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# **Seismic design in Europe and the Russian Federation**

Steel and reinforced concrete buildings of moderate importance



Bachelor's thesis

Construction engineering

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**ABSTRACT**

This Bachelor's thesis is dedicated to the seismic design in Europe and Russian Federation. The main goal was to provide the information needed for an English-speaking specialist to perform seismic design in the two regions above. The thesis was commissioned by Pöyry Oy.

The thesis is based on the latest normative standards valid in the two regions: Eurocode 8 and SP 14.13330.2014. The background information needed to write the thesis is structural engineering knowledge learned at HAMK UAS and the help of the specialists at Pöyry Oy combined with various books and internet articles. In the appendix, a practical example of seismic design of a steel building is provided.

The outcome is a comparison of both standards. The Russian norms demonstrated to be much less detailed, especially in the phase of detailing after the structural analysis. Therefore, a specialist must be experienced enough and have sufficient background so as to design earthquake resistant buildings in Russia.

**Keywords** Seismic design, Eurocode 8, Russia, Earthquakes.

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## FOREWORD

This Bachelor's thesis "Seismic design in Europe and the Russian Federation" was written for the purpose of further development of structural design for European companies in the Russian Federation.

The idea to make a research project for the topic mentioned was offered by Pöyry Oy. So, I would like to thank this organisation for the financial support. Also special thanking words must be told to Pekka Narinen, without his enthusiasm the work presented could not be written.

In addition, it is vital to mention the essential help of my company supervisor Pekka Marjamäki, an experienced specialist in structural engineering and especially seismic design. His guidance helped significantly in applying the theoretical knowledge from books to practical structural design, as well as in the searching for the suitable sources of information.

After that, I would like to thank all the staff of HAMK University of Applied Sciences for the knowledge and advice that I was taught in the degree programme in Construction Engineering. Besides, I want to express my gratitude to Jarmo Havula, who was the main lecturer in structural engineering and spent a lot of extra time guiding me on the basics of Finite Element Software.

Besides that, I want to state my gratefulness to my department at Pöyry Oy, especially my section manager Aleksi Pöyhönen, who also contributed to this Bachelor's thesis by helping with Russian engineering standards. The friendly environment and advice from my colleagues in addition to the gained professional experience made the process of writing this thesis much more efficient.

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Appendix 1 Seismic design of a steel structure

## 1 INTRODUCTION

### 1.1 Topic definition

The purpose of this Bachelor's thesis in engineering is to examine the design of structures made of steel or reinforced concrete in the areas with a high probability of earthquake occurrence. The regions of interest are mainly Europe and the Russian Federation. Therefore, buildings must correspond to the specific sections of national building codes that the thesis is based on. The codes stated above are SP 14.13330.2014 for Russia and Eurocode 1998 for the EU.

### 1.2 Scope

At first, it is necessary to specify the main subject of the chapters below. This Bachelor's thesis explains the structural design procedure of earthquake-resistant buildings in Europe and the Russian Federation. The goal principally distinguishes this project from most of the books and Internet articles: besides rewriting the regulations and additional background, a detailed comparison between the two standards will be provided. In other words, after any criteria of Eurocodes are stated, its equivalent from the Russian standard is listed with the differences and similarities underlined. As an outcome, there is a possibility for international companies and specialists to be able to perform seismic design in Russia. The reason is that a simple translation of SP is not enough, as far as all the regulations need to be explained and probably analyzed in order to ensure the absence of principal mistakes. In addition, a special attention is paid to the clauses that may be understood in two ways to avoid fundamental misunderstanding. That is why this thesis may be time-worthy not only for those who want to get acquainted with the basics of seismic design, but also for a specialist that is highly experienced in earthquake engineering and wants to do business in Russia.

### 1.3 Overview of contents

The content presented allows one to be able to perform a seismic analysis in the Russian Federation or any part of Europe. This is achieved via a sequent cover of all the relevant stages of an earthquake design. Every stage explanation is based on both standards mentioned above. Therefore, links to corresponding clauses in the building codes are always provided.

However, the purpose was quite far from re-writing the formulas and regulations from the official documents. To avoid extra paper consumption, this thesis must be read strictly together with the building codes specified above. Most of the equations and demands are not written, only links to their location in the standards are given. In this project there is the background that helps one to understand their meaning. Nevertheless, it may

not be the case with the Russian standards because there is no official translation to English. This is why some key expressions and tables will be listed to give an English-speaking reader the required information.

It must be noted that the thesis provides the information only about the buildings of average importance, it is not intended for a specific usage (e.g. nuclear power plants or sea platforms).

As a second part, there is a practical example of a real building that is intended to resist an earthquake. The goal is to provide a working example that appears as a final summarization of all the numerous requirements. The methods used are considered to be most popular and generally adopted worldwide or at least, officially valid in the regions of interest. They include different analysis types with computer software combined with manual calculations when needed. As a result, common questions that a designer might face throughout a work process are basically covered.

### 1.4 Background of the topic

Unfortunately, it is not possible to make a guide for a person unfamiliar with some certain background, because it will require endless resources. In the next sentences there are the essential requirements for a reader.

First of all, seismic design is only one part of a structural design. This is why it is vital that a person is familiar with the way of determining building resistance to loads as well as with the methods of their estimation.

Secondly, the knowledge of Russian and European standards will be a good advantage and allow one to get the maximum benefit out of everything written here.

One more advantage is, of course, an ability to work with finite element programs. Whereas a conventional design may be done using paper and a calculator even nowadays, it is almost impossible for the earthquake case.

It may be necessary to state a vague frame of this study, in order for a reader to navigate easier. Nowadays, seismic design may be separated into the three phases, independently of a building code:

- Region seismicity definition
- Structural analysis
- Modifying a building to correspond to specific material requirements

These will be the stages that determine the sequence of the thesis.

### 1.5 Methods of research

In order to present a sequent and logical explanation, various sources were used, starting from books and Internet articles and ending with videos on seminars. They all are stated in the related part. To ensure better understanding of the theoretical part, there is a practical example in the appendix 1. It shows the seismic design procedure for a steel structure.



## 2 EARTHQUAKES

### 2.1 Earthquake definition and threat assessment

At first, a clear definition of an earthquake must be provided. According to Wikipedia, it means punches and vibrations of the Earth surface, caused by natural (mostly tectonic processes) and artificial (e.g. explosions) reasons. Anyhow, only the description does not represent the impact of this phenomena on human beings.

Fortunately, plenty of books and articles have been written about that. For example, Amita Sinhvai gives a detailed estimation in her book “Understanding earthquake disasters”. Based on the author’s explanations, earthquakes seem to be one of the most devastating kind of disasters nowadays, every year making thousands of people homeless, or even dead. (Amita Sinhvai - “Understanding earthquake disasters”, 2010)

In addition to what is written above, it is necessary to underline that up-to-date urbanisation rate tends to be the highest in the history of mankind. This is why Sinhvai states that seismic threat grows exponentially with global population and new cities’ appearing.

Earthquakes happen much more frequently than one can imagine, but, as stated by Sinhvai, they do not occur uniformly all over the World. To paraphrase, it means that while some countries may suffer from an excessive seismic activity more than 10 times a year, the other have never faced a terrible disaster described in this paragraph. In Figure 1 one can find the data from CATDAT organization about the year 2012 that hopefully gives a clear proof of the idea.

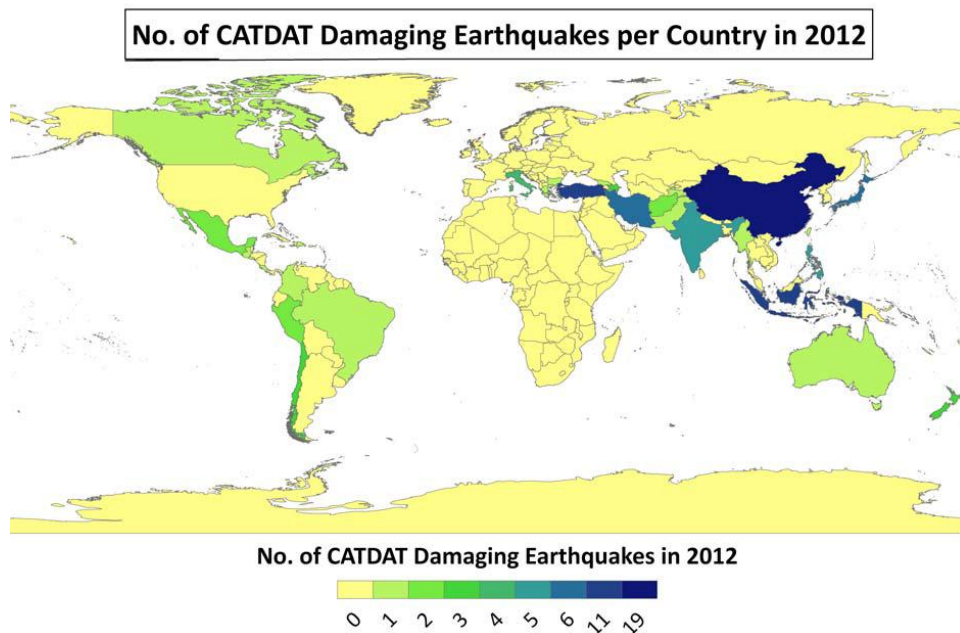


Figure 1 Number of damaging earthquakes per country in 2012 (<http://earthquake-report.com/2013/01/07/damaging-earthquakes-2012-database-report-the-year-in-review/>)

Whereas some regions are considered to be totally non-seismic, it is required to point out that only significantly damaging earthquakes are commonly taken into account. To confirm this, Wikipedia writes in the related article that very often no one even notices a seismic activity, although it still takes place. To visualize this point, one is offered to take a look at Figure 2 below that is related to Finland.

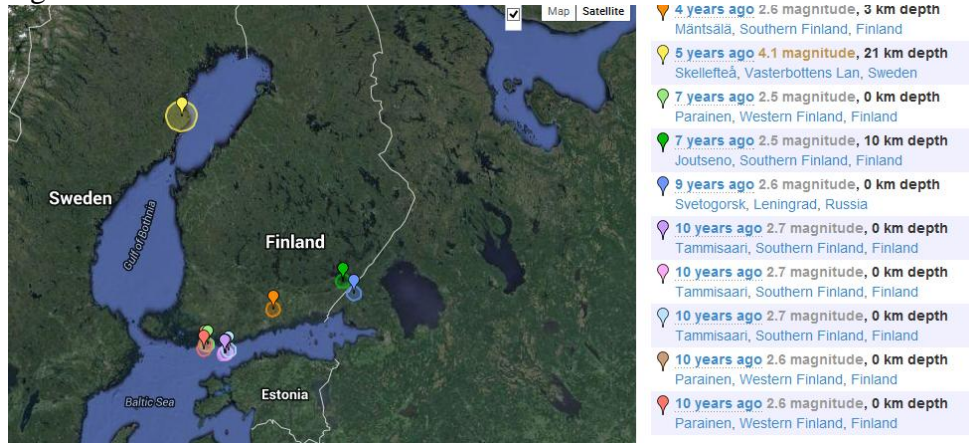


Figure 2 Earthquake monitoring in Finland  
(<http://earthquaketrack.com/p/finland/recent>)

A quick summary of everything above gives a clear reason why structural seismic design is vital in some regions. It is common knowledge that a properly designed building which does not lose the functionality throughout a catastrophe plays an extremely important role not only during the seismic activity itself, but also after it, which provides the possibility to liquidate the consequences easily.

### 2.2 Hazard estimation

In the previous clauses the terms “seismic” and “non-seismic” regions were mentioned plenty of times. It would be reasonable to give an explanation of what is meant by them.

There is no clear and generally adopted definition, because an earthquake hazard may be assessed differently, based on many criteria. The criteria may be, for example, earthquake magnitude or intensity and then a lot of values for their frequency and probability of occurrence on a certain territory. In the limits of this thesis, the following expressions are proposed:

- Seismic region – a territory, where special provisions to a structural design of buildings (seismic design) are required by national authorities due to a high probability of earthquake occurrence.
- Non-seismic region – all the other territories.

Therefore, the only seismicity criteria important for the structural design are the ones defined by a local government. In the chapters below one will find a detailed comparison between seismicity classification systems in Europe and the Russian Federation that are chosen for the design of earthquake resistant buildings.

### 2.2.1 Classification in Europe.

In most of the countries in the European Union, the seismicity of the region is based on the following factors:

- Earthquake effect strength
- Probability of exceedance in a certain period
- Return period

At first, an earthquake effect is discussed. This represents the action on structure produced by the phenomena. It obviously may be measured with many units. As for Europe, a reference ground acceleration in  $\text{m/s}^2$  is chosen for an earthquake strength definition.

Secondly, after the reference acceleration is acquired, it is necessary to figure out how possible it is that a stronger earthquake occurs throughout a specified time. This time is chosen to be either 10 or 50 years. The value to be chosen depends on particular steps of seismic design that are described later. As for the probability itself, it is measured in percents.

Finally, a return period means the average time step of an earthquake with the reference ground acceleration stated above. In other words, how long it takes for the phenomena to happen again.

### 2.2.2 Classification in Russian Federation

In comparison with the European system, the factors for region seismicity estimation in Russia do not change at all. On the contrary, there are some differences in the values and units which stand for them.

The effect strength is measured by earthquake intensity. Wikipedia expresses this term as “the severity of ground shaking on the basis of observed effects in an area of the earthquake occurrence.” To estimate it, a special intensity scale named “Medvedev–Sponheuer–Karnik scale” (also known as MSK-64) is used that one can see in Figure 3. According to it, the intensity is measured in degrees.

<b>I. Not perceptible</b>	Not felt, registered only by <a href="#">seismographs</a> . No effect on objects. No damage to buildings.
<b>II. Hardly perceptible</b>	Felt only by individuals at rest. No effect on objects. No damage to buildings.
<b>III. Weak</b>	Felt indoors by a few. Hanging objects swing slightly. No damage to buildings.
<b>IV. Largely observed</b>	Felt indoors by many and felt outdoors only by very few. A few people are awakened. Moderate vibration. Observers feel a slight trembling or swaying of the building, room, bed, chair etc. China, glasses, windows and doors rattle. Hanging objects swing. Light furniture shakes visibly in a few cases. No damage to buildings.
<b>V. Fairly strong</b>	Felt indoors by most, outdoors by few. A few people are frightened and run outdoors. Many sleeping people awake. Observers feel a strong shaking or rocking of the whole building, room or furniture. Hanging objects swing considerably. China and glasses clatter together. Doors and windows swing open or shut. In a few cases window panes break. Liquids oscillate and may spill from fully filled containers. Animals indoors may become uneasy. Slight damage to a few poorly constructed buildings.
<b>VI. Strong</b>	Felt by most indoors and by many outdoors. A few persons lose their balance. Many people are frightened and run outdoors. Small objects may fall and furniture may be shifted. Dishes and glassware may break. Farm animals may be frightened. Visible damage to masonry structures, cracks in plaster. Isolated cracks on the ground.
<b>VII. Very strong</b>	Most people are frightened and try to run outdoors. Furniture is shifted and may be overturned. Objects fall from shelves. Water splashes from containers. Serious damage to older buildings, masonry chimneys collapse. Small landslides.
<b>VIII. Damaging</b>	Many people find it difficult to stand, even outdoors. Furniture may be overturned. Waves may be seen on very soft ground. Older structures partially collapse or sustain considerable damage. Large cracks and fissures opening up, rockfalls.
<b>IX. Destructive</b>	General panic. People may be forcibly thrown to the ground. Waves are seen on soft ground. Substandard structures collapse. Substantial damage to well-constructed structures. Underground pipelines ruptured. Ground fracturing, widespread landslides.
<b>X. Devastating</b>	Masonry buildings destroyed, infrastructure crippled. Massive landslides. Water bodies may be overtopped, causing flooding of the surrounding areas and formation of new water bodies.
<b>XI. Catastrophic</b>	Most buildings and structures collapse. Widespread ground disturbances, tsunamis.
<b>XII. Very catastrophic</b>	All surface and underground structures completely destroyed. Landscape generally changed, rivers change paths, tsunamis.

Figure 3 MSK-64 intensity scale  
[http://en.wikipedia.org/wiki/Medvedev%E2%80%9364\\_Sponheuer%E2%80%933Karnik\\_scale](http://en.wikipedia.org/wiki/Medvedev%E2%80%9364_Sponheuer%E2%80%933Karnik_scale)

The probability of exceedance and return period depend directly on each other in Russia. The first one is measured strictly in 50 years.

### 2.2.3 Comparison and conclusion.

As can be summarized from everything above, the seismicity of a region in both Europe and Russia is evaluated with an earthquake of a reference strength and its frequency of happening on any territory. In both regions, the probability of exceedance and the return period remain constant all over the territory considered, while according to them the reference strength is measured.

To summarize everything, if the strength value for a specified frequency and possibility is too small, a region is considered to be non-seismic, i.e. no structural seismic design is needed for all the buildings in that territory. As for the limits of seismic strength, they are provided in the standards which are described in the following chapters.

## 3 BUILDING CODES

As it was stated in the very first paragraph, seismic design is governed by the law. A designer is obligated to follow certain standards that are adopted by a local government. Below one will find a short description of the standards in Europe and Russian Federation.

### 3.1 Eurocode 8

In the area of the European Union structural design of buildings must correspond to Eurocodes. Wikipedia defines them as “a set of harmonized technical rules developed by the European Committee for Standardization for the structural design of construction works”. To say it simply – a number of regulations that one has to obey if he wants his building permitted for construction. All the Eurocode system is separated into parts that a designer has to apply for a particular guidance. There is a whole part dedicated to an earthquake design which is EN1998. This partition itself is also divided into several documents that are listed below:

- Part 1: General rules, seismic actions and rules for buildings
- Part 2: Bridges
- Part 3: Assessment and retrofitting of buildings
- Part 4: Silos, tanks and pipelines
- Part 5: Foundations, retaining structures and geotechnical aspects
- Part 6: Towers, masts and chimneys

Because this thesis is limited to usual industrial buildings, the most referred document will be EN1998-1 that will further be named as “Eurocode” or “EC8”. In case some other parts are linked, the full name of the standard will be given.

The European Union comprises a lot of countries, so it is impossible to give the same coefficient values for all of them. For that purpose the special local standards named “National annexes” were introduced, giving the possibility for country authorities to influence the design in their territory. For example, Finland does not have a national annex to EC8 at all, because the country is totally non-seismic.

### 3.2 SNiP II-7-81

In the Russian Federation, all the structural design is governed by a building code named SNiP. This is the abbreviation in Russian that means “Building norms and regulations”.

Just as Eurocodes, the Russian standard is not a single document and is divided into topics, each of them containing a number of documents with a unique number.

When it comes to Earthquakes, a designer must apply to the partition II “Structural design” where there is the topic “Urban design” and finally the document named SNiP II-7-81 “Seismic design”. In contrast with the European norms, it is a single document that covers everything needed from foundations to fire safety.

However, Wikipedia tells that SNiP system was introduced in 1955, that time the first documents were published. Since that, structural design has made a huge step forward, which could not be ignored in the standards. This is why nowadays there often are so-called “actual versions” of various SNiP documents named SP (“set of regulations” in the Russian trans-

lation). The latest edition of the seismic document is named SP 14.13330.2014. Its first page tells a reader that it is nothing but the latest version of SniP II-7-81.

This Bachelor's thesis is based on the latest document available nowadays (January 2015) that is specified above. In the pages below its name will be shortened to "SP" or "SniP" which mean the same up-to-date standard. Sometimes, the references are made to the original SniP, published in 1981. In that case, it will be specified and the document linked to will be called "original SniP".

In addition to the obligatory norms, there also exist official comments and guidance to most of the documents. It contains typical generally adopted solutions with calculations examples that may serve as acceptable explanations for the designer's choice. In this project, the references will be made to the official guidance to the original SniP II-7-81.

## 4 MODELING AND PRE-DESIGN STAGE

### 4.1 Seismicity definition and design situations

#### 4.1.1 Europe

In Europe an earthquake hazard is estimated separately for two different design situations. According to EC8, a building must be verified in the two limit state systems:

- Ultimate limit state
- Damage limit state

As it was specified earlier, the main value to estimate the possible earthquake strength is the reference ground acceleration that is given by the national authorities.

For the ultimate limit state Eurocode suggests to measure the acceleration that has a 10% probability of exceedance in 50 years and the return period 475 years. The value of 50 years is a constant independently of a country, whereas the other numbers may be selected by a local government. For the return periods other than stated, the acceleration value is found by multiplying the reference one by the factor  $\gamma_I$ .

For the damage limitation criteria the possibility of a greater earthquake and its return time are measured in 10 years, with the recommended values from EC8 equal to 10% and 95 years respectively for the probability and return period. When only the ULS ground acceleration is given by national specialists, the value for the DLS may be obtained via the factor  $\gamma_I$ .

The design ground acceleration equals the reference one times the reliability factor  $\gamma_I$  that is equal to 1 if the reference return period is used. For the structures of high importance, the bigger return period must be used. To achieve this, the standard distinguishes all the buildings into four importance classes, giving the  $\gamma_I$  coefficient for each of them in the section 4.2.5 of EC8.

Of course, it is up to a local government to divide regions into seismic and non-seismic. Anyway, the standard recommends to consider the areas with  $a_g$  less than  $0.78\text{m/s}^2$  or  $a_g \cdot S$  less than  $0.98\text{m/s}^2$  to be low-seismic. In the expression above  $S$ , stands for the soil factor that is described later and  $a_g$  is the design ground acceleration.

Non-seismic areas according to EC8 are those where  $a_g < 0.39\text{m/s}^2$  or  $a_g \cdot S < 0.49\text{m/s}^2$ . This is according to the clauses of 3.2.1.

### 4.1.2 Russian Federation

In Russia there also are two different design situations. Although, their meaning is quite far from being the same with Eurocodes. Based on the sub-chapter 5.2 of SP one may find the following cases:

- Design earthquake (DE). It is aimed to maintain a building usable (based on its purpose) during and after an earthquake. Although this is the exact translation from the standard, it does not anyhow mean serviceability limit state. The design earthquake situation involves the building check at ultimate limit state under a seismic phenomenon based on several factors. The analysis must be linear elastic.
- Maximum design earthquake (MDE) situation. This aspect must prevent a building from a global or local collapse that may be dangerous for people. A principal difference from a simple ULS check (DE) is that the consequences of a building failure are dangerous not only for the inhabitants, but also for the environment. For example, if a nuclear power-plant collapses, the result is devastating for everything inside a several-kilometers radius. The analysis must be non-linear. In practice, the definition of this situation makes it obligatory only for the buildings of primary importance (like the power-plant above or hospitals). Therefore, this case is not observed in the current thesis.

The design earthquake situation must be applied to all buildings within a seismic design. The maximum design earthquake case is required to verify only for high-responsibility buildings that one may find in the fields 1 and 2 of the table 3 in SP. Due to its size and simplicity, it is not listed here.

In Russia it is needed to define the region seismicity at first. For that purpose, in the annex of SP there are the maps of the Russian Federation (OCP-97) with the reference earthquake intensity. In total, there are three maps: A, B and C. Each of them corresponds to the catastrophe of certain intensity with the probability of exceedance 10% for the map A, 5% and 1% for the maps B and C respectively. The possibility is measured in 50 years, just as in Eurocodes. According to the sub-chapter 4.3 of SP, the map is chosen by a contractor. However, it is advised to use the help of a special organization. For the MDE situation, a seismo-tectonic expertise must be carried out, the map must be selected according to its results.

Secondly, SP requires to find out the site seismicity. This will be the one used in structural analysis. However, the clause 4.4 strictly forbids selecting this value without the engineering expertise that accounts soil, groundwater and seismo-tectonic investigations.

Anyway, if the map A has been selected to define the region seismicity, SP allows to choose a preliminary value based on the soil type according to Table 1 of SP.

Finally, if the site seismicity turned out to be less than 7 degrees according to the map (return period) chosen, the site is considered to be non-seismic according to the applicability criteria of SP (Chapter 1).



## 4.2 Structural simplicity criteria

It is common knowledge that most of the building codes propagandize and encourage to use simple and generally adopted solutions. This becomes even more important when speaking about earthquakes. In the book “Designers’ guide to Eurocode 8: design of structures for earthquake resistance” (further called Designers’ guide to EC8) the authors describe the recent tests that revealed a better resistance of symmetric rightly shaped buildings. So it is not a surprise, that both SNIIP and Eurocode insist on avoiding extraordinary building appearance. (Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioll, Paolo Pinto, Andre Plumier “Designers’ guide to Eurocode 8: design of structures for earthquake resistance”)

To encourage both contractor and designer, various benefits are provided for the buildings corresponding to regularity criteria: starting from the design load reduction (therefore, less material quantity needed for a frame) and ending with design simplifications procedure that an engineer might adore.

### 4.2.1 Europe

EC8 divides structural simplicity into two regularity criteria: in plan and in elevation. A possible outcome of taking them into account is best described by Table 1 below (Table 4.1 in Eurocode 8):

Table 1 The outcome from simplification criteria in EC8

Regularity		Allowed Simplification		Behaviour factor
Plan	Elevation	Model	Linear-elastic Analysis	(for linear analysis)
Yes	Yes	Planar	Lateral force <sup>a</sup>	Reference value
Yes	No	Planar	Modal	Decreased value
No	Yes	Spatial <sup>b</sup>	Lateral force <sup>a</sup>	Reference value
No	No	Spatial	Modal	Decreased value

<sup>a</sup> If the condition of 4.3.3.2.1(2)a) is also met.

<sup>b</sup> Under the specific conditions given in 4.3.3.1(8) a separate planar model may be used in each horizontal direction, in accordance with 4.3.3.1(8).

#### 4.2.1.1.1. Regularity in plan

Eurocode gives its requirements for a regular in plan building in the clause 4.2.3.2. The main idea is to make a structure approximately symmetric, whereas keep the mass and stiffness centers near to each other. In addition, the floor behavior comparable to a rigid diaphragm is quite important. The definition of rigidity is given in the part 4.3.1 of the standard. This helps a building to distribute lateral forces more evenly in case of eccentrically applied mass (e.g. imposed load). On Figures 4 and 5 below one may find the difference in structural behavior between a similar structure with and without rigid floor panels.

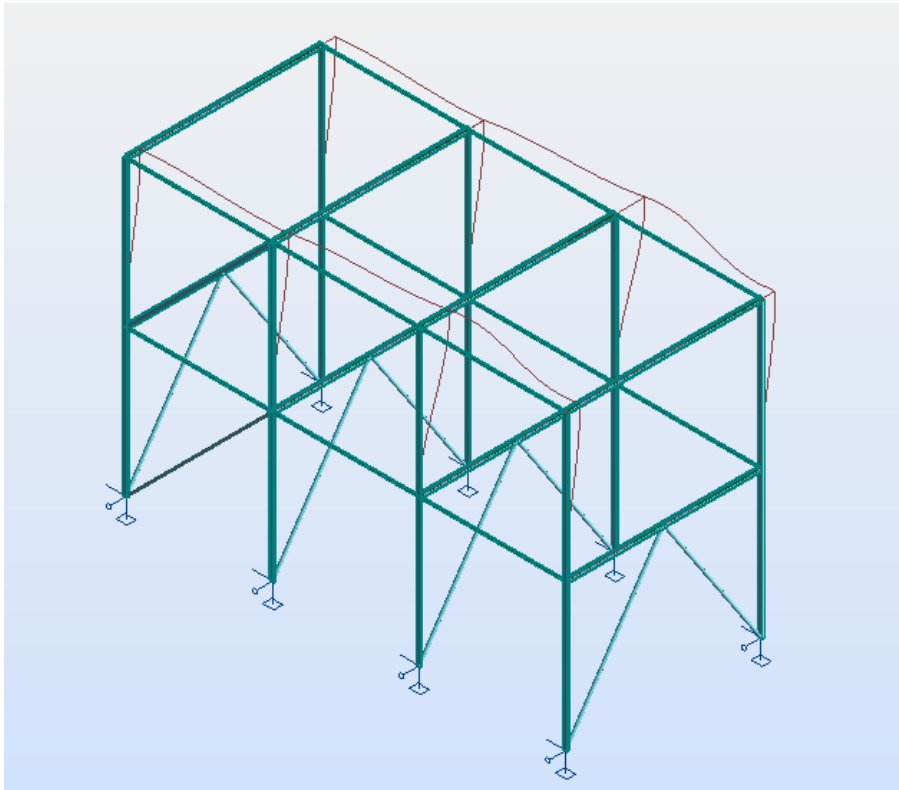


Figure 4 Lateral deformations in the structure without rigid diaphragms

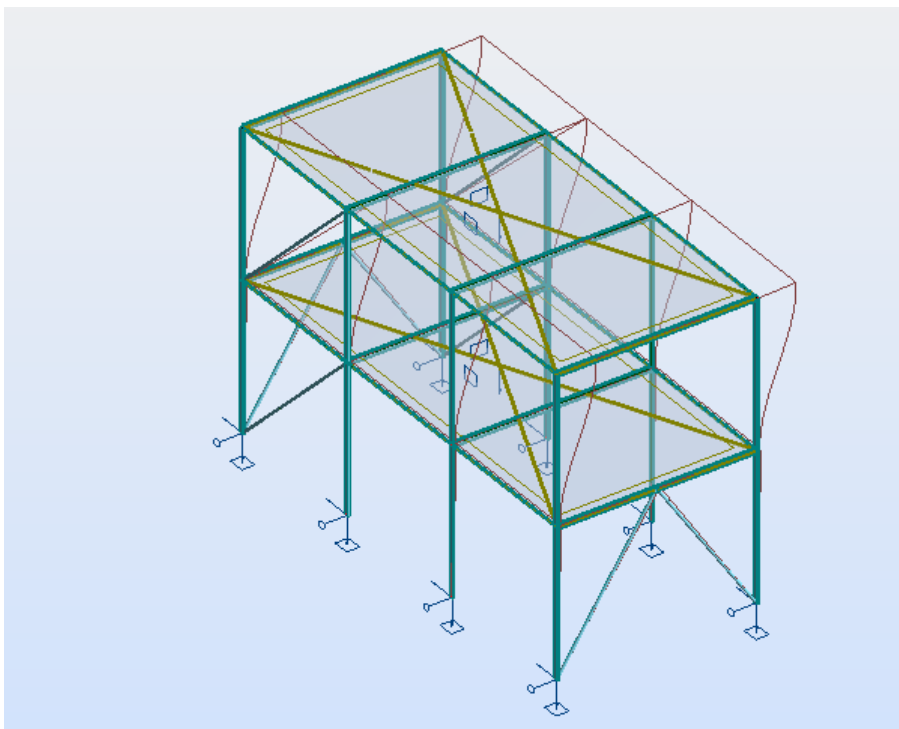


Figure 5 Lateral deformations in the structure with rigid floor

Besides, Eurocode also pays attention to adequate torsional stiffness. It is discussed more detailed in the chapter about the Russian standard.

#### 4.2.1.2. Regularity in elevation

The other set of requirements concerns building regular side-view. Again, the focus is held on the masses and stiffness. Therefore, the demands for regularity in elevation that are written in the section 4.2.3.3 concentrate on a small variety in storey masses and bracing systems that go from the foundation to the roof.

#### 4.2.2 SNIIP

In the Russian standards, there is only one generally adopted structural simplicity criterion. Other requirements are not gathered upon one code clause, so in this Thesis they will be dealt with in the chapters corresponding to the relevant benefits given by them.

A building can be considered to be regular (in other words, to have a simple structural frame) if it satisfies the conditions given in the sub-chapter 5.3 of SP:

- First and second vibration modes of a structure are not torsional about its vertical axis (an example of torsional form may be found below)
- Maximum and average values of lateral displacement at every floor do not differ more than by 10% for all the lateral vibration modes of a building.
- Period values for all the accounted vibration modes must differ at least by 10%.
- The condition of 4.1 must be fulfilled.
- Satisfy the condition of the table 7.
- There are no large holes or openings in floor diaphragms that significantly weaken their stiffness.

A brief explanation of the requirements should be provided. Most of them are based on a so-called “modal analysis” that is clearly described in the next chapter. Anyway, the reasons that may cause an undesirable outcome and make a building irregular can be given here.

The first demand is actually the same as the clauses from 6 to 9 of 4.2.3.2 in EC8. These criteria are based on the comparison of building lateral and torsional stiffness. The goal is to have the latter much bigger. As it is written in the Designers’ guide to EC8, this prevents simultaneous twisting and a lateral response throughout an earthquake that is considered uncontrollable and dangerous. So, both standards may be used to perform a preliminary check of each other. The guide authors also underline that there is nothing to be afraid of if the lateral stiffening elements of the structure are distributed as even as the mass. Unfortunately, this can be hardly ever achieved in practice whereas these bracing members occupy quite much of usable space. However, a designer may be almost sure that his building is not regular if most of the lateral restraining system is concentrated near the mass centre, as shown in Figure 6 below.

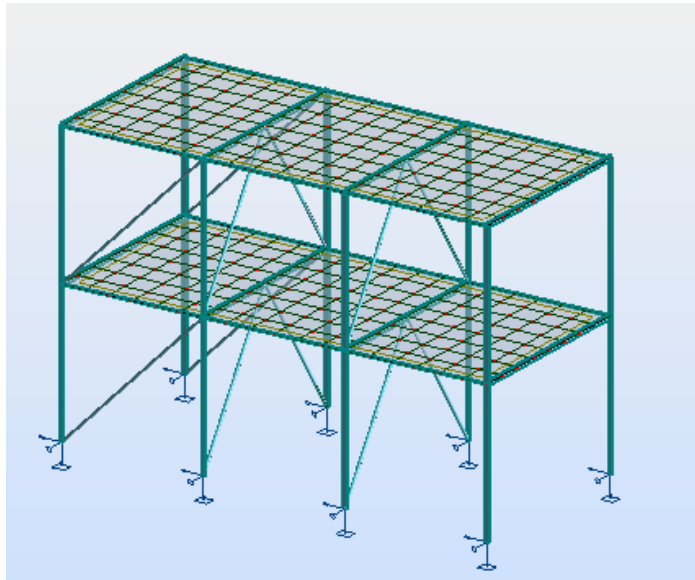


Figure 6 A frame with the first vibration mode being torsional

The second requirement means the estimation for the floor rigidity. In the same manner as in Europe, it does not require doing a full seismic analysis: the displacements are estimated from the modal analysis.

The criterion three will be discussed more in the analysis part. As for the condition 4.1, it says that a building must correspond to the ductility requirements that will also be explained later.

Table 7 of SP determines a building maximum height with respect to its frame type.

### 4.3 Load combinations and seismic mass

#### 4.3.1 Eurocode 8

Generally, the limit state system applied in both Russia and Europe involves partial safety factors to account for the possible overload in reality and to obtain the most inconvenient internal forces in members.

Nevertheless, during an earthquake the forces undergone by a building are very dissimilar to those in everyday life. To account for this, Eurocode offers special different load combinations for two purposes:

- Definition of building mass during an earthquake
- Combination of seismic actions with other forces

It is vital to describe the key distinction between the cases above. A seismic effect on a structure depends directly on mass distribution inside it. In other words, before the analysis is done it is required to assess the mass of the building that will interact with a ground motion, so-called “seismic mass”. Its estimation is given in 3.2.4 of EC8. The characteristic permanent loads are combined with the quasi-permanent live load ( $Q_k^* \psi_2$ ), with an additional reduction due to the non-rigid connection of mass to a sup-

port, as written in Designers' guide. To paraphrase, people are not glued to a floor, which causes a dissimilar structural response during an earthquake comparing to a concrete beam that is fixed to columns. The mass obtained after this combination is the one that must be used in all the analysis types described further to obtain a seismic action effect (4.3.1(10)).

The second case is needed after the ground motion effect on a structure has been found with any of the analysis types below. This combination is needed to obtain the internal forces in a member to be compared with their resistance. To put it simply, this is the combination for the structural analysis in a seismic design situation. In 4.4.2.2 the standard refers to the seismic combination according to EC1990. It involves the characteristic dead load, the design seismic action and the quasi-permanent value of the live load. In this case, the variable load  $Q$  is not anyhow reduced because of its possible fixing to a structure.

#### 4.3.2 SNIIP

The Russian standard distinguishes the same situations for the load definition process.

Firstly, building mass that must be considered in modeling for seismic force definition is discussed. The part 5.5 of SP refers to the clause 5.1 in order to find out the mass. There the special combination factor  $n_c$  (an equivalent to  $\psi$ ) is introduced for the design values of dead and live loads. There is no necessity to explain the inequalities in the load classification in the two standards, since one may find in "Building frame design" or "Designer's handbook" (L. Mailyan, D. Mailyan, E. Lukashevich, U. Veselev, G. Verzhbovskiy, V. Lagutin, L. Mailyan, A. Mailyan - "Designer's handbook" «Справочник современного проектировщика»; V. Setkov, E. Serbin – "Building frames design" «Строительные конструкции расчет и проектирование»).

In terms of this thesis, only the seismic case is taken into account. Stated in SP20.13330.2011 (loads and combinations), this case belongs to the special load combination type. As specified in the point 4.3, the partial safety factor  $\gamma$  then is equal to 1. Besides, the part 5.5 of SP13.13330.2014 clarifies that only those loads which produce inertial mass must be considered. To say it simply, only the masses that rest on a structure (e.g. wind should not be considered) must be considered, while the full list of the loads to be neglected may be found in 5.1 of SP. A conclusion of everything above will be the formula for building mass calculation according to SNIIP:

$$m = G_k + \sum Q_{ik} * n_{c,i}$$

Where the coefficient  $n_c$  is taken from the table below (table 2 of SP). The default combination factor  $\psi$  from the SP about load combinations is neglected. Hopefully, one has noticed that the mass connection to the structure is accounted neither in the seismic mass definition, nor in seismic combinations, in contrast with EC8.

Table 2 Table 2 of SP

Load type	Combination factor $n_c$
Dead	0.9
Live long-term	0.8
Live short-term	0.5

To combine the earthquake effect with other components, the same rule is followed. One must use the partial safety factor 1 and only the combination factor  $n_c$ . Wind and thermal loads are also ignored. Concluding everything from the previous paragraph, the design internal force in a member  $F_{Ed}$  is equal to:

$$F = G_k + \sum Q_{ik} * n_{c,i} + A_{Ed}$$

Where A is the earthquake effect and is superpositioned in any direction.

#### 4.4 Modeling

##### 4.4.1 General provisions

###### 4.4.1.1. Spatial versus planar model

In a construction design there is always a question of whether a spatial (three-dimensional) or two several planar models should be used to obtain internal member forces. Before the large variety of computer software became available, there had always been a strong desire to limit the analysis work to one 2D model in each horizontal direction.

In most cases that can be quite easily achieved, but not in the seismic one. The reason comes from the possible torsion influence of an earthquake which is not very simple to represent via planar analysis. Based on this, it is vital to ensure a building uniform lateral displacement when affected by an earthquake. So, it is not a surprise that in both codes there are particular demands that a building must meet in order to be analyzed in 2D.

According to Designers' guide, in a planar model a structure is assumed to consist of a number of plane frames connected by a rigid diaphragm. The bracing elements that resist lateral forces in the perpendicular direction are not modeled.

After everything above, one might have a reasonable question: is it worth spending time on adjusting a building frame to the simplification criteria, whereas nowadays even free FEM software is capable of performing a full three dimensional analysis for a moderate building? Of course, there are many opinions as usual. For example, the authors of Designers' guide believe that 2D modeling for linear elastic analysis is "not worthwhile." From the other point of view, it is always a noble goal to achieve structural simplicity for the causes described previously.

4.4.1.2. FEM modeling

Whereas today 99% of analysis is done using computer programs based on the finite element method, it becomes essential to clarify some possible problems that a designer may face. In the both building codes there is said nothing about it, since an engineer can choose the solving method by himself.

Firstly, the importance of joint regions must be underlined. This means considering the eccentricity due to members connection. In other words, keep in mind that a beam is usually connected to a column face, but not to its neutral axis, thus causing a bending moment. Generally, it accounts all the design situations, not only the seismic one. However, an earthquake resistance is based mainly on a bracing system, so the aspect becomes even more vital.

Various methods to account for the eccentricity have been proposed in Designers' guide and "Design of steel structures". The most convenient is to always model a rigid massless element between the neutral axes of members and their actual ends, as it is illustrated in the Figure 7. (Luis Simoes da Silav, Rui Simoes, Helena Gervasio - "Design of steel structures")

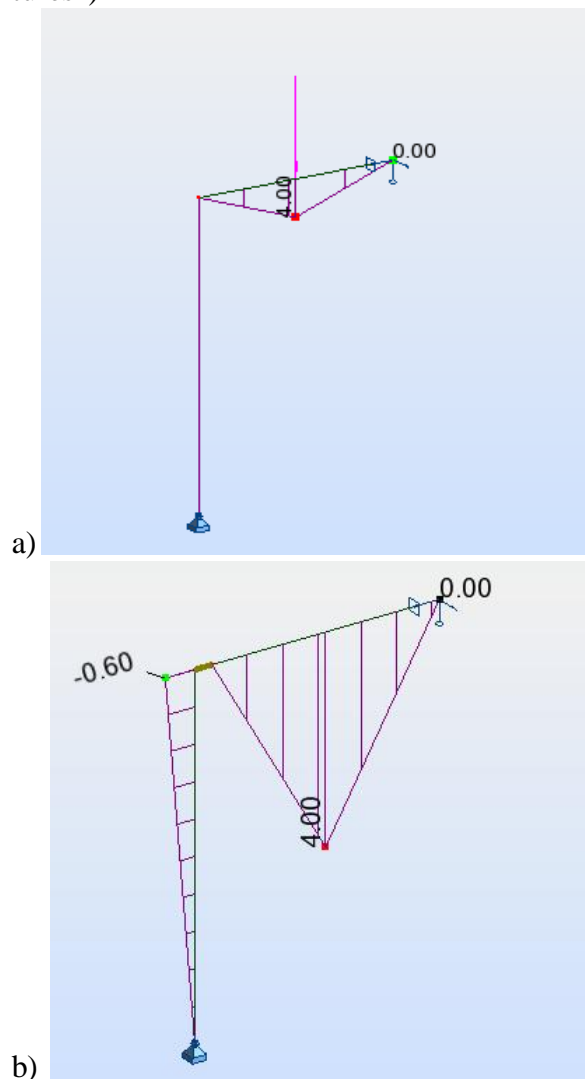


Figure 7 Internal bending moment diagrams of a beam and a column connected to each other in the two ways: a - with an idealized, center-to-center beam to column connection. Bending moment occurs only in the beam; b - the beam is attached to the column face, but not to the neutral axis. The distance from the face to the centroid is connected with a rigid link. Now there is the  $-0.6 \text{ kN}\cdot\text{m}$  moment in the column.

Secondly, the usage of shell elements should be avoided. According to the Designers' guide, the main reason is that most of the detailing rules in the standards were developed for prismatic beam elements. They do not take into account, for example, a shear lag or shear deformations that are obtained in the stress distribution of shell elements. For this reason, in case of a concrete frame, it is quite desirable to design a floor as a rigid membrane and model T-beams as line elements with linearly distributed loads applied to them. Besides, the authors of "Design of steel structures" underline possible difficulties when connecting slab parts to beams if a composite action is desired (most of the cases with T-beams). It will be required to rigidly connect the nodes of shells to the beam ones.

The third point comes as a question from the second one. Usually, shell elements are used not only for floors, but also for walls. In this case the guide authors offer very easy solution by replacing a shell wall with a huge beam element with the cross section of the wall in plan.

The offered way corresponds highly to the EC requirements and is a perfect representation of the elastic analysis method of walls according to "Reinforced concrete design to Eurocodes". Saying briefly, one way to design a RC wall in a conventional (non-seismic) situation is to assume elastic stress distribution based on the same easy formulas that are applied to beams. After the analysis different (shear, bending or even torsion) stresses are obtained and a huge cross-section is separated into one meter wide parts that are treated as columns. (P.Bhatt, T.J. MacGinely, Ban Seng Choo - "Reinforced concrete design to Eurocodes")

This solution provides wide possibilities for the subsequent hand or other software analysis. The wall beam must be divided into parts (with fixed connection to each other, of course) between the stories. Nevertheless, no one has ever made a staircase shaft without openings. To account a hole location, a separate beam with the length equal to the opening height and a net cross section of the shaft in plan is put in the middle. The connections of other frame beams and a wall beam with an opening must be arranged via rigid links in order to account torsion. An illustration of the method is given in Figure 8.



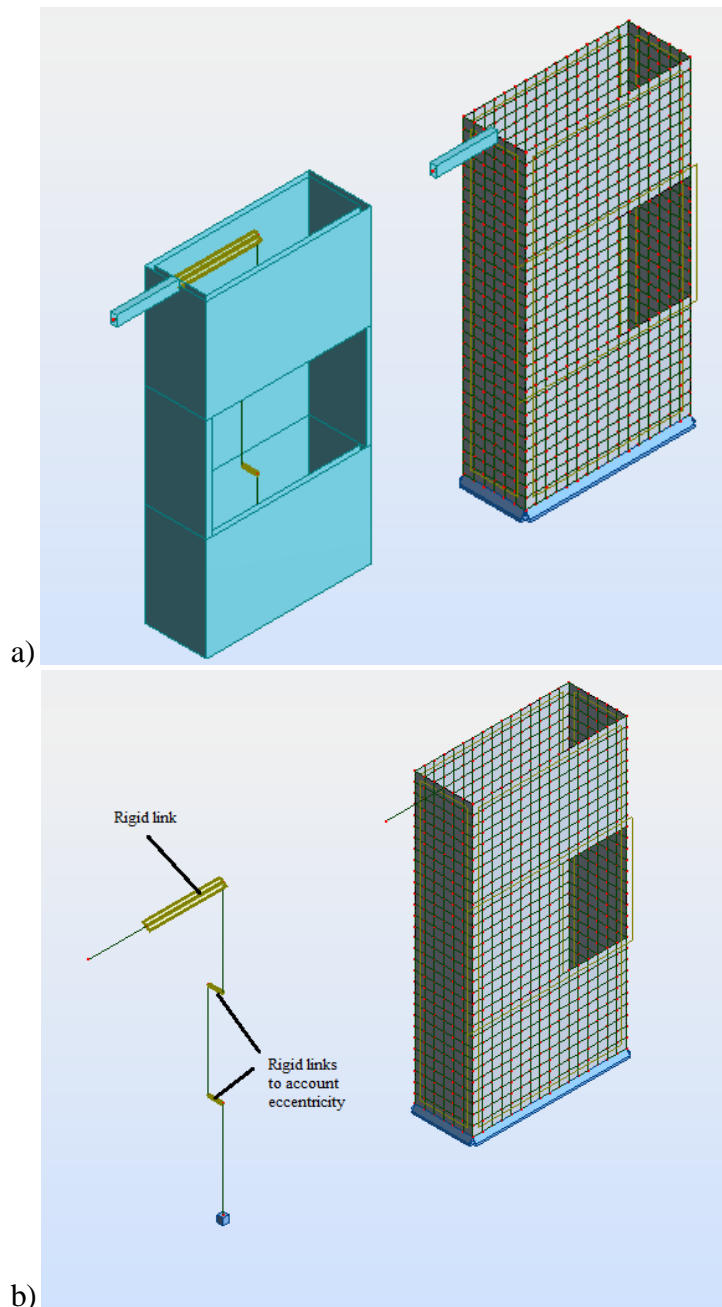


Figure 8 An example of replacing a shell staircase shaft containing an opening with a set of beam elements: a – section shape visual representation is on, b – section shapes are off to underline rigid links

One should also keep in mind that despite the easiness of this method it does not represent local stresses around the area of a hole, so a separate analysis based on the end forces and moments of the wall-beams should be carried out to detail the reinforcement.

In addition to hole problems, locally concentrated moments from coupling beams and the same kind of stresses caused by a wall connected to a foundation slab need to be analyzed with different modeling method, for example, strut and tie. A good description of local analysis of joints or holes can be found in Narayanan and Beeby. (R.S. Narayanan and A. Beeby - “Designers’ guide to EN1992-1-1 and EN1992-1-2”)

### 4.4.2 Eurocode

EC8 offers plenty of modeling simplifications so as to encourage a designer to keep a regular shape of a building. However, with the computer software available nowadays these allowances may seem quite unnecessary. All the modeling demands are listed in the part 4.3.1, while their brief explanation is provided in the next paragraphs.

Firstly, the standard allows the usage of two separate planar models for the structures regular in plan and satisfying the criteria in 4.3.3.1(8). Currently this may be useful only for a non-linear analysis that is not covered within the thesis.

Secondly, the statement 4.3.1(6) obligates to account cracking in the stiffness of all the load bearing elements made of concrete or masonry. The main reason lies in the specific rules for energy dissipation that will be described later. Those rules assume a ductile behavior with the yielding of reinforcement. Therefore, the internal member forces in reality will be based on concrete with the cracks corresponding to yielded steel. That is quite contrary to the usual design, where Narayanan and Beeby advise usage of gross element attributes at ULS. Unless it is not achievable to estimate this material property, the statement 4.3.1(7) allows accounting the 50% value of the initial concrete or masonry stiffness. In addition, torsion rigidity must be treated carefully, since the diagonal cracks resulting from twisting gradually reduce torque stiffness. Though it is not stated in EC8, Designers' guide recommends accounting a very small percentage of its gross value. The authors of "Worked examples to EC8" used about 10% in their calculations. Nevertheless, in a design earthquake situation (not while the construction process itself, when not all the frame elements have been assembled yet) torsion moments for beams happen quite rarely.

As a third point, the foundation deformability is vital according to the clause 4.3.1(9) even if it has a positive influence on internal forces. In practice it may be complicated to achieve if the foundation design is done by another company. In case of pad footings, it may also lead to iterative procedures. Usually, basement elasticity reduces the design forces from an earthquake, resulting to smaller pads needed, which reduces the pressure area on soil and increases deformability. Masses for the cases with rigid floors may be lumped at their gravity centre.

Finally, EC8 offers one more simplification for designers. It distinguishes a building frame into primary and secondary seismic elements. The latter may not be taken into account in the analysis, unless their effect to the lateral stiffness exceeds 15% of that from the primary members (4.4.2(4)). The regulations for their detailing are given in 4.2.2 of the standard. Secondary elements are not obligated to correspond to the plenty of ductility detailing rules (they are discussed in the relevant chapters). Anyway, they must still be able to carry the gravity loads with accounted second order effects resulting from the building deformation throughout an earthquake. Unless a computer program is capable of disabling element stiffness during the response calculation as it is described in Designers' guide, a sepa-

rate analysis might be necessary. This is how one more simplification turns into additional work.

#### 4.4.3 SNiP

The Russian standard in the clause 5.5 differentiates two possible types of modeling that significantly influence the further analysis procedure. These inequalities are stated in the relevant chapters of this thesis.

For the regular structures, it is allowed to use a so-called cantilever model. A literal translation will give the term “console model”, which shows the different terminology in the two languages. The main idea is that a whole building is represented as a fixed-end column divided into stories that displace laterally throughout a seismic activity. The illustrating picture from SP is presented below in Figure 9.

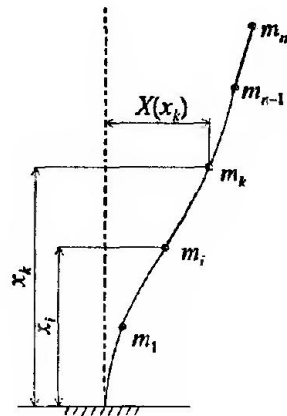


Figure 9 A cantilever model of a building (SP 14.13330.2014)

Masses are allowed to be concentrated in the nodes of a “column,” in other words, lumped at gravity centers of floors the same way it is advised in Eurocodes.

Based on the further regulations concerning the seismic action direction and analysis types, one may also use two separate planar models with this method, though it is not stated in the standard.

However, there are obvious disadvantages in a cantilever-type model. First of all, the analysis result will not give any torsion about vertical axis. Secondly, non-rigidity of floor diaphragms may cause various spatial displacements. Therefore, the buildings with a high possibility of the effects above (non-regular frames) must be designed using the spatial model.

The spatial model must account multi-directional deformations of a structure. Masses in that case should be applied to the nodes of the structural members. In case of a vertical seismic action effect, it is needed to uniformly distribute the mass along beam elements in order to cause relevant bending moments.

In contrast with Eurocodes, the official guidance to the original SNiP allows the usage of non-cracked (gross) concrete stiffness, although there will be cracks at ultimate limit state (statement 3.25). However, in case of a RC truss the tensile stiffness must be the one of the reinforcement, ignoring concrete (clause 3.37).

In both cases the clause 5.10 obligates to account ground elasticity in a structural model, disregarding whether the effect is favorable or unfavorable.

## 5 DEFINITION OF ACTIONS ON STRUCTURE

It is common knowledge that in 99% of cases in structural design the linear static analysis is used. Explaining this a bit more precisely, one defines a magnitude of a force and applies it to a structure in full accordance with the most simple mechanics laws. Generally, all the actions that are applied to a building in reality, e.g. wind, imposed load or self-weight, act very simply. Therefore, their magnitude and direction may be easily defined based either on statistics or material properties. The results of the definition will correspond to actual life with a high precision.

Unfortunately, this is not the case when it comes to an earthquake. The reason is that its nature differs quite much from snow or an imposed load. The high magnitude of actions and vibration motion in addition to a very rapid direction change make the usage of static analysis extremely difficult, whereas there occurs a notable contribution from a building frame itself.

This is why both standards propose various analysis types for structures subjected to earthquakes. While different building frames produce an unequal response to this phenomenon, there are special requirements for every analysis type. In this thesis, the explanation will be limited only to the linear analysis types, since they may be applied in most cases.

### 5.1 Eurocode

Eurocode 8 proposes four possible analysis types, which must be convenient for every designer, since one is given the opportunity to choose between them. From the other side, there are certain requirements that a structure must meet if an engineer wants to use a particular way of earthquake force definition. With this sort of limitation, the authors of Eurocode want to encourage the usage of simple and generally adopted solutions. The necessity of that was explained previously. In this project, only the linear analysis methods will be described.

### 5.1.1 Modal response spectrum analysis

#### 5.1.2 General definition

In spite of many possible ways to determine the internal forces in members, EC8 offers one reference solution that may be applied to all kind of structures without any limitation. It is called the modal response spectrum analysis.

CSI knowledge base defines it as “a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure.”

The definition, however, does not describe what is done throughout this procedure. In a few paragraphs below a short explanation is provided that is based on the book “Designer’s guide to EC8.”

As a brief introduction, it is necessary to say that all the structures when subjected to an earthquake motion from the ground tend to behave like a very stiff inverted pendulum. In other words, it waves back and forward with a certain frequency. This is called a structure’s “natural” or “fundamental” frequency and shape. The fundamental frequency is always the lowest one. Nevertheless, almost every building has more than one vibration shape, as it is written in Wikipedia. For a complicated frame there is quite a high opportunity of activating those shapes during an earthquake. The figure 10 shows the first and second vibration mode shapes of a simple structure.

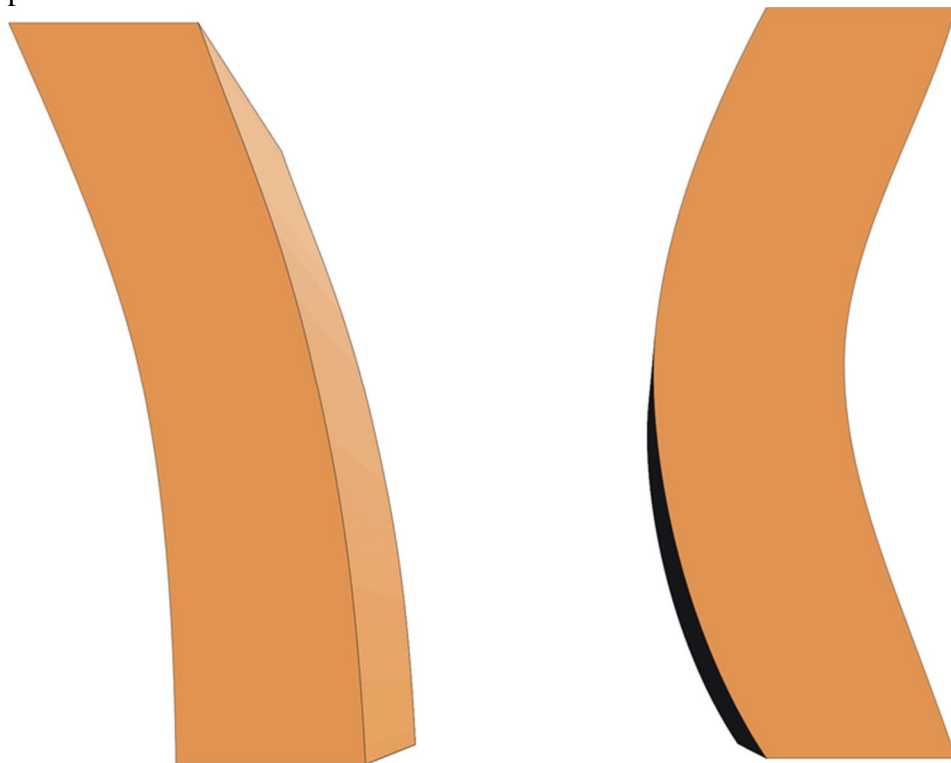


Figure 10 First and second vibrational modes of a simple structure ([http://en.wikipedia.org/wiki/Seismic\\_analysis](http://en.wikipedia.org/wiki/Seismic_analysis))

The process of definition of modal shapes and frequencies is called a modal analysis. This is the first step of a modal response spectrum analysis.

After all the required modes are obtained, an earthquake motion may finally be applied to a structure. For this purpose a so-called “response spectra” is used. It is a curve that represents any desired type of response, e.g. acceleration, velocity of displacement, at a certain period. The examples of a spectrum are given in Figure 11. It is essential to underline that the curve represents not the ground motion, but the response of a single degree of freedom structure to it. In other words, it shows not the ground acceleration, but the acceleration of a building subjected to an earthquake.

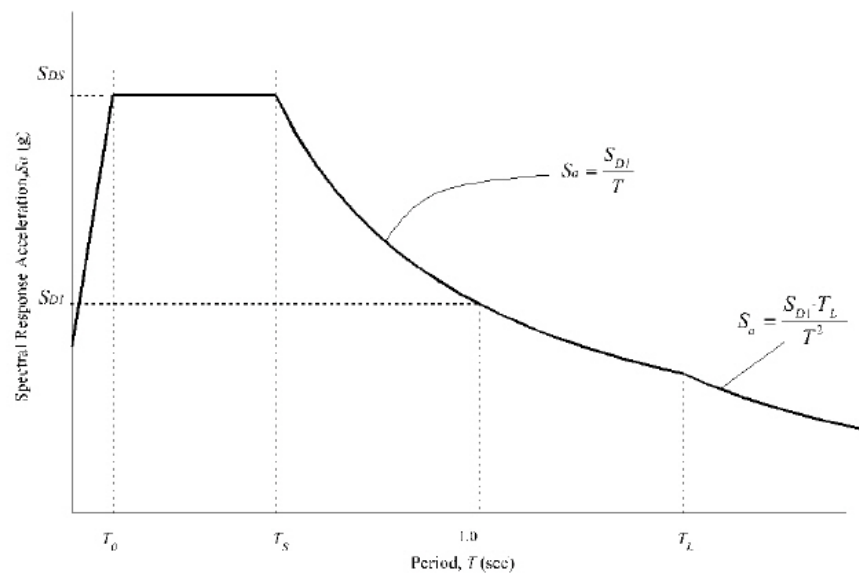


Figure 11 An example of acceleration response spectra (<http://www.isatsb.com/Seismic-Ground-Motion-Values.php>)

To obtain a seismic action effect via a modal response spectrum analysis for a vibration mode  $i$ , the following procedures are done:

- The acceleration  $S_a(T)$  value that corresponds to the frequency of the mode  $i$  is taken.
- According to the period and acceleration, the spectral displacement  $S_d(T)$  is obtained.
- The spectral displacement value is multiplied by the modal shape factor and modal participation factor (that accounts the mass) for the mode  $i$ .
- The product above is a nodal displacement, according to that the internal forces in members are found.

This is vital to realize that in a MRS analysis the procedure is reverse comparing to a usual static approach, where forces are applied to a member and the displacements are calculated based on them.

As far as all the modes have a chance to exist in reality, this is required to find the combination of them all, which will be the final magnitude to be used in structural design. Unfortunately, it does not mean a simple sum-

marization. In real life, an earthquake shock is a very rapid, short-time phenomenon. This means that not all vibration modes occur at the same moment. Therefore, a maximum *possible* effect due to their interaction is searched for. There are several commonly-known rules to do that, and it is up to a building code to choose between them.

### 5.1.3 Spectrum definition

As it was mentioned above, the seismic action in Eurocodes is represented by an acceleration response spectrum. Initially, it is required to define a so-called “elastic response spectrum”. The definition procedure is clearly provided in the clauses of 3.2.2.1 to 3.2.2.3. Anyway, it would be wise to provide here a short summary with additional commentaries.

An elastic response spectrum depends on the following factors:

- Design ground acceleration
- Soil factor
- Period limit values
- Curve shape
- Damping

Design ground acceleration is based on the reference one which is a major characteristic of region seismicity and is given by local authorities. Consequently it must be transferred to a design value via the simple procedures described in the sub-chapter 3.2.1 of EC8.

When it comes to the soil, it should be noted that the reference ground acceleration is measured on the rock soil that belongs to the type A soil in accordance with EC classification. For the other ground types the earthquake may be more damaging; therefore, the acceleration value is multiplied by the soil factor  $S$ . The ground type definitions are given in Table 3.1 of the standard. As for the factor itself, it can be chosen by national authorities, though some recommended values are provided in EC8.

The period limits  $T_b$ ,  $T_c$  and  $T_d$  are usually described in national annexes, too, but in case of their absence the recommended values are proposed. Their purpose is to affect a spectrum shape.

In addition to that, a spectrum shape may also be proposed by the local authorities. This is something that the soil factor and the period limit depend on. Eurocode 8 offers two possible spectrum curves with the different factor values for each of them.

The reference value for the damping factor is 1 for 5% viscous damping, which is quite rarely changed. Nevertheless, if a designer uses the value other than 5%, the appropriate rules are given in the clause 3.2.2.2(3).

It is common knowledge that an earthquake is not singly directed. This is why the standard obligates to separate the effect into components. Two horizontal components must be represented by the same spectrum. The

rules for defining a vertical elastic response spectrum (that, obviously, must be used for the vertical component) are given in 3.2.2.3. These regulations use separate vertical design ground acceleration and do not require considering a soil type.

After an elastic response spectrum has been defined, the standard requires an engineer to obtain a design one that may finally be used in a MRS analysis.

Nowadays, the most popular global structural analysis type is linear elastic, despite the wide range of computer software. However, for the ductile material like steel it is extremely cost-ineffective not to account its plastic capacity. For that purpose, various moment redistribution rules that allow to partially utilize the plastic resistance of a cross section are written in the relevant standards (EC1993-1-1 for steel and 1992-1-1 for reinforced concrete). These requirements help to produce cost-effective solutions while staying in the range of global elastic analysis (L. Gardner, D.A. Nethercot – “Designers’ guide to EN 1993-1-1”).

The same method is used for the seismic design. Since the MRS analysis is a linear one, all the internal member forces obtained afterwards correspond to the theory of elasticity. If they were the final values, the response spectrum method would never be used by anyone because of the huge out coming price for a structural frame.

This is why the definition of a design spectrum is required. Its main difference from an elastic one is the reduction of forces due to the plastic capacity of a building frame. The outcome values will represent internal member forces after plastic deformations of specific members that may be used in the resistance verification.

However, it is vital that no stability failure occurs because of the excessive plastic deformations in the members responsible for lateral stability. The ability of a structure to undergo cycling variable plastic deformations without collapsing is called its “energy dissipation capacity”.

The reduction factor due to that capacity is represented by the letter  $q$  and is named “behavior factor” in EC8. Its value is obtained by meeting a set of obligations, specific for every material. They all will be described in the next chapters.

Besides this, the clause 4.4.2.2(1) clearly allows to use the moment redistribution rules from EC3, 4 and 2 at the same time with the forces reduction due to the energy dissipation capacity.

As for the rules of decreasing the elastic response spectrum to the design one, they are given in the sub-chapter 3.2.2.5. This is how the modal response spectrum analysis allows to utilize the significant value of material ductility while remaining linear.



### 5.1.4 Vertical component of a seismic action

Generally, the vertical component of an earthquake effect is not taken into account. The reason is that in most cases the axial stiffness of columns is much larger than the lateral stiffness of a frame.

Therefore, Eurocode obligates to consider the vertical component only in the cases when it causes significant bending stresses in beams. The rules are described in the chapter 4.3.3.5.2.

Even if a structure has some elements that are very sensitive to up and down vibrations, the standard permits applying these effects only to them, without the necessity to subject the whole building to the vertical component (clause 4.3.3.5.2(3)).

### 5.1.5 EC8 requirements for MRS analysis

As it was stated above, this analysis type can be used for all structures. However, there still are some rules for the analysis itself that are given in the chapter 4.3.3.3 of the standard.

The chapter starts with the regulations for modal analysis that the further effect determination will be based on. The principal criterion is the minimum number of modes to be taken into account, which is stated in the sub-chapter 4.3.3.3.1.

The Eurocode gives the following requirements that must be satisfied for a modal analysis:

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

When it is impossible to satisfy the condition given, the standard offers a simplified approach in the clause 4.3.3.3.1(5).

Whereas it is clear with the first criterion (one simply increases the number of modes until he reaches the 90% value), it might be quite complicated to find all the modes with the effective modal mass value higher than 5%.

The clauses of 4.3.3.3.2 specify the rules for the combination of effects of all the modes considered. This is the “maximum possible effect” described at the end of the clause 5.1.2 of this work. As for the code rules, a brief summarization is provided below, without a simple re-writing of the document.

If the periods between every subsequent mode differ by more than 10%, the Square Root of Sum of Squares (SRSS) may be used to obtain the maximum possible effect.

On the contrary, in case of closely spaced periods the standard requires to perform more accurate calculations, e.g. according to CQC-rule. CSI online encyclopedia explains that it is necessary to account cross correlation between mode shapes. Anyway, Designers' guide gives a wise piece of advice in his book: since nowadays calculations are performed with computer software, this won't be resource-consuming to use CQC rule in all cases.

### 5.1.6 Accidental torsion effects

The final requirement for MRS analysis in EC8 considers the effects from the accidental torsion eccentricity. The reason of this necessity is that there will never be a perfectly symmetric structure in reality, so it is a must to keep some degree of imperfection during a design procedure. According to CSI, the best way to account torsion is still through a static analysis. Therefore, in the sub-chapter 4.3.3.3 there are the rules for applying torsional effects on structure based on the lateral force method of analysis (even if a building does not satisfy the requirements for it). That will be described below.

## 5.2 Lateral force method of analysis

### 5.2.1 Definition and requirements

Although it was underlined before that a seismic action should be obtained by a dynamic analysis, there is still a possibility for a designer to use a linear static one.

In the chapter 4.3.3.2 Eurocode 8 offers a way to assess an earthquake effect on a structure by representing it as a set of lateral static forces. Obviously, this is not the most precise option, so it will not be a surprise to see that a structural frame has to meet certain conditions which are defined in the clauses of 4.3.3.2.1.

The main idea of Eurocode requirements is based on the structural simplicity needed. To substitute such a difficult phenomenon as an earthquake for a set of static forces, the building must not differ a lot from a simple pendulum discussed above. The purpose is to minimize the effect from all the modes other than the first while keeping the natural shape simple.

One may already notice that this method still requires performing a modal analysis. Therefore, it makes hand calculations resource ineffective. To eliminate this drawback, simplified rules for estimating a natural period are proposed in the sub-chapter 4.3.3.2.2. As an outcome we get the method that is much simpler than a modal response spectrum analysis. It is supposed to push a designer forward to structural simplicity.

The low influence of subsequent modes is achieved by natural period limitations. As for a natural shape simplification, the building is limited to be regular in elevation.

### 5.2.2 Effect determination

The force obtaining process is clearly described in the sub-chapters 4.3.3.2.2 and 4.3.3.2.3. To say the main idea briefly, a design spectrum is used to find the response acceleration value at a structure's natural period. Afterwards, it is multiplied by a design mass of a building during an earthquake and a correction factor to obtain its modal mass. The product equals to the total force applied to a building basement by an earthquake, also known as "seismic base shear."

The natural shape is considered to be simple and approximated by a set of lateral displacements. The static force applied to every storey is found as a relevant part of the total seismic base shear depending on a storey mass and its height above basement or its modal displacement in the natural vibration mode. One may observe a comparison of "simple" and "complicated" natural shapes (based on the regularity in elevation) in Figure 12. The first building is regular in elevation, while the all the mass of the second one is located on its top with a varying storey lateral stiffness.

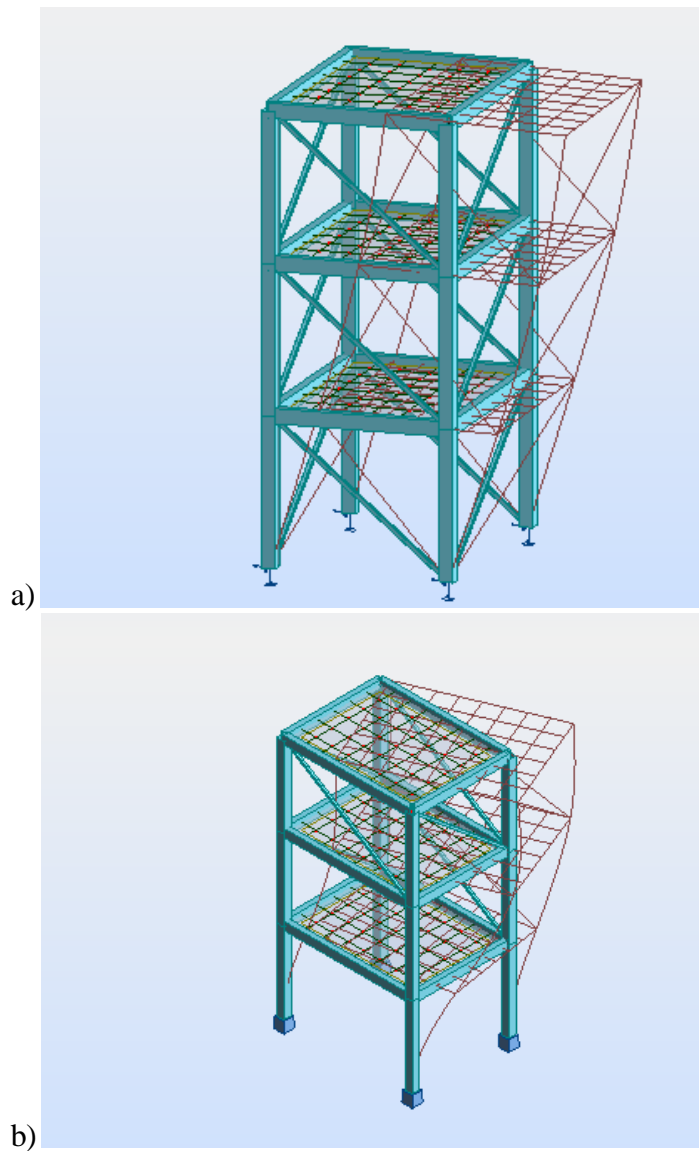


Figure 12 a - simple natural mode shape; b - complicated natural mode shape because structure main mass and stiffness is located on the top storey - seismic force cannot be evaluated with a set of storey-lateral forces.

In some cases, a later force method may serve as a quick check of whether the dynamic analysis was done correctly. A designer may have a good sleep if the total seismic base shear found by the static approach does not vary a lot from the MRS value of the first mode.

### 5.2.3 Combinations of seismic action components

Earlier it was mentioned that an earthquake effect must be divided into components: 2 lateral (in structure's X and Y direction) and, if needed, 1 vertical.

Although the standard requires to perform a separate structural analysis for each of them, the final effect (that will be used for resistance check) must be based on their simultaneous action according to the clause 4.3.3.5.1(1).

The word “simultaneous” does not mean simply adding one to another with a plus (or minus) sign.

For that purpose, various rules are given in the sub-chapter 4.3.3.5 not only for the two horizontal components, but for a case with a vertical one. Commonly, the method given in 4.3.3.5(3) is used in real design projects because of its simplicity and ease of application when it comes to software calculations. One thing that is not mentioned in EC, is that a designer has to create all kind of effect combinations with a plus and minus sign so as to obtain the force in all the directions, which will lead to bi-directional resistance check.

For the structures that are simple in plan with completely independent bracing systems in the two main directions, the clause (8) permits to treat the horizontal components separately, without estimating their simultaneous effect. The example of such a frame one can find in Figure 13. As it is notable, the relevant action component does not cause any stresses in the bracings of the other direction.

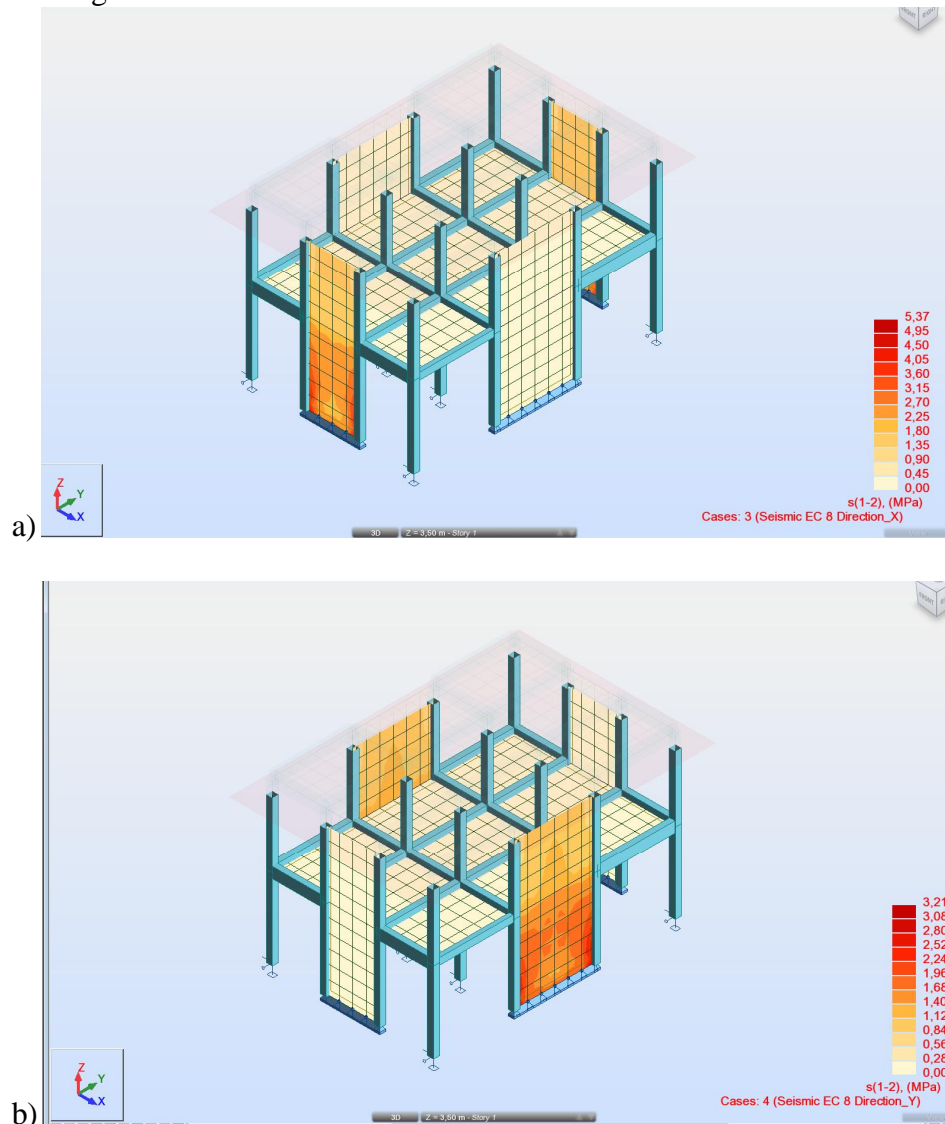


Figure 13 a - regular in plan building with the stresses from the seismic action component in X direction; b - the same, but in Y direction

## 5.3 SNiP

In the Russian standard it is stated nowhere what kind of analysis is used to determine internal member forces due to an earthquake. Instead of that, SNiP simply gives the formulae. However, the guidance for the original SNiP II-7-81 introduces a clarification. The statement 2.4 makes one treat a seismic effect as a static force. Based on the clause 5.5 it is allowed to assume masses and forces concentrated on structure's nodes (or on the gravity centers of rigid floor slabs for a cantilever model).

At first, it is still necessary to perform a modal analysis. Secondly, according to the expressions (1) and (2) of the mentioned sub-chapter, the design seismic action effect (either force or moment) for a node in the mode  $i$  should be found as follows:

$$S_i = K_0 * K_1 * m * A * \beta * K_\psi * \eta_k$$

Where all the  $K$  are various reliability coefficients,  $m$  is the mass of the structure associated with the node,  $A$  is the ground acceleration,  $\beta$  is the coefficient for period for the vibration mode  $i$  that depends on the curve shape (see 5.3.1 of this work) and  $\eta_k$  is the modal shape coefficient (also for the mode  $i$ ). Most of the values are very easy to find, the only uncertainty might occur with  $\eta_k$ . The formulae are given in 5.7 and 5.8.

a) For a spatial model:

$$\eta_{ik}^j = \frac{U_{ik}^j \sum_{p=1}^n \sum_{l=1}^3 m_p^l U_{ip}^l r_l}{\sum_{p=1}^n \sum_{j=1}^6 m_p^j (U_{ip}^j)^2}$$

Where  $l$  (from 1 to 3) is one of the 3 axis,  $j$  is the force direction (from 1 to 3 axial, from 4 to 6 moment about an axis);  $p$  indicates a node ( $n$  is the total number of all the nodes);  $m$  is the mass (or mass inertia moment);  $r$  is the cosine of the angle between action direction and the axis  $l$ ;  $U$  corresponds to the modal displacement (or rotation), whereas  $k$  is the node for which the effect is calculated.

b) For a cantilever one:

$$\eta_{ik} = \frac{X_i(x_k) \sum_{j=1}^n m_j X_i(x_j)}{\sum_{j=1}^n m_j X_i^2(x_j)}$$

In this case  $j$  is a node out of all the  $n$  nodes (floor gravity centers) and  $X$  stands for a modal displacement. As one may notice, in this case there is only one direction and the rotations are neglected. All the other letters represent the same as for the spatial formula.

So, one can observe a variation of a lateral force method of analysis that considers a certain number of vibration modes, but not only the first as in EC8.

Fortunately, it is not necessary for a designer to do the manual calculations. The reason comes from the official explanatory note to SNiP that was made by its authors. In that document it is clearly stated that the for-

mulae above in practice represent exactly the same effect as a modal response spectrum analysis.

### 5.3.1 Design spectrum definition

Nowadays, manual calculations may be found extremely seldom. This is a logical consequence of very high spread of computer software. In most of it, e.g. Autodesk RSA, a designer has to simply specify the reliability coefficients, while the rest is done by the program. Nevertheless, a seismic design according to SNiP might be quite an exotic feature in a computer program, whereas a modal response spectrum analysis is available almost everywhere. For example, in Autodesk RSA2015 at this moment there is no possibility to use earthquake design according to the latest SP.

This is why it is essential to explain how one can derive a design spectrum by himself.

#### 5.3.1.1. Ground acceleration

Design ground accelerations  $A$  in SNiP depends directly on the region seismicity, defined previously. It equals 1, 2 and  $4\text{m/s}^2$  for the site seismicity of 7, 8 and 9 degrees respectively (clause 5.5)

#### 5.3.1.2. Reliability factors

The coefficient  $K_0$  is based on the building importance and can be found in Table 3. Generally, it can be taken as 1 for the buildings of an average importance.

The coefficient  $K_1$  depends on the plastic and damage capacity of a frame. Basically, it has the same function as the behaviour factor  $q$  in EC8. It is found according to Table 4 of the current SP. The contractor has the right to select the  $K_1$  value equal to 1 if he does not want the building to have inelastic deformations or significant cracks.

As for  $K_\psi$ , it accounts the shape of a building frame and is taken in accordance with Table 5 of SP. For most of the structures it is 1, unless a building is too high and narrow in the plan (e.g. towers).

Table 3 Table 4 of SP14.13330.2014

Type of a structural frame	$K_1$
1. Frames with no damage or plastic deformations allowed	1
2. Ductile frames where insignificant damages are permitted with respect to people and equipment safety made of:	
Timber	0.15
Steel without vertical diagonal or diaphragm bracing system	0.25
Steel with the bracings listed above	0.22

RC walls	0.25
RC blocks	0.3
RC frame without vertical diagonal or diaphragm bracing system (moment resisting frame)	0.35
The same as above, but with masonry infill	0.4
RC frame with the bracings listed above	0.3
Masonry	0.4
3. Low-responsibility buildings	0.12

5.3.1.3. Curve shape factor

The factor  $\beta$  represents two possible spectrum curves. For the ground types I and II the clause 5.6 tells to use the first shape, the other one is for the soils III and IV. The spectrum shapes and the function of the factor can be found in Figure 14.

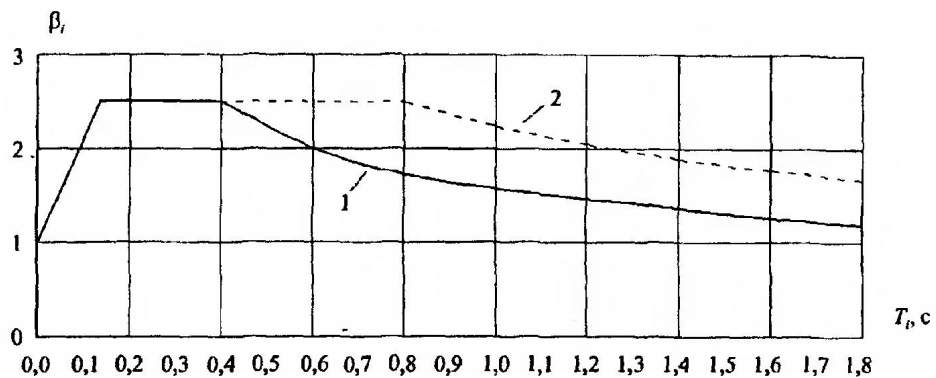


Figure 14 Two possible spectrum shapes from SP.

Shape 1:

$$T_i \leq 0,1 \text{ c } \beta_i = 1 + 15 T_i;$$

$$0,1 \text{ c } < T_i < 0,4 \text{ c } \beta_i = 2,5;$$

$$T_i \geq 0,4 \text{ c } \beta_i = 2,5(0,4 / T_i)^{0,5}.$$

Shape 2:

$$T_i \leq 0,1 \text{ c } \beta_i = 1 + 15 T_i;$$

$$0,1 \text{ c } < T_i < 0,4 \text{ c } \beta_i = 2,5;$$

$$T_i \geq 0,4 \text{ c } \beta_i = 2,5(0,4 / T_i)^{0,5}.$$

But  $\beta_i$  must always be more or equal than 0.8.

5.3.1.4. Design response spectrum formula

Based on everything above, one may use the following design spectrum in computer software:

$$S(T) = K_0 * K_1 * A * \beta * K_\psi$$



For the vertical response spectrum, the coefficient  $K_\psi$  must always be equal to 1, and the whole spectrum is multiplied by the factor 0.75 (clause 5.12).

### 5.3.2 Requirements for modal analysis

According to the sub-chapter 5.9 of the standard, the minimum number of modes to be taken into account should satisfy the following criteria:

- the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure for two horizontal directions and 75% for the vertical one;
- all modes with effective modal masses greater than 5% of the total mass are taken into account.

The combination of modal responses generally follows the same logic as EC8. The clause 5.11 tells that when the periods of sequent modes differ more than by 10%, the SRSS rule can be used. Otherwise, the correlation of modes is considered with the formula:

$$N_p = \pm \sqrt{\sum_{i=1}^n N_i^2 + \sum_{i=1}^{n-1} \rho_i |N_i N_{i+1}|}$$

Where  $\rho_i=2$ , if  $T_{i+1}/T_i > 0.9$ ; and  $\rho_i=0$ , if  $T_{i+1}/T_i < 0.9$

From this formula one can easily recognize the 10% combination method. However, SP gives certain requirements for the signs of outcome internal forces, as far as the combinations are quadratic. As specified in 5.11, the signs must correspond to those from the modes with the maximum modal mass involved. In most computer software, it is achieved through specifying a main mode for each direction. Unless so, the signs are taken always positive (because of squared values). It may result in having, e.g. both truss chords in compression (while the truss is in bending), like in Figure 15. As far as in a computer analysis displacements are calculated first, this does not anyhow affect a deformation shape (e.g. the shape of a bent truss with both compression chords will still be curved as if the lower one was in tension).

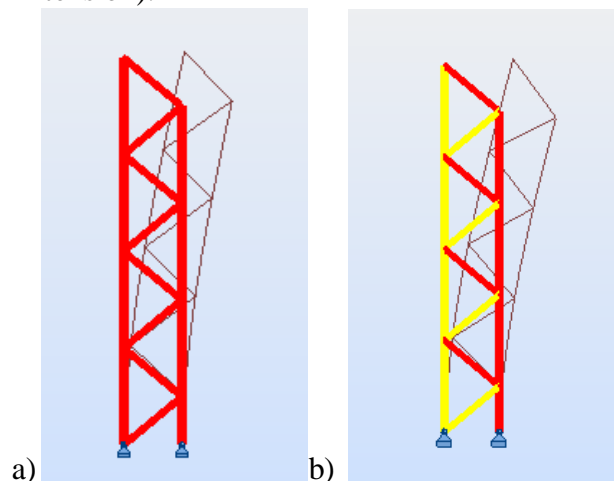


Figure 15 Stress state and deformation in the vertical truss. Red means compression, yellow is for tension. a) – the direct values after a quadratic combination were used; b) – the “main mode” was specified to define the sign of internal forces.

Since an earthquake may have any direction in reality, different positive and negative load combinations should always be specified in an analysis program. Therefore, one will get all the members subjected to both tension and compression values in different load cases. Generally, it is enough for members check, so the rule may be ignored, unless there are certain conditions described in the next sentences. The first one is when performing the stability verification for foundations, in that case having an axial tension and compression at the same time becomes vital. The second occurs when an engineer has to apply a static lateral force in combination with the results of a dynamic analysis. Usually, the only force of this type is wind load which is ignored in seismic combinations. Therefore, this case is quite rare.

Based on everything above, one is advised to use the described approach even when doing a seismic design according to EC8, though in the European standard it is said nothing about it.

### 5.3.3 Torsion effects

As for the torsion effects, the sub-chapter 5.16 of SP requires their considering only for a cantilever model if any building dimension in a plan is more than 30m.

The torsion is represented as a moment equal to the seismic shear multiplied by the eccentricity at least  $0.1B$ , where  $B$  is the relevant building dimension in the plan. The necessity comes from the various locations of a storey centers of mass and stiffness that is ignored in the calculation process of a cantilever. In a spatial model it will be accounted automatically and the standard does not demand any accidental eccentricity in addition to an actual one, contrary to EC8. However, a designer might want to ensure some safety. For that purpose he may either use the method from Eurocodes, or try to distribute loads unequally along the floor surface in purpose to reach a high eccentricity between mass and stiffness centers. Unfortunately, some sources claim that this is not possible for the live load as far as in both standards its quasi-permanent value is used which cannot be changed due to favorability. This idea comes from the standard EN 1990, where the favorable live load is multiplied by the partial factor  $\gamma_Q=0$ . Since in the seismic combinations the partial factors  $\gamma$  are not used, technically, one must not consider favorability at all. In spite of that, the standards carry a vague responsibility for collapsed buildings, that is why it is essential for an engineer to ignore some convenient rules and make a couple of analysis cases with all the live loads concentrated as far as possible from the stiffness centre.

### 5.3.4 Lateral force method of analysis

As it was earlier mentioned, SNiP does not distinguish its regulations for different analysis types, as it is done in Eurocodes. However, a plenty of simplification procedures are offered. Below one may find a set of requirements and regulations that allow him to determine internal forces in members with no practical difference from the EC lateral force method of analysis.

Firstly, a cantilever model must be used for the structural analysis.

Secondly, according to the clause 5.9 only the first vibration mode can be considered if the structure corresponds to the regularity criteria and has a natural period smaller or equal than 0.4s.

As the last criterion, a building must have five or fewer storeys, and their mass and stiffness must not differ significantly (almost the same as EC regularity in elevation). This is due to the clause 5.8

These obligations lead to the following benefit:

The coefficient  $\eta_k$  is defined according to the formula 7:

$$\eta_k = \frac{x_k \sum_{j=1}^n m_j x_j}{\sum_{j=1}^n m_j x_j^2}$$

In this case  $x$  stands for the distance of the mass to the top of the foundation, while  $j$  and  $m$  mean the same as in the previous formula.

Therefore, the lateral force applied to a node based on the formula (2) and re-written to replace various importance factors with the design spectrum value is:

$$F = S(d) * m * x_k * \eta_k$$

It is notable that an approach common to EC8 rules for a simple natural mode shape is used.

### 5.3.5 Combination of seismic action components

The rules for the combination of the components are given in the clauses of 5.3. According to them, the seismic action in reality may have any direction (disregarding the axis vector of a structure).

Anyway, SNiP also offers some simplifications in this term. For the building frames that correspond to regularity criteria the standard allows not to combine horizontal components. In this case, the earthquake force must act separately along the structure's two main lateral axes.

For the buildings outside the regularity requirements an earthquake force must not be divided into components, its spatial effect must be considered in order to account a possible torsion and non-rigidity of floors. The direction of the resultant must cause the maximum possible internal forces in structural members. In most cases, those can be obtained when applying the force along the mean line of the two bracing systems (45 degrees in case of orthogonal bracings).

When the vertical component is considered, it must always act at the same time with the two (or each of them separately) horizontal ones.

### 5.3.6 Vertical component of a seismic action

SNiP follows the same logic as Eurocodes, so generally no vertical action of an earthquake is considered. In the clause 5.4 one can find the cases when it is vital (hence, accounted in calculations). The vertical component must be considered in the design of:

- Any kind of cantilevers
- Span-elements of bridges
- Elements with the span of more than 24m
- Structure design against equilibrium loss
- Masonry buildings.

### 5.4 Non-structural elements

It is common knowledge that any building consists not only of a load bearing frame. There may also be partitions, parapets and stair handlings. These elements are usually considered non-structural and accounted in modelling only by their self-weight. However, this is not the case with an earthquake. The reason is that a seismic phenomenon applies lateral dynamic loads to everything that has mass. This is why even a partition wall that does not carry anything during everyday life can get broken when subjected to an earthquake. To avoid it, both standards introduce a verification procedure.

#### 5.4.1 Eurocode 8

In Europe, only the elements that can be dangerous for people or the building itself during an earthquake must be verified in a seismic design situation. In EC8, the whole chapter 4.3.5 is dedicated to provide needed instructions. The European norms demand an engineer to account non-structural elements in the three following ways, depending on their importance:

- Elements that may influence the dynamic response of a building must be modelled with their connections.
- Parts that are of great importance have to be analyzed based on their realistic dynamic model and appropriate design spectrum.
- All the other elements can be designed by applying a static force to them as it is described further.

The simplified procedure for the third type is written in 4.3.5.2 to 4.3.5.4. These clauses demand applying a static force to the gravity centres of elements. One should also keep in mind the transmission of the force applied to the main structure which is especially vital in case of fixing design. So, the standard gives the following expression to determine an earthquake force for non structural elements:

$$F_a = (S_a \cdot W_a \cdot \gamma_a) / q_a$$

Where:

- $\gamma_a$  is the importance factor from 4.3.5.3 (typically equals 1, unless an element is dangerous either to people or to environment)
- $q$  is the behaviour factor for the non-structural elements that is found in 4.3.5.4 in a much simpler way than this factor for a design spectrum
- $W$  is the weight of an element
- $S$  is the seismic coefficient derived in 4.3.5.2(3). Its purpose is to perform a lateral force method of analysis for a non-bearing element with only the first vibration mode accounted. The element is assumed to rest on a moving tough (hard as the ground of A type) surface.

#### 5.4.2 SNiP

In contrast to Eurocodes, the Russian standards do not specify any general rules for all the type of non-structural elements. However, some typical building parts belonging to this category must be designed to resist seismic action. Based on the sub-chapters 5.12 to 5.14, the following design methods are applicable:

- Partition walls, technical equipment joints and concrete panel joints must be designed to resist a horizontal earthquake force defined by the general formulae above. The value of  $\beta_\eta$  is the same as for the relevant building storey, but not less than 2.
- Secondary structures continuing upwards (e.g. parapets) with an insignificant mass comparing to a building itself in addition to the joints of heavy objects on the ground floor (e.g. monument fixing) have to be able to withstand the horizontal seismic action based on the general formulae. But in this case the product of  $\beta_\eta$  is always equal to 5.
- For the cantilever structures with a relatively small mass (for example, balconies) must be designed for the vertical seismic action with the  $\beta_\eta$  value of 5. This is very convenient for a MRS analysis, because a designer does not need to fully consider the vertical component that will be added later as a simply calculated static force (but only if these light cantilevers are the only components that must resist a vertical earthquake impact).

## 6 VERIFICATION AT DIFFERENT LIMIT STATES

In both standards the limit state system is used. “Steel structures” define it as the state of a structure during which it is not anymore suitable for usage. Though it is not directly related to the topic of this thesis, a short introduction to the limit state systems in Europe and Russia will be given.

Nowadays the most common limit states are ultimate and serviceability states. In the previous chapter the damage limit state system was introduced. It has many things in common with the second one.

In the Russian Federation one will find only two cases: 1<sup>st</sup> limit state and the 2<sup>nd</sup> one. They mean exactly the same as the ultimate and serviceability states respectively.

### 6.1 Europe

In Eurocode 8 the verification process demands are written in the part 4.4. First of all, it is mentioned that no verification is allowed to be done if the design lateral force from a non-seismic linear elastic situation is bigger than that obtained via an earthquake analysis with the low behaviour factor. It is quite obvious, because internal member forces will not exceed the resistance value that was chosen without considering the plastic capacity of a material. Anyway, some general requirements still must be met as it is stated in 4.4.1.

#### 6.1.1 Ultimate limit state

The verification procedure must correspond to the fundamental design equation  $E_d < R_d$ . Where  $R_d$  is the material resistance based on the relevant parts of the Eurocode in addition to specific ductility requirements from EC8. As for the effect, it is represented by the seismic design situation force with second order effects accounted.

These effects (also called P- $\Delta$  effects) are usually obtained via a non-linear analysis. However, the MRS method is strictly linear, so it cannot perform the iterations necessary to obtain internal second order moments. This is why the standard uses the same approach as material-specific Eurocodes, which involves a simplified method.

As it is written in 4.4.2.2, the necessity to consider P- $\Delta$  deformations depends directly on the ratio of the secondary moments from the occurred eccentricities to the initial moments at a storey level. Resulting from how small it is, various regulations (from performing a full second order analysis to completely ignoring its impact) have to be obeyed. In practice this means that in case of a high storey displacement a designer is capable of selecting more stiff bracing elements that allows him to perform a simplified analysis. It must also be noted that no global initial imperfections according to EC2 and EC3 are applied to a building.

Besides, a building must have adequate ductility as it will be described in the next chapters, which allows reducing the characteristic elastic spectrum to a designed value.

As one more obligation, EC8 introduces special rules for seismic joints that may be found in 4.4.2.7 of the standard. The main idea is to limit the displacements of nearby buildings so that no collision takes place.

### 6.1.2 Damage limitation state

Eurocode 8 determines the damage limit state as the limitation of an inter-storey drift. One might notice the same logic as for the serviceability limit state. At this limit state an earthquake with a higher return period must be chosen, which means it will be weaker than a reference one. The reduction factors and displacement limitation values may be obtained from 4.4.3.

## 6.2 SNiP

The Russian standard recommends checking building resistance throughout an earthquake only at ULS (point 5.18). Anyway, in the same way as in Eurocodes it is stated that a customer may give his own reason to check SLS and an undesirable damage or deformation of certain elements. It must be noted that the official guidance does not allow bending moment redistribution in RC frames for non post-tensioned main beams (clause 3.73), contrary to EC8.

Besides, an additional safety factor for material resistance named “working condition coefficient” must be used at ULS at the same time with all other factors, including the working condition coefficient for an average design situation (5.15). It is taken from Table 6 of SP.

### 6.2.1 Seismic joint requirements

In contrast to EC8, SNiP treats seismic joints in a more detailed way. Almost the whole sub chapter 6.1 is dedicated to this question. So, according to the rules of SP the following summarizations can be made.

These kinds of joint becomes obligatory whenever a structure has a complicated plan shape or nearby parts with the height inequality at least 5 meters and significant differences in mass or stiffness.

A seismic joint should typically be realized through twin frames or walls. In addition, a reasonable gap width requirements are given in 6.1.6 to be not less than the sum of design structural displacements of the nearby parts with some minimal borders. One must keep in mind that the displacement calculation follows exactly the same logic as EC8, which means that according to Table 4 of SP the coefficient  $K_1$  (being an equivalent to a behavior factor in the European norms) for the deformation assessment must be 1 in all cases.

A seismic joint is classified as a potentially dangerous structure, so it must not exist at the building parts with a constant occupancy according to 6.1.2.

## 7 SPECIFIC MATERIAL REQUIREMENTS

### 7.1 Methods of energy dissipation

In the design spectrum definition part for both codes one faced the variable named “behavior factor” that accounts plastic capacity of a material. While a building subjected to an earthquake force  $F_{ed}$  cannot withstand it based on the theory of elasticity, in reality the internal stresses in critical regions will be lower for the reason of plastic deformations. For example, if a diagonal bracing made of elastic-perfectly plastic material is attached to a column and subjected to axial tension, the column will never have to resist the force more than the plastic resistance of the diagonal: the force physically cannot get bigger. No matter how strongly the bar is pulled, the material will yield and the bracing will elongate, leaving the internal tension equal to the plastic resistance. If the bracing was elastic, the internal stresses in the column would increase linearly. The modal response spectrum analysis assumes all the members perfectly elastic, while in reality they are not, which is beneficially used in the seismic design. This phenomenon is known as energy dissipation. There are plenty of ways to achieve it, the method above (with plastic deformations) is called “energy dissipation through material ductility.”

Both norms accept ductility as one possible way to reduce the design seismic force, as stated in 4.1 of SP and 2.2.2 of EC8. Other methods, e.g. base isolation, are outside the scope of this thesis because any building may be designed based on the ductility requirements.

Sometimes, the energy dissipation factor allows a designer to reduce the seismic force by more than three times. For sure, no one has ever seen a beam that is three times stronger in plastic resistance comparing to the elastic one. According to “Steel Structures”, a rectangular solid section made of a ductile material can resist only 1.5 times more moment than the same beam if it was brittle. (U. Kudishin, E. Belenya, V. Ignatyeva, A. Pukhovskiy, G. Vedenikov, B. Uvarov, S. Astryab, V. Val’, U. Sokolov, T. Morachevskiy, D. Streletskiy - “Steel structures” «Металлические конструкции»)

This is why it is obligatory to ensure that during an earthquake a building will develop plenty of plastic hinges to reach a plastic mechanism. The more plastic hinges are needed for a mechanism, the greater amount of energy will dissipate in them.

The conclusion of this sub-chapter may be the following: it is not enough to use a material which can develop plasticity in order to reduce a seismic force. A frame must meet the ductility requirements to ensure that there will be enough plastic deformations in reality. These demands in EC8 and SNiP will be explained further.



## 7.2 Soft-storey mechanism

There is one general method to maximize the amount of plastic hinges. In multi-storey frames inelastic deformations must occur only in bracing elements or beams, but not in columns. The reason is that if columns fail, the mechanism may be locked within one storey. Therefore the building will have not too many plastic hinges. This kind of a mechanism where plastic hinges develop only in columns is called “soft-storey.” One will find its illustration and comparison to bracing-failure mechanism in the Figure 16. As it is notable, the number of plastic hinges can be much bigger when the bracing elements yield and the columns remain elastic.

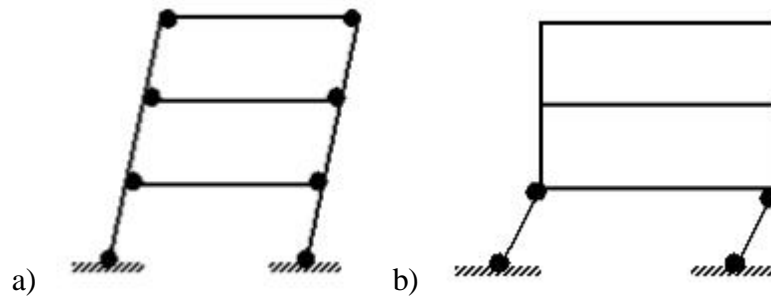


Figure 16 Different failure mechanisms for a moment resisting frame: a – bracing elements failure, b – soft-storey mechanism.  
([http://www.luxinzheng.net/publication1/PF\\_THU.htm](http://www.luxinzheng.net/publication1/PF_THU.htm))



Figure 17 A soft-storey mechanism example  
([http://en.wikipedia.org/wiki/Soft\\_story\\_building](http://en.wikipedia.org/wiki/Soft_story_building))

As one can see in Figure 17, bracing failure demands many more plastic hinges and the number needed increases with number of floors. This means a lot of energy will be dissipated if a designer is capable of avoiding the other failure.

This is why Eurocode forbids a soft-storey mechanism in the clause 4.4.2.3 of EC8. As for SNIIP, it only recommends avoiding it in the statements 6.8.1 of SP and 3.63 of the guidance to the original SNIIP.

### 7.3 General requirements for structural frames

Independently of the material used, the two building codes introduce various demands for structural frames.

#### 7.3.1 EC8

In Eurocode there is only one fundamental equation (number 4.29) that must be satisfied for all joints of primary seismic beams and columns at all storeys except the last one:

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$$

Where  $\sum M_{Rc}$  is the sum of moment resistance of columns framing the joint and  $\sum M_{Rb}$  is the same for beams.

This is the general way to prevent a soft-storey failure.

##### 7.3.1.1. Capacity design of members

From the previous chapter it can be understood that it is desirable for one structural member (a beam) to fail before the other (a column). In that case the resistance of the latter has to be bigger. This approach is called “capacity design” and is the main one used in Eurocodes to ensure a building ductile behaviour. Columns are capacity designed to be able to carry the load while the beam has already failed.

Unfortunately, an engineer faces the fundamental problem of structural design when using this method. One is able to calculate  $M_{rd}$  of a beam, but the formulae from Eurocode involves plenty of safety factors. This means that in reality a beam may be much stronger than estimated by calculations. Usually, it is a great advantage, but not in the seismic case. So, a designer cannot be sure that a beam actually fails before the column. Although equal safety factors are used, they may correspond to reality in case of a vertical member and be too much on the safe side for the beam.

For that purpose EC8 removes all the safety factors for the dissipative elements, because they are expected to reach the limit first. The values for the non-dissipative ones stay the same. Moreover, even characteristic values for materials correspond to 95% of overestimation. This is why it is wise to take the resistance of dissipative members even bigger than characteristic values. This makes a designer totally sure that plastic deformations (failure) will take place at the desired location.

All the rules above are written particularly for every material and frame type in the relevant section of EC8. As a conclusion from here, a designer must remember the following: since energy dissipation is required to hap-

pen through a certain ductile behaviour, e.g. flexural bending, at a certain location, e.g. beams, all the other members and failure modes, e.g. shear, must be prevented by the rules of capacity design.

### 7.3.2 SNiP

SP has plenty of advice and regulations when it comes to the building frame. However, most of them have “detailing” meaning containing the rules for e.g. maximum building height depending on the frame type. All the rules are simply written in the sub-chapters 6.1-6.7 of SP. Nevertheless, there are some recommendations, especially in the official guidance that promote to the energy dissipation through ductility, besides the demands written in 7.2 of this thesis.

Clauses 6.92 and 6.84 of SNiP obligate to locate prefabricated column joints at a position of low bending moment values. Although in a calculation model a column is uniform and solid along its height, which means that joints are supposed to rigidly connect one prefabricated element to another, at a construction site errors are more likely than at a workshop. If it is possible, the standard encourages a single prefabricated column to cover as many floors as possible. This prevents accidental occurrence of a soft-storey mechanism.

## 7.4 Reinforced concrete structures

### 7.4.1 Eurocode 8

The whole chapter 5 of the European norms is dedicated to the design and detailing of reinforced concrete structures. All the rules are clearly explained in 140 pages of the document. Therefore, to avoid unnecessary repetition of all the expressions, within this thesis only the basic information is provided. As a result a reader will have the image of RC structures behavior under earthquake conditions, which helps him to approach the particular rules with a sufficient degree of understanding.

#### 7.4.1.1. Frame types

Concrete as a material is an extremely convenient material because it allows an endless number of various geometric shapes to be used in elements. Therefore, energy may be dissipated not only in prismatic members, but also in other elements like walls. So, the standard distinguishes the following frame types with different requirements for each of them:

- a) frame system
- b) dual system (either frame or wall equivalent)
- c) ductile wall system (coupled or uncoupled)
- d) system of large lightly reinforced walls
- e) inverted pendulum systems
- f) torsionally flexible system

These are illustrated in Figure 18.

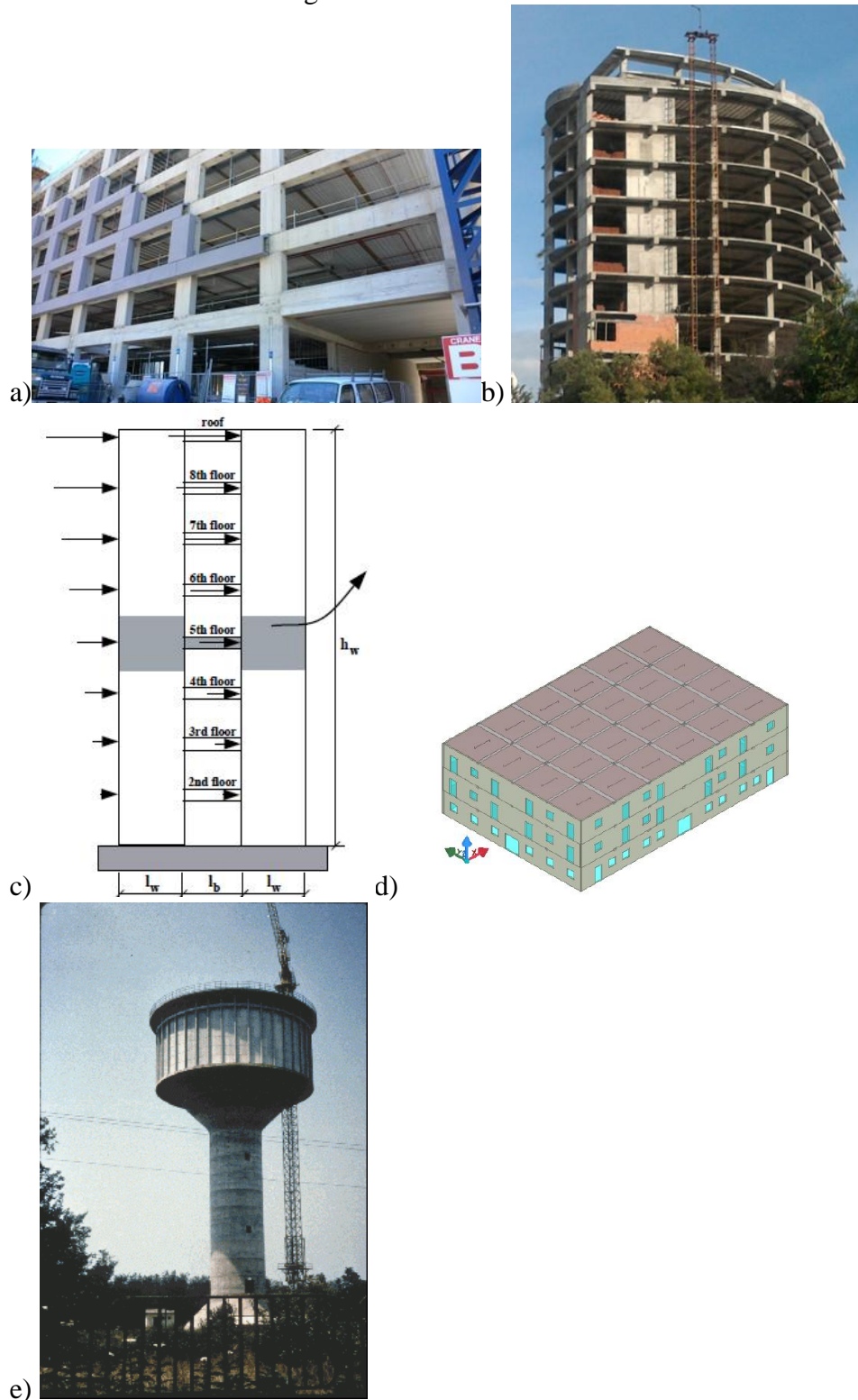


Figure 18 Different load resisting systems  
(<http://www.johnmartin.com/earthquakes/eqshow/images/64701417.GIF>;  
[http://www3.nd.edu/~concrete/research\\_files/Precast\\_Coupling\\_beams\\_for\\_walls\\_files/precastcoupling1.pdf](http://www3.nd.edu/~concrete/research_files/Precast_Coupling_beams_for_walls_files/precastcoupling1.pdf);  
<http://www.nexus.globalquakemodel.org/gem-building-taxonomy/overview/glossary/moment-frame--lfm>)

Every type has its own behavior factor and a building can be classified separately into its two main orthogonal directions, which is very convenient for portal frames. However, it cannot be done for the last type because the earthquake effect in that case is quite spatial.

A building is considered to have the frame lateral resisting system if at least 65% of the total seismic base shear is resisted by spatial frames, e.g. moment connections of beams and columns. The energy is dissipated in the plastic hinges of beams according to the general rule to prevent the soft-storey mechanism.

A ductile wall system is one where 65% or more of the seismic base shear is counteracted by ductile walls. It can be coupled, if most of the wall-resistance is governed by the so-called “coupled walls” or uncoupled respectively. If two or more single walls are connected uniformly along their height with special beams and this results in the reduction of the sum of the wall base bending moments by at least 25% comparing to the case without those beams, the walls are considered coupled. An example can be found in Figure 18, case c). The plastic hinges need to be formed at the base of walls and in coupling beams, if there are any.

A dual system represents the combination of the two above, when the values are between 35% and 65% for both frames and walls. It can be a wall or frame equivalent, depending on whether most of the base shear is resisted by walls or frames respectively.

As for the system of large lightly reinforced walls, the standard defines it as “a wall system if, in the horizontal direction of interest, it comprises at least two walls with a horizontal dimension of not less than 4,0 m or  $2h_w/3$ , whichever is less, which collectively support at least 20% of the total gravity load from above in the seismic design situation, and has a fundamental period, for assumed fixity at the base against rotation, less than or equal to 0,5 s.” This differs a great deal from all the other ones because of its energy dissipation method. In the walls mentioned, it is achieved not through a ductile behavior, but via their large stiffness. Being almost rigid large walls are able to temporarily uplift the mass resting on them and transform the earthquake energy to the potential one of that mass. The rest is dissipated by soil.

An inverted pendulum frame has more than half of the total mass located at the highest third of a building.

A building is considered torsionally flexible if it does not satisfy the expression 4.1b from EC8 which is a part of regularity in the plan criteria.

### 7.4.1.2. Ductility classes

In a usual design situation an engineer has the opportunity to optimize the ratio of steel to concrete so as to create cost-effective solutions. For earthquake conditions Eurocodes also give a possibility to choose between a high reduction of a design seismic action accompanied by the plenty of de-

tailing rules up to leaving the spectrum value in an elastic range and paying no attention to dissipative behavior at all.

The division is achieved via so-called “ductility classes”. In total, there are three of them: low, medium and high (DCL, DCM and DCH respectively). The higher the class, the less will be the earthquake effect, but a designer will have to use more reinforcement and concern many rules. The document introduces the ductility types in Chapter 5.2.1 and provides various detailing rules for each of them.

It is always up to an engineer to choose the degree of energy dissipation, anyway EC8 does not recommend the DCL-approach for buildings except in low-seismicity regions.

#### 7.4.1.3. Behavior factor definition

From the design point of view, the main goal of the detailing for ductility is to reduce an earthquake effect with a behavior factor  $q$ . For RC structures it is defined based on 5.2.2.2 by the following expression:

$$q = q_0 k_w \geq 1.5$$

Where  $q_0$  is the basic behavior factor and the coefficient  $k_w$  defined by 5.2.2.2(11) is needed only for wall and torsionally flexible systems. Its purpose is to ensure a sufficient wall slenderness, so that flexural deformations develop.

The basic values are obtained according to Table 4:

Table 4 The values of  $q_0$

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3,0\alpha_u/\alpha_1$	$4,5\alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4,0\alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

For the buildings that are non-regular in elevation the table value must be reduced by 20%.

The final  $q$  value can reach the magnitude of 6 with the help of the ratio of  $\alpha_u$  to  $\alpha_1$ , defined in the sub-clauses (4) to (8). Alpha coefficients are the values that one multiplies by the design seismic action so as to obtain: the first plastic hinge for  $\alpha_1$ ; a plastic mechanism for  $\alpha_u$ .

The previous procedure looks quite laborious, because besides iterations, a non-linear analysis must be used. The amount of work may be cost-ineffective, so the document offers simplification methods in the sub-clauses mentioned. The result depends on the building regularity in the plan.

A reader may also notice the minimum value of 1.5 that is considered reasonable due to a high probability of greater material capacity and usual rounding of needed reinforcement value. (Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioll, Paolo Pinto, Andre Plumier “Designers’ guide to Eurocode 8: design of structures for earthquake resistance”)

#### 7.4.1.4. Possible material overstrength

Concluded from the capacity design philosophy, it is on the safe side to consider a material stronger than its characteristic value. For that purpose the document introduces a special factor  $\gamma_{Rd}$ . This is only one coefficient, but it accounts several phenomena, such as strain hardening of steel and the 95% exceedance of a characteristic value. The magnitude depends on the formula and a ductility class needed. For DCM it varies between 1 and 1.1, whereas for DCH the value of 1.3 can be stated.

#### 7.4.1.5. Capacity design of RC members

Most of the pages of EC8 chapter 5 are devoted to the capacity design in order for the plastic deformations to locate at the places of interest for every frame type, while preventing other failure modes during the dissipation process.

Firstly, the document determines to avoid the shear failure of RC elements. As it was stated previously, the best way to dissipate energy is through flexure. In the case of a steel member, plastic shear deformations may also be useful, since beam elements are mostly done from the profiles non-susceptible to shear buckling. Unfortunately, it cannot be arranged with concrete because the material is extremely brittle. Although RC elements comprise transverse reinforcement, the cycling nature of an earthquake effect gives a potential danger of crushing the concrete in compression. (Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioll, Paolo Pinto, Andre Plumier “Designers’ guide to Eurocode 8: design of structures for earthquake resistance”)

So, it is necessary that dissipative members do not fail in the shear manner not only in a seismic design situation, but also during the formation of plastic hinges. When a frame is subjected to lateral loads, in a beam connected rigidly to a column opposite bending moments develop at beam ends, resulting in a constant shear force throughout the beam, as illustrated in Figure 19.

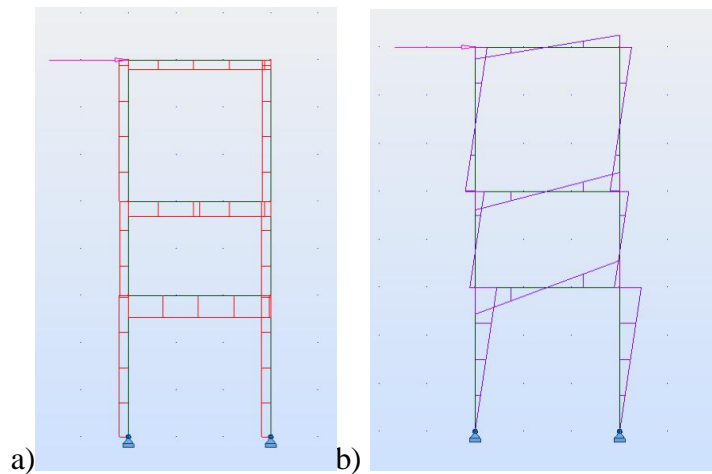


Figure 19 Internal forces due to lateral load: a – shear force, b – bending moment

So, the goal for primary seismic beams is to withstand the shear resulting from gravity loads and moments at beam ends. The moments obviously are taken to be the plastic capacity of a section multiplied by an over-strength factor. In the document this situation is described in 5.4.2.2.

Even more dangerous situation occurs for primary seismic columns. Besides avoiding plastic hinges, they have to withstand the shear stress from the joint that are shown in Figure 20.

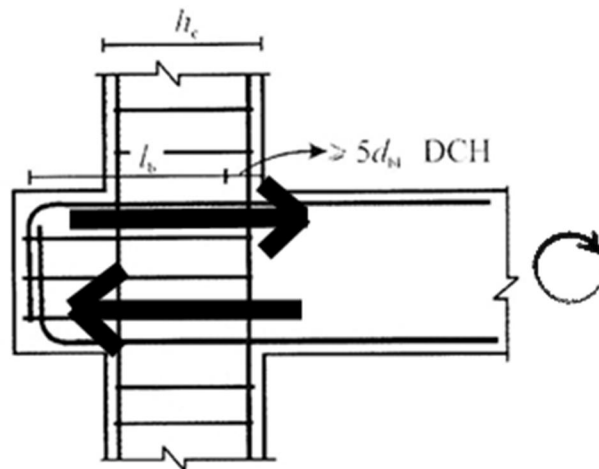


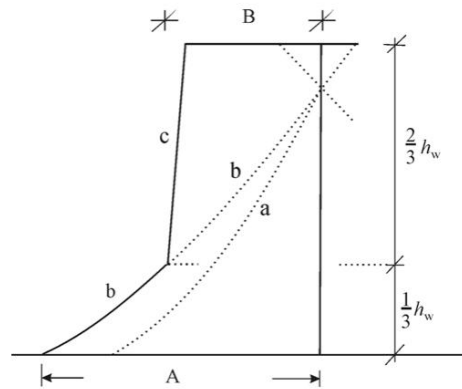
Figure 20 Shear force in a column due to a moment connection (EN1998-1)

The main danger of this phenomenon is that it is not covered by the FEM analysis of beam-elements, because they are represented as lines without an actual cross section. So the clause 5.5.2.3 introduces the method to estimate it manually. However, it is needed only for DCH. For all other classes the resulting shear is assumed to be resisted enough by general ductility detailing rules.

In addition to bar-members, walls must also be much stronger in shear than in bending. Moreover, when a plastic hinge occurs at a wall base, shear forces might increase for the reason of bigger lateral forces, causing a plastic hinge formation. The rules are given in 5.4.2.4 to account that.



Besides, for slender walls it is agreed to consider the effect of higher modes. As one can see in Figure 10 of this thesis, the second mode creates shear forces that are distributed less uniformly along the height. So, it is assumed to replace the design shear force diagram (envelope) with the one in Figure 21 below:



- Key**
- a shear diagram from analysis
  - b magnified shear diagram
  - c design envelope
  - A  $V_{\text{wall,base}}$
  - B  $V_{\text{wall,top}} \geq V_{\text{wall,base}}/2$

Figure 21 Assumed shear force distribution for slender walls (EN1998-1)

One more criterion is the obvious need of only certain frame types to satisfy the fundamental equation 4.29 for preventing the soft-storey mechanism. Since for wall systems it is desired to dissipate energy in the basement, it is purposeless to design columns against the soft-storey failure manner: they are not primary seismic members. The full list of capacity-design requirements is written in 5.2.3.3 of EC8.

#### 7.4.1.6. Material requirements

Obviously, not all the materials manufactured nowadays are able to provide a sufficient ductility. This is why to ensure a needed number of plastic deformations without a fracture or brittle failure, the standard introduces specific obligations for concrete and reinforcing steel used in manufacturing of primary seismic elements.

As it was stated earlier, the demands depend directly on the ductility class of a building. For example, for DCL the only requirement is the usage of B or C class of reinforcing steel, while DCH approach will need concrete class at least C20/25 and reinforcing steel of class C, which has the best elongation at ultimate stress. All the rules are specified in the paragraphs of EC8 devoted to a relevant ductility class design.

## 7.4.2 SNIp

The Russian building code introduces a lot of obligations for RC frames in addition to typical solutions in the official guidance. All can be found in 6.7 and 6.8 of SP. In this thesis only the demands that contribute to energy dissipation will be observed, with the links to relevant clauses of the standard afterwards.

### 7.4.2.1. Frame arrangement and types

In contrast to the European norms, SNIp does not offer that much variety in lateral force resisting systems paying more attention to the elements that a building is made of. Within this thesis only moment resisting frames, frames with bracings and pure wall systems (they can be found in Figure 22) will be covered, since the rest involve mostly masonry prefabricated-elements. In the second type the bracings can be arranged via diagonal bars or shear walls (diaphragms).

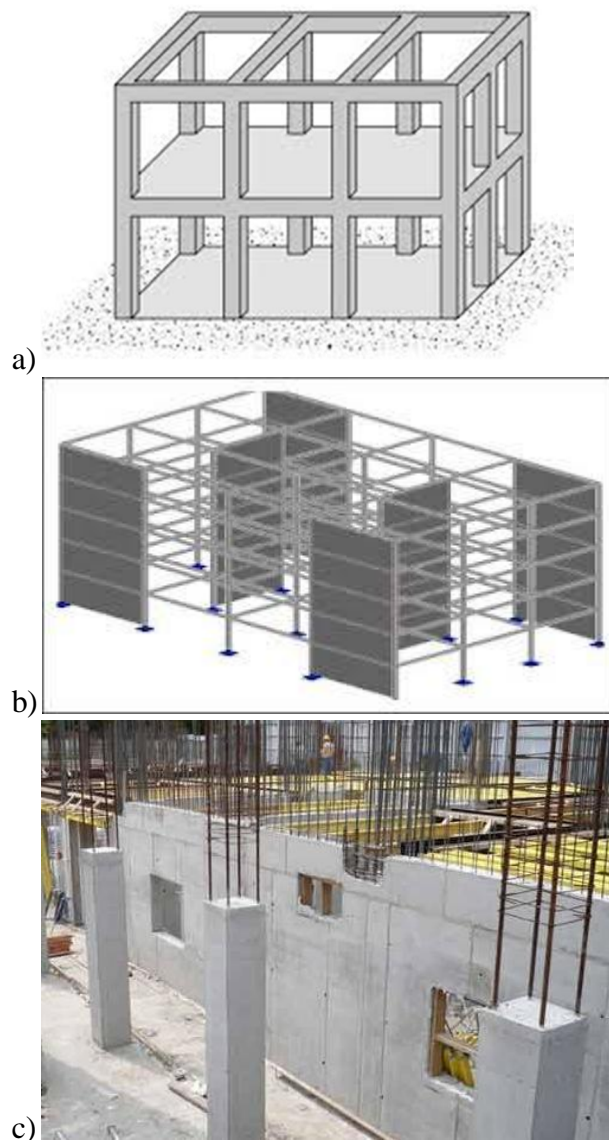


Figure 22 a – moment resisting frame, b – frame with bracings, c – pure wall system  
(<http://www.nexus.globalquakemodel.org/gem-building-taxonomy/overview/glossary/moment-frame--lfm>)

The behavior factors differ for all three kinds, so do the requirements. Anyway, there are general regulations for planning that encourage the ductile response of a building.

At first, the clause 6.8.1 forbids the buildings with more than 9 floors to consist of a moment resisting frame, bracing elements must be provided. In my opinion, this is needed to maintain small lateral deformations. The reason is that in the Russian standards no global second order effects are accounted (even for a usual design situation). So, the only way to limit their impact is to ensure that a building is horizontally stiff enough, which is difficult to be achieved in tall moment resisting buildings.

Secondly, when the stiffness of the first floor is smaller than others, the standards recommend make the columns of steel.

#### 7.4.2.2. Rules for moment resisting frames and frames with bracing elements

The regulations below help to achieve the required energy dissipation capacity and are obligatory for both moment resisting buildings and frames with walls or diagonal bracings.

##### 7.4.2.2.1. Rotational capacity of a section

As it was stated earlier, the only way to dissipate energy in reinforced concrete structures is through flexure. This is why elements with plastic hinges must have an adequate rotational capacity, which is governed by the standard.

At first, the relative height  $\xi_R$  of compression zone in sections subjected to bending must be limited in accordance with the usual design rules of the current SP about RC structures. Besides, the limit value must be decreased by multiplying it by the safety coefficients 0.85, 0.7 and 0.5 for the site seismicity of 7, 8 and 9 degrees respectively. The demand aims to prevent concrete reaching its ultimate strain by keeping the neutral axis position higher than usual. This prepares a section for greater rotations in reality than assumed during the design stage. However, if the analysis is non-linear the requirement should be ignored, as far as the rotations and plastic capacity will already be accounted. (6.7.2)

Secondly, the reinforcement must also have an adequate elongation limit so as not to fracture during an element rotation. Besides the allowed rebar classes described in 6.7.3-6.7.7, there is also the demand for all the reinforcement to have an ultimate strain not less than 2.5%.

#### 7.4.2.2.2. Additional anchorage and lapping requirements

It has always been the biggest problem in RC structures to ensure a correct ending of rebars in order for them to adequately transform axial forces. So, one will not be surprised to see that anchorage for energy dissipation elements needs to withstand higher forces to allow a sufficient plastic deformation in members.

When the design seismicity is 9 degrees, rebars with the diameter of more than 28mm cannot be used without special anchorage devices. It is a common knowledge that large-diameter bars are difficult to be anchored. (6.7.8)

Whenever the rebars are arranged by tying stirrups, the latter must be anchored by bending their ends around longitudinal bars towards the gravity center of a section. The continuing distance after the bent must be at least 6 times the diameter of a stirrup. (6.7.11)

Lapping of longitudinal reinforcement without welding is allowed; however, it must correspond to extra demands:

- The bar diameter must be less than 20mm
- When the design seismicity is 9 degrees, special anchorage devices should be provided
- The design lap length according to the current RC-SNiP is increased by 30%
- In the lapping zone transverse reinforcement must be provided with the spacing not more than 8 times the diameter of a rebar.
- Not more than 50% of tensile reinforcement is allowed to lap within one section.
- Prefabricated columns with the length up to 10.7m must consist of solid longitudinal bars without any lapping or welding.
- In prefabricated columns lapping is totally forbidden.

When lapping is not allowed, the transmission of forces between reinforcing can be arranged via welding. At first, it is necessary to introduce the desired method of welding. For usual, non-seismic buildings bars are commonly lapped and then welded. This is not the best way for an earthquake case, since a big rotational capacity is required for energy dissipation. Therefore, most of the examples in the official guidance recommend so-called “bath welding”, when bars are placed inside a special bracket and melted metal is poured inside. It is illustrated in Figure 23. It provides a better welding quality comparing to a usual method. (6.7.12)

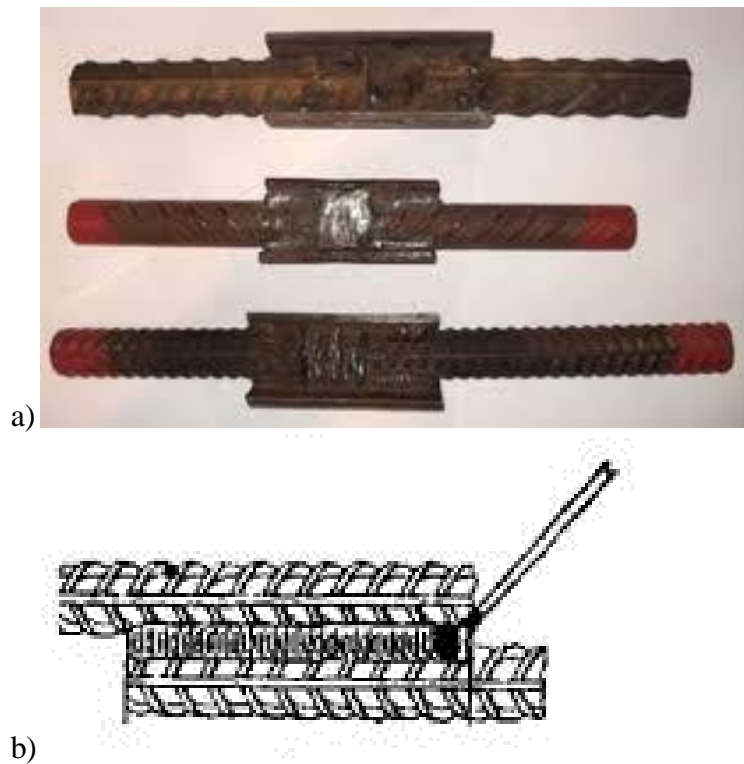


Figure 23 a – bath welding; b – usual lap welding  
(<http://www.gosthelp.ru/text/svarkametallokonstrukciji.html>;  
[http://www.zaopkti.spb.ru/services07\\_416.html](http://www.zaopkti.spb.ru/services07_416.html))

#### 7.4.2.2.3. Capacity design against shear failure and stirrup arrangement

In the same manner as in Eurocodes, the Russian standards pay attention to preventing a shear failure of elements. Although the philosophy of capacity design is not introduced and methods offered look more like construction solutions.

The clause 6.8.8 encourages to pay more attention to the joint zones in moment-resisting frames. The reason is that dissipation will take place there, so an additional shear reinforcement (Figure 24) must be provided to avoid an undesirable failure. The standard limits the spacing of stirrups to 100mm or 200mm for the frames with bearing diaphragms. This rule ends after the distance of  $1.5h$ , where  $h$  is a member depth.

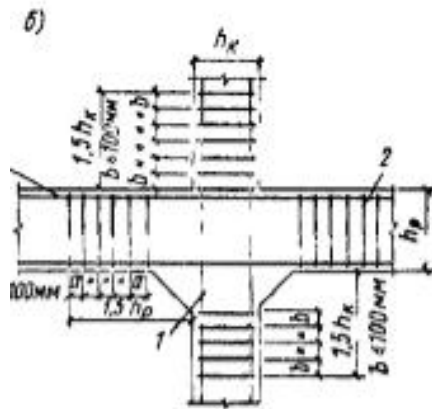


Figure 24 Additional shear reinforcement in the joint zones

SNiP, however, does not give any regulations for the joint shear resistance needed to withstand the stress caused by tension and compression of the attached beam in bending. The approach is assumed not to differ from the one used in a usual design situation. Anyway, this can always be checked with the method given in 3.78 of the official guidance (Figure 25).

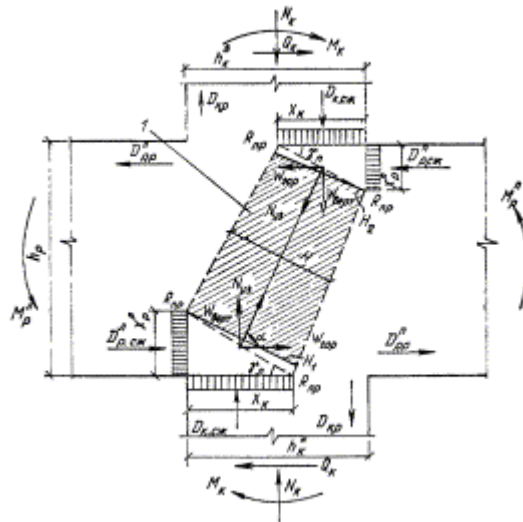


Figure 25 Internal forces in the joint regions (official guidance illustration)

Obviously, stirrups are used not only for resisting shear forces, but also to prevent compression reinforcement buckling. Of course, in the seismic situation the real compression strain will exceed the design value. Therefore, extra rules to prevent that are given. The standard demands to account them only for the elements where compressed steel is accounted. In spite of that, ignoring the compression rebars in calculations does not protect them from buckling in reality. So, it will be a good idea to follow the requirements below in all the members where the reinforcing steel may get compressed.

When the design seismicity is 8 or 9 degrees, stirrup spacing must be limited to not more than:

- The least of 400mm, or 12d for tied stirrups and 15d for welded ones, when the design resistance of steel is less than 450MPa

- 300mm, 10d for tied and 12d for welded, when the design resistance of steel is more than 450MPa
- For all the steel types the spacing of stirrups in columns of moment resisting frames does not exceed 0.5h
- In braced buildings the previous rule for columns apply with the value of  $1 \cdot h$ , where h is the smallest dimension of column cross-section. The minimum stirrup diameter is 8mm in the last two cases. In all cases d means the least diameter out of all compressed rebars. (6.7.9)

If the reinforcement ratio exceeds 3%, the stirrup spacing is limited to the least of 8d or 250mm.

#### 7.4.2.3. Specific rules for braced frames

The regulations below have to be satisfied in addition to the general rules above.

Firstly, the definition of a frame with bracings is given. In such frames columns must not represent more than 50% of lateral stiffness on every floor. The reason is that the behavior factor for such frames reduces the seismic action more than for moment-resisting ones. So, it must be assured that enough energy will dissipate in these bracings.

Lateral force resistant walls must be arranged in both orthogonal directions, symmetrically with respect to building mass center. Therefore, it should be at least two walls for each direction that are not located in the same plane. They have to continue along the whole frame height, although it is allowed to reduce the size at top storeys. The reason is that a seismic shear decreases from building basement. However, the stiffness difference must not be more than 20% between sequent floors. (6.8.9)

#### 7.4.2.4. Specific rules for pure wall systems

In the section 6.11 SP defines these buildings as ones where at every storey at least 80% of stiffness is achieved with wall elements. In the same manner as in EC8, one may observe the best behavior factor for these systems.

A designer may find the regulations for wall systems in the sub chapter 6.11. Again, most of them are construction requirements and only a few help to achieve energy dissipation.

Firstly, lapping rules are not that strict for wall elements comparing to moment resisting bars. Lapping is allowed for the bars less than 20mm in diameter. The lapping length does not need to be increased comparing to the usual design rules. When the seismicity is 9 degrees, special anchorage devices must be used. (6.11.10)

All the storeys must have their stiffness not less than 50% of a storey below. (6.11.2)

To ensure a sufficient number of inelastic deformations, in buildings with bearing external walls there should be at least two shear walls at any direction. (6.11.3)

### 7.5 Steel structures

#### 7.5.1 Eurocode

The European standard gives specific rules for steel buildings in the section 6. Again, it is unnecessary to simply list them all, so only the basic idea is provided. After this, a designer is capable of following the requirements of the code.

##### 7.5.1.1. Frame types and ductility classes

Eurocode 8 treats steel structures in the same way as concrete ones. Buildings are differentiated based on the method they resist lateral forces. The standard introduces the following frame types:

- a) Moment-resisting frames
- b) Frames with concentric bracings
- c) Frames with eccentric bracings
- d) Inverted pendulum structures
- e) Structures with concrete walls
- f) Moment resisting frames with infills
- g) Combinations of moment resisting frames with concentric bracings

In the moment resisting frames horizontal forces are counteracted by beams in bending that are connected to columns with the ability to transfer moments.

Buildings with concentric bracings dissipate energy with the help of plastic deformations mainly in tension diagonals. For the compression ones the dissipation is also possible, however, in most cases prevented by buckling.

The third building type involves bracing elements connected with an eccentricity to a main chord centerline that results in secondary bending moments in the chord, as shown in Figure 26.



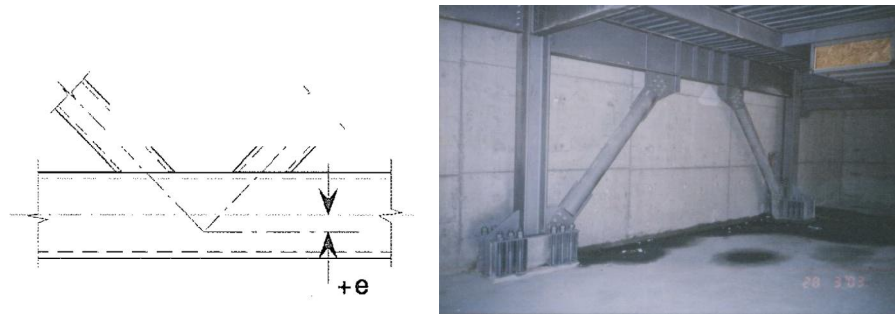


Figure 26 Eccentric connection of bracings (<http://web.iku.edu.tr/courses/insa/ce007/>)

The chord zone between the bracing elements is called a seismic link. This is the place where dissipation takes place, while the horizontal forces are still resisted by bracings themselves.

As for the type d), it does not anyhow differ from the one defined in the previous chapter about concrete. The three last kinds represent the combinations of earlier described solutions.

Within this thesis only certain types will be covered in details, since they alone cover all the basic concepts of a building behavior during an earthquake. All the other kinds do not differ significantly, so a reader does not need any additional information besides that written in the relevant parts of EC8.

When it comes to the ductility classes, one does not find any differences comparing to RC-structures. The approaches are described in 6.1.2 of EC8, though they mainly repeat the sentences of the previous chapter. In a similar manner, an engineer is capable of choosing the degree of ductility by himself. More dissipative behavior will reduce the design seismic force, but requires more careful detailing (and, probably, more of expensive welding operations) for a capacity design that is chosen to lock plastic deformations at desired places.

### 7.5.1.2. Behavior factors

In purpose to reduce an earthquake effect Eurocodes introduce the behavior factor  $q$  that depends on a frame type and ductility class as described in the part 6.3.2. The factors for the steel buildings are listed in Table 5.

Table 5 Behavior factors  $q$  for steel buildings

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_w/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_w/\alpha_1$
d) Inverted pendulum	2	$2\alpha_w/\alpha_1$
e) Structures with concrete cores or concrete walls	See section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha_w/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section 7	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_w/\alpha_1$

Again the buildings not regular in elevation are penalized with 20%  $q$  reduction. As for the alpha values, they are obtained in the same way as for RC structures. The method can be time-ineffective for the specialists non-familiar with the pushover analysis type. Therefore, a simplified definition is given, too.

#### 7.5.1.3. Material overstrength

Steel is an alloy that is significantly more uniform than other materials. Moreover, all the elements are manufactured in factories, which allows a more precise estimation of the real material strength, but not characteristic one that corresponds to the 95% possibility of non-exceedance. This results into various procedures for the capacity design. As it was written earlier, this approach must consider the actual strength of a material in dissipative members. For steel buildings the standard allows to rely on the results of testing according to 6.2. Nevertheless, in the absence of specific measures, the standard introduces the partial factor  $\gamma_{ov}=1.25$  that is multiplied by 1.1.

#### 7.5.1.4. Capacity design of moment-resisting frames

In the buildings with moment-resistant connections (Figure 27) the energy is assumed to dissipate either in beams or in the joints. Therefore, connections are not obligated to be fully rigid. Partial strength connections can also undergo sufficient plastic deformations involving, for example, an end-plate in bending or plate shear.



Figure 27 Moment resisting steel frame (<http://www.allstructure.com/projects.htm>)

Despite whether the dissipation takes place in beams or columns, the main idea is to satisfy the fundamental equation 4.29 to avoid the soft-storey failure.

Although steel is capable of working well in plastic shear deformations, moment resisting frames are assumed to reduce the design earthquake force through flexure. That means that a shear and axial failure must be prevented in addition to lateral torsional buckling. For that purpose the part 6.6.2 of the standard limits the usage ratio ( $F_{Ed}/F_{Rd}$ ) of relevant failure components.

When a designer is sure that only a bending plastic failure is possible in primary seismic beams, it is necessary to take care of columns. The rules are provided in 6.6.3.

All the primary columns must have enough extra capacity. It is obtained by multiplying the usage ratio of the beams joining the column by the material overstrength factor. In addition, shear forces in the column web resulting from a moment connection are also taken into account, as it is done in RC-structures.

When it comes to connections, it is needed to decide if they are dissipative or not. For the second case EC8 obligates to provide enough bearing capacity taking into consideration a possibly stronger material in attached beams (6.5.5). But if an engineer wants to allow plastic deformations in joints he has to ensure a sufficient rotational capacity in full accordance with 6.6.4.

### 7.5.1.5. Frames with concentric bracings

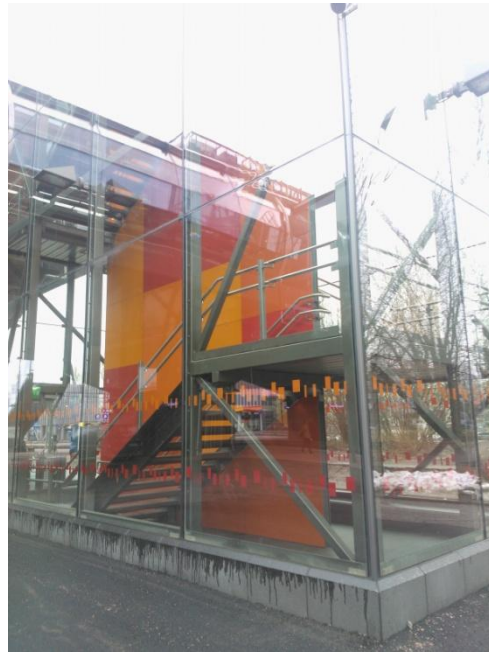
In the buildings where lateral stability is governed by diagonal bars connected either in the centerlines of load-bearing elements, or with the eccentricity allowed in EC1993-1-8, the design seismic action is reduced via plastic deformations in tension diagonals. Unfortunately, it is hardly ever possible to involve the compression ones for the reason of flexural buckling. Only if the slenderness of diagonal bars does not exceed the limit value from EC3 6.3.1.2(4), it can be designed only for the cross-section resistance. Therefore, it may contribute to energy dissipation. (Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioll, Paolo Pinto, An-

dre Plumier “Designers’ guide to Eurocode 8: design of structures for earthquake resistance”)

The rules for these buildings are presented in 6.7. Obviously, they do not need to conform to the fundamental equation 4.29 about moment resistance of beams and columns. Anyway, this is a more difficult case because now beams, columns and connections have to avoid failure while bracings undergo plastic deformations.

Firstly, a reader may observe in Figure 28 various kinds of bracing arrangements, each of them has specific requirements to be followed.





d) Figure 28 Various types of diagonal bracings (<https://courses.cit.cornell.edu/arch262/notes/07b.html>; <https://courses.cit.cornell.edu/arch262/notes/07b.html>; <http://www.shutterstock.com/pic-107442737/stock-photo-cross-bracing-of-steel-structure.html>; [https://www.sfsa.org/tutorials/castconnex/SCC\\_02.htm](https://www.sfsa.org/tutorials/castconnex/SCC_02.htm))

On Figure 28 above: a stands for X-bracings, b means V or “chevron” bracings, c stands for simple diagonal bracings and d for K-bracings.

The general approach is to consider only tension diagonals during a structural analysis, because buckled compression ones will not contribute to the lateral stiffness at all. For this reason, K bracings are strictly forbidden because they cannot work without compression bars - lateral forces will be resisted by moments in columns, which is potentially dangerous.

Nevertheless, V-bracings are allowed. Although they have the same drawback as K-ones, the possible plastic hinges will occur in beams that is much safer. Anyway, the dissipation still takes place only in the diagonals, no flexural effect of beams should be relied on. This is why for V-bracings the standard obligates to model both tension and compression bars, with the latter resisting all the compression in a design seismic situation.

While the tension bars undergo plastic deformations, beams and columns have to stay within their bearing capacity. This is achieved with the help of 6.7.4. First of all, a designer must ignore the contribution of diagonals to the resistance of gravity loads, as far as they will certainly buckle, only beams and columns must be considered. Secondly, the usage ratio for bracings multiplied by their possible material overstrength should be smaller than the one of the elements they are connected to. After yielding of diagonals, there will be no increase in the internal forces of beams and columns, so the lateral stability won't be lost.

## 7.5.1.6. Frames with eccentric bracings

When the diagonal bars are connected eccentrically, shear forces and bending moments result in the seismic links (Figure 29). This allows to dissipate energy inside them, while the bracings are assumed to remain elastic and be capacity designed against failure together with beams and columns. The rules are provided in Chapter 6.8 of the standard.

Based on them, seismic links are distinguished into three categories according to their length: short, long and intermediate. This results into various way of energy dissipation that will affect the capacity design. In the short links the main ductile deformations happen through shear, in the long ones through bending, while the last is the combination of them.

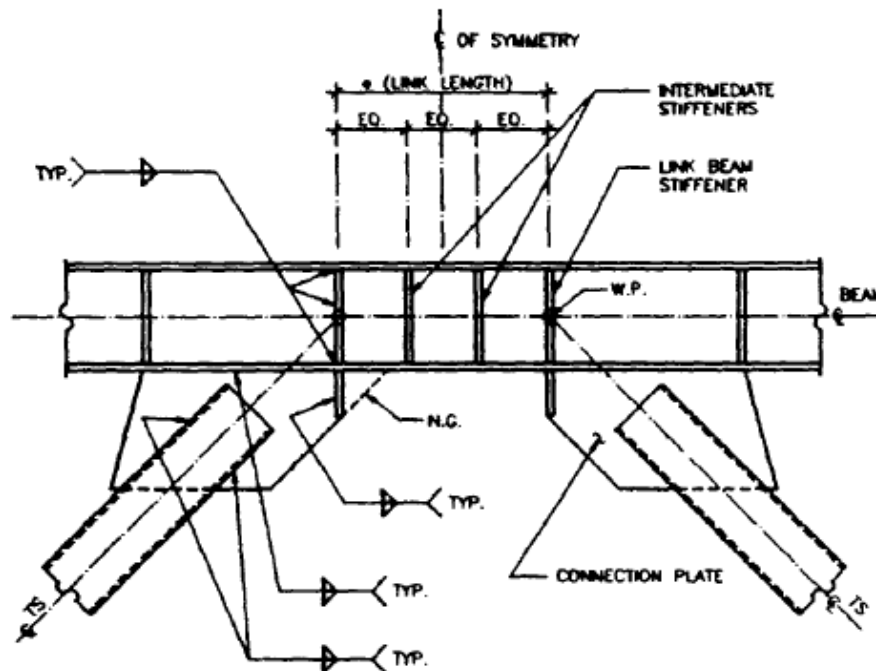


Figure 29 A seismic link example, Photo from AISC Steel Tip "Seismic Design Practice for Eccentrically Braced Frames", R. Becker and M. Ishler

Eurocodes give the formulae to define the resistance of a link in shear and bending that depend on the design value of axial force. The classification of length is based upon the ratio of moment to shear resistance.

In addition, there are the regulations for detailing on a link zone that require providing an adequate amount of web stiffeners as a reader may see in the Figure 29. This is needed to avoid transverse buckling of the web in a seismic link, allowing the desired plastic deformations to happen before other failures.

When it comes to non-dissipative members, there are a lot of them to be capacity designed: diagonal bars, beams (outside the region of a seismic link), and columns. However, such difficulty is well rewarded with a quite high value of a behavior factor. The formulae to prevent an early failure of non-dissipative elements is quite similar to the one about concentric bracings, but the usage ratio is now taken for seismic links with respect whether they are long (moment resistance is taken) or short (shear capacity is

analyzed). In addition, the same overstrength must be acquired by the connection of links.

### 7.5.2 SNiP

The Russian norms differentiate steel structures into moment resisting frames and braced ones. An interesting fact is that the behavior factor is better for the latter, despite the fact that flexure provides a better energy dissipation. In concrete buildings there is the same situation, but it can be reasoned by the presence of shear walls that work a lot in bending.

The rules for steel construction can be found in the section 6.9. Contrary to RC structures, most of them have an obvious goal to enlarge the ductility of a frame.

#### 7.5.2.1. General rules

SNiP starts with special requirements for the material used. It is not a surprise that these regulations apply only to the dissipative members. In their design only low-carbon steel with an ultimate strain at least 20% must be used. Obviously, this is needed for large plastic deformations without a fracture. (6.9.5)

The standard also allows to use different lateral resistance methods in the two building directions. In other words, stability in Y direction may be assured by a column fixed connection while in X there can be diagonal bracings. In that case bracing bars in X direction must be arranged in all rows of columns. (6.9.6) There are, however, some questions with a behavior factor in this case. For the buildings analyzed separately in the two orthogonal directions different design spectra can be used, whereas for spatial models it is not possible. SP does not give any rules about it, so this is up to a designer to select the factor.

#### 7.5.2.2. Moment-resisting frames

For moment-resisting systems all the columns must be made of sections with an equal slenderness of the two main axes. According to “Steel structures”, in case of an I profile the width has to be equal to two times the height of the cross section as to acquire an equal second moment of area. Therefore, only closed sections like tubes or square hollow sections are allowed. (6.9.1). This rule concerns only the buildings that have moment resisting connections providing a lateral stability in all directions. As for the combined systems like, for example, portal frames, opened sections such as I-beams are not forbidden.

In rigid beam-to-column joints regions web stiffeners should be provided for columns as shown in Figure 30. The required thickness is calculated based on the general rules for plate elements. Nevertheless, the official guidance gives the minimum thickness defined by the formula:

$t = (0,1 \div 1,2) t_{w, b}$ , where  $t_{w, b}$  is the thickness of the beam web.

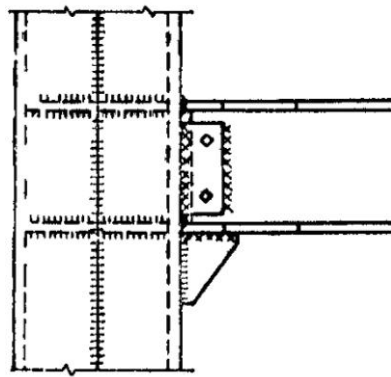


Figure 30 Column stiffeners arrangement

It is notable that installing this kind of plate inside a cold-formed tube will cause plenty of additional work comparing to a welded column.

The clause 6.9.2 insists on avoiding plastic deformations in joint regions, which differs significantly from Eurocodes. Anyway, their response to a cycling inelastic behavior has not been studied enough, so it is more reliable to locate dissipation regions in beams.

In the official guidance statement 3.83 the soft-storey mechanism is strictly prohibited. But the document itself, as one may read in the beginning, is not obligatory.

Besides, the guidance also uses the capacity design in the same clause. It recommends providing the joint resistance itself bigger than the plastic resistance of the attached beam. One must keep in mind that this approach does not concern strain hardening, so it is always good to add extra capacity to a joint.

To avoid unreasonably thick welds, it is recommended to enlarge the profile width in joint zones as shown in Figure 31. (6.9.4)

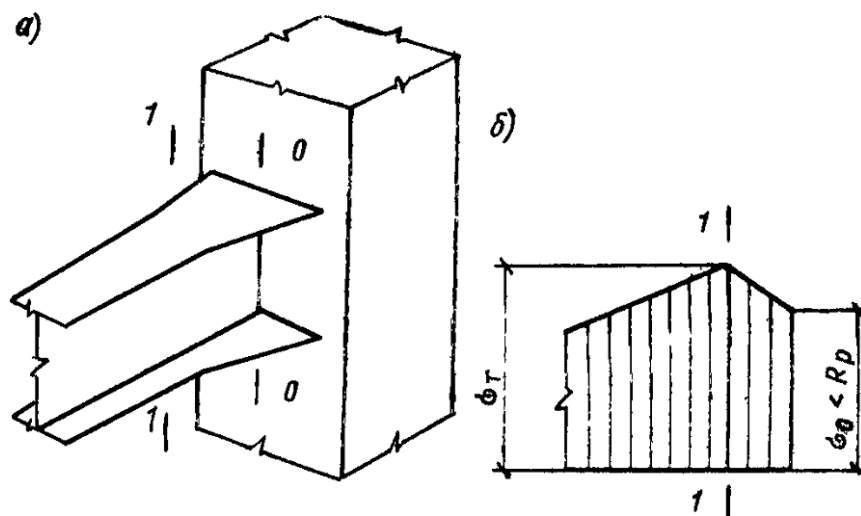




Figure 31 Reducing normal stresses in a fixed beam-to-column connection: a – increasing the beam width, б – axial stress diagram for the beam upper flange

While in Eurocodes the rotational capacity of an element is represented by a cross-section class, in Russia the limiting of the slenderness of section elements is used. Whereas in a seismic case large deformations are expected, the slenderness of the web and flanges is governed by the following rules applied to dissipative members (main beams):

- $h_w/t_w \leq 50$
- for the outstand flanges  $c/t \leq 0.25t\sqrt{E/R_y}$ , where  $c$  and  $t$  stand for the same as in EC3-1-1 table for the cross section classes and  $R_y$  is the design yield strength.

### 7.5.2.3. Braced frames

For the buildings with diagonal bracings I and cross profiles are not forbidden to be used as columns. Unfortunately, one will not find any regulations for these buildings in SP. So, to ensure a ductile response of a structure a designer has to apply the official guidance.

The clause 3.83 recommends using special energy-dissipation devices for diagonal bracings. For example, a ring-energy dissipater, tube-energy dissipater or beam-energy dissipater.

It must be noted that no one forbids to assume dissipation in the diagonals themselves, although no rules for their detailing are given.

## 8 CONCLUSION

Both standards have proved to be reliable by many years of practice. This means that the methods given correspond to the reality to a sufficient extent. Anyway, there is one principal difference between the two standards.

In general, Eurocode 8 has demonstrated to be more “user-friendly”. In all the chapters before actual formulas and regulations there are the descriptions of methods used, which a designer may use as keywords if he wants to acquire additional background. This is something that SNIp lacks. In the Russian standard one will rarely find the purpose of a requirement. So, to use it safely an engineer has to apply for other sources, e.g. official guidance.

Besides, one more inequality is caused by the previous. Providing enough background, EC8 gives many more alternatives allowing one to regulate the design process. This is quite notable after taking a look at various ductility classes, or possible structural analysis types. In SNIp alternations are always based on simplifications and have the purpose not to give the opportunity to choose, but to ensure safe and conventional structural solutions.

Anyhow, the main drawback of SNIp for a non-experienced engineer is clearly visible after the structural analysis phase. SP offers quite high reduction of a design seismic action due to possible plastic deformations and does not link it with ductility requirements.

Nevertheless, it is vital to give a reader the correct goal of this conclusion, which is different from writing a list of advantages and drawbacks of the building codes. The main aim is that a reader will have an adequate assessment of his own skills and be able to match them with the background needed for both standards. The sentences above form a brief image of what a designer will face. Obviously, design in Russia demands a significantly higher level of understanding in order to ensure safety. This is why it is recommended to firstly get acquainted with plenty of additional sources like the official guidance or, as an alternative, Eurocode 8.



## 9 SOURCES

### 9.1 Websites

- Wikipedia.org
- Steelconstruction.info
- wiki.csiamerica.com
- knowledge.autodesk.com and forums.autodesk.com

### 9.2 Books

- 1) Amita Sinhal - “Understanding earthquake disasters”
- 2) Michael N. Fardis, Eduardo Carvalho, Amr Elnashai, Ezio Faccioli, Paolo Pinto, Andre Plumier “Designers’ guide to Eurocode 8: design of structures for earthquake resistance”
- 3) U. Kudishin, E. Belenya, V. Ignatyeva, A. Pukhovskiy, G. Vedenikov, B. Uvarov, S. Astryab, V. Val’, U. Sokolov, T. Morachevskiy, D. Streltskiy - “Steel structures” («Металлические конструкции»)
- 4) L. Mailyan, D. Mailyan, E. Lukashevich, U. Veselev, G. Verzhbovskiy, V. Lagutin, L. Mailyan, A. Mailyan - “Designer’s handbook” («Справочник современного проектировщика»)
- 5) Luis Simoes da Silva, Rui Simoes, Helena Gervasio - “Design of steel structures”
- 6) P.Bhatt, T.J. MacGinley, Ban Seng Choo - “Reinforced concrete design to Eurocodes”
- 7) R.S. Narayanan and A. Beeby - “Designers’ guide to EN1992-1-1 and EN1992-1-2”
- 8) V. Setkov, E. Serbin – “Building frames design” («Строительные конструкции расчет и проектирование»)
- 9) L. Gardner, D.A. Nethercot – “Designers’ guide to EN 1993-1-1”

### 9.3 Pictures

1. Number of damaging earthquakes per country in 2012 (<http://earthquake-report.com/2013/01/07/damaging-earthquakes-2012-database-report-the-year-in-review/>)
2. Earthquake monitoring in Finland (<http://earthquaketrack.com/p/finland/recent>)
3. MSK-64 intensity scale ([http://en.wikipedia.org/wiki/Medvedev%E2%80%93Sponheuer%E2%80%93Karnik\\_scale](http://en.wikipedia.org/wiki/Medvedev%E2%80%93Sponheuer%E2%80%93Karnik_scale))
4. First and second vibrational modes of a simple structure ([http://en.wikipedia.org/wiki/Seismic\\_analysis](http://en.wikipedia.org/wiki/Seismic_analysis))
5. An example of acceleration response spectra (<http://www.isatsb.com/Seismic-Ground-Motion-Values.php>)
6. Different failure mechanisms for a moment resisting frame ([http://www.luxinzheng.net/publication1/PF\\_THU.htm](http://www.luxinzheng.net/publication1/PF_THU.htm))

7. A soft-storey mechanism example  
([http://en.wikipedia.org/wiki/Soft\\_story\\_building](http://en.wikipedia.org/wiki/Soft_story_building))
8. Different load resisting systems  
(<http://www.johnmartin.com/earthquakes/eqshow/images/64701417.GIF>;[http://www3.nd.edu/~concrete/research\\_files/Precast\\_Coupling\\_beams\\_for\\_walls\\_files/precastcoupling1.pdf](http://www3.nd.edu/~concrete/research_files/Precast_Coupling_beams_for_walls_files/precastcoupling1.pdf);  
<http://www.nexus.globalquakemodel.org/gem-building-taxonomy/overview/glossary/moment-frame--lfm>)
9. Various types of diagonal bracings  
(<https://courses.cit.cornell.edu/arch262/notes/07b.html>;  
<https://courses.cit.cornell.edu/arch262/notes/07b.html>;  
<http://www.shutterstock.com/pic-107442737/stock-photo-cross-bracing-of-steel-structure.html>;  
[https://www.sfsa.org/tutorials/castconnex/SCC\\_02.htm](https://www.sfsa.org/tutorials/castconnex/SCC_02.htm))
10. Moment resisting steel frame  
(<http://www.allstructure.com/projects.htm>)
11. Eccentric connection of bracings  
(<http://web.iku.edu.tr/courses/insaat/ce007/>)
12. Welding photos and pictures  
(<http://www.gosthelp.ru/text/svarkametallokonstrukciji.html>;  
[http://www.zaopkti.spb.ru/services07\\_416.html](http://www.zaopkti.spb.ru/services07_416.html))
13. Concrete frame pictures  
(<http://www.nexus.globalquakemodel.org/gem-building-taxonomy/overview/glossary/moment-frame--lfm>)

#### 9.4 Others

Video from the seminar «САПРяжение 2012», seismic design in Auto-desk Robot

## Seismic design of a steel structure

### Building introduction

The purpose of this appendix is to show in practice most of the things written above. A picture of the analyzed building is presented further.

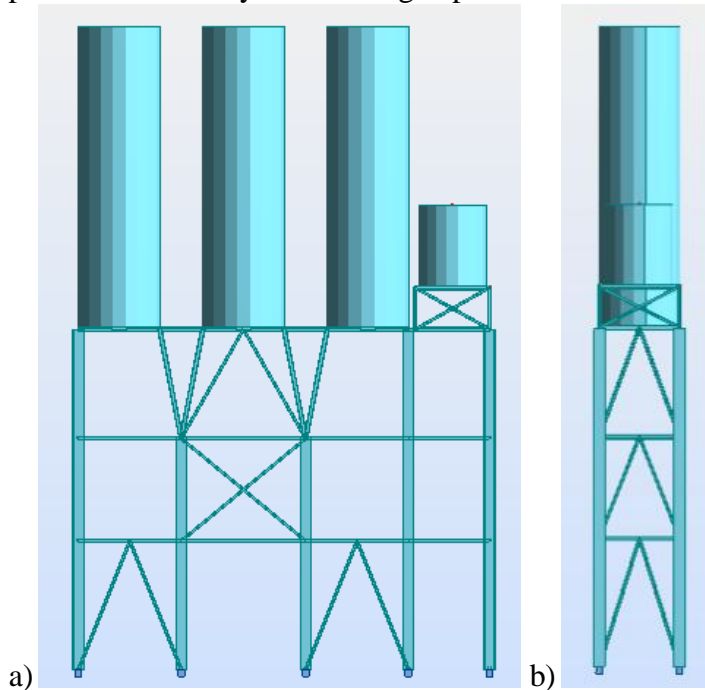


Figure 32 Analyzed building: a) – front view; b) – side view

This is a coal-silo located in Chile. Initially it was designed and analyzed for an earthquake resistance using American standards. Therefore, one may notice some aspects not common for a design in Europe: for instance, the angle of bracings which is less than  $30^\circ$ . The structural arrangement was not changed, unless it was needed for the purposes of the seismic design, but this will be specified later.

The frame is divided into three parts with a varying structural purpose: supporting steel frame, silo tanks (three big ones for coal and a small one for bio-content) and gallery on top of them.

### Loads

The building is quite specific, so the loads cannot be defined from EN 1990. All of them were given by the customer.

#### Dead load

Description	Magnitude	Location
Self weight of handrails, gratings, etc.	$0.5 \text{ kN/m}^2$	Platforms under coal and bio silo
Secondary structures in the gallery	$28 \text{ kg/m}^2 = 0.28 \text{ kN/m}^2$	Gallery roof
	$15 \text{ kg/m}^2 = 0.15 \text{ kN/m}^2$	Gallery walls
	$40 \text{ kg/m}^2 = 0.4 \text{ kN/m}^2$	Gallery floor
Belt	$85 \text{ kg/m} = 0.85 \text{ kN/m}$	Gallery
Pulley	$563 \text{ kg} = 5.63 \text{ kN}$	Galley
Coal conveyor bridge	$25500 \text{ kg} = 255 \text{ kN}$	Gallery
Bio conveyor bridge	$13700 \text{ kg} = 137 \text{ kN}$	Top of the bio silo
Tripper in the gallery	$9000 \text{ kg} = 90 \text{ kN}$	Any point in the gallery
Coal silo content	$864000 \text{ kg} * 3 = 8640 \text{ kN} * 3$	Coal silo

Bio silo content	33300kg=333kN	Bio silo
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### Live load

Description	Magnitude	Location
Imposed load	3 kN/m <sup>2</sup>	Platforms under coal and bio silo
Maintenance loads	1500 kg/m <sup>2</sup> =15kN/m <sup>2</sup>	Gallery floor
	100kg/m <sup>2</sup> =1kN/m <sup>2</sup>	Gallery roof
Conveyor bridges live load	53kN	Gallery
	32kN	Top of the bio silo

Wind load is not the case of interest according to 5.1 of the latest SP and the combination factor  $\psi_0=0$  in EN1990.

### Modeling

Modeling and analysis was done in Autodesk Robot Structural Analysis Professional 2012. Columns, beams and bracings were modeled as prismatic beam elements. Based on 4.3.1(2) of EC8, the beams were offset from the columns with very stiff, massless beam elements to account eccentricity. The beams have pinned joints with columns and the latter are rigidly connected to the foundation. The clause 6.3.1(3) of EC8 forbids the usage of K bracings, so they were replaced with the V-ones. Although in SNiP it is not said anywhere about possible bracing types, the same changes were still applied in order to ensure an equal stiffness of each row of columns.

The silos were not modeled as shell structures – not for the reasons described in 4.4.1.2 of this work, that apply mostly to concrete. This was done to avoid unnecessary complicated modeling that may cause errors and have a negative effect on analysis results. Instead of that, the behavior of the silo-structure was described by a contractor. According to his explanations, a huge steel cross-section (6000x47.5) was selected for the beam elements that represent coal silos in the model. Big beams were braced with rigid elements in order to obtain the same dynamic properties (e.g. natural frequency) as the complicated shell-beam structure of a coal silo. The bio-silo was modeled with a huge beam-element too.

The mass on the gallery roof is really insignificant, it will have a small effect on the gallery walls structure. Therefore, the gallery was included only as nodal masses on top of the coal-silo beams. This ensures the same effect on the supporting structure caused by an earthquake.

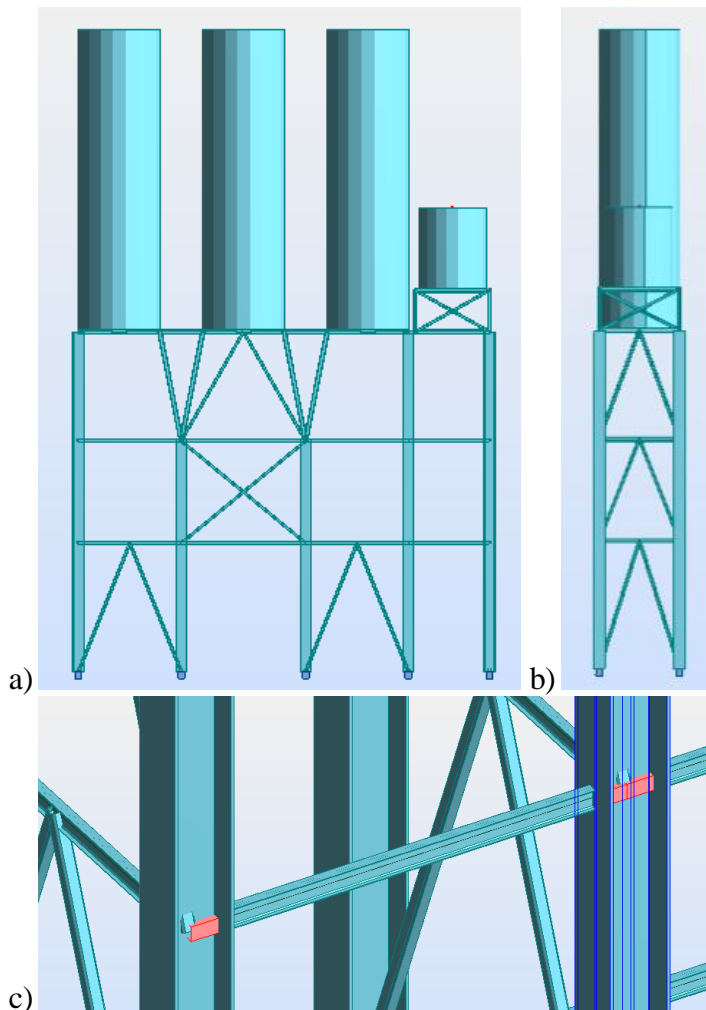


Figure 33 Initial analysis model in Robot: a) – front view; b) – side view; c) rigid mass-less beams to account joint eccentricity.

According to the clause 6.1.2 of SP, this structure must be separated with a seismic joint for the reason of a significant mass difference between the coal silos and the bio-tank. Leaving the building unseparated will result in the following deformation shape:

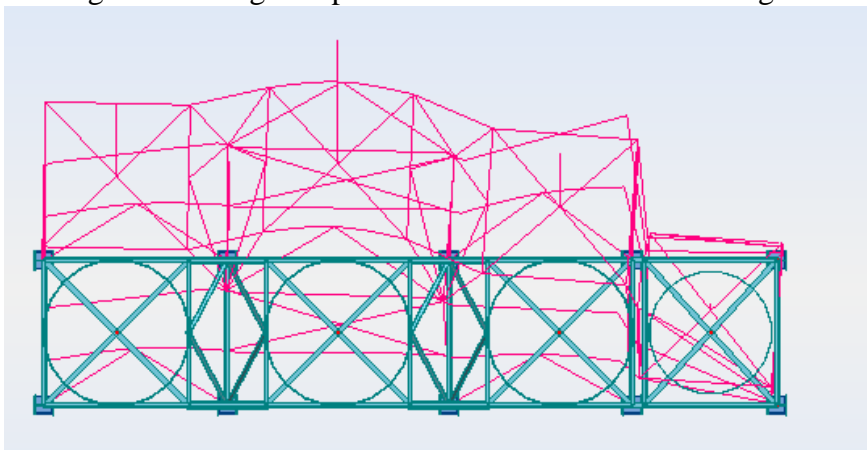


Figure 34 Natural mode shape of the initial model, top view.

Independently of the design code, this shape will result in the internal forces that are quite difficult to be resisted by generally adopted structural solutions. This is why in



both cases the structure has a seismic joint, by duplicating the row of columns between the coal silos and the bio tank.

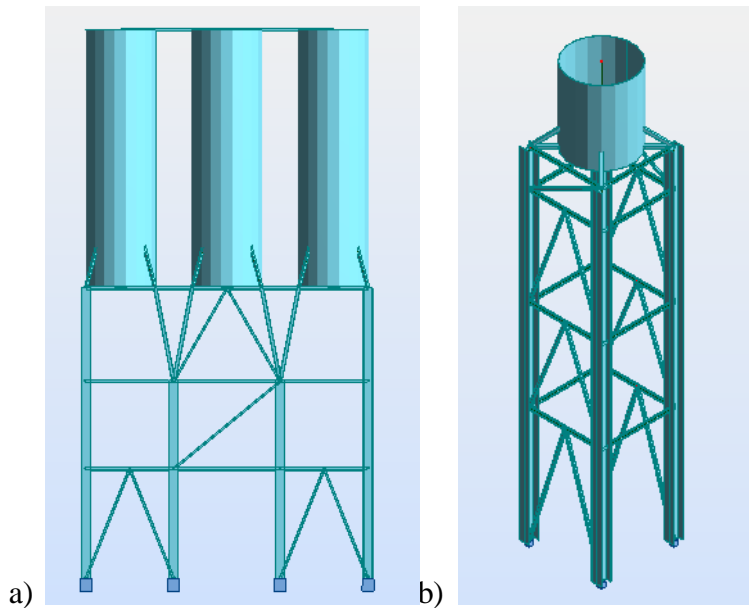


Figure 35 Final model: a) Coal silo structurally independent unit; b) Bio silo independent unit. Units are separated with a seismic joint

The critical unit is, of course, the one with three silos, it will be used as the example of seismic design.

Both standards obligate to account ground elasticity. In most cases it has a positive influence on the results, in other words, reduces the internal forces. Therefore, in this case to stay on the safe side it was neglected and the supports are fully rigid.

As for the X-bracings, clause 6.7.2(2) of the standard obligates to account only tension diagonals throughout the analysis. In robot, there is a feature to make a truss-element resist only tension or compression force, but this method uses iterations, sequentially, is non-linear. The MRS analysis is linear, so one will not obtain correct results. The only way to fulfill the demand of the code is to simply delete compression bars from the analysis model. This makes a designer to create at least two models: one with the tension diagonals in each direction, to evaluate the seismic response in both positive and negative sides.

### **Eurocode 8 design**

#### **Modal analysis**

As it was said in 5.3.3 of this thesis, it is always a must to consider possible torsional response of a structure. For example, the case when only one silo (e.g. the right one) is full with coal and other stay empty can be fatal during an earthquake because of enormous mass. Though it is not said in any standards, this situation must be considered. So, in both cases (SniP and Eurocode), there were three models to be analyzed and designed: bio silo unit, coal silo unit with all the silos filled with coal, coal silo unit with only one side silo containing coal while others were empty. In addition, the location of the tripper on top of the gallery is not fixed, so, in the eccentric (torsional) case it is placed on top of the filled, side silo.

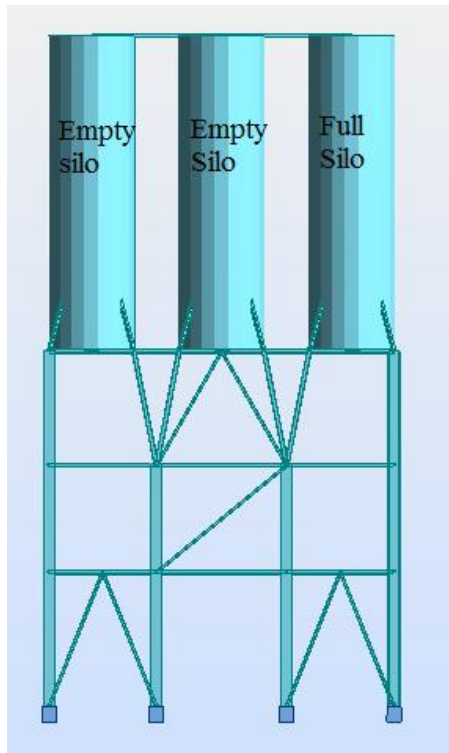


Figure 36 Torsion case load arrangement. Plenty of mass concentrated on the right silo will cause a torsional response when the ground moves perpendicular to the plane of the page

To obtain the seismic mass that is accounted in the modal analysis, the rules given in 3.2.4(2) of EC8 were used. Based on EN 1991-1-1 classification, the building belongs to the category E1, so the value  $\phi$  equals to 1 (table 4.2 of EC8). The standard EN 1990 gives the value  $\psi_2=0.8$ . So, the final part of the live load to be considered in analysis is:  $Q_k * \psi_E = Q_k * 0.8 * 1$ .

The self-weight (including mass of the coal) was taken as a characteristic value. The limit for mass participation is 90%, no vertical component of seismic action is needed to be considered according to 4.3.3.5.2 of the standard.

### Accidental torsional eccentricity

The rule from 5.1.6 of this work that introduces the accidental eccentricity through a static force application is only one possible option, suitable for manual (hand) design. The other way, also not forbidden by the standard is to account it directly throughout a modal analysis. In most of the software, there is such a function, e.g. in Robot it is possible to specify the relative eccentricity by an automatic addition of nodal masses. This is how it was accounted in our case, the value of 5% was input.

The results of the modal analysis are presented below, only two modes were needed to reach the 90% value of mass participation in the directions of interest for the usual case and three modes were needed for the case with torsion.

Case with three full silos:

Case	Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)
6	1	0.58	1.72	0	95.24	0
6	2	0.86	1.16	97.2	95.25	0

Case with only right silo filled with coal:

Case	Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)
------	------	----------------	--------------	----------------	----------------	----------------

6	1	0.74	1.35	0.01	87.72	0
6	2	1.22	0.82	96.57	87.74	0.04
6	3	1.78	0.56	96.6	93.93	0.04

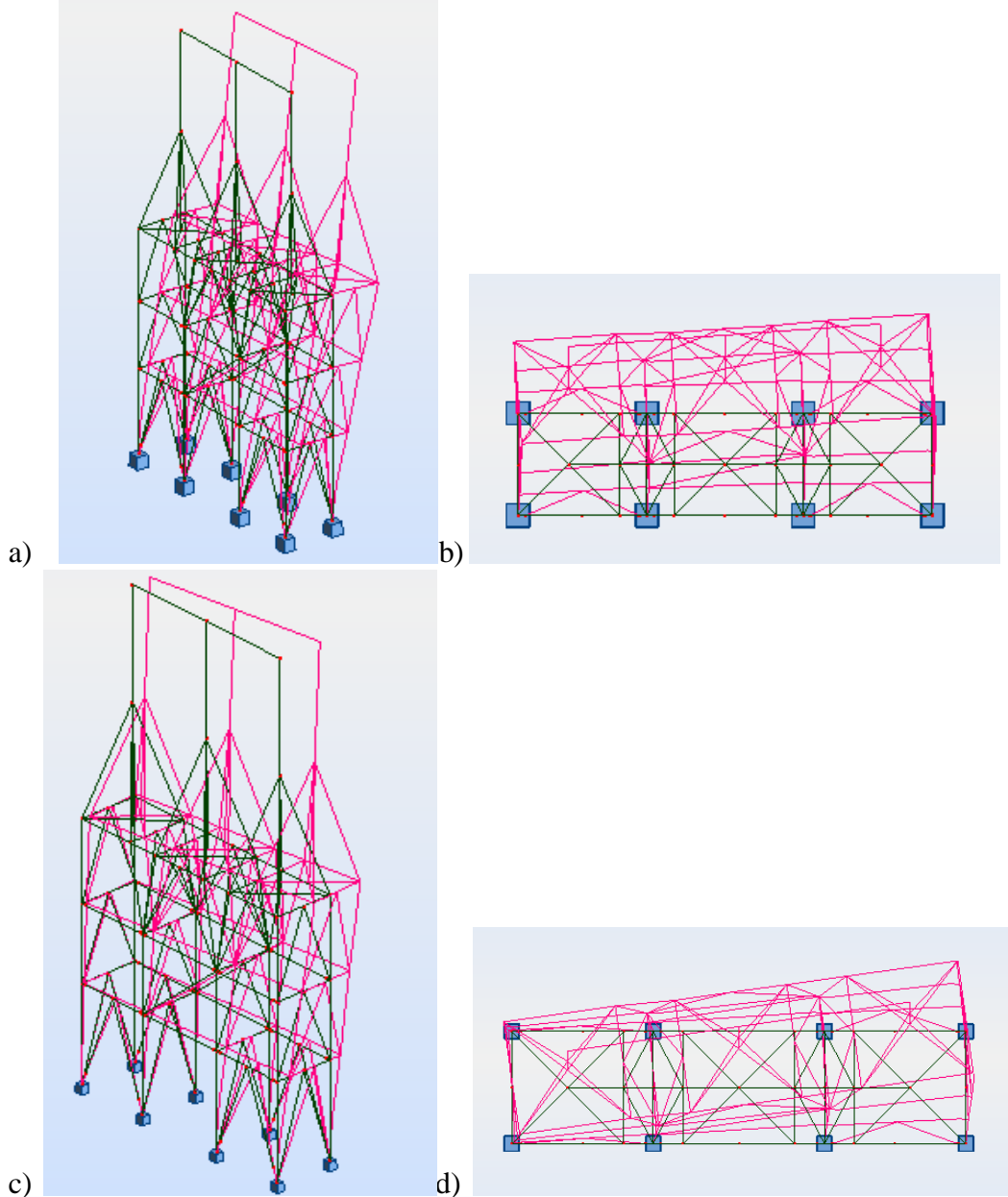


Figure 37 Natural mode shapes: a – case with 3 silos full; b – top view. A small rotation is caused by the accidental eccentricity of 5%; c – case with only right silo full; d – top view.

### Design spectrum definition

Based on NCh433-1996 (Chilean norms), the building rests on the soil of type 2, that corresponds to the type B class in EC8. The reference ground acceleration on the ground type  $a_{gR}=3.924\text{m/s}^2$ . The second shape of the spectrum was selected.

Based on 4.2.3.2 and 4.2.3.3 of EC8, the structures (each of the two units) are non-regular in a plan (due to the absence of the floor diaphragms) and elevation (because of most of the mass concentrated on the top storey). This was accounted in the behavior factor selection.

### Behavior factor

The unit with coal silos resists lateral forces mainly by diagonal V-bracings. The ductility class chosen is DCH.

The factor  $q$  was chosen using Table 6.2 of Eurocode 8.

The behavior factor was reduced by 20% because of non-regularity in elevation (6.3.2(2)). So, the final  $q$ -value equals to:

$$q=2.5*0.8=2$$

### Elastic response spectrum

At first, the elastic response spectrum was defined based on the rules of 3.2.2.2 of the standard. The recommended values of  $T$  were selected. The building has a usual importance level, so the reference return period is used and the factor  $\gamma_I=1$ . So, the design ground acceleration is equal to the reference one.

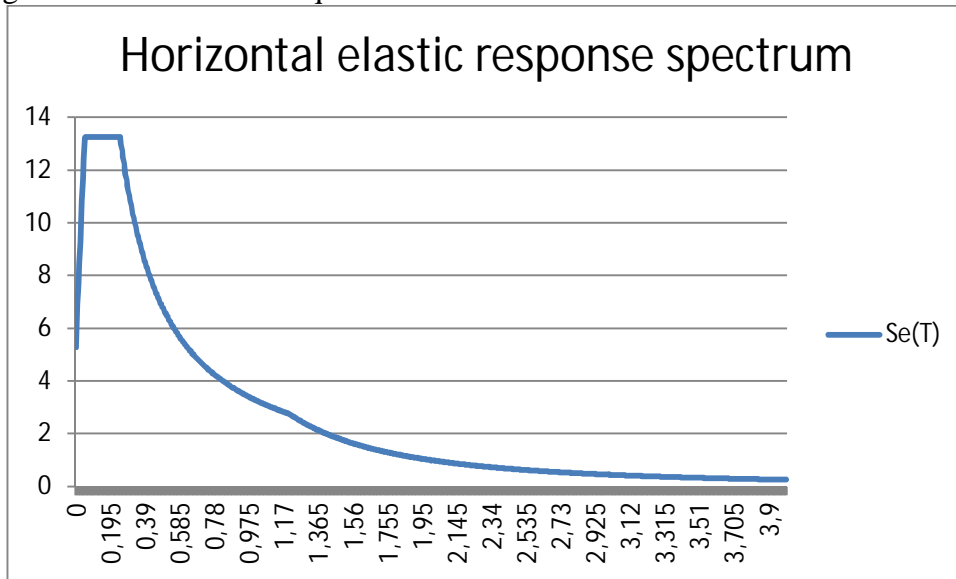
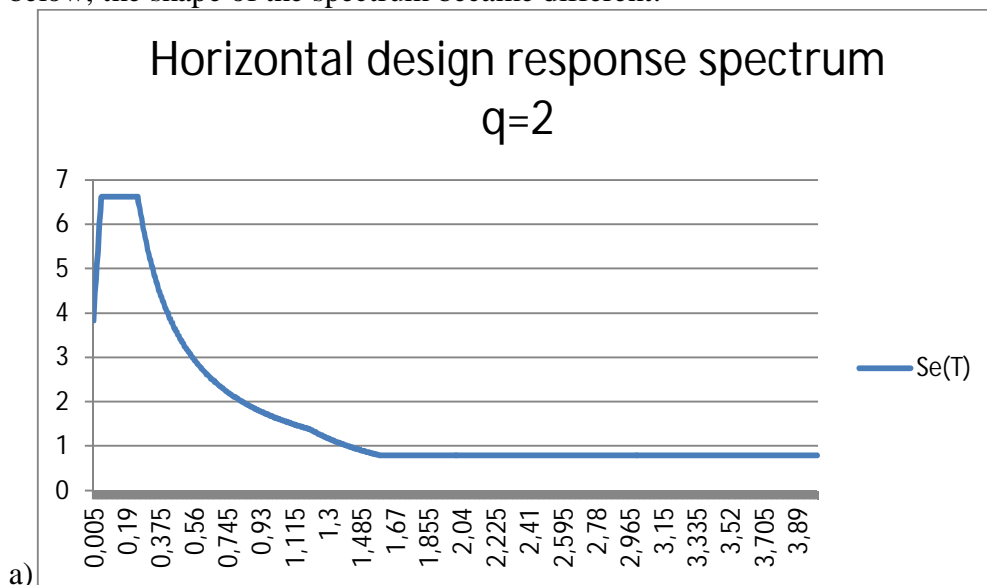


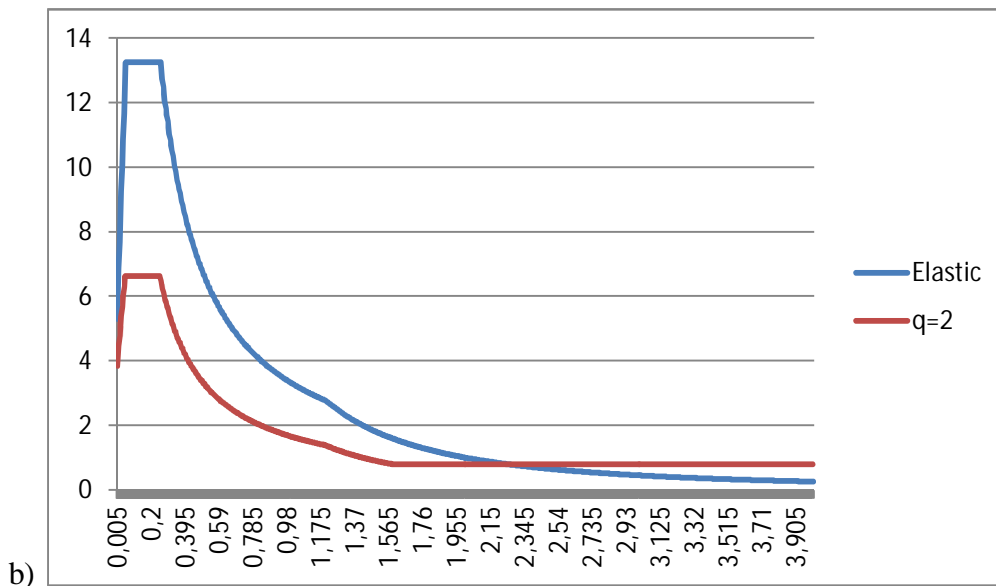
Figure 38 Horizontal elastic response spectrum

### Design response spectrum

To obtain the curve used in the MRS analysis, the elastic spectrum was reduced with the factor  $q$ . The reduction was done according to 3.2.2.5 of the code. As one may notice below, the shape of the spectrum became different.



a)



b) Design response spectrums: a) behavior factor equal to 2; b) comparison.

**Modal response spectrum analysis results**

A good way to check whether the analysis has been done correctly is to check the resulting base shear in the main direction (the direction of the building drift in the natural mode) – it should be approximately equal to the mass of the structure multiplied by the acceleration value for the natural period. The mass of the building with three full silos is 3656.14t. So, the base shear for the first mode of the MRS analysis in the direction of the building inclination in the natural shape should be:  $3656.14t \cdot 10^3 \cdot 0.7848m/s^2 = 2869.4kN$ . The analysis has shown the value of 2682kN. The difference occurs because not all the mass participated in the natural mode, so the results should be acceptable. The results are listed further. To obtain the maximum possible effect according to 4.3.3.3.2 of Eurocode 8, the CQC rule was used: the summarization is done by the software, so no simplifications are needed.

Three full silos

Direction of analysis	Seismic base shear (CQC)
Y	2682kN
X	5052kN

Torsional case

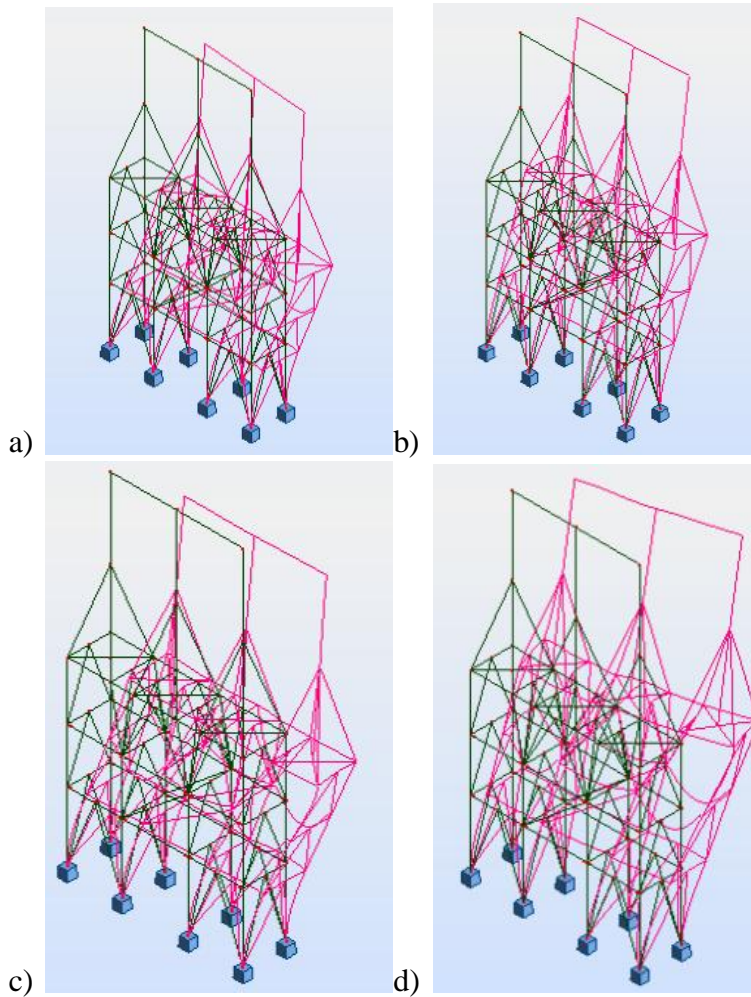
Direction of analysis	Seismic base shear (CQC)
Y	2001
X	3622

Horizontal seismic actions were combined using the rules of 4.3.3.5.1(3) of EC8. The main modes were specified to return the internal effects positive or negative signs lost during quadratic CQC combinations. In both cases, it was the main mode 1 for Y direction and main mode 2 for X.

**Seismic combinations**

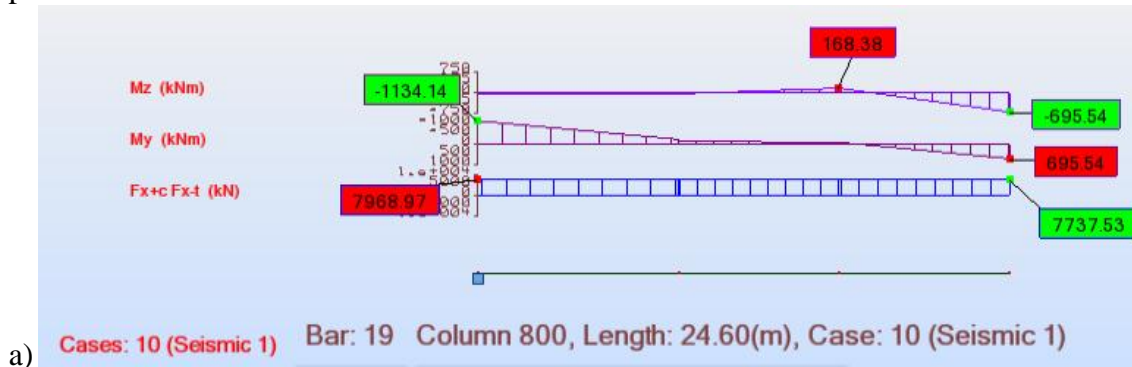
Earthquake effects were combined with other actions based on the rules of 6.4.3.4 of EN1990:

$G_k + Q_k \cdot \psi_2 + E_d$ , where  $\psi_2 = 0.8$  for the category E and  $E_d$  is the design earthquake effect. One may observe the deformation in the most dangerous combinations (when the seismic effect direction is the same as the direction of the accidental torsion eccentricity) below:



Deformation in the seismic design situation: case with all silos full, a – first seismic combination ( $E_d=Ex+0.3E_y$ ); b – second seismic combination ( $E_d=0.3*Ex+E_y$ ); torsion case, c – first combination; d – second combination.

Both cases will be checked for all the limit states. But for the capacity design of beams and columns it is needed to determine the most dangerous one. In the figures with deformation shapes it is obvious that the most axial force will come on the upper-right column, so its internal forces are compared. It must be noted that it is a critical column only under the specified combinations, but in reality an earthquake will act in both directions, so the columns and bracings will be arranged with a perfect symmetry in a plan and have identical sections and materials.



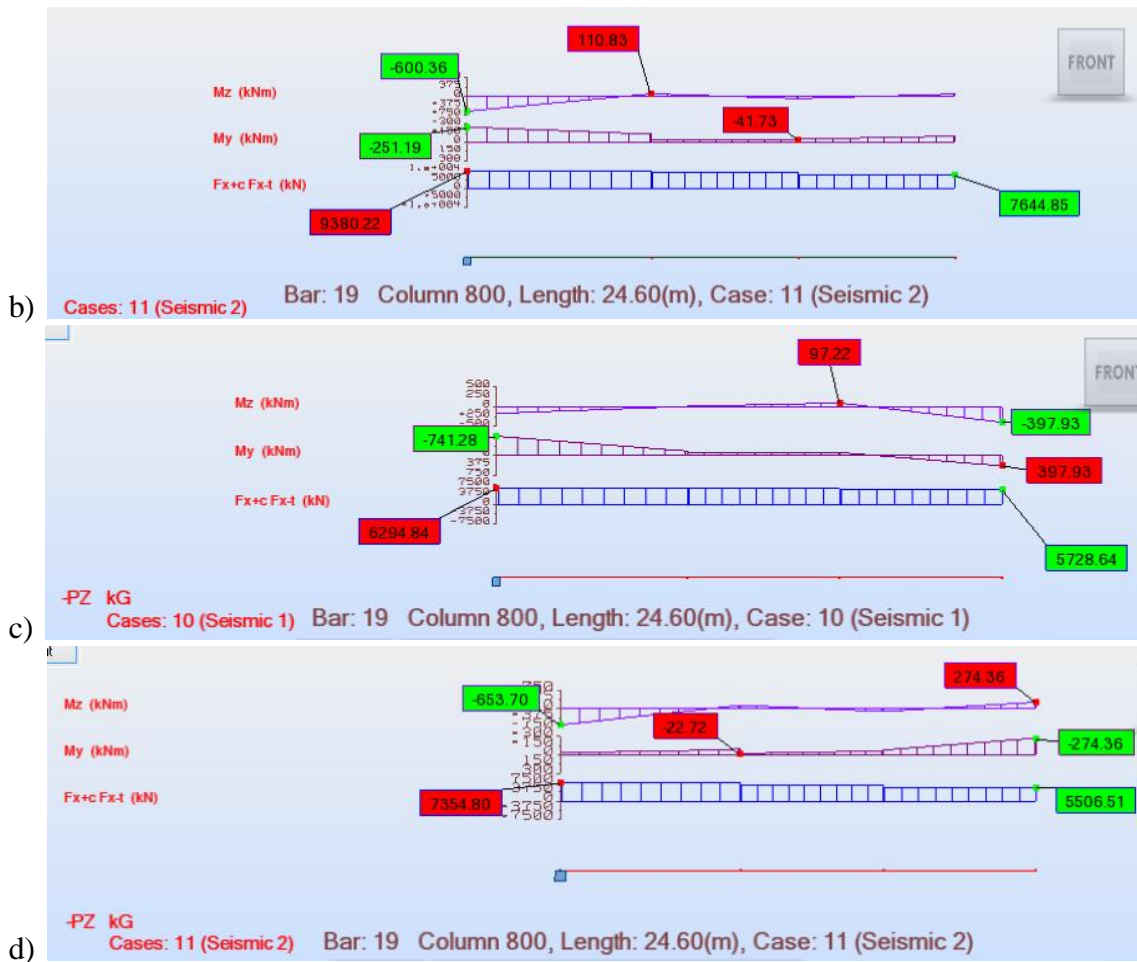


Figure 39 Internal forces in the critical column: a/b – case with three full silos; c/d – torsion case.

For the reason of higher internal forces, the case with three full silos will be used for the capacity design of beams and columns. All the cases will be checked at ULS and DLS.

### Damage limitation state check

The building is checked at a damage limit state using the rules given in 4.4.3 of the code. The goal is to limit the damage caused by an earthquake with the lower return period: a weaker one, because strong catastrophies happen more seldom.

The building certainly has brittle technical equipment attached to it, so the interstorey drift is restricted as specified in 4.4.3.2(1) a). The importance class is assumed to be II, so the recommended value of  $v$  is 0.5. The storey drifts were assessed using the "structural storeys" feature in Robot, afterwards multiplied by the behavior factor as demanded in 4.3.4 of EC8. The results are listed below.

Three silos full:

Damage limitation state	q	2	v	0.5		
Storey	h	d X	d Y	dr*v X	dr*v Y	limit
1	9.3	0.014	0.015	0.014	0.015	0.0465
2	7.4	0.017	0.014	0.017	0.014	0.037
3	7.9	0.014	0.016	0.014	0.016	0.0395

Torsion case:

Damage	q	2	v	0.5		
--------	---	---	---	-----	--	--

limitation state						
Storey	h	d X	d Y	dr*v X	dr*v Y	limit
1	9.3	0.01	0.011	0.01	0.011	0.0465
2	7.4	0.013	0.01	0.013	0.01	0.037
3	7.9	0.011	0.01	0.011	0.01	0.0395

### Second order effects

Obviously, it is more convenient to design a structure without considering the second order effects. The clause 4.4.2.2(2) allows that if they cause the moment less than 0.1 of the first order bending. The verification procedure is listed further.

Three silos full:

Storey	Ptot	Vtot X	Vtot Y	d X	d Y	h	Tetta X	Tetta Y
1	34554.72	5037	2697	0.015	0.018	9.3	0.02213	0.049596
2	33769.92	5002	2680	0.017	0.02	7.4	0.031019	0.068112
3	33010.8	4909	2642	0.016	0.019	7.9	0.027239	0.060101

Torsion case:

Storey	Ptot	Vtot X	Vtot Y	d X	d Y	h	Tetta X	Tetta Y
1	34554.72	3590	2188	0.01	0.011	9.3	0.0207	0.037359
2	33769.92	3540	2146	0.013	0.01	7.4	0.033517	0.04253
3	33010.8	3410	2059	0.011	0.01	7.9	0.026959	0.040588

In both cases ratio  $\theta$  is smaller than 0.1, therefore, no second order effects need to be accounted.

### Energy dissipation concept

In the seismic design, the acceleration values for the spectrum were reduced with the behavior factor. This means that when the base shear exceeds the design value during a real earthquake (it definitely will), plastic deformations will develop in certain members that will stop the internal force increasing in other ones. In other words – if you attach an elastic-perfectly plastic beam to a column and start pulling it, the column will never have to resist the force more than  $A \cdot f_y$  ( $A$  is the cross section area of the beam).

So, it is necessary to explain how our building reduces the design internal forces in its bearing elements. The energy dissipation takes place in tension diagonal bars, the compression ones will certainly buckle. Beams and columns are capacity designed, so that they are able to withstand that  $A \cdot f_y$  force, with respect to the strain hardening and  $f_y$  being probably stronger than the characteristic value.

### Ductility detailing

Frames with V-bracings are the most uncomfortable for capacity design. The reason comes from the two rules of the code:

- Compression diagonals must be accounted (6.7.2(2))
- In addition, they must be designed to resist buckling (6.7.3(6))

As it was said in this work, the energy is dissipated in tension bars, while compression ones are assumed to buckle. However, EC8 demands them to withstand at least the design seismic effect without a capacity loss. Usually, the reduction factor for the cross-sectional strength due to flexural buckling is about 0.5 (in a good case). This means, that the section is about two times stronger in plastic tension. The clause 6.7.4(1) obligates beams and columns to which the diagonals frame to be able to withstand the seismic action equal to  $F_{Ed} = F_G + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot F_E$ , where  $\Omega$  represents the ratio of section plastic resistance to the design axial force in bracings. Assuming the recommended value of  $\gamma$  1.25, the seismic action for beams and columns may reach  $1.25 \cdot 1.1 \cdot 1/0.5 \cdot F_E = 2.75 \cdot F_E$ , that is even higher than the maximum behavior factor.



Other types of diagonal bracings are not required to resist buckling, so the design force may be almost equal to the tension resistance ( $\Omega \sim 1$ ), which in addition to a higher  $q$  values and absence of beam capacity design makes them much more suitable for dissipative behavior.

It would be cheaper in all cases to design the concerned building in accordance with the non-dissipative behavior. However, this is not the purpose of the thesis. Therefore, the capacity design procedure is performed as it is required for the ductility class high.

### Slenderness of the diagonals

In accordance with 6.7.2 of Eurocode 8, the diagonal bars must have their non-dimensional slenderness within the following limits:

- Less or equal than 2 for V-bracings
- Between 1.3 and 2 for X-bracings

The verification can be seen below.

Slenderness check for V-bracings, $f_y=355\text{MPa}$					
Section	Length	$I_y=I_z$	A	Ncr	$\lambda$
200x8	8.57	35662500	5924	1006397	1.445563
250x10	9.75	87066700	9257	1898287	1.315735
220x12	7.96	64868500	9366	2121909	1.25178
220x10	8.43	57824600	8057	1686462	1.302305
250x12	8.76	98594200	10806	2662945	1.200233
260x12.5	9.09	115478800	11704	2896633	1.197663

There was a problem with the X-bracings. The intersection reduces the buckling length only about one axis. Even if no compression bars are modeled, this reduction must be taken into account, because it makes the diagonals too stocky while there is the lower limit for the non-dimensional slenderness (compression bars are not modeled because they are assumed to buckle soon enough, otherwise the columns will be overloaded with a non-accounted axial force). It is not difficult to design a slender bar – X-bracings do not have to be designed for buckling, so their resistance is governed only by tension. Therefore, the cross-section area, which can be easily adjusted. Nevertheless, X-bracings are the only horizontal force resisting element in one direction on the second floor. If their  $\Omega$ -ratio (see 6.7.4) varies significantly from the bracings of other stories, it will lead to a non-uniform dissipation throughout an earthquake: the second floor will yield while overstrengthened (due to buckling resistance) tension diagonals of other stories remain elastic. So, even if X-bracings are not designed for flexural buckling, in this case they have to be almost in the same way overstrengthened as if their  $F_{b,Rd}$  was controlled. This, in addition to the limits of  $\lambda$  require custom welded boxes to be used. The most optimal solution with respect to all the limits is the profile below:

b	h	t	A	$I_y$	$I_z$	Ncrz	$\lambda_z$	Ncry	$\lambda_y$
150	300	25	20000	207291667	63541667	3881377	1.352497	3165549	1.49763

### Cross-section classes

To ensure enough ductility for DCH approach, the code requires all the diagonals (dissipative members) to have the cross-section class 1 (6.5.3). All the bars are made of S355 steel, and the bending moment due to the self-weight is neglected. So, based on Table 5.2 of EN1993-1-1, their  $c/t$  ratio is restricted to be smaller than  $33 \cdot \epsilon = 26.85$ .

Section	$c/t$
200x8	18
250x10	18

220x12	10.333333
220x10	15
250x12	12.833333
260x12.5	12.8
300x150x25	10

### Capacity design of beams and columns

The case with three full silos have demonstrated higher internal forces in columns, so the capacity design will be based on it.

The arrangement of the diagonals is symmetric in the plan for every storey, so the requirement of 6.7.1(3) of the code is automatically fulfilled. The capacity design is fully based on the “reverse” usage ratio  $\Omega$  that must be inspected for all the diagonals.

Section	NPIRd	N Ed	$\Omega$
250x10	3265.74	1136	2.874771
220x12	3304.31	1257	2.628727
220x10	2842.38	925	3.072843
260x12.5	4129.3	1875	2.202293
300x150x25	7100	3295	2.15478
200x8	2090	663	3.152338
250x12	3812.34	1590	2.397698

One can notice that the X-bracing bars have the ratio smaller than the maximum times 0.75 (6.7.3(8)). Anyway, with the increase of cross-section (resistance) the stiffness also increases and the design force becomes more (the X-bracings “share” the lateral forces with the columns that are fixed to the foundation). The current value was the highest that was possible to achieve with respect to the slenderness limits. However, it will not be used in the capacity design verification.

To check beams and columns with the equation 6.12 (6.7.4) of the code, separate load combinations were created in Robot, where besides the gravity loads the seismic action multiplied by  $1.1 \cdot \gamma_{ov} \cdot \Omega = 1.1 \cdot 1.25 \cdot 2.2 = 3$  was added. Although the standard requires to verify the resistance based on the value of  $M_{Ed}$  in the seismic situation, for continuous columns it is safe to consider the full moment resulting from the yielded diagonal.

Besides, the energy is not dissipated through the bending of beams to which the V-bracings frame. So, they need to withstand the forces from the yielding diagonals without the formation of plastic hinges. The principles are listed in 6.7.4(2) of the standard. A beam must not fail while the tension diagonal yields and the compression one adds some post-buckling axial force, equal to 0.3 times the plastic resistance of the diagonal in tension. The critical case is the beam above the bracings 250x12, so it was designed locally against the effects above:

Angle	22.39723
NPIRd	3812
Vertical force	2467.108
Horizontal force	1888.211

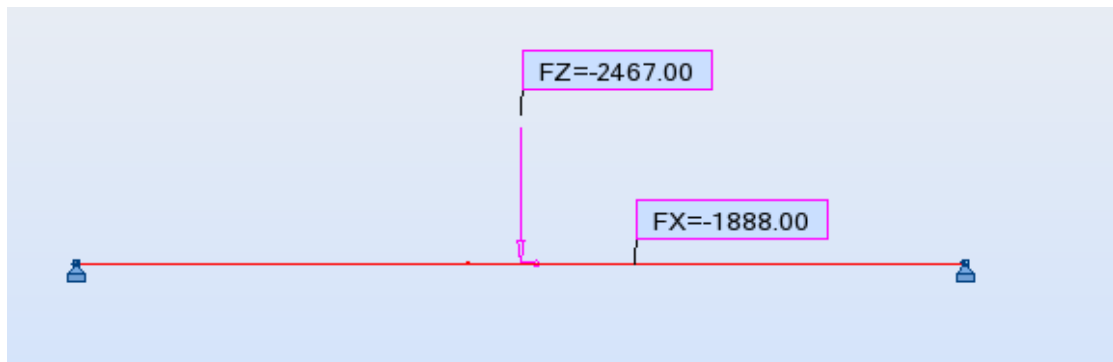


Figure 40 Local analysis of a critical beam

For the reason of very strong bracings, only HEB1000 profile is suitable.

### **Capacity design of connections**

The sub-chapter 6.5.5 requires the connection of dissipative members to be stronger with respect to the possible material overstrength. In our case, the dissipation takes place in the diagonal bars, so the joints of beams and columns do not have to be capacity designed. An example for one bracing is listed further:

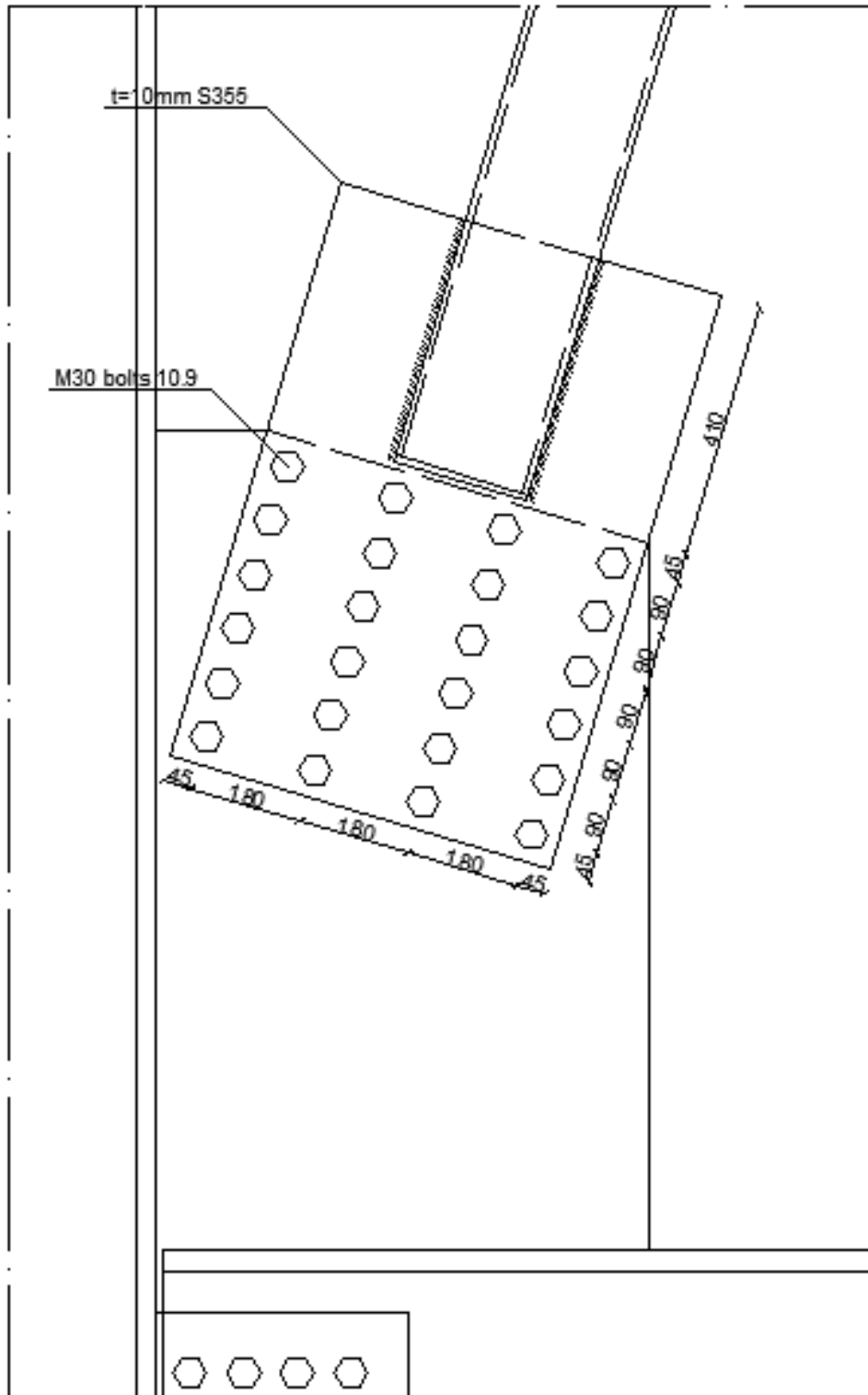


Figure 41 Suggested connection for the bracing 220x10

The connections are designed based on the rules of EN1993-1-8, with the resistance requirements from EC8, as described above.

The input data for the bracing bar:

Bracing	220x10		$\gamma_{ov}$	1.25
$f_y$	355		$\gamma_{M2}$	1.25

## Seismic design in Europe and the Russian Federation

$N_{Pl,Rd}$	2842.38		$f_u$	490
Required joint resistance	3908.273			

Bolts data and arrangement on the connection plate:

Bolts data					
Class	10.9	Diameter	30	Non-threaded area	707
$f_{ub}$	1000	Hole diameter	33	Threaded area	561
$f_{yb}$	900	Number of bolts	24		

Arrangement				
e1	45		$t_{pl}$	10
e2	45			
p1	90			
p2	180			
Shear resistance of bolts				
$\alpha_v$		0.5		
$F_{v,Rd}$		5385.6		

Bearing resistance of the plate				
	Edge end	end inner	Inner edge	Inner inner
$k_1$	2.118182	2.5	2.118182	2.5
$\alpha_d$	0.454545	0.454545	0.659091	0.659091
$\alpha_b$	0.454545	0.454545	0.659091	0.659091
Number	2	2	10	10
$F_{b,Rd}$	226.4529	267.2727	1641.783	1937.727
Total	4073.236			
Check the rule 6.5.5(5)				
$F_{v,Rd}$		$1.2 * F_{b,Rd}$		
	5385.6	4887.884	OK	
Block tearing				
$A_{nt}$	2280			
$A_{nv}$	15270			
$V_{eff,1,Rd}$	4023.489			

Welds	
Length	400
a	10
$\beta_w$	0.9
$f_{vw,d}$	251.4681
$F_{w,Rd}$	4023.49

## SNiP Seismicity definition

To define the site seismicity for the design, Table 1 of the standard was used. In reality, it will require a seismo-tectonic expertise to be carried out. Since the building is located outside the Russian Federation, the OCP-97 maps were not used. The seismicity is selected to be 9 degrees, because in that case the ground acceleration used in the design spectrum will be almost the same ( $4\text{m/s}^2$ ). The ground type corresponds to the type II of the SP classification.

**Modal analysis**

The building does not correspond to the regularity criteria stated in 5.3. The reason is that in the torsion case the maximum value of floor displacement differs from the average by more than 10% (see the picture of modal analysis). This torsion cannot be analyzed by a cantilever-type model, so the spatial model was used. In addition, SNIIP does not restrict the modeling of diagonal bracings, so both tension and compression diagonals were modeled in X-bracings.

For the load to mass conversion, the coefficients were chosen according to 5.1 of SP. The dead load was multiplied by the factor 0.9, and the imposed one by 0.5. This is a bit smaller than the factors in Eurocodes, where the self-weight is never reduced.

The structure is designed for the Design Earthquake situation.

The clause 5.16 does not obligate to account the accidental torsion eccentricity for the spatial model, so it was not defined.

Since the mass participation for the horizontal components was enough in the first two vibration modes, the final number of modes accounted was chosen to be three, because of the demands in 5.9 of SNIIP. The results of the modal analysis are listed below:

Three silos full:

Case	Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)
6	1	0.69	1.44	0	94.73	0
6	2	1	1	97.98	94.73	0
6	3	1.14	0.88	97.98	94.74	0

Torsion case:

Case	Mode	Frequency (Hz)	Period (sec)	Rel.mas.UX (%)	Rel.mas.UY (%)	Rel.mas.UZ (%)
6	1	0.91	1.1	0	88.13	0
6	2	1.44	0.7	96.15	88.13	0.04
6	3	1.94	0.52	96.15	92.22	0.04

**Design spectrum definition**

To obtain the design seismic action effects, the modal response spectrum analysis was used. The spectrum was defined as described in 5.3.1 of this thesis.

$K_0=1$  according to Table 3 of the standard;

The behavior factor  $K_1=0.22$  for the steel structures with diagonal bracings (Table 4 of SP).

$K_{\psi}=1.0$ , because the building is not a tower or any other relatively high structure (Table 5 of SP);

The ground acceleration was left  $3.924\text{m/s}^2$  (but not 4, as required for the site seismicity of 9 degrees), so that the comparison is more notable.

The clause 5.6(3) gives the type 1 spectrum for the ground category II.

With regard to all the coefficients, the design spectrum can be seen below:

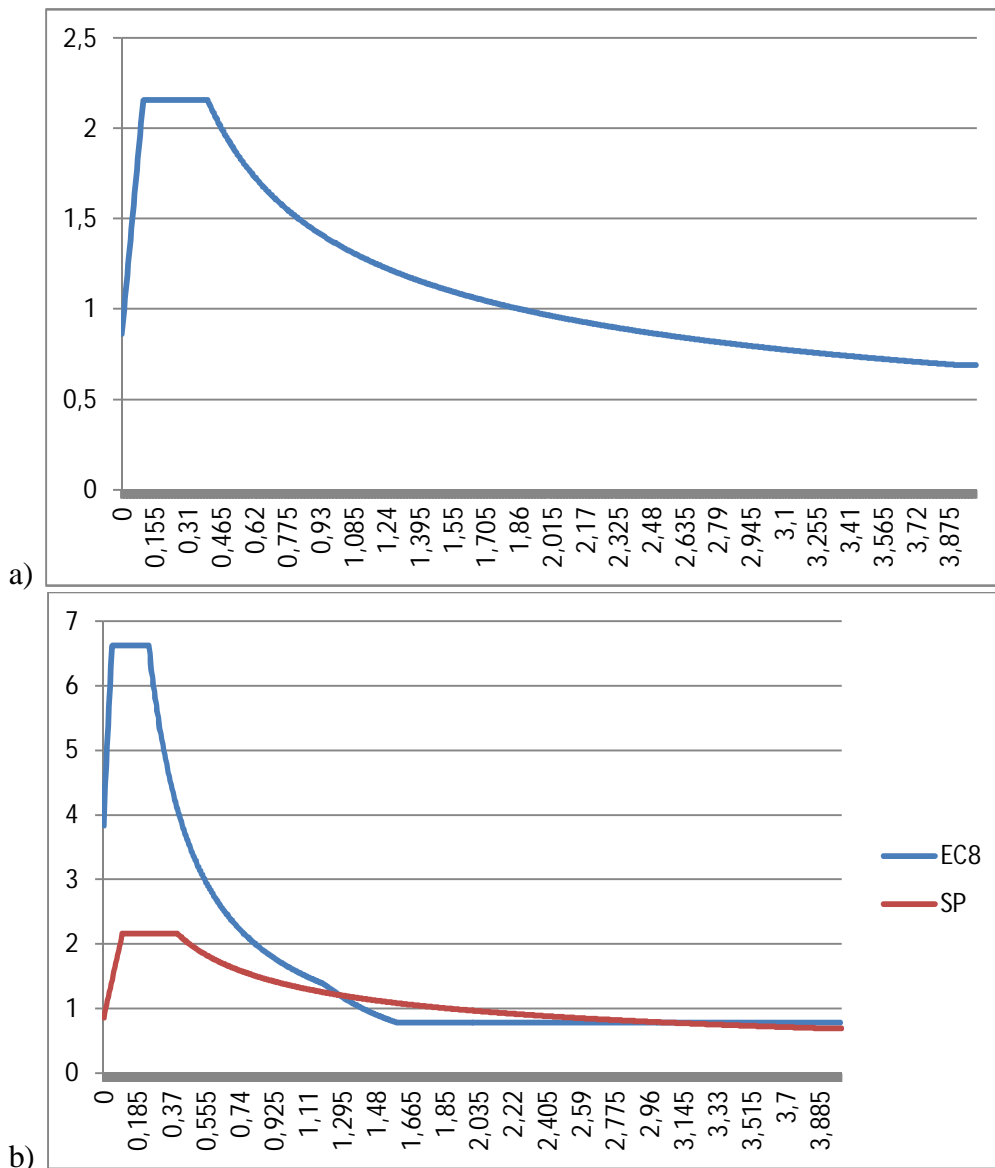


Figure 42 A) - Design spectrum for MRS analysis, b) – comparison with EC8 spectrum.

It is notable that the values may be higher than those in Eurocodes, in spite of the better behavior factor. Anyhow, the curve itself is much smoother in SNiP, a higher natural period is gradually more beneficial in the European standards. As it was said in this thesis, one way to increase the period is to account ground elasticity. Now we see that it is really worth doing when designing according to Eurocodes.

The total seismic mass of the building is 3139t for the case of three full silos. A quick check of the MRS analysis results:

$$F_b = 3138890.54 \text{kg} * 1.137471274 \text{m/s}^2 * 0.9473 (\text{mass participation percentage}) = 3382 \text{kN}$$

The program has shown almost the same results (3379.64kN), so the results are trusted.

Three silos full:

Direction	Seismic base shear (10% rule)
Y	3379.64kN
X	4189.93kN

Torsion case:

Direction	Seismic base shear (10% rule)
Y	1781.80kN
X	2443.12kN

### **Seismic combinations**

For the case of a spatial model, the standard requires not to divide the effect into components. The maximum modal effect was obtained with the 10% combination rule. The seismic combinations were created according to 5.1 of SP:

$F_{Ed}=0.5*Q_k+0.9*G_k+E_d$ , where  $E_d$  is the design seismic effect.

### **Ultimate limit state check**

The statement 5.18 allows to check the buildings only at ULS. In addition, the additional working condition coefficient is taken into account as specified in the 5.15 of SP. When performing the check for buckling, it depends on the element slenderness ( $l_0/i$ ). The critical bracings have their slenderness ratio 70, so according to Table 6 the  $m_r$  coefficient is interpolated to 1.1 for the buckling check. As for the section verification (beams in bending and shear), the factor  $m_r$  was input as 1.3.

To introduce here an exact comparison, the ULS check of members was performed according to Eurocode 3. The Russian standards for steel have different rules and material design resistance definition besides dissimilar characteristic yield strength. This is outside the scope of the thesis, so the European norms were used. In SNiP, the design material strength is multiplied by the working condition coefficient. In our case it was accounted by dividing the design effect by it in separate combinations in Robot, which is basically the same.

### **Ductility detailing**

The only energy dissipation regulation for the braced frames that one may find in SP is to locate the dissipation zones outside the joints. However, no rules for the capacity design are given. So, one has to apply other sources of information, e.g. Eurocode 8.

### **Conclusions**

This appendix has the goal to represent in practice everything written in the conclusion chapter of the theoretical part. Obviously, it has demonstrated the necessity of alternations available in Eurocodes: the non-dissipative approach would be much more economic. Higher behavior factors in case of a moment resisting frame might have changed this, although according to steelconstruction.info the connections are much more expensive for the reason of welding. Anyway, the standard offers plenty of various methods, and it is up to the designer to choose.

As for SNiP, one may see that there are the requirements for the ductile behavior (locate the dissipation zones outside the joints, etc.), but there is no guidance. The document named “The official guidance” introduces only a set of solutions that are not universal.