14: Machin that maces hollow core elements, picture from a tour at Spenncon in Hjoerungavaag

2.11 Soft storey

A soft storey may be at any level of the building. Often the ground floor is soft, because a more open floorplan solution and higher panelled ceiling is expected. In the upper floors, there are walls, which are the bracing elements, but in the ground floor columns replace these walls. Therefore, ground floor is regarded as soft in the horizontal direction. The columns are often damaged by the relative displacements caused by cyclic loading. When the plastic deformations at the top and bottom end of the columns lead to a dangerous sway mechanism then it is often inevitable to avoiding a collapse. (Bachmann & l'environnement 2003)

2.12 AutoCAD

Autodesk AutoCAD is a program that is used to draw building plans, facades, section, details and reinforcement drawings. In Autocad you can draw both in 2D and 3D, but there are engineers that will rather use Revit in the 3D drawings. It is possible to collect the drawing from AutoCAD and export them to Revit, and vice versa.

2.13 Revit

Autodesk Revit is a BIM (Building Information Modeling)-program, which is used for showing the construction in 3D. The software is used for construction, architectural, and HVAC(heating, ventilation and air-condition) drawings. With Revit you can take the file and export it in to a file that the FEM-Design program can read.

2.14 FEM-Design

FEM-Design (finite element method) is a finite element program used to analyse and design the load-capacity of concrete, timber and steel structures, according to Eurocodes. The program is based on the CAD (computer-aided design) tools, that makes it easier to draw the building in 3D or import it form BIM-software. The results is shown in different 3D-graphs, contour lines, colour palettes or sections.

2.15 Different shape method in Fem-design

2.15.1 Static linear shape method

Static linear shape is a simplified fundamental modal shape method. The loads increase linearly along the height with approximated horizontal displacements. This method is usable, if the whole foundation is on the same horizontal plane or non-elastic. (StruSoft 2010)

2.15.2 Static, mode shape

In the static mode shape, the distribution of shear forces in the base is according to the fundamental mode shapes. This method takes into account the total mass of the structure, and not the effective mass. (StruSoft 2010)

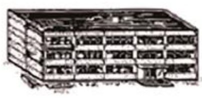
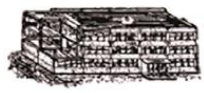
2.15.3 Modal analysis

The Modal analysis gives the possibility to investigate all the directions (x, y, and z). The method is used to calculate the structural response under different ground motions, by summation of more vibration shapes. (StruSoft 2010)

2.16 Damage degrees

There are different categories for damage after an earthquake, see Table 2-3. Grade 1, gives no or slight structural damage. Grade 2, is moderate damage, some cracks in the structural components. It is used to describe small repairable damages of structures, after an earthquake. However, a building in this category can still be regarded as intact. Grade 3 denotes heavy damaged structure after an earthquake. The building needs serious repairing of its important structural elements. Grade 4 denotes heavy damage of the structure and very heavy non-structural elements damage. Furthermore, some parts of the construction may fail, like local collapse of a few columns or plates. The building often has to be demolished and rebuilt. Grade 5 denotes collapse of important construction parts of the building.

Table 2-3: The different damage degrees, (Verderame et al. 2014)

DS1	DS2	DS3	DS4	DS5
				
Grade 1: Negligible to slight damage *(no structural damage, slight non-structural damage)*	**Grade 2: Moderate damage** *(slight structural damage, moderate non-structural damage)*	**Grade 3: Substantial to heavy damage** *(moderate structural damage, heavy non-structural damage)*	**Grade 4: Very heavy damage** *(heavy structural damage, very heavy non-structural damage)*	**Grade 5: Destruction** *(very heavy structural damage)*
Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills	Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels	Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels	Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor	Collapse of ground floor or parts (e. g. wings) of buildings
$\min\left(\Delta_{cr}^{inf} ; \Delta_{cr}^{RC}\right)$	$\min\left(\Delta_{max}^{inf} ; \Delta_{y}^{RC}\right)$	$\min\left(\Delta_{ult}^{inf} ; \Delta_{spalling}^{RC} ; \Delta_{buckling}^{RC}\right)$	Δ_{ult}^{RC}	Δ_{coll}^{RC}

3 Method

Two three-dimensional finite element models have been constructed using Fem-Design software. The first model is with precast concrete walls and the second one with cast on-site reinforced concrete walls. The next step was to analyse the models and comparative evaluate their results.

3.1 The models

The basement is constructed with solid concrete walls with a thickness on 200mm, which is supported in all directions and with concrete columns with the section 300x300mm, which is also supported in all directions. The floor in the upper parts of the building are built with hollow core slabs, drawn as a normal concrete slab with a reduced mass, with a thickness of 400mm, and on-site casted balconies. In the top floor, the plate’s construction goes all the way out to the outer wall, compared to other floors where it stops at the balconies. If there are balconies, the rest of the floor goes out to the outer wall. The plates are supported by concrete prefabricated beams, that are in reality an L-beam in the same size, and drawn as a rectangular beam with the size 250x600mm. The beams lie on top of hollow core steel columns 150x150x10mm. There are some smaller beams with the size 200x300, in the walls, that are really shelves moulded into the walls for support of the hollow core slabs.

3.1.1 The precast concrete element model

The middle floors are constructed with precast concrete elements that is connected with bolts of 60mm in diameter in the horizontal gaps, and steel welding of 200mm at the vertical gaps in concrete elements. The elements is two floors high in most parts of the building, in the front at the third floor it is only over one floor cause the top floor is pulled inn and the outer walls is replaced with timber walls.

3.1.2 The on-site casted concrete model

The outer walls in the building are drawn in on-site casted concrete, except a timber wall in the top floor at the front of the building, to minimize the loads.

3.2 Materials

The following materials have been used for the construction and the analysis of the building:

Concrete C25/30

Steel columns S355

Steel connections S355

3.3 Loads

The payloads are collected form (*Eurokode 1 : Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-1: General actions : Densities, self-weight, imposed loads for buildings : Del 1-1 : Allmenne laster : Tetthet, egenvekt og nyttelaster i bygninger* 2008). The building is according to Table A - 1 in the category A, areas for indoor activity and home activity. From Table A - 2 payload on floors, the loads used for balconies is 2,5 kN/m^2, staircases is 2,0 kN/m^2 and floor load is 2,0 kN/m^2, is the payload that is used in the models in Fem-Design.

The snow loads are collected form Table A - 3, our building in Fosnavaagen city in Heroey municipalities. $S_{k,0}$ = 2,5 kN/m^2. The snow load on the roof is 2,0 kN/m2, see Appendix A: Hand calculations for the calculations. The snow load accumulation is 2,0 kN/m^2, it is smaller than the balcony payload so in the model that is what's used.

The wind load for Heroey municipalities are collected form Table A - 3, $V_{b,0}$ = 30 m/s. The wind pressure on the windward side is 1,28 kN/m^2, and the wind pressure on the leeward side is -0,8 kN/m^2, see Appendix A: Hand calculations.

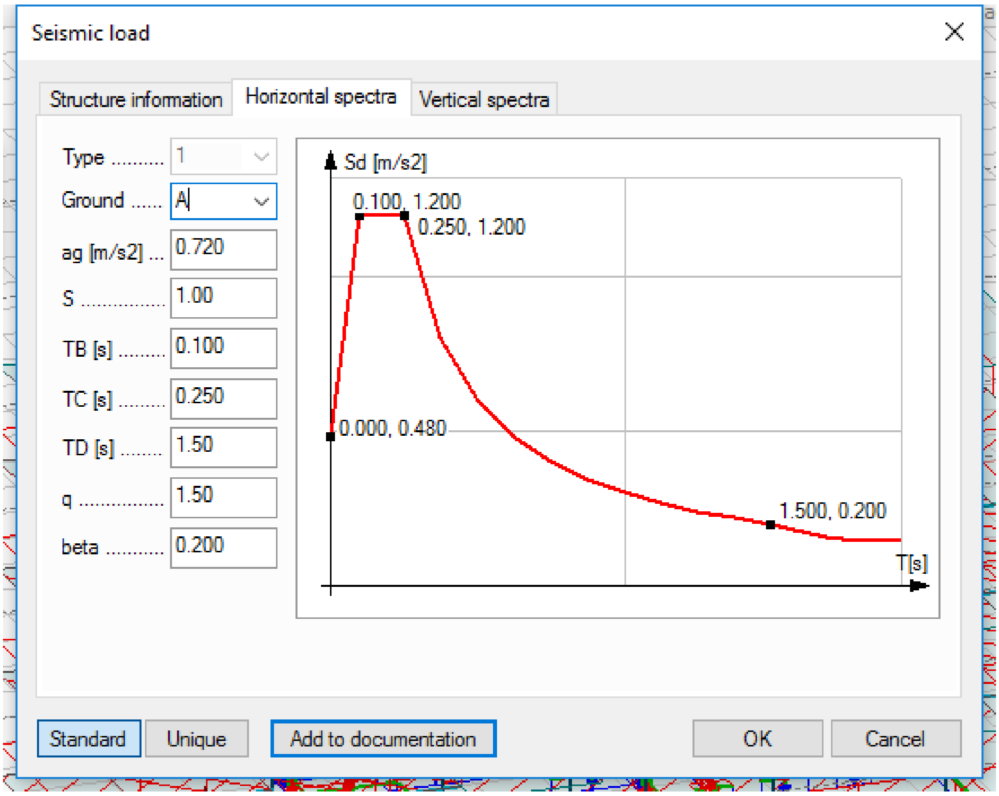


Figure 3-1: Seismic loads in FEM-Design, Horizontal spectra.

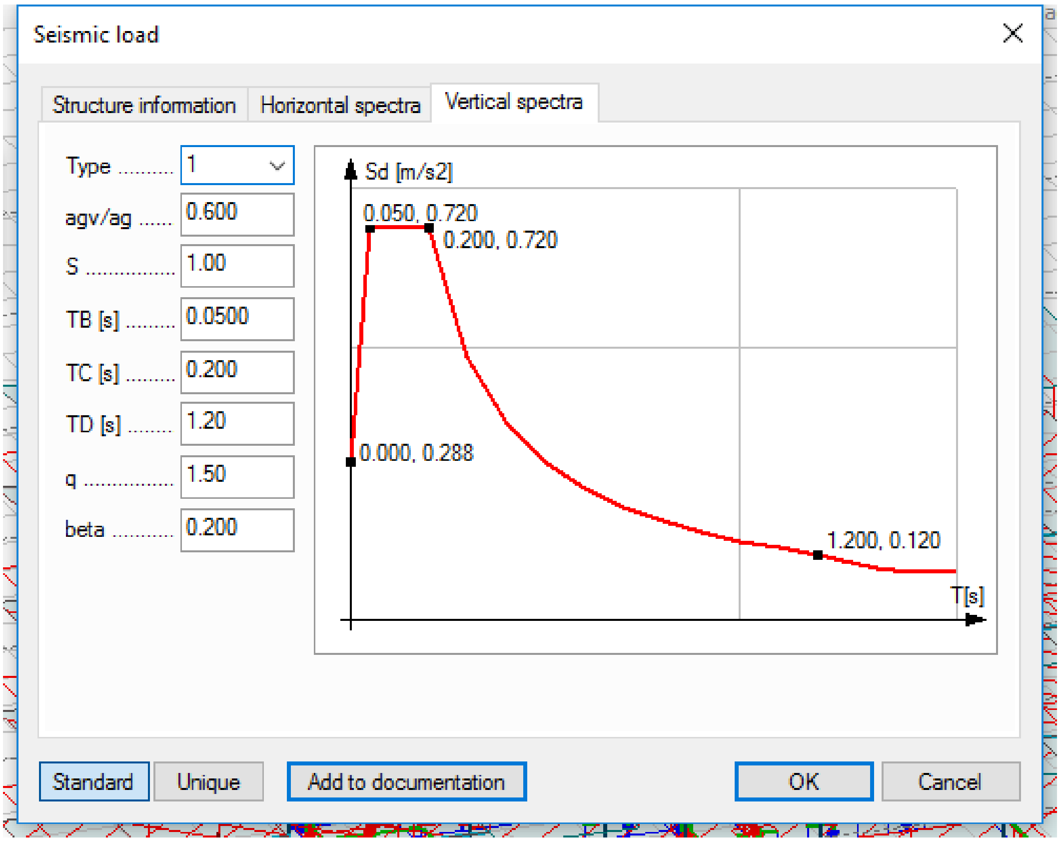


Figure 3-2: Seismic loads in FEM-Design, Vertical spectra

The seismic loads that have been used in FEM-Design analysis, are calculated in Appendix A: Hand calculations. The ground type is in category A see Table A - 8. The ground acceleration a_{g40Hz} = 0.9 m/s^2 from Figure A - 3 with the calculation in Appendix A: Hand calculations is a_g = 0.72 m/s^2. From Table A - 9 with the ground type A, S = 1.0, T_B(s) = 0.10, T_C(s) = 0.25, T_D(s) = 1.5. The ductility factor q = 1.5 from Table A - 10, the building is in a site with low ductility. The dimensioning spectra for elastic analysis β = 0.2 according to (*Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger* 2014). The horizontal spectra is shown in Figure 3-1.

The parameters that describe the vertical spectra is a_g = 0.6, T_B(s) = 0.05, T_C(s) = 0.2, T_D(s) = 1.2 see Table A - 11, so S and q are set as the same as in the horizontal spectra, depicted in Figure 3-2.

According to the Eurocodes, wind and earthquake loading cannot happen at the same time. In Table 3-1 is the mass conversion and wind is set as zero. The other factors is from Table A - 12, ψ_2 category A: indoors residential areas gives the factor 0.3, for snow loads is the factor 0.2. The constructions dead load has a factor on 1.0.

Table 3-1: Load case in FEM-Design.

Factor	Name
1.000	Dead Load
0.300	Payload
0.300	Balcony load
0.200	Snow load
1.000	stair load
--	Wind load north
--	Wind load south
--	Wind load east
--	Wind load west

3.4 Calculation shapes in FEM-Design

When performing seismic analysis using FEM-Design a static linear shape is considered, shown in Figure 3-3 and Figure 3-4

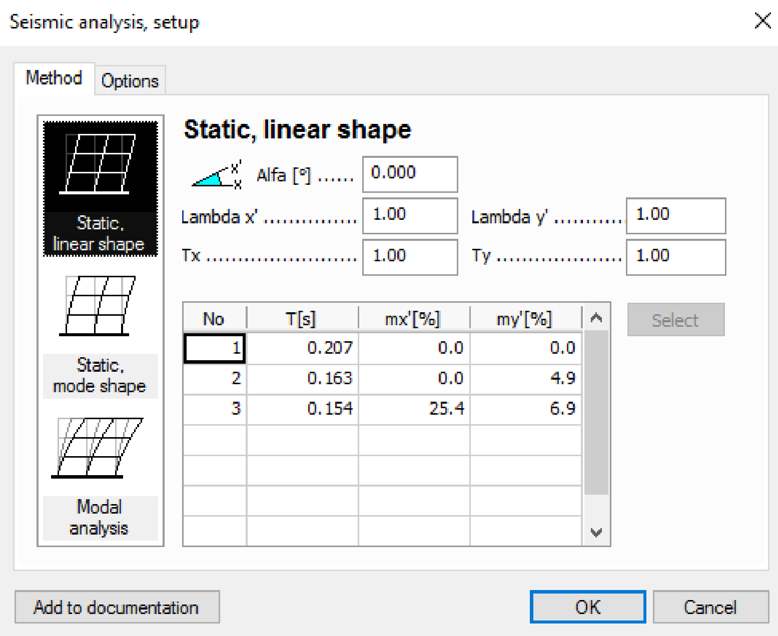


Figure 3-3: Seismic analysis using FEM-Design

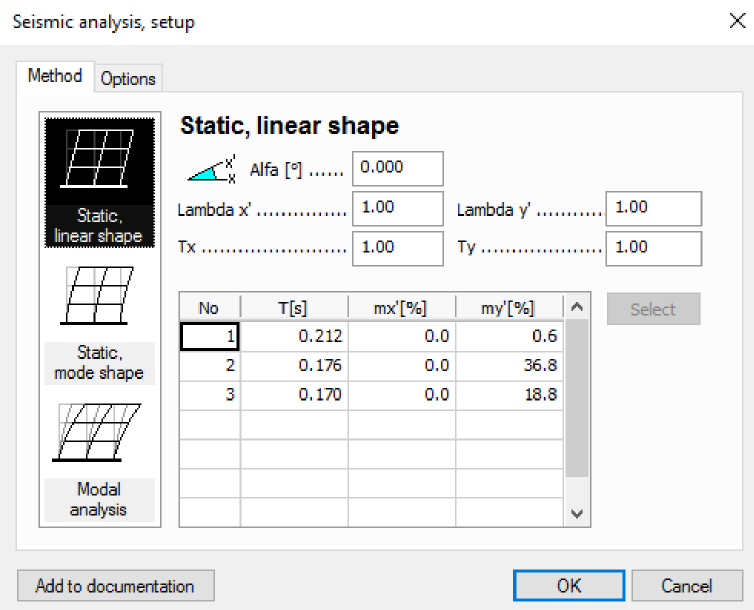


Figure 3-4: Seismic analysis using FEM-Design

4 Results

4.1 Precast element model

The displacements due to seismic analysis are shown in Figure 4-1.

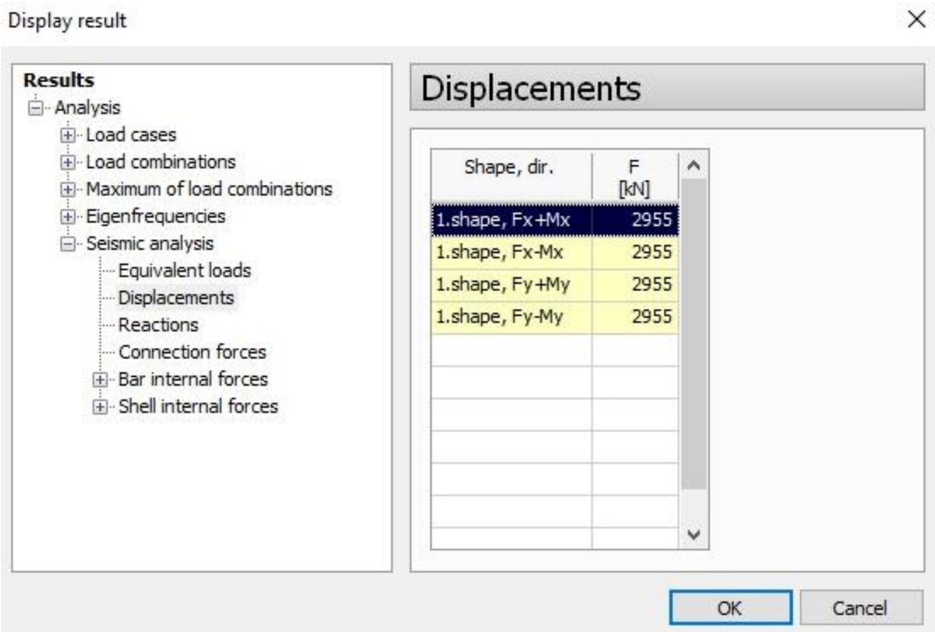


Figure 4-1: Displacements due to seismic analysis

Seismic displacement in the model in $F_x + M_x$ is on the top of the wall in the front to the right, with 1.09mm, according to Figure 4-2

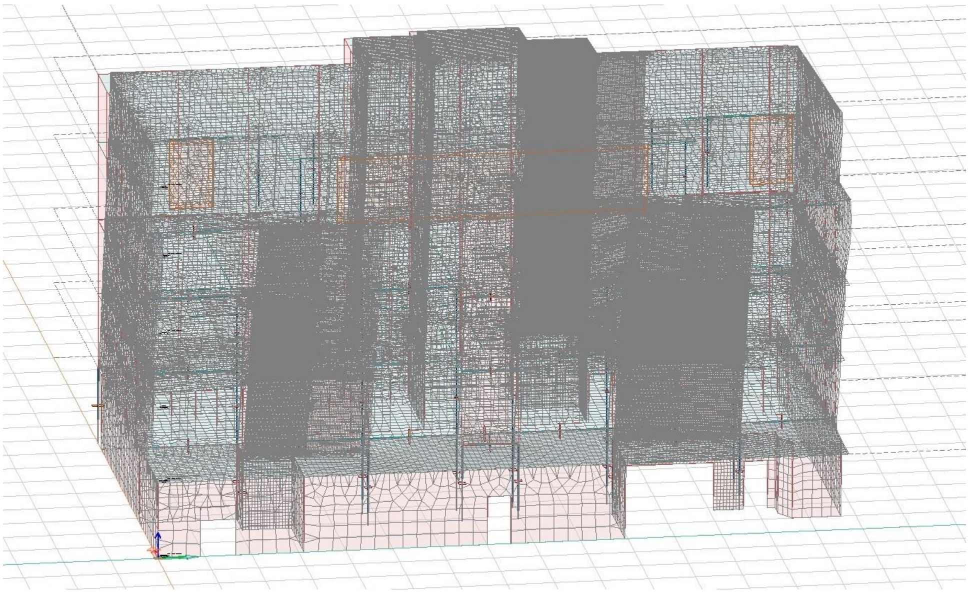


Figure 4-2: Seismic displacements, Fx + Mx

Seismic displacement in the model in F_x - M_x is on the top of the wall in the front to the right, with 0.905mm, according to Figure 4-3.

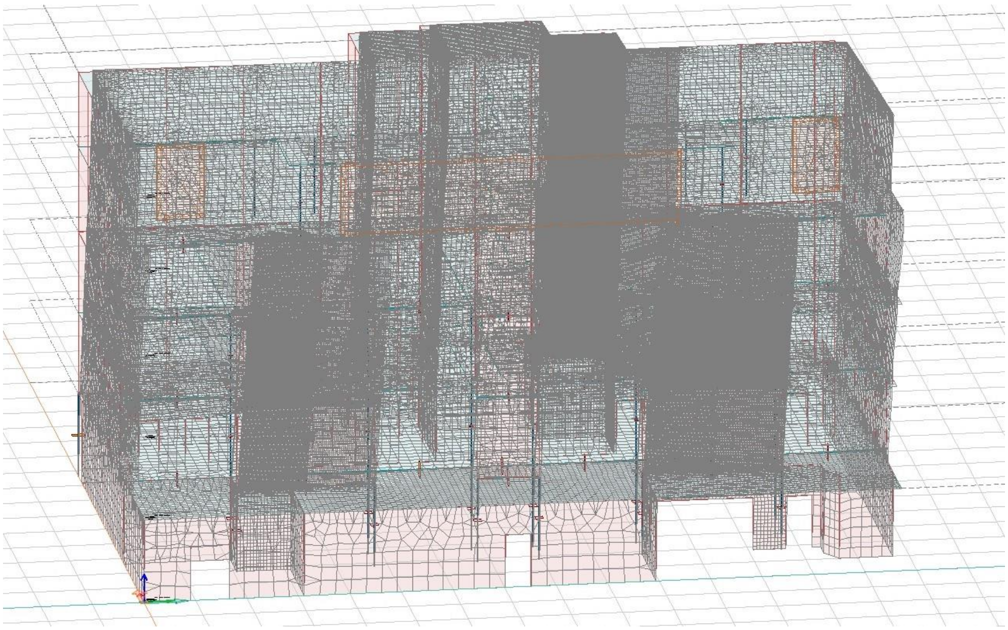


Figure 4-3: Seismic displacements, Fx – Mx

Maximum seismic displacement in the model in F_y + M_y is on the top of the wall in right corner of the staircase, with 1.07mm, according to Figure 4-4.

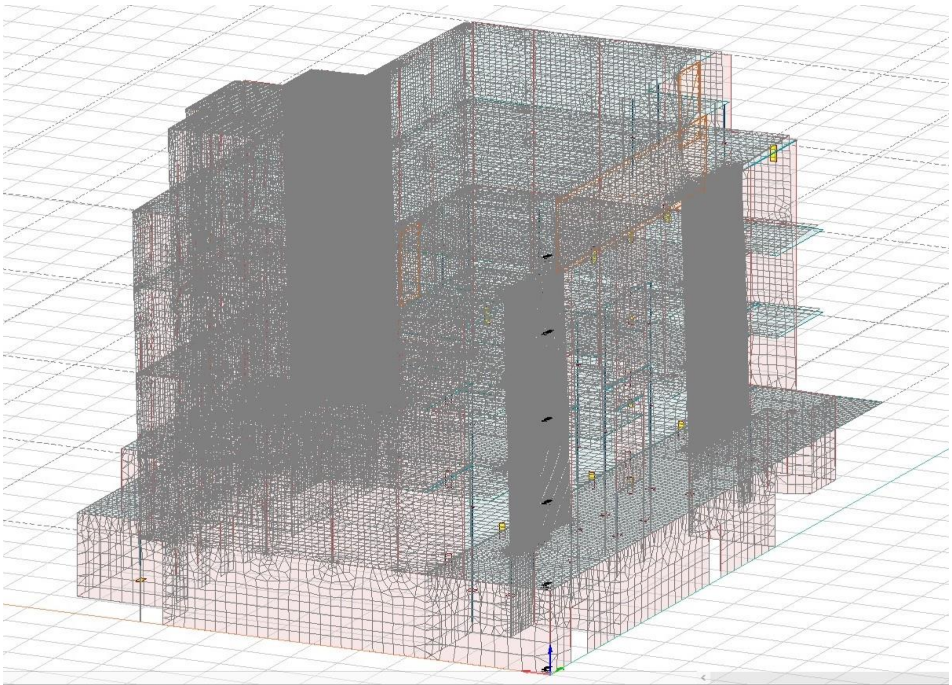


Figure 4-4: Seismic displacements, Fy + My

Seismic displacement in the model in F_y - M_y is on the top of the wall in left corner of the staircase, with 1.08mm, according to Figure 4-5.

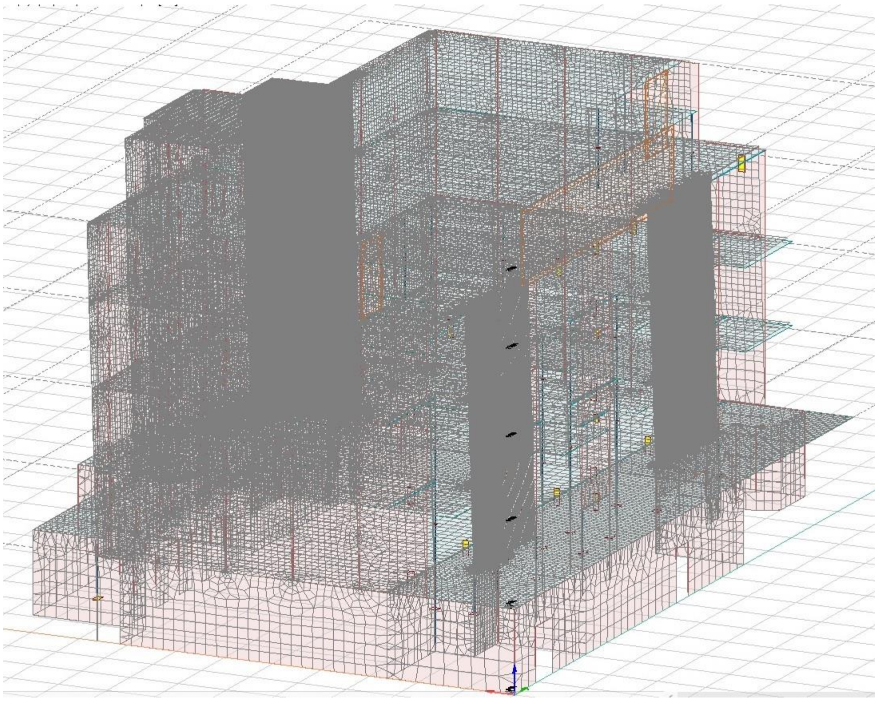


Figure 4-5: Seismic displacements, Fy – My

The eigenfrequencies is shown in Figure 4-6, and the vibration shape in Figure 4-7, 4-8, 4-9.

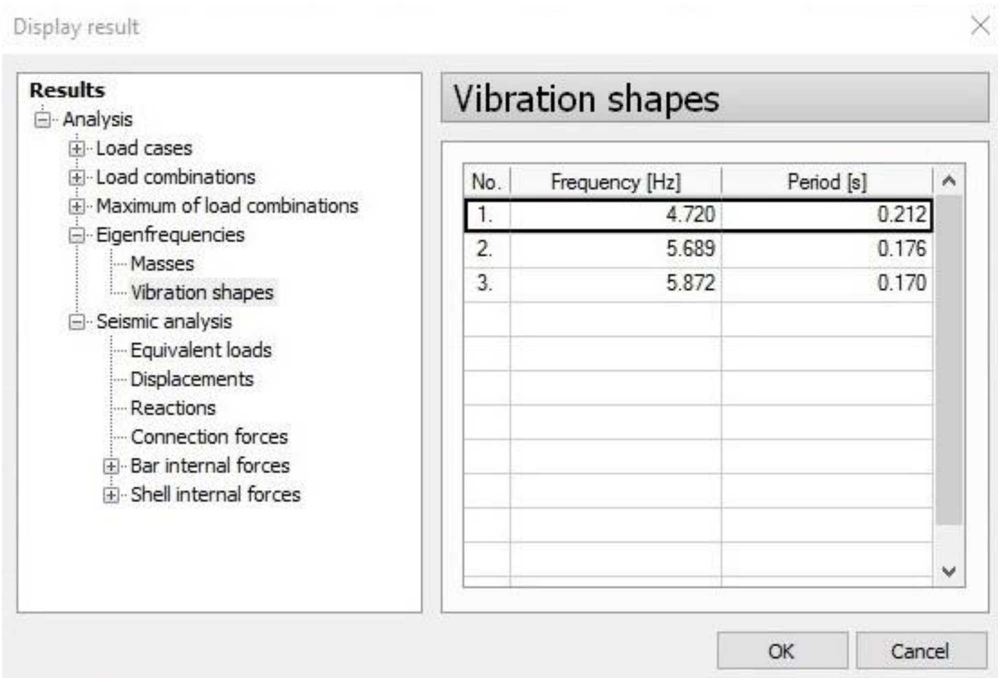


Figure 4-6: Eigenfrequencies, Vibration shapes

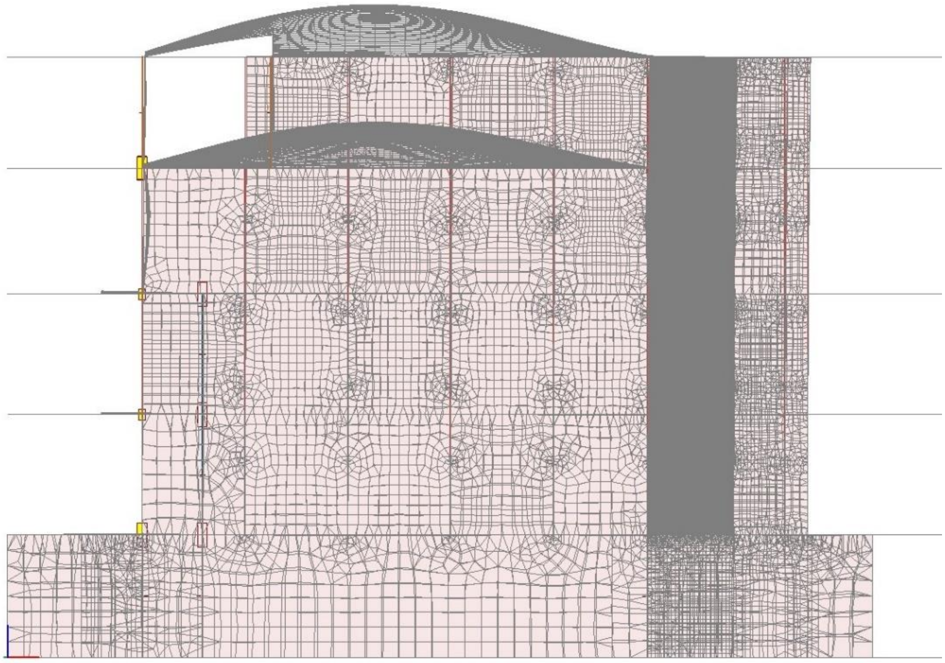


Figure 4-7: Vibration shape 1

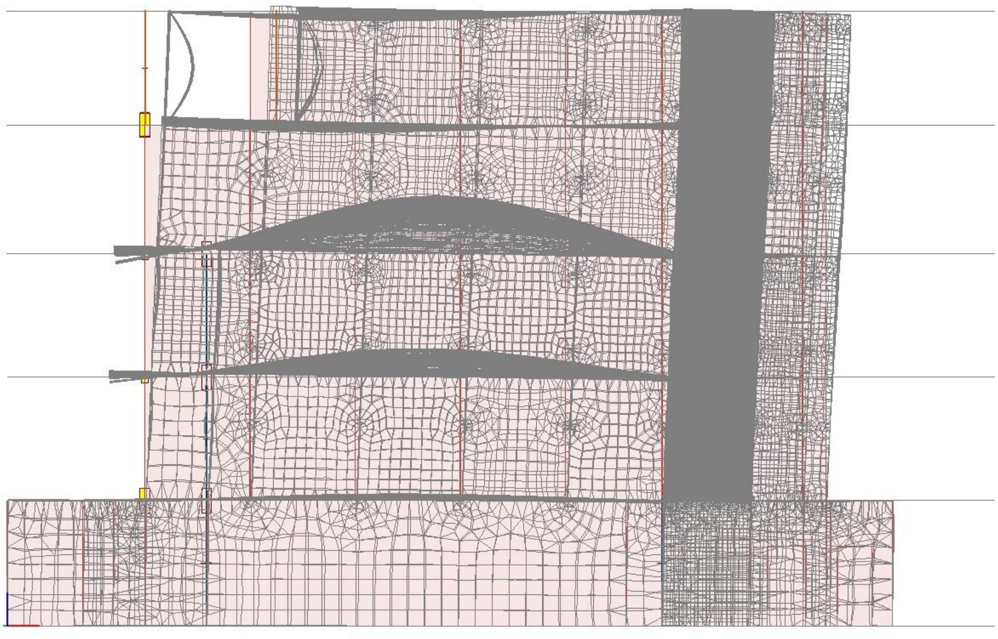


Figure 4-8: Vibration shape 2

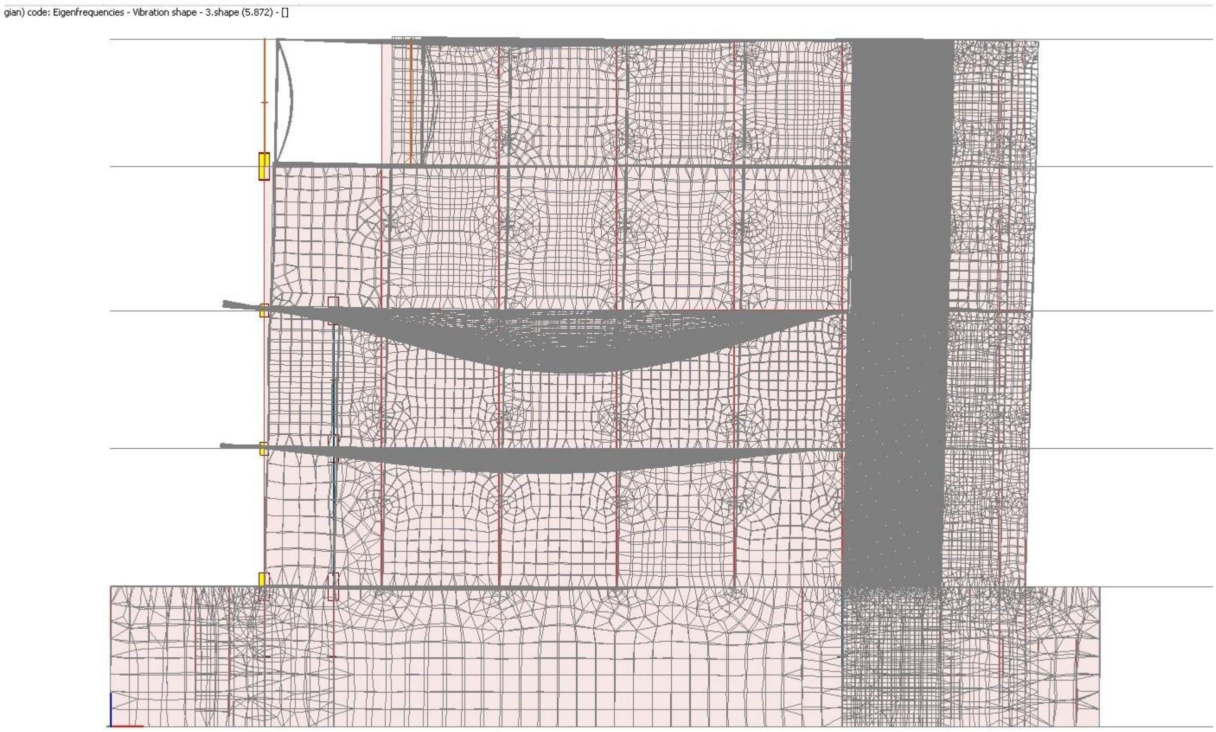


Figure 4-9: Vibration shape 3

The normal internal forces in the bar elements are shown in Figure 4-10, and the distribution of the normal forces in Figure 4-11.

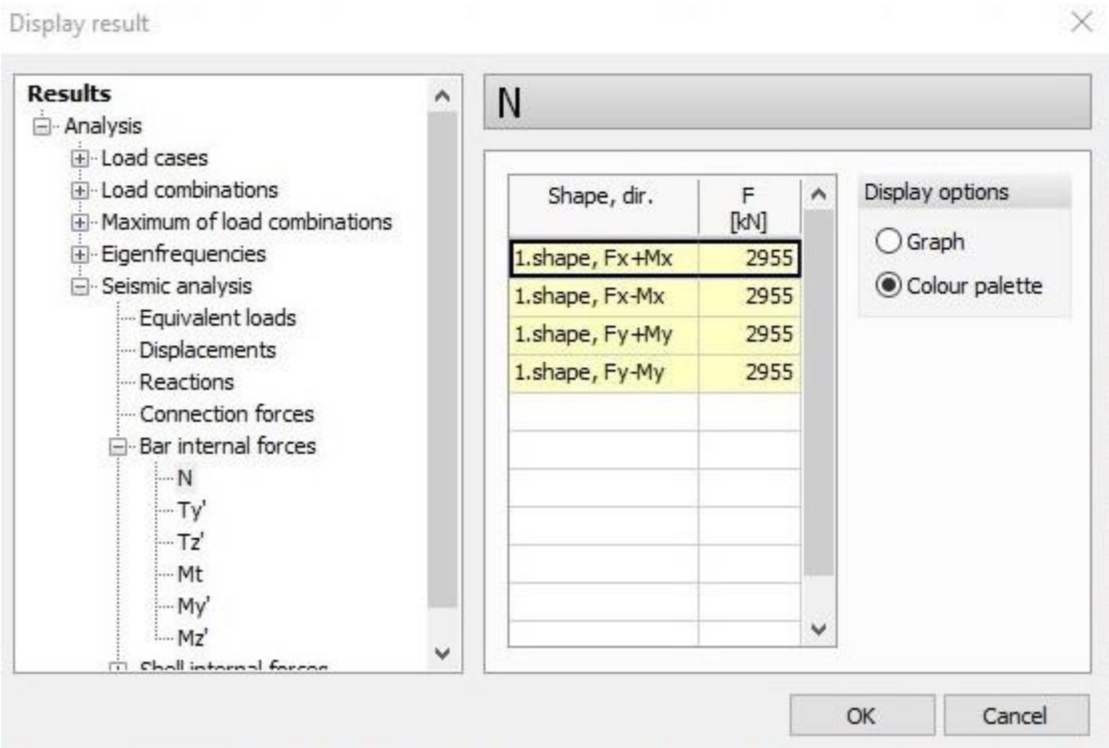


Figure 4-10: Bar internal forces

The maximum internal normal force is in the columns are in the back with a force on 113 kN, according to Figure 4-11.

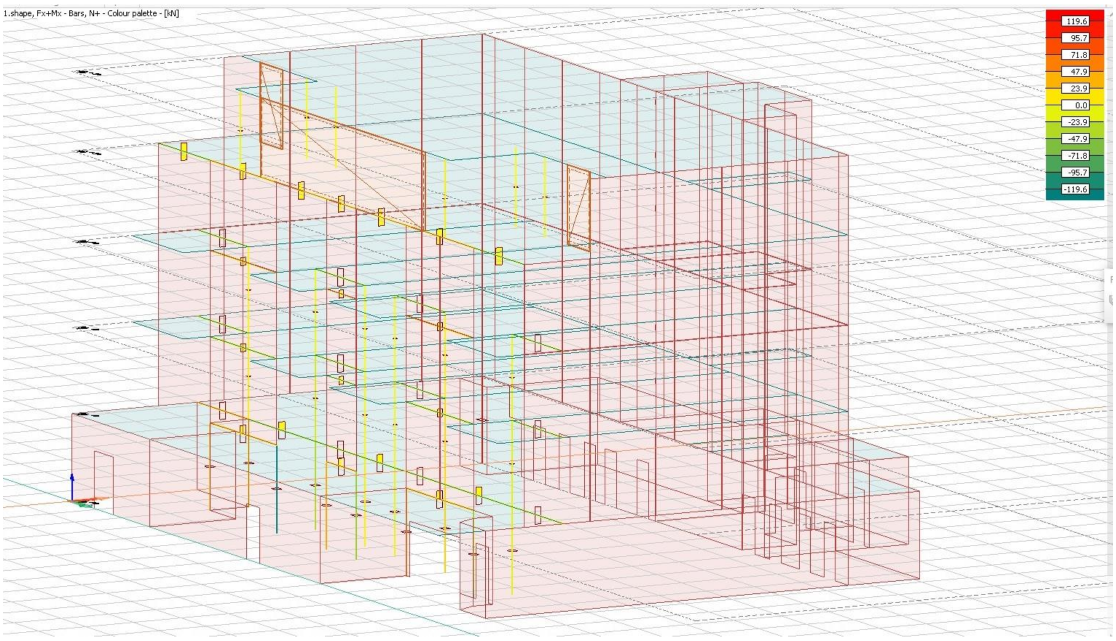


Figure 4-11: Normal internal forces

The internal forces in the shell elements are shown in Figure 4-12, and the distribution of the forces in Figure 4-13.

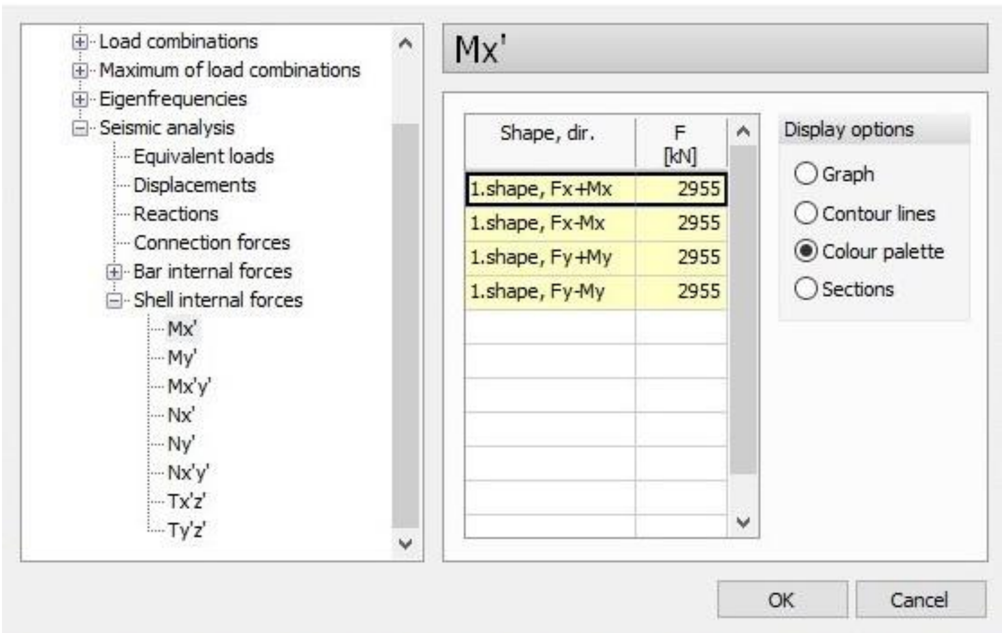


Figure 4-12: Shell internal forces

The maximum internal force M_x, in the shell elements are in the top plate with a force on 13 kN, according to Figure 4-13.

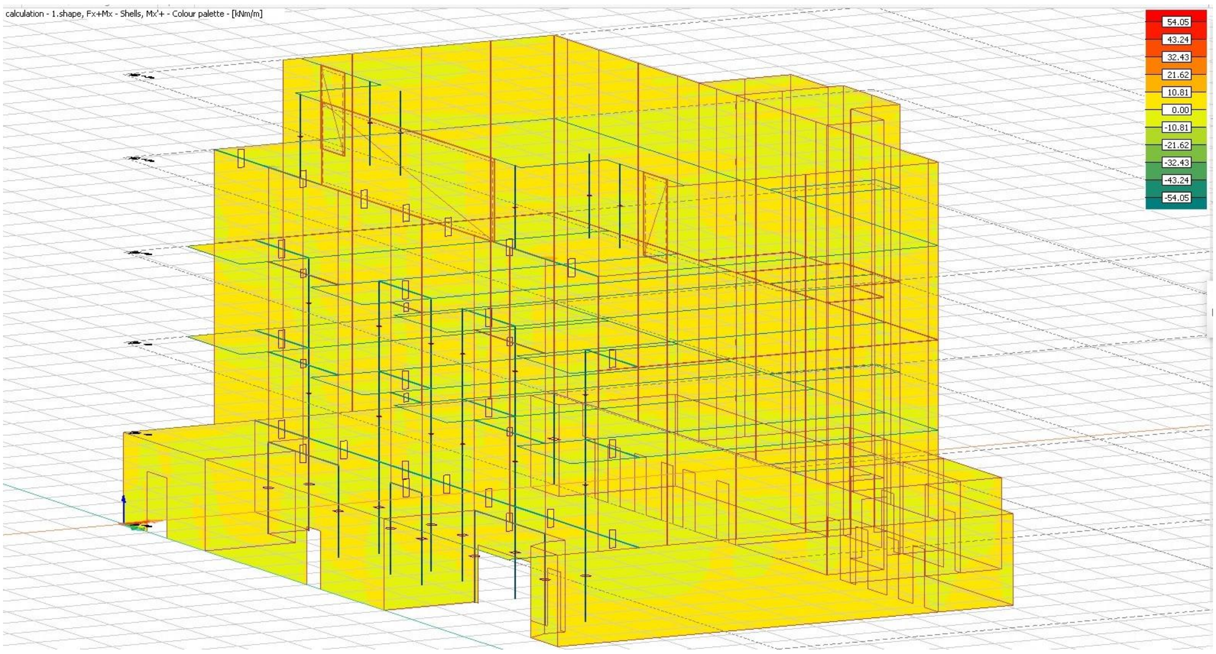


Figure 4-13: Shell internal forces

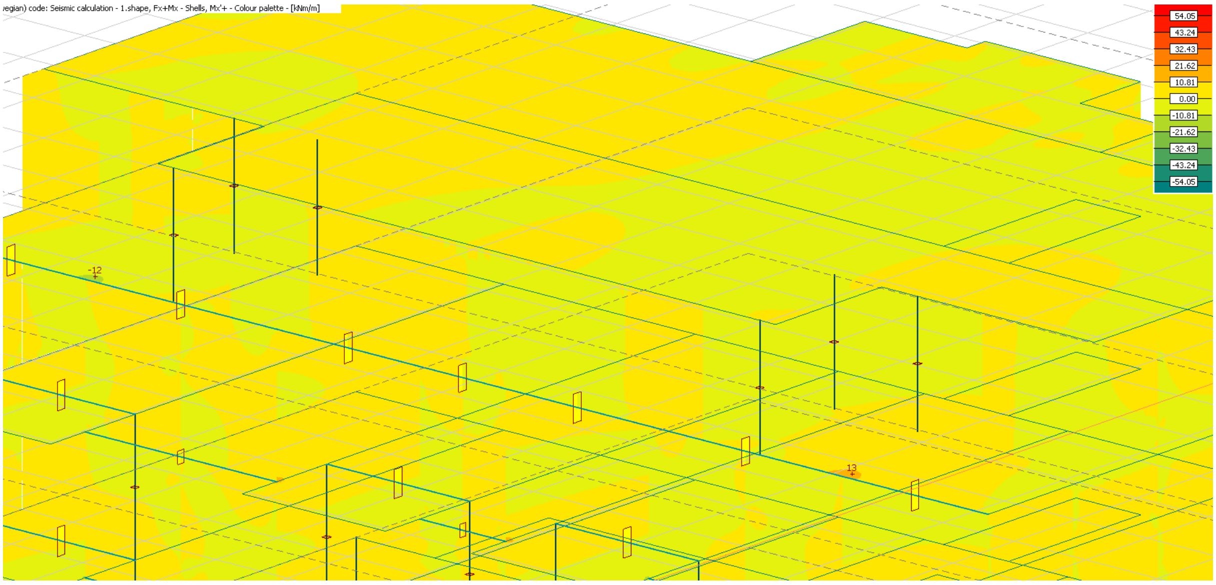


Figure 4-14: Shell internal forces with distribution

4.2 On-site casted model

The displacements due to seismic analysis are shown in Figure 4-15.

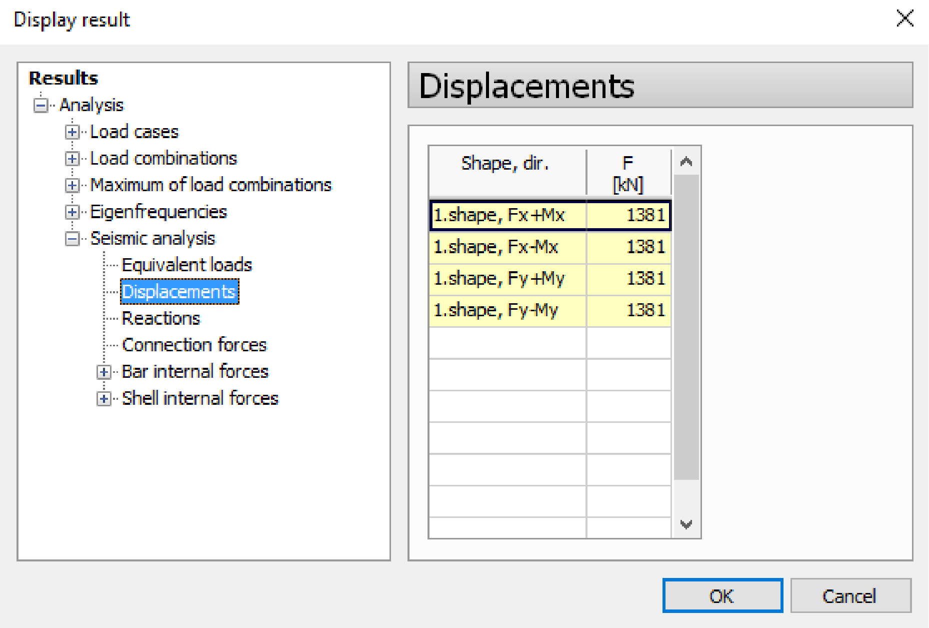


Figure 4-15: Displacements due to seismic analysis

Seismic displacement in the model in $F_x + M_x$ is on the top of the wall in the front to the right, has been found equal to 0.438mm, as shown in Figure 4-16.

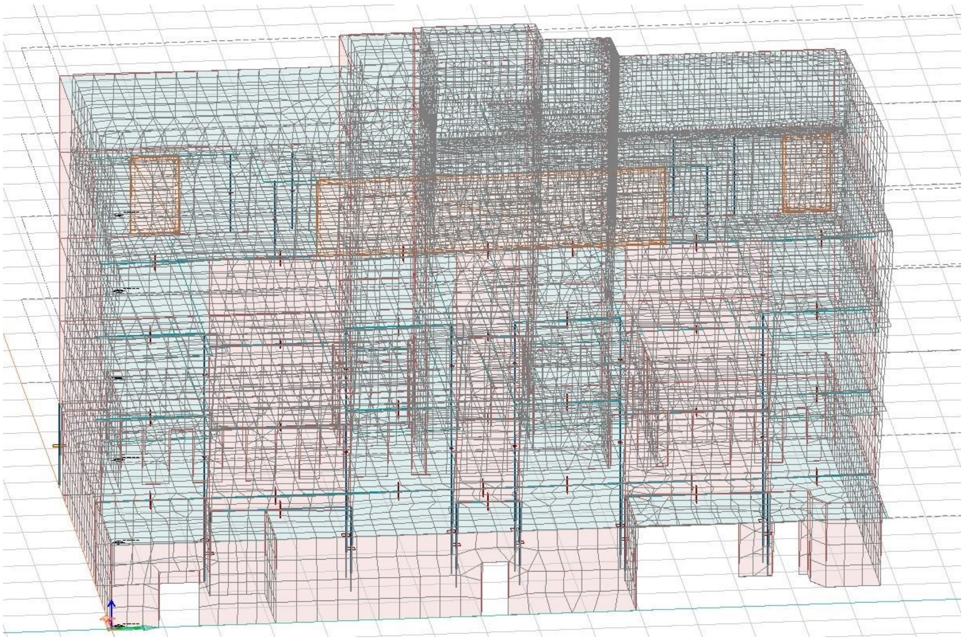


Figure 4-16: Seismic displacements, Fx + Mx

Seismic displacement in the model in F_x - M_x is on the top of the wall in the front to the right, has been found equal to 0.36mm, as shown in Figure 4-17.

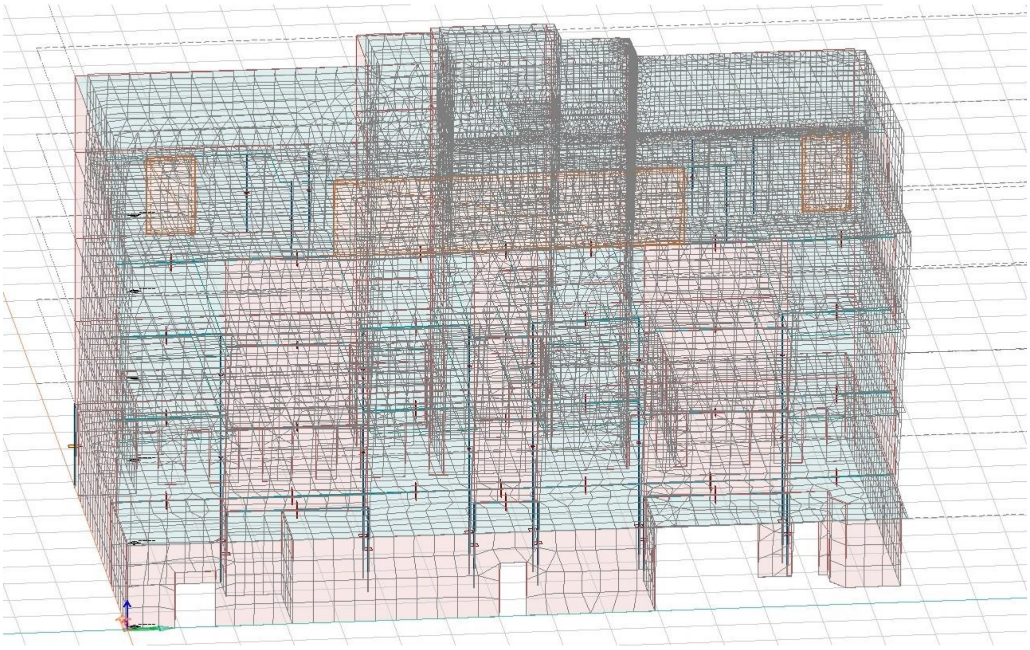


Figure 4-17: Seismic displacements, Fx – Mx

Seismic displacement in the model in F_y + M_y is on the top of the top of the wall in right corner of the staircase, is found equal to 0.391mm, according to Figure 4-18.

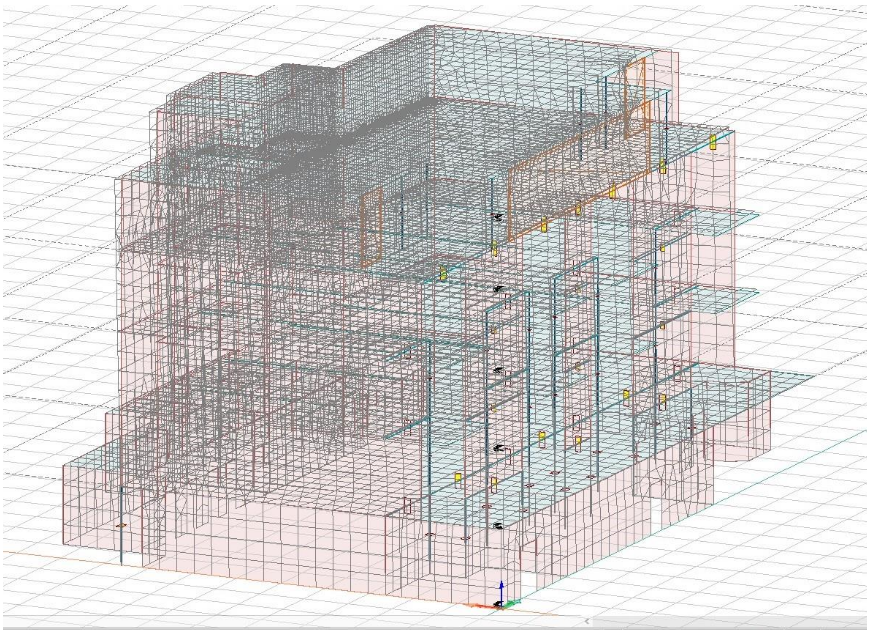


Figure 4-18: Seismic displacements, Fy + My

Seismic displacement in the model in F_y - M_y is on the top of the top of the wall in left corner of the staircase is equal to 0.391mm, as depicted in Figure 4-19.

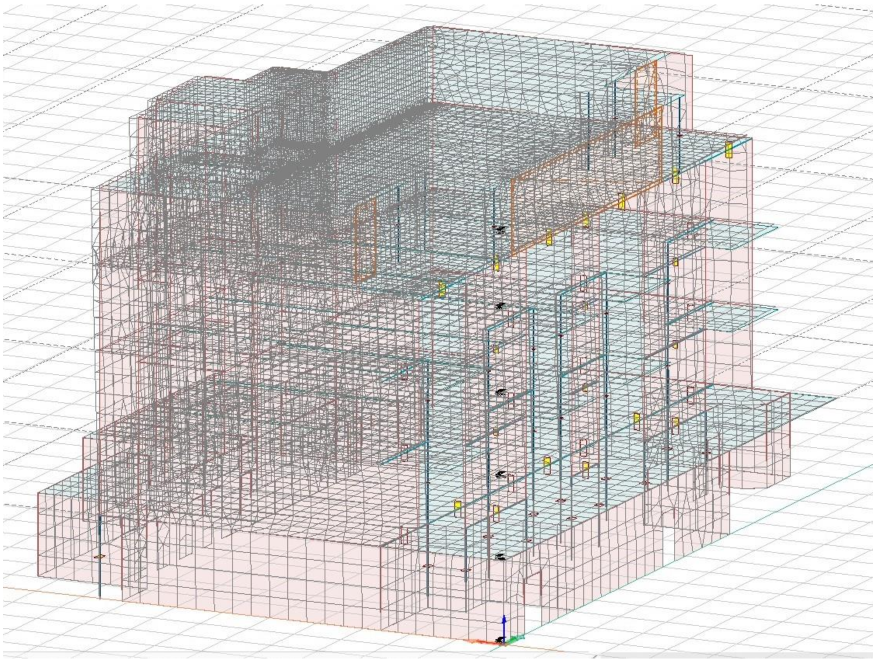
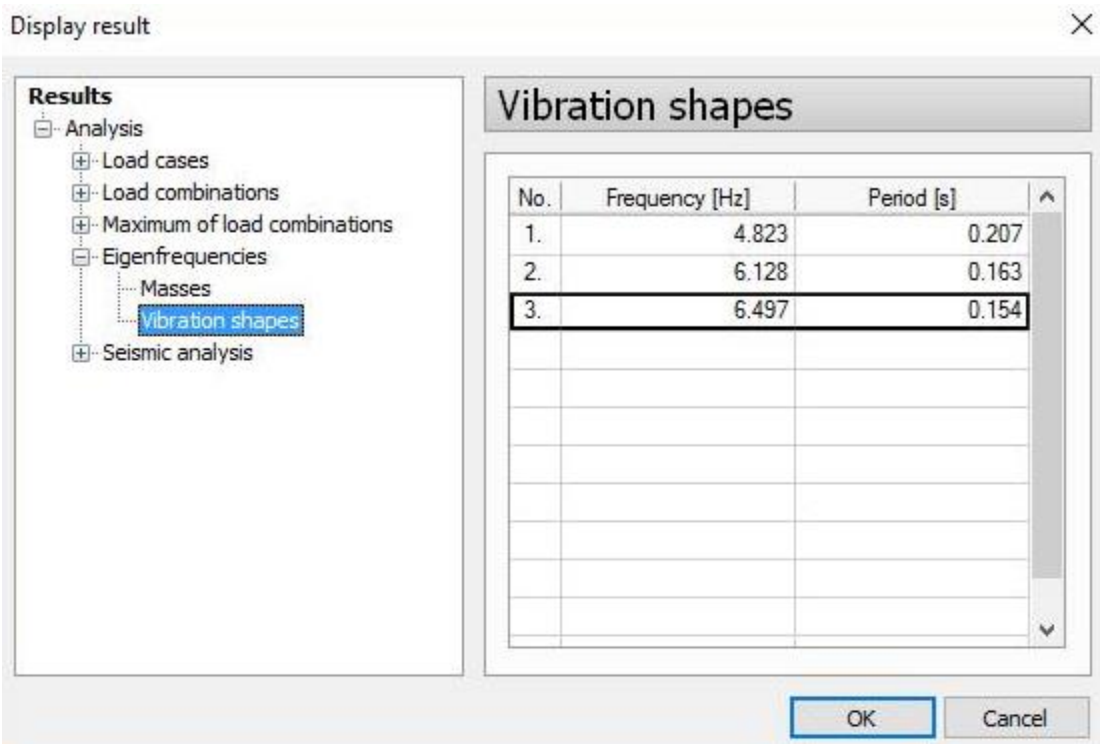


Figure 4-19: Seismic displacements, Fy - My

The eigenfrequencies are shown together with the vibration shape in Figure 4-20, Figure 4-21, Figure 4-22 and Figure 4-23.



No.	Frequency [Hz]	Period [s]
1.	4.823	0.207
2.	6.128	0.163
3.	6.497	0.154

Figure 4-20: Eigenfrequencies and Vibration shapes

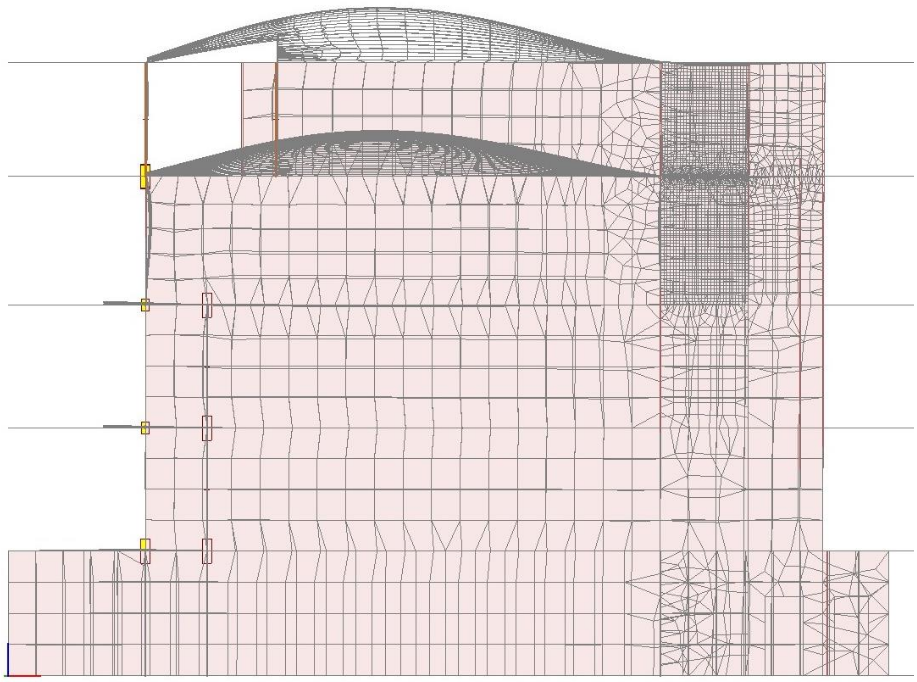


Figure 4-21: Vibration shape 1

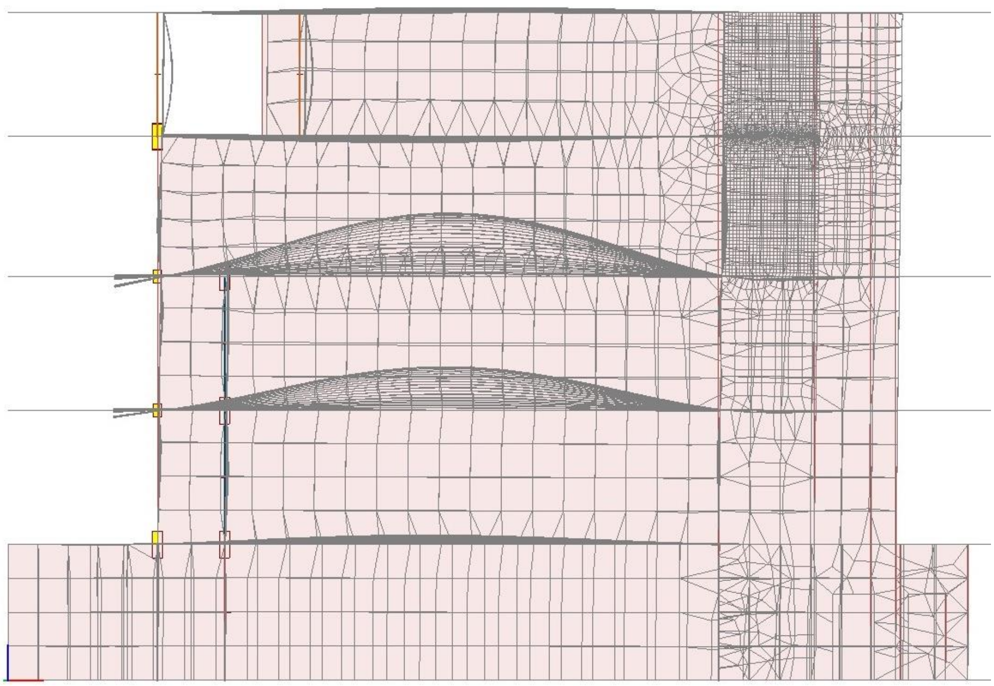


Figure 4-22: Vibration shape 2

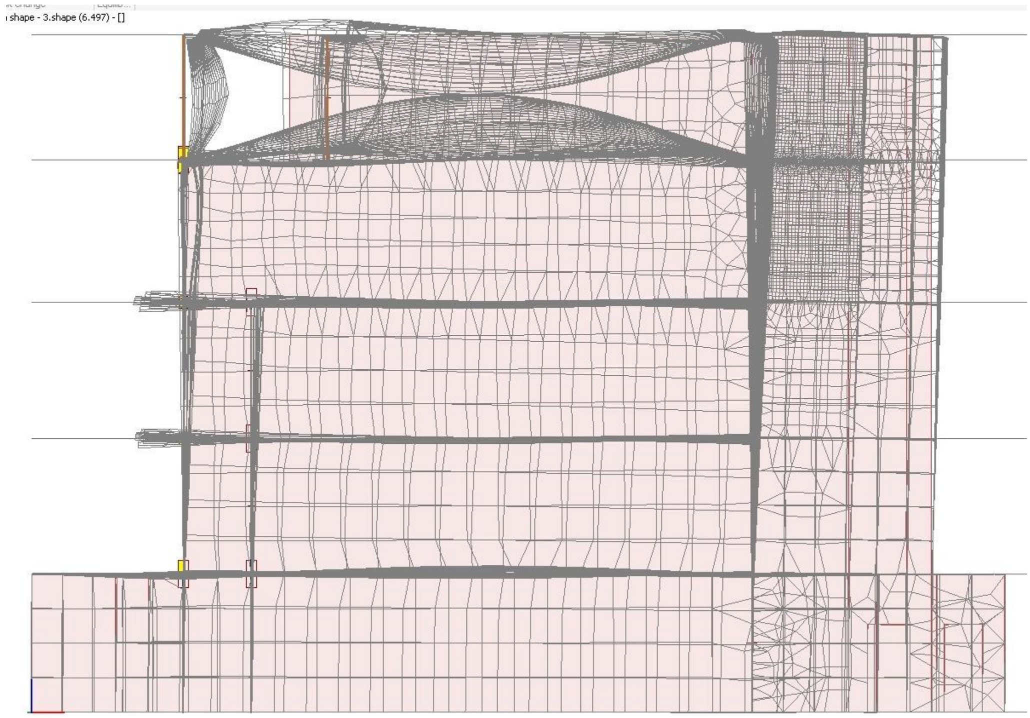


Figure 4-23: Vibration shape 3

The normal internal forces are depicted in Figure 4-24 and the distribution of the normal forces is shown in Figure 4-25

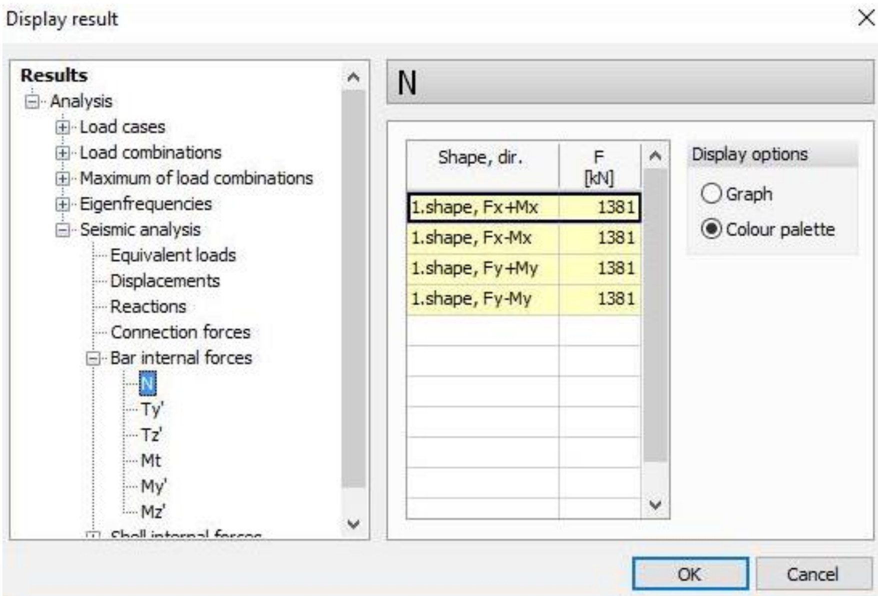


Figure 4-24: Normal internal forces

The maximum internal normal force is in the column in the back with a force on 61,7kN, according to Figure 4-25.

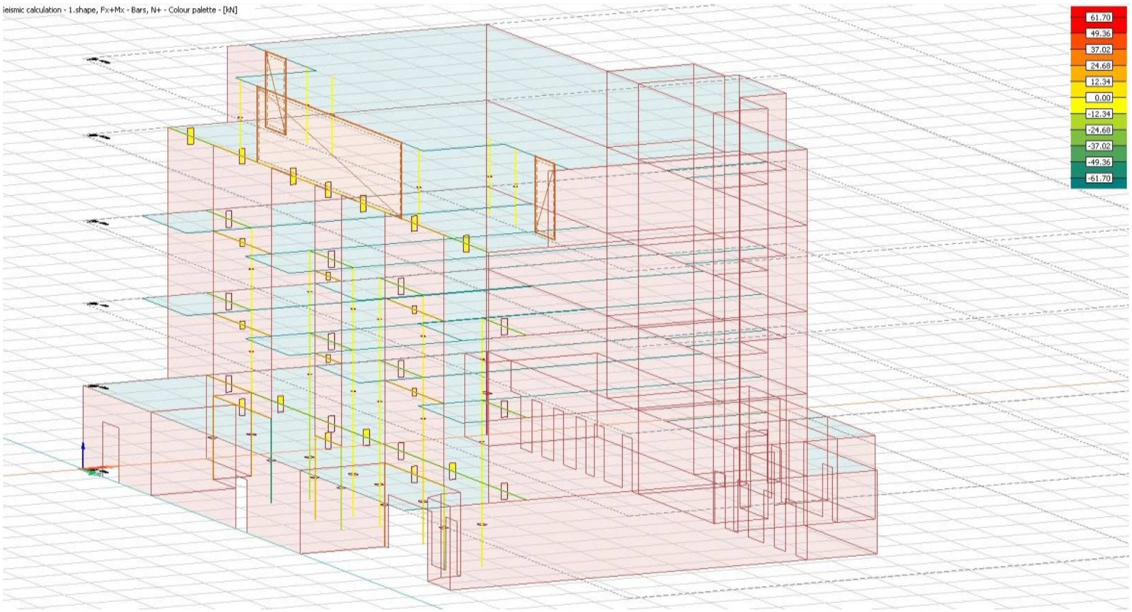


Figure 4-25: Normal internal forces

The internal forces in the shell elements are shown in Figure 4-26, and the distribution of the forces in Figure 4-27.

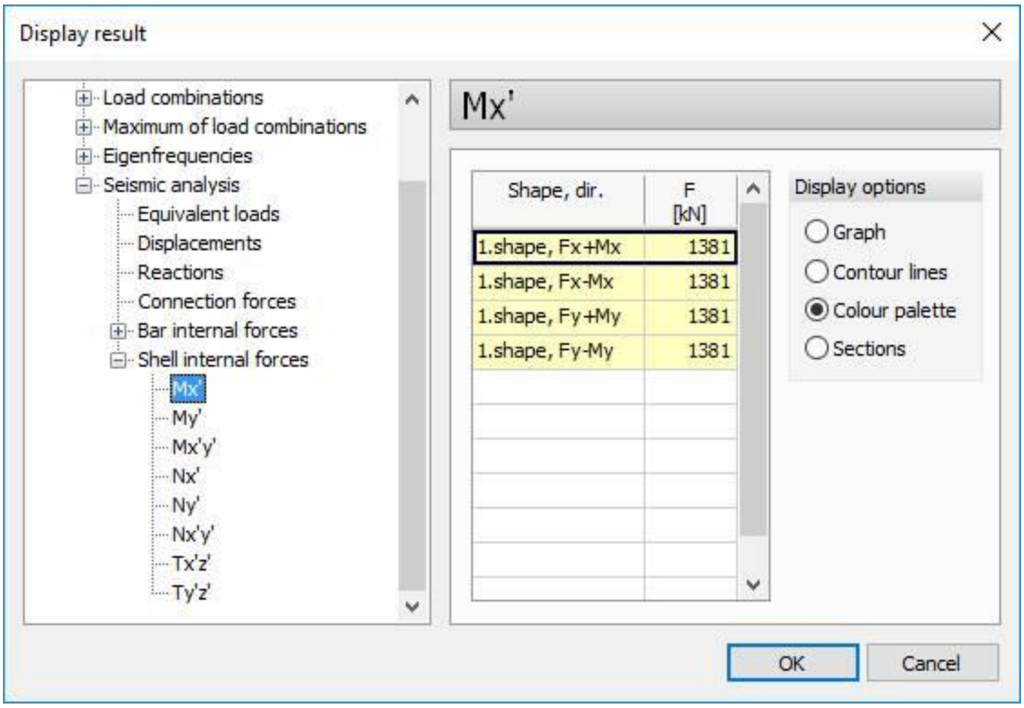


Figure 4-26: Shell internal forces

The maximum internal force M_x, in the shell elements are in the top plate with a force equal of 6 kN, according to Figure 4-27.

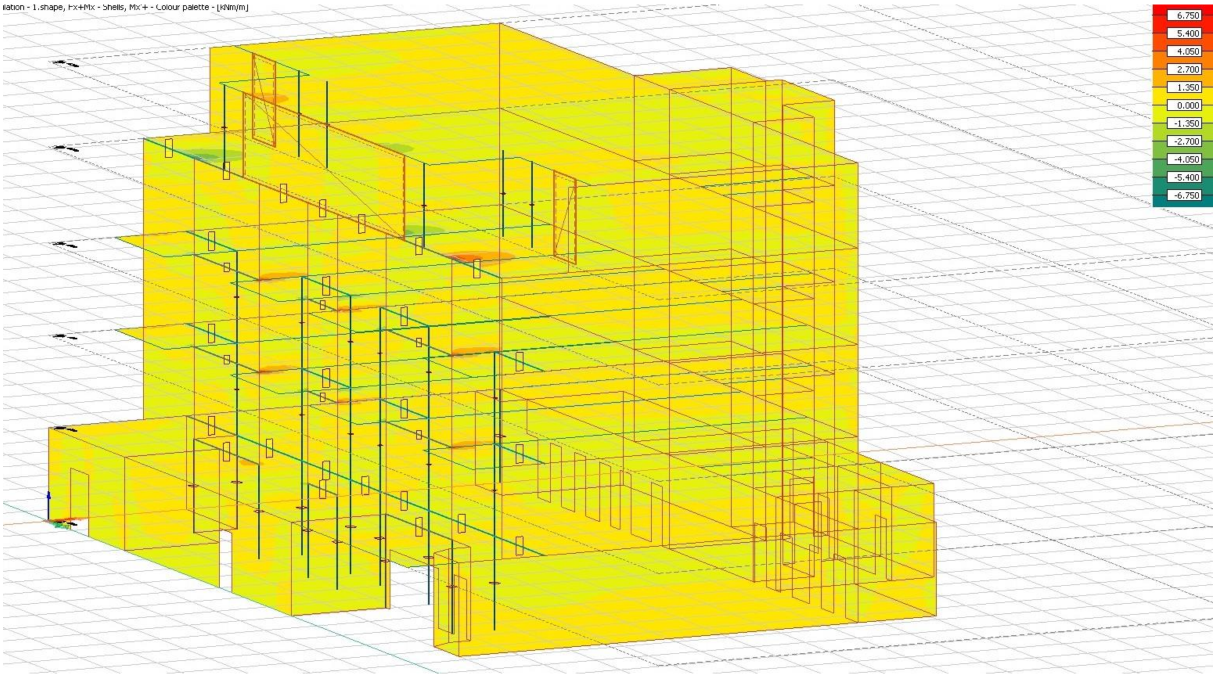


Figure 4-27: Shell internal forces

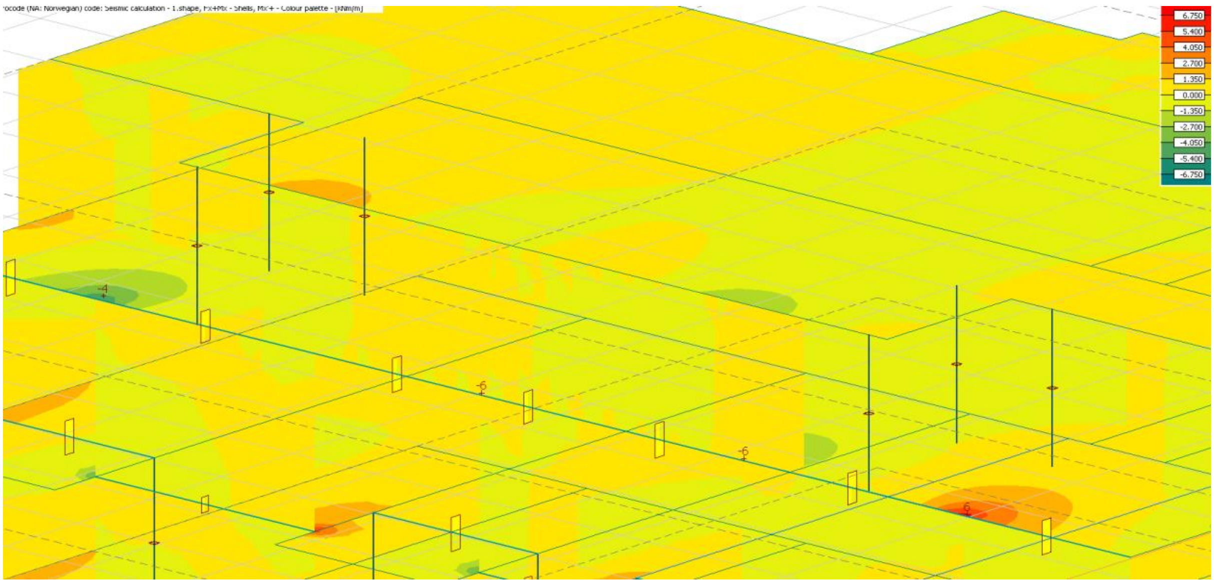


Figure 4-28: Shell internal forces distribution

5 Discussion

Both the models are classified as grade 1 according to Table 2-3. This means that the models behave extremely well, with no or slight structural damage, during a moderate earthquake event. In case a larger earthquake occurs, it will probably be at grade 2, which is still acceptable.

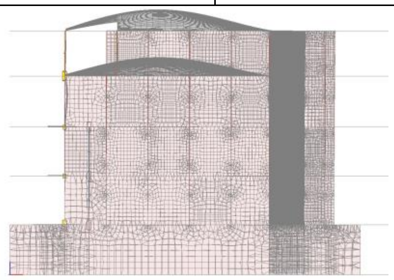
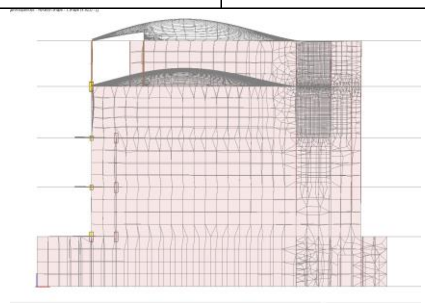
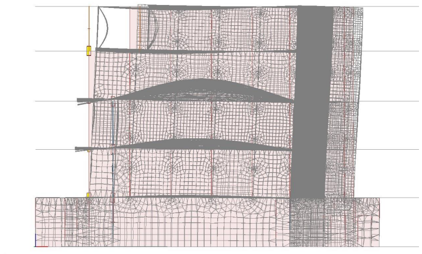
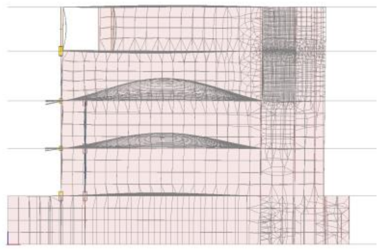
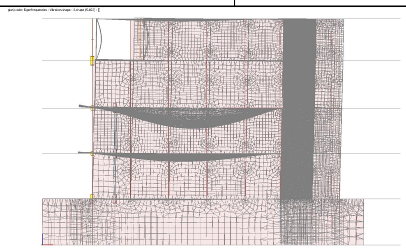
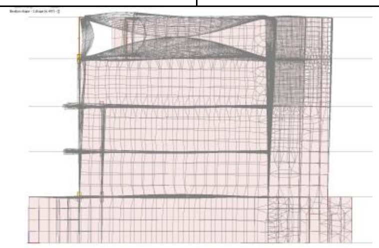
In the tables below, there is a comparison between the calculated displacements, vibration shapes and normal internal forces for the two models. There is a slight difference in the two models, but it is not big enough to have fatal consequences.

Table 5-1: Displacements comparison

	Precast element model	On-site casted model
Displacement force	2955 kN	1381 kN
Displacements Fx + Mx	1.090mm	0.438mm
Displacements Fx - Mx	0.905mm	0.360mm
Displacements Fy + My	1.070mm	0.391mm
Displacements Fy - My	1.080mm	0.391mm

The displacements found in both of the models are quite small. The on-site casted model is more rigid and has smaller displacements than the prefabricated element model.

Table 5-2: Eigenfrequencies, Vibration shapes, comparison

Vibration shapes	Precast element model		On-site casted model	
Number	Frequency[Hz]	Period [s]	Frequency[Hz]	Period [s]
1.	4.720	0.212	4.823	0.207
				
2.	5.689	0.176	6.128	0.163
				
3.	5.872	0.170	6.497	0.154
				

The vibrations shape is quite similar in both models. The time span in the analysis has been found to be very low, which is beneficial for the structural response. If the periods were longer, then the material may be damaged or cracked due to fatigue. The two first vibration shapes are very similar in both cases.

The precast element model has a maximum displacement equal to 71mm in the roof and 62mm in the top slab. The on-site casted model have a maximum displacement equal to 74mm in the roof and 61mm in the top slab, as shown in

Table 5-2. During the second vibration shape, the maximum displacements are located in the middle floors, as presented in Table 5-2. The slab with the biggest displacement is equal to 52mm in the prefabricated element model, and equal to 77mm in the on-site casted model. In the third vibration shape the models act a bit different. In the prefabricated element model there is a maximum displacement equal to 74mm in the middle slab. Whereas the on-site casted model has a displacement of 52mm downwards in the top slab and 55mm upwards in the next upper slab.

Table 5-3: Normal internal forces in bar elements comparison

	Precast element model	On-site casted model
Bar internal forces	2955 kN	1381 kN
Column in the back	113 kN	61.7 kN
Column in front	-120 kN	-52.3 kN

In the normal internal forces in the bars, the largest force is in the column in the back, which is marked red in Figure 4-11 and Figure 4-25.

Table 5-4: Shell internal forces comparison

	Precast element model	On-site casted model
Upper slab red mark	13 kN	6 kN
Upper slab green mark	-12 kN	-6 kN

In the shell internal forces the slabs have the largest force. The largest force is in the next upper slab in the front with the balconies. The largest displacements are marked in red for positive and green for the opposite force.

If the models were to be tested for larger earthquakes, I believe that they would perform well, but I expect there to be some moderate damages. To improve the model's behaviour, if they were to be placed in a more earthquake prone area, I would suggest moving the staircase in to the rectangular building. Fewer corners are better. I would also change the formation of the concrete columns in the ground floor, with a single concrete wall, so all the loads are transferred to the ground without any deviation. I would also take out the column at the back, which is marked with red in Figure 4-11 and Figure 4-24; to optimize the walls and make them into a corner.

The prefabricated element building has been found to be the best construction technique for the building study. The reason is that there are only moderate earthquakes in the area, and the walls can safely resist the seismic loads. The weak points of the building is the steel connections between the precast elements. Therefore, the building parts need to be assembled properly in order to minimize this disadvantage. Taken into the consideration the cost and the climate conditions in Norway, the prefabricated element building is definitely the best choice.

6 Conclusion

Overall, the seismic performance of the building under investigation has been found as very good due to the design techniques and the modern materials. Of course, there is always room for improvement.

Some changes in the building design that would improve the seismic behaviour of the structure are:

1) Eliminate the plan and the vertical irregularities of the building.
2) Eliminate asymmetrical horizontal bracing.
3) Eliminate/ minimize discontinuities in stiffness and resistance.

7 Further work

For future work, it would be intriguing to analyse the building for larger earthquakes compared to those occurring in Norway. This hypothesis would investigate the limits of the building, both in terms of ductility and capacity.

Another idea is to model and analyse the building with the new improved characteristics, as suggested in the previous paragraph. A modal analysis of the building where different connections for the wall elements are investigated is also challenging.

Finally, another option is to study the same building at a location with other ground characteristics, and see the difference at the seismic behaviour. This would be the worst case scenario for Norway.

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Appendix A: Hand calculations

A.1 Payload

Table A - 1: Use Categories (Eurokode 1 : Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-1: General actions : Densities, self-weight, imposed loads for buildings : Del 1-1 : Allmenne laster : Tetthet, egenvekt og nyttelaster i bygninger 2008)

Kategori	Spesifikk bruk	Eksempel
A	Arealer for inneaktiviteter og hjemmeaktiviteter	Rom i boligbygg og hus; sengerom og behandlingsrom i sykehus; soverom i hoteller og gjestgiverier; kjøkken og toaletter.
B	Kontorarealer	
C	Arealer der personer kan samles (med unntak av arealer som er definert i kategori A, B og D[1])	**C1**: Arealer med bord osv., f.eks. i skoler, kafeer, restauranter, spisesaler, leserom, resepsjoner osv.
		C2: Arealer med faste seter, f.eks. arealer i kirker, teatre eller kinosaler, konferanserom, forelesningssaler, forsamlingssaler, venterom medregnet forhall på jernbanestasjoner osv.
		C3: Arealer uten hindringer for personer i bevegelse, f.eks. arealer i museer, utstillingsrom osv., og ankomstområder i offentlige bygg og administrasjonsbygg, hoteller, sykehus, jernbanestasjonshaller.
		C4: Arealer med muliget for fysiske aktiviteter, f.eks. dansesaler, gymnastikkrom, scener osv.
		C5: Arealer som lett overfylles, f.eks. i bygg for offentlig bruk, som konsertsaler, idrettshaller medregnet tribuner og atkomstområder og jernbaneperronger.
D	Forretningsarealer	**D1**: Arealer i vanlig detaljhandel.
		D2: Arealer i varehus.

[1] Det gjøres oppmerksom på 6.3.1.1(2), særlig for C4 og C5. Se NS-EN 1990 når det må tas hensyn til dynamiske effekter. For kategori E, se tabell 6.3.

MERKNAD 1 Oppdragsgiver og/eller det nasjonale tillegget kan fastsette at arealer som normalt kan settes i kategori C2, C3, C4, avhengig av bruk, kan settes i kategori C5.

MERKNAD 2 Underkategorier til A, B, C1 til C5, D1 og D2 kan gis i det nasjonale tillegget.

MERKNAD 3 Se 6.3.2 for lagrings- eller industrivirksomhet.

Table A - 2: Payloads on floors (Eurokode 1 : Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-1: General actions : Densities, self-weight, imposed loads for buildings : Del 1-1 : Allmenne laster : Tetthet, egenvekt og nyttelaster i bygninger 2008)

Kategorier for belastede områder	**q_k [kN/m^2]**	**Q_k [kN]**
Kategori A		
– Gulv	1,5 til 2,0	2,0 til 3,0
– Trapper	2,0 til 4,0	2,0 til 4,0
– Balkonger	2,5 til 4,0	2,0 til 3,0
Kategori B	2,0 til 3,0	1,5 til 4,5
Kategori C		
– C1	2,0 til 3,0	3,0 til 4,0
– C2	3,0 til 4,0	2,5 til 7,0 (4,0)
– C3	3,0 til 5,0	4,0 til 7,0
– C4	4,5 til 5,0	3,5 til 7,0
– C5	5,0 til 7,5	3,5 til 4,5
Kategori D		
– D1	4,0 til 5,0	3,5 til 7,0 (4,0)
– D2	4,0 til 5,0	3,5 til 7,0

A.2 Snow load

Table A - 3: Snow loads in Herøy municipalities, (Eurokode 1: : Laster på konstruksjoner. Del 1-3. Allmenne laster. Snølaster = Eurocode 1: Actions on structures : Part 1-3: General actions, Snow loads 2008)

Halsa	4,5	150	1,0	–
Haram	2,5	150	1,0	–
Hareid	3,0	150	1,0	–
Herøy	2,5	150	1,0	–
Kristiansund	2,5	150	1,0	–
Midsund	3,0	150	1,0	–
Molde	3,5	150	1,0	–

$s = \mu * s_k$

$\mu = 0,8$ ($0^o \leq \alpha \leq 30^o$)

$s_{k,0} = 2,5$ kN/m2

$s = 0,8 * 2,5 = \underline{2,0 \text{ kN/m2}}$

A.3 Snow load on the lower balconies

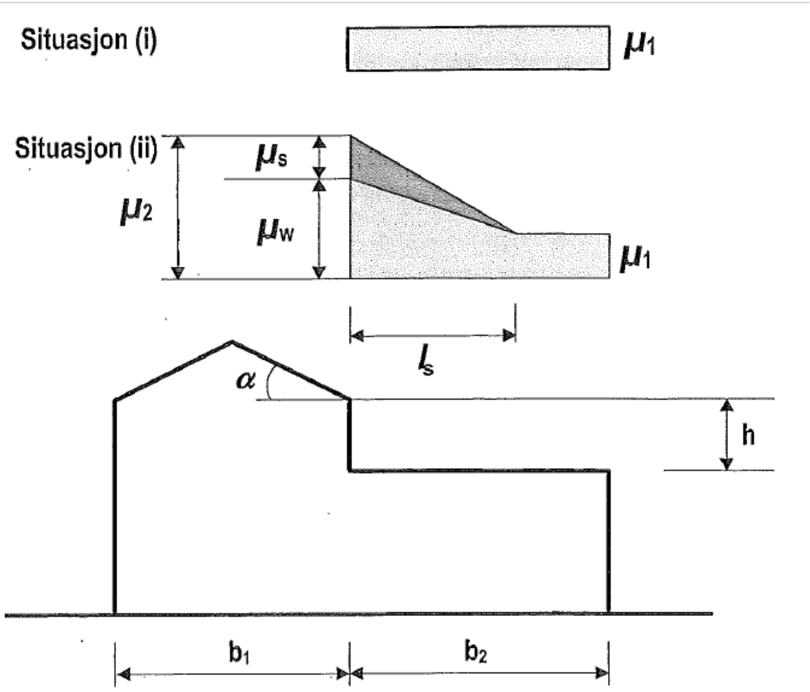


Figure A - 1: Form factor for snow loads on roofs adjacent to high buildings, (Eurokode 1: : Laster på konstruksjoner. Del 1-3. Allmenne laster. Snølaster = Eurocode 1: Actions on structures : Part 1-3: General actions, Snow loads 2008)

μs = 0 for $\alpha \leq 15^o$

$$\mu w = \frac{(b2 + b2)}{2h} \leq \frac{\gamma h}{sk}$$

μw = (b1 + b2) / (2 * h) ≤ ɣ * h / sk

= (13,62 + 3,63) / (2 * 13,8) ≤ 2 * 13,8 / 2,5

= 0,625 ≤ 11,04

0,8 ≤ μ ≤ 4 (recommended scope, in the national addition)

Using μw = 0,8 because it is the nearest the smallest value

μ2 = μw + μs

= 0,8 + 0

= 0,8

s = μ2 * s_k = 0,8 * 2,5 = 2 kN/m2

The snow loads due to wind on the balconies is smaller than the normal payload on the balconies so we can use the balcony payload instead of the snow load on the balconies.

A.4 Vindlast:

Ålesund	29	Møre og Romsdal
Vanylven	30	Møre og Romsdal
Sande	30	Møre og Romsdal
Herøy	30	Møre og Romsdal
Ulstein	30	Møre og Romsdal
Hareid	29	Møre og Romsdal

Table A - 4: $v_{b,0}$ [m/s] for Fosnavågen in Herøy municipality.(Eurokode 1: Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-4: General actions. Wind actions : Del 1-4 : Allmenne laster. Vindlaster 2009)

$V_{b,0}$ = 30 m/s

Table A - 5: terrain category and terrain parameter, (Eurokode 1: Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-4: General actions. Wind actions : Del 1-4 : Allmenne laster. Vindlaster 2009)

Terrengkategori		z_0 m	z_{min} m
0	Kyststrøk som er eksponert for åpent hav	0,003	1
I	Innsjøer eller flatt og horisontalt område med lite vegetasjon og uten hindringer	0,01	1
II	Område med lav vegetasjon som gress og spredte hindringer (trær, bygninger) med avstand minst 20 ganger deres høyde	0,05	2
III	Område med vegetasjon eller bygninger eller med spredte hindringer med avstand minst 20 ganger deres høyde (landsbyer, forstadsterreng, permanent skog)	0,3	5
IV	Område der minst 15 % av overflaten er dekket av bygninger, og deres gjennomsnittlige høyde overskrider 15 m	1,0	10
MERKNAD Terrengkategoriene er illustrert i A.1.			

Terrengkategori II

Z = 17,1 m

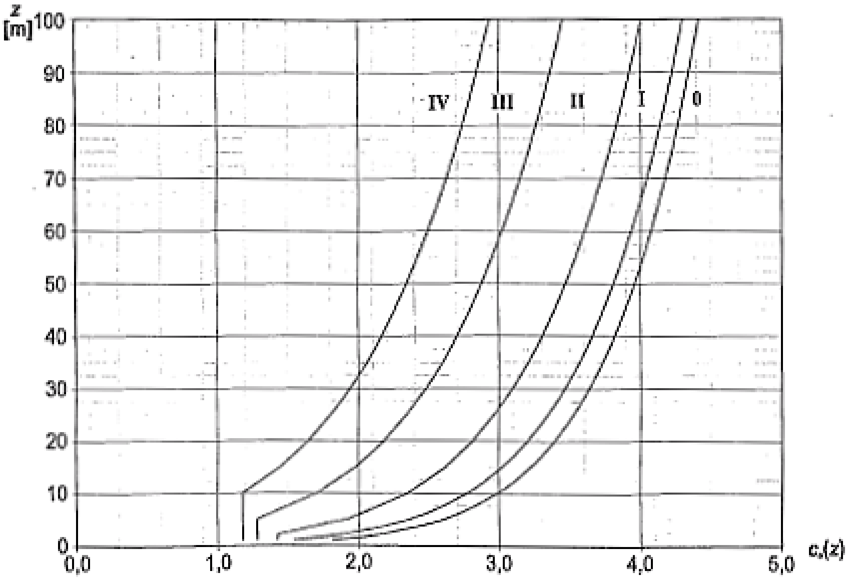


Figure A - 2: Illustrations of exposure factor $C_e(z)$ for C_0 = 1.0, k_l = 1.0, (Eurokode 1: Laster på konstruksjoner = Eurocode 1: Actions on structures. Part 1-4: General actions. Wind actions : Del 1-4 : Allmenne laster. Vindlaster 2009)

$C_e(z)$ = 2,8 for C_0 = 1,0 og k_1 = 1,0

q_b = ½ * ρ * vb2 = ½ * 1,25 * 30^2 = 562,5 N/m^2

$q_p(z) = c_e(z) * q_b$ = 2,8 * 562,5 = 1575 N/mm^2 ~ 1,6 kN/m^2

Karakteristisk vindlast: $q_p(z)$ = 1,6 kN/m^2

Wind towards the long side: h/d = 17,1 / 29,3 = 0,58

Wind towards the short side: h/d = 17,1 / 21,5 = 0,80

Using sone 1 from table 7.1

D: $c_{pe,10} = q_b(z)$ * 0,8 = 1,6 * 0,8 = <u>1,28 kN/m^2</u>

E: $c_{pe,10} = q_b(z)$ * -0,5 = 1,6 * -0,5 = <u>-0,8 kN/m^2</u>

A.5 Load from staircase

Concrete: 25 kN/m3

Average thickness: 250 mm = 0,25 m

Length from the bottom to the first repo: 1590 mm → 1,59 / 2 = 0,795 m

Length from the first repo to the second: 2150 mm → 2,15 / 2 = 1,075 m

Length from the second repo to the top: 1590 mm → 1,43 / 2 = 0,715 m

Load on the bottom part of the stair: 25 kN/m3 * 0,25 m * 0,795 m = <u>4,97 kN/m</u>

Load on the first repo of the stair: 25 kN/m3 * 0,25 m * (0,795 + 1,075) m = <u>11.69 kN/m</u>

Load on the second repo of the stair: 25 kN/m3 * 0,25 m * (1,075 + 0,715) m = <u>11,19 kN/m</u>

Load on the top part of the stair: 25 kN/m3 * 0,25 m * 0,715 m = <u>4,47 kN/m</u>

A.6 Seismic Loads

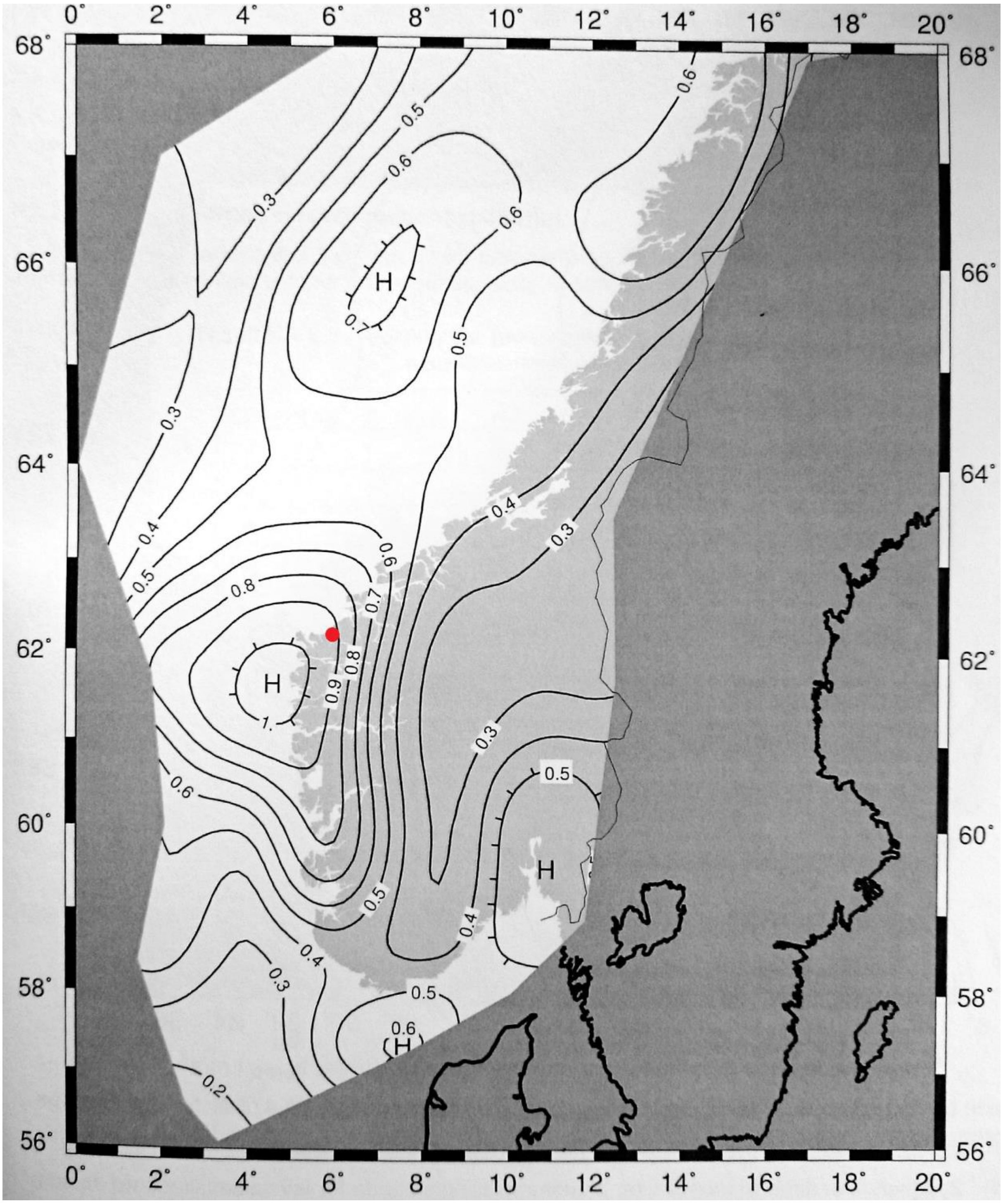


Figure A - 3: Seismic zones in the south of norway, ag40Hz in m/s2. The figure is from the Norwegian addition to the Eurocode 8. (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

Table A - 6: Prior information table when selecting seismic class, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

Byggverk	I	II	III	IV
Byggverk der konsekvensene av sammenbrudd er særlig store				X[1)]
Viktig infrastruktur: sykehus, brannstasjoner, redningssentraler, kraftforsyning og lignende			(X)	X
Høye bygninger, mer enn 15 etasjer		(X)	X	
Jernbanebruer [2)]			X	(X)
Veg- og gangbruer [2)]		(X)	X	(X)
Byggverk med store ansamlinger av mennesker (tribuner, kinosaler, sportshaller, kjøpesentre, forsamlingslokaler osv.)		(X)	X	
Kaier og havneanlegg		X	(X)	
Landbaserte akvakulturanlegg for fisk		X	(X)	
Tårn, master, skorsteiner, siloer	(X)	X	(X)	
Industrianlegg		X	(X)	
Skoler og institusjonsbygg		(X)	X	
Kontorer, forretningsbygg og boligbygg		X	(X)	
Småhus, rekkehus, bygg i én etasje, mindre lagerhus osv.	X	(X)		
Støttemurer med høyde lavere enn 3 m langs veger i klasse II [3)]	X	(X)		
Kulverter	X	(X)	(X)	
Landbruksbygg	(X)			
Kaier og fortøyningsanlegg for sport og fritid	(X)			

[1)] For byggverk der konsekvensene av sammenbrudd er særlig store, for eksempel ved atomreaktorer og lagringsanlegg for radioaktivt avfall, store dammer og marine konstruksjoner bør jordskjelvrisikoen vurderes spesielt, eventuelt basert på en risikoanalyse.

Lagertanker for flytende gass og store hydrokarbonførende rørledninger over land er behandlet i NA til NS-EN 1998-4.

[2)] Se veiledende tabell for valg av seismisk klasse for bruer i NA til NS-EN 1998-2.

[3)] For støttemurer langs jernbane, støttemurer langs veger med høyde over 3 m og støttemurer langs viktige veier (klasse III) benyttes samme seismiske klasse som for vegen eller jernbanen

Table A - 7 : Values for seismic factor, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

Seismisk klasse	γ_I
I	0,7
II	1,0
III	1,4
IV	2,0

Table A - 8: Ground types, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

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.			

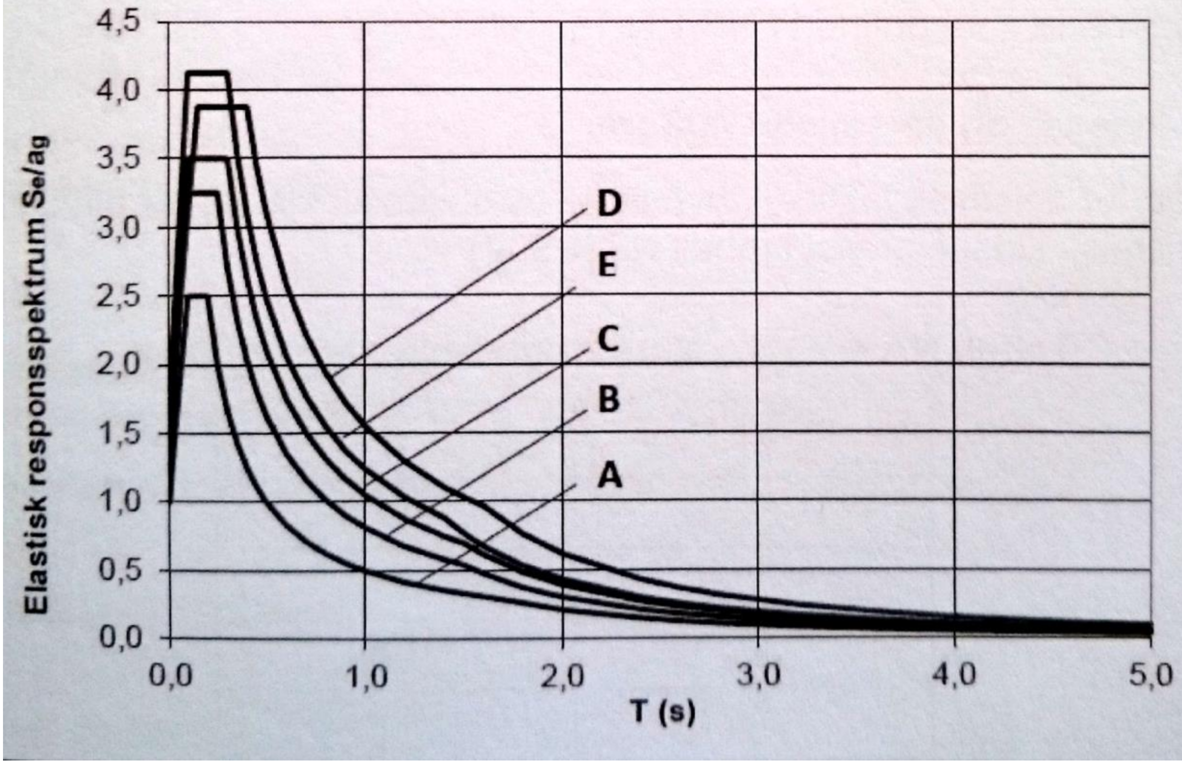
[1)] Hvis minst 75 % av konstruksjonen står på fjell og resten på løsmasser, og konstruksjonen står på ett kontinuerlig fundament (platefundament), kan grunntype A benyttes.

[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.

Table A - 9: Values of parameters describing the recommended elastic response spectra, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

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



$a_{g40Hz} = 0.9$

$a_{gR} = 0.8 * a_{g40Hz}$

$= 0.8 * 0.9$

$= 0.72$ [m/s^2]

$\gamma = 1$

$a_g = \gamma * a_{gR}$

x

= 1 * 0.72

= 0.72 [m/s^2]

Table A - 10: Dimensioning Principles, ductility classes and upper limits of reference values for constructions, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

Dimensjoneringsprinsipp	Konstruksjonens duktilitetsklasse	Område for referanseverdier for valg av konstruksjonsfaktor q
Prinsipp a) Konstruksjon med lite energiabsorpsjon	DCL (Lav)	$\leq 1{,}5$
Prinsipp b) eller c) Energiabsorberende konstruksjon	DCM (Middels)	≤ 4 Også begrenset av verdiene for DCM i tabell 7.2
	DCH (Høy)	Som for DCM

To adjust the loads in FEM-Design for the earthquake simulations it is used the table form the Eurocode. (*Eurokode : Grunnlag for prosjektering av konstruksjoner = Eurocode : Basis of structural design* 2008)

Table A - 11: Values of parameters describing the vertical elastic response spectrum, (Eurokode 8: Prosjektering av konstruksjoner for seismisk påvirkning = Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings : Del 1 : Allmenne regler, seismiske laster og regler for bygninger 2014)

	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Vertikalt responsspektrum	0,6	0,05	0,20	1,2

Table A - 12: Factors to adjust the earthquake simulations, (Eurokode : Grunnlag for prosjektering av konstruksjoner = Eurocode : Basis of structural design 2008)

Last	ψ_0	ψ_1	ψ_2
Kategorier for nyttelaster i bygninger (se NS-EN 1991-1-1)			
Kategori A: innendørs bostedsarealer	0,7	0,5	0,3
Kategori B: kontorarealer	0,7	0,5	0,3
Kategori C: arealer hvor personer kan samles	0,7	0,7	0,6
Kategori D: forretningsarealer	0,7	0,7	0,6
Kategori E: lagerarealer	1,0	0,9	0,8
Kategori F: traffikkarealer, kjøretøyvekt ≤ 30kN	0,7	0,7	0,6
Kategori G : trafikkarealer, 30kN < kjøretøyvekt ≤ 160kN	0,7	0,5	0,3
Kategori H : tak	0	0	0
Snølaster på bygninger (se NS-EN 1991-1-3)*			
Finland, Island, Norge, Sverige	0,70	0,50	0,20
Øvrige CEN-medlemsland, for steder med beliggenhet H > 1000 m o.h.	0,70	0,50	0,20
Øvrige CEN-medlemsland, for steder med beliggenhet H ≤ 1000 m o.h.	0,50	0,20	0
Vindlaster på bygninger (se NS-EN 1991-1-4)	0,6	0,2	0
Temperatur (ikke brann) i bygninger (se NS-EN 1991-1-5)	0,6	0,5	0
MERKNAD ψ-verdiene kan fastsettes i det nasjonale tillegget.			
* For land som ikke er nevnt nedenfor, se relevante lokale vilkår.			

Appendix B: Pictures from the site

Figure B - 1: Picture 1 from the assembly of the construction

Figure B - 2: Picture 2 from the assembly of the construction

Figure B - 3: Picture 3 from the assembly of the construction

Figure B - 4: Picture 4 from the assembly of the construction

Appendix C: Pictures from the AutoCAD file

Figure C - 1: The basement floor, with the axis system.

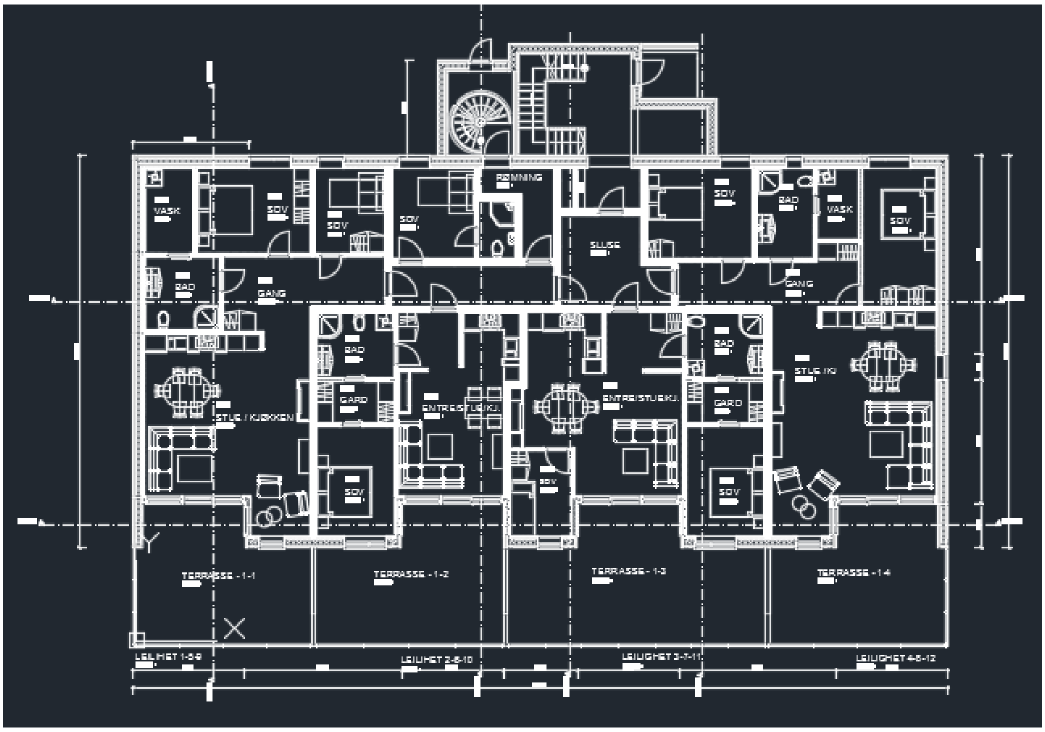


Figure C - 2: The first floor

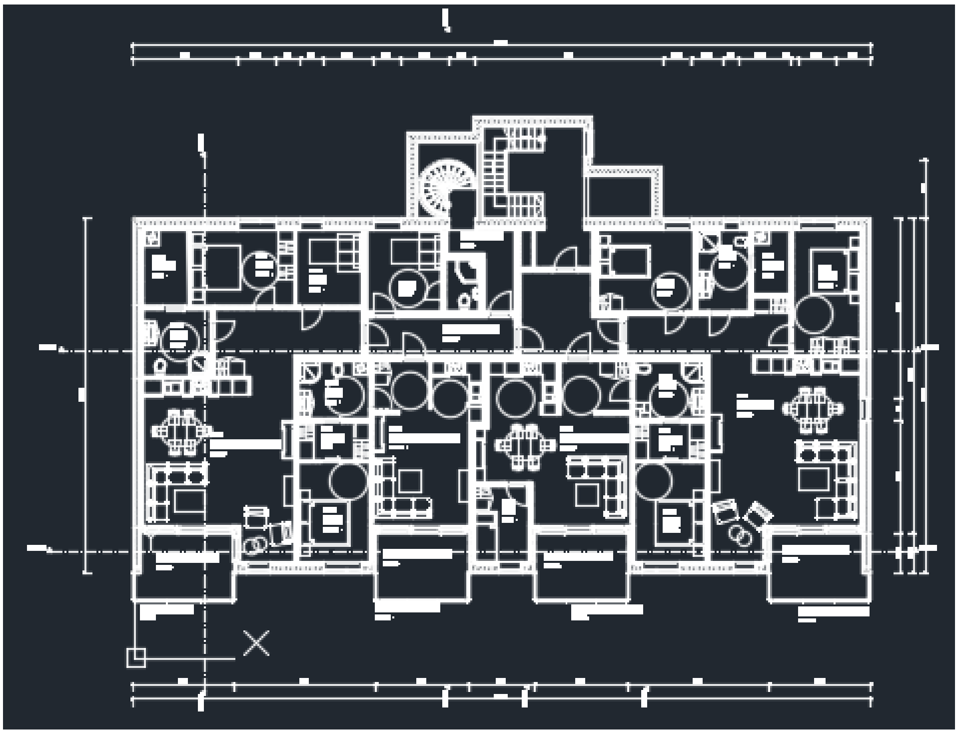


Figure C - 3: Second and third floor

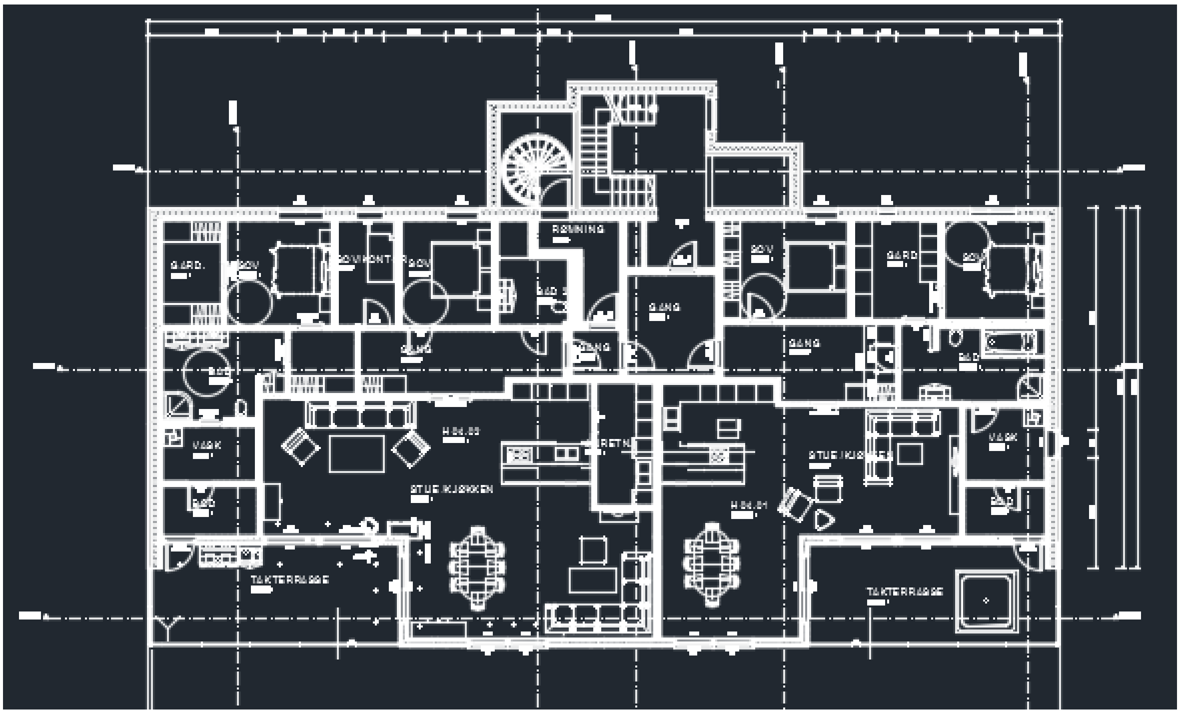
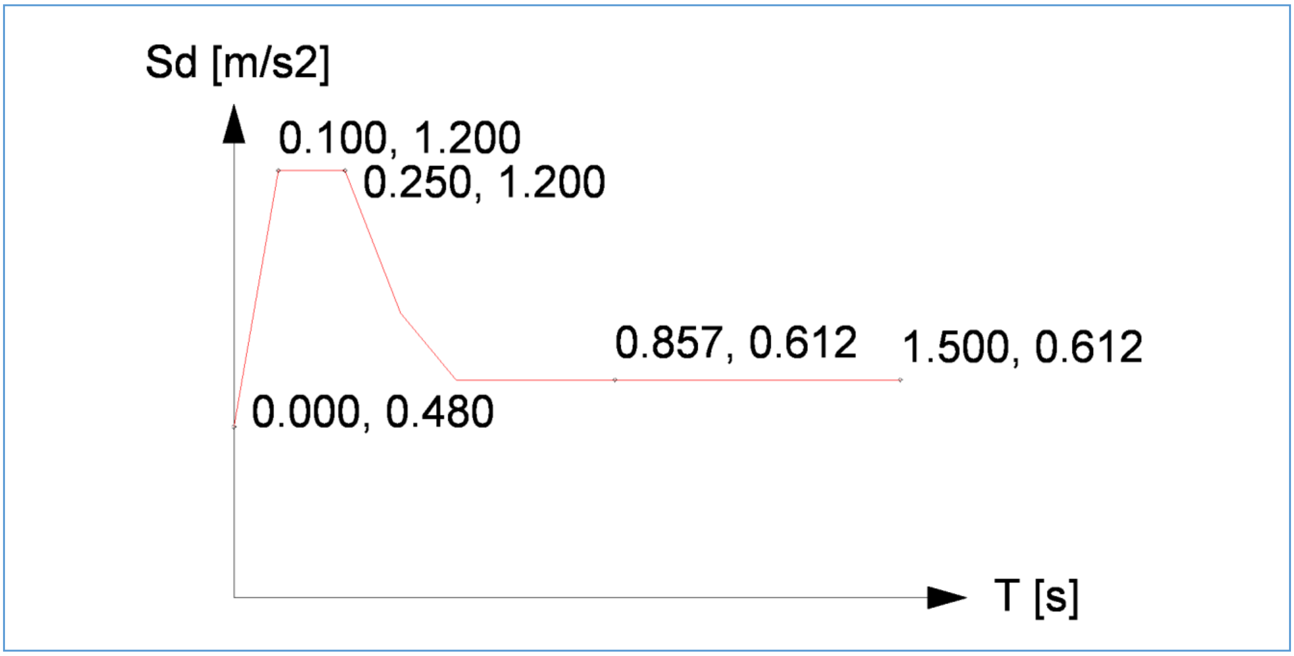


Figure C - 4: Fourth floor

Appendix D, Seismic values from the prefabricated element model

Seismic load, structure information (3 items)

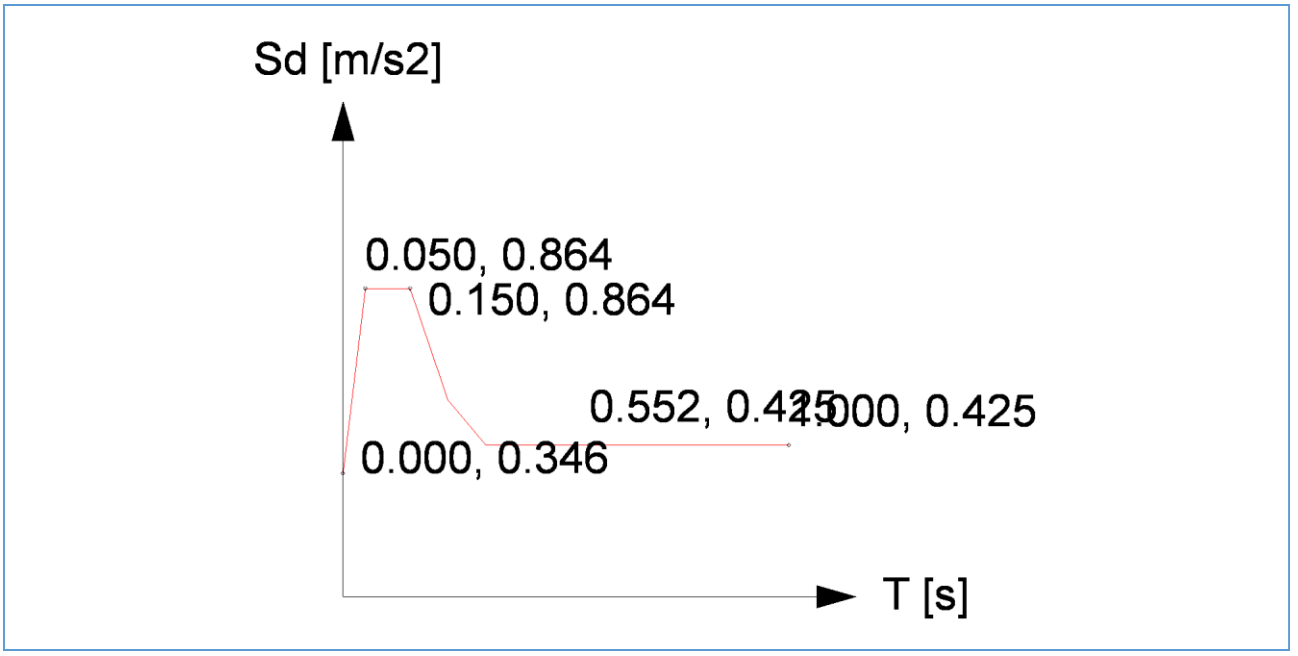
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.850



Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.720
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.150
TD [s]	1.000
q	1.500
beta	0.820

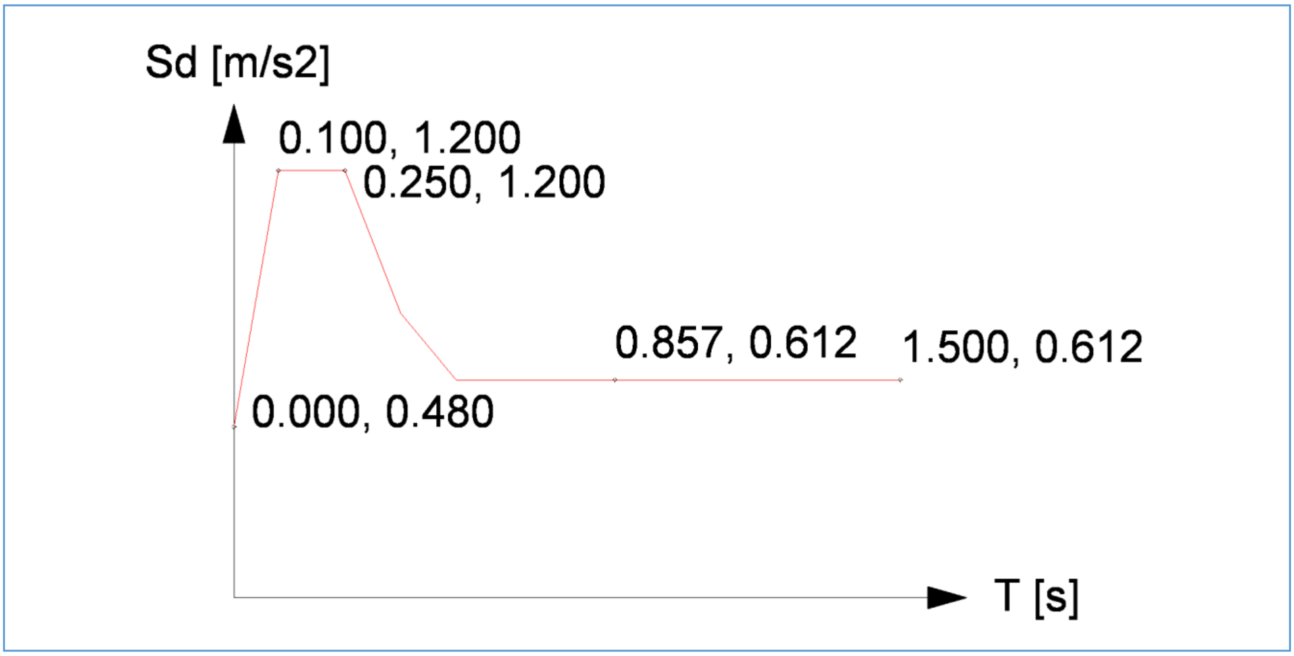
Seis. calc.: static - linear shape (8 items)

Value	Quantity
Alfa (angle of x-x')	0.000 [rad]
Lambda x'	1.000
Lambda y'	1.000
Tx'	1.000

Value	Quantity
Ty'	1.000
Combination rule	Ex "+" 0.3Ey "+" 0.3Ez...
Signed result	Yes
Torsional effect	5.0 [%]

Seismic load, structure information (3 items)

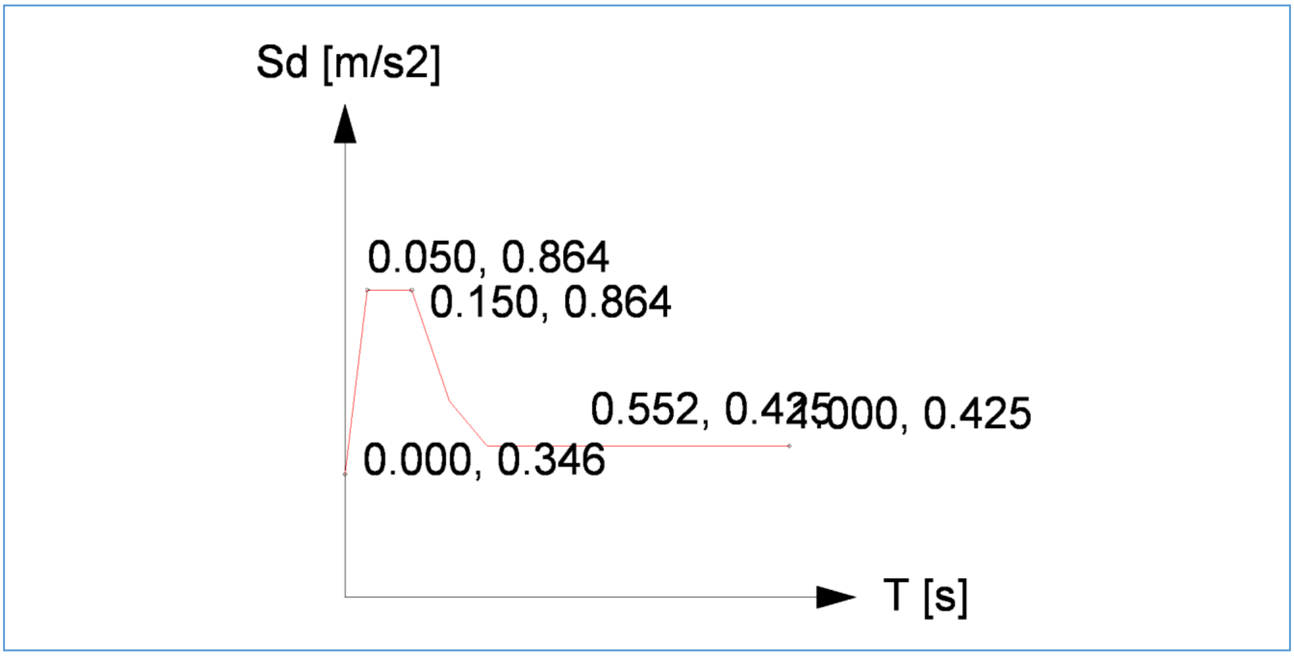
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.850



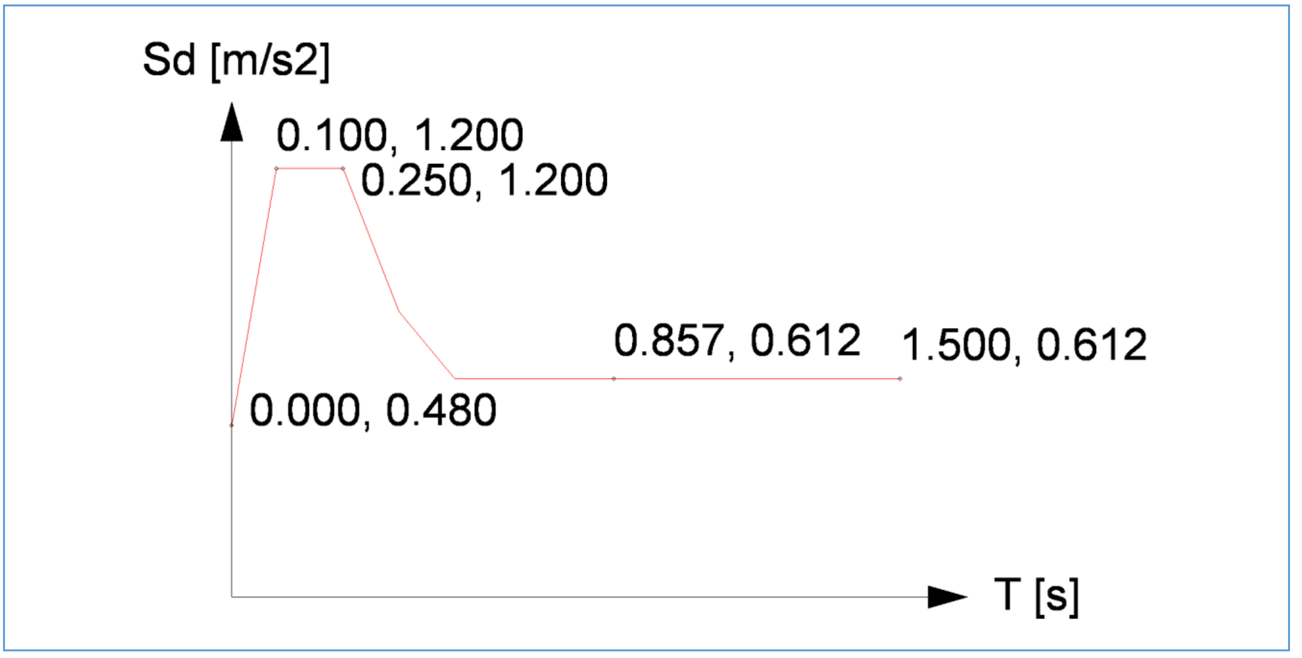
Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.720
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.150
TD [s]	1.000
q	1.500
beta	0.820

Seismic load, structure information (3 items)

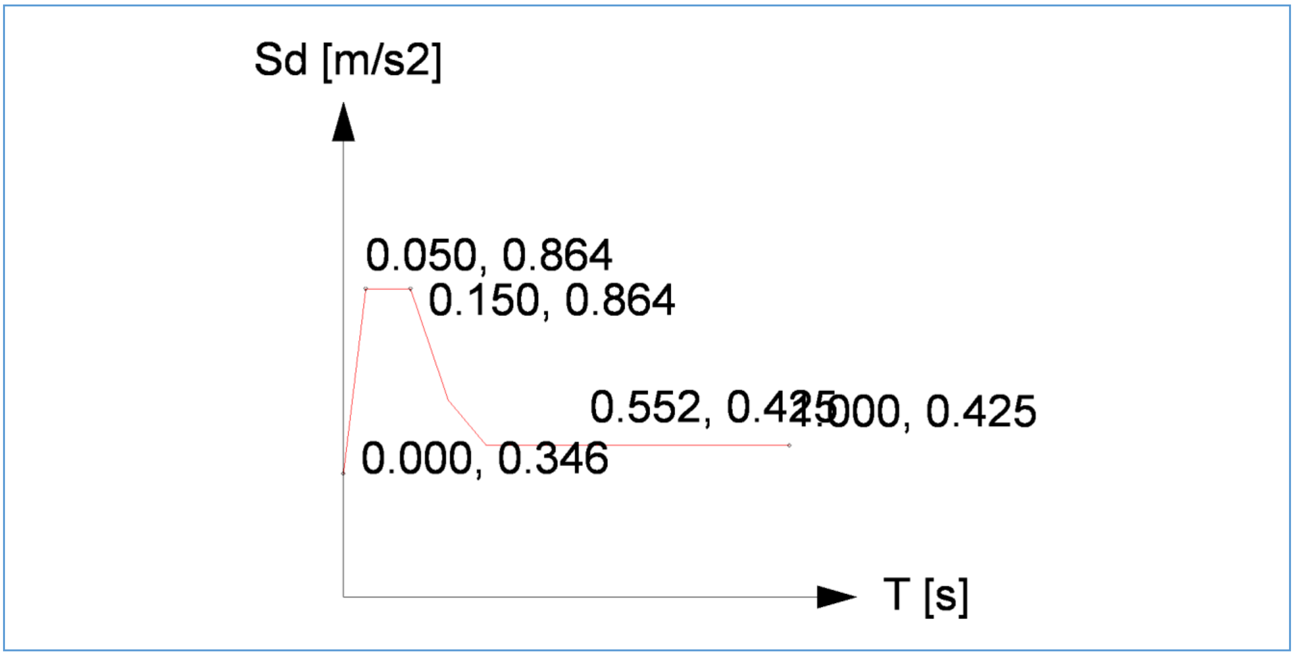
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.850



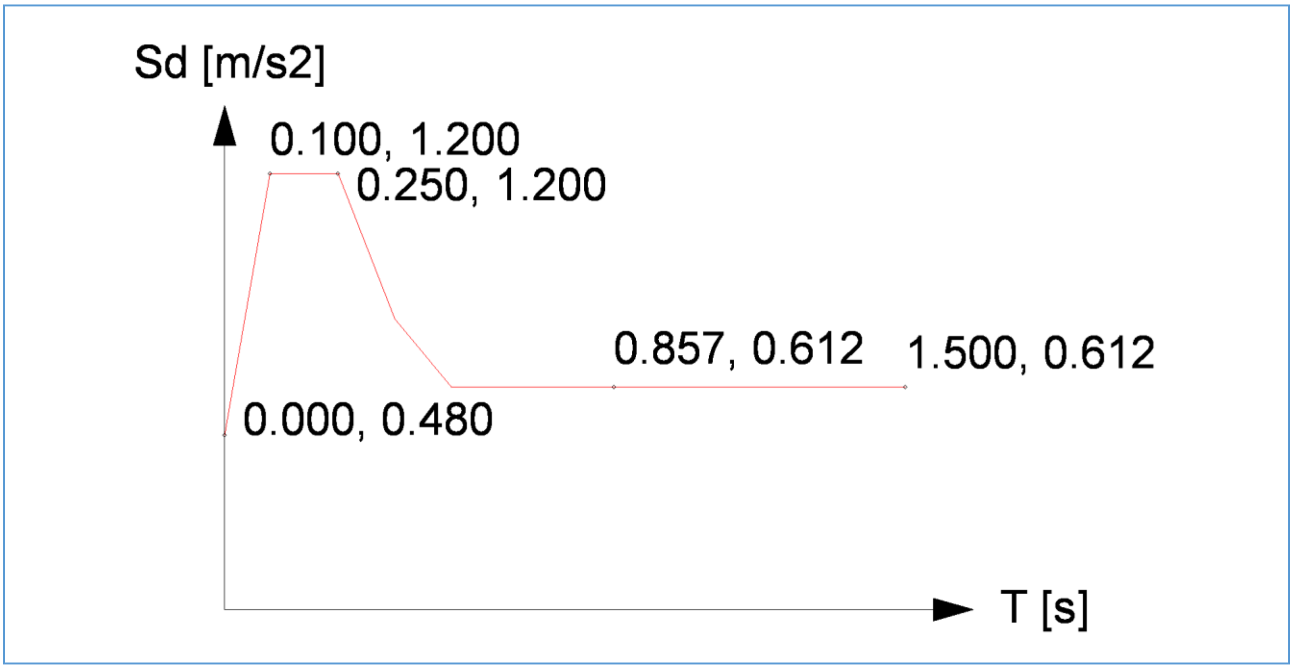
Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.720
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.150
TD [s]	1.000
q	1.500
beta	0.820

Seismic load, structure information (3 items)

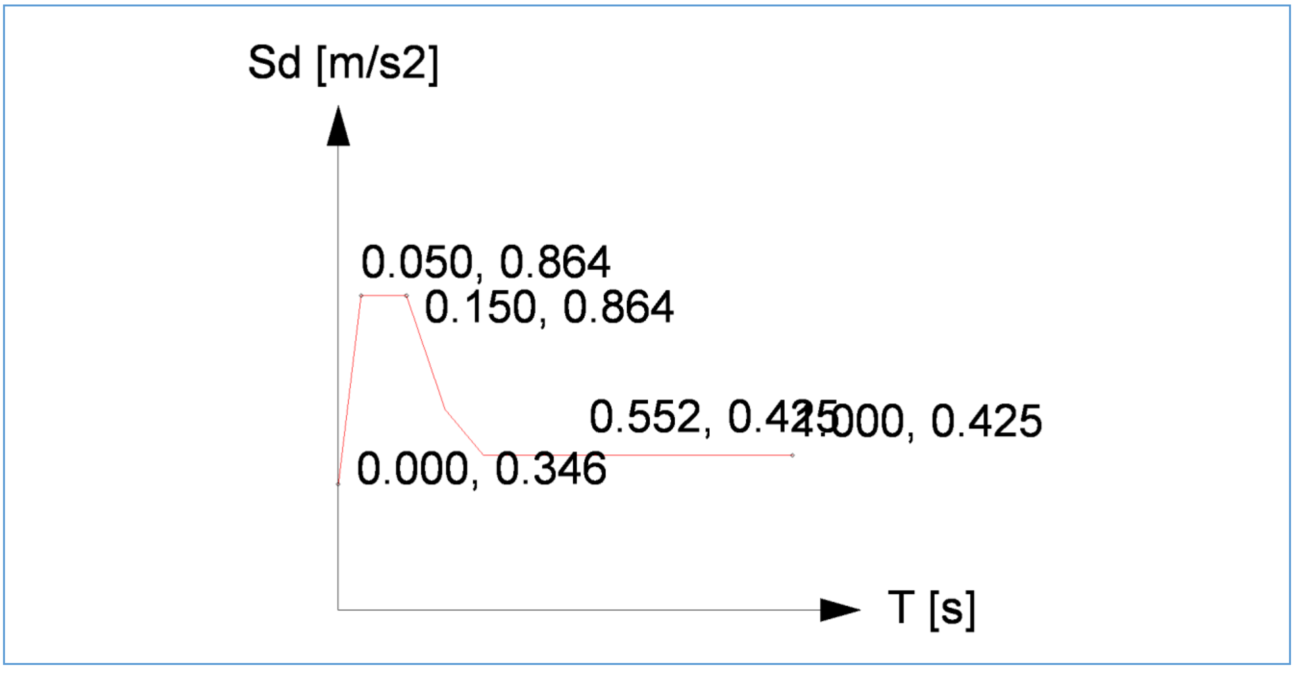
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.850



Seismic load, vertical sp., standard (8 items)

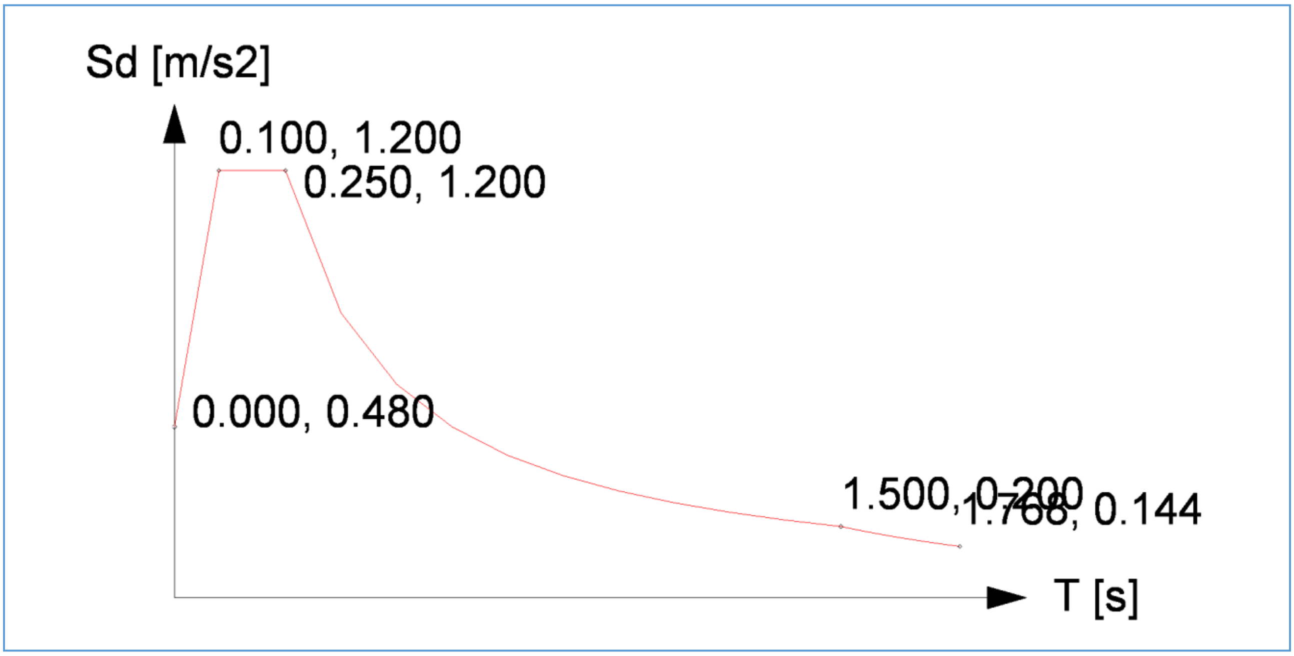
Value	Quantity
Type	1
agv/ag [m/s2]	0.720
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.150
TD [s]	1.000
q	1.500
beta	0.820

Appendix E, Seismic values from the on-site casted model

Seismic load, structure information (3 items)

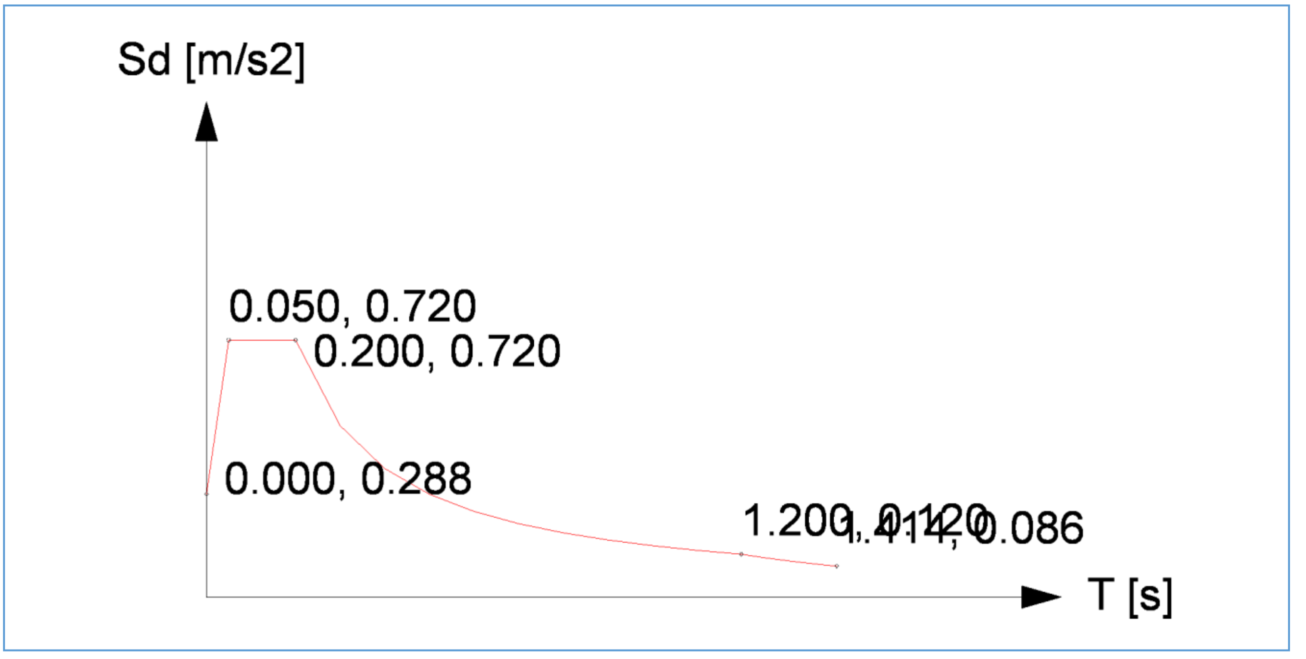
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.200



Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.600
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.200
TD [s]	1.200
q	1.500
beta	0.200

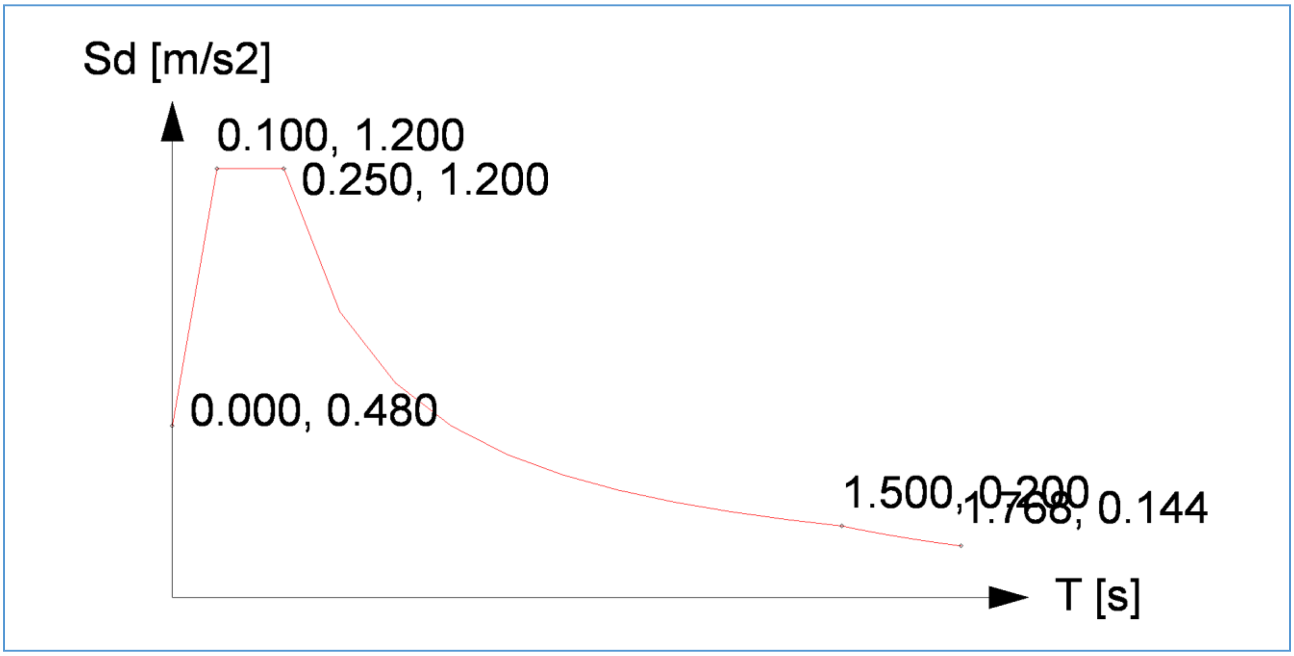
Seis. calc.: static - linear shape (8 items)

Value	Quantity
Alfa (angle of x-x')	0.000 [rad]
Lambda x'	1.000
Lambda y'	1.000
Tx'	1.000

Value	Quantity
Ty'	1.000
Combination rule	Ex "+" 0.3Ey "+" 0.3Ez...
Signed result	Yes
Torsional effect	5.0 [%]

Seismic load, structure information (3 items)

Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.200

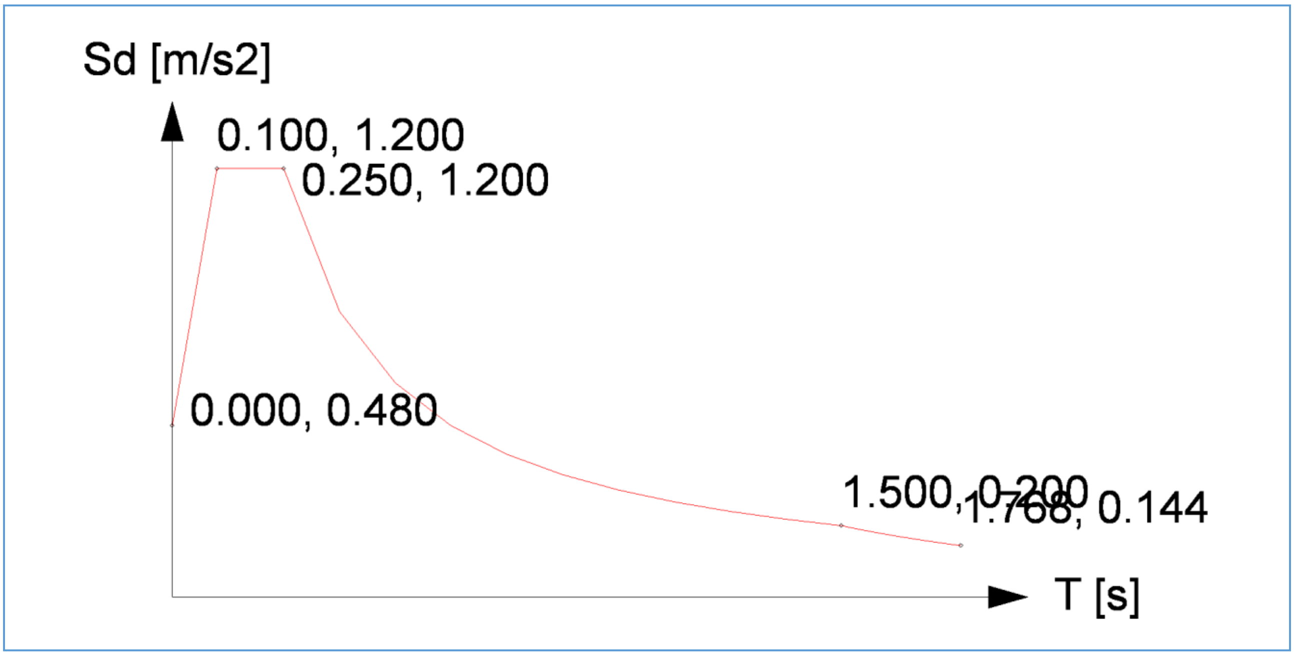
Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.600
S	1.000
TB [s]	0.050

Value	Quantity
TC [s]	0.200
TD [s]	1.200
q	1.500
beta	0.200

Seismic load, structure information (3 items)

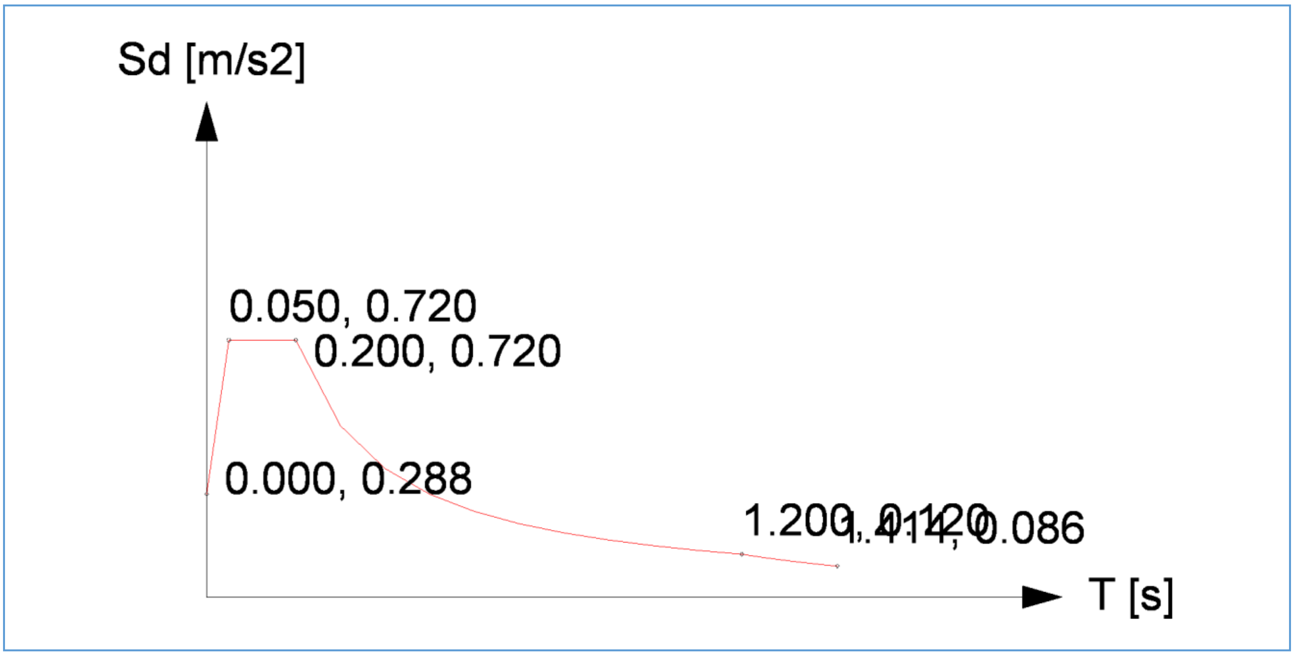
Value	Quantity
Structure type	Building structure
xi (damping factor) [%]	5.000
qd (behaviour factor for displacements)	1.000



Seismic load, horizontal sp., standard (9 items)

Value	Quantity
Type	1
Ground	A
ag [m/s2]	0.720
S	1.000
TB [s]	0.100

Value	Quantity
TC [s]	0.250
TD [s]	1.500
q	1.500
beta	0.200



Seismic load, vertical sp., standard (8 items)

Value	Quantity
Type	1
agv/ag [m/s2]	0.600
S	1.000
TB [s]	0.050
TC [s]	0.200

Value	Quantity
TD [s]	1.200
q	1.500
beta	0.200

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