Comparison of two stabilizing systems of steel structure including the effect of earthquake design



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Summary

This master thesis takes a look at steel structures during a design phase. The stabilizing system of a steel structure is an important part of the design. The properties and mechanisms of two different stabilizing structures were looked at, X-bracing frame and moment resisting frame. The different connection types were studied to gain an understanding of the stabilizing systems. The horizontal loads are also an important part of any design with wind loads and earthquake design. Some basic study of dynamics and earthquake design was therefore necessary. Also the wind loads were calculated according to the necessary standards.

There is an idea that a moment resisting frame leads to higher material consumption. The question was therefore posed: is it really a misconception that a moment-resisting frame has higher material consumption? Does earthquake design change the overall comparison of x-bracing frame and moment frame? Our case were two structures from an earlier bachelor project with a six story steel frame, one with x-bracing and the other with moment-resisting frame. The structures were designed and analysed in Robot Structural Analysis. The result shows that the moment resisting frame had less material consumption than the x-bracing frame, while the x-bracing frame withstood earthquake forces better than the moment frame. This leads to the conclusion that the moment-resisting frame is preferred but bracing should definitely be considered in more earthquake prone areas or with high-raised structures.

Innhold

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1 | Introduction

Steel structures are an important part of the construction industry today. The strength and material properties of steel make it a preferred choice in construction. This report looks at the aspects of a steel structure with regards to the difference between a moment resisting frame and a cross bracing frame. The horizontal loads such as wind and earthquake imposed on a structure will be an important part of this study.

This master thesis includes different issues regarding the seismic design of the structures for earthquake- induced ground motion loads. Seismic design of the structures is a complicated subject due to the complexity action of the seismic loads to the structure. A seismic design engineer needs have an understanding of the dynamics involved in ground motion. Today, many new buildings in Norway need to be approved for seismic design according to the demands of Eurocode 8 and the associate national annex. Seismic design is a relatively new requirement in Norway and therefore it is an important part of the project design of a structure.

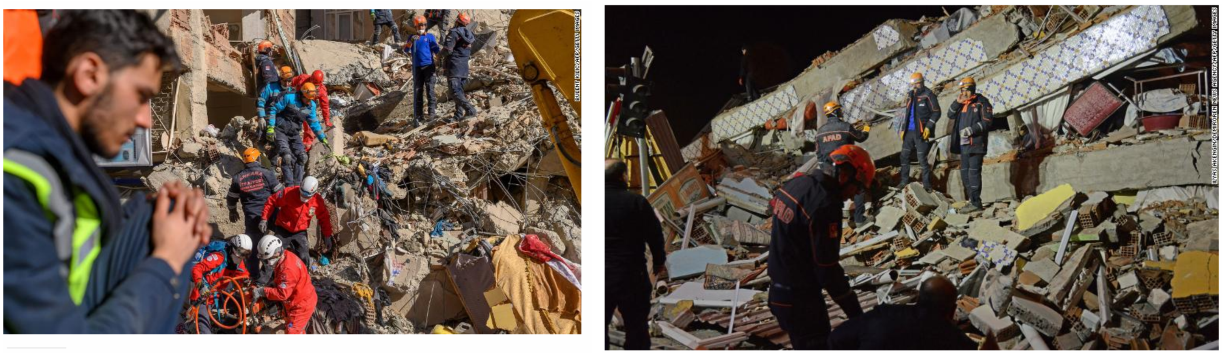
Our case is a structure from an earlier bachelor report that we will be studying further. The six stories, seven including a roof terrace, steel structure will be analysed with an emphasis of the differences between moment resistant frame, and a frame with bracing. The connection design as well as material properties will be necessary when understanding the differences in cross-bracing and moment resisting frame.

The purpose of this study is to learn the advantages and disadvantages of the two structures in a design phase. Factors such as material consumption, bracing systems, and horizontal loads will be the basis in this report when comparing the two structures. Our goal is to evaluate the properties of different stabilizing systems in a steel structure in a design phase and to produce educated suggestions for future studies and design.

2 Significance of the work

Earthquake is a naturally destructive force that involves the potential loss of life and destruction of infrastructures. Large numbers of building can be significantly damaged following extreme earthquakes. In order to prevent this disaster, it is important to design earthquake resistant structures that is very essential for our modern society. Earthquake design and construction technologies have undergone fast development over the last 100 years. Engineers are always developing a new technology in structures that is able to protect human life and material values of the buildings. The philosophy of the proper seismic design for buildings and infrastructures is used to prevent collapse of buildings under large earthquakes and maintain full operation immediately after earthquakes. Therefore, a civil engineer should have proper knowledge of seismic design that can be acquired by learning of building codes and other important documents such as Eurocode 8 and National Annex which is relevant in seismic design. This helps to find out the best solution in design phase of the structure in order to decrease the probability of undesirable events such as damage or totally collapse of the structures during an earthquake.

A poor seismic designed structure can be damaged partly or collapse when it is exposed to an earthquake. For example, we can see the earthquake that happened in a city in Turkey. The latest earthquake dated 24.01.2020 that stuck the Elazig city that is located in the southern part of Turkey with magnitude 6.8. The earthquake took the life of around 36 people and more than 1500 were injured. Around ten buildings were totally collapsed and several buildings were damaged.[1]. In this situation, the people, including the injured, may have been unable to return to their houses and consequently fall into financial difficulties.



Figur 2.1: Rescue workers search through the rubble in Elazig[1]

In Norway, earthquake design is relatively new, and many ask why it is applicable here. However, Norway is located in the region of North-Europe that experiences earthquake many times almost everyday,but they are from low to moderate seismic actions. The probability of a strong earthquake is low, but some expert says medium ones could hit Norway and Northern Europe as it happened in 1904. In 1904, Oslo fjord was exposed to an earthquake with a magnitude of 5.6 that caused only material losses[11]. So, it is very important to have earthquake design. In Norway, earthquake design was introduced in 2004 with new design requirements in order to update the reliability based design of structures, mostly focused on new buildings.

Therefore, the main goal of this master thesis is to design and construct safe and secure buildings during an earthquake. For constructing safe and secure buildings, it is important to understand the behavior of the structure and structural properties such as ductility, damping, stability and deformability. So, the buildings require special design for earthquakes. Our master thesis has considered to design buildings with two different stabilizing system that can meet the requirements of appropriate design and strength to withstand the seismic load. The construction of these two structures can have a high contribution to the modern society, if it is designed and analysed properly.

Further, it is also necessary to consider the material consumption of one building comparing to the two stabilizing system, parallel to the seismic design. It is more reasonable to design buildings with less material consumption for the society. It is important to focus on the material costs and reduce the emission of harmfull gases during material production.

3 Theory

3.1 Steel Structures

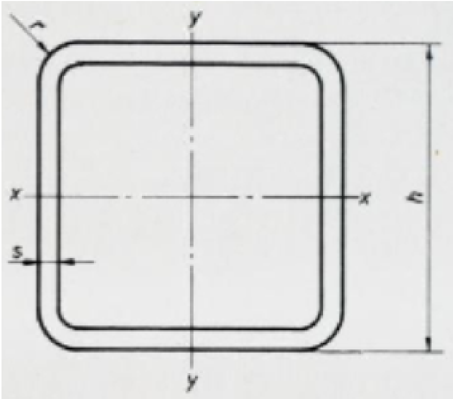
3.1.1 Definition for the steel structure

Steel is an alloy of iron and carbon with contribution of 0.2% of the weight of steel. If the percentage of carbon in the alloy is less than 0.2% is called wrought iron which is soft and malleable. If the alloy contains more thane 3% carbon it is called cast iron. Structural steel is carbon steel with controlled amount of manganese, phosphorous, silicon, sulfur, and added oxygen. Moreover, carbon steel can be categorized according to its carbon content: mild steel(0.2-0.25%carbon), medium steel(0.45-0.85%) and spring steel(0.85-1.85%). The notable properties of steel are determined as follows:

- Modulus of elasticity E =$2.1 \times 10^5 N/mm^2$
- shear modulus G= $8.1 \times 10^4 N/mm^2$
- Poisson's ratio v = 0.3 [2]
- Ductility: (steel is- a ductile material, it can sustain large plastic deformation before failure).
- High strength:(400-500MPa)
- Predictable material properties: (high degree of certainty in prediction of its material properties, steel actually shows elastic behavior relatively to stress level).[12]

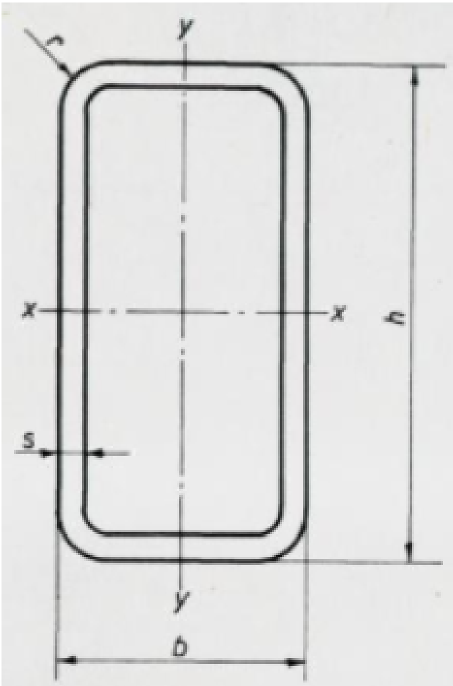
The above mentioned properties makes steel to be attractive and widely-used as building material in modern constructional industry especially in earthquake resistance buildings.

A steel structure is combination of elements that should tolerate their share of applied load and to transmit them safely to the ground. These structural components or elements can be subjected to forces either axial, bending, torsion or combination loads. Axial load can be either tensile or compressive and the elements that tolerate these loads are called tension member or compression member. Columns is an example of a structural element which tolerates compressive load. Generally, all steel structures are constructed of components such a tension member which subjected to tensile force, compression components which subjected to compressive loads, and flexural components which subjected to bending. The elements of a steel structure are rolled to a basic cross- section in a steel- element production plant. These elements can be connected to each other with help of bolts, rivets, pins or welding in order to form the structure. The connection between these elements are called joints. Based on fixity or stability, the joints are classified as rigid which able to transmit the moments, flexible that can transfer axial or shear loads and semi-rigid that has properties of rigidity and flexibility. Steel structures have many advantages such as their smaller weight- to strength ratio hence the slimmer cross section, installation period, scrap value, recycling and so on. Steel section takes appropriate shape in a steel production plant. The process of shape making is called hot rolling process. This can be rolled into different shape and size according their designed demands. There is a standard for the production, regarding to the dimensions, weight and geometrical properties of various of sections. Depend on these standard, steel structures classified into H- section , I- section , Square Hallow Section (SHS), Rectangular Hallow Section(RHS). Square Hallow Section(SHS) are the most used hot- rolled steel section in the buildings. [7]. Here, the figures below shows the illustration of SHS and RHS.



Dimensjoner								
h mm	s mm	r mm	F cm^2	G ~kg/m	0 m^2/m	$J_x=J_y$ cm^4	$W_x=W_y$ cm^3	$i_x=i_y$ cm
35	2,7	2,7	3,42	2,69	0,135	5,91	3,38	1,31
	3,2	3,2	3,98	3,12	0,134	6,65	3,80	1,29
	4,0	4,0	4,82	3,78	0,133	7,65	4,37	1,26
40	2,7	2,7	3,96	3,11	0,155	9,13	4,57	1,52
	2,9	2,9	4,23	3,32	0,155	9,63	4,82	1,51
	3,0	3,0	4,36	3,42	0,155	9,87	4,94	1,51
	4,0	4,0	5,62	4,41	0,153	12,0	6,00	1,46

Figur 3.1: Hot-rolled Square Hallow Section(SHS) [2]



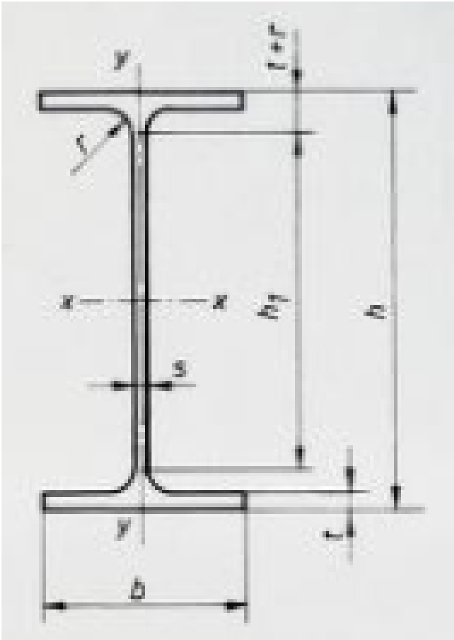
Dimensjoner						For bøyningsakse x—x			
hxb mm	s mm	r mm	F cm^2	G ~kg/m	0 m^2/m	J_x cm^4	W_x cm^3	i_x cm	W_{px} cm^3
50x25	2,7	2,7	3,69	2,90	0,145	11,1	4,44	1,73	5,71
50x30	2,9	2,9	4,23	3,32	0,155	13,4	5,36	1,78	6,74
	4,0	4,0	5,62	4,41	0,153	16,8	6,72	1,73	8,70
60x40	2,9	2,9	5,39	4,23	0,195	25,9	8,63	2,19	10,7
	3,0	3,0	5,56	4,36	0,195	26,7	8,90	2,19	11,0
	4,0	4,0	7,22	5,66	0,193	33,3	11,1	2,15	14,0

Figur 3.2: Hot-rolled Rectangular Hallow Section(RHS) [2]

I-section steel is a type of steel cross section looks like the character"I". The inner surface of the upper and lower flange of I-section has sloop 1:6 which makes the flange thin outside and thick inside. They are mainly used in beams. The cross section of I-beam has better pressure bearing and tensile resistant, but the section size is too narrow to resist twisting. I-beam is small in length and high in height, so it can bear load only in one direction. I-section are not prioritize to use them as in columns of the building, because they are not stable enough.

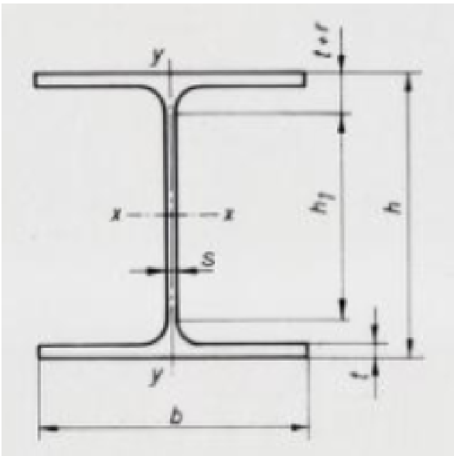
H- steel sections are designed to take axial forces, mostly use as in column of the building. H-section steel has a deep grove, large thickness and can tolerate load in two direction. It has mechanical properties per unit weight which can save a lot of material and construction time. In comparison to the I-section, the flanges of the hot rolled H-section are wider, have greater lateral stiffness, and are more resistant to bending. [13]

The Norwegian Steel Association handbook part 1 is about steel sections and presents different dimensions,weight and geometrical properties of different sections.[2]



Betegnelse IPE 1)	G kg/m 2)	Mål i mm						F cm²	Overflate	
		h	b	s	t	r	h_1 3)		m²/m	m²/t
80	6,00	80	46	3,8	5,2	5	59	7,64	0,328	54,8
100	8,10	100	55	4,1	5,7	7	74	10,3	0,400	49,5
120	10,4	120	64	4,4	6,3	7	93	13,2	0,475	45,6
140	12,9	140	73	4,7	6,9	7	112	16,4	0,551	42,6
160	15,8	160	82	5,0	7,4	9	127	20,1	0,623	39,4

Figur 3.3: Hot-rolled medium width I-beam [2]



Betegnelse HE-A 1)	G kg/m 2)	Mål i mm						F cm²	Overflate	
		h	b	s	t	r	h_1 3)		m²/m	m²/t
100	16,7	96	100	5	8	12	56	21,2	0,561	33,6
120	19,9	114	120	5	8	12	74	25,3	0,677	34,1
140	24,7	133	140	5,5	8,5	12	92	31,4	0,794	32,2
160	30,4	152	160	6	9	15	104	38,8	0,906	29,8
180	35,5	171	180	6	9,5	15	122	45,3	1,02	28,9

Figur 3.4: Hot-rolled wide- flange (HE-A) H-beam [2]

3.1.2 Structural element of steel frame

The main element of steel structural buildings are columns, beams, slabs and vertical bracing systems and etc.

3.1.2.1 Columns

In multi-story buildings wide flange I-section columns are usually selected, by having sufficient strength and stability against flexural buckling in the direction of both main axes of the cross-section. The steel columns could be arranged as pinned or fixed at their bases. Columns on multi-story buildings are continuous along the height of the building and beam span between them as a principle. The continuity of the column is always provided at the selected cross-section (I-section) for reducing the value of bending moment by using bolted connection with flange and web plate, or through the top and base plates on the hollow-section. In columns with hollow section, the continuity of the column could be secured by full penetration of butt welds, or the continuity of the column could be secured inside the concrete slab in the structural system without a frame action.[3]. In addition, H-section column can also be selected in multi-story steel structure. H-section columns are wide flange steel shapes and are also widely used in steel structure buildings. It is an economical section with more optimized area distribution and more reasonable strength to weight ratio.

Figur 3.5: Continuity of column with double cross I-section [3]

3.1.2.2 Beams

Beams support the floor element and transfer their vertical load to the columns. The beams connected the column head at floor level in both main directions. The beam spans might be between 6 and 18 meters long with depth range between 80 and 600 mm, or it could be larger in specific buildings. Beams are usually I-section either hot rolled or built up. However, H-section can be also used in different types of steel constructions. The beam webs resist shear force, while the beam flange resists most of the bending moment.

Besides the strength cross-section, beams can be verified for lateral-torsional buckling (LTB) stability at ultimate limit state in both construction and service stage. It can also be verified stages against excessive deformation at serviceability limit state (SLS). The significance of these checks in beams structural beam element becomes more important in large steel structures [3].

3.1.2.3 Slabs

In steel multi-story buildings, slabs play a significant role on the transfer of vertical loads to the steel beams while working as a floor diaphragms and provided to distribute horizontal loads to the columns. In most cases slabs supported by the top flange of the steel beams or placed within depth of beam‘s as alternative [3]. There are different types of slabs that used for steel multi-story buildings, for example, hollow core slabs. Hollow core slabs are a precast, pre-stressed concrete elements that are used as flooring for building constructions. These types of slabs have four to six longitudinal cores, and the main purpose are to decrease weight of materials in the floor and to maintain maximum strength.

3.1.3 Beam-to-column joints/connections

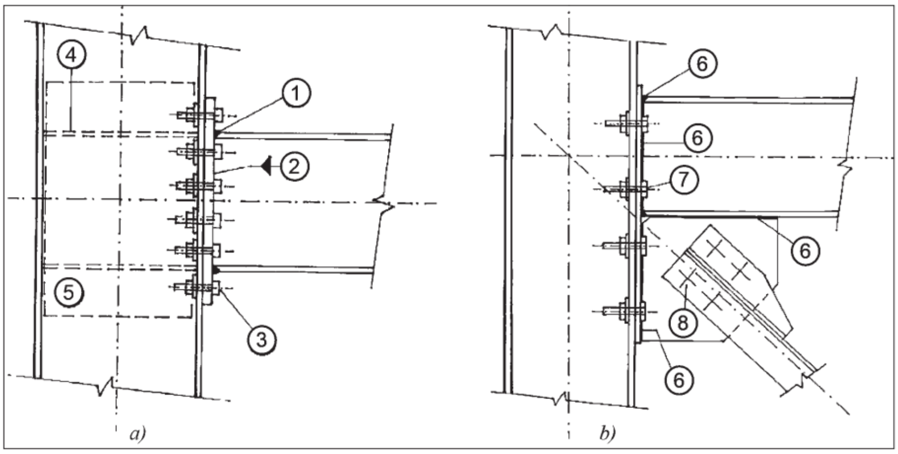
The word “connection” is always describing the interface between connected parts and their connecting means (bolts, welds), while “joint” is the overall area of the connection and in addition end parts of beam and column including column flange, web panel and stiffening elements.

Beam-to-column connections are very important element for steel structures relative to stiffness and strength. The joint configuration depends on many parameters such as the connection type, welded or bolted, the column shape, the angle of inclination between the beam and column. Normally, the beam-to-column joints are considered in design and analysis either pinned or fixed. However, semi-rigid is preferred in real practice in order to allow relative rotation between the connected members and developing moments[3].

3.1.3.1 Classification of joints

Joints are classified in respect to stiffness and strength. The beam to column connection and joints can be classified in to three parts, such as: [3]

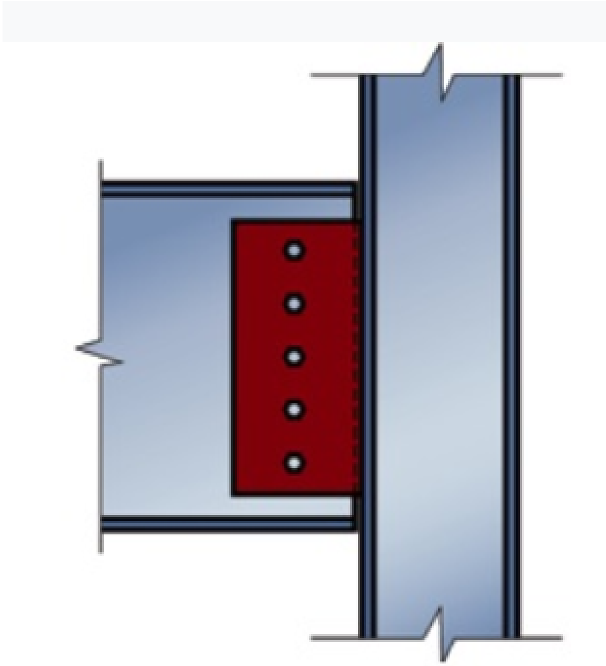
- Simple/Pinned connection: joint with moment capacity smaller than 25% of the moment capacity of the connected members.
- Rigid/ Full strength connection: joint with moment capacity higher than the moment capacity of the connected members.
- Semi-rigid/Partial strength connection: joint with neither fixed nor pinned



Figur 3.6: Connection’s rotation [4]
a) Moment resistant connection
b) Diagonal bracing connection

Pinned/Simple connections:

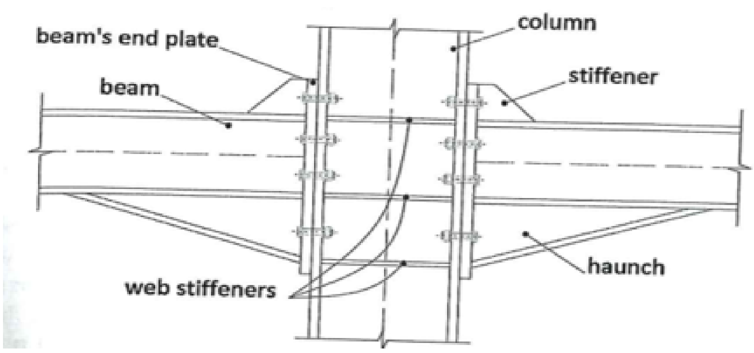
Pinned connection is able to resist only force, and having sufficient rotational capacity. The transformation of shear force is usually realized by bolting the web of the beam to the column through a fin plate that is welded to the web or flange of the column as shown in the figure below.



Figur 3.7: Pinned beam-to-column connection [5]

Rigid connections:

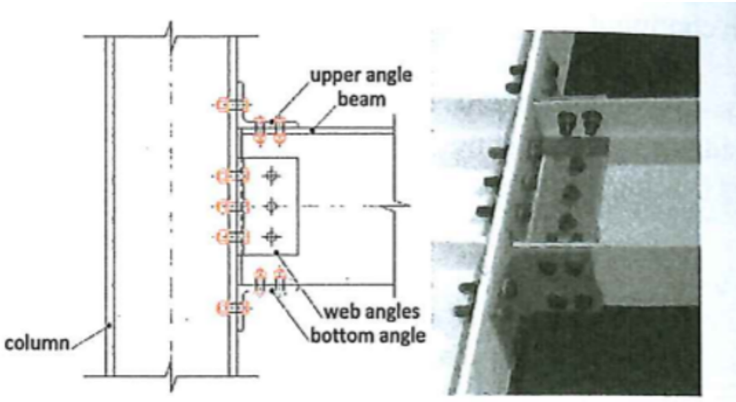
This type of connection is able to resist both force and moments, and having adequate rigidity as the angle between the connected members remains unchanged during loading. Rigid beam-to-column joints usually arranged by welding at the beam to end plate and bolted the column flange. In order to increase the lever arm, the end plate usually extended beyond the top flange of the beam. If the height of the end plate is not sufficient to provide the required strength and rigidity, a haunch is welded to the beam for having adequate strength as shown in the figure below.[3].



Figur 3.8: Rigid beam-to-column connection [3]

Semi-rigid connections:

In this type of connection, the change of the angle between connected members appears when a moment applies on the connection. They are able to resist bending moment with change of the angle from the initial between connect members. This connection always represents by moment-rotation (M-ϕ) curve. In practice, semi-rigid connection provided the structure to resist horizontal loading by vertical bracing systems. The connection of semi-rigid is usually arranged as both flange and web of the beam are connected to the column through the angle section[3].

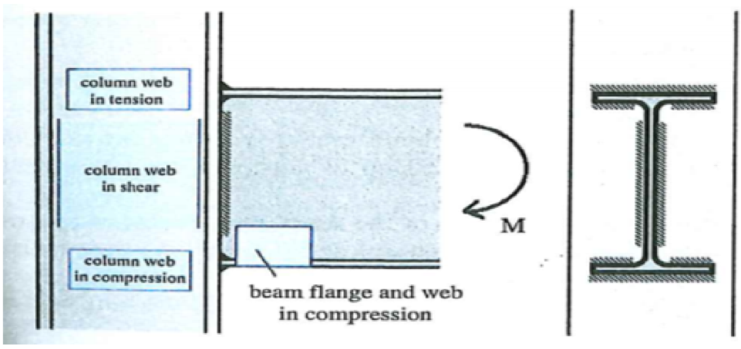


Figur 3.9: Semi-rigid beam-to-column connection [3]

In connection, both beams and columns of I and H-section, and the beam is connected to the strong axis of the column either by welding or bolting

3.1.3.2 Welded Beam-to-column joints

In I-shaped or H-shaped beam and column, the beam flanges and webs are welded to the column flanges. The column web will be in tension, compression and shear, both beam flanges and webs are in compression when the individual joints are subjected to a bending moment.



Figur 3.10: Welded joints [3]

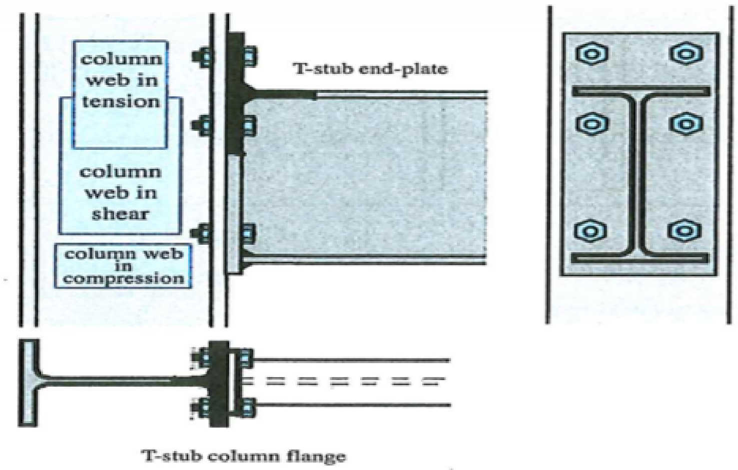
Column web in compression and tension: the compression is occurred when the load transfer from the beam flange to the column web. Meanwhile, the transfer load in tension is like compression with opposite direction of stress. In this situation, yielding and buckling of the column web are possible failure modes.

Column web in shear: A column section is subjected to a concentrated shear force as a result of compression or tension force transferred from the beam flange.

For increasing resistance in the column web, stiffeners can be added. If horizontal stiffeners are added in the column web in the extension of beam flange, the resistance in tension and compression is increased due to stiffeners transfer the concentrated forces. In addition, the shear resistance is increased due to formation of plastic hinge in the column flanges.

3.1.3.3 Bolted end-plate beam-to-column joints

Bolted connection is more frequently used than other connections method due to flexibility of assembly. It is easy to operate, and no special equipment is required. This type of connection is made possible through the use of welding. The end-plate is connected to the beam web or end through welding and bolted to the column flange. The individual components such as a joint similar in respect to the column, i.e. column web in tension, compression and shear, and the compression zone of the beam. The tension zone behaves like two T-stubs, one referring to the column flange and one referring to the end-plate as shown in the figure below:



Figur 3.11: Bolted end-plate joints [3]

So, we can see detail information about bolts and welding in the next subsections (Eurocode 3 section).

3.1.4 Eurocode 3

Eurocode 3 is very important in designing of steel buildings and civil engineering works. In this section mainly focused on the design of connections of steel structure. The Eurocode 3 is used as a source for this chapter[6].

3.1.4.1 Steel cross-section classes

The importance of the classification of the cross-section is to identify the resistance and rotational capacity of the cross-section which is restricted by the local buckling resistance. There are four types of classes of cross-sections:

Class I cross-section: Cross-sections develop plastic moment and have sufficient for plastic hinge to form.

Class II cross-section: Cross-sections develop a plastic moment, but have limited rotation capacity due to local buckling.

Class III cross-section: Cross-sections cannot develop plastic moment resistance due to local buckling, however, it can develop the elastic and yield moment.

Class IV cross-section: Cross-sections develop a limit moment smaller than elastic moment due to early local buckling.

Therefore, width to thickness ratio is used to classify the cross-section parts when it subjected to the compression (either totally or partially).

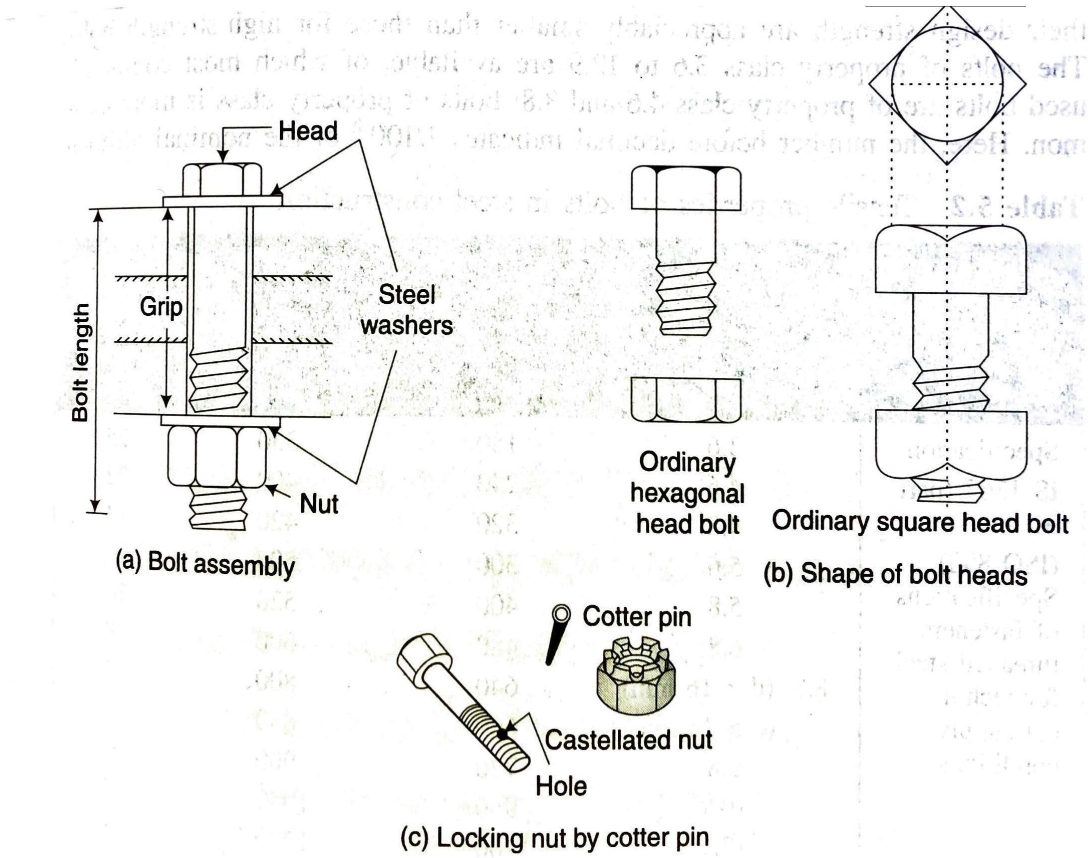
3.1.4.2 Connections with Bolts

Bolts are composed of head and shank which has a threaded and and unthreaded parts. Together with washer, nuts, and lock washer they form bolting assemblies.

Based on EN 1993-1-8: the yield strength $f_y b$ and the ultimate tensile strength $f_u b$ for bolt classes $4.6, 4.8, 5.6, 5.8, 6.8, 8.8 and 10.9$ illustrated in the figure below.

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

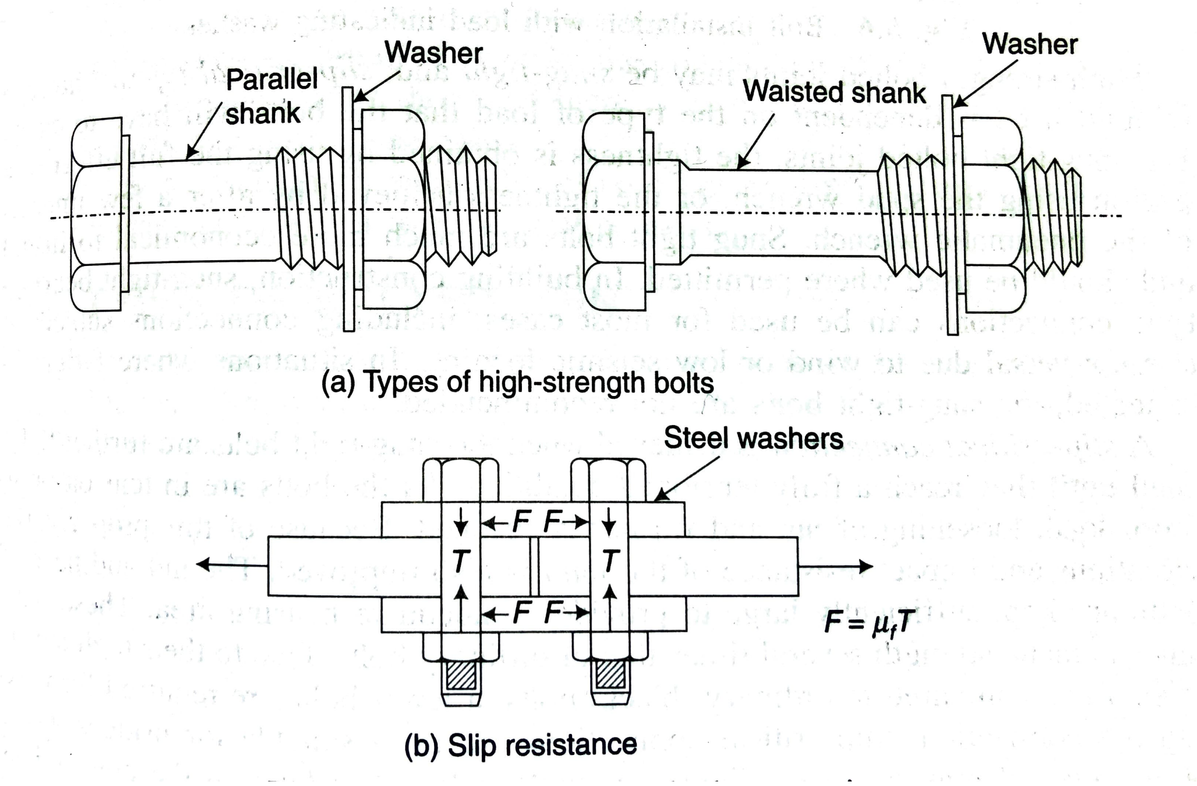
Figur 3.12: Values for yield strength $f_y b$ and tensile ultimate tensile strength $f_u b$[6]



Figur 3.13: Bolt configuration[7]

Preloaded bolts:

Bolt collection of clauses 8.8 and 10.9 relevant to the requirement 1.2.4 Reference Standard given in EN-1993. Group 4 applies for high strength structural bolting with controlled tightening in accordance with requirements. Standard group 7 is about execution of steel structures or requirement for execution of steel structures. This group can be used preloaded bolts.



Figur 3.14: High-strength bolt [7]

Categories of bolted connections

When bolted connection acts under shear condition the design should be performs based on the following categories:

Category A – in this category bolt from class 4.6 up to class 10.9(including class 10.9) should be used. This category practically used for shear connection.

Category B – slip resistance at serviceability state. In this category preloaded bolts class 8.8 and 10.9 should be used. At the serviceability limit state slip should not occur definitely.

Category C – Slip resistance at ultimate state. In this category only preloaded bolts classes 8.8 and 10.9 should be used. Slip must not occur at the ultimate limit state.

When connection acts under tension condition the design should be performed in accordance to categories D and E.

Category D (non-preloaded)- in this category bolts from class 4.6 up to and including class 10.9 can be used. This category not recommended to use in the connection that subjected to periodic tensile loading. They can be used in connection designed to resist wind loads.

Category E (preloaded)-in this category preloaded classes 8.8 and 10.9 with controlled tightening in accordance with Reference Standard group 7 which given in EN 1993.

Here, the figures below shows the illustration of bolt connection and it's categories.

Figur 3.15: Bolted connection[8]

Category	Criteria	Remarks
Shear connections		
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ AC2 $\sum F_{v,Ed} \leq N_{net,Rd}$ AC2	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
Tension connections		
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.

Figur 3.16: Categories of bolted connection[6]

Note: The design force $F(t, Ed)$ should include any force due to prying force.

Prying force: where fasteners are required to carry an applied force, they should be designed to resist the additional forces due to prying. Bolts subjected to both shear and tensile forces should also satisfy the criteria that given in figure 3.14. If preloaded bolt is not exactly used in the design calculation for the purpose of the slip resistance but is required for the inspection or quality measurement (for example durability), then the level of preloaded can be specified in the National Annex.

3.1.4.3 Welded connections

Joining of two structural elements by welding means is called welded connection. Welded connections have the following privileges than other types fasteners to be used for joining a steel structures connection:

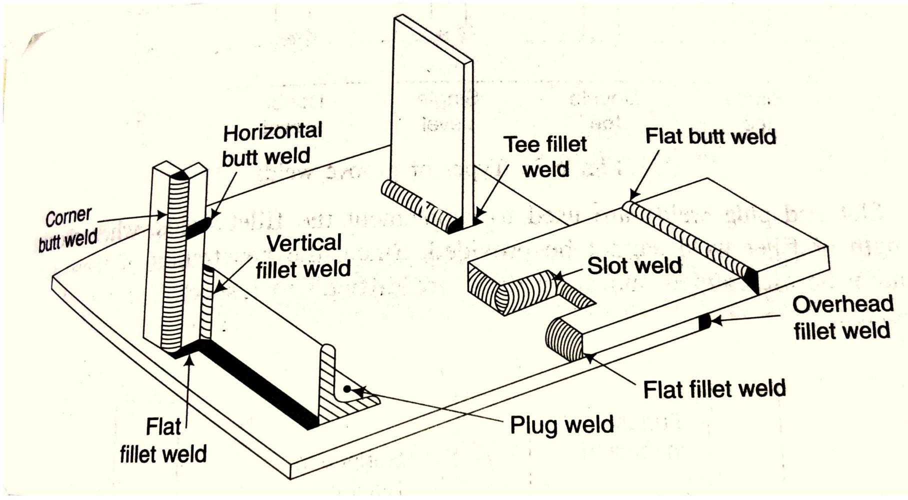
- Welded design provides the opportunities to achieve more effective use of material. Welding is the only process that makes a one piece of construction.
- Welding process save the material consumption, as well as the weight of the building.
- Avoid hole on the structural components that leads to reduction of load bearing ability of the structure.
- Welded joints have better performance for fatigue loads, load impacts and vibration.

Figur 3.17: Welded connection[9]

More over a properly welded joint is more stronger than the jointed material. In a seismic resistance structure applied lateral forces, it is important that the structure should have sufficient stiffness to resist these applied lateral loads and avoid deformation. Fused joints created a rigid structure in comparison to the non rigid structure made of other types of joints. The compactness and high rigidity of welded joints permit design assumption to be performed more accurately. Using welding to joint a connection of the structural element increases the rigidity in the connection, resulting in reduction of the cross sectional area of the beam (width and depth) which is relevant to seismic design of earthquake resistance structure. As well as using welding allow architect and building engineers freedom in choice on designing phase of the structure.

Classification of welding:

Welded joints can be classified depending on the type of welding are: fillet welds, butt welds, plug weld, slot weld, spot weld etc. Depend on the positioning of welds are classified into flat weld, horizontal weld and overhead weld. On the type of joints, they classified into butt welded, lap welded, tee welded and corner welded.[7]



Figur 3.18: Types and positions of welds [7]

Eurocode 3 covers design of the fillet welds, fillet welds all around butt welds, plug and flare groove welds. Butt weld can be either full or partial penetration butt welds. Fillet weld can be used for connecting parts where fusion face from an angle of between $60°to120°$. Angles smaller than are also permitted, but in such cases the weld should be considered to be partial penetration.

3.1.5 Stabilizing system

The main function of stabilizing system is to transfer horizontal loads such as wind and earthquake to ground. In steel buildings, there are two ways to provide lateral stability and resist horizontal forces. The two ways are, moment resisting frame and vertical bracing systems. There are different types of vertical bracing systems, however, we have only cross bracing system been used in this thesis.

3.1.5.1 Moment resisting frame (MRF)

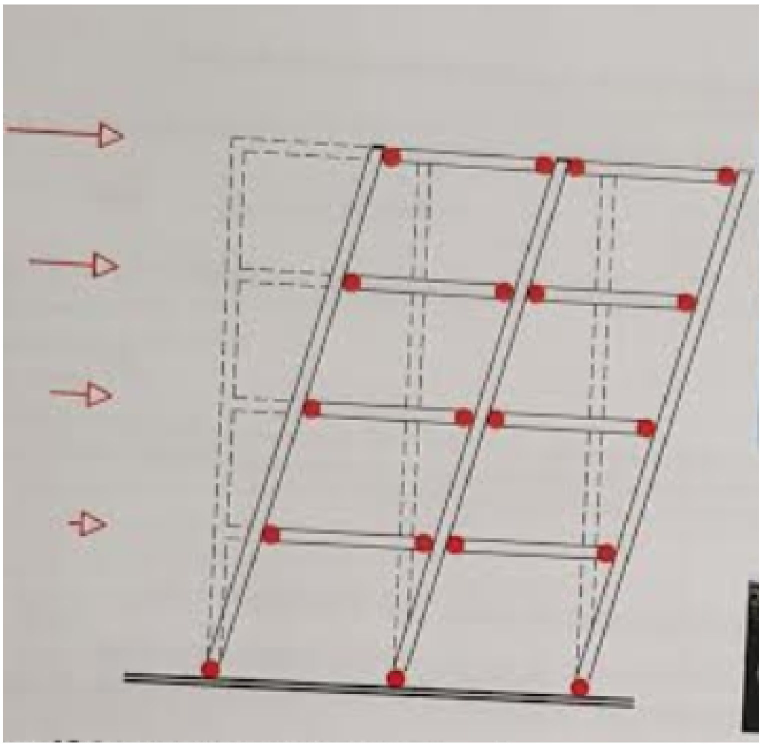
Moment resisting frame is a rectilinear assemblage of beams and columns that require rigid connection between beams and columns. MRF provided lateral stability to the structure by developing bending moment and shear force in frame members and connections/joints. In seismic region, MRF requires the arrangement of full strength/rigid connection in both main directions, producing 3D frame. The horizontal forces resisted by the frame action. 3D moment resisting frame have large margin possibilities to redistribute force and develop post elastic behavior due to large static redundancy. The column can be hollow-section or crossed double I-section for providing sufficient rigidity in both directions. Or MRF arrange only in the perimeter of the building as alternative, and the internal frames are supplied with the simple beam-to-column connection(pinned connection)[3].

MRF are used frequently in low -rise and mid-rise buildings located in high seismicity region due to high ductility. In these types of structures, we can use different types of requirements either strong-column weak-beam design requirement or strong-beam weak-column requirement. Strong-column weak-beam design requirement in large column section and overdesign in low-rise long span buildings.This means, the column has high stiffness than beam. As we know stiffness is inversely proportional to length, therefore, the column is more stiffer than beam because of less in length. In these structures, the strong-column weak-beam design approach is used to allow for plastic hinges to develop in the beams prior to the columns and increases the ductility of the structure to prevent collapse. However, low redundancy and lack of redistribution capacity are the main disadvantages of using this approach (Strong-column weak-beam)[14]. Generally, Strong-column weak beam design determined by the following points:

- The structure can have only local failure
- Can show sign of failure
- Exposed to partial collapse

For a MRF, the behavior factor q can be choose up to 4 in DCM (ductility class medium). Here in the construction below, the beams deform plastically (plastic deformation), and all plastic joints together gives a compatible deformation [10]. Eurocode 8 advice that the energy-absorbing parts should be designed in plastic hinges in the beams near to the joint points. However, the energy-absorbing zones in some places of columns is allowed by Eurocode 8 such as:

- At the top of the frame
- In the top of the column in the top floor
- In the top and bottom of the columns in single floor if the requirements of axial force in the column is satisfied.



Figur 3.19: Illustration of frame construction in DCM [10]

For MRF, the main source of ductility is rotational capacity of plastic hinge. Plastic hinge is used to determine the deformation of beam section where plastic bending occurs, or energy damping device to allow plastic deformation. Moment resisting beam-to-column connection provide to resist horizontal force by flexure and shear in the beam and column. Therefore, the ductility of moment resisting frame develops through flexural yielding of beams, shear yielding of columns panel zone, and flexural yielding of columns. However, the flexural yielding of column can lead to soft story collapse, so it is not preferable in seismic region [15].

Design and detailing rules according Eurocode 8

In seismic region, structural element and structure as a whole should have sufficient ductility for able to develop plastic deformation through introducing of dissipative energy in seismic event. In strong earthquake, moment resisting frame intended to develop plasticity through the formation of adequate number of plastic hinge that provided with sufficient overstrength. [16].

In moment resisting frame, plastic hinge should be design in the beams or in the beams-to-columns connection, not designed in the columns according EC 8 4.4.2.3 (global and local ductility conditions). This concept or requirement is not considered at the base of the frame and at the top level of multi-story buildings and for single story buildings.

In this frame, the dissipative zones should have adequate ductility and resistance depend on where the dissipative zone is located. If the dissipative zones are located in the connections, the connected member should have enough overstrength for allowing the development of cyclic yielding in the connections. If the dissipative zone located in the structural members, the none-dissipative parts and connection of the dissipative parts of the remained structures must have adequate over strength for allowing the development of cyclic yielding in the dissipative parts.

Beams in MRF: the beam should have enough resistance against lateral and lateral-torsional buckling assuming a plastic hinge at its most loaded end, under the seismic design situation. In these end cross-section of plastic hinges, it is to be ensure that the plastic resistance and rotation capacity are not limited by axial compression and shear force. To this end of the beam, for the cross-section of class I and II to secure the formation of hinges, and verified by the following equations (EC 8, 6.6):

$$\frac{M_{Ed}}{M_{Pl,Rd}} \leq 1.0 \tag{3.1}$$

$$\frac{N_{Ed}}{N_{Pl,Rd}} \leq 0.15 \tag{3.2}$$

$$\frac{V_{Ed}}{V_{Pl,Rd}} \leq 0.5 \tag{3.3}$$

Where,

$V_{Ed} = V_{Ed,G} + V_{Ed,M}$

N_{Ed}, M_{Ed}, and V_{Ed} is the design axial force, bending moment, and shear force respectively.

$N_{Pl,Rd}$, $M_{Pl,Rd}$, and $V_{Pl,Rd}$ is design resistance (according EN:1993).

$V_{Ed,G}$ is design value of shear force due to non-seismic action

$V_{Ed,M}$ is design value of shear force due to application of plastic moments.

Columns: columns should be verified for the capacity values of compression force considering the most unfavorable combination of axial force and bending moments.(Eurocode 8, 6.6.3)

In column the N_{ED}, M_{Ed}, and V_{Ed} can be computed as follows:

$$N_{ED} = N_{Ed,G} + 1.1\gamma_{ov}.\Omega N_{Ed,E} \tag{3.4}$$

$$M_{Ed} = M_{Ed,G} + 1.1\gamma_{ov}.\Omega M_{Ed,E} \tag{3.5}$$

$$V_{Ed} = V_{Ed,G} + 1.1\gamma_{ov}.\Omega M_{Ed,E} \tag{3.6}$$

Where,

$N_{ED,G}$, $M_{Ed,G}$, and $V_{Ed,G}$ is compression force, bending moment , and shear force in the column due to non-seismic action.

$N_{ED,E}$, $M_{Ed,E}$, and $V_{Ed,E}$ is compression force, bending moment, and shear force in the column due to design seismic action.

γ_{ov} is overstrength factor

Ω is the minimum value of $\Omega_i = \frac{M_{Pl,Rd.i}}{M_{Ed.i}}$, where $M_{Ed,i}$ is the design value of bending moment i in the seismic design situation and $M_{Pl,Rd,i}$ is the corresponding plastic moment.

The column shear force V_{Ed} resulting from structural analysis should be satisfy by the equation of:

$$\frac{V_{Ed}}{V_{Pl,Rd}} \leq 0.5 \tag{3.7}$$

The shear resistance of framed web panel of beam-to-column connection should also be satisfied by the following equation:

$$\frac{V_{Wp,Ed}}{V_{wp,Rd}} \leq 1.0 \tag{3.8}$$

Where,

$V_{Wp,Ed}$ is design shear force in the web panel, and $V_{Wp,Rd}$ is shear resistance (shear buckling resistance) of the web panel.

Beam-to-column connection in MRF:

If the structure is intended to design the dissipative energy in the beams, the beam-to-column connection should be checked the capacity design criteria, to possess sufficient over-strength against the beams. At the plastic hinges, it is required that the cross-section develop the full plastic strength. Moreover, it is required that the joints have sufficient rotational capacity, to offer the possibility of a moment redistribution, during the formation of plastic hinges, and of the development of final full plastic mechanism, without the appearance of local and buckling phenomena. The rotation capacity of θ_p of the plastic region is defined by:

$$\theta_p = \frac{\delta}{0.5L} \tag{3.9}$$

where, δ is the beam‘s deflection at the mid span, and L is length of the beam‘s span.

The rotation capacity of the plastic region should not be less than 35 mrad (milliradian) in the structure of DHC (high ductility class) and not less than 25 mrad for a DCM (medium ductility class with q greater than 2).

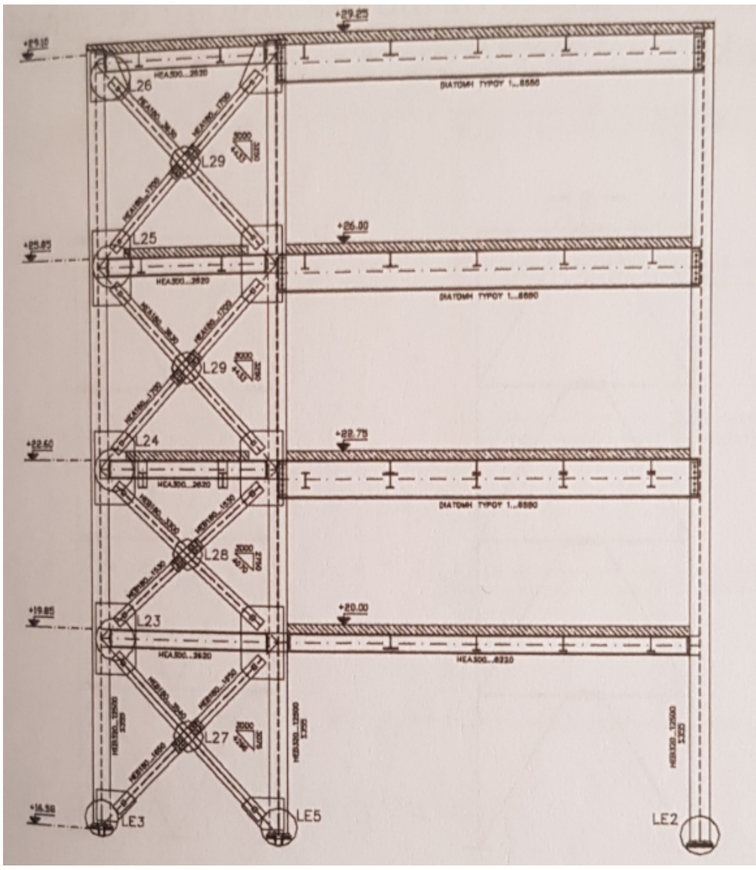
The member of frame in the connection should demonstrate to be stable at the ultimate limit state (ULS). Column design capacity should be taken from the plastic capacity of the connections, if partial strength connections are used. However, the column elastic deformation is excluded the evaluation of rotational capacity of plastic hinge region.(Euro code 8,6.6.4)

3.1.5.2 Cross-bracing frame (CBF)

Cross-bracing (x-bracing) is a popular kind of the concentrated braced frame with high strength and stiffness, and with compacted architectural form frequently used to resist wind and seismic loading. Regarding the fulfillment of the serviceability requirement (the need to limit inter-story drift demand under seismic events having a return period comparable with the service of life of the structure), x-braced system provide best solution by having maximum lateral stiffness compared to MRF. The ductile response of the x-bracing depends on the capability of bracing members of sustaining large inelastic displacement is being reverse without significant loss of strength and stiffness[17].

In cross- braced system, the axial force is developed in the member of bracing for resisting horizontal force, and considered only the diagonals under tension in resistance of the horizontal force. Cross bracing are considered as concentric type of bracing which are very slender, has high tensile capacity and posses very low compressive buckling. In comparison to moment resistance frame the energy dissipation during an earthquake is low. So, due to poor inelastic behavior under sever seismic loading, X-bracing is not preferred to use in very high seismisity region. This means, more energy is dissipated in a brace yielding in tension rather than in a brace buckling in compression. The energy dissipation of the X bracing system impresses by post buckling brace behaviour. This characteristic is different for slender(thin) braces than the thick braces. Slender or thin braces has more abilities of energy dissipation than thicker braces, but thick braces are able to resist more loading cycles and larger inelastic deformation. Connections in concentric bracing should be stronger than the other members of bracing system [18].

In cross-bracing two diagonal members intersect each other at mid-span and show rather complex behavior than single diagonal bracing during cyclic loading. This type of bracing is intended to dissipate energy and sustain large deformation demands through buckling in compression and yielding in tension. However, the design of cross-bracing have a limited capacity philosophy that requiring the strength of the brace connecting element and frame members to exceed the expected yield strength of the brace. That means, cross-bracing with active tensile diagonals, where horizontal forces can be resisted by the tensile diagonals only (T-O model) [19].



Figur 3.20: Concentric cross-braced system [3]

The structural model of braced frame is illustrates in the figure above that determine and design the connection of beams, columns and braces gusset plates in the model. During analyses of the structure, all the members such as beams, columns and gusset plates are divided in to sub-elements using square or cubic finite elements when we use FEM model. According the standard seismic design codes, beams and columns of the braced frame must behave elastically while the bracing have to be the only parts that deform plastically during a seismic strong motion [19].

Design and detail rules of concentric x-bracing

In terms of seismic design, the tensile diagonals are dissipative members of the structure and used to develop yielding of tensile diagonal before the failure of the connection at the end of the diagonal, as well as before yielding or buckling of the beams and columns. Eurocode 8 is allowed the dissipative elements to be in connection between the member of the structure. The beam and column are considered to resist gravity load, not taking into account the bracing system [16].

The requirement of Eurocode for concentric x-bracings of multi-story have the following additional remark:

a)The structure of multi-story, the limitation of the x-diagonal‘s slenderness ratio is $1.3 \leq \lambda \leq 2.0$, however in structure up two story, the limitation of slenderness ratio does not apply.

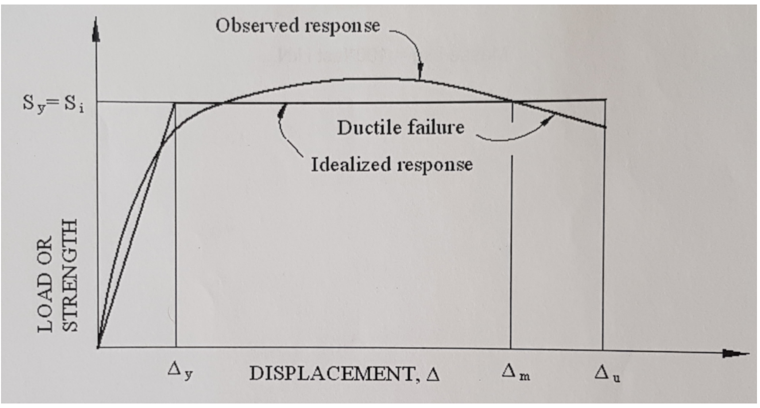
b)To obtain the homogeneous dissipative behavior of the diagonals, at all levels, it should be verified that the maximum value of overstrength factor Ω does not differ from the minimum Ω by more than 25%. Or $\Omega \leq 1.25$.

c) For regular structure, the behavior factor q is equal to 4 for both medium and high ductility class.

d) To calculate the internal force, EC8 allows to perform a linear elastic analysis a T-O model (tension only) where the contribution of compression diagonals is neglected.

3.1.6 Ductility

Ductility is the ability of materials to undergo significant plastic deformation with out fracture when it exposed to the applied load. On the other hands, ductility is the ability of the structure to undergoes large plastic deformation with out significant loss of strength, or the ratio between ultimate deformation and the yield deformation [20]. When the material is more ductile, it has high ability to deform under applied loading. Ductility is measured by the amount of permanent deformation that indicates by the stress-strain curve.



Figur 3.21: Strength versus deformation[10]

The figure above shows the strength versus deformation. This implies that how the strength is required for the structure to hold linear elastic or elastoplastic during the ground motion.

Moment resisting frame and cross-braced frame have commonly used as lateral load resisting structure element in steel buildings. Moment resisting frame provide ductility through yielding, but due to their flexibility, they do not satisfy the stiffness criteria. The diagonal x-braced provide more lateral stiffness to the structure comparing with MRF [21].

3.1.6.1 Choice of ductility class for earthquake resistance steel structure

The earthquake load that causes the structure being displaced. The displacement of the structure will follow the ground’s acceleration direction. The structure will move from its rest position, the vibration period of the structure depends mainly on the magnitude of the earthquake, geographical conditions, material properties of the structure such as stiffness and ductility.

When steel structures subjected to earthquake loads,it behaves better than other structures made of another materials due to high strength and ductility. A steel structure consist of many structural members having low sensitivity of bending resistance of the structural elements to the presence of occurring axial force. The structural elements (beam, column) and their connections (joints) make several plastic hinges that lead to energy dissipation of seismic loads. Therefore, the choice of ductility is important for the designing of individual structural elements and their connections. Based on ductility classification and behavior factor used in design, the Eurocode 8 defines the cross-sectional class for all dissipative steel members [4].

The capacity of steel members to dissipate energy is located in the dissipative areas. Hence with higher ductility the use of cross-sectional I is important. That means the given cross- section can form plastic hinge with rotation capacity without any reduction of its resistance.

3.2 Earthquake Design

3.2.1 Dynamics

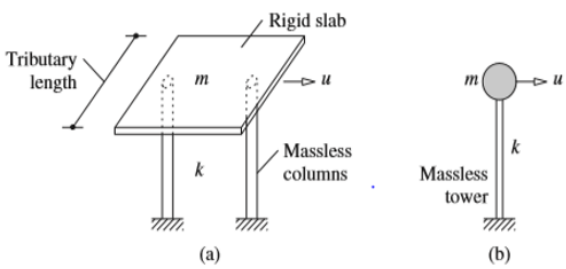
Dynamics is classical mechanics that describes how the force influences the body movement, stability and equilibrium. So, the knowledge of basic principle of dynamics is used to understand how seismic force act on a structure when the earthquake happens.

The load effects from the earthquake on building occur due to the ground acceleration and displacement causes the structure to move and deform. The shaking and the structural response in relation to the earthquake can vary from one type of building to another. When the earthquake happened, some buildings would be damaged totally, while other buildings may not be, regardless they exposed to the same magnitude of the earthquake. In this condition, the import parameters are mass and stiffness of the buildings as well as their distributions besides the geographical location and basic conditions. Therefore, the theory of dynamics of this project is based on the book ''Dynamics of Structures" that is written by Chopra [20]. All figures and text have this source in the Dynamics section unless specified otherwise.

N.B: The theory of dynamics have been taken direct from our previous pre-project, we have just only summarized.

3.2.1.1 Single degree of freedom

A single degree of freedom system (SDF) needs just one parameter to define its position at any time interval. The minimum number of separate parameters needs to determine the position of the parts of a system at any interval of time defines the degrees of freedom (DOFs) of the system. For structural dynamics analysis, a simple structure has only one DOF, lateral displacement. This is different from static analysis where two joint rotations would be included in the analysis. And this system is to be considered as an idealized of one-story structure. Each structural member of the real structure contributes to the mass, stiffness, and damping (dissipation energy) properties of the structure. The idealized single degree of freedom where mass m is supported of mass.



Figur 3.22: Idealized of SDF

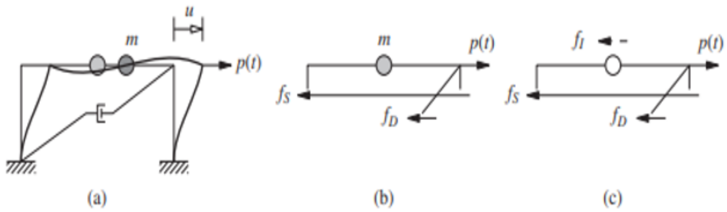
Simple structures can be determine as a structure with a concentrated mass and a massless frame. The frame has stiffness (k) in the lateral direction. There will be an increase in the excitation of the structure, When an external lateral force is applied to this structure. This excitation can lead to a displacement or deformation of the structure and that can be defined with the help of differential equation. The equation can be expressed as follows:

$$m\ddot{u} + ku = 0 \tag{3.10}$$

where m is mass, $\ddot{u}$ is acceleration, k is stiffness and u is deformation.

3.2.1.2 Equation of motion

The equation of motion is derived from a simple one-story frame with viscous damping and external force. The equation varies with time as shown in the figure below:



Figur 3.23: Applied force on SDF system

Newton's second law give the following equation;

$$p - f_D - f_s = m\ddot{u} \tag{3.11}$$

The equation of motion for the deformation u(t) of a simple structure is depend on the figure above. The system is assumed to be linear elastic. The equation of motion for an inelastic system is similar but the equation $f_s = ku$ must be replaced as this is only for linear deformations. This gives the following equation of motion for an inelastic system:

$$m\ddot{u} + c\dot{u} + f_s(u) = p(t) \tag{3.12}$$

If a structure is subjected to a ground motion, the equation of motion may be rewritten as:

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g(t) \tag{3.13}$$

The result is the equation governing the structure subjected to ground motion. This equation is quite similar to the other equations of motion. Therefore, the forces, external force and ground acceleration are the same. The effective earthquake force can replace the ground motion.

$$p_{eff} = -m\ddot{u}_g(t) \tag{3.14}$$

The effective earthquake force is therefore proportional to the mass of the structure but acting opposite to the ground acceleration.

3.2.1.3 Undamped free vibration

Free vibration is the state when a structure is moved from its resting position and then allowed to move freely without any dynamic excitation. The concept of free vibration gives a better understanding of the natural frequency and damping force of an SDF system.

The free vibration is initiated by giving the mass some displacement u(0) and velocity $\dot{u}(0)$ at time zero.

$$u = u(0), \dot{u} = \dot{u}(0) \tag{3.15}$$

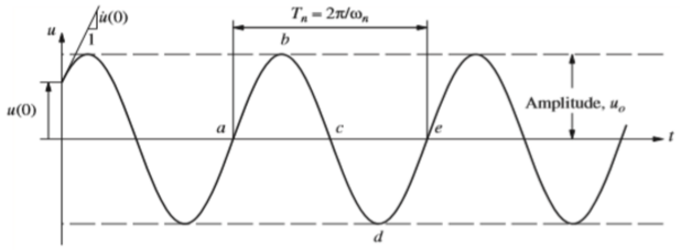
The solution to the homogeneous differential equation can be expressed by the following equation:

$$u(t) = u(0)cos\omega_n t + \frac{\dot{u}(0)}{\omega_n} sin\omega_n t \tag{3.16}$$

Where,

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

ω_n is the natural frequency of vibration



Figur 3.24: Undamped free vibration

The movement in above figure repeats itself by every $\frac{2\pi}{\omega_n}$ second. This simple harmonic motion has a natural period of vibration(Tn) that stretches from point a-e. Tn is the time required to complete one cycle of vibration for an undamped system. The units here are radians per second.

$$T_n = \frac{2\pi}{\omega_n} = \frac{1}{f_n} \tag{3.17}$$

The amplitude of the system is denoted as u_0. The term of undamped natural vibration is used to describe the properties of a system that is allowed to vibrate without any external influence. $\frac{1}{T_n}$ cycles are completed per second by the system. The units for f_n are hertz(Hz) cycles per second.

The concept of natural frequency of vibration is used both for ω_n and f_n. Both the natural frequency of vibration ω_n and the natural cyclic frequency of vibration (f_n) depend on the stiffness and mass of the structure.

3.2.1.4 Damped free vibration

The other type of free vibration is damped vibration, which is the force from surrounding (frictional force) contributes to diminish the amplitude of oscillation until the system is at rest.

Putting external force p(t)=0 and use the equation of motion will be:

$$m\ddot{u} + c\dot{u} + ku = 0 \tag{3.18}$$

Dividing the equation by mass(m) and it becomes as follows:

$$\ddot{u} + 2\zeta.\omega_n^2\dot{u} + \omega_n^2 u = 0 \tag{3.19}$$

$$\zeta = \frac{C}{2m\omega_n} = \frac{C}{C_{cr}} \tag{3.20}$$

Where C_{cr} is critical damping coefficient and can be written as $C_{cr} = 2m\omega_n = 2\sqrt{km} = \frac{2k}{\omega_n}$

ζ- is the damping ratio

There are different types of motion as mentioned below:

$\zeta = 1$ or $C = C_{cr}$ critical damping

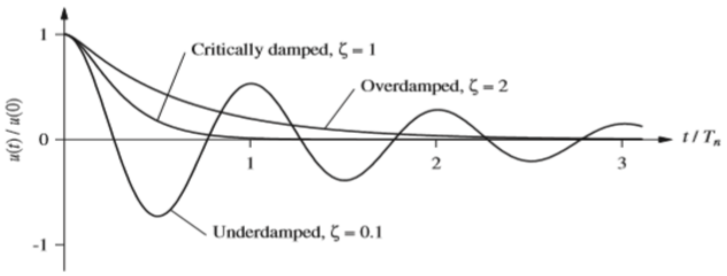
$\zeta = 0$ no damping

$\zeta > 1$ or $C > C_{cr}$ is overdamping

$\zeta < 1$ or $C < C_{cr}$ is underdamping

The damping coefficient c is a measure of resistance offered by damper against the motion or in other words this is a measure of the energy that dissipates in a cycle of free vibration or in a cycle of forced vibration.

C_{cr} or critical damping coefficient is the smallest value of the c that prevents oscillation completely. It is the line dividing between the oscillatory and non-oscillatory motion. For structures like buildings, bridges, dams, offshore structures and other structures the value of $C < C_{cr}$ takes in consideration a damping ratio less than 0.10. And in this thesis, we use an underdamping system where the body oscillates from and back at the equilibrium position and amplitudes decreases linearly.



Figur 3.25: Damped free vibration

The equation that governed for the underdamped systems will express as follows:

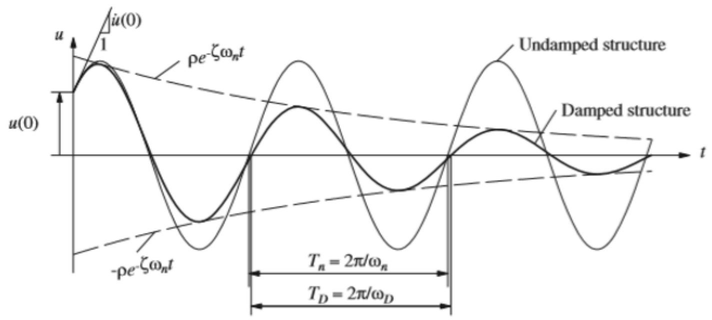
$$u(t) = e^{-\zeta\omega_n t}\Big[u(0)cos\omega_D t + \frac{\dot{u} + \zeta.\omega_n u(0)}{\omega_D sin\omega_D t}\Big] \tag{3.21}$$

Where ω_D is the natural frequency of the damped vibration equal to $\omega_n\sqrt{1-\zeta}$ or:

$$\omega_D = \omega_n\sqrt{1-\zeta} \tag{3.22}$$

T_D- the natural period of damped vibration, $T_D = \frac{2\pi}{\omega_D}$ is referred to the natural period T_n without damping by the following equation:

$$T_D = \frac{T_n}{\sqrt{1-\zeta^2}} \tag{3.23}$$



Figur 3.26: Damped and undamped structure

The figure above shows that the displacement of amplitude for an undamped system will be the same in all vibration cycles, while for the damped system oscillation with amplitude is decreasing with every cycle of vibration. The figure shows that the deformation amplitude decreases exponentially with time.

3.2.1.5 Undamped forced vibration

A system that derived with external force is forced vibration. The undamped forced vibration is exposed to harmonic external force with its frequency, and the force defined by; $p(t) = p_0 sin\omega t$ or $p_0 cos\omega t$.

p_0- is an amplitude or the maximum value of the force, its frequency ω is called exciting frequency or forcing frequency.

$p(t) = p_0 sin\omega t$ gives the differential equation of motion that represents to the forced harmonic forced vibration system and written as follows:

$$m\ddot{u} + ku = p_0 sin\omega t \tag{3.24}$$

If displacement $u(0) = 0$ and velocity $\dot{u}(0) = 0$. The equation has a complementary solution $u_c(t)$ imposing at initial condition, and the particular solution $u_p(t)$ describes the response of the free vibration.

$$u_p(t) = \frac{p_0}{k}\frac{1}{1-(\omega/\omega_n)^2} sin\omega t \tag{3.25}$$

$$u_c(t) = Acos\omega_n t + Bsin\omega_n t \tag{3.26}$$

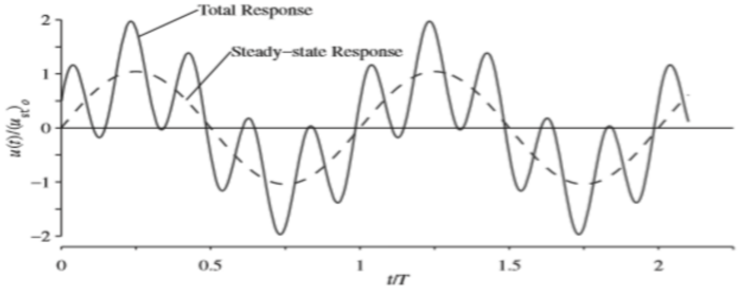
A and B are constant, determined by imposing the initial conditions: $u = u(0)$ and $\dot{u} = u(0)$. The final result is the summation of complementary and particular solution can be written as:

$$u(t) = Acos\omega_n t + Bsin\omega_n t + \frac{p_0}{k}\frac{1}{1-(\omega/\omega_n)} sin\omega t \tag{3.27}$$

A and B determined by imposing the initial conditions: $A = u(0)$ and $\dot{u} = u(0)$

$$B = \left[\frac{\dot{u}(0)}{\omega_n} - \frac{p_0}{k}\frac{\omega/\omega_n}{1-(\omega/\omega_n)^2}\right] \tag{3.28}$$

Plotting the equation 3.28 for $\omega/\omega_n = 0.2$, initial displacement $u(0) = 0.5\frac{p_0}{k}$ and initial velocity $\dot{u}(0) = \omega_n\frac{p_0}{k}$, as shown in the figure below.



Figur 3.27: Harmonic forces for undamped system

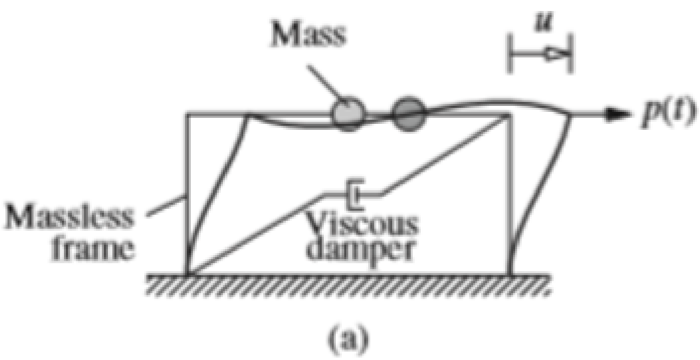
From the figure above, the steady state dynamic response- is a sinusoidal oscillation at the forcing frequency and the maximum value of static deformation can be written in the following two equations:

$$u(t) = u_{st0}\Big[\frac{1}{1-(\omega/\omega_n)^2}\Big]sin\omega t \tag{3.29}$$

$$u_{st0} = \frac{p_0}{k} \tag{3.30}$$

When we plotted the ratio of forced frequency to the natural frequency ω/ω_n it is easy to understand how the system displaced in relation of the acting force.

If $\omega/\omega_n < 1$ or $\omega < \omega_n$, it indicates that, deformation or displacement u(t) and external force u(p) have the same direction. As it is mentioned in the figure below, when the force acts to the right then the system would be displaced to the right. The displacement is said to be in phase with the applied force.



Figur 3.28: External force and displacement

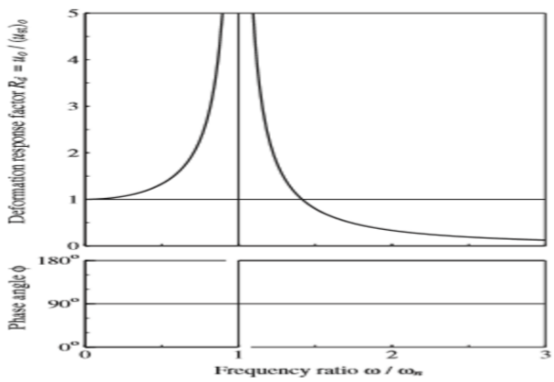
But if $\omega/\omega_n > 1$ or $\omega > \omega_n$, indicates that displacement u(t) and external force will have opposite direction, when the force acts to the right then system would be displaced to the left direction. The displacement is said to be out of phase of applied force.

To comprehend better the concept of phase mathematically, equation 3.29 can be rewrite in terms of amplitude u_0 of the vibratory displacement (dynamic deformation) u(t)and phase angle ϕ:

$$u(t) = u_0 sin(\omega t - \phi) = (u_{st0})(R_d sin(\omega t - \phi) \tag{3.31}$$

Where R_d is the displacement response factor, and can be written as;

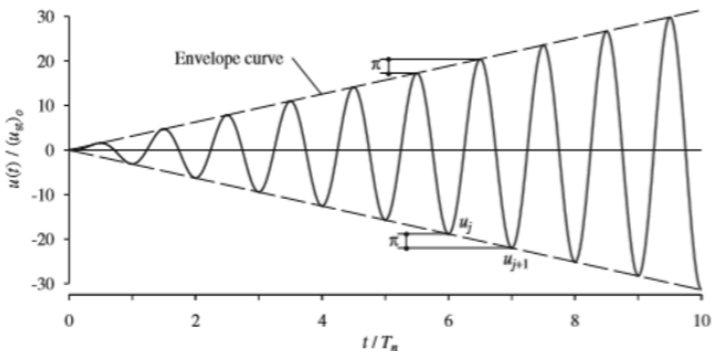
$$R_d = \frac{u_0}{u_{st0}} = \frac{1}{1 - (\omega/\omega_n)^2} \tag{3.32}$$



Figur 3.29: Deformation response and frequency ratio

If the forced frequency is larger than the natural frequency the strain is going to zero, and the construction can have very long natural frequency during the earthquake, and the vibration will be very fast.

When the forced vibration reaches at maximum deformation response factor, is resonance frequency. For undamped system the resonant frequency is a natural frequency with deformation response factor is uncontrolled. However, the dynamic response is to be infinite when the frequency ratio is toward 1. This leads to vibrate forcely and the amplitude increases linearly with time as shown below in the figure. The real structures will reach the yielding point, and the system could fail if the deformation continues. In addition, the stiffness decreases, and its natural frequency would no longer equal to the forced vibration.



Figur 3.30: Resonance for undamped system

3.2.1.6 Damped forced vibration

When the external force is applied to the system, it oscillates and is exposed to the effect of damping . The force varies harmonically with the amplitude and frequency. Considering the equation below for damped motion and external force is:

$$m\ddot{u} + c\dot{u} + ku = p_0 sin\omega t \tag{3.33}$$

The equation is to be solved when the external force subjected to the system, with initial condition of deformation and velocity; $u(0) = 0$ and $\dot{u}(0) = 0$

Then the equation has complementary and particular solutions that describe the systems response can be written as follows:

$$u_p(t) = Csin\omega t + Dcos\omega t \tag{3.34}$$

$$u_c(t) = e^{-\zeta\omega_n t}(Acos\omega_D t + Bsin\omega_D t) \tag{3.35}$$

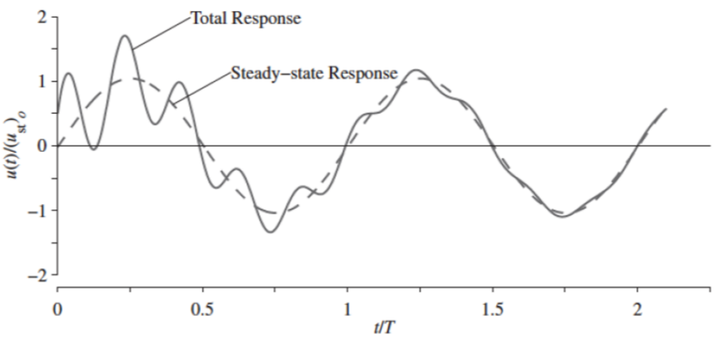
$$C = \frac{p_0}{k}\frac{1-(\omega/\omega_n)^2}{[1-(\omega/\omega_n)^2]^2 + [2\zeta(\omega/\omega_n)]^2} \tag{3.36}$$

$$D = \frac{p_0}{k}\frac{-2\zeta\omega/\omega_n}{[1-(\omega/\omega_n)^2]^2 + [2\zeta(\omega/\omega_n)^2]^2} \tag{3.37}$$

The complete solution will be rewritten, when constant A and B is described as initial condition:

$$u(t) = e^{-\zeta\omega_n t}(Acos\omega_D t + Bsin\omega_D t) + Csin\omega t + Dcos\omega t \tag{3.38}$$

Equation 3.38 can be plotted below as describing the steady state response $u_p(t)$ and total response u(t) with the starting of damping ratio 5%, $\omega/\omega_n = 0.2$, $u(0) = 0.5p_0/k$ and $\dot{u}(0) = \omega_n p_0/k$.



Figur 3.31: Response of damped system to harmonic force

As we shows in the figure 3.31, the solid line represents the total response and the dashed line represents the forced response. The difference between the two graphs are free response that depends on the frequency ratio and damping ratio, and later becomes negligible. This graph also shows that the largest deformation can occur before the systems reached the steady state.

For damped system, the constant A, B, C, and D can be determined when the response for $\omega = \omega_n$ and the initial condition is equal to zero:

$$A = u_{st0}/2\zeta \tag{3.39}$$

$$B = u_{st0}/2\sqrt{1-\zeta^2} \tag{3.40}$$

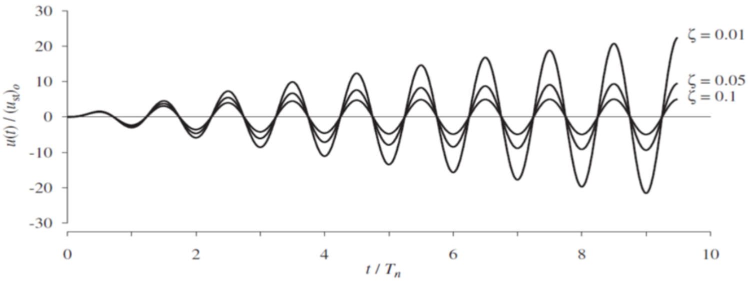
$$C = 0 \tag{3.41}$$

$$D = -u_{st0}/2\zeta \tag{3.42}$$

The total response from equation 3.38 with these constants can be rewritten as:

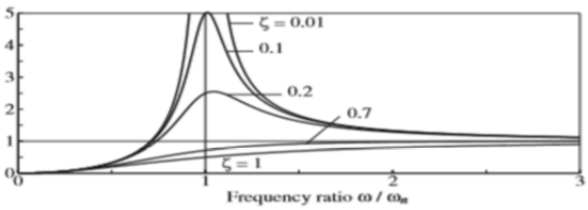
$$u(t) = u_{st0}\frac{1}{2\zeta}\Big[e^{-\zeta\omega_n t}\Big(cos\omega_D t + \frac{\zeta}{\sqrt{1-\zeta}}sin\omega_D t\Big) - cos\omega_n t\Big] \tag{3.43}$$

The total response with different value of damping ratio that applied to the system can be shown in the figure below.



Figur 3.32: Resonance with different damping ratio

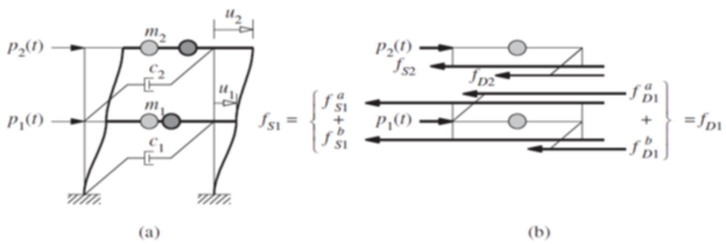
According to figure above, damping in the system is minimum when the damping ratio is large (0.1) and oscillates very low. However, when the damping ratio is small (0.01), damping in the system becomes large, it oscillates very high and the response amplitude becomes smaller and smaller.The figure below determines better on how to illustrate the dynamic amplification with different damping ratio.



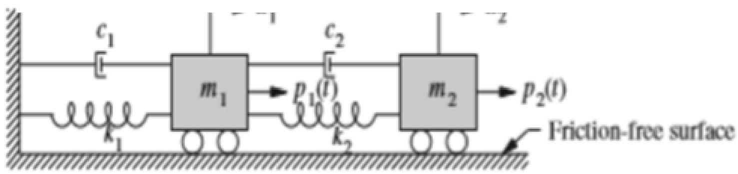
Figur 3.33: Dynamic amplification factor with different damping ratio

3.2.1.7 Multi degree of freedom system

In previous above section, we learned single degree of freedom and how a one-story frame construction is defined. In this section we will study more complicated construction having multi-degree of freedom (MDFs). So, it needs to develop the system to include the frame building construction with MDF, for example, a two-story building with lumped mass on the top. Just in the case of SDFs we assume that the linear viscous damping mechanism is dissipated the energy in the structure. This energy dissipation is followed with displacements u_1 and u_2 in each floor in horizontal direction.



Figur 3.34: Earthquake-induced stress in the frame structure: a)Two story frame b) Force acting on the two masses



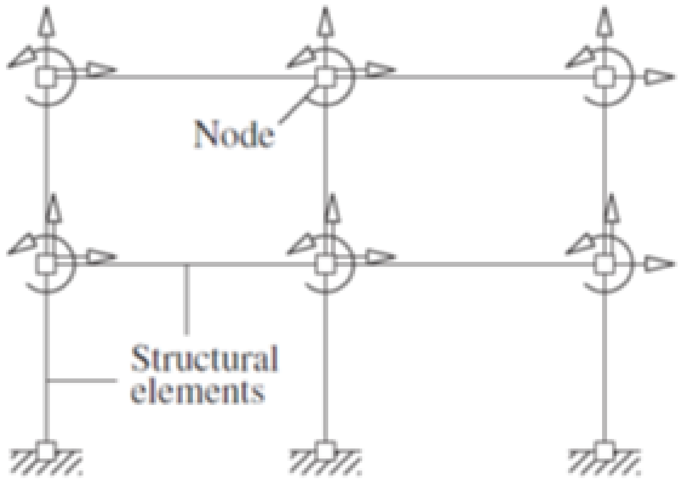
Figur 3.35: Two degree of freedom in a mass-damper-spring system

The force is subjected to each floor mass as shown in the figure above. The forces such as the external force, resisting force, and damping force can be expressed by the equation as:

$$m_j \ddot{u}_j + f_{Dj} + f_{Sj} = p_j(t) \tag{3.44}$$

Where m_j consist of masses m_1 and m_2 which acts on the top of each floor. These include the external forces $p_j(t)$, the elastic or resisting force f_{Sj} and damping force f_{Dj}.

The above equation 3.44 is not governed for complicated structures. Hence, we need to find the solution for the complicated frame constructions having multi degrees of freedom. A frame structure can be idealized as a collection of elements such as beam, column, and walls connected at nodal points or nodes. The displacement of nodes (joints) is the degree of freedom. A node in two - dimensional frame has three degrees of freedom (DOFs), two translation and one is rotation. As well as a node in a three- dimensional frame has six degrees of freedom- three translation and three rotation about the x,y, and z axes. In the figure below illustrates a frame construction with 18 degree of freedom.



Figur 3.36: Axial and rotational deformation, 18 DOFs

3.2.1.8 Stiffness matrix

Relating the external force f_{Sj} on the stiffness component of the structure to the resulting displacement. This relationship for the linear system is obtained by the superposition method and the concept of stiffness influence coefficient. The stiffness influence coefficient k_{ij} is the force required along degree of freedom i due to unit displacement at degree of freedom j.

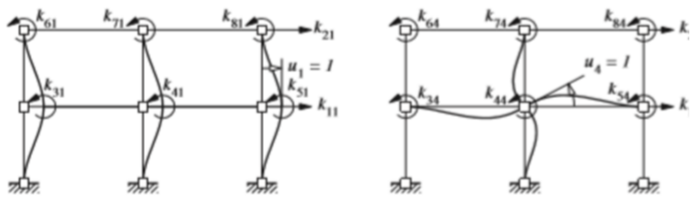
The resistance force f_{Sj} at the degree of freedom, i contributed with displacement u_j, j=1 to N is obtained by superposition as:

$$f_{Si} = k_{i1}u_1 + k_{i2}u_2 + \cdots + k_{ij}u_j + \cdots + k_{iN}u_N \tag{3.45}$$

The equation above shows the force relates with deformation in SDF and can be written in the matrix form for MDF system as:

$$\begin{bmatrix} f_{s1} \\ f_{s2} \\ \vdots \\ f_{sN} \end{bmatrix} = \begin{bmatrix} k_{11} & k_{12} & \cdots & k_{1j} & \cdots & k_{1N} \\ k_{21} & k_{22} & \cdots & k_{2j} & \cdots & k_{2N} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ k_{N1} & k_{N2} & \cdots & k_{Nj} & \cdots & k_{NN} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ \vdots \\ u_n \end{Bmatrix} \tag{3.46}$$

In the above equation, k is the stiffness matrix of the structure. The frame structure that subjected to deformations in the degree of freedom u as shown in the figure below.



Figur 3.37: Deformations for frame buildings with different degrees of freedom

3.2.1.9 Damping matrix

As we know, the energy dissipation mechanism of a vibrating structure can be idealized by equivalent viscous damping system. Taking into account this assumption relating the external force f_{Dj} (damping force) acting to damping component of the structure to the velocity $\dot{u}_j$. By imparting a unit of velocity along degree of freedom j and keeping the velocities in another degree of freedom as zero. Eventually these velocities generate internal damping force that resists the velocities and external forces are necessary to balance these forces.

The damping influence coefficient c_{ij} is the external force in the degree of freedom i due to unit velocity in degree freedom j. The damping force f_{Di} at degree of freedom i contributed with the velocities $\dot{u}_j$, j=1 to N is obtained by superposition:

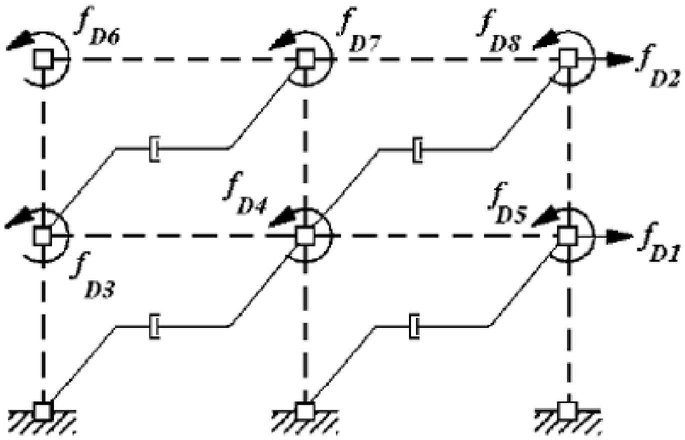
$$f_{Dj} = c_{i1}\dot{u}_1 + c_{i2}\dot{u}_2 + \cdots + c_{ij}\dot{u}_j + \cdots + c_{iN}\dot{u}_N \tag{3.47}$$

Rewrite the above equation in the matrix form for MDFs as:

$$\begin{bmatrix} f_{D1} \\ f_{D2} \\ \vdots \\ f_{DN} \end{bmatrix} = \begin{bmatrix} c_{11} & c_{12} & \cdots & c_{1j} & \cdots & c_{1N} \\ c_{21} & c_{22} & \cdots & c_{2j} & \cdots & c_{2N} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ c_{N1} & c_{N2} & \cdots & c_{Nj} & \cdots & c_{NN} \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \vdots \\ \dot{u}_n \end{Bmatrix} \tag{3.48}$$

c- is the damping matrix.

It is difficult to solve the coefficient of c_{ij} of the damping matrix directly from the dimension and size of the structural elements. Therefore, damping for MDFs can be computed by numerical value for damping ratio based on experimental data as for SDF system.



Figur 3.38: Damping component in frame building

3.2.1.10 Mass matrix

Relating the external force f_{Ij} which acts on the mass component of the structure to the acceleration $\ddot{u}_j$. Applying a unit acceleration along the degree of freedom, and the acceleration in all other degrees of freedom is taking as zero. Based on D' Alembert's principle, the fictitious inertia resists these accelerations, hence external force is necessary to balance these inertia forces. The mass influence coefficient m_{ij} is the external force in degree of freedom i due to unit acceleration along degree of freedom j. For example, the forces m_{i1} are required in different degree of freedom to balance the inertia force related with $\ddot{u}_1 = 1$ while all the other keeping zero, $u_j = 0$ as shown in the figure below (fig. 3.34b).

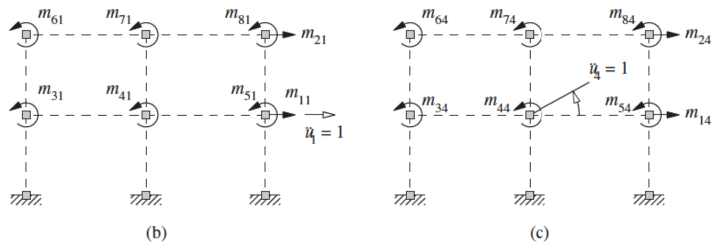
The inertia force f_{Ii} at the degree of freedom, i contributed with acceleration $\ddot{u}_j$, j=1 to N is defined by the equation as follows:

$$f_{Ii} = m_{i1}\ddot{u}_1 + m_{i2}\ddot{u}_2 + \cdots + m_{ij}\ddot{u}_j + \cdots + m_{iN}\ddot{u}_N \tag{3.49}$$

The matrix for the above equation is:

$$\begin{bmatrix} f_{I1} \\ f_{I2} \\ \vdots \\ f_{IN} \end{bmatrix} = \begin{bmatrix} m_{11} & m_{12} & \cdots & m_{1j} & \cdots & m_{1N} \\ m_{21} & m_{22} & \cdots & m_{2j} & \cdots & m_{2N} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ m_{N1} & m_{N2} & \cdots & m_{Nj} & \cdots & m_{NN} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \vdots \\ \ddot{u}_n \end{Bmatrix} \tag{3.50}$$

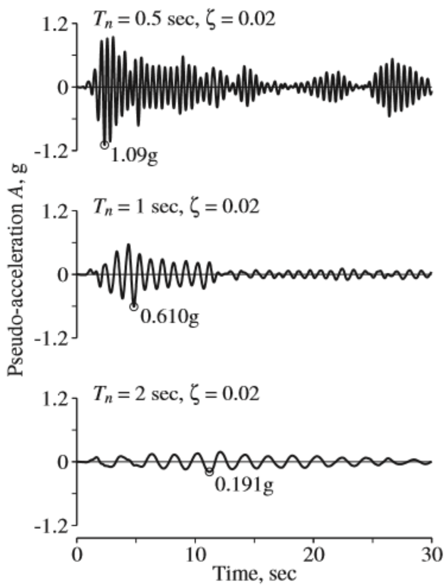
Where m is mass matrix



Figur 3.39: Acceleration of mass in frame construction, b) Mass influence coefficient for $\ddot{u}_1 = 1$, c) Mass influence coefficient for $\ddot{u}_4 = 1$

3.2.1.11 Response spectrum

The concept of response spectrum is introduced by M.A Biot, that describes the characterizing of ground motion and their effects on structures. In earthquake engineering, the response spectrum provided to understand the practical approach of structural dynamics to design of structures and used to develop the requirements of lateral force in building codes. This basic concept in earthquake design, the response spectrum provided to plot a peak response from different SDF systems when it exposed the ground motion. The figure below shows the acceleration response spectrum at different natural vibration period and the same damping ratio.

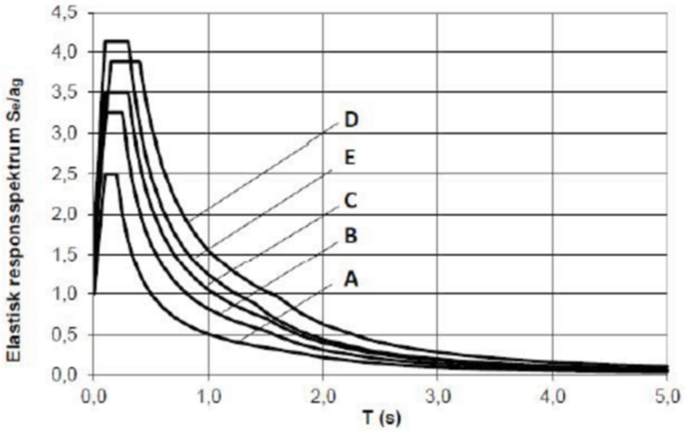


Figur 3.40: Acceleration response at different natural vibration of period

The acceleration response spectrum in the figure above, is as a function of natural vibration period T_n or natural frequency f_n related to the peak deformation of the system.

The response spectrum for a given ground motion can be constructed by using numerical method. For computing this method, it should be define and select ground acceleration, deformation response, natural vibration and damping ratio in order to generate the earthquake response spectrum.

All linear elastic buildings that exposed to the ground acceleration makes response spectrum. In modal analysis, the peak value of the modal response for every swing mode (vibrational mode) can be obtained from response spectrum of the ground motion. When every simple response is combined, can be calculated the total response of the whole buildings.



Figur 3.41: Elastic response spectrum with the ground types

3.2.2 Eurocode 8

3.2.2.1 Engineering and approaches of earthquakes

Eurocode 8 applies to the design and construction of buildings and other civil engineering works in the seismic region (low and hazard seismic region). The main aim is to secure the human lives, limit the damage and protect the important structure to operates continually when the earthquakes happened.

During earthquakes, the force is transformed as a vibration from the ground to the overlying of buildings and again back to the ground. The loads from the buildings and the ground become very complex. The direction, intensity, and variations in the strength of earthquakes very affected to the exposed buildings. As a result, it is difficult to find the rational method of calculating and dimensioning of the earthquakes. Therefore, Eurocode 8 gives as a reference that based on the reference peak ground acceleration on type A ground (rock), a_{gR} with additional parameters that requires for specific structures.

The reference peak acceleration corresponds to the reference return period T_{NCR} of the seismic action for the non-collapse requirement that chosen by the national authority of the seismic zone. In this case, the reference probability in 10% is 475 years in one return period, T_{NCR}. (EC 8, 3.2.1)

In this case, it needs to know the ground behavior and conditions. For identifying the ground conditions, we should have used a suitable investigation depending on the importance class of structures and particular condition of the projects. Based on the ground investigation and deeply geology study can be determined the seismic action. The ground classification that describes by stratigraphic profiles and parameters can be ground type A, B. C, D, and E, (see later on the table). (EC 8, 3.1.1).

Moreover, the vibration of the earthquake makes the building to oscillate. The strength of the oscillation depends on the difference between the ground vibration and the fundamental vibration period of the building. When the earthquake's vibration is subjected to the building, the force will occur on different floors of the buildings, depending on the ability building's on how to absorb energy or damping (EC8, 3.2.2.5 (3)p). The absorption of energy of the building is related to the behavior factor, q that describes the ductility. And different parameters could have used to calculate the seismic force F_b on the base level of the buildings (base shear force). Base shear force is the summation of the total horizontal force of each floor of the buildings (EC8, 4.3.3.2.3 (2)p).

3.2.2.2 Requirement for design

Many countries including Norway have established how to incorporate Eurocodes in their national standard. Eurocodes provided a large scope and update the design rule of structure (construction) in Europe. The most important standards of the Eurocodes are:

1. EN 1990 Eurocode 0: Basis of structural design
2. EN 1991 Eurocode 1: Action on structures
3. EN 1991 Eurocode 2: Design of concrete structures
4. EN 1991 Eurocode 3: Design of steel structures
5. EN 1991 Eurocode 4: Design of composite steel and concrete structures
6. EN 1991 Eurocode 5: Design of timber structures
7. EN 1991 Eurocode 6: Design of masonry structure
8. EN 1991 Eurocode 7: Geotechnical design
9. EN 1991 Eurocode 8: Design of structures for earthquake resistance
10. EN 1991 Eurocode 9: Design of aluminium structures

However, this thesis mainly focused on Eurocode 8 that specifies general rules, performance requirements, detail seismic action and hazard, and analytical procedure of the seismic design of steel frame buildings. There is the two-level ground motion that specified by this Eurocode:(EC8, 2.1, 2.2.2, and 2.2.3):

- The reference ground motion relates to the reference probability of exceedance in 10% in 50 years, or 475 years in the return period. Under this condition, there is no local or global collapse of the structures. This refers to the ultimate limit states.
- The reference ground motion with the probability of exceedance 10% in 10 years, or with a return period of 95 years, and the structure is able to have sufficient resistance and stiffness. This refers to the damage limit state. This requirement is not applied in Norway

3.2.2.3 Ductility according Eurocode 8

Ductility is the ability of material to deform permanently when it exposes to stress. During deformation of the material in the elastic region, energy has been observed or dissipated.

According the Eurocode, the steel buildings should have to design into two concept of energy dissipative behavior for resisting the earthquakes (EC8, 6.1.2). The concepts are:

Low-dissipative structural behavior (low ductility class) Low ductility (DCL) is defined as if the behavior factor q is less or equal to 1.5 when one construction has low ductility, it is easy to calculate the seismic force. The concrete or steel construction can be designed or dimensioned in low ductility class if it expressed by the following equations[10]:

$$a_g.S < 0.25g = 2.45m/s^2 \tag{3.51}$$

High-dissipative structural behavior (medium and high ductility class)

DCM / DCH are abbreviations for ductility class medium / high, and here the exclusion criteria do not apply. In medium ductility class the behavior factor q should be between 1.5 and 4. In

DCH, the value of q is above 4, and DCH cannot be used in Norway. In DCM, it is assumed that a plastic rearrangement occurs in load-bearing nodes and elements that affects results for having larger displacements compared to DCL.

Dimensioning of medium ductility have special requirement to design the supporting system for horizontal forces[10].

3.2.2.4 Ductility and behavior factor

From the EC8, the behavior factor is accounting for the ability of the structure to dissipate energy. On the other hand, how many energies absorbed and distributed in the constructions is accounted by q. The behavior factor reflects the ductility of different structures such as; types of structures, material type, and design concepts.

Design concepts

As we have seen the concept design in ductility section (low dissipative and dissipative structural behavior), the structures to resist earthquakes though elastic behavior (linear elastic response), the structures are designed to low ductility class. The upper limit of the reference value of q is between 1.5 and 2. This belongs to low seismicity and $a_g.S = 0.8a_{gR40Hz}.\gamma_1$ is not greater than 0.1g ($0.98m/s^2$) (EC8, 6.1.2, 4.4.1 and 3.2.1 (4)P). However, for constructions such as steel structure, this limit can be increased to 0.25g ($a_g.S < 0.25g = 2.45m/s^2$). And this could have verified by shear force at the foundation level due to earthquake must be less than the load of the other combination of actions (EC8,4.4.1).

Tabell 3.1: Behavior factor and ductility class

Ductility class	Reference value of behavior factor q	Required cross sectional class
DCM (some times called DCM+ or DCL+	$1.5 < q \leq 2$	1, 2 or 3
DCM	$2 < q \leq 4$	1 or 2
DCH	$q > 4$	1

The structures that resist earthquake actions through inelastic behavior (non-linear response), the value of q is greater than the upper limit (dissipative structural behavior). The structures are designed to the ductility classes DCM or DCH, where the value of q is between 1.5 and 4, in Norway not used higher than 4. We can see in the figure above, the behavior factor and ductility class.

Material type

The structures that designed in dissipative zone, should have considered the material properties (yield strength and toughness). The requirement of the yield strength of the steel design structure in dissipative zone have to satisfy the following conditions (EC8. 6.2 (3)P):

- The maximum yield strength of the steel of dissipative zone is; $f_{y,max} \leq 1.1\gamma_{ov}.f_y$
- The structure is designed for both dissipative and non-dissipative zones, the nominal yield strength of steel for non-dissipative zone and the maximum yield strength for dissipative zones.
- The actual yield strength, f_y,act of steel for every dissipative zone is determined from measuring and the actual over strength factor, γ_{ov} act is for computing of each dissipative zone.

Where, γ_{ov} is over strength factor in the design, and the f_y is the nominal yield strength specified for steel grade.

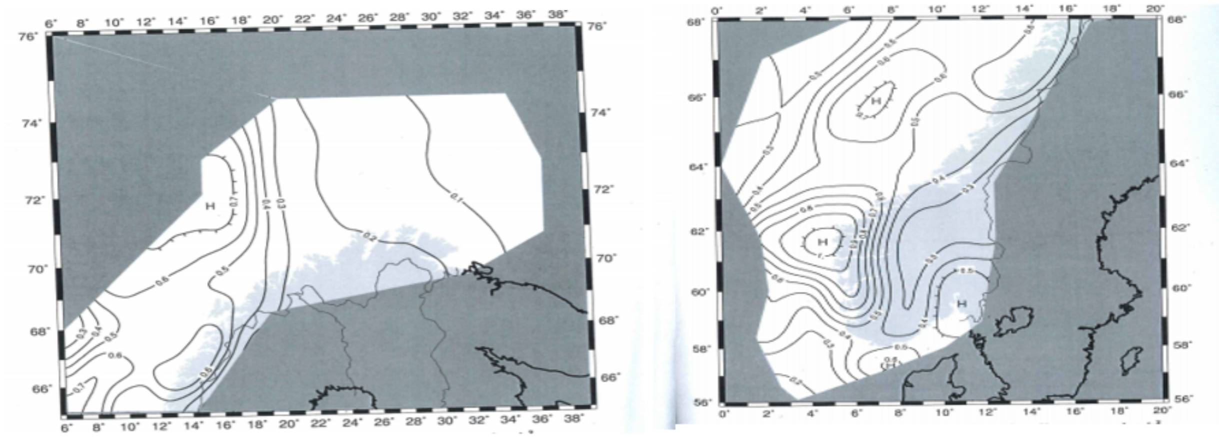
In addition, the toughness of the steel and welds should satisfy the requirement of the seismic action with regarding the lowest service temperature. And in bolted connection of the primary seismic member of building (seismic members other than beams and columns) have to use high strength bolt, 8.8 or 10.9 bolt grade.

Structural types

There are different structural types in steel buildings relating to their primary resisting structural behavior under seismic actions. The structural types are; moment resisting frames (mainly resist flexural force), frame with concentric bracing (mainly resists to axial force), frame with eccentric bracing (mainly resists to axial loads, and the energy dissipative by means of cyclic bending or shear in the seismic links), moment-resisting combined with concentric bracing and etc.

3.2.2.5 Ground acceleration

Norway is a country that is considered a low seismicity region and seismic design could be prioritized. In low seismicity, the designed ground acceleration multiplied by S is not greater than 0.5 g ($0.49m/s^2$), and the behavior factor q is not given a value higher than 1.5 and is not required to important class I.



Figur 3.42: Seismic zone for Northern and Southern Norway

The most important input in seismic calculation of the elastic response in Norway is the ground acceleration with a frequency 40Hz in the earthquake in return period of 475 years. The parameter a_{gR} is multiplied with the importance factor γ_1 to give the designed ground acceleration on type A ground. $a_g = a_{gR}.\gamma_1$ (EC8, 3.2.1). And the reference peak ground acceleration in type A will be $a_{gR} = 0.8a_{gR40Hz}$. Then the design ground acceleration for ground type A can be expressed below.

$$a_{gR} = 0.8a_{gR40Hz}.\gamma_1 \tag{3.52}$$

N.B: In the maximum areas, 0.05 m/s2 is added to the ISO-curve of value is used for a_{gR40Hz}.

Tabell 3.2: Values of importance factor with the importance class

Importance class	Buildings	Importance factor, γ_1
I	Buildings of minor importance for public safety e.g agricultural buildings	0.7
II	Ordinary, buildings not belonging in the other categories	1.0
III	Buildings whose seismic resistance is important in view of the consequences associated with a collapse e.g school, office buildings etc..	1.4
IV	Buildings whose integrity during earthquake is a vital importance for civil protection, e.g hospitals, fire station and power plants etc.	2.0

3.2.2.6 Importance class of buildings and importance factor

According the negative impacts for human life, on how to consider public safety and civil protection after earthquake happened and considered the consequence of collapse on social and economic, buildings are classified into 4 classes. These classes shaped by different importance factor, the value of γ_1 is being different for many seismic zones as shown in table 3.2 (table NA.4(901)).

In our master task, we have used importance class III and the value of q is 2.5.

3.2.2.7 Exclusion criteria

For understanding the characteristics of the building during an earthquake, it is important to find out which ground condition is best to design the seismic structure. The engineers could have to analyse whether the constructions are possible to design for the earthquake or not. Basis this analysis, realized to design the seismic performance of structures. The exclusion criteria is heavily depend on the weight of the buildings, the vibrational period, regularity and complexity of the structures. The following points of criteria can be defined to design the seismic structures:

Very low seismicity:

This requirement is not fulfilled according to NS-EN:1998 (EC 8, 3.2.1(5)p)

$$a_g.S < 0.05g = 0.45m/s^2 \tag{3.53}$$

Construction type:

The structures that classified under importance class I (like agricultural building, small house and fishing port) is not fulfilled with the verification of adequate safety according to NS-EN:1998 (Ec8, 3.2.1(5)p)

'Design spectrum Sd(T):

Here the verification of adequate safety is fulfilled if

$$S_d(T) < 0.05g = 0.45m/s^2 \tag{3.54}$$

In this design spectrum, the criteria are related with:

- Behavior factor $q \leq 1.5$
- No reduction of the elastic flexural and shear stiffness properties

- The structure rigidity fixed to the ground.

Sizes of forces:

If the shear force at foundation level due to earthquake is less than the forces of the other combination actions, the additional capacity control for the earthquake can be excluded [10]p.49.

$$F_b < (1.05 \times vind + 1.05 \times skjev) \times \frac{\gamma_{c,brudd}}{\gamma_{c,DCL}} \tag{3.55}$$

And the requirements that are found in the points of EC8, 2.2.4.1(1 and 4)p, 2.2.4.2 and 2.2.4.3) must be fulfilled.

.

4 Research question

In this report we are comparing design of two steel frame buildings. Their preliminary design of the buildings was performed as part of an earlier bachelor thesis in HIOF[22].

According to the anecdotal evidence presented in this bachelor thesis, it is a general believe among structural engineers and building designers in Norway that moment resistant frames are less economical than traditional cross-bracings as use of the former leads to excessive material consumption. The bachelor thesis compared these two types of stabilising systems and found the moment resistant frame to be more economical than cross-bracings.

However, the earthquake design was outside the scope of this bachelor thesis and the building design in the bachelor thesis was incomplete and was not subjected to any external control. Thus, the calculations in bachelor thesis might be erroneous.

Thus our research question is:

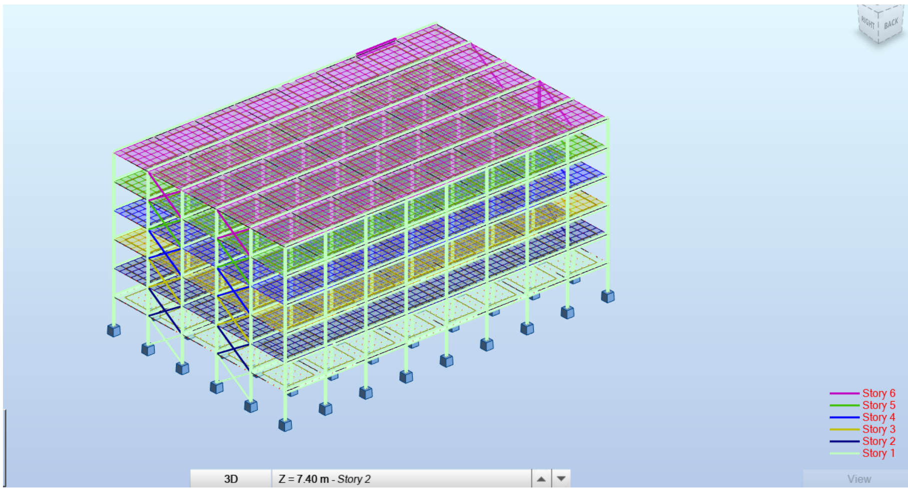
Do the moment resistant frames still show to be more economical than traditional cross-bracings when the earthquake loads are taken into account?

The sub questions are:

1. Do the bachelor project results hold against external scrutiny?
2. How does the building stabilizing system hold against earthquake loads?

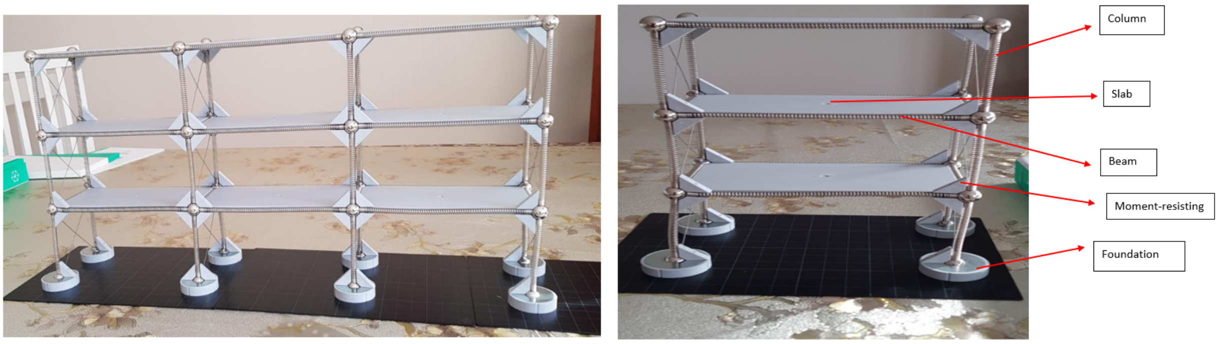
5 | Case-study

The case used in this master thesis is taken from an earlier bachelor thesis [22] . This is a six story building, where the final story is a roof terrace. The proposed building is designed in two options: moment resistance and cross bracing steel frame with hollow-core slabs. The location is chosen to be Oslo, Norway in order to achieve highest possible ground acceleration with regards to earthquake analysis. A model of the structure is made in Robot Structural Analysis. The structure is 30 × 48 with bay widths of 6m in both X and Y direction. There are eight span in the X and five span in the Y directions.The story height is 3.7m, as it mentioned earlier the final floor functions as a roof terrace for entertainment and other activities, so it surrounded by walls height of 2.5m. The total height of the building including height of surrounding walls is 24.7m.

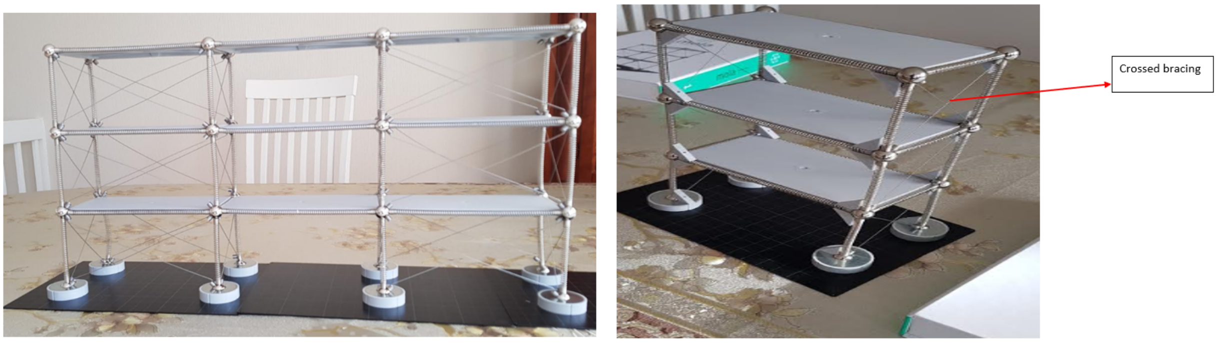


Figur 5.1: Model of the building in Robot

The Mola educational model (figures 5.2 and 5.3) is used to gain a better understanding of our design, moment resisting and cross bracing buildings in practice. The buildings are constructed with three spans in the X direction and one span in the Y direction, while the actual buildings are comprised of eight spans in X and five spans in the Y direction. The Mola model provides facilities for simple illustration of the structural elements such as joints, connections and bracing system in different combinations. We found out that the usage of Mola model helps to understand and comprehend better the technical solution and design of the real structures.



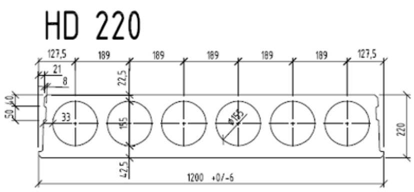
Figur 5.2: Model of the moment resistance building



Figur 5.3: Model of the cross bracing building

The structure is made up of steel elements comprised of beams, columns, and bracing. The columns for the moment resisting building are HEB 240 and the beams are IPE 550. The columns for the cross bracing building are the same as the moment resisting frame, HEB 240, while the beams are IPE 600. The bracing for both moment resisting and cross bracing buildings is HUP $120 \times 120 \times 6.3$. The length and width of the building was originally 30×49, but after reviewing the hollow core slabs the length was reduced to 48m to fit the width of the hollow core slabs. The hollow core slab used in this building is of the type HD220 REI120. Product details of HD 220 are taken from a typical producer in Norway [23].

Tverrsnitt	Egenvekt		Fugemørtel
	(kg/m²)	(kg/lm)	(kg/m²)
HD200	255	306	16
HD220	304	365	17
HD265	370	444	21
HD285	420	503	22
HD320	400	480	25
HD340	450	540	26
HD400	504	605	31
HD420	553	663	33
HD500	614	737	39
HD520	663	797	41



Figur 5.4: Hollow-core slab and its cross- section

5.1 Earthquake analysis

The type of earthquake analysis we have chosen to use in this project is the Modal Seismic Analysis. This requires first a modal analysis which utilises swing modes and then a seismic analysis. More about swing modes and Eurocode 8 is found in the Theory section of the pre-project. The

Tabell 5.1: Input data for earthquake design

Factor	Value unit	Description
g	$1.0 m/s^2$	Ground acceleration
a_g	$1.176 m/s^2$	Design ground acceleration
γ_1	1.4	importance class III
Ground type	D	Loose to medium cohesion-less soil
S	1.55	Factor for soil
$T_B(s)$	0.15	Classifies areas of change in the design spectrum
$T_C(s)$	0.40	Classifies area of change in the design spectrum
$T_D(s)$	1.60	Classifies area of change in the design spectrum
β	0.20	Lower limit of the design spectrum

acceleration used for earthquake analysis in the area of Oslo, Norway is $1.0 m/s^2$. Below is a table of the necessary factors used in Modal Seismic analysis. The factors are according to Eurocode 8(EN 1998-1:2004) The load combinations utilized for earthquakes are according to Eurocode 0(NS-EN 1990:2002). Here is also the equation for design ground acceleration:

Design ground acceleration is:

$a_g = 0.8 \times a_{gR40Hz} \times \gamma_1 = 0.8 \times 1.05 \times 1.4 = 1.176 m/s^2$

5.2 Wind calculation

The wind load is derived from Eurocode1 EN 1991-1-4. The fundamental basic wind velocity for Oslo, Norway was Vb,0= 22m/s. The complete calculation for wind loads is available in appendix 1.

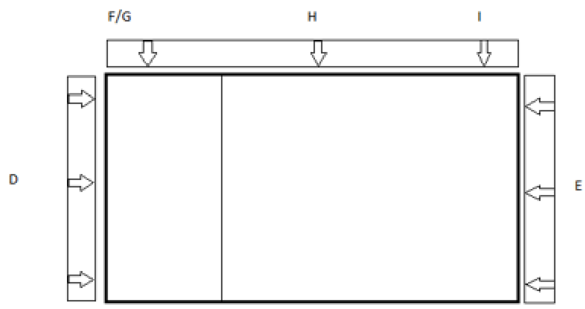
$W = (c_{pe} + c_{pi}).q_p(z)$

$q_p(z) = (1 + 7 \times 0.23) \times \frac{1}{1.25} \times 20.9^2 = 712.55 N/m2 = 0.712 kN/m2$

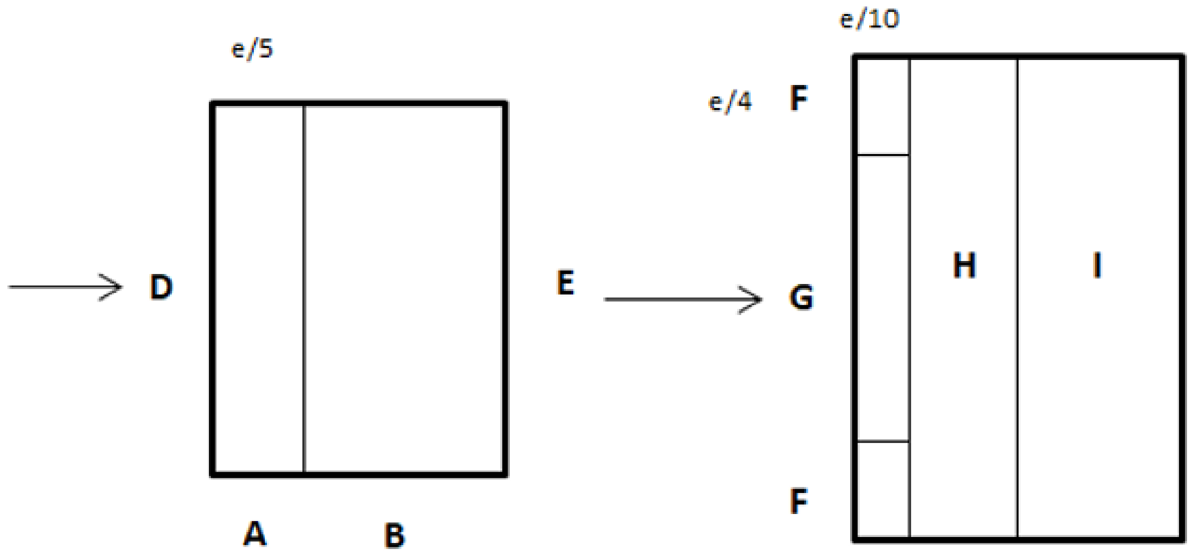
z- in this case will be the height 22.5m.

The loads will be in kN/m

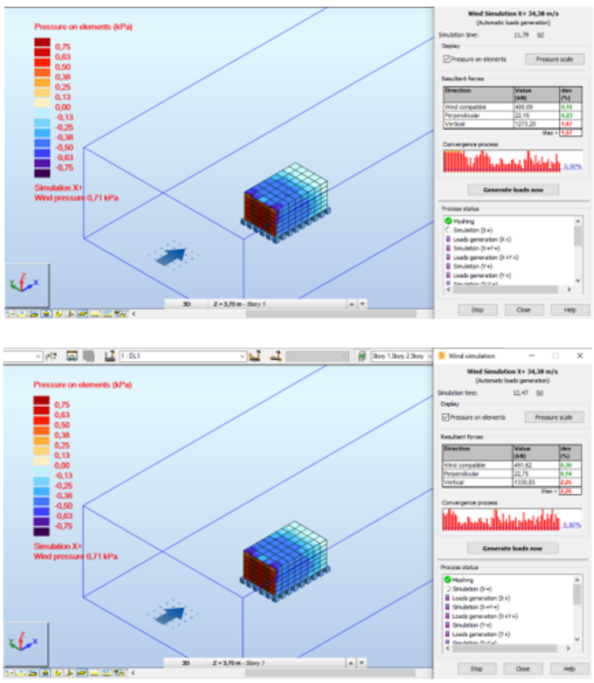
A : 24.03 B : 17.62 D : 12.82 E : 11.69 G : 17.62 F : 24.03 H : 16.02 I : 8.01



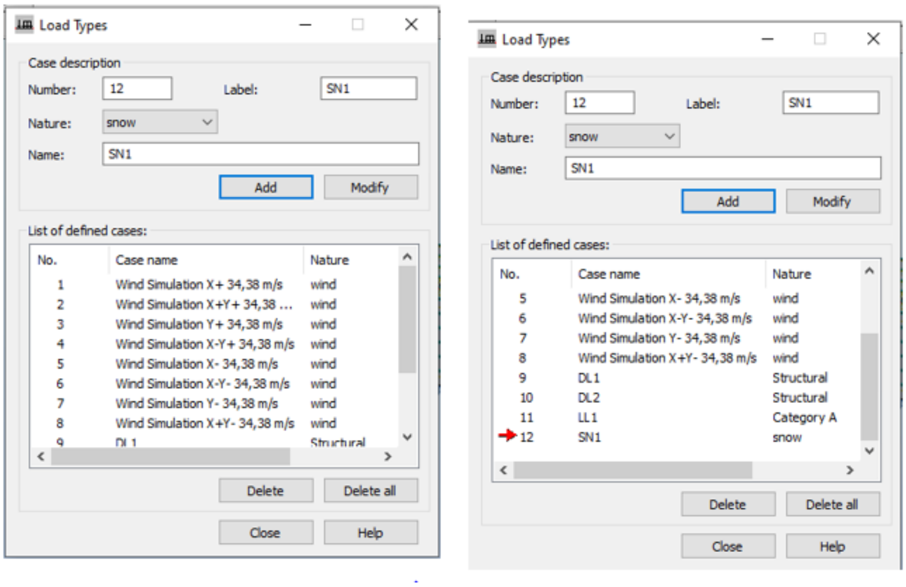
Figur 5.5: Wind forces acting on the facade of the building



Figur 5.6: Illustration of vertical walls and the roof of the buildings



Figur 5.7: Wind simulation analysis taken from RSA



Figur 5.8: Wind load acting on the building from different directions

5.3 Loads

All the loads are derived and calculated using Eurocode 1. Eurocode 0 also gives a load to mass conversion when considering seismic analysis. The load combinations can be set both manually and automatically in Robot Structural Analysis (Robot). We have chosen to use the automatic load combinations in Robot. This is because the combinations Robot generates is much more than we could manually calculate and this gives sufficient results. Robot uses both the codes EN 1991(Action on structures) and EN 1990(Basis of structure design), when generating combinations. The method for the calculation of the loads can be found in the appendix 1.

The loads used on the model are shown in table 5.2.

Tabell 5.2: Loads on model

Loads	kN/m^2	Load to mass conversion	Comments
Dead Load (DL1)	self weight	1	Automatic in Robot
Dead Load (DL2)	2.5	1.0	10 cm concrete floor
Snow load	2.88	0.2	Flat roof
Live Load	3.0	0.3	Office space
Wind Load	1.07	0	In different direction

6 | Methods

6.1 Procedure/Course of action

Literature study is an important part of gaining the necessary information for the thesis. The study of dynamics, steel structures, load combinations, Eurocodes (0, 1, 3, and 8) are performed to gain an understanding of the topics covered in this thesis. This literature study is done based on the study of different books, articles, and journal reports based on sources like Science Direct, Google Scholar, and Oria. The credibility of the source is evaluated accordingly for each source. Things like timeliness and authority is evaluated for each source. Is the author an expert on what he is writing? When was the article written? Has it been peer-reviewed? These are examples of questions we ask when evaluating a source. It was also important to learn Robot Structural Analysis software in detail, in order to learn about the software and methods of analysis. This was achieved by watching different videos on YouTube and gathering information from the Autodesk platform.

The methods for completing the thesis from start to finish is described in this section. First we had a literary review of the dynamics of structures and steel structures. After some preliminary research we decided to focus on certain bracing systems applied to steel structures with a focus on horizontal loads. The purpose was to learn more about these types of structures and to design these types of structures. Our case became two steel structures from an earlier bachelor thesis. The next step was to further define a problem or question we wanted to answer. Our research question was then formed from our research and case. In order to answer our research question, more research and analysis was necessary. We had to design and analyse our building in some type of analysis software. We had already chosen Robot Structural Analysis since we were familiar with this program. The study of the connections type and details of the hollow slab for our case were imperative in this phase. We also needed to calculate the loads acting on the structure. This prompted research for wind and snow loads, as well as load combinations. The earthquake forces were also an important part of the design, as modal seismic analysis was chosen as the analysis method for earthquake. Much of the research on earthquake design came from our pre-project. Steel structures and connection design was emphasized in the theory as this was very important in order to answer our research question. The next phase was to start analysis and compilation of the results. The analysis was performed several times to ensure some reliability to the result. Two different group members performed the analysis separately; this was also to give reliability to the result. The results were then recorded and analysed accordingly. Our analysis prompted further analysis and a redesign of the structures, this redesign and analysis followed the same procedures as the first results. Finally, the conclusion and discussion are based on our results and findings. The discussion should also show eventual flaws or give some critical views to eventually our process or research. The final step is to go over the project and look for flaws or eventual mistakes and either correct these or, in some cases, admit the uncertainties in the discussion section.

6.2 Seismic analysis in relevance to Eurocode 8

The objective of this method is to perform the seismic analysis, based on the integration of Eurocode 8. The parameters which are taken into account in the Eurocode 8 is used to design and calculate the seismic force. In order to analyse the dimension of earthquake in low ductility class and medium ductility class (see. Eurocode 8), it is necessary to know the following key points:

1. Determine the importance class and importance factor, $\gamma 1$

   The four importance class is defined in the table EN:NA 4(902), and determine the buildings which importance class have based on risk and consequence of collapse. In addition, importance class determines which importance factor will be used for controlling the seismic design.

2. Determine the ground types

   The ground type should be identified before the design phase of the construction. The different ground types, A-E can see in the table EN:NA 3.1.

3. Determine the soil factor, S

   The soil factor can be identified based on the ground type. The soil factor can control the exclusion criteria of very low seismicity.

4. Determine the ground acceleration at the frequency of 40Hz, a_{g40Hz}

   The peak reference ground acceleration at frequency of 40Hz is determined based on the cart of seismic zone in the national annex.

5. Define the value of behaviour factor, q

   The behaviour factor is defined based on the ductility class. This behaviour factor, q is accounting for the ability of structures to dissipate energy.

6. Define the design ground acceleration, a_g

   The design ground acceleration is defined based on the importance factor and peak reference ground acceleration (a_{g40Hz}).

7. Define the breaking parameter of the vibrational period, T_B, T_C, T_D

   The different value of breaking parameters is defined in table NA: 3.3. These value of T_B,T_C,T_D are parameters that defined breaking point in the elastic response spectrum S_d(T) and related with ground type.

8. Define the horizontal shear force, F_b

   The base shear force, F_b can be defined with the help of design response spectrum that multiplies with mass of stories and correction factor.

9. Calculate the design response spectrum, S_d(T)

   The design response spectrum can be calculated by using Equations 3.13-3.16 in Eurocode 8.

10. Define the fundamental vibration period T, and the coefficient of the building's rigidity C_t

    The first fundamental vibration period T_1 can be calculated by using equation 4.6 in Eurocode 8, and the coefficient of the building's rigidity can be defined based on which type of construction (steel, concrete).

11. Calculate the seismic loads in the building

    The seismic force of buildings could be calculated or designed by using either lateral force method or seismic modal analysis if the exclusion criteria are not fulfilled.

12. Design of buildings to resist earthquakes.

    The buildings can be design against earthquake, the seismic load combines with the other load combination in relation of Eurocode 8.

6.3 Modal analysis

The main aim of this analysis is to determine the structure‘s swing modes and frequencies under free vibration. Modal response is applied to the buildings which do not satisfy by the lateral analysis method. The requirement which is not fulfilled by the following condition:

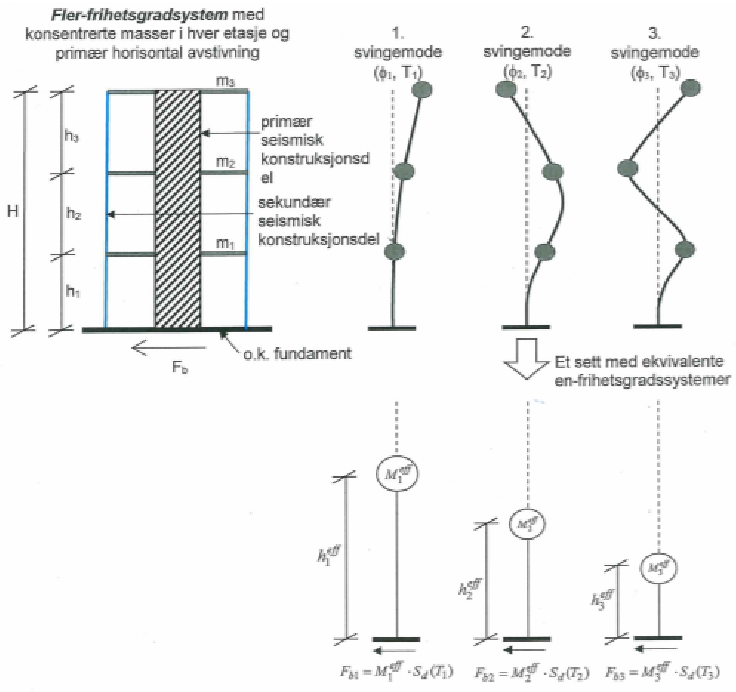
- The building does not meet the criteria for regularity in elevation
- Building‘s first fundamental vibration period is larger than 4Tc or 2 sec.

Modal analysis method is often based on a linear dynamic analysis and used as a field of application for earthquake design of Norwegian buildings. This method is also characterized by the following condition [10]:

1. Calculate the maximum force and deformation
2. Construction’s ability to absorb energy (q) direct in the design response spectrum (S_d) is taking into account, such analysis become linear elastic
3. Assume linear response is applicable for the most buildings
4. The reference method of NS-EN 1998-1 always based on the software tools.

6.3.1 Modal response analysis

Modal response spectrum analysis is a method to estimate the structural response to a transient dynamic event. It means that this analysis is based on natural mode (is a function of the natural frequency of motion and its damping) of single degree of freedom system. The shape of natural vibration mode is depending on the stiffness ratio and the first fundamental vibration period. The fundamental vibration period has a significant influence on the response of the buildings. If the building modelled as multi-degree of freedom system, the mass in the system is either distributed or concentrated at different level in the construction. Meanwhile, the primary structure gives stiffness to the construction. Each of the natural vibration modes (swing modes) in the nth mode, the mass and stiffness defined as M_n and K_n of the equivalent to the single of freedom system with equal fundamental vibration period as a real system [20] [10].



Figur 6.1: Modal response[10]

Each swing mode has its own deformation and fundamental vibration period. The combination of results from all swing modes determines the building's total force and deformation. According to Eurocode 8, the modal response analysis is applied to the buildings if the requirement for regularity in elevation and fundamental vibration period is not satisfied with lateral force analysis. The requirement of the response of all vibration modes that consider the modal analysis will be satisfied if the following includes:

- The total effective modal mass included in the system will be at least 90% of the total mass of the structures.
$$\sum M_n^{eff} \geq 0.9m_{tot} \tag{6.1}$$
- All modes with effective modal mass will be greater than 5% of the total mass of the structure.

However, if the requirement of the response of all vibration mode is not satisfied with the modal analysis (for example, in building with significant contribution from torsional modes), it requires minimum number of modes (k) that includes in the Eurocode. The value of k could satisfy both the two following conditions:

$$k \geq 3\sqrt{n} \tag{6.2}$$

$$T_k \leq 0.2s \tag{6.3}$$

Where k is the number of modes taken into account, n is the number of stories above the foundation or the top of a rigid basement, and T_k is the period vibration of mode k.

6.3.2 Combination of modal response

Based on the modal combination rule ([20] p.563), we have different methods (such as SRSS and CQC) used to combine different responses. The modal combination rules give excellent response estimates for structures and well separated natural frequencies. According to the Eurocode (4.3.3.3.2), the two vibration modes such can be combined independently if their fundamental periods (T_i and T_j) satisfies the following conditions:

$$T_j \leq 0.9T_i \tag{6.4}$$

If the relevant modal response which is independently each other, we will use SRSS combination rule. And the peak seismic action effect can be written as follows:

$$E_E = \sqrt{\sum E_{Ei}^2} \tag{6.5}$$

Where, E_E is seismic action effect (force, displacement) and E_{Ei} is the value of seismic action effect due to mode i.

If the response in two vibration modes of i and j is not satisfied with the SRSS rule of combination, we have able to use an accurate procedure for combination of modal response, the CQC rule. This is most relevant with robot analysis.

$$E_E = \sqrt{\sum_{j=1}^{k} \sum_{i=1}^{k} \rho_{ij} E_{Ej} E_{Ei}} \tag{6.6}$$

Where ρ_{ij} is the correlation coefficient for modes of i and j.

6.4 Software

Today, we can find a large number of software used for calculation of static and dynamic forces. But there is no fixed solution that which software is the ideal one in order to give proper and more accurate result. Many software producers deliver their product to the market not having the same qualities or functions as they offer them. Just the user has responsibilities for selecting the right and suitable software in order to calculate the project more accurate. The result should be checked for the purpose of to avoid mistakes.

Some of these software packages are very advanced and complicated while others are simple to use them. There are number of software packages which are available in the market today such as:

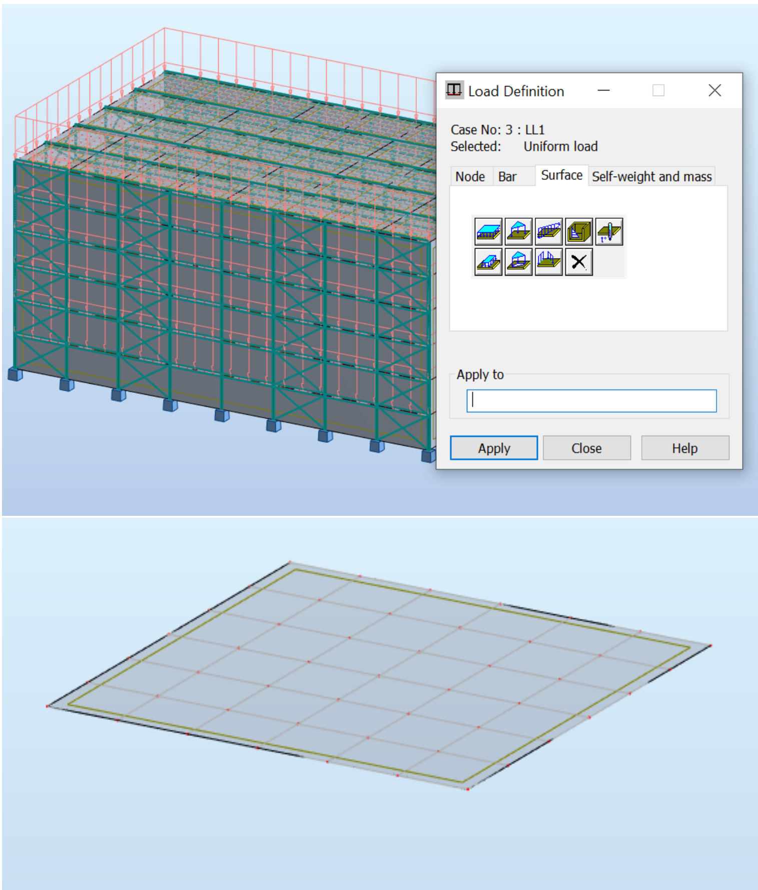
- Abaqus
- Sap 2000
- Ansys
- ETAB
- Mark
- Robot structural analysis (RSA)

We have used the Robot software in our thesis, because it is an advanced tool for analysis of dynamic forces such as earthquakes. Robot provides facilities carrying out standardized seismic analysis such as Lateral force method, Response spectrum, Time history, Push over. As well as it has facilities for dimensioning based on Eurocodes.

6.4.1 Modeling in Robot structural analysis

Robot structural analysis (RSA) is an advanced Autodesk program that used to analyse dynamic and seismic loads. This modelling-program often based on the element method. Robot programs have an ability to calculate a large and small projection in relevance of one construction. Robot software can have a little difference in the calculation tools based on the production year. Our building is modelled in RSA 2019, and done by two group members in order to get a better result. It is very important that the building must be modelled correct as possible, and designed in relevance to Eurocode 8. It means that the design of the building should be fulfilled the general requirements of this Eurocode.

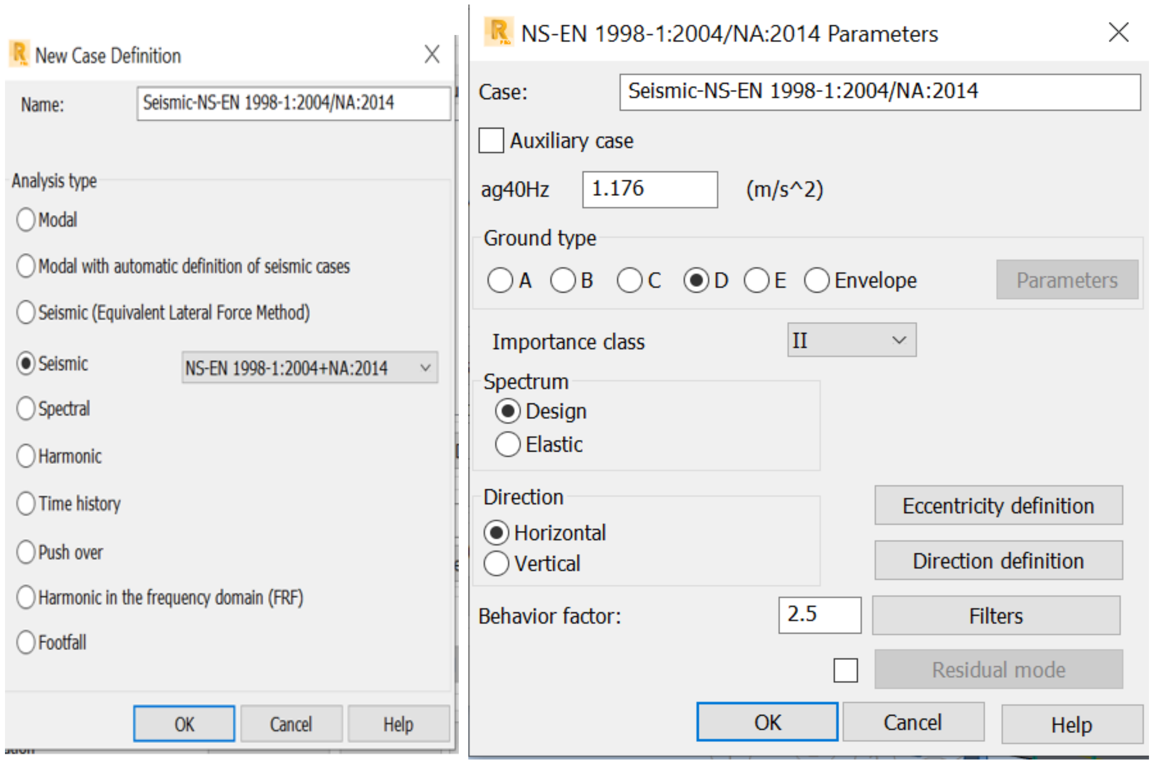
The design structural element such as beam, column and slabs can be modelled by the help of different program tools. After modelling the structural element, it should be define all the load types as shown in the figure 6.2. All variable and permanent loads can be added to every floor of the structure in accordance with Norwegian standard. When the model is completed , the meshing function should be used. Meshing divides the structure into small elements that can be done manually or automatically [24]. Here in this thesis, meshing is done automatically only in the slabs of the building.



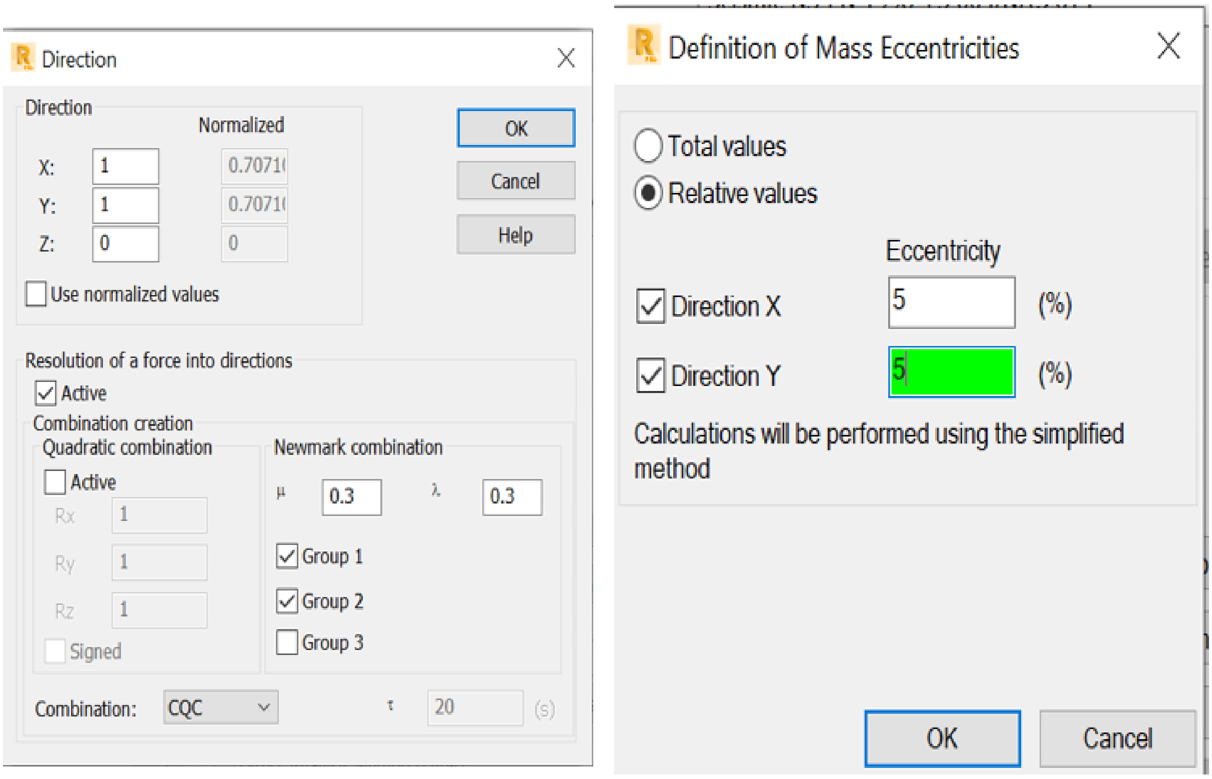
Figur 6.2: Load definition and meshing of slab

After that, we run a modal analysis to get the dynamic response and swing modes of the building. It is possible to enter seismic load, if the period of vibrations and swing modes reached at least 90% of the effective modal mass in X and Y-direction. All the calculations have been done in Robot are based on the input from Eurocode 8. Finally, member of verification is done in order to verify the structural elements of the building.

Figure 6.3 and 6.4 shows the detail information on how to sett the input data and calculate in Robot program.



Figur 6.3: Analysis type and seismic parameters



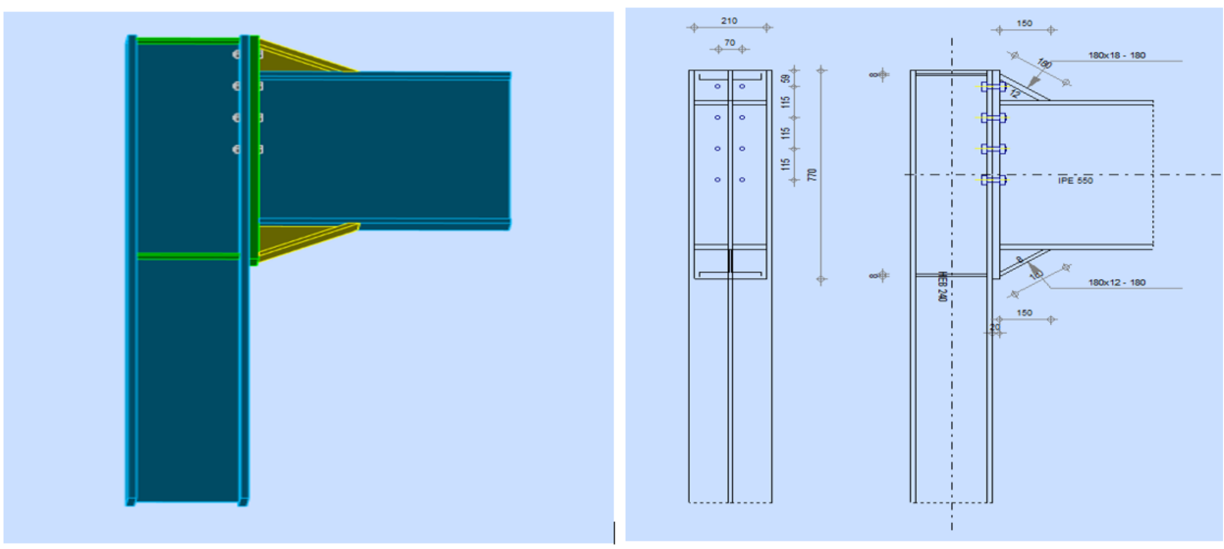
Figur 6.4: Definition of direction and mass eccentricities

7 Results

Here the analysis and results of the two structures will be presented in detail. The structures were compared to find out the pros and cons of each structure. Each structure has been analyzed separately in Robot from two different computers, in order to ensure better results, as also mentioned in Method. The analysis below showed that the structures were not highly efficient and, therefore, the decision was made to redesign both buildings. The material usage and analysis will draw a conclusion regarding a moment resisting frame versus cross-bracing frame.

7.1 Moment resisting frame

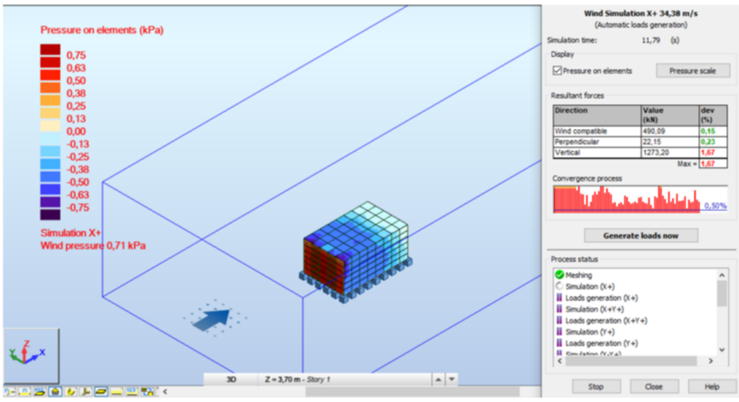
This moment resisting frame has fixed connection both for the beams and columns, but the bracing on the short sides are pinned.



Figur 7.1: Connection design for moment-resisting frame from robot model

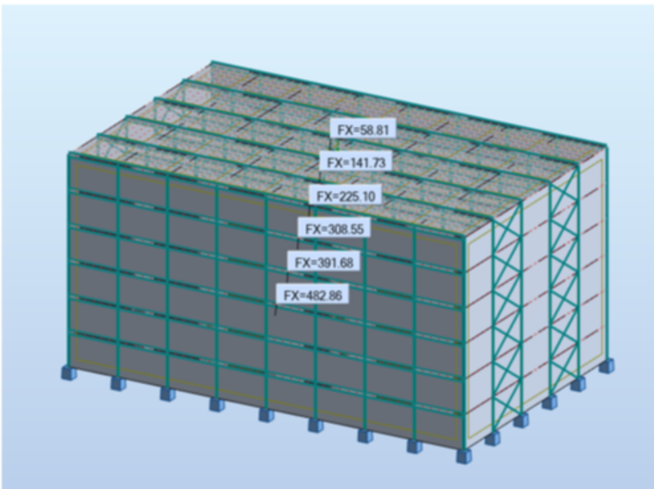
7.1.1 Wind

The wind forces are calculated as shown in Case and Attachments. After the calculations, the peak wind pressure was used in the simulation in Robot. The simulation created forces for 8 different directions. The figure below shows the pressure on the structure in the X+ direction. The red and orange-like colors shows the pressure while the blue shows where there is suction or negative forces.

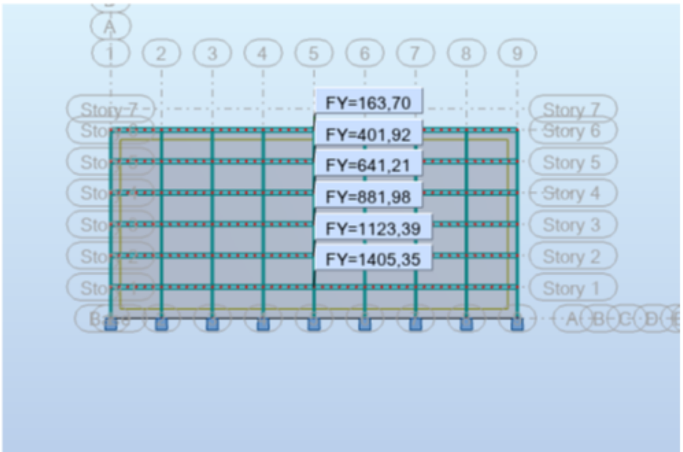


Figur 7.2: Wind forces in X+ direction

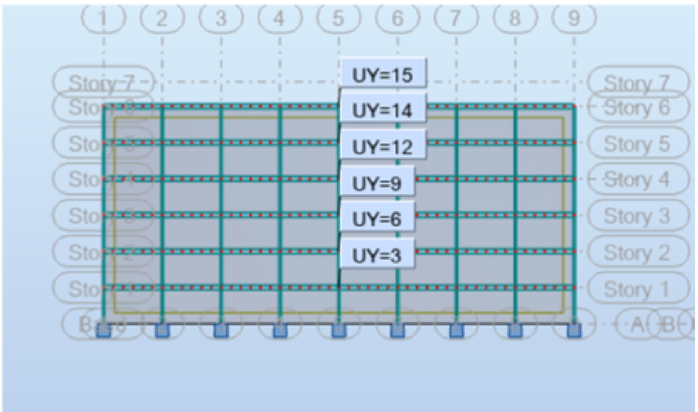
The wind forces create some pressure on the building. Base shear force and deformations was looked at when considering only wind forces, as shown in the figures 7.3-7.5. The base shear force is bigger in the y direction, which comes from wind pressure on the long side of the building, then the x direction. The deformations due to wind are relatively low when compared to earthquake analysis. Although, the shear force in Y-direction are much higher than the shear forces in X-direction as shown in the figures below. This is because the surface area is much higher in the X-direction. The forces in the Y direction enact on the X-direction surface.



Figur 7.3: Base shear force for wind forces (X-direction)



Figur 7.4: Base shear force for wind forces (Y-direction)



Figur 7.5: Deformation in Y-direction from wind forces

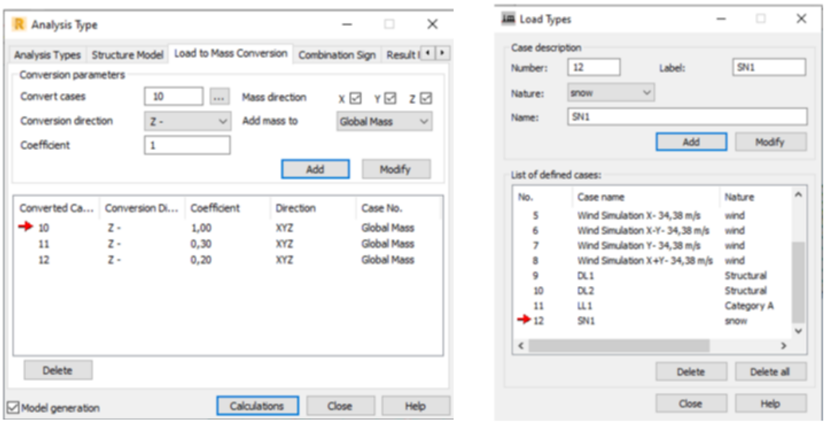
7.1.2 Earthquake analysis

The earthquake analysis was done according to the Modal Seismic analysis method, as this was shown to give a more accurate result then the lateral force method. This is shown in the Case section. First, the modal analysis was done and 18 swing modes were chosen. This was to ensure 90% mass participation and too follow recommendations of three modes per floor (Ref. Eurocode8) . The following figure shows details from the swing modes. Notice the period becomes lower as the frequency increases in the latter swing modes. This is good as the structure becomes more acceleration sensitive in the spectral regions. The ductility demand also increases in this region (See Dynamics of structures in our pre-project).

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)
10/ 1	0.60	1.66	85.80	0.00	0.00	85.80	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 2	0.69	1.45	85.80	79.64	0.00	0.00	79.64	0.00	5955181.39	5955181.39	5955181.39
10/ 3	1.01	0.99	85.80	79.64	0.00	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 4	1.79	0.56	95.15	79.64	0.00	9.35	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 5	2.18	0.46	95.15	94.61	0.00	0.00	14.97	0.00	5955181.39	5955181.39	5955181.39
10/ 6	2.94	0.34	98.16	94.61	0.00	3.02	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 7	3.21	0.31	98.16	94.61	0.00	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 8	3.77	0.27	98.16	98.12	0.00	0.00	3.50	0.00	5955181.39	5955181.39	5955181.39
10/ 9	3.98	0.25	99.40	98.12	0.00	1.24	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 10	4.82	0.21	99.88	98.12	0.00	0.48	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 11	4.91	0.20	99.88	99.39	0.00	0.00	1.28	0.00	5955181.39	5955181.39	5955181.39
10/ 12	5.38	0.19	100.00	99.39	0.00	0.11	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 13	5.66	0.18	100.00	99.39	0.00	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 14	5.71	0.18	100.00	99.85	0.00	0.00	0.45	0.00	5955181.39	5955181.39	5955181.39
10/ 15	5.84	0.17	100.00	99.85	60.69	0.00	0.00	60.69	5955181.39	5955181.39	5955181.39
10/ 16	5.94	0.17	100.00	99.85	60.69	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 17	6.00	0.17	100.00	99.85	60.69	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39
10/ 18	6.14	0.16	100.00	99.85	60.69	0.00	0.00	0.00	5955181.39	5955181.39	5955181.39

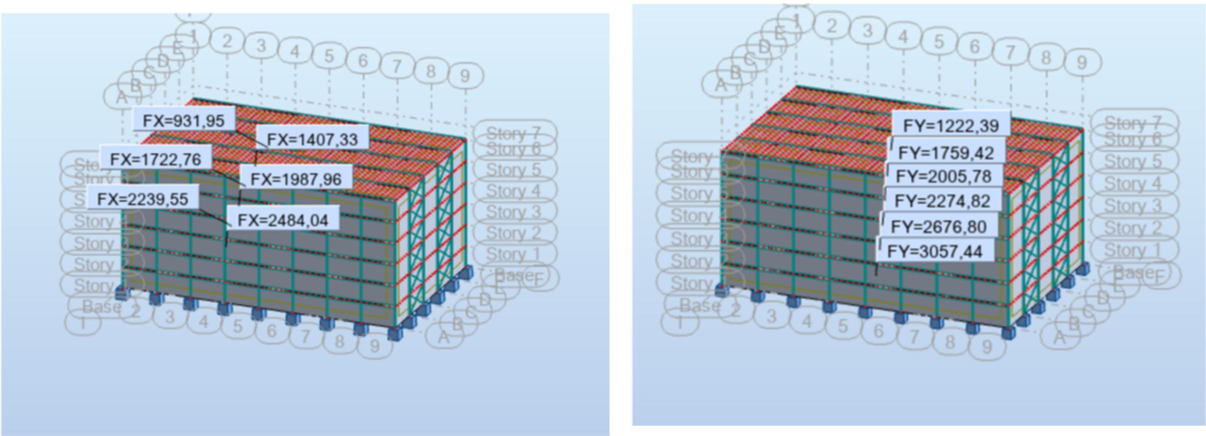
Figur 7.6: Swing modes for moment resisting structure

The load to mass conversion was utilized as according to Eurocode 8. All of dead load 2 (DL2) was converted to mass for the earthquake analysis, while 20‘% of the snow load (SN1) and 30% of the live load (LL1) were converted to mass. The figure below show the load to mass conversion and the some of the load types.

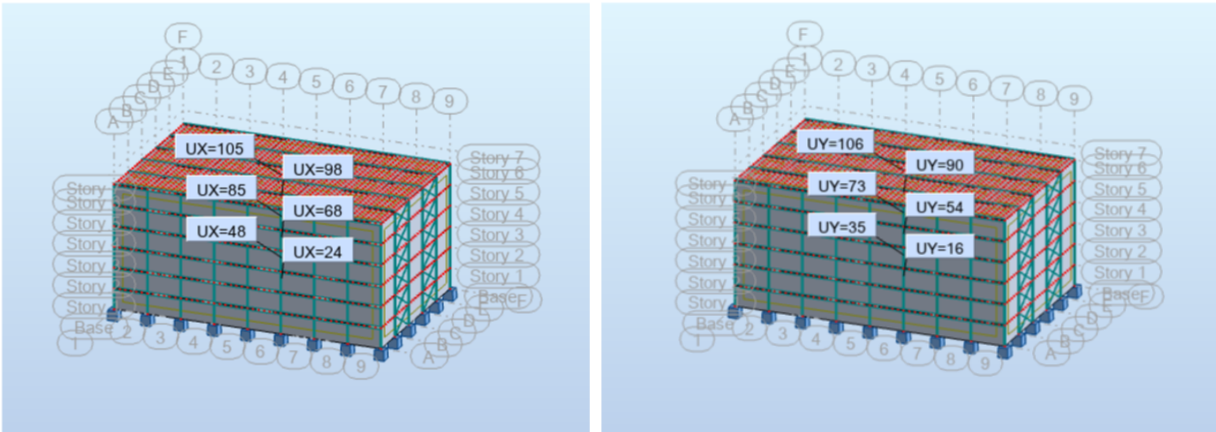


Figur 7.7: Load to mass conversion and main load type acting on the building

The base shear forces acting on the structure are shown in the figures below, as well as the deformations. The results show slightly higher base shear in the y-direction with a base shear of 3057 kN. The deformations are however quite similar, which shows the effect of the cross bracing on the sides of the building. The stiffness of the structure is increased, as this will be shown in the cross-bracing structure later in results. The deformations in mm are shown according to the center of the floor, similar to the base shear.



Figur 7.8: Base shear force in X-direction and Y-direction



Figur 7.9: Deformation in X-direction and Y-direction

The story drift of the building amounted to 24mm in the X-direction as the highest story drift. The inter-story drift is checked according to EN 1998-1:2004 (equation 4.32). This equation was chosen because there are no walls or non-structural elements in the structure at the present moment.

$$d_r.v \leq 0.0075h \tag{7.1}$$

where

$$d_r = Drift_e.q \tag{7.2}$$

Also q is the behaviour factor, V is reduction factor with a value of 0.4 for importance class III, and h is the height of the floor.

The equation 7.1 gives:

$24mm \times 2.5 \times 0,4 \leq 0,0075 \times 3700$

$24 \leq 27.75$

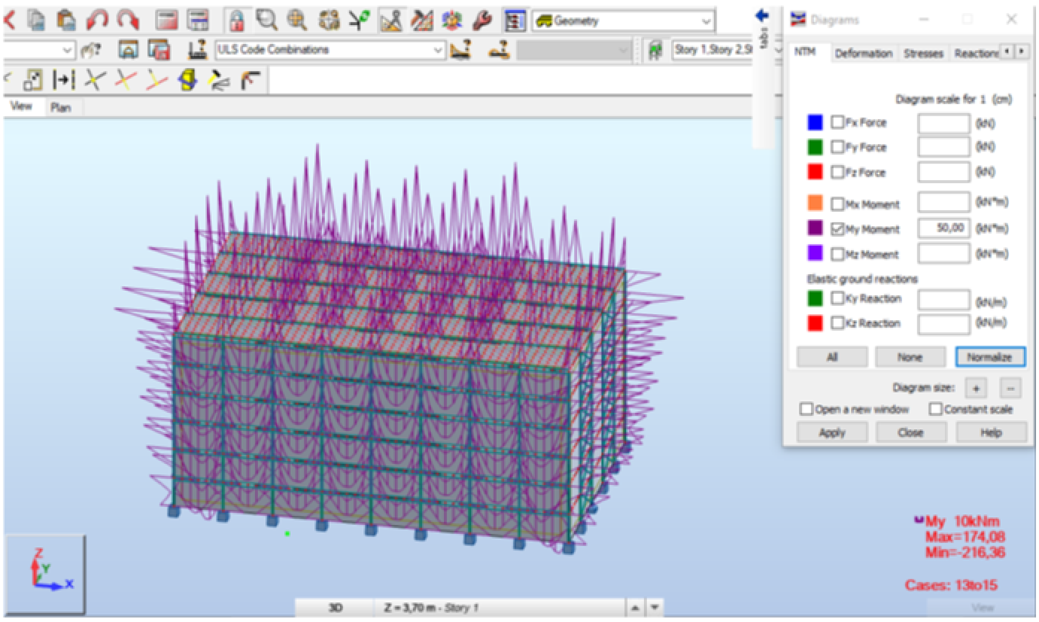
The story drift is approved according to Eurocode 8. The stiffness of the structure as a whole is good enough pertaining to ground shaking. This shows that the story drift was close to the limit and will be compared to the other structure in order to draw some conclusions pertaining to the behaviour of each structure.

7.1.3 Forces (DL1, DL2, LL1, SN1 and Wind)

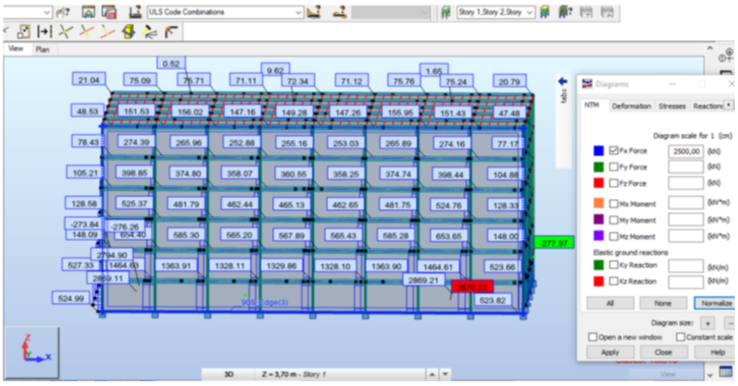
Here are some figures showing the effect of forces, other than earthquake, on the structure. All the forces are shown as combinations in the ultimate limit state (ULS). The figure 7.10 shows the maximum forces on the structure. Moment (My) on the beams is shown as well as the forces on the columns. The maximum moment on the beams are 118.91 kNm and the maximum on the columns is 2870.22kN.

	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
MAX	2870,22	32,21	207,46	0,55	118,91	63,86
Bar	47	319	596	573	322	272
Node	99	327	373	376	384	334
Case	ULS/73	ULS/3	ULS/73	ULS/22	ULS/79	ULS/1
MIN	-418,25	-32,06	-207,46	-0,55	-216,36	-63,86
Bar	51	314	583	613	596	319
Node	109	322	348	381	373	381
Case	7	ULS/3	ULS/73	ULS/22	ULS/73	ULS/1

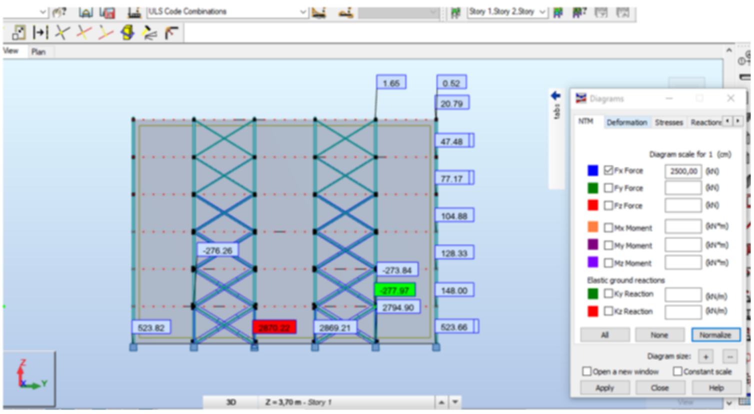
Figur 7.10: Maximum and minimum forces



Figur 7.11: Moment on the beams



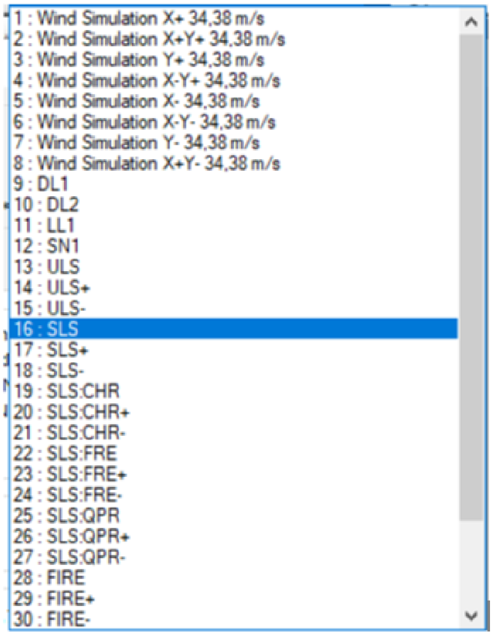
Figur 7.12: Forces on the column



Figur 7.13: Forces in the bracing

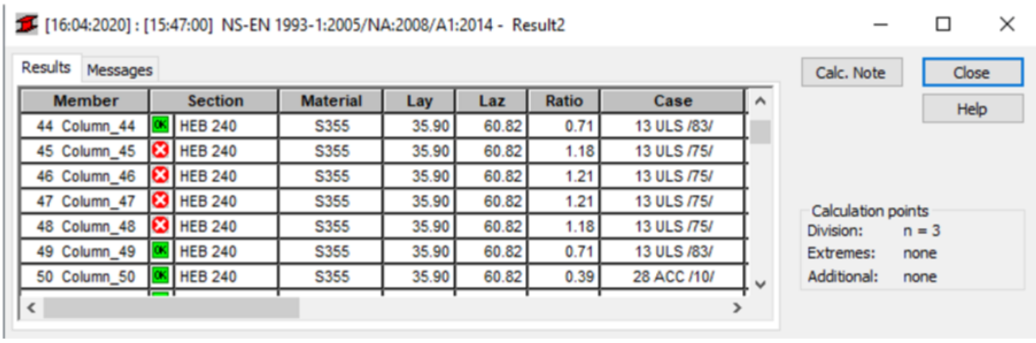
7.1.4 Member verification

The member verification shows how the elements of the structure react to the loads. The member verification is according to Eurocode 3. All beams and bracing were approved, while 32 HEB 240 columns were not approved. The figure below shows the loads and the code combinations that can be utilized in Robot. Full automatic combinations were chosen.



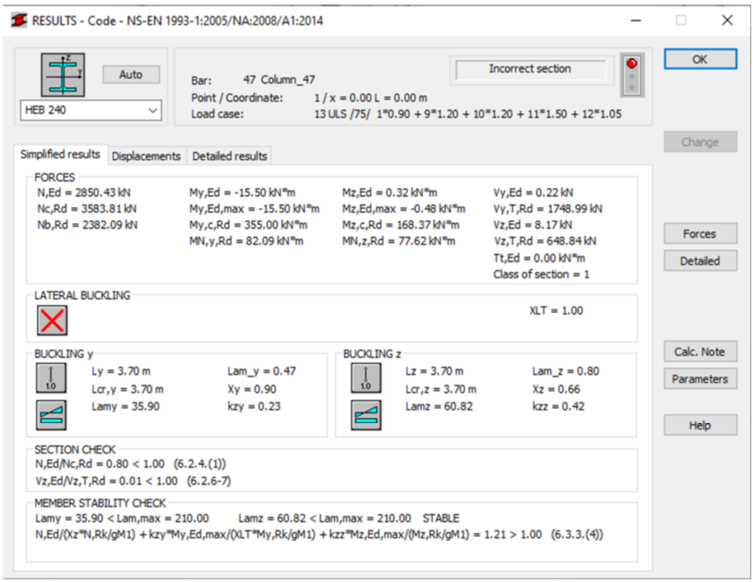
Figur 7.14: Loads and combinations

The columns had 32 members that were not approved with the highest ratio being 1.21. The members were not approved in the members stability check. The results are very informative and well shown in Robot.



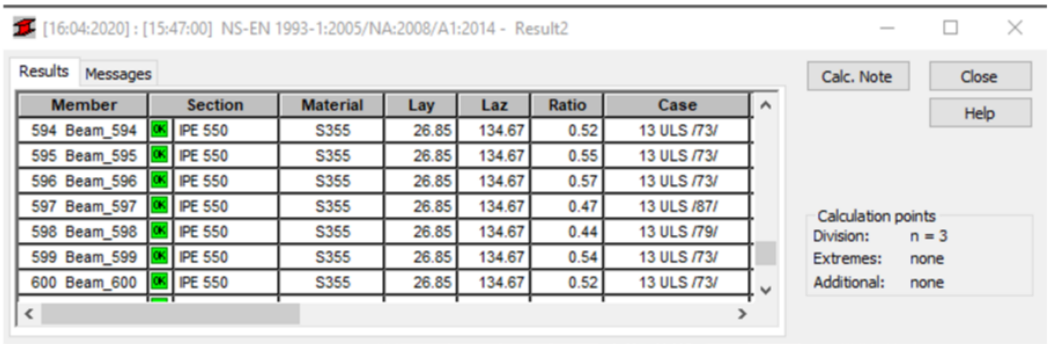
Member	Section	Material	Lay	Laz	Ratio	Case
44 Column_44	HEB 240	S355	35.90	60.82	0.71	13 ULS /83/
45 Column_45	HEB 240	S355	35.90	60.82	1.18	13 ULS /75/
46 Column_46	HEB 240	S355	35.90	60.82	1.21	13 ULS /75/
47 Column_47	HEB 240	S355	35.90	60.82	1.21	13 ULS /75/
48 Column_48	HEB 240	S355	35.90	60.82	1.18	13 ULS /75/
49 Column_49	HEB 240	S355	35.90	60.82	0.71	13 ULS /83/
50 Column_50	HEB 240	S355	35.90	60.82	0.39	28 ACC /10/

Figur 7.15: Columns verification



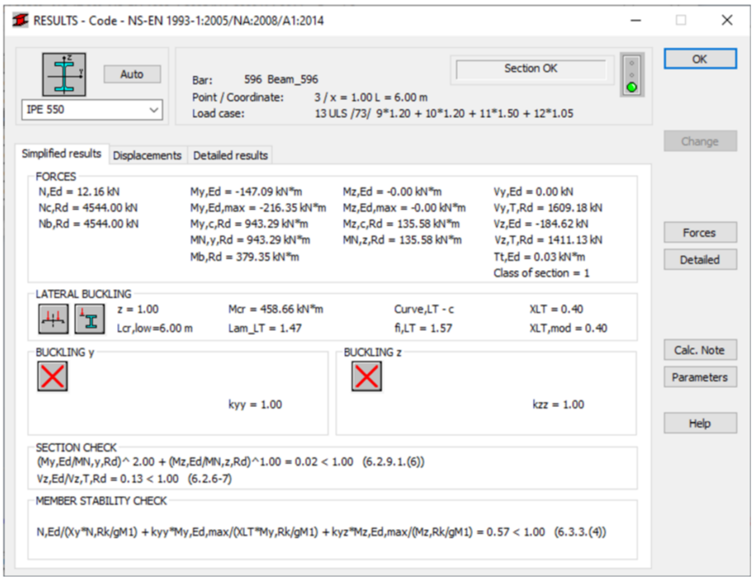
Figur 7.16: Results of verification details for column 47

The beams were all approved with the highest utilized beam at 0.57 ratio. This shows that a small adjustment in the beam size could be possible, although this would affect the entire structure. The loads case as shown in figure 7.14 is a combination of DL1, DL2, wind load X+, LL1, and SN1. This is included different factors, which becomes quite a large load case including wind and snow. This load case came up a lot as the toughest load case on the member verification list.



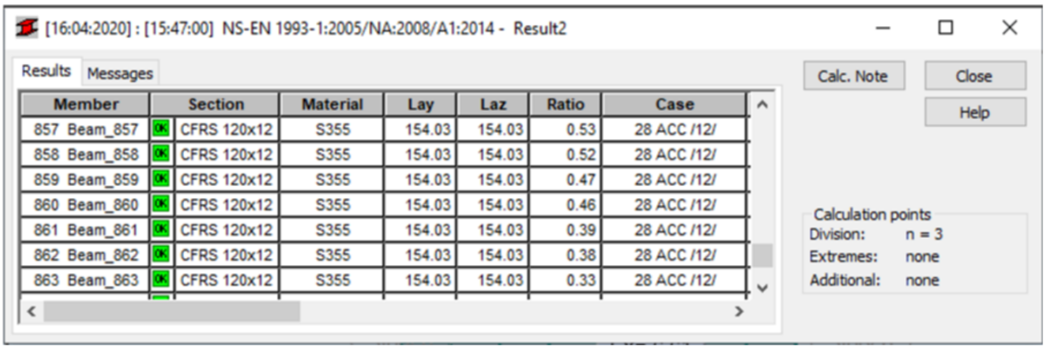
Member	Section	Material	Lay	Laz	Ratio	Case
594 Beam_594	IPE 550	S355	26.85	134.67	0.52	13 ULS /73/
595 Beam_595	IPE 550	S355	26.85	134.67	0.55	13 ULS /73/
596 Beam_596	IPE 550	S355	26.85	134.67	0.57	13 ULS /73/
597 Beam_597	IPE 550	S355	26.85	134.67	0.47	13 ULS /87/
598 Beam_598	IPE 550	S355	26.85	134.67	0.44	13 ULS /79/
599 Beam_599	IPE 550	S355	26.85	134.67	0.54	13 ULS /73/
600 Beam_600	IPE 550	S355	26.85	134.67	0.52	13 ULS /73/

Figur 7.17: Beams verification



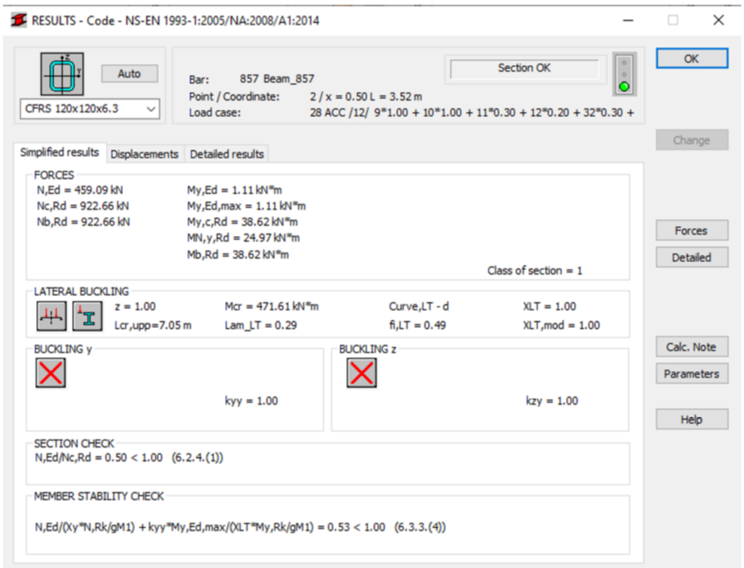
Figur 7.18: Verification of beam 596

The bracing has a ratio of 0,52 and 0,53 at the two highest utilized members. The bracing system had a most used load case which included earthquake forces. The load case combined ground shaking the dead loads and converted mass from snow load and live load. The figures of cross-bracing are shown below. The utilization of the cross-bracing could be better. Either the cross-section could be redefined or the placement and number of cross-bracing could heavily effect their utilization.



Member	Section	Material	Lay	Laz	Ratio	Case
857 Beam_857	CFRS 120x12	S355	154.03	154.03	0.53	28 ACC /12/
858 Beam_858	CFRS 120x12	S355	154.03	154.03	0.52	28 ACC /12/
859 Beam_859	CFRS 120x12	S355	154.03	154.03	0.47	28 ACC /12/
860 Beam_860	CFRS 120x12	S355	154.03	154.03	0.46	28 ACC /12/
861 Beam_861	CFRS 120x12	S355	154.03	154.03	0.39	28 ACC /12/
862 Beam_862	CFRS 120x12	S355	154.03	154.03	0.38	28 ACC /12/
863 Beam_863	CFRS 120x12	S355	154.03	154.03	0.33	28 ACC /12/

Figur 7.19: Bracing verification

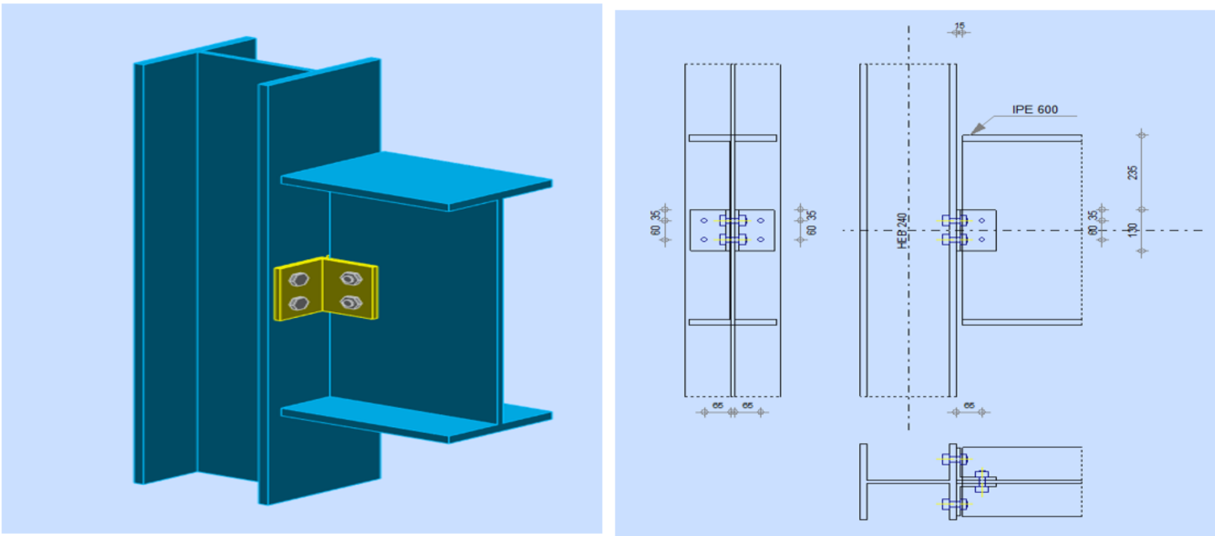


Figur 7.20: Verification on bracing beam 857

The moment-resisting structure was not approved on account of the HEB 240 not being approved. Also, as mention before, the beams were not utilized higher then 0,57 and could therefore be utilized better. The structure could definitely be redesigned so that the material consumption and utilization can see an improvement. Especially, considering the comparison of these two structures on both material consumption and behaviour from load cases have a great impact on the conclusion. Therefore, after the results from the cross-bracing structure, the structures will be slightly redesigned in order to optimize the results.

7.2 Cross-bracing frame

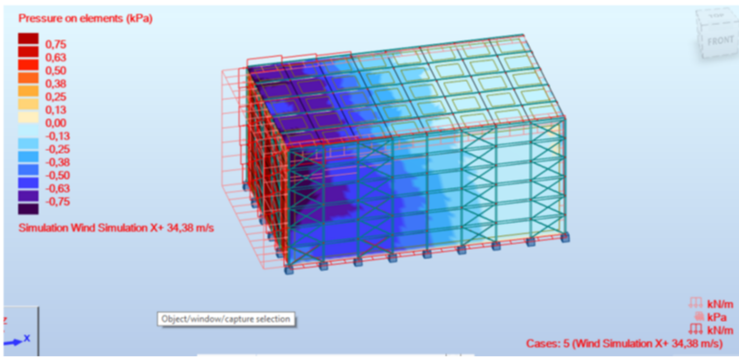
The cross-bracing frame has 144 bracing elements as opposed to the 48 bracing elements on the moment resistant building. All the bracing elements have pinned-pinned connections, meaning that they are pinned at both ends of the element. Also, in this structure the beams are pinned-pinned. The results follow the same format as the bracing structure, where wind, other loads (DL1,DL2,LL1, SN1), earthquake results, and member verification are shown. Finally, it will be seen if the bracing structure is approved or not.



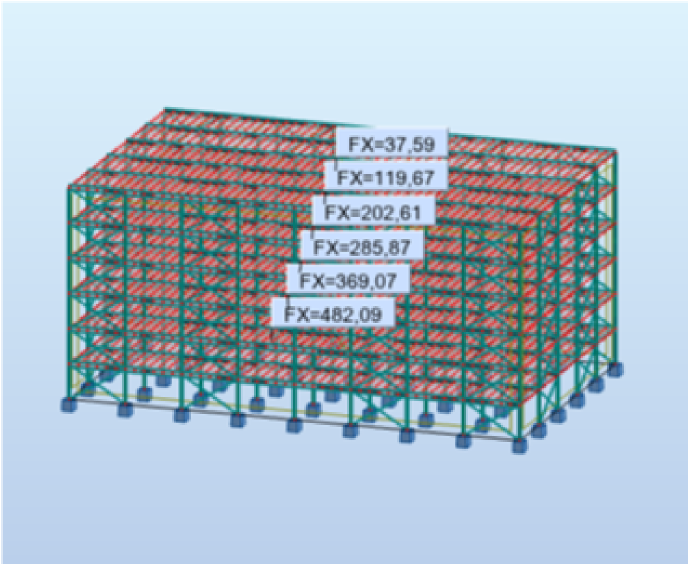
Figur 7.21: Connection design for cross- braced frame from robot model

7.2.1 Wind

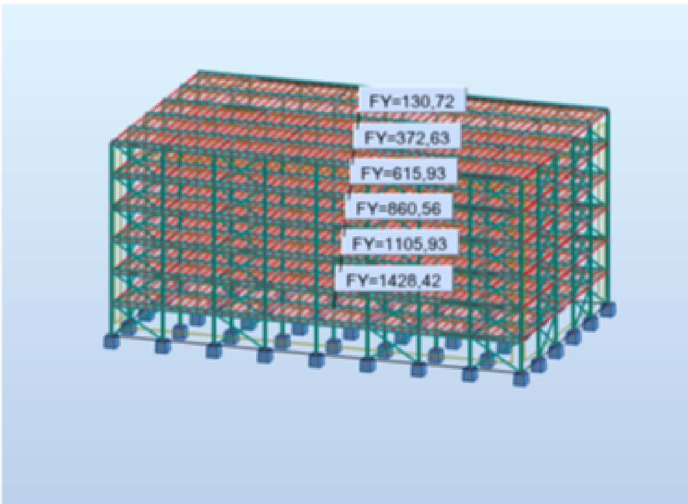
The wind forces are generated exactly as in the moment-resisting structure. The wind simulation program in Robot simulates the cases in 8 different directions, with each direction receiving a different load case that can be looked at individually. Figure 7.22 shows the wind load case X+ as it is generated as a surface load around the structure. The base shear and deformations from the wind loads were quite similar to the moment structure with only minor differences.



Figur 7.22: Wind load on surface



Figur 7.23: Base shear force from wind load in X-direction



Figur 7.24: Base shear force from wind load in Y-direction

7.2.2 Earthquake analysis

The earthquake analysis of the cross-bracing structure followed the same procedure as the moment structure. The period and frequency are slightly different to the moment structure in the modal analysis. Again, modal seismic analysis was utilized using the same parameters as in the moment structure. The parameters for the modal seismic analysis can be found in the case.

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)
31/ 1	0,66	1,52	0,00	78,79	0,00	0,00	78,79	0,00	5997556,98	5997556,98	5997556,98
31/ 2	1,07	0,93	81,00	78,79	0,00	81,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 3	1,27	0,79	81,00	78,79	0,00	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 4	2,11	0,47	81,00	94,43	0,00	0,00	15,64	0,00	5997556,98	5997556,98	5997556,98
31/ 5	3,33	0,30	94,79	94,43	0,00	13,79	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 6	3,69	0,27	94,79	98,00	0,00	0,00	3,57	0,00	5997556,98	5997556,98	5997556,98
31/ 7	4,03	0,25	94,79	98,00	0,00	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 8	4,86	0,21	94,79	99,33	0,00	0,00	1,33	0,00	5997556,98	5997556,98	5997556,98
31/ 9	5,71	0,18	94,79	99,82	0,00	0,00	0,49	0,00	5997556,98	5997556,98	5997556,98
31/ 10	5,75	0,17	98,20	99,82	0,00	3,41	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 11	5,80	0,17	98,20	99,82	59,77	0,00	0,00	59,77	5997556,98	5997556,98	5997556,98
31/ 12	5,88	0,17	98,20	99,82	59,77	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 13	5,94	0,17	98,20	99,82	59,77	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 14	6,02	0,17	98,20	99,82	59,77	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 15	6,06	0,17	98,20	99,82	66,66	0,00	0,00	6,89	5997556,98	5997556,98	5997556,98
31/ 16	6,20	0,16	98,20	99,82	66,66	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98
31/ 17	6,23	0,16	98,20	99,95	66,66	0,00	0,12	0,00	5997556,98	5997556,98	5997556,98
31/ 18	6,39	0,16	98,20	99,95	66,66	0,00	0,00	0,00	5997556,98	5997556,98	5997556,98

Figur 7.25: Swing modes for cross-bracing structure

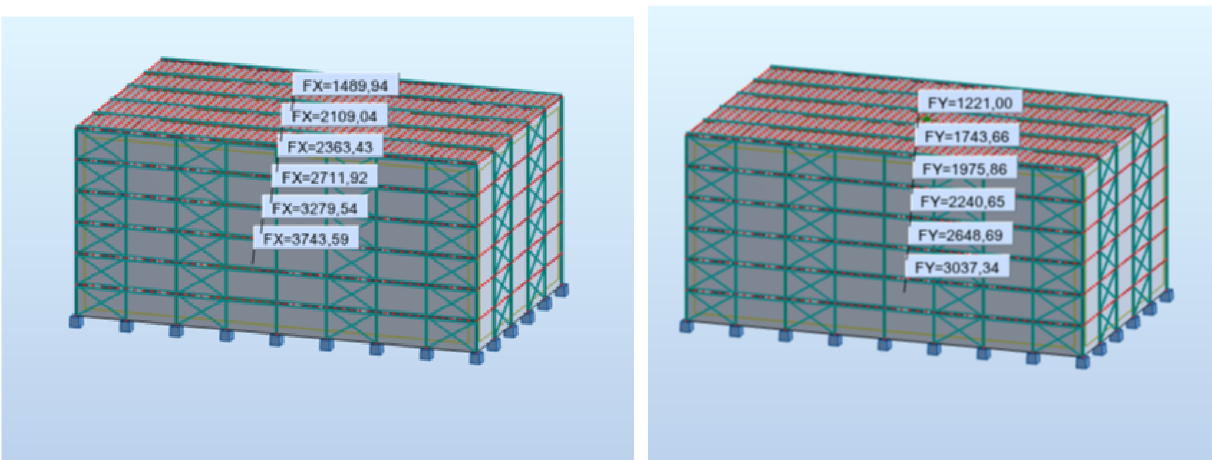
The base shear forces and deformation can be seen in the figures below. The base shear force in the X-direction was quite a bit higher than for the moment resisting frame, but at the same time the deformations were lower. This could be on account of the bracing in the X-direction. This could be due to minimal changes in the Y directions for both buildings. You can also see a change in the highest story drift in the X-direction from 24mm in the moment frame to 16mm in the cross-bracing structure. The base shear and deformations in the Y-direction were quite similar to the moment structure.

The highest story drift in the cross-bracing structure (dr UY) was 20mm. The story drift was approved as seen below.

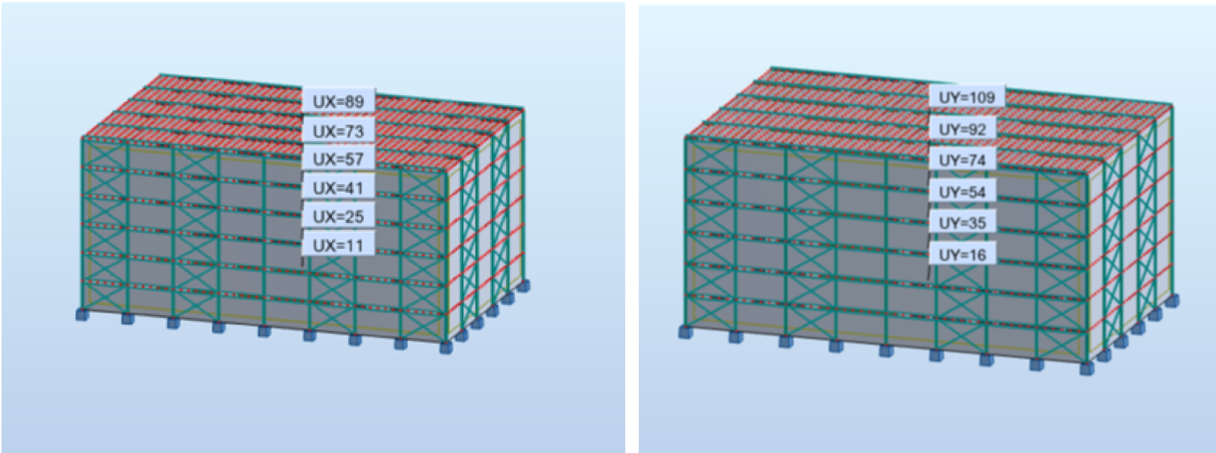
From equation 7.1, the story drift for cross-bracing can be calculated as follows;

$20mm \times 2.5 \times 0,4 \leq 0,0075 \times 3700$

Gives $20mm \leq 27.75mm$



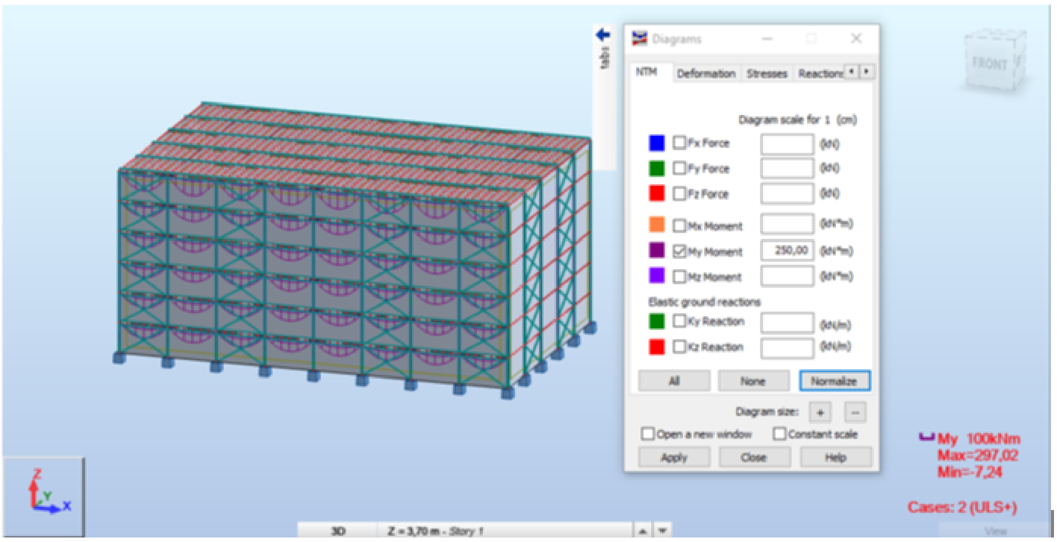
Figur 7.26: Base shear force in X and Y-direction



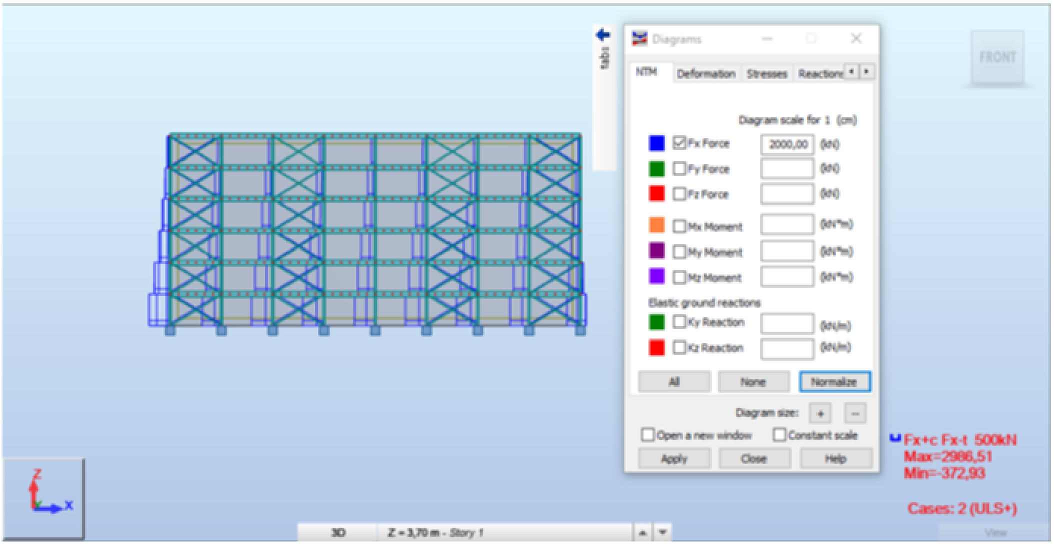
Figur 7.27: Deformation in X and Y-direction

7.2.3 Forces (DL1, DL2, LL1, and SN1)

Here you can see the moment diagrams and forces on the bars resulting from the ULS code combinations. The maximum moment on the beams amounted to 297.02 kNm. As you can see the moment are zero at the joints due to the pinned-pinned connections for the beams. The columns had forces close to 3000kN at the bottom of the structure.



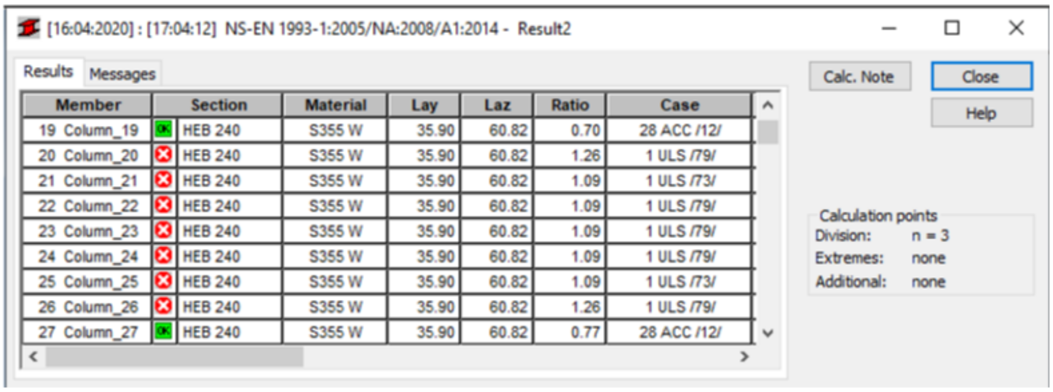
Figur 7.28: Moment on the beams



Figur 7.29: Force on the column

7.2.4 Members verification

The member verification here shows the result of the bracing structure. There were 35 HEB 240 columns that were not approved. The figures show that the utilization of some columns was very high with 126% in some cases. The code combination used here was a combination of wind, snow, dead loads, and live load.



[16:04:2020] : [17:04:12] NS-EN 1993-1:2005/NA:2008/A1:2014 - Result2

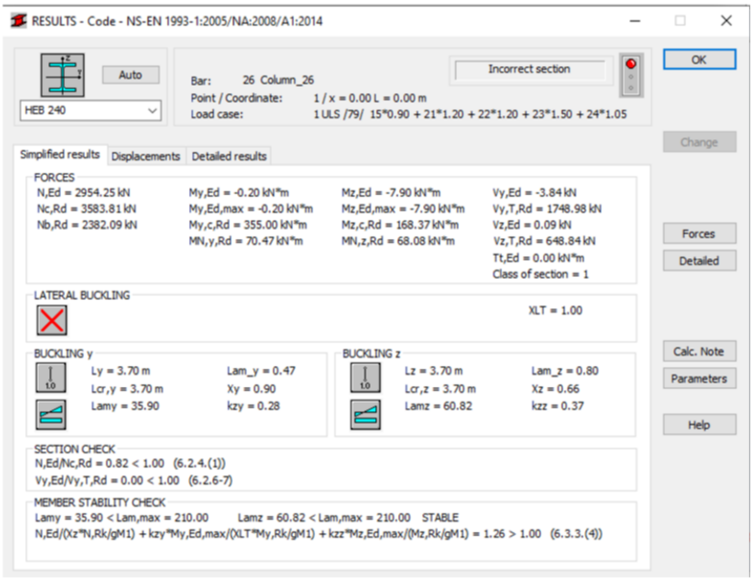
Results | Messages

Member	Section	Material	Lay	Laz	Ratio	Case
19 Column_19	HEB 240	S355 W	35.90	60.82	0.70	28 ACC /12/
20 Column_20	HEB 240	S355 W	35.90	60.82	1.26	1 ULS /79/
21 Column_21	HEB 240	S355 W	35.90	60.82	1.09	1 ULS /73/
22 Column_22	HEB 240	S355 W	35.90	60.82	1.09	1 ULS /79/
23 Column_23	HEB 240	S355 W	35.90	60.82	1.09	1 ULS /79/
24 Column_24	HEB 240	S355 W	35.90	60.82	1.09	1 ULS /79/
25 Column_25	HEB 240	S355 W	35.90	60.82	1.09	1 ULS /73/
26 Column_26	HEB 240	S355 W	35.90	60.82	1.26	1 ULS /79/
27 Column_27	HEB 240	S355 W	35.90	60.82	0.77	28 ACC /12/

Calc. Note | Close | Help

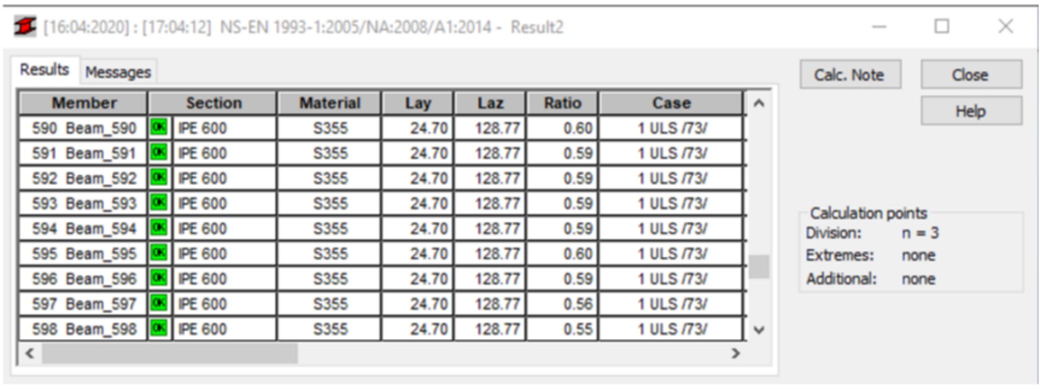
Calculation points
Division: n = 3
Extremes: none
Additional: none

Figur 7.30: Column verification bracing structure



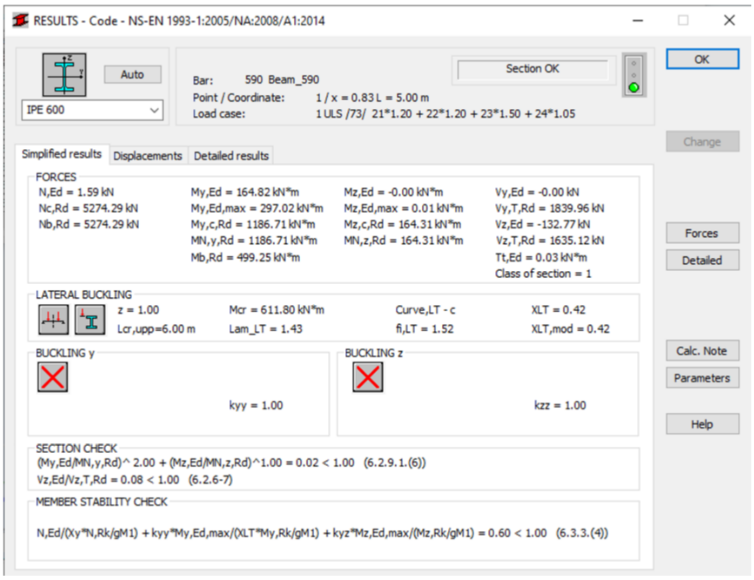
Figur 7.31: Verification details for column 26

All the IPE 600 beams were approved but the utilization increased slightly compared to the moment structure. The utilization was at 60% at the highest, compared to 57% utilization using IPE 550 beams at the moment structure.



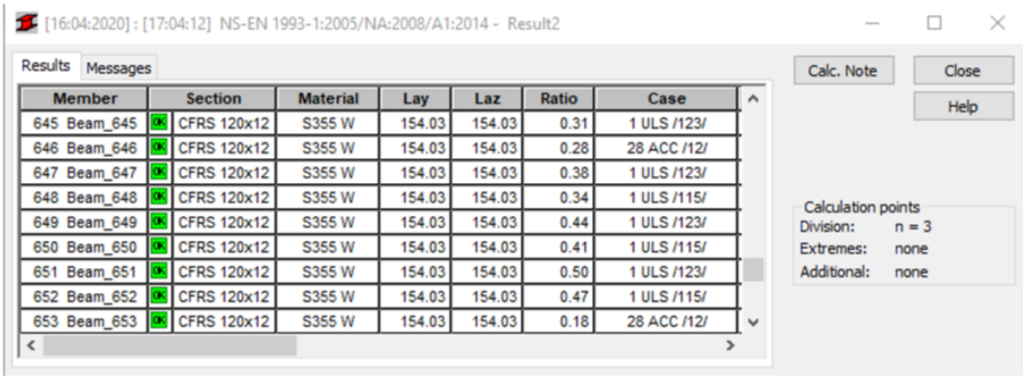
Member	Section	Material	Lay	Laz	Ratio	Case
590 Beam_590	OK IPE 600	S355	24.70	128.77	0.60	1 ULS /73/
591 Beam_591	OK IPE 600	S355	24.70	128.77	0.59	1 ULS /73/
592 Beam_592	OK IPE 600	S355	24.70	128.77	0.59	1 ULS /73/
593 Beam_593	OK IPE 600	S355	24.70	128.77	0.59	1 ULS /73/
594 Beam_594	OK IPE 600	S355	24.70	128.77	0.59	1 ULS /73/
595 Beam_595	OK IPE 600	S355	24.70	128.77	0.60	1 ULS /73/
596 Beam_596	OK IPE 600	S355	24.70	128.77	0.59	1 ULS /73/
597 Beam_597	OK IPE 600	S355	24.70	128.77	0.56	1 ULS /73/
598 Beam_598	OK IPE 600	S355	24.70	128.77	0.55	1 ULS /73/

Figur 7.32: Beams verification for cross-bracing structure



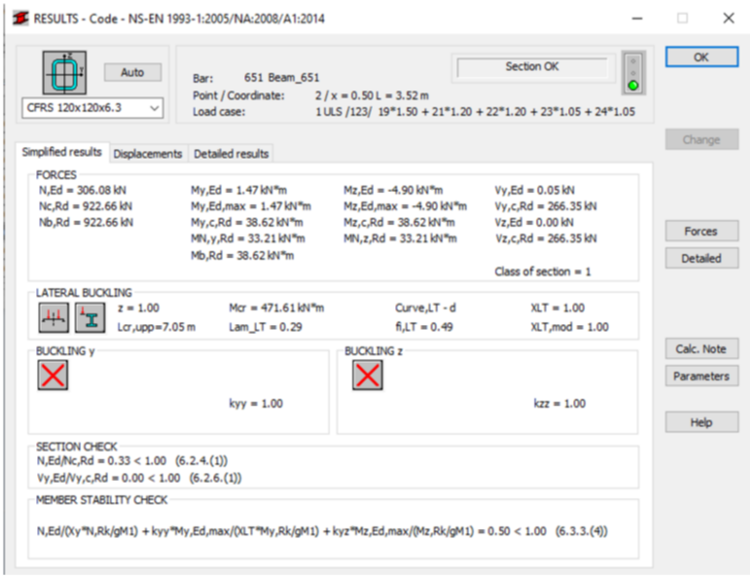
Figur 7.33: Verification details for beam 590

The bracing was also verified with the highest ratio being 0.5. The code combinations that were highest for the cross bracing members were combinations were wind is a big factor or in some cases the earthquake combinations (ACC).


[16:04:2020] : [17:04:12] NS-EN 1993-1:2005/NA:2008/A1:2014 - Result2

Member		Section	Material	Lay	Laz	Ratio	Case
645 Beam_645	OK	CFRS 120x12	S355 W	154.03	154.03	0.31	1 ULS /123/
646 Beam_646	OK	CFRS 120x12	S355 W	154.03	154.03	0.28	28 ACC /12/
647 Beam_647	OK	CFRS 120x12	S355 W	154.03	154.03	0.38	1 ULS /123/
648 Beam_648	OK	CFRS 120x12	S355 W	154.03	154.03	0.34	1 ULS /115/
649 Beam_649	OK	CFRS 120x12	S355 W	154.03	154.03	0.44	1 ULS /123/
650 Beam_650	OK	CFRS 120x12	S355 W	154.03	154.03	0.41	1 ULS /115/
651 Beam_651	OK	CFRS 120x12	S355 W	154.03	154.03	0.50	1 ULS /123/
652 Beam_652	OK	CFRS 120x12	S355 W	154.03	154.03	0.47	1 ULS /115/
653 Beam_653	OK	CFRS 120x12	S355 W	154.03	154.03	0.18	28 ACC /12/

Figur 7.34: Cross-bracing Verification



Figur 7.35: Verification details for cross-bracing beam 651

The structure was not approved due to the HEB columns. Also here the utilization of the beams was low enough to try to redesign the structure. The new structure will be redesigned with slightly bigger cross-sections on the columns and smaller cross-section on the beams. The results will show how the material consumption and behaviour of the structures is in comparison to each other. The results show that the cross-bracing frame has less inter-story drift for earthquake force but slightly higher utilization of beams and columns. A more detailed comparison to find eventual differences will be shown after the results of the redesigned structures.

7.3 Redesign of moment-resisting and cross- bracing frame

The redesign of the structure consisted mainly in reducing the size of the beams. IPE 450 was first chosen for both structures but this proved to be too low for the bracing frame. Therefore, the final design consisted of IPE 500 beams for the bracing frame and IPE 450 for the moment frame. All the columns were slightly strengthened to HEB 260 from HEB 240. The bracing was not redesigned on either of the buildings. Although redesign of the bracing could have a profound effect on the structure, especially if you consider both cross section and placement. Here the results of the new structure are shown simultaneously in the three following sections, Earthquake Analysis, Forces and Member Design. There will also be comparison to the 1st design and between the moment and bracing structures of the new design.

7.3.1 Earthquake analysis

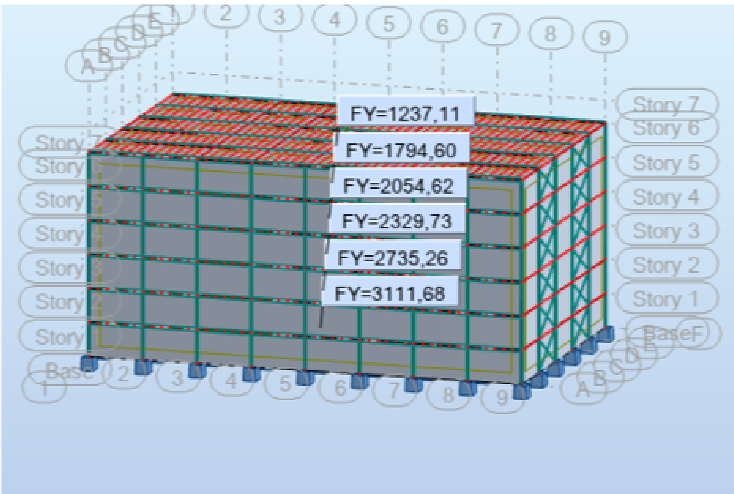
Here are the results of the modal analysis as well as base shear and the resulting deformations. The total mass is slightly smaller after the conversion of DL2, 30% of LL1, and 20% of the snow load. The figures below show the swing modes for the two structures.

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)
31/ 1	0,59	1,70	84,44	0,00	0,00	84,44	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 2	0,68	1,47	84,44	79,10	0,00	0,00	79,10	0,00	5917600,96	5917600,96	5917600,96
31/ 3	1,01	0,99	84,44	79,10	0,00	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 4	1,79	0,56	94,08	79,10	0,00	9,64	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 5	2,17	0,46	94,08	94,39	0,00	0,00	15,29	0,00	5917600,96	5917600,96	5917600,96
31/ 6	3,03	0,33	97,50	94,39	0,00	3,43	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 7	3,22	0,31	97,50	94,39	0,00	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 8	3,77	0,27	97,50	97,91	0,00	0,00	3,52	0,00	5917600,96	5917600,96	5917600,96
31/ 9	4,25	0,24	99,09	97,91	0,00	1,59	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 10	4,93	0,20	99,09	99,26	0,00	0,00	1,35	0,00	5917600,96	5917600,96	5917600,96
31/ 11	5,35	0,19	99,80	99,26	0,00	0,70	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 12	5,73	0,17	99,80	99,26	0,00	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 13	5,79	0,17	99,80	99,79	0,00	0,00	0,53	0,00	5917600,96	5917600,96	5917600,96
31/ 14	5,84	0,17	99,80	99,79	59,85	0,00	0,00	59,85	5917600,96	5917600,96	5917600,96
31/ 15	5,91	0,17	99,80	99,79	59,85	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 16	5,98	0,17	99,80	99,79	59,85	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96
31/ 17	6,03	0,17	99,80	99,79	66,49	0,00	0,00	6,64	5917600,96	5917600,96	5917600,96
31/ 18	6,05	0,17	99,80	99,79	66,49	0,00	0,00	0,00	5917600,96	5917600,96	5917600,96

Figur 7.36: Swing modes for moment resisting structure

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)
13/ 4	2,15	0,47	76,09	94,37	0,00	0,00	15,65	0,00	5953528,95	5953528,95	5953528,95
13/ 5	2,84	0,35	93,69	94,37	0,00	17,61	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 6	3,77	0,27	93,69	97,92	0,00	0,00	3,54	0,00	5953528,95	5953528,95	5953528,95
13/ 7	3,90	0,26	93,69	97,92	0,00	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 8	4,82	0,21	93,69	97,92	57,53	0,00	0,00	57,53	5953528,95	5953528,95	5953528,95
13/ 9	4,92	0,20	93,69	97,92	57,53	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 10	4,96	0,20	93,69	99,20	57,53	0,00	1,28	0,00	5953528,95	5953528,95	5953528,95
13/ 11	4,97	0,20	93,69	99,27	57,53	0,00	0,08	0,00	5953528,95	5953528,95	5953528,95
13/ 12	5,07	0,20	93,69	99,27	57,53	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 13	5,09	0,20	93,69	99,27	63,40	0,00	0,00	5,88	5953528,95	5953528,95	5953528,95
13/ 14	5,23	0,19	97,42	99,27	63,40	3,73	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 15	5,23	0,19	97,42	99,28	63,40	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 16	5,33	0,19	97,42	99,28	63,40	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 17	5,46	0,18	97,42	99,28	63,40	0,00	0,00	0,00	5953528,95	5953528,95	5953528,95
13/ 18	5,51	0,18	97,42	99,28	69,89	0,00	0,00	6,49	5953528,95	5953528,95	5953528,95

Figur 7.37: Swing modes for cross-bracing structure



Figur 7.38: Base shear force for redesign moment-resting frame in Y-direction

The figure below show the base shear and story drift of the first design and the new design of both buildings. As expected, the base shear and story drifts are quite similar from the first design to the new design. The biggest change is from the moment frame to the cross-bracing frame, especially in the X-direction. As you can see here the story drift and base shear change dramatically. The base shear increases while the story drift decreases in the cross-bracing structure. The story drift is approved in both structures as can be seen in the equation 7.1 where we take the highest story drift from the moment structure.

$$25mm \leq 27.75mm$$

Stories	Base shear moment		Story drift moment		Base shear bracing		Story drift bracing	
	FX(kN)	FY(kN)	dr UX (mm)	dr UY (mm)	FX(kN)	FY (kN)	dr UX (mm)	dr UY (mm)
1st floor	2484.04	3057.44	24	16	3743.59	3037.34	11	16
2nd floor	2239.55	2676.8	24	19	3279.54	2648.69	14	19
3rd floor	1987.96	2274.82	20	19	2711.92	2240.65	16	19
4th floor	1722.76	2005.78	17	19	2363.43	1975.86	16	20
5th floor	1407.33	1759.42	13	17	2109.04	1743.66	16	18
6th floor	931.95	1222.39	7	16	1489.94	1221	16	17

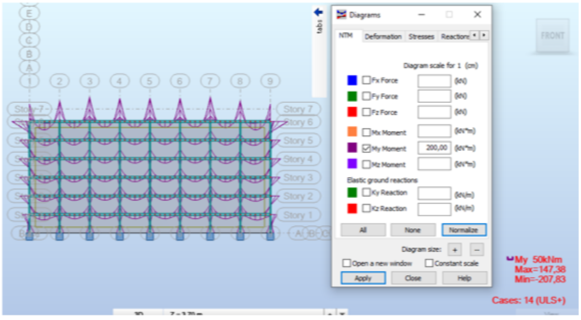
Figur 7.39: Base shear force and story drift of first design

Stories	Base shear moment		Story drift moment		Base shear bracing		Story drift bracing	
	FX(kN)	FY(kN)	dr UX (mm)	dr UY (mm)	FX(kN)	FY (kN)	dr UX (mm)	dr UY (mm)
1st floor	2479.01	3111.68	21	16	3778.71	3089.55	11	16
2nd floor	2235.08	2735.26	25	18	3338.71	2705.34	14	19
3rd floor	1982.83	2329.73	21	19	2787.93	2299.23	15	18
4th floor	1724.89	2054.62	18	18	2429.73	2027.04	15	19
5th floor	1419.92	1794.60	13	17	2139.83	1779.05	16	17
6th floor	958.69	1237.11	7	15	1493.44	1235.45	15	16

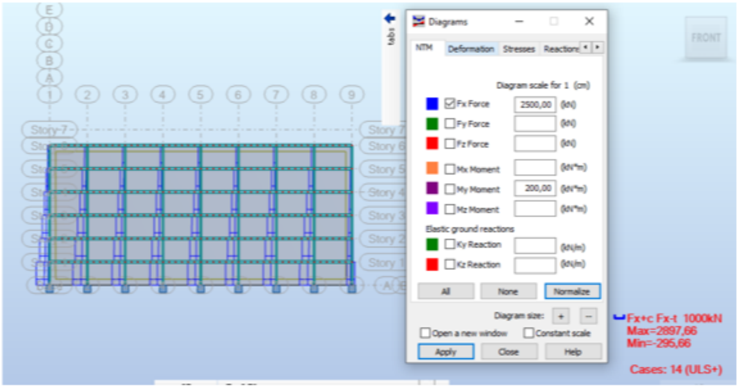
Figur 7.40: Base shear force and story drift of redesign/new design

7.3.2 Forces

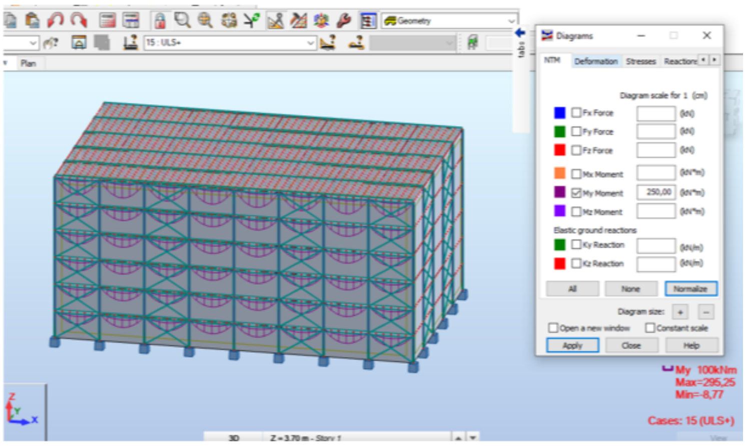
Here are the moment and force diagrams of the new designs. The minimum and maximum values are not much different to the first design. The moment diagrams show the pinned-pinned beams in the cross-bracing really well as it is obvious that there is no moment in the connections. Although some moment may be possible in the connection of the bracing structure due to eccentricities in the joint connections.



Figur 7.41: Moment diagram for redesign of moment-resisting structure



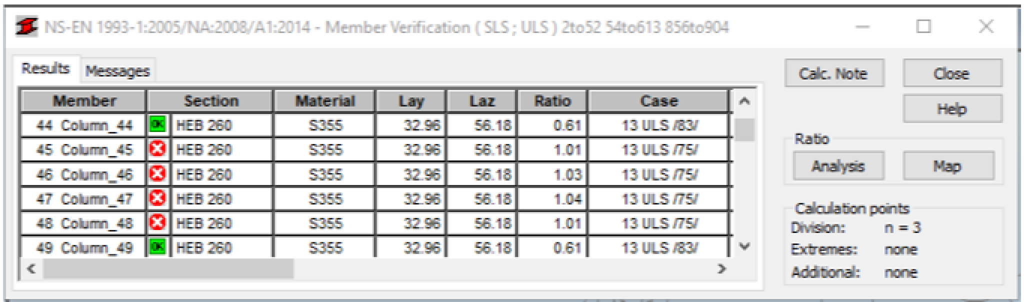
Figur 7.42: Force diagram for redesign of moment-resisting structure



Figur 7.43: Moment diagram for redesign of cross-braced structure

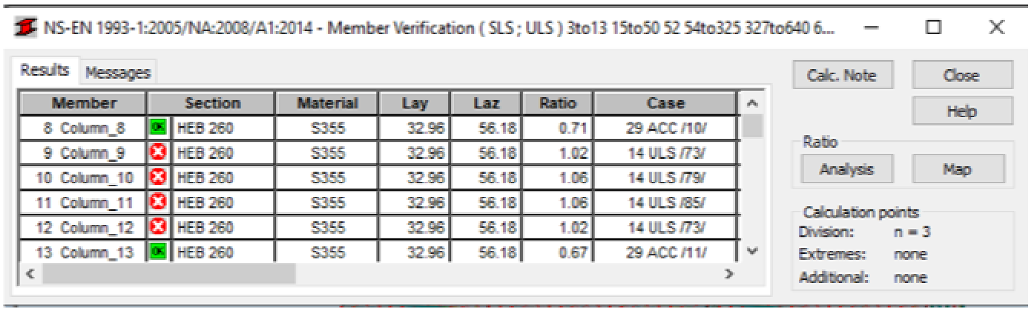
7.3.3 Members of verification

The member verification gave some interesting results. The columns were adjusted to HEB 260 and most of the columns were approved. As seen in the figures below, some columns were not approved but are very close to the 100% utilization. Therefore, considering all the factors, one could make an argument that the columns are strong enough after producing more calculation and documentation on the necessary columns. The column verification was quite similar when comparing the moment-resisting and cross-bracing structures. Each structure had 8 columns that were not approved in verification. The critical combination was wind combined with dead load, snow load, and live load (0.9 Wind + 1.2 DL1and2 + 1.5 LL1 + 1.05 SN1).

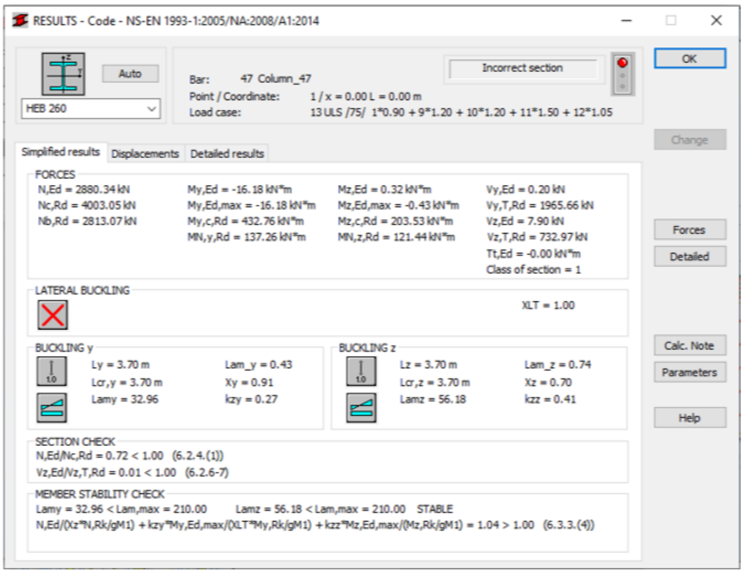


Member	Section	Material	Lay	Laz	Ratio	Case
44 Column_44	OK HEB 260	S355	32.96	56.18	0.61	13 ULS /83/
45 Column_45	HEB 260	S355	32.96	56.18	1.01	13 ULS /75/
46 Column_46	HEB 260	S355	32.96	56.18	1.03	13 ULS /75/
47 Column_47	HEB 260	S355	32.96	56.18	1.04	13 ULS /75/
48 Column_48	HEB 260	S355	32.96	56.18	1.01	13 ULS /75/
49 Column_49	OK HEB 260	S355	32.96	56.18	0.61	13 ULS /83/

Figur 7.44: Column verification of redesign moment-resisting structure

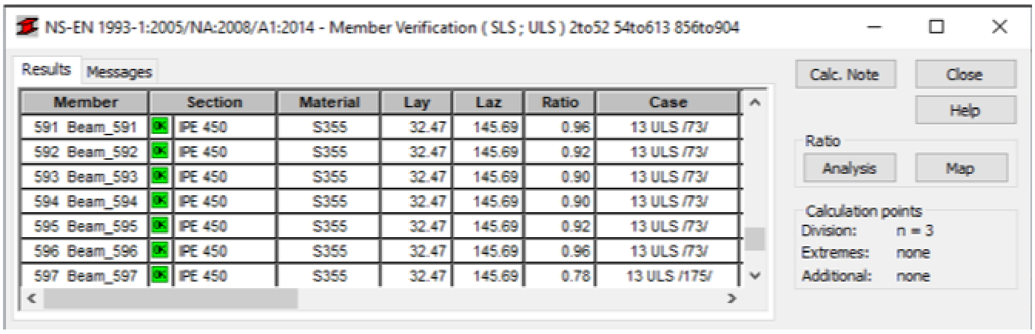


Figur 7.45: Column verification of redesign cross-braced structure



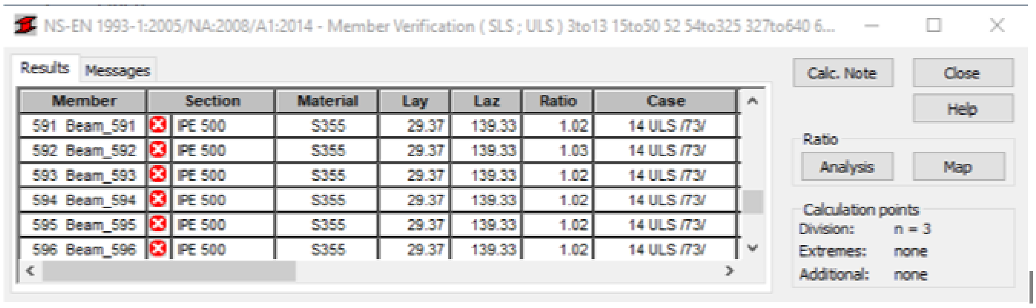
Figur 7.46: Verification details for column 47 in moment-resisting structure

The beam verification showed some key differences in the two structures. While the moment structure could be held up by IPE 450 beams, the bracing structure needed IPE 500 beams. The top floor beams were particularly exposed, with both live load and snow loads barrelling down on them, including the dead loads. As a result, many of the beams in the bracing structure had just over 100% utilization. There were 16 beams that were not approved in the bracing structure. The beams in the moment frame had highest utilization at 96 percent. The beams in the cross-bracing structure could again be approved through more calculation as they were right over 100% utilization. Here the critical combination involved live load and snow. Why the pinned-pinned connections on the beams gave higher utilization is something that can be studied further. One reason could be that the moment became higher in the middle of the beam, whereas in the moment structure the connections had to take some of the moment.



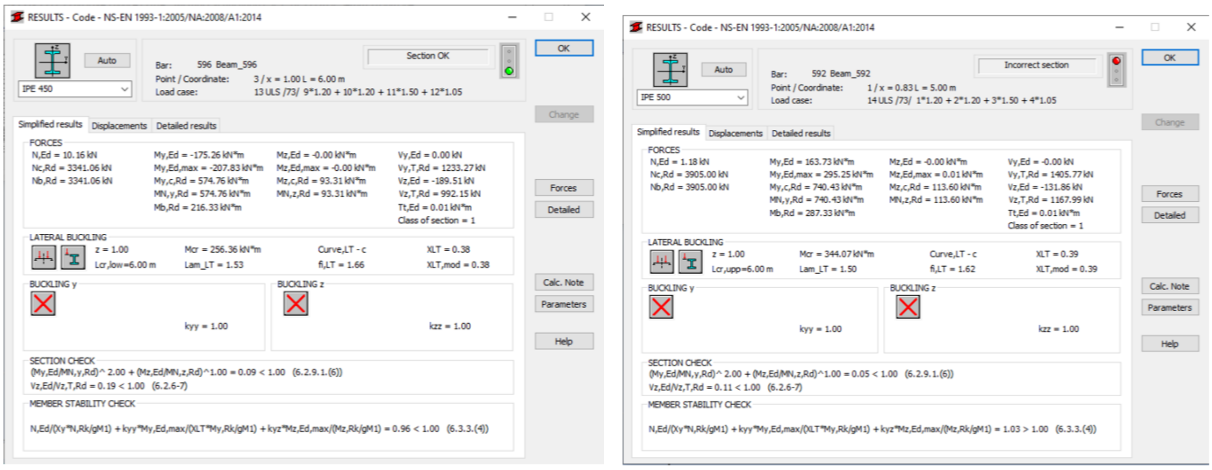
Member		Section	Material	Lay	Laz	Ratio	Case
591 Beam_591	OK	IPE 450	S355	32.47	145.69	0.96	13 ULS /73/
592 Beam_592	OK	IPE 450	S355	32.47	145.69	0.92	13 ULS /73/
593 Beam_593	OK	IPE 450	S355	32.47	145.69	0.90	13 ULS /73/
594 Beam_594	OK	IPE 450	S355	32.47	145.69	0.90	13 ULS /73/
595 Beam_595	OK	IPE 450	S355	32.47	145.69	0.92	13 ULS /73/
596 Beam_596	OK	IPE 450	S355	32.47	145.69	0.96	13 ULS /73/
597 Beam_597	OK	IPE 450	S355	32.47	145.69	0.78	13 ULS /175/

Figur 7.47: Beam verification of redesign moment-resisting structure



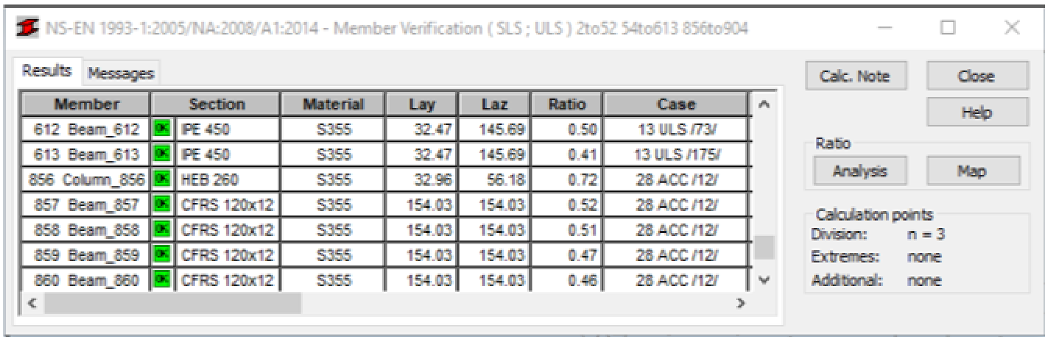
Member		Section	Material	Lay	Laz	Ratio	Case
591 Beam_591	✗	IPE 500	S355	29.37	139.33	1.02	14 ULS /73/
592 Beam_592	✗	IPE 500	S355	29.37	139.33	1.03	14 ULS /73/
593 Beam_593	✗	IPE 500	S355	29.37	139.33	1.02	14 ULS /73/
594 Beam_594	✗	IPE 500	S355	29.37	139.33	1.02	14 ULS /73/
595 Beam_595	✗	IPE 500	S355	29.37	139.33	1.02	14 ULS /73/
596 Beam_596	✗	IPE 500	S355	29.37	139.33	1.02	14 ULS /73/

Figur 7.48: Beam verification of redesign cross-bracing structure



Figur 7.49: Verification details for beam 596 and 592 in moment-resisting and cross-bracing structure

The bracing verification was quite similar to the first design with a ratio of around 0.5 at the highest, considering both structures. The critical combinations on these members involved either earthquake forces or wind forces.

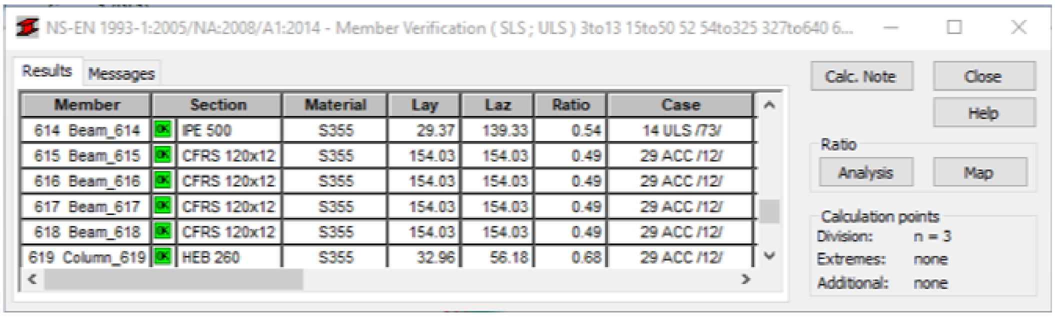


NS-EN 1993-1:2005/NA:2008/A1:2014 - Member Verification (SLS ; ULS) 2to52 54to613 856to904

Member	Section	Material	Lay	Laz	Ratio	Case
612 Beam_612	IPE 450	S355	32.47	145.69	0.50	13 ULS /73/
613 Beam_613	IPE 450	S355	32.47	145.69	0.41	13 ULS /175/
856 Column_856	HEB 260	S355	32.96	56.18	0.72	28 ACC /12/
857 Beam_857	CFRS 120x12	S355	154.03	154.03	0.52	28 ACC /12/
858 Beam_858	CFRS 120x12	S355	154.03	154.03	0.51	28 ACC /12/
859 Beam_859	CFRS 120x12	S355	154.03	154.03	0.47	28 ACC /12/
860 Beam_860	CFRS 120x12	S355	154.03	154.03	0.46	28 ACC /12/

Calculation points: Division: n = 3; Extremes: none; Additional: none

Figur 7.50: Bracing verification for redesign of moment-resisting structure



Figur 7.51: Bracing verification for redesign of cross-bracing structure

The final design gave a slight optimization of the design of the structures. The mass of the final structures show a difference over 30.000kg or 30 tons between the two structures, with the moment structure being the lighter structure. For earthquake purposes, the bracing structure was able to withstand deformations slightly better than the moment structure. Therefore in earthquake prone areas or areas with strong winds, bracing could enhance the structure. There are some more figures and example reactions on one structure in attachments of appendix 2. The connection design will go more into detail between the pinned connection and a fixed connection can be shown in the appendix 3.

7.4 Material consumption

Steel is a relatively expensive building material, so it is often that we have a responsibility to choose economic sizes and shapes according to the actual loads on the building to avoid overdesign. Because of the the higher cost of steel, we should find a best way to reduce the weight and size of some of steel members in the structure. As a result, our task is tried to compare two different structures in order to get the best option in relevance of material consumption.

The material consumption of moment resisting frame and x-bracing frame is calculated by hand calculation and robot structural analysis. Here, in the figures 7.52 and 7.53 are shown the hand calculation of the material consumption of two different stabilizing structures.

Moment Resistant Frame Building

Type of Steel Section	Structural element	Quantity/Number	Length[m]	Mass[kg/m]	Weight[kg] Per Unit	Total Weight [kg]
HEB 240	Column	324	3.7	83.2	308	99792
IPE 550	Beam	288	6	91	546	157248
HUP 120X120X6.3	Cross Bracing	48	7.049	22.3	157.2	7545.6
TOTALL WEIGHT=						264585.6

Cross Bracing Frame Building

Type of Steel Section	Structural Element	Quantity/Number	Length[m]	Mass[kg/m]	Weight[kg] Per Unit	Total Weight [kg]
HEB 240	Column	324	3.7	83.2	308	99792
IPE 600	Beam	288	6	104.5	627	180576
HUP 120x120x6.3	Cross Bracing Y direction	48	7.049	22.3	157.2	7545.6
HUP 120X120X6.3	Cross Bracing X Direction	96	7.049	22.3	157.2	15091.2
TOTALL WEIGHT =						303004.8

Figur 7.52: Comparison of material consumption of moment resistance frame and cross- bracing frame of the first design

The material consumption that calculated automatically in robot structural analysis is shown in the figure 7.54. Here, we have used a mass to load conversion and included the mass of the slabs, as a result it gave us higher material consumption than normally hand calculation.

Weight comparison between Moment Resistant Frame (MRF) and Cross Bracing Frame (CBF) after re-designing:

Moment Resistant Frame (MRF)

Type of steel section	Structural element	Quantity/Number	Length[m]	Mass[kg/m]	Weight[kg]per unit	Total weight[kg]
HEB-260	Column	324	3.7	68.2	252.34	81758.16
IPE-450	Beam	288	6	77.6	456.6	134092.8
HUP 120x120x6.3	Cross bracing elements in Y direction	48	7.049	22.3	157.2	7545.6

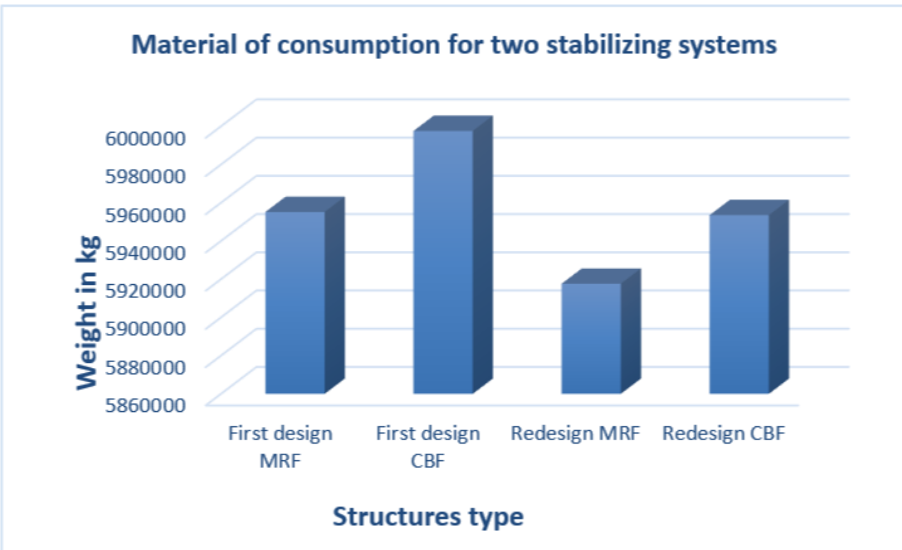
Total weight = 220804.56

Cross Bracing Frame (CBF)

Type of steel section	Structural element	Quantity/Number	Length[m]	Mass(kg/m]	Weight[kg]per unit	Total weight[kg]
HEB-260	Column	324	3.7	68.2	252.34	81758.16
IPE-500	Beam	288	6	90.7	544.2	156729.6
HUP-120x120x6.3	Bracing element in Y direction	48	7.049	22.3	157.2	7545.6
HUP-120x120x6.3	Bracing element in X direction	96	7.049	22.3	157.2	1509.2

Total weight= 261124.56

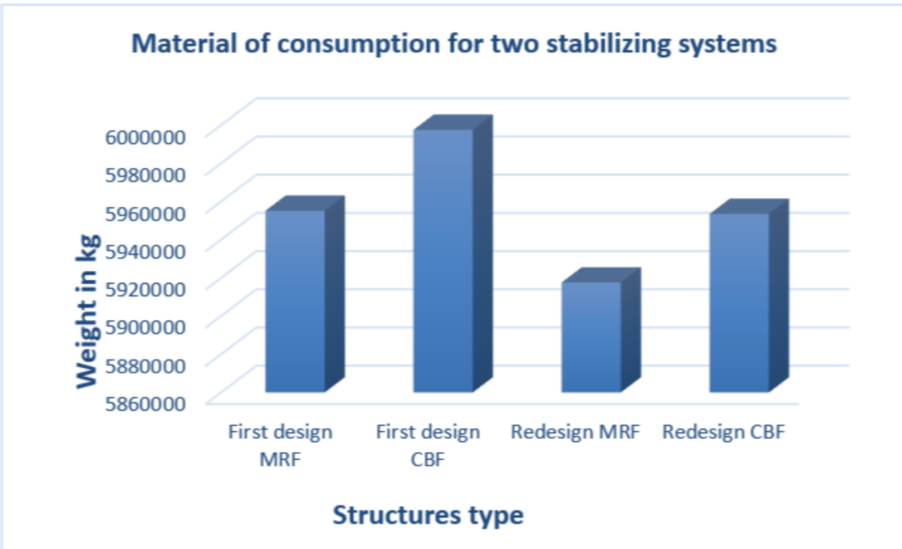
Figur 7.53: Comparison of material consumption of moment resistance frame and cross- bracing frame after redesign



Figur 7.54: Comparison of material consumption b/n MRF and CBF from Robot structural analysis

8 Discussion

The results show that the x-bracing frame has higher material consumption. This was, in part, due to the larger cross-section type in the beams. The x-bracing system gave a higher utilization of the beams. This may be due to the higher moment in the beams. While the moment resisting structure has some moment transferred to the connection, the x-bracing structure has no moment in the connection. Therefore the beam in the x-bracing structure has a higher moment. This again effects the verification of the beams. This and the extra bracing members give a result of significantly higher material consumption in the x-bracing structure.



Figur 8.1: Comparison of material consumption b/n MRF and CBF from Robot structural analysis

The earthquake design of the structure changes the picture slightly. The earthquake force is a horizontal force and effects the building significantly. It is well known that bracing members are good against horizontal forces. While the deformation from the wind forces was small in these relatively low structures, the differences could be seen more clearly after the earthquake design. The moment frame had a higher story drift then the x-bracing frame. Although both buildings were approved, the moment resisting frame came close to the maximum story drift. The earthquake design showed that the design of a building can become more of a challenge with higher heights or higher earthquake forces. Factors like mass, ductility, and stiffness of a building can have a profound effect on the earthquake design. How much deformation and story drift should be allowed in earthquake design is also something that could be studied further.

The analysis was performed by two group members. This was to ensure some precision and control to the results. Overall the results were quite similar though, ensuring some reliability in the results. The verifications and analysis were performed several times to ensure more accurate results. Preferably analysis should be performed several times and also utilizing different analysis programs to ensure the reliability of the results.

The definition of the structure is very important in the analysis. The connections from the beams to column had to be defined as pinned and similarly from bracing to column. The connections from column to column were fixed in all cases. The moment structure did not need any special definitions in the connection except from the bracing to the columns, which was pinned. The beams in the moment structure had fixed connections. The hollow slab was also defined as one way for the forces, and the measurements etc. are seen in Case.

It is possible that some definitions in the structure are inaccurate. Especially the connections between the hollow slab and beam, there might be a need to define the connection more accurately. This would require more studies into the behaviour of hollow slabs and how to properly define their interaction with the load bearing columns. Also, if eventually there would be added some concrete walls or non-structural elements that would contribute to the stiffness of the structure, this could have some effect on the beams and columns. Therefore the analysis and results give a pinpoint to the behaviour of the structure but are not entirely conclusive.

9 | Conclusion

The overall conclusion of the bachelor thesis regarding the moment resisting frames being more economical than traditional cross bracings was confirmed after verification of structural design. However, the design of the structures studied from an earlier bachelor project were shown to be slightly inaccurate. The utilization of the beams was quite low, while the columns were slightly overworked. The design of the structures studied from an earlier bachelor project were shown to be slightly inaccurate. The utilization of the beams was quite low, while the columns were slightly overworked. This prompted us to redesign the structure accordingly.

The results of the redesign gave similar results to the bachelor project. The cross-bracing frame required a larger beam size than the moment frame. The beam size and amount of cross-bracing members were the only material differences in the two structures. This gave the result that the cross-bracing structure had a higher material consumption than the moment structure. Our results show that it is a misconception that moment resisting frames require more material consumption, as our bracing frame weighed 30 tons more than the moment frame and had 92 more bracing elements.

Both designs were verified according to the appropriate Euro codes and national standards. The loads and combinations were applied accordingly. Both structures were verified, although with some members being just over the 100 percent utilization. This can arguably be justified with the necessary calculations and documentation. The redesign made both structures weigh slightly less and therefore they became more optimized and sustainable.

The earthquake analysis gave some interesting results. The earthquake acceleration value was chosen for the most earthquake prone areas in Norway. Although the base shear was high for the cross-bracing frame, the story drift was lower than for the moment frame. The moment structure became more deformation sensitive while the bracing structure was more acceleration sensitive when considering the ground motion. The bracing structure is definitely more robust when considering earthquake analysis. If the height or ground motion was increased, the earthquake design would become an increasingly more important factor. While both building were approved for story drift, the story drift of the moment frame was very close to the maximum.

The results show that the moment resisting frame may be more preferable. The material consumption and the construction of the structure seem to be an advantage, although the construction of the connections is not considered here. If the structure was going to be built today, we would have to go for the moment resisting frame. The results also show the advantages of bracing and they are definitely an enhancement against the horizontal loads like wind and earthquake. If the structure was higher or had more extreme horizontal loads, the bracing members of the structure could be more predominant.

10 | Suggestions for further work

For further studies, the behaviour of hollow slabs and or the effect of having a concrete wall, either load bearing or non-structural could be investigated further. Also, the size and positioning of bracing could be studied further. Where does the bracing have most effect in a structure? If effective placement of bracing could supplement number of bracing elements, this would be an interesting topic to study further. The connection design can also be studied further considering horizontal loads. If you consider connection design with regard to horizontal loads, there are many topics to further investigate. Some examples of topics are connection construction, fatigue, ductility, and connection design.

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Appendices

A Appendix 1: Eurocode 0 and 1

A.1 Eurocode 0 (Basic structural- load combinations)

Basic variables- classification of actions (section 4.1.1) Action can be classified by variation of their time and divided into three as follows.

- Permanent action (G), ex. Self-weight of structures, fixed equipment's and road surfacing, indirect action caused by shrinkage and uneven settlements.
- Variable action (Q), ex. Imposed loads on buildings floor, beams and roofs, wind action or snow load.
- Accidental action (A), ex. Explosions, or impact from vehicles.

Note: some actions like snow loads and seismic actions may considered either accidental or variable actions.

Representative value of variable action: There are four different representative of variable actions such as:

- Characteristic value (Q_k)
- Combination value of variable action (ψ_0 Q_k)
- Frequent value of variable action (($\psi_1 Q_k$). This type of variable is selected more for buildings.

- The Quasi-permanent value of variable action ($\psi_2 Q_k$)

Verification by partial factor method: Design value of action F_d (section 6.3.1)

$F_d = \gamma_f F_{rep}, with\ F_{rep} = \psi F_k$

Where,

F_k is the characteristic value of action

F_{rep} is relevant representative value of action

γ_f is partial factor for the action of unfavorable deviation action

Design value of the effect action $\mathbf{E}_d$ $(section 6.3.2)$: For specific load case the design value of the effect action is;

$E_d = \gamma_{Sd} E\left\{\gamma_{f,i} F_{rep,i}; a_d\right\} i > 1$

Where,

a_d is design value of the geometrical data (see section 6.3.4)

γ_{Sd} is partial factor taking account of uncertainties

Combination of action for seismic design situation (section 6.4.3.4) The general format of effect of action should be expressed by:

$\mathrm{E}_d = E\left\{G_{k,j}; P; A_{Ed}; \psi_{2,i}Q_{k,i}\right\}$

$j \geq 1$; $i \geq 1$

Combination action in the brackets { } can be expressed as:

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

Where, A_{Ed} is design value for seismic action.

Combination of action for accidental design situation (section 6.4.3.3) The general format of effect of action should be expressed by:

$\mathrm{E}_d = E\left\{G_{k,j}; P; A_d; (\psi_{1,1} \text{ or } \psi_{2,1})\mathrm{Q}_{k,1}; \psi_{2,i}Q_{k,i}\right\}$

$j \geq 1$; $i > 1$

Combination action in the brackets { } can be expressed as:

$\sum_{j\geq 1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) + \sum_{i>1} \psi_{2,i}Q_{k,i}$

Where, A_d is design value for accidental action.

Combination of actions for serviceability limit states:

1. Characteristic combination:

   $\mathrm{E}_d = E\left\{G_{K,j]; P; \psi_{0,i}Q_{K,i}}\right\} j \geq 1; i > 1$

2. Frequent combination:

   $\mathrm{E}_d = E\left\{G_{k,j}; P; \psi_{1,1}Q_{k,1}; \psi_{2,i}Q_{k,i}\right\} j \geq 1; i > 1$

3. Quasi-permanent combination:

   $\mathrm{E}_d = E\left\{G_{k,j}; P; \psi_{2,i}Q_{k,i}\right\} j > 1; i \geq 1$

Application for buildings Annex A1:

The recommended value of ψ factors for buildings action;

Table A1.1

The characteristic value of all permanent action can be written as, $\mathrm{G}_{sup} if the total resulting action effect is unfa$

Table A1.3 is considered on the design value of action for using in accidental and seismic combination actions.

Note: the partial factors for actions for ultimate limit state and serviceability limit state in accidental and seismic design situation, should be equal to 1 and value is given in table A1.1.

Eurocode 0 (Basic structural- load combinations)

Basic variables- classification of actions (section 4.1.1)

Action can be classified by variation of their time and divided into three as follows.

- Permanent action (G), ex. Self-weight of structures, fixed equipment's and road surfacing, indirect action caused by shrinkage and uneven settlements.
- Variable action (Q), ex. Imposed loads on buildings floor, beams and roofs, wind action or snow load.
- Accidental action (A), ex. Explosions, or impact from vehicles.

Note: some actions like snow loads and seismic actions may considered either accidental or variable actions.

Representative value of variable action: There are four different representative of variable actions such as:

- Characteristic value (Q_k)
- Combination value of variable action ($\psi_0 Q_k$)
- Frequent value of variable action (($\psi_1 Q_k$). This type of variable is selected more for buildings.
- The Quasi-permanent value of variable action ($\psi_2 Q_k$)

Verification by partial factor method:

Design value of action$F_d(section 6.3.1)$

$F_d = \gamma_f F_{rep}$, with F$_{rep}$ =ψF_k

Where,

F_k is the characteristic value of action

F_{rep} is relevant representative value of action

γ_f is partial factor for the action of unfavorable deviation action

Design value of the effect actionE_d(section 6.3.2): For specific load case the design value of the effect action is;

$E_d = \gamma_{Sd} E\left\{\gamma_{f,i} F_{rep,i}; a_d\right\}$ i>1

Where,

a_d is design value of the geometrical data (see section 6.3.4)

γ_{Sd} is partial factor taking account of uncertainties

Combination of action for seismic design situation (section 6.4.3.4) The general format of effect of action should be expressed by:

$E_d = E\left\{G_{k,j}; P; A_{Ed}; \psi_{2,i} Q_{k,i}\right\}$

$j \geq 1$; $i \geq 1$

Combination action in the brackets $\{\}$ can be expressed as:

$\sum_{j\geq1} G_{k,j} + P + A_{Ed} + \sum_{i\geq1} \psi_{2,i} Q_{k,i}$

Where, A_{Ed} is design value for seismic action.

Tabell 1: The recommended value of ψ factors for buildings action

Imposed loading in buildings, (see in EN 1991-1-1)	ψ_0	ψ_1	ψ_2
Category A and Category B: domestic, residential area and office area	0.70	0.50	0.30
Snow loads on buildings (see EN 1991-1-3); Finland, Iceland, Norway and Sweden	0.70	0.50	0.20
Wind loads on building (see in EN 1991-1-4)	0.60	0.20	0

Combination of action for accidental design situation (section 6.4.3.3) The general format of effect of action should be expressed by:

$E_d = E\{G_{k,j}; P; A_d; (\psi_{1,1} \text{ or } \psi_{2,1})\mathrm{Q}_{k,1}; \psi_{2,i}Q_{k,i}\}$

$j \geq 1$; $i > 1$

Combination action in the brackets { } can be expressed as:

$\sum_{j\geq1} G_{k,j} + P + A_d + (\psi_{1,1} \text{ or } \psi_{2,1}) + \sum_{i>1} \psi_{2,i}Q_{k,i}$

Where, A_d is design value for accidental action.

Combination of actions for serviceability limit states:

1. Characteristic combination:

   $E_d = E\left\{G_{K,j];P;\psi_{0,i}Q_{K,i}}\right\}$ j$\geq$ 1; $i > 1$

2. Frequent combination:

   $E_d = E\{G_{k,j}; P; \psi_{1,1}Q_{k,1}; \psi_{2,i}Q_{k,i}\}$ j$\geq$ 1; $i > 1$

3. Quasi-permanent combination:

   $E_d = E\{G_{k,j}; P; \psi_{2,i}Q_{k,i}\} j > 1; i \geq 1$

Application for buildings Annex A1 is given in the table 1; the recommended value of ψ factors for buildings action.

The characteristic value of all permanent action can be written as, G_{sup} if the total resulting action effect is unfavorable and G_{inf} if the total result action effect is favorable as we can see in the table A1.3.

Table A1.3 is considered on the design value of action for using in accidental and seismic combination actions.

Note: the partial factors for actions for ultimate limit state and serviceability limit state in accidental and seismic design situation, γ should be equal to 1 and ψ value is given in table A1.1.

A.2 Eurocode 1 (Action on structures)

A.2.1 EN 1991-1-1, self weight and imposed load for buildings

Self-weight of construction should be classified as a permanent fixed action, while imposed load should be classified as a variable free action (section 2.1 and 2.2).

Design situation: we can see the design situation based on the following two loads.

Permanent loads: The total self-weight of structure and non-structure member should be taken as in combination of actions as a single action (section 3.2)

Imposed loads on building (section 6): Imposed loads that specified in this part are modelled the loads that distributed uniformly, line loads or concentrated loads or combination of the loads.

- For design of a floors: the imposed load in multi-stories assumed to be distributed uniformly (fixed action), and the total imposed load is reduced by a reduction factor,α_A.
- For design of column and walls: the imposed load should be placed at all unfavorable locations. The total imposed loads may be reduced by the reduction factor, α_n.

From table 6.2, the imposed load on the floors of the building could be $q_k = 2.0 - 3.0kN/m^2$ and $Q_k = 1.5 - 4.5kN$ for category B (office area).

The reduction factor is expressed by the following equations:

$_A = \frac{5}{7}\psi_0 + \frac{A_0}{A} \leq 1.0$

where, A_0 is basic area, and equal to $10m^2$, A is the loaded area

$\alpha_n = 2 + \frac{(n-2)\psi_0}{n}$

where, n is number of stories (n>2)

Note: national annex is alternative method.

A.2.2 EN:1991-1-4, Wind load

Basic values (section 4.2):

The fundamental value of basic wind velocity, Vb,0 is characteristic 10 minutes mean wind velocity Vm, not respective of wind direction and time of years, at 10m above ground level in open country terrain.

The basic wind velocity shall be calculated:

$V_b = C_{dir}.C_{season}.V_{b,0}$

Where, V_b is basic wind velocity, defined as a function of wind direction and time of years

C_{dir} is the direction factor, the recommended value is = 1 , see better in the national annex.

C_{season} is season factor, recommended value is 1 and refer in the national annex.

Wind force (section 5.3):

Wind force is the whole structure or structural component should be determined. Wind force F_w, acting on the structure of structural component can be explained directly by using the following expression:

$F_w = C_sC_d.C_f.q_p(Ze).A_{ref}$

Where, C_sC_d is structural factor, always taking as 1 for the frame building less than 100m high (see section 6.2)

C_f is the force coefficient for structure or structural element, and takes approx. 1.8

A_{ref} is the projected area of structure that exposed to wind loading, depending the types of structure or structural element.

$q_p(Ze)$ is the peak velocity pressure, and can write as $q_p(Ze) = C_e(Z).q_b$

where, q_b is basic velocity pressure, and expressed as $q_b = \frac{1}{2}\rho.V_b^2$

ρ is density of wind, recommended value is $1.25kg/m^3$ or find in the national annex.

$q_p(Ze) = (1 + 7I_v(Ze).\frac{1}{2}\rho V_m(Ze)^2$

where,

$I_v(Ze)$ is turbulence intensity at the reference height Z_e

$I_v(Ze) = \frac{k_I}{C_0(Ze).ln(\frac{Z_e}{Z_0})}$

for the case of $Z_e \geq Z_{min}$

K_I is turbulence factor, and considered as $K_I = 1.000$ (see in section 4.4)

$C_0(Ze)$ is orography factor, and considered as 1.000

$V_m(Ze)$ is mean wind velocity at the reference height Z_e depends on the terrain roughness.

$V_m = C_r(Ze).C_0(Ze).V_b$

Where, $C_r(Ze)$ is roughness factor

$C_r(Ze) = K_r.ln(\frac{Z_e}{Z_0}$, in case $Z_e \geq Z_{min}$

K_r is terrain factor, can be expressed as, $K_r = 0.19(\frac{Z_0}{Z_{0,II}})^{0.07}$

From table 4.1 terrain category II, $Z_0 = 0.5m$ and $Z_{min} = 2.0m$

And, $K_r = 0.19(\frac{0.5}{0.5})^{0.07}$ will be 0.1900

A.2.3 EN:1991-1-3, Snow load

Exceptional snow load on the ground (section 4.3):

The locations where the exceptional snow loads on the ground can occur and determined by:

$S_{Ad} = C_{esl}.S_K$

Where,

S_{Ad} is the design value of exceptional snow load on the ground for the given location.

C_{esl} is coefficient for exceptional snow loads, the recommended value is 2.0, or refer on the national annex.

S_k is the characteristic value of snow loads on the ground in the given locations. The snow load on the ground Sk for Norway is from $1.75 - 9.5kN/m^2$. For Grimstad/Oslo, S_k is $6.35kN/m^2$ (refer annex C; European ground snow load maps)

Snow load combinations:

Combinations value $\psi_0 S_K$:

$\sum_{j\geq1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$

The combination factor 0 is applied to the snow load effect when the dominating load effect is due to some external load like wind.

Frequent value $\psi_1 S_K$:

The frequent value $\psi_1 S_K$ is chosen so that the time, it is exceeded is 0.10 of the reference periods.

$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$ (Eurocode 0 eq. 6.15b)

Quasi-permanent $\psi_2 S_K$***:***

The quasi-permanent $\psi_2 S_K$ is usually chosen so that the proportion of time, its exceeded is 0.5 of the reference periods.

$\sum_{j\geq 1} G_{k,j} + P + \sum_{i>1} \psi_{2,1} Q_{k,i}$

N.B:

For locations where exceptional load may occur, the ground snow load may treat as an accidental action value, and the load combination can be written as:

$\sum_{j\geq 1} G_{k,j} + P + A_d + (\psi_{1,1} or \psi_{2,1}) Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$

(from Eurocode 0 eq. 6.11b)

Snow load on the roof (section 5.1):

Properties of the roof must be considered on:

- The shape of the roof
- Its thermal properties
- The roughness of its surface
- Amount of heat generated under the roof
- The proximity of nearby buildings
- Surrounding terrain

Snow loads on the roof can be determine as:

1. For persistent/transient design situations $S = \mu_i C_e C_t S_k$
2. For accidental design situation where exceptional snow load is accidental action $S = \mu_i C_e C_t S_{Ad}$
3. For accidental design situation where exceptional snow drift is accidental action $S = \mu_i S_k$

   Where,

   μ_i is snow load shape coefficient (see in annex b and section 5.3, and well represented in figure 5.1 and 5.2 and table 5.2).

   $C_e$is the exposure coefficient, and taking as Ct = 1 (see table 5.1)

   C_t is the thermal coefficient, and used for reduction of snow loads, and taking as Ct = 1 for all case

   Note: In regions with possible rainfalls on snow, and causing melting and freezing, the snow load on the roof dramatically increased.

   Roof shape coefficients:

   There are several different roof shape coefficients, such as:

   - Monopitch roof
   - Pitched roof

- Multi-span roof
- Cylindrical roof
- Roof butting and close to falls construction works

The roof shape coefficient depends on the roof angle; for example, $\mu_1 = 0.8$ if $0^0 \leq \alpha \leq 30^0$

The snow load shape; $\mu_1 = 0.8$ and $\mu_2 = \frac{\gamma.h}{S_k}$ with the restriction of $0.8 \leq \mu_2 \leq 2.0$

Where,

γ is the weight density of the snow, taking as $2kN/m^3$ (according section 6.2)

The bulk weight density of snow is different in relevance with time (see annex E):

Fresh snow $= 1.0kN/m^3$

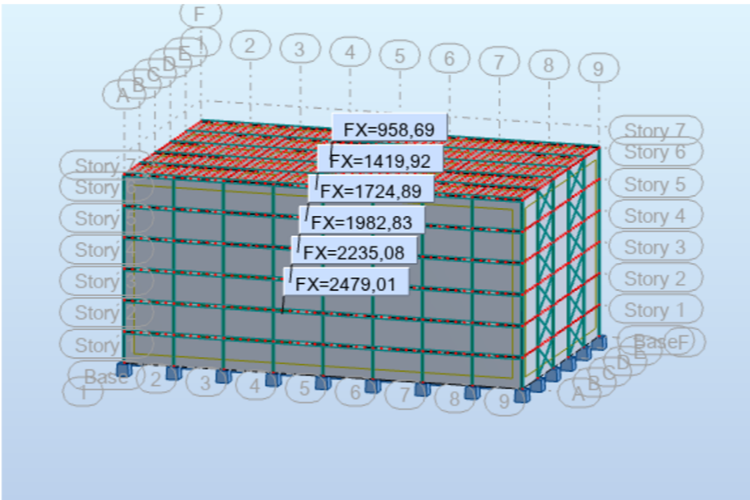
Settled snow (several hours or several days after fall) $= 2.0kN/m^3$

Old snow (several weeks or months) $= 2.5 - 3.5kN/m^3$

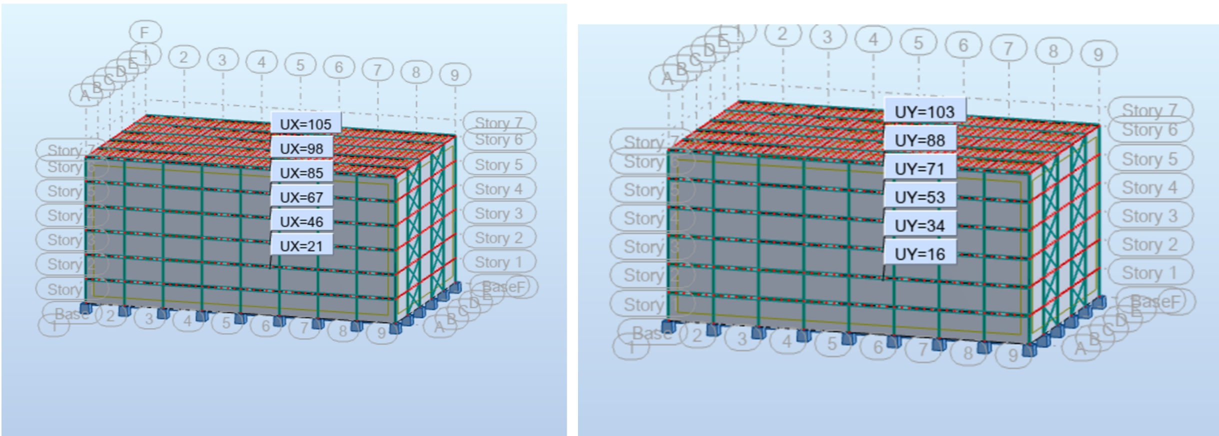
Wet snow $= 4kN/m^3$

B Appendix 2: Redesign attachment

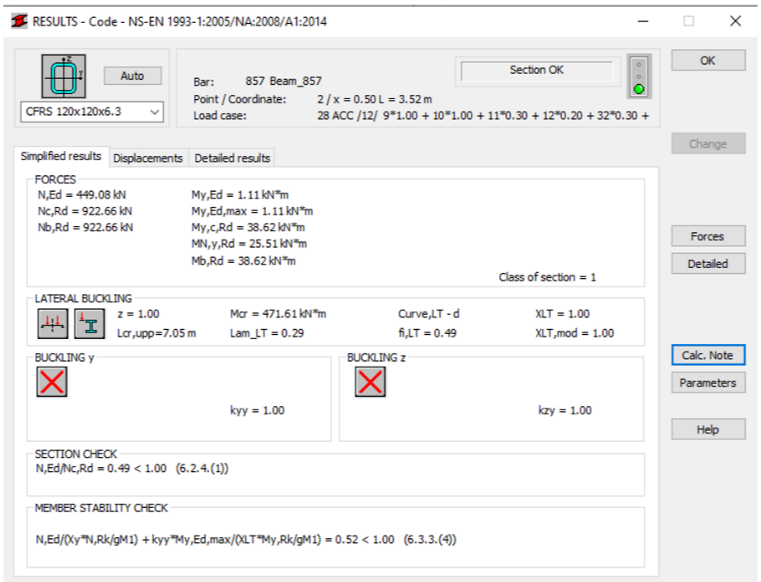
B.1 Moment-resisting frame



Figur 1: Base shear force for moment-resisting structure

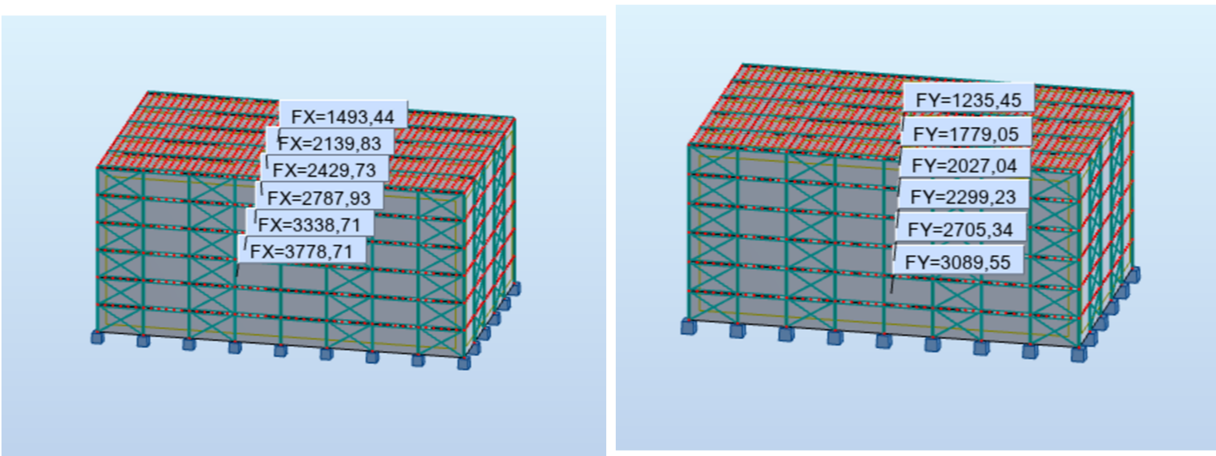


Figur 2: Deformation for moment-resisting structure in X and Y-direction

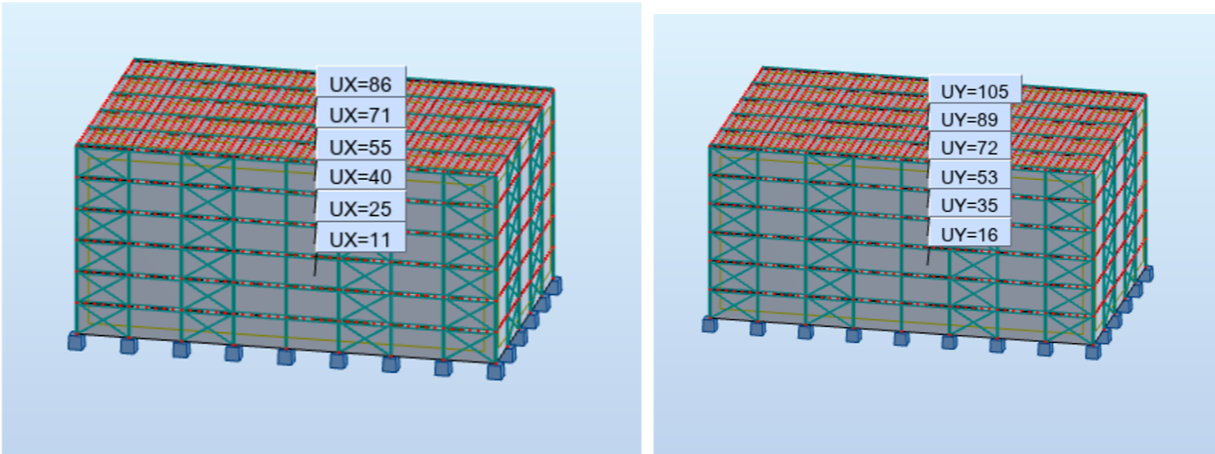


Figur 3: Beam verification for moment-resisting structure

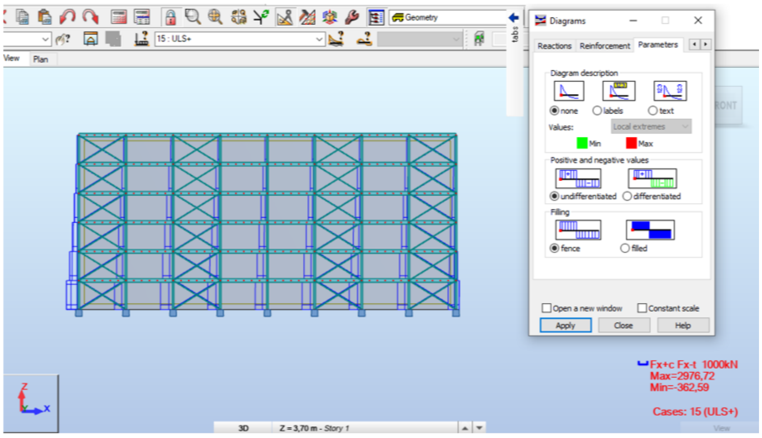
B.2 Cross-bracing frame



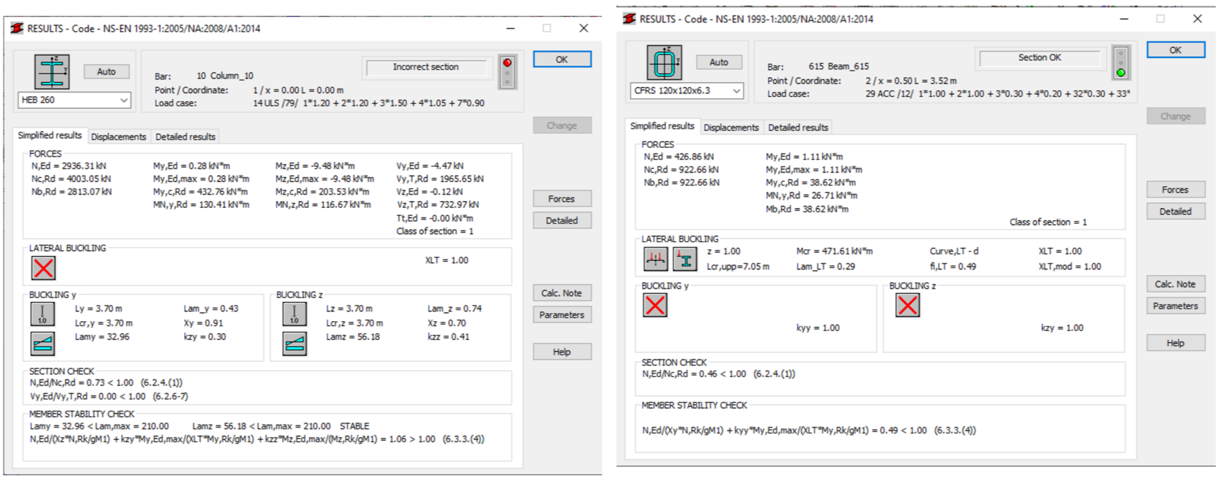
Figur 4: base shear force for cross-bracing structure in X and Y-direction



Figur 5: Deformation for cross-bracing structure in X and Y-direction



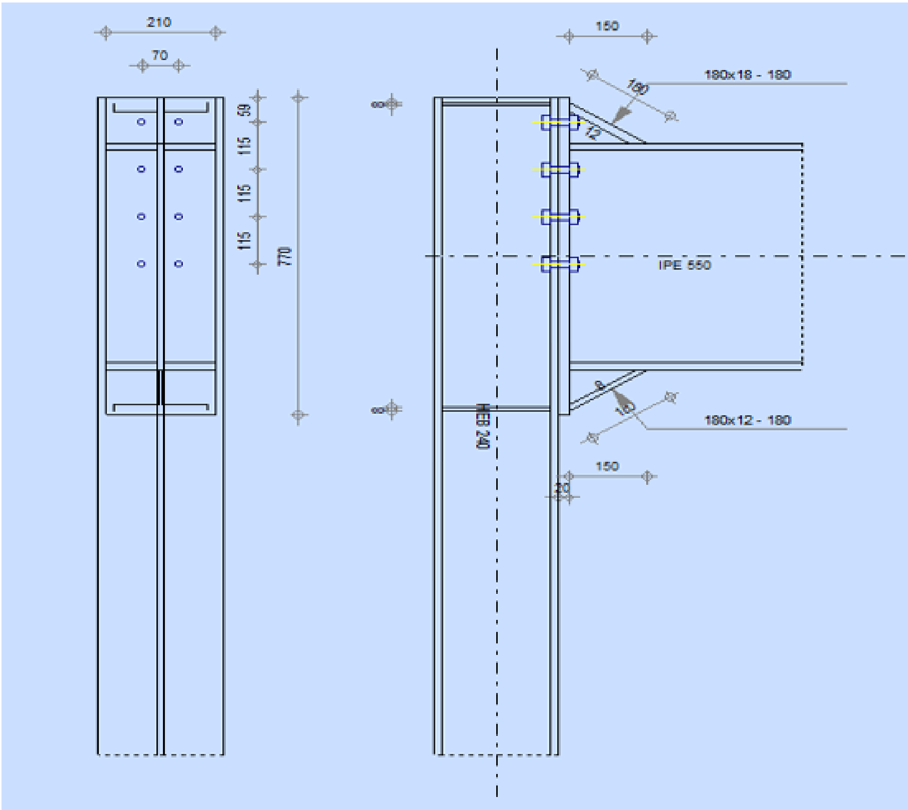
Figur 6: Column verification for cross-bracing structure



Figur 7: Verification of column and bracing for cross-bracing structure in X and Y-direction

C Appendix 3: Design of beam-to-column connection

C.1 Fixed connection for MRF



Figur 8: Fixed connection

COLUMN Section: HEB 240

$\alpha = -90.0$ [Deg] Inclination angle

$h_c = 240[mm]$ Height of column section

$b_{fc} = 240[mm]$ Width of column section

$t_{wc} = 10[mm]$ Thickness of the web of column section

$t_{fc} = 17[mm]$ Thickness of the flange of column section

$r_c = 21[mm]$ Radius of column section fillet

$A_c = 10600[mm2]$ Cross-sectional area of a column

$I_{xc} = 112600000[mm4]$ Moment of inertia of the column section

Material: S355

$f_{yc} = 355.00[MPa]$ Resistance

BEAM Section: IPE 550

$\alpha = 0.0[Deg]$ Inclination angle

$h_b = 550[mm]$ Height of beam section

$b_f = 210[mm]$ Width of beam section

$t_{wb} = 11[mm]$ Thickness of the web of beam section

$t_{fb} = 17[mm]$ Thickness of the flange of beam section

$r_b = 24[mm]$ Radius of beam section fillet

$r_b = 24[mm]$ Radius of beam section fillet

$A_b = 13440[mm2]$ Cross-sectional area of a beam

$I_{xb} = 671200000[mm4]$ Moment of inertia of the beam section

Material: S355

$f_{yb} = 355.00[MPa]$ Resistance

BOLTS

The shear plane passes through the UNTHREADED portion of the bolt.

$d = 16[mm]$ Bolt diameter

Class = 8.8 Bolt class

$F_{tRd} = 90.43[kN]$ Tensile resistance of a bolt

$n_h = 2$ Number of bolt columns

$n_v = 4$ Number of bolt rows

$h_1 = 59[mm]$ Distance between first bolt and upper edge of front plate

Horizontal spacing $e_i = 70[mm]$

Vertical spacing $p_i = 115; 115; 115[mm]$

PLATE

$h_p = 770[mm]$ Plate height

$b_p = 210[mm]$ Plate width

$t_p = 20[mm]$ Plate thickness

Material: S235

$f_{yp} = 235.00[MPa]$ Resistance

UPPER STIFFENER

$w_u = 180[mm]$ Plate width

$t_{fu} = 18[mm]$ Flange thickness

$h_u = 100[mm]$ Plate height

$t_{wu} = 12[mm]$ Web thickness

$l_u = 150[mm]$ Plate length

$\alpha = 33.7$ [Deg] Inclination angle

Material: S235

$f_{ybu} = 235.00[MPa]$ Resistance

LOWER STIFFENER

$w_d = 180[mm]$ Plate width

$t_{fd} = 12[mm]$ Flange thickness

$h_d = 100[mm]$ Plate height

$t_{wd} = 8[mm]$ Web thickness

$l_d = 150[mm]$ Plate length

$\alpha = 33.7[Deg]$ Inclination angle

Material: S235

$f_{ybu} = 235.00[MPa]$ Resistance

COLUMN STIFFENER

Upper:

$h_{su} = 206[mm]$ Stiffener height

$b_{su} = 115[mm]$ Stiffener width

$t_{hu} = 8[mm]$ Stiffener thickness

Material: S235

$f_{ysu} = 235.00[MPa]$ Resistance

Lower:

$h_{sd} = 206[mm]$ Stiffener height

$b_{sd} = 115[mm]$ Stiffener width

$t_{hd} = 8[mm]$ Stiffener thickness

Material: S235

$f_{ysu} = 235.00[MPa]$ Resistance

FILLET WELDS

$a_w = 8[mm]$ Web weld

$a_f = 13[mm]$ Flange weld

$a_s = 8[mm]$ Stiffener weld

$a_{fu} = 5[mm]$ Horizontal weld

$a_{fd} = 5[mm]$ Horizontal weld

MATERIAL FACTORS

$\gamma_{M0} = 1.00$ Partial safety factor

$\gamma_{M1} = 1.00$ Partial safety factor

$\gamma_{M2} = 1.25$ Partial safety factor

$\gamma_{M3} = 1.25$ Partial safety factor

LOADS

Ultimate limit state

$Mb1, Ed = 111.68[kN \times m]$ Bending moment in the right beam

$V_{b1,Ed} = 102.97[kN]$ Shear force in the right beam

$N_{b1,Ed} = -4.82[kN]$ Axial force in the right beam

$M_{b2,Ed} = 58.16[kN \times m]$ Bending moment in the left beam

$V_{b2,Ed} = 72.86[kN]$ Shear force in the left beam

$N_{b2,Ed} = -3.45[kN]$ Axial force in the left beam

$M_{c1,Ed} = 30.69[kN \times m]$ Bending moment in the lower column

$V_{c1,Ed} = 16.96[kN]$ Shear force in the lower column

$N_{c1,Ed} = -1146.67[kN]$ Axial force in the lower column

$M_{c2,Ed} = -25.80[kN \times m]$ Bending moment in the upper column

$V_{c2,Ed} = -13.74[kN]$ Shear force in the upper column

$N_{c2,Ed} = -947.16[kN]$ Axial force in the upper column

RESULTS

BEAM RESISTANCES

COMPRESSION

$A_b = 13440[mm^2]$ Area EN1993-1-1:[6.2.4]

$N_{cb,Rd} = A_b f_{yb}/\gamma_{M0}$

$N_{cb,Rd} = 4771.20[kN]$ Design compressive resistance of the section EN1993-1-1:[6.2.4]

SHEAR

$A_{vb} = 9233[mm^2]$ Shear area EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb}(f_{yb}/\sqrt{3})/\gamma_{M0}$

$V_{cb,Rd} = 1892.29[kN]$ Design sectional resistance for shear EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed}/V_{cb,Rd} \leq 1.0 = 0.05 < 1.00$ verified

COLUMN RESISTANCES

WEB PANEL - SHEAR

$_{b1,Ed} = 111.68[kN \times m]$ Bending moment (right beam) [5.3.(3)]

$M_{b2,Ed} = 58.16[kN \times m]$ Bending moment (left beam) [5.3.(3)]

$V_{c1,Ed} = 16.96[kN]$ Shear force (lower column) [5.3.(3)]

$V_{c2,Ed} = -13.74[kN]$ Shear force (upper column) [5.3.(3)]

$z = 636[mm]$ Lever arm [6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed})/z - (V_{c1,Ed} - V_{c2,Ed})/2$

$V_{wp,Ed} = 68.79[kN]$ Shear force acting on the web panel [5.3.(3)]

$A_{vs} = 3324[mm^2]$ Shear area of the column web EN1993-1-1:[6.2.6.(3)]

$A_{vc} = 3324[mm^2]$ Shear area EN1993-1-1:[6.2.6.(3)]

$d_s = 742[mm]$ Distance between the centroids of stiffeners [6.2.6.1.(4)]

$M_{pl,fc,Rd} = 6.16[kN \times m]$ Plastic resistance of the column flange for bending [6.2.6.1.(4)]

$M_{pl,stu,Rd} = 0.90[kN \times m]$ Plastic resistance of the upper transverse stiffener for bending [6.2.6.1.(4)]

$M_{pl,stl,Rd} = 0.90[kN \times m]$ Plastic resistance of the lower transverse stiffener for bending [6.2.6.1.(4)]

$V_{wp,Rd} = 0.9(A_{vs} \times f_{y,wc})/(\sqrt{3}\gamma_{M0}) + Min(4M_{pl,fc,Rd}/d_s, (2M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd})/d_s)$

$V_{wp,Rd} = 632.18[kN]$ Resistance of the column web panel for shear [6.2.6.1]

$V_{wp,Ed}/V_{wp,Rd} \leq 1.0 = 0.11 < 1.00$ verified

CONNECTION RESISTANCE FOR COMPRESSION

$N_{j,Rd} = Min(N_{cb,Rd} 2F_{c,wb,Rd,low}, 2F_{c,wb,Rd,upp}, 2F_{c,wc,Rd,low}, 2F_{c,wc,Rd,upp})$

$N_{j,Rd} = 2150.70[kN]$ Connection resistance for compression [6.2]

$N_{b1,Ed} = 0$

$N_{b1,Ed}/N_{j,Rd} \leq 1.0 = 0.00 < 1.00$ verified

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$F_{tj,Rd} = [kN]$ Reduced bolt row resistance

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd} = 300.47[kN \times m]$ Connection resistance for bending [6.2]

$M_{b1,Ed}/Mj,Rd \leq 1.0 = 0.37 < 1.00$ verified

CONNECTION RESISTANCE FOR SHEAR

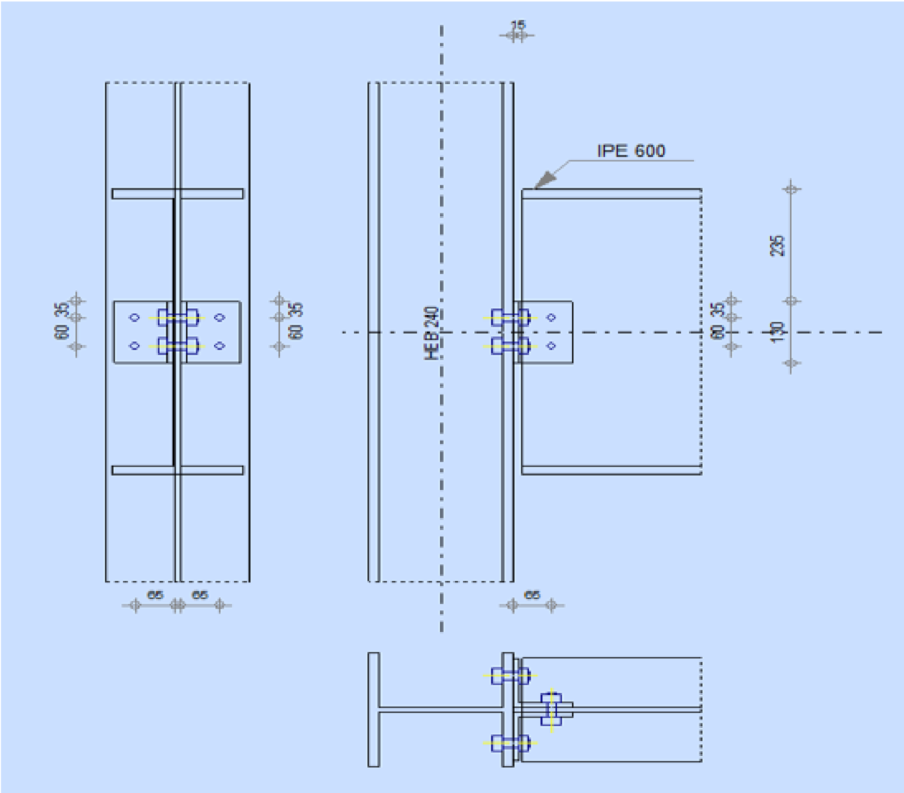
$F_{vj,Rd} = [kN]$ Reduced bolt row resistance

$V_{j,Rd} = n_h \sum F_{vj,Rd}$

$V_{j,Rd} = 481.08[kN]$ Connection resistance for shear EN:1993-1-1[Table 3.4]

$V_{b1,Ed}/V_{j,Rd} \leq 1.0 = 0.21 < 1.00$ verified

C.2 Pinned connection for cross-braced frame



Figur 9: Pinned connection

COLUMN

Section: HEB 240

Bar no.: 317

$\alpha = -90.0$ [Deg] Inclination angle

$h_c = 240[mm]$ Height of column section

$b_{fc} = 240[mm]$ Width of column section

$t_{wc} = 10[mm]$ Thickness of the web of column section

$t_{fc} = 17[mm]$ Thickness of the flange of column section

$r_c = 21[mm]$ Radius of column section fillet

$A_c = 10600[mm^2]$ Cross-sectional area of a column

$I_{yc} = 112600000[mm^4]$ Moment of inertia of the column section

Material: S355

$f_{yc} = 355.00[MPa]$ Design resistance

$f_{uc} = 490.00[MPa]$ Tensile resistance

BEAM

Section: IPE 600

Bar no.: 615

$\alpha = 0.0[Deg]$ Inclination angle

$h_b = 600[mm]$ Height of beam section

$b_b = 220[mm]$ Width of beam section

$t_{wb} = 12[mm]$ Thickness of the web of beam section

$t_{fb} = 19[mm]$ Thickness of the flange of beam section

$r_b = 24[mm]$ Radius of beam section fillet

$A_b = 15600[mm^2]$ Cross-sectional area of a beam

$I_{yb} = 920800000[mm^4]$ Moment of inertia of the beam section

Material: S355

$f_{yb} = 355.00[MPa]$ Design resistance

$f_{ub} = 490.00[MPa]$ Tensile resistance

ANGLE

Section: VL 100x100x10

$h_k = 100[mm]$ Height of angle section

$b_k = 100[mm]$ Width of angle section

$t_{fk} = 10[mm]$ Thickness of the flange of angle section

$r_k = 12[mm]$ Fillet radius of the web of angle section

$l_k = 130[mm]$ Angle length

Material: S355

$f_{yk} = 355.00[MPa]$ Design resistance

$f_{uk} = 490.00[MPa]$ Tensile resistance

BOLTS

BOLTS CONNECTING COLUMN WITH ANGLE

The shear plane passes through the UNTHREADED portion of the bolt.

$Class = 8.8$ Bolt class

$d = 16[mm]$ Bolt diameter

$d_0 = 18[mm]$ Bolt opening diameter

$A_s = 157[mm^2]$ Effective section area of a bolt

$A_v = 201[mm^2]$ Area of bolt section

$f_{ub} = 600.00[MPa]$ Tensile resistance

$k = 1$ Number of bolt columns

$w = 2$ Number of bolt rows

$e_1 = 35[mm]$ Level of first bolt

$p_1 = 60[mm]$ Vertical spacing

BOLTS CONNECTING ANGLE WITH BEAM

The shear plane passes through the UNTHREADED portion of the bolt.

$Class = 6.8$ Bolt class

$d = 16[mm]$ Bolt diameter

$d_0 = 18[mm]$ Bolt opening diameter

$A_s = 157[mm^2]$ Effective section area of a bolt

$A_v = 201[mm^2]$ Area of bolt section

$f_{ub} = 400.00[MPa]$ Tensile resistance

$k = 1$ Number of bolt columns

$w = 2$ Number of bolt rows

$e_1 = 35[mm]$ Level of first bolt

$p_1 = 60[mm]$ Vertical spacing

MATERIAL FACTORS

$\gamma_{M0} = 1.00$ Partial safety factor [2.2]

$\gamma_{M2} = 1.25$ Partial safety factor [2.2]

LOADS

Case: ULS (1+2) $\times 1.35 + (3 + 4 + 5 + 6 + 7 + 8 + 9 + 10 + 11 + 12) \times 1.50$

$N_{b,Ed} = 32.25[kN]$ Axial force

$V_{b,Ed} = -42.69[kN]$ Shear force

$M_{b,Ed} = 0.00[kN * m]$ Bending moment

RESULTS

FORCES ACTING ON BOLTS IN THE COLUMN - ANGLE CONNECTION

Bolt shear

$e = 71[mm]$ Distance between centroid of a bolt group of an angle and center of the beam web

$M_0 = 0.5 \times V_{b,Ed} \times e$

$M_0 = 1.52[kN \times m]$ Real bending moment

$F_{Vz} = 0.5 \times |Vb, Ed|/n$

$F_{Vz} = 10.67[kN]$ Component force in a bolt due to influence of the shear force

$F_{Mx} = 25.26[kN]$ Component force in a bolt due to influence of the moment

$F_{x,Ed} = F_{Nx} + F_{Mx}$

$F_{x,Ed} = 25.26[kN]$ Design total force in a bolt on the direction x

$F_{z,Ed} = F_{Vz} + F_{Mz}$

$F_{z,Ed} = 10.67[kN]$ Design total force in a bolt on the direction z

$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$

$F_{Ed} = 27.42[kN]$ Resultant shear force in a bolt

$F_{Rdx} = min(F_{bRd1x}, F_{bRd2x})$

$F_{Rdx} = 101.63[kN]$ Effective design capacity of a bolt on the direction x

$F_Rdz = \mathrm{min(F}_{bRd1z}, F_{bRd2z})$

$F_{Rdz} = 101.63[kN]$ Effective design capacity of a bolt on the direction z

$|F_{x,Ed}| \leq F_{Rdx}$, $|25.26| < 57.91$ verified

$|F_{z,Ed}| \leq F_{Rdz}$, $|10.67| < 57.91$ verified

$F_{Ed} \leq F_{v,Rd}$, $27.42 < 57.91$ verified

FORCES ACTING ON BOLTS IN THE ANGLE - BEAM CONNECTION

Bolt shear

$e = 74[mm]$ Distance between centroid of a bolt group and center of column flange

$M_0 = M_{b,Ed} + V_{b,Ed} \times e$

$M_0 = -3.14[kN \times m]$ Real bending moment

$F_{Nx} = |N_{b,Ed}|/n$

$F_{Nx} = 16.12[kN]$ Component force in a bolt due to influence of the longitudinal force

$F_{Vz} = |V_{b,Ed}|/n$

$F_{Vz} = 21.34[kN]$ Component force in a bolt due to influence of the shear force

$F_{Mx} = 52.30[kN]$ Component force in a bolt due to influence of the moment on the x direction

$F_{Mz} = 0.00[kN]$ Component force in a bolt due to influence of the moment on the z direction

$F_{x,Ed} = F_{Nx} + F_{Mx}$

$F_{x,Ed} = 68.42[kN]$ Design total force in a bolt on the direction x

$F_{z,Ed} = F_{Vz} + F_{Mz}$

$F_{z,Ed} = 21.34[kN]$ Design total force in a bolt on the direction z

$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$

$F_{Ed} = 71.67[kN]$ Resultant shear force in a bolt $F_{Rdx} = min(F_{bRd1x}, F_{bRd2x})$

$F_{Rdx} = 153.60[kN]$ Effective design capacity of a bolt on the direction x

$F_{Rdz} = min(F_{bRd1z}, F_{bRd2z})$

$F_{Rdz} = 153.60[kN]$ Effective design capacity of a bolt on the direction z

$|F_{x,Ed}| \leq F_{Rdx}$, $|68.42| < 153.60$ verified

$|F_{z,Ed}| \leq F_{Rdz}$, $|21.34| < 153.60$ verified

$F_{Ed} \leq F_{v,Rd}$, $71.67 < 77.21$ verified (0.93)

D Appendix 4: Reference of plan of procedure

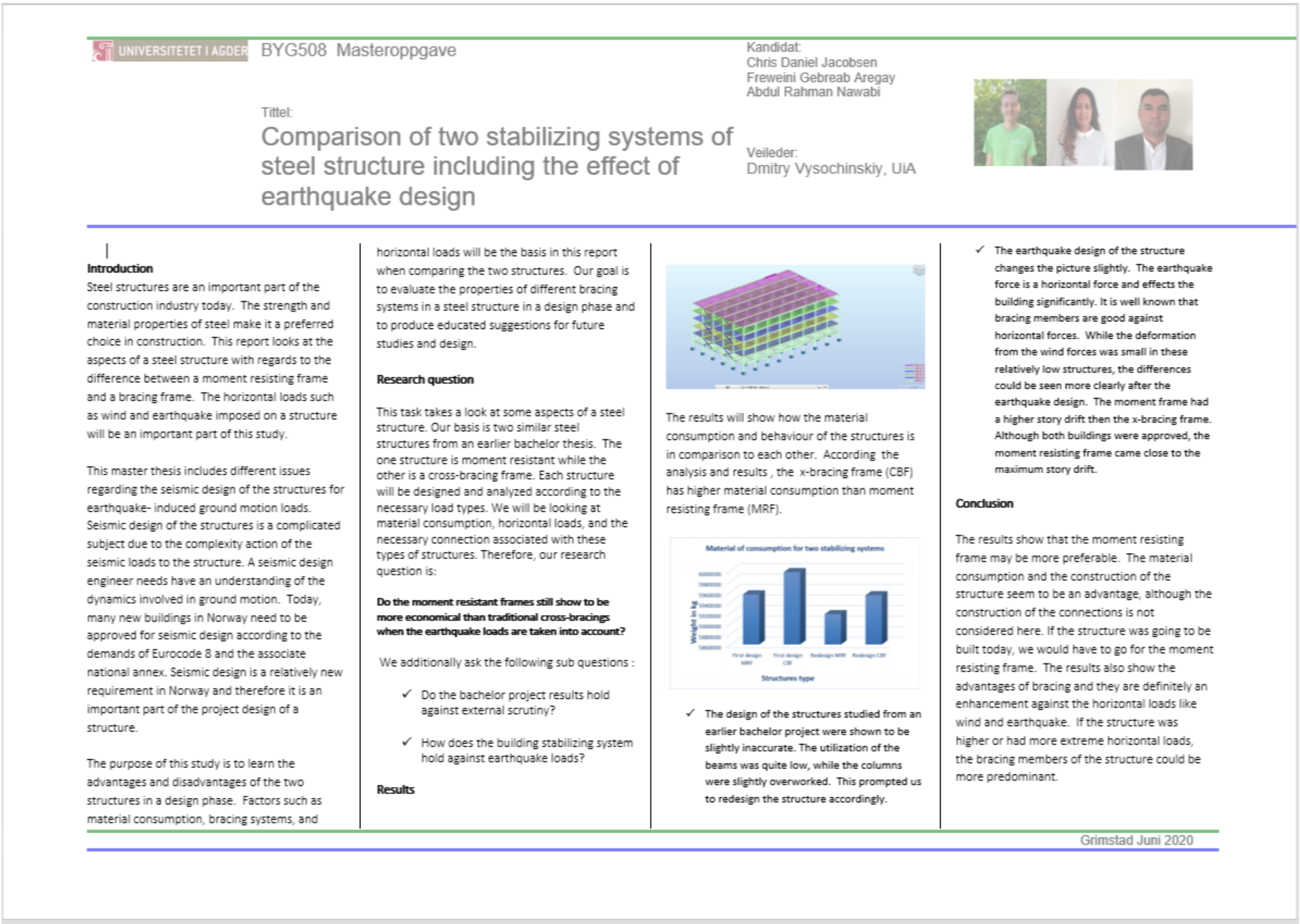
Task Name	Duration	Start	Finish	Predeces	Gantt
Gjennomgang av dynamik og utarbeiding av prosjekt	5 days	Mon 06.01.20	Fri 10.01.20		Abdul;Chris;Freweini
Veiledning	**106 days**	**Thu 09.01.20**	**Thu 04.06.20**		
Literature Review and Research Question	6 days	Mon 13.01.20	Mon 20.01.20		Abdul;Chris;Freweini
Case- defining the case	4 days	Thu 16.01.20	Tue 21.01.20		Freweini;Abdul;Chris
Ductility and stiffness review-reading articles and Dynamics of structures	10 days	Mon 20.01.20	Fri 31.01.20		Abdul;Chris;Freweini
Moment stiff frame and X Bracing-reading and writing	16 days	Tue 21.01.20	Tue 11.02.20		Freweini
Ductility and stiffness- reading and writing theory	12 days	Thu 23.01.20	Fri 07.02.20		Freweini

Task Name	Duration	Start	Finish	Predeces	Gantt
Case- writing	3 days	Fri 24.01.20	Tue 28.01.20		Chris
Ductility and stiffness review-reading articles and Dynamics of structures-writing	7 days	Thu 30.01.20	Fri 07.02.20		Abdul;Freweini
Modellering og case	19 days	Tue 04.02.20	Fri 28.02.20		Chris;Abdul
Programs- what software to use, model method, learning software, eventually FEM design	11 days	Tue 04.02.20	Tue 18.02.20		Chris;Freweini
Method- defining the methods used in this project	14 days	Tue 04.02.20	Fri 21.02.20		Freweini;Abdul;Chris
FEM design theory	5 days	Fri 07.02.20	Thu 13.02.20		Chris
Eurocode 8- Reading and writing theory	5 days	Thu 13.02.20	Wed 19.02.20		Freweini
Eurocode 3-Reading	3 days	Thu 13.02.20	Mon 17.02.20		Abdul
Eurocode 3-Writing	3 days	Tue 18.02.20	Thu 20.02.20		Abdul
Steel structure-reading and writing	5 days	Thu 20.02.20	Wed 26.02.20		Abdul;Freweini
Connections	5 days	Thu 20.02.20	Wed 26.02.20		Freweini;Chris

Task Name	Duration	Start	Finish	Predeces	Gantt
Connections	5 days	Thu 20.02.20	Wed 26.02.20		Freweini;Chris
Load and load combinations-theory and case	3 days	Mon 02.03.20	Wed 04.03.20		Chris;Abdul
Method-writing	4 days	Thu 05.03.20	Tue 10.03.20		Chris;Freweini
Results from analysis- several attempts	15 days	Thu 05.03.20	Wed 25.03.20		Chris
Social Perspective	6 days	Mon 16.03.2	Mon 23.03.20		Freweini
Results-hand calculations	3 days	Thu 19.03.20	Mon 23.03.20		Freweini;Chris
Samle og skrive resultat	11 days	Thu 02.04.20	Thu 16.04.20		Chris;Freweini;Abdul
Redigering første gang hele rapporten	7 days	Fri 17.04.20	Mon 27.04.20		Abdul;Freweini
Skrive Discussion og analyse	9 days	Fri 17.04.20	Wed 29.04.20		Chris
Skrive konklusjon	2 days	Thu 30.04.20	Fri 01.05.20		Chris
Redigering andre gang hele rapporten	17 days	Fri 01.05.20	Mon 25.05.20		Abdul;Chris;Freweini
Introduction and cover page	3 days	Wed 20.05.20	Fri 22.05.20		Chris

Task Name	Duration	Start	Finish	Predeces	Gantt
Results-hand calculations	3 days	Thu 19.03.20	Mon 23.03.20		Freweini;Chris
Samle og skrive resultat	11 days	Thu 02.04.20	Thu 16.04.20		Chris;Freweini;Abdul
Redigering første gang hele rapporten	7 days	Fri 17.04.20	Mon 27.04.20		Abdul;Freweini
Skrive Discussion og analyse	9 days	Fri 17.04.20	Wed 29.04.20		Chris
Skrive konklusjon	2 days	Thu 30.04.20	Fri 01.05.20		Chris
Redigering andre gang hele rapporten	17 days	Fri 01.05.20	Mon 25.05.20		Abdul;Chris;Freweini
Introduction and cover page	3 days	Wed 20.05.20	Fri 22.05.20		Chris
Innlevering prosjekt	0 days	Fri 05.06.2	Fri 05.06.20		05.06
~~lage powerpoint for presentasjon~~	~~4 days~~	~~Tue 26.05.20~~	~~Fri 29.05.20~~		Chris;Abdul;Freweini
~~øve på presentasjon~~	~~8 days~~	~~Mon 01.06.2~~	~~Wed 10.06.20~~		Freweini;Abdul;Chris
~~Presentasjon~~	~~1 day~~	~~Fri 12.06.20~~	~~Fri 12.06.20~~		Chris;Abdul;Freweini

E Appendix 5: A3 poster



UNIVERSITETET I AGDER BYG508 Masteroppgave

Tittel:

Comparison of two stabilizing systems of steel structure including the effect of earthquake design

Kandidat:
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Abdul Rahman Nawabi

Veileder:
Dmitry Vysochinskiy, UiA

Introduction

Steel structures are an important part of the construction industry today. The strength and material properties of steel make it a preferred choice in construction. This report looks at the aspects of a steel structure with regards to the difference between a moment resisting frame and a bracing frame. The horizontal loads such as wind and earthquake imposed on a structure will be an important part of this study.

This master thesis includes different issues regarding the seismic design of the structures for earthquake- induced ground motion loads. Seismic design of the structures is a complicated subject due to the complexity action of the seismic loads to the structure. A seismic design engineer needs have an understanding of the dynamics involved in ground motion. Today, many new buildings in Norway need to be approved for seismic design according to the demands of Eurocode 8 and the associate national annex. Seismic design is a relatively new requirement in Norway and therefore it is an important part of the project design of a structure.

The purpose of this study is to learn the advantages and disadvantages of the two structures in a design phase. Factors such as material consumption, bracing systems, and horizontal loads will be the basis in this report when comparing the two structures. Our goal is to evaluate the properties of different bracing systems in a steel structure in a design phase and to produce educated suggestions for future studies and design.

Research question

This task takes a look at some aspects of a steel structure. Our basis is two similar steel structures from an earlier bachelor thesis. The one structure is moment resistant while the other is a cross-bracing frame. Each structure will be designed and analyzed according to the necessary load types. We will be looking at material consumption, horizontal loads, and the necessary connection associated with these types of structures. Therefore, our research question is:

Do the moment resistant frames still show to be more economical than traditional cross-bracings when the earthquake loads are taken into account?

We additionally ask the following sub questions :

- ✓ Do the bachelor project results hold against external scrutiny?
- ✓ How does the building stabilizing system hold against earthquake loads?

Results

The results will show how the material consumption and behaviour of the structures is in comparison to each other. According the analysis and results , the x-bracing frame (CBF) has higher material consumption than moment resisting frame (MRF).

Material of consumption for two stabilizing systems

Weight in kg

First design MRF | First design CBF | Redesign MRF | Redesign CBF

Structures type

- ✓ The design of the structures studied from an earlier bachelor project were shown to be slightly inaccurate. The utilization of the beams was quite low, while the columns were slightly overworked. This prompted us to redesign the structure accordingly.
- ✓ The earthquake design of the structure changes the picture slightly. The earthquake force is a horizontal force and effects the building significantly. It is well known that bracing members are good against horizontal forces. While the deformation from the wind forces was small in these relatively low structures, the differences could be seen more clearly after the earthquake design. The moment frame had a higher story drift then the x-bracing frame. Although both buildings were approved, the moment resisting frame came close to the maximum story drift.

Conclusion

The results show that the moment resisting frame may be more preferable. The material consumption and the construction of the structure seem to be an advantage, although the construction of the connections is not considered here. If the structure was going to be built today, we would have to go for the moment resisting frame. The results also show the advantages of bracing and they are definitely an enhancement against the horizontal loads like wind and earthquake. If the structure was higher or had more extreme horizontal loads, the bracing members of the structure could be more predominant.

Grimstad Juni 2020

F Appendix 6: Minutes of meeting with supervisor

In this master thesis, we have had meeting with supervisor once a week for discussing of the main point of the task until a mid of march. However, due to Covid-19 situation, our meeting have undergone through Teams (video meeting) once a week. The points that we have discussed are summarized below:

Meeting dated: 09-01-20

Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Meeting location: UiA
Discussed the following points:

- How to dimension the earthquake?
- Ductility and elasticity demand
- How can we ensure that the building has sufficient ductility?
- Exclusion criteria
- Research question
- Why earthquake is relevant to Norway?
- Comparison of ductility classes of low and middle in steel building.
- To coordinate of agreement next plan decision on master thesis.

Meeting dated: 16-01-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Meeting location: UiA
Discussed the following points:

- Future plan
- Research question
- Literature review
- Selecting of soft wear
- What are differences between ductility low and meddle classes?
- Symmetric and nonsymmetric in-plan buildings
- Comparison of cross bracing and moment resistance steel structures

- Find a bachelor thesis

Meeting dated: 23-01-20

Participants: Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Meeting location: UiA
Discussed the following points:

- Some proposals for research question
- What affect does ductility have under earthquake analysis when we compare cross bracing and moment resistance frame structures?
- Behaviour of cross bracing system and cross bracing system applied seismic loads
- Comparison of two bracing systems under seismic design
- Verification of structural elements (beams and columns)
- Designing of joints based on EC 8
- Different modelling approaches

Meeting dated: 30-01-20
Participants:
Freweini Gegbreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Meeting location: UiA
Discussed the following points:

- Research question should set at the top
- Make a plane in Excel
- Joint rotation capacity
- Find out books regarding to the joint designing
- To select the research question of our master project
- There is a misconception that moment resistance frame buildings are not relevant in Norway due to excessive material consumption
- Will material consumption of two different moment resistance and cross bracing systems change in accordance to the earthquake design?

Meeting dated: 07-02-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen

Abdul Rahman Nawabi

Meeting location: UiA Discussed the following points:

- Decision research question and future plan (action plan)
- Got a bachelor thesis as a case study
- Comparison of two stabilizing systems, moment resistance and cross bracing
- Check and verify the bachelor thesis
- How to arrange correctly the chapters of our master thesis
- Apply wind and seismic load and.
- Check if the material consumption of these two systems differs.
- Design should be based on EC 8
- Review EC 3, classification of joints and connections
- Role of hallow core slabs in seismic design
- Theory should be based on the dynamic of structures, EC 0,1,3 and 8

Meeting dated: 27-02-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Meeting location: UiA
Discussed the following points:

- Wind simulation, wind distribution from different side on the building
- Different type of joints rigid, semi-rigid and pinned
- Globally analysis of two bracing systems
- Make a model of the structures using MOLA educational model

Meeting dated: 19-03-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- How to put data in LaTeX overleave
- Hallow-core slab's global analyse

- Report of new analysis of the project what we have done till now If there are different values and data getting after analysing of bachelor project, than the new findings should be registered and recorded
- Differences between bolted and welded connection

Meeting dated: 27-03-20

Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- Results of two bracing systems
- If there is lower utilization of structural elements, then find out why?
- Implement earthquake analysis and verify bachelor thesis findings
- Compare material consumption of two bracing, moment resistance and cross bracing systems.
- Will re- designing of the project have impacts on material consumption of the structures?
- Delivery of the fist gotten results till next meeting

Meeting dated: 02-04-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- We have the primary analysis, some constructional not approved, it means that the result gotten by earlier bachelor students should corrected in the next redesigning step
- Some columns are overestimated of previous students
- Base share force is more high
- Some obtained results are different between two groups who performed analysis, it can be due to the differences of RSA software version
- Effect of regularity in elevation EC 8
- Results shows that deformation in cross bracing system is less than to moment resistance frame
- Use of MOLA model in order to have better understanding of different stabilizing systems

Meeting dated: 08-04-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi
Video meeting via Teams
Discussed the following points:

- Discussion about obtained results
- 28 columns of the first floor are not verified
- Re-design of the project to get more accurate results
- Have make a contact with Paul in order to get access to Norwegian Steel Association catalogue for column and beam size and weight proportion.
- There is high utilization in columns
- Our results show that previous bachelor students are overdesigned the structures, so it is necessary to re- design the project.
- Changing stiffness of steel structures component will lead to last distribution in the whole building
- Next meeting will be hold on Thursday 16-04-20

Meeting dated: 16-04-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- Utilization degree of the constructional elements
- Load combination of static and dynamic loads
- The primary structure is not verified
- To ensure over the quality of the designed structures, designing performed in two computers in order to get more exact result for our project
- Static determined and static undetermined systems
- Joint's eccentricity

Meeting dated: 23-04-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- Changing cross sectional areas of one the structural element leads to the force impact is changes so, some part of the structure cannot be verified
- If we use pined connection between beam and columns, then the utilization of columns will increase
- Static determined and static undetermined system, example truss
- In the static determined system if we change load magnitude one part of structure's element, then we have not to re- calculate whole the structure. But in static the undetermined we have to design for the whole structure.

Meeting dated: 30-04-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- delivery of the draft of project to supervisor
- Beams on the top floor don't verified
- Change the size of I steel sections
- The top of the hollow core covers by isolation and mortar
- Read about floor types
- Eccentricity in the joints

Meeting dated: 14-05-20
Participants:
Freweini Gebreab Aregay
Chris Daniel Jacobsen
Abdul Rahman Nawabi

Video meeting via Teams
Discussed the following points:

- Discussion of the results and first draft of the project
- Editing of the project
- Conclusion should be corrected
- Coordination between the case and conclusion
- Forced damped vibration

- Advantages and disadvantages of moment resistance and cross bracing systems