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Seismic Design according to Eurocode 8 and AzDTN 2.3-1 code: Case study of multistorey building in Baku

Master degree in Civil Engineering – Building Construction

Amir Alasgarov

Leiria, October of 2020

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Dissertation report under the supervision of Professor Joao Paulo Veludo Vieira Pereira, Professor Hugo Filipe Pinheiro Rodrigues and Professor Khalid Mehemmed Oglu.

Leiria, October of 2020

Originality and Copyright

Dedication

To my mom.

Acknowledgments

First and for most I must give many thanks to Professor Joao Veludo, who trained me as an engineer and made a huge contribution to my future. The experience that Professor Veludo gave me is of great value, and always found time for my doubts and questions. Professor Joao Veludo always showed the highest level of support in any field and guided me along the true path. I am infinitely grateful to the professor and appreciate the effort that he put into my thesis.

Also, I am grateful for Professor Hugo Rodrigues who made a timely contribution to the work, and shared his great experience with us.

I am grateful for the Head of Civil Engineering department, Professor Azar Akhmadov from Baku Engineering University, because of his experience shared with me at the beginning of my journey, which created a strong foundation for to build on.

Finally, I am thankful to the whole department of Civil Engineering of the Polytechnic Institute of Leiria, whose doors were always open for me.

Abstract

Detailed and entire research in the comparison of seismic behaviour of reinforced concrete structures under European seismic code and Azerbaijan seismic code are not yet provided. However, there are big interests from the Azerbaijan Republic to involve European codes as state construction norms in Azerbaijan. Because of this, comparison has been made to help Azerbaijan move to European Standards.

The following aspects were taken into account in order to make a comparison of seismic codes: design states, structural types, ground conditions, important classes, seismic zones, horizontal elastic response spectrum, base shear force and distribution of the horizontal seismic forces. Chapter 4 compares results of the case study in Chapter 3. To make a seismic analysis, the existing constructed structure was taken into account to apply seismic codes of Europe and Azerbaijan. The Robot Structural Analysis software was used for modelling structure and analysing it behaviour and results.

The several aspects of both seismic codes are quite similar, such as design limit states, seismic zones, but the most similar aspect observed in the research is that of the characteristics of ground types. The almost 80% of difference in base shear force is observed for studied building. Also, studies show that Azerbaijan code is much more conservative in terms of shape of elastic response spectrum in poor soil conditions than European seismic code.

Basically, overall results of research show that Azerbaijan Construction Norms, in terms of seismic design, are much more conservative in all aspects comparing with European codes. The main reason for this is the high seismicity of number of regions of Azerbaijan. Also, to consider is the cost of construction materials in Azerbaijan is way less that the cost in Europe.

Keywords: seismic design, seismic analysis, reinforced concrete structure, elastic response spectrum, seismic combinations, ground types.

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List of Abbreviations and Acronyms

ULS	Ultimate Limit State
SLS	Serviceability Limit States
LSD	Limit State Design
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PGD	Peak Ground Displacement
SDOF	Single Degree of Freedom
FEM	Finite Element Method
NCR	No-Collapse Requirement
DLR	Damage Limitation Requirement
SPT	Standard Penetration Test
OCR	Over-consolidation Ratio
RC	Reinforced Concrete
FSLS	First Stage Limit State
SSLS	Second Stage Limit State
CC	Corner Column
IC	Inner Column
EC	Edge Column
EB	Edge Beam
IB	Inner Beam

1. Introduction

1.1.Scope

The definition of "earthquake" according to Cambridge Dictionary [43] is "a sudden violent movement of the earth's surface, sometimes causing great damage". There are more terms which describes earthquakes such as, tremor, quake, subsurface seismic activity, temblor.

Through many years people have faced earthquakes. This has subsequently led to the development of the ability to get along with periodic movement of the earth's tectonic plates and to be prepared for their occurrence.

The map of seismic hazard presented in Figure 1.1.

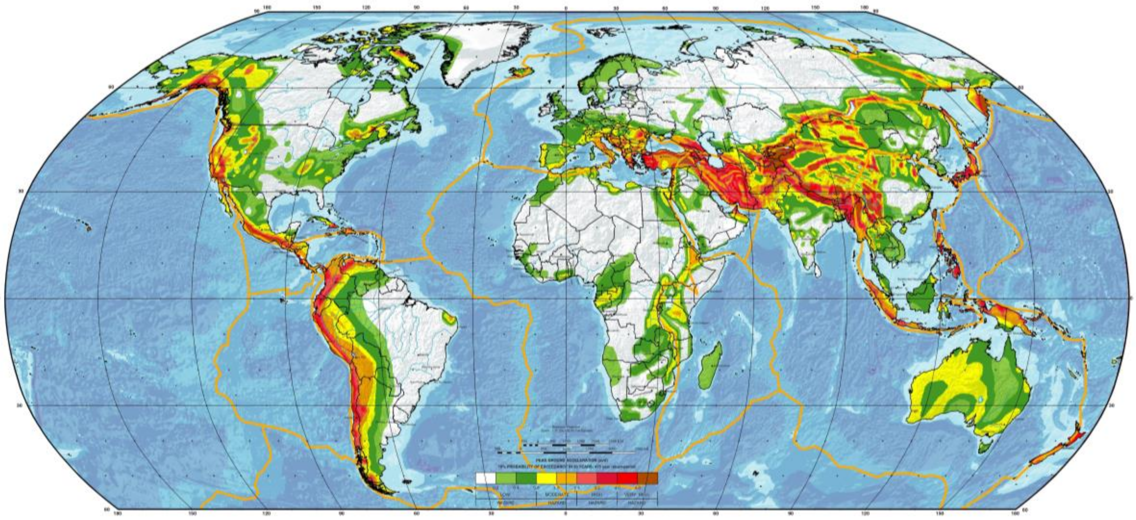
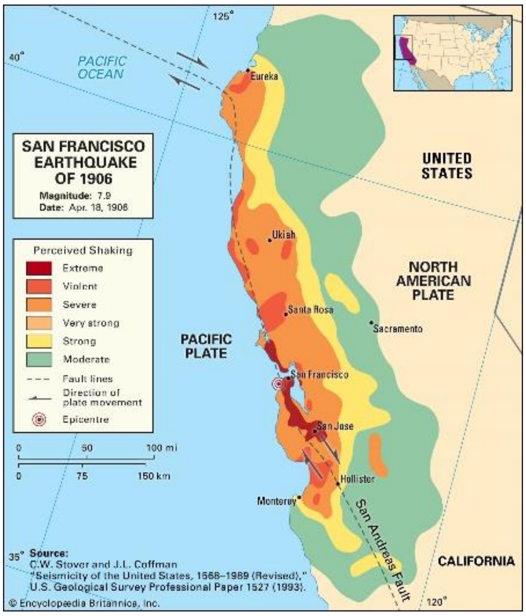
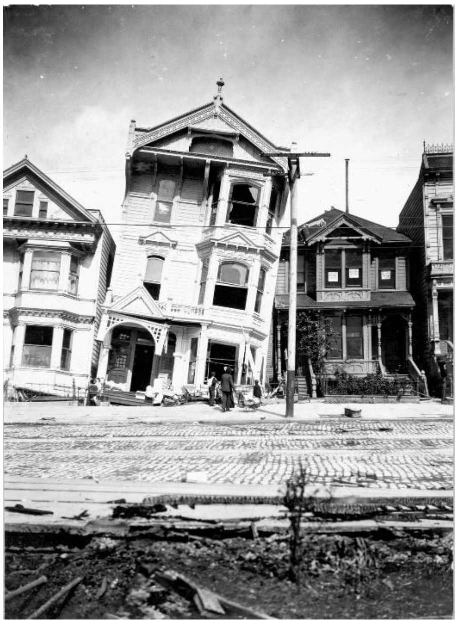


Figure 1.1: Global seismic hazard map

Structural engineers have to provide proper seismic design to reach the main goal which is humans safety and reduction of the material losses. One example of a tragedy is an earthquake in San Francisco in 1906 (see Figure 1.2) with a magnitude 7.9, which was the cause of the deaths of 3000 people, and lots of damage to infrastructure and buildings [41].



a) Hazard map of eartquake



b) Soil failure

c) After Earthquake

Figure 1.2: San Francisco earthquake of 1906. "By courtesy of Encyclopædia Britannica, Inc., copyright 2020; used with permission."

Figure 1.2(b) shows that the underlying soil condition has direct relationship with the earthquake response of structure. The properties of the ground type at a given site can be characterized through adequate geotechnical investigations.

Typical causes of damages of reinforced concrete structures can be divided into the following types [42]:

- Shear and flexural failure;
- Inadequate capacity and detailing of the joints;
- Structural irregularities, in plan or in elevation, “weak-storey”, “soft-storey”;
- Short-column mechanism;
- “Strong-Beam Weak Column”.

The scheme of typical damages on structures are presented in Figure 1.3.

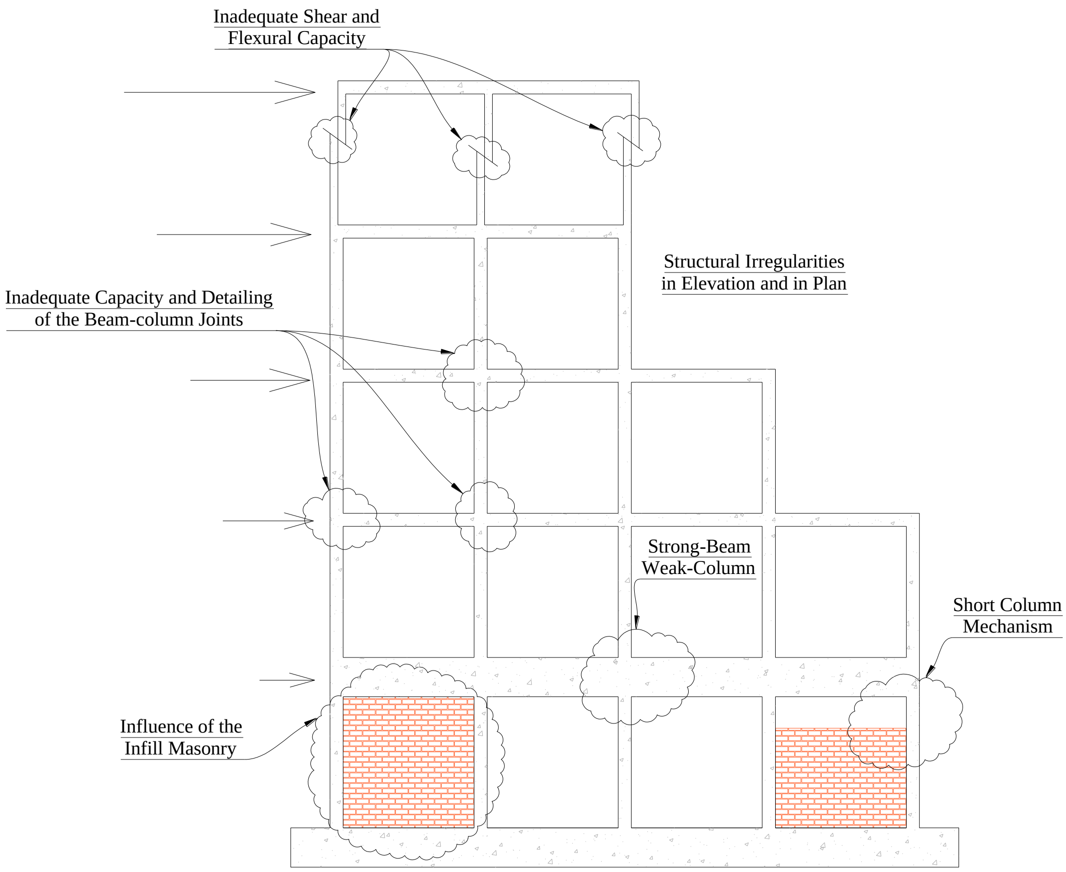


Figure 1.3: Typical causes of damage and failure of RC structures

“Strong-Beam Weak-Column”

Concrete structures and those which do not incorporate seismic resistant design criteria have poor column-to-beam and column-to-slab connections. With emphasis on design for static loads, slabs tend to be very stiff and much stronger than columns [33]. Columns deform and plastify long before beams or slabs. The consequences of “Strong-Beam Weak-Column” case are presented on Figure 1.4 (a, b). This is due to lack of confinement and poor detail of the transverse reinforcement.

a) b)

Figure 1.4: "Strong-Beam Weak-Column" failure [33]

Inadequate capacity of the joints

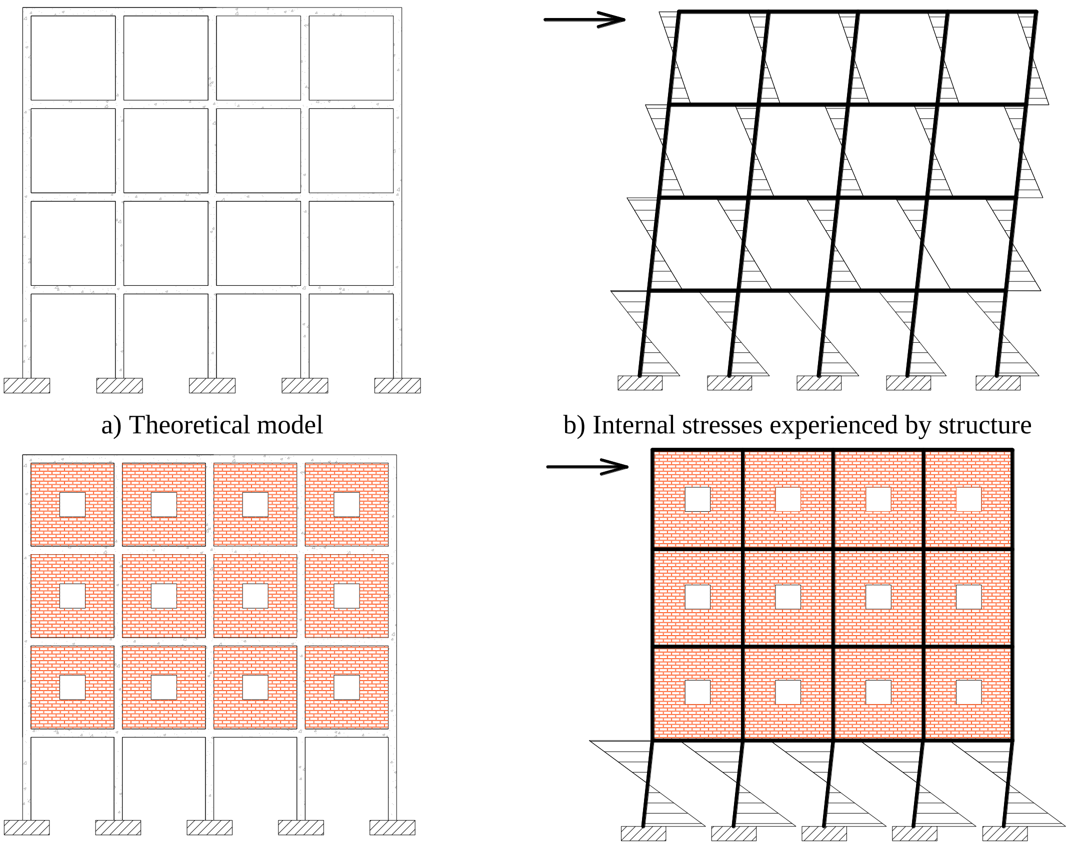
The poor performance of inadequate moment-resistant, non-ductile brittle reinforced concrete frames is dramatically illustrated in Figure 1.5 (a, b), which despite its lightness and carrying no loads other than its self-weight has developed plastic hinges in column base and top with permanent non-recoverable deformations.

a) b)

Figure 1.5: Inadequate detailing of the joints [34]

Structural irregularities

Soft-storey configuration in structures is a type of construction where any one storey of the building is more flexible (less stiffness) when compared with other storeys. This may be located at the bottom as shown in Figure 1.6 (c), or at any intermediate points, where the storey above or below it may be stiffer compared to itself. This is considered to be a weak element in the perspective of seismic forces. Figure 1.6 shows an example of structure which ground soft-storey, experienced tremendously big shear stress in columns in first storeys, which leads to collapse of the structure. The presence of walls in upper storeys makes them much stiffer compared to the ground storey. This makes the upper storeys to behave like a single block.



a) Theoretical model

b) Internal stresses experienced by structure

c) Model with rigid upper storeys

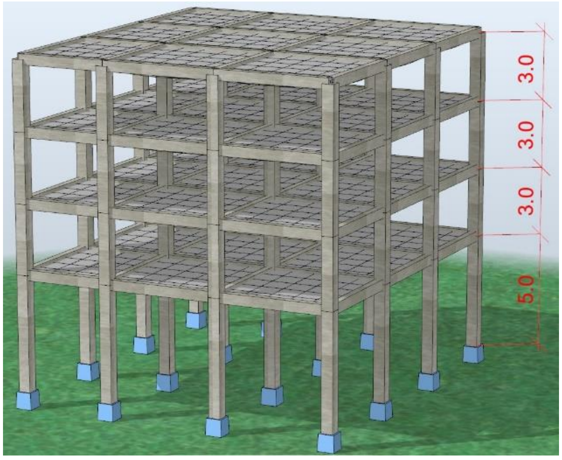
d) Internal stresses experienced by structure

Figure 1.6: Distribution of internal forces along the height of the building

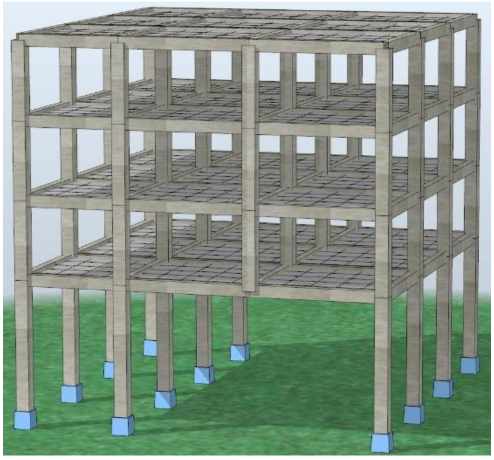
Many structural damages recorded due to earthquake have had major problems of change in stiffness and strength along their vertical configuration. It is not only essential to have symmetry along the horizontal direction, i.e. in the plan, but also in the vertical direction.

This is a factor that assures lateral stiffness. Abrupt changes in the vertical plan should be avoided to the maximum.

Another soft-storey example is presented in Figure 1.7. The presence of huge differences between storeys' height in structure (see Figure 1.7 (a)), is one of the common examples of poor behaviour under seismic action, which leads to collapse of the structure. Figure 1.7 (b) shows soft-storey arrangement where the columns are arranged in a discontinuous manner. This itself has problems in a discontinuity in the load transfer, which becomes severe under seismic forces.



a) Irregular in elevation structure



b) Irregular in plan structure

Figure 1.7: Different types of structural irregularities

The behaviour of structure with irregularities in elevation after earthquake are presented in Figure 1.8 (a, b).

a) Behaviour of irregular structure under seismic action

b) Structure experienced permanent drift

Figure 1.8: Structure with ground soft storey after Al-Hoceima earthquake 24/02/2004 [34]

The ground storey of the building presented in Figure 1.8 (a) includes open plan shops on the ground storey with densely populated apartments above. This was a classic candidate for soft-storey damage. The building has 6-degree permanent drift inclinations (see Figure 1.8 (b), due to seismic actions.

Short columns

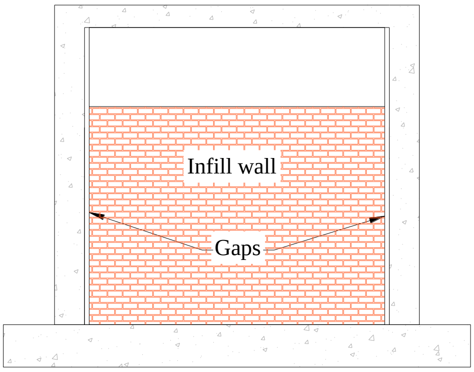
The short-column effect takes place in many structures, while structure's frame infill by masonry walls include openings for windows and other portions of columns sandwiched by infill masonry short-column effect appear. Example of short column failure shown in Figure 1.9.

a) Al- Hocema 2004 [34]

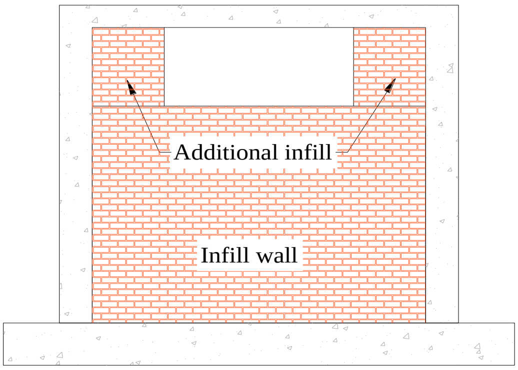
b) Adana-Ceyran 1998 [37]

Figure 1.9: Example of short column failure

One of the best ways to eliminate the short-column effect is to separate the infill wall from the bounding structural frame with an adequate gap as presented in Figure 1.10 (a), that would allow the column to freely bend [37], also adding infill wall segments, see Figure 1.10 (b), that would slightly reduce the opening width next to short column.



a) Gaps to reduce shear forces



b) Additional infill to reduce shear forces

Figure 1.10: Solutions to reduce shear forces in “short column” issue

Shear and flexural failure

Due to shear forces experienced by column it can fail in any place between joints as far as shear force is constant along the height of the column [39]. Examples of shear failure due to lack of shear resistance is diagonal crack as shown in Figure 1.11 and Figure 1.12.

Figure 1.11: Shear failure of a column of Shinkansen bridge. 2004, Japan [40]

Figure 1.12: Diagonal shear crack in lightly reinforced concrete pier of the Wu Shu bridge in Taichung [39]

Flexural failure is always accompanied by horizontal cracks and loss of concrete cover. Flexural capacity of corroded column decreases due to deteriorated concrete cross-section and reduced steel bar area. Furthermore, corrosion of transverse reinforcement reduces the modulus of elasticity of steel bar and as a result, the confinement rate decreases. Therefore, corroded RC column may not be able to develop the full flexural capacity [38]. Figure 1.13 shows the consequences of insufficient flexural ductility.

a) San Fernando Road Overhead damage in the 1971 San Fernando earthquake

b) Hashin Epressway, Pier 46, damage in the 1995 Hyogo-Ken earthquake

Figure 1.13: Examples of flexural failure due to seismic action [40]

1.2.Subject, Relevance and Main Goals of the Work

The main goal of work is to compare main aspects of construction and design of the seismic code of Azerbaijan Republic [6] with the Eurocode 8 [3]. The best approach to compare different seismic codes is to take an existing structure applying two codes to compare results, which was essentially presented by the author. By using “Autodesk Robot Structural Analysis” [44] software, the author made a three-dimensional model to evaluate differences presented by applied codes. The elastic response spectrum and base shear of studied structure were compared. The factors and coefficients available for comparison have been compared in the second chapter.

Azerbaijan is a small, resource-rich country located on the far east end on the European continent. The country is actively moving towards practices used and followed by the European Union, including the European standards. The author, being an international student, will contribute to the future of the republic of Azerbaijan, and help the transition to European building standards.

1.3. Thesis Structure

The first chapter of the thesis presents a brief introduction of the entire report and touches on the importance of seismic design.

The second chapter presents the fundamental aspects of each code, in terms of seismic design.

The third chapter presents a brief introduction to studied structure in which EN 1998-1 [3] and AzDTN 2.3-1 [6] were applied. Also, methodology of performance is described.

The fourth chapter presents the main results and the analysis of important parameters such as results of base shear, displacement drift, ground acceleration and analysis of some of the structural members according to seismic codes applied.

The fifth chapter presents a conclusion and summation of work done.

The appendices A to E include additional figures and tables to clarify ideas written in the main text. Appendices A and B show more architectural sketches of facades and structural plans of several storeys. Appendix C shows several structural elements made in a three-dimensional model. Appendix D shows values of partial factors used in computation of earth load. Appendix E shows more displacement shape modes.

2. Seismic Action According to Eurocode 8 and AzDTN 2.3-1 Code

2.1. Introduction

In 1975, the Commission of the European Community established a set of harmonized technical rules for the design of construction works. The first European codes were generated in the 1980's. The Structural Eurocode programme comprises ten standards generally consisting of several parts. Eurocode 8 (part 1), denoted in general by EN 1998-1 [3], applies to the design and construction of buildings and civil engineering works in seismic areas. Eurocode 8 [3] is composed of six parts dealing with different types of construction, such as buildings, bridges, silos, pipelines, retaining structure and chimneys. EN 1998-1 [3] is used to design buildings in seismic regions and is subdivided into ten chapters.

AzDTN 2.3 -1 "Construction in Seismic Areas" [6], is based on Russian seismic code SNIP II - 7 -81* "Construction in Seismic Areas". AzDTN 2.3-1 [6] was established in 2010 and comprise one part. The SNIP II - 7 -81* loses its validity after 2010. Azerbaijan code touches on some topics to design buildings for seismic resistance.

This chapter includes the most important rules about seismic action and seismic design according to Eurocode 8 (EC8) [3] and AzDTN 2.3-1 [6]. In section 2.4 both seismic codes were compared in important parameters considering seismic design.

2.2. Seismic Analysis According to Eurocode 8

2.2.1. Requirements and Limit States

The design of buildings under seismic action should obey two requirements described in European seismic code. The first requirement asks that after seismic action aftershock structure should be strong enough to withstand and have residual load bearing capacity to save human lives. That requirement is named "no-collapse requirement".

– *No collapse Requirement (NCR).*

The design seismic action is expressed in terms of:

a) The reference seismic action associated with a reference probability of exceedance, P_{NCR}, in 50 years or a reference return period, T_{NCR}.

b) The importance factor γ_I, described hereinafter, consider reliability differentiation.

The other, but no less important requirement of EN 1998-1 [3] is named “damage limitation requirement” and requires that the construction and design of structure should be strong enough to prevent occurrence of damage which leads to unreasonable expenses in relation to the cost of construction.

– *Damage Limitation Requirement (DLR).*

The design seismic action is expressed in terms of:

a) The seismic action for DLR has a probability of exceedance, P_{DLR}, in 10 years and a return period T_{DLR}.

b) Recommended P_{DLR} = 10%, which corresponds to T_{DLR} = 95 years

In order to satisfy the fundamental requirements European construction code requires that structure meet two limit states described in EN 1990 [1], ultimate limit state, (ULS) which concerns with safety of people and safety of the structure and serviceability limit state (SLS) which is concerned with functioning of the structure and comfort of people.

2.2.2. Seismic Action and Soil Parameters

Three parameters are used for a quantitative definition of the soil profile, such as value of the average shear wave velocity ($v_{s,30}$), the number of blows in the standard penetration test (N_{SPT}) and undrained cohesive resistance (c_u). The average shear wave velocity ($v_{s.30}$) computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \tag{1}$$

Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 2.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

Ground types A to D range from rock or other rock-like formations to loose cohesionless soils or soft cohesive soils.

Ground Type E is essentially characterised by a sharp stiffness contrast between a surface layer and the underlying much stiffer formation.

Two additional soil profiles (S1 and S2) are also included in Table 2.1. For sites with ground conditions matching either one of these ground types, special studies for the definition of the seismic action are required.

Table 2.1: Ground types according Eurocode 8 [3]

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30sm)	c_u(kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	>50	>250
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 – 50	70-250
D	Deposits of loose-to-medium cohesionless soil, or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high-water content	<100 (indicative)	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Seismic action and zones

The seismic action to be considered for design purposes should be based on the hazard assessment. Seismic hazard is normally represented by hazard curves that depict the exceedance probability of a certain seismologic parameter. It is widely recognized that peak values of the ground motion parameters are not good descriptors of the severity of an earthquake and of its possible consequences on construction. Hence the more recent trend is to describe the seismic hazard by the values of the spectral ordinates.

In Eurocode 8 [3], the seismic hazard is described by the value of the reference peak ground acceleration on ground type A (a_{gR}). The reference peak ground acceleration (a_{gR}), for each seismic zone, corresponds to the reference return period (T_{NCR}), chosen by the National Authorities. Structures, except ordinary ones, are designed to fulfil the no collapse requirement under a design ground acceleration determined by expression (2). The design acceleration (a_g) in the described below expression (2) corresponding to ground type A.

$$a_g = \gamma_I a_{gR} \tag{2}$$

The value of the importance factor γ_I (see Table 2.2) in expression (2) is equal to 1.0 for structures of ordinary importance. Values of the importance factor other than 1.0 are considered to correspond to mean return periods other than the reference, T_{NCR}.

Table 2.2: Importance classes and factors according to Eurocode 8 [3]

Importance class	Buildings	Importance factor γ_I
I	Buildings of low importance such as agricultural buildings.	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, such as museums and archives.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, such as hospitals and fire stations.	1.4

The seismic hazard at a site can be represented by a hazard curve showing the exceedance probabilities associated with different levels of a given engineering seismology parameter, such as peak ground acceleration (PGA), velocity (PGV), displacement (PGD) and duration, for a given period of exposure.

Methods for evaluating earthquake input for different levels include zonation map-based procedures and site-specific studies. Map-based procedures, such as those normally provided by national authorities in Europe, use maps of the peak ground acceleration to define the seismic input at one or more different hazard levels and under different site conditions. According to EN 1998-1 the recommended choice is the use of two types of spectra, Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5.5 it is recommended that Type 2 spectrum is adopted, otherwise Type 1 should be considered. The European Seismic Hazard Map (Figure 2.1), shows Peak Horizontal Ground Acceleration to be reached with 10 % probability in 50 years, corresponding to the average recurrence of such ground motions every 475 years, as prescribed by the national building codes in Europe for standard buildings.

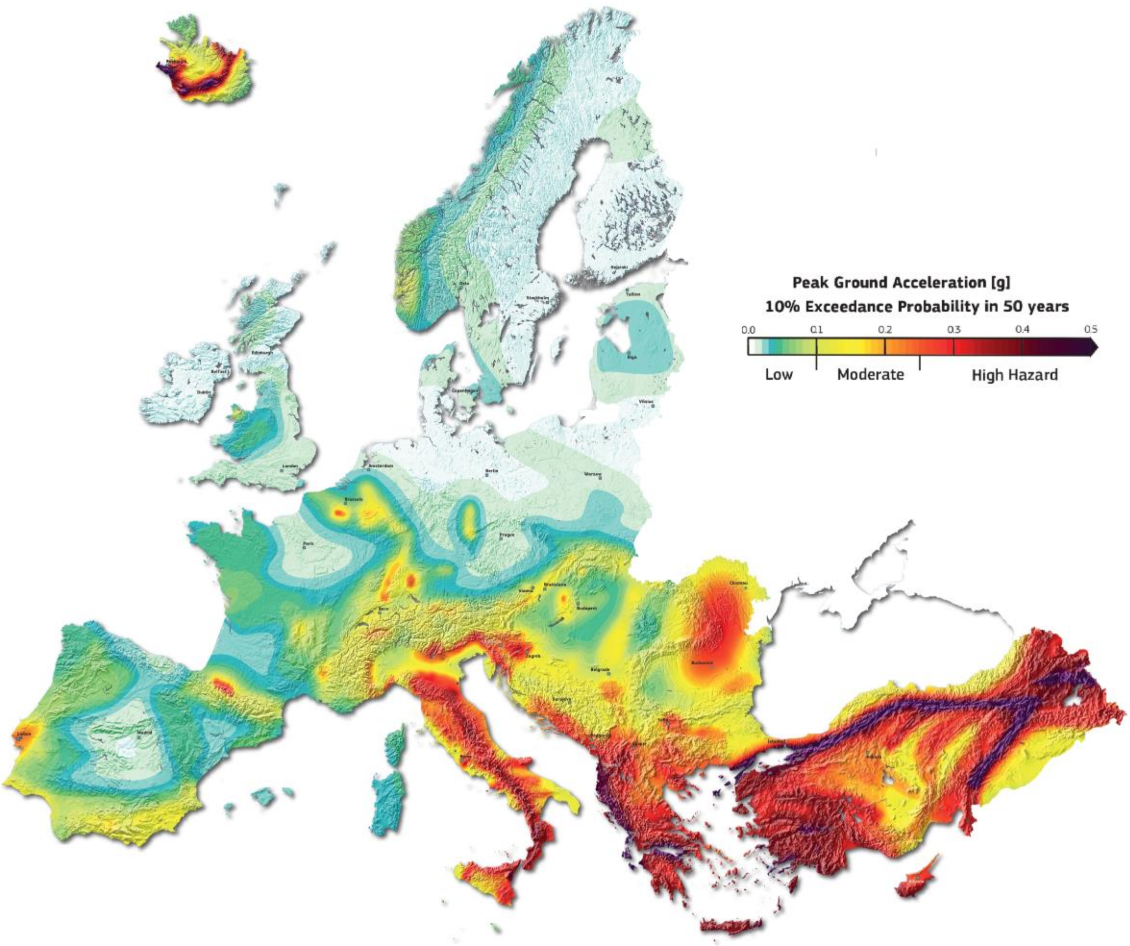


Figure 2.1: Seismic hazard map of Europe

Horizontral elastic response spectrum

The earthquake ground motion at a given site is described by the response spectrum, which may be elastic, inelastic or design. The elastic response spectrum is the theoretical response of a single degree of freedom (SDOF) system in the elastic range. The inelastic response spectrum is the theoretical response of a SDOF system with inelastic load deformation characteristics. The design response spectrum is smoothed and adjusted spectrum taking into account non-theoretical features and requirements for safe design, which mean providing a minimum base shear for long period structures.

Horizontal components of ground motion are mainly caused by secondary shear S waves. The wavelength of these seismic waves is longer than that of primary P waves, see Figure 2.2. S-waves are more destructive and dangerous than P- waves, due to larger amplitude and transversal movement.

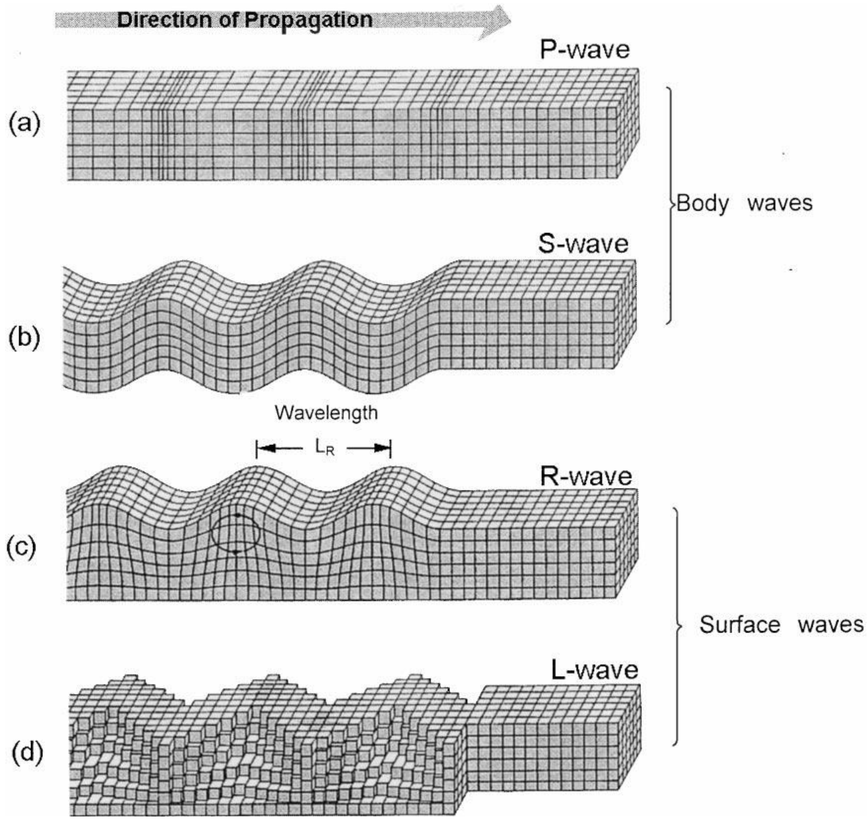


Figure 2.2 Four types of earthquake waves [29]

Horizontal components of the seismic action are defined in Eurocode 8 through the horizontal elastic response spectrum given in EN 1998-1 [3], where $S_e(T)$ is the value of the elastic response spectrum for the vibration period T of a linear SDOF system and is defined by following expressions:

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2,5 - 1) \right] \tag{3}$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \tag{4}$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right] \tag{5}$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5\left[\frac{T_C T_D}{T^2}\right] \tag{6}$$

where

$S_e(T)$ is the elastic response spectrum;

T is the vibration period of linear SDOF system;

a_g is the design ground acceleration on type A ground ($a_g = \gamma_1 a_{gR}$);

T_B is the lower limit of the period of the constant spectral acceleration branch;

T_C is the upper limit of the period of the constant spectral acceleration branch;

T_D is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor;

η is the damping correction factor with a reference value of η = 1 for 5% viscous damping, and determined by following expression:

$$\eta = \sqrt{10/(5+\zeta)} \ge 0{,}55 \tag{7}$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage.

For each ground type values of T_B, T_C, T_D and soil factor S, varies from country to county and presented in National Annex.

There are two type of elastic response spectra, which distinguish by surface-wave magnitude M_s. Type 1 elastic response spectra refers to surface-wave magnitude Ms, greater than 5,5, consequently Type 2 refers to surface-wave magnitude M_s less than 5,5.

M_s is the surface wave magnitude which is a scale of earthquake based on Rayleigh surface waves travelling in top layers.

For the five ground types A, B, C, D and E the recommended values of the parameters soil factor (S) and vibration period on a point B (T_B), C (T_C) and D (T_D) are given in Table 2.3.

The basic spectral shape is composed by three branches presented in Figure 2.3.

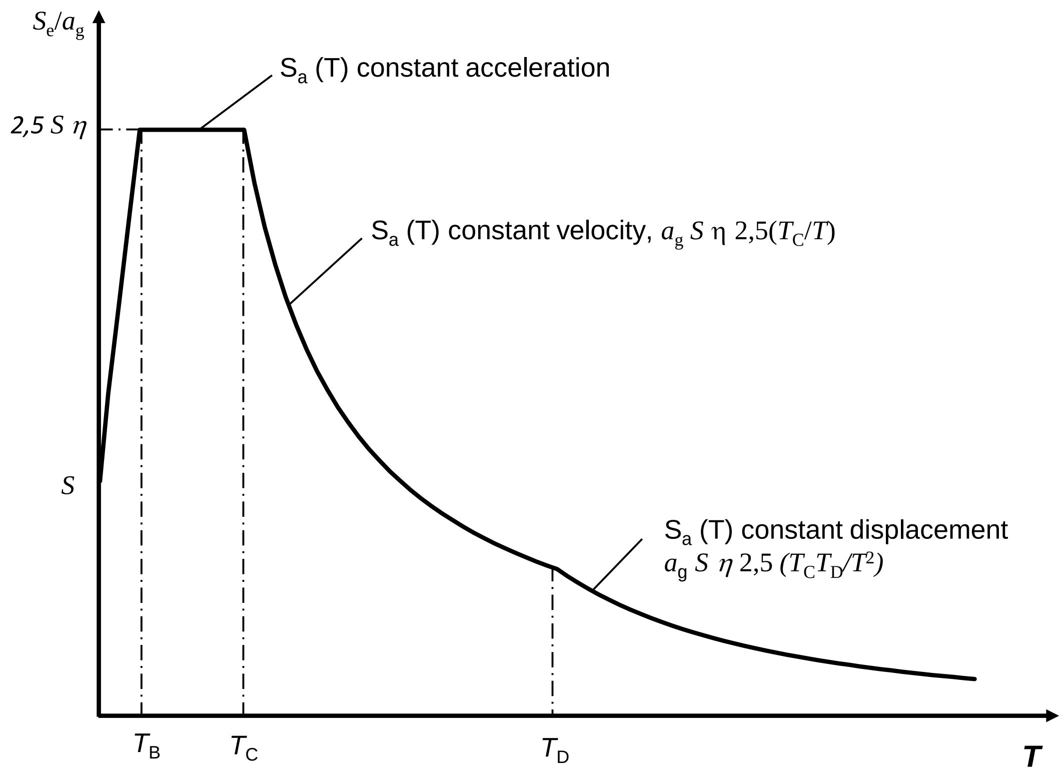


Figure 2.3: Basic shape of the elastic response spectrum according to EN 1998-1 [3]

Table 2.3 describe values of parameters S, T_B, T_C and T_D for high magnitude earthquakes Type 1 ($M_s > 5{,}5$).

Table 2.3: Values of the parameters for Type 1 elastic response spectra

Ground Type	S	T_B (s)	T_C(s)	T_D(s)
A	1,00	0,4	0,4	2,0
B	1,20	0,5	0,5	2,0
C	1,15	0,6	0,6	2,0
D	1,35	0,8	0,8	2,0
E	1,40	0,5	0,5	2,0

Table 2.4 describe values of parameters S, T_B, T_C and T_D for low magnitude earthquakes Type 2 ($M_s < 5{,}5$).

Table 2.4: Values of the parameters for Type 2 elastic response spectra

Ground Type	S	T_B (s)	T_C(s)	T_D(s)
A	1,00	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,50	0,10	0,25	1,2
D	1,80	0,10	0,30	1,2
E	1,60	0,05	0,25	1,2

Figure 2.4 present elastic response spectra Type 1 for five ground type A to E.

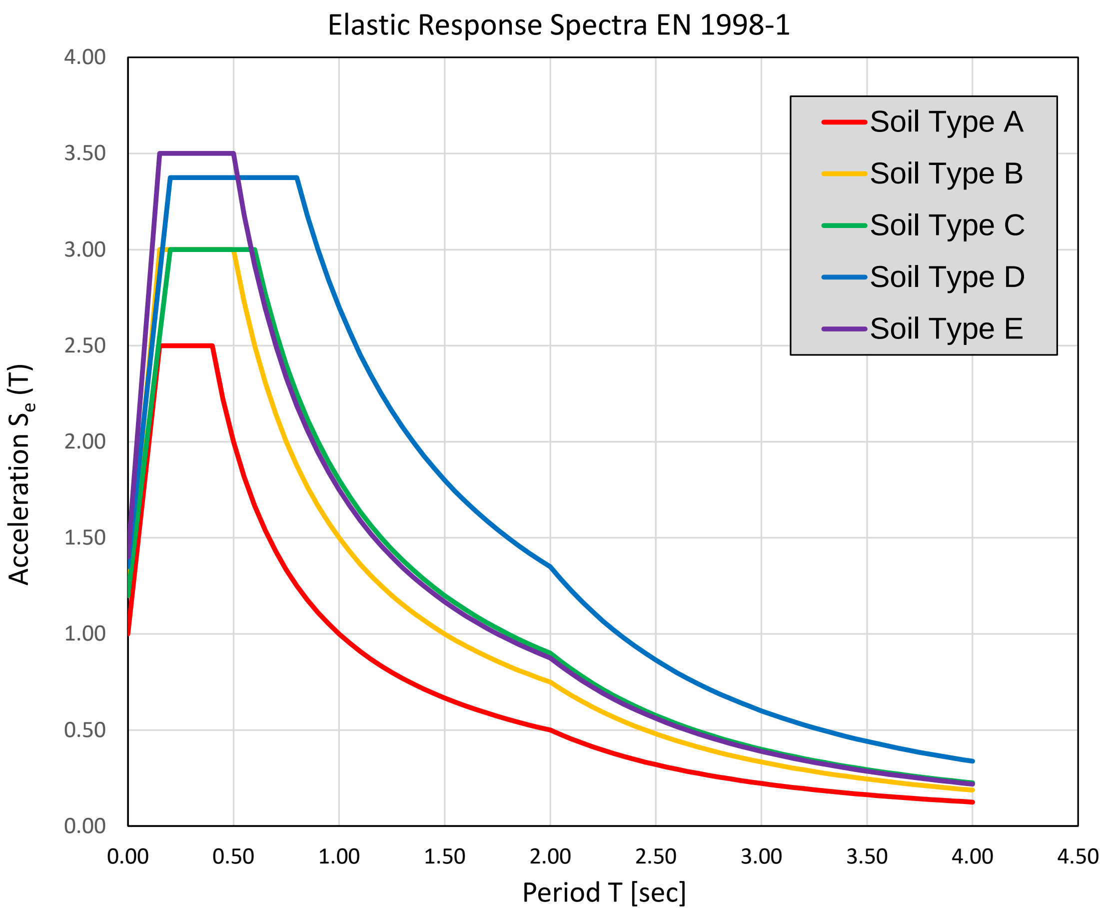


Figure 2.4: Elastic response spectra Type 1 for five soil types (5% damping)

The maximum value of spectral response acceleration for constant spectral acceleration branch for soil type A, B, C, D and E are 2,5, 3,0, 2,875, 3,375 and 3,5 respectively. The beginning of the constant displacement response range for all types of soil is on 2,0 seconds.

Ground types S_1 and S_2 described in Table 2.1 require special studies for obtaining values of soil factor S, T_B, T_C, T_D.

The elastic acceleration response spectrum, $S_e(T)$, could be transformed to elastic displacement response spectrum, $S_{De}(T)$, by the following expression:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \tag{8}$$

Expression (8) is suitable for vibration period not greater than 4,0 s.

Vertical elastic response spectrum

The vertical elastic response spectrum, $S_{ve}(T)$, expressed by following expressions (9-12).

$$0 \le T \le T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3,0 - 1)\right] \tag{9}$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \tag{10}$$

$$T_C \le T \le T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \left[\frac{T_C}{T}\right] \tag{11}$$

$$T_D \le T \le 4s : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \left[\frac{T_C \cdot T_D}{T^2}\right] \tag{12}$$

In Table 2.5 presented values to determine vertical elastic response spectra.

Table 2.5: Recommended values of parameters describing the vertical elastic response spectra

Spectrum	a_{vg}/a_g	T_B (s)	T_C (s)	T_D (s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

2.2.3. Buildings Design Under Seimic Actions

In order to reach more predictable behaviour of structure under seismic action, the following principals should be applied to design of structure [3]:

- uniformity, symmetry and redundancy;
- adequate foundation;
- diaphragmatic behaviour at storey level;

– bi-directional resistance and stiffness;

– torsional resistance and stiffness

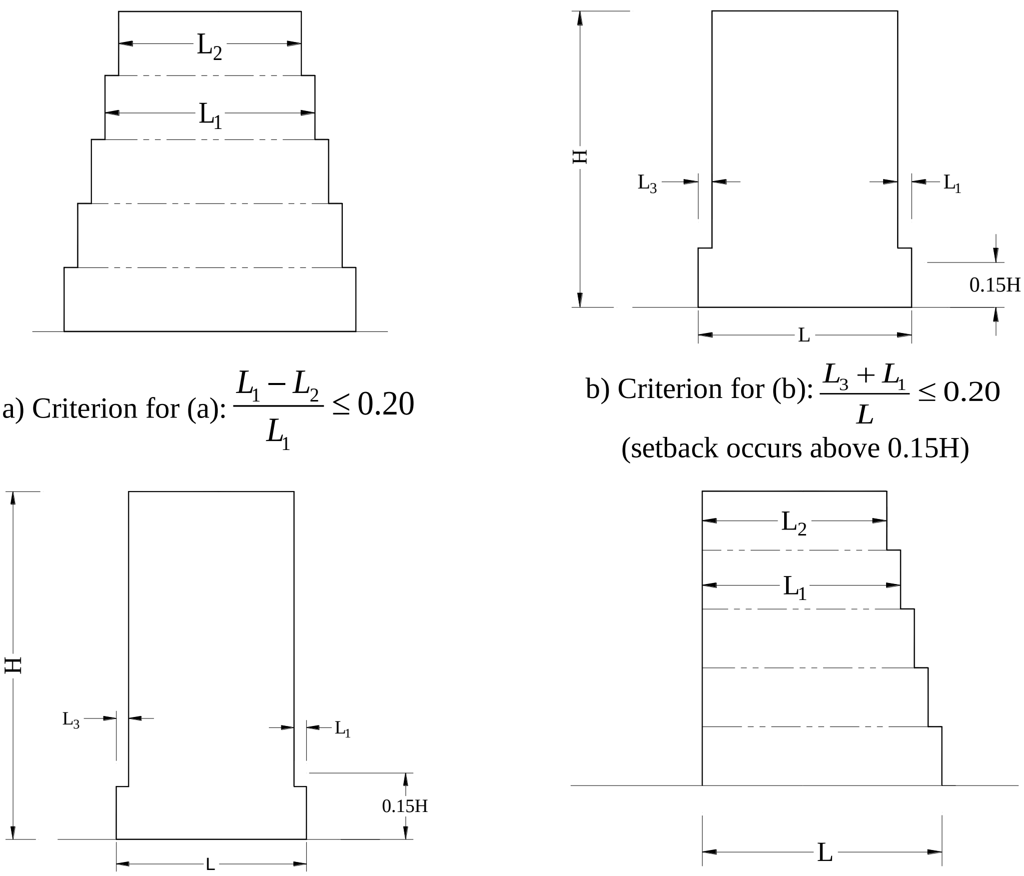
– structural simplicity.

Also, buildings should be regular in elevation. To reach regularity in elevation several conditions must be satisfied, such as:

– systems which resist to lateral load (shear walls, cores, frames) must be uninterrupted from foundation to the last storey of building;

– mass of the individual storeys and lateral stiffness shall change gradually, and do not present abrupt changes from foundation to the last storey of building;

In case building include several setbacks in elevation, following rules shall be applied:



a) Criterion for (a): $\frac{L_1 - L_2}{L_1} \le 0.20$

b) Criterion for (b): $\frac{L_3 + L_1}{L} \le 0.20$
(setback occurs above 0.15H)

Criterion for (c): $\frac{L_3 + L_1}{L} \le 0.50$
(setback occurs below 0.15H)

Criteria for (d): $\frac{L - L_2}{L} \le 0.30$

$$\frac{L_1 - L_2}{L_1} \le 0.10$$

Figure 2.5: Criteria for regularity of buildings with setbacks EN 1998-1 [3]

Included combinations were used to verify Ultimate Limit State (ULS) as well as Serviceability Limit State (SLS).

Ultimate Limit State

- Combinations of actions for seismic design situations:

$$E_d = \sum_{j\geq1} G_{k,j} \text{"+"} A_{\mathrm{Ed}} \text{"+"} \sum_{\mathrm{i}\geq1} \psi_{2,\mathrm{i}} Q_{\mathrm{k,i}} \tag{13}$$

where:

$\gamma_{G,j}$ is the partial factor for permanent action *j*;

$G_{k,j}$ is the characteristic value of permanent action *j*

$Q_{k,1}$ is the characteristic value of the leading variable action *1*

$Q_{k,I}$ is the characteristic value accompanying variable action *i*;

A_{Ed} is the design value of seismic action $A_{Ed} = \gamma_I A_{Ek}$;

$\psi_{2,I}$ is the factor for quasi-permanent value of a variable action *i*

In order to obtain loads for security verification of Ultimate Limit State, most unfavourable load assumption must be taken.

Serviceability Limit State

For Serviceability Limit State following combinations used.

- Quasi-permanent load combination:

$$E_d = \sum_{j\geq1} G_{k,j} \text{"+"} \sum_{i\geq1} \psi_{2,\mathrm{i}} Q_{\mathrm{k,i}} \tag{14}$$

- Characteristic load combination:

$$E_d = \sum_{j\geq1} G_{k,j} \text{"+"} Q_{\mathrm{k},1} \sum_{i>1} \psi_{0,\mathrm{i}} Q_{\mathrm{k,i}} \tag{15}$$

Table 2.6 shows recommended values of ψ factor used in combinations.

Table 2.6: Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area, vehicle weight ≤ 30 kN	0,7	0,7	0,6
Category G: traffic area, 30 kN < vehicle weight ≤ 160 kN	0,7	0,5	0,3
Category H: roofs	0	0	0

The horizontal components, in both directions X and Y, of the seismic action (E_{Edx}, E_{Edy}), should be applied simultaneously. The combinations below should be used for determine action effect due to seismic action.

$$E_{Edx} \text{"+"} 0,30 E_{Edy} \tag{16}$$

$$0,30 E_{Edx} \text{"+"} E_{Edy} \tag{17}$$

where

E_{Edx} represents the action effects due to the application of the seismic action along the chosen horizontal axis *x* of the structure;

E_{Edy} represents the action effects due to the application of the seismic action along the orthogonal horizontal axis *y* of the structure.

The seismic action in one of the directions should also include 30 % of seismic actions other direction.

In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, the calculated centre of mass at each storey *i* shall be considered as being displaced from its nominal location in each direction by an accidental eccentricity:

$$e_{ai} = \pm 0,05 \cdot L_i \tag{18}$$

where

e_{ai} is the accidental eccentricity of storey mass I from its nominal location, applied in the same direction at all storeys;

L_i is the storey-dimension perpendicular to the direction of the seismic action

Distribution of the horizontal seismic forces

Horizontal forces F_i, shall be applied to structure to imitate seismic action effects and determined by following expression:

$$F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_j \cdot m_j} \tag{19}$$

where

F_i is the horizontal force acting on storey i;

F_b is the seismic base shear in shear in accordance with expression 20;

s_i,s_j are the displacement of masses m_i, m_j in the fundamental mode shape;

m_i,m_j are the storey masses.

The horizontal forces F_i, should computed by expression 21, in case horizontal displacement increasing linearly along the height.

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \tag{20}$$

where

z_i,z_j are the heights of the masses m_i,m_j above the level of application of the seismic action.

The horizontal forces F_i shall be linearly distributed to the whole height of structure.

Base shear

For both horizontal direction seismic base shear force F_b, shall be determined by expression (21).

$$F_b = S_d(T_1) \cdot m \cdot \lambda \tag{21}$$

where

$S_d(T_1)$ is the ordinate of the design spectrum at period T_1;

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the mass of the building, above the foundation or above the top of a rigid basement.

λ is the correction factor, the value of which is equal to: λ = 0,85 if $T_1 \leq 2\ T_C$ and the building has more than two storeys, or λ = 1,0 otherwise.

The fundamental period T_1, for buildings which heights do not exceed 40 meters could be approximated by expression

$$T_1 = C_t \cdot H^{3/4} \tag{22}$$

where

C_t presented in Table 2.7

H is the height of the building, from the foundation or from the top of a rigid basement.

Table 2.7: Values of C_t for expression (22)

Structure type	C_t
Moment resistant space steel frames	0,085
Moment resistant space concrete frames	0,075
Eccentrically braced steel frames	0,075
All other structures	0,050

The value of C_t, for structures with concrete or masonry shear walls could be determined by expression (23).

$$C_t = 0,075 / \sqrt{A_c} \tag{23}$$

where

$$A_c = \sum \left[A_i \cdot (0,2 + (l_{wi} / H))^2 \right] \tag{24}$$

and

A_c is the total effective area of the shear walls in the first storey of the building, in m^2;

A_i is the effective cross-sectional area of shear wall i in the direction considered in the first storey of the building, in m^2;

l_{wi} is the length of the shear wall i in the first storey in the direction parallel to the applied forces, with the restriction that l_{wi}/H should not exceed 0,9.

Also, fundamental period could be obtained by expression (25)

$$T_1 = 2 \cdot \sqrt{d} \tag{25}$$

where

d is the lateral elastic displacement of the top of the building, due to the gravity loads applied in the horizontal direction.

2.2.4. Particular Factors and Rules

Structural types

According to how structures respond to seismic action concrete buildings shall be classified into several structural types, such as:

- Torsionally flexible systems
- Dual system of frames and walls
- System of large lightly reinforced walls
- Inverted pendulum systems
- Frame systems
- Wall systems either coupled or uncoupled walls.

Inverted pendulum systems and torsionally flexible systems have specific undesirable features, for that reason values of behaviour factor q lower. The reason to reduce behaviour factor is to keep responses closer to the elastic range.

Concrete buildings could be classified into two types of structural systems, first in one horizontal direction and second in another horizontal direction, excluding torsionally flexible systems.

Behaviour factor

The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural system according to the relevant ductility classes. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions. The behaviour factor q, is the value which depend on structural systems and materials.

Concrete buildings may alternatively be designed for low dissipation capacity and low ductility, and neglecting the specific provisions.

For each design direction the upper limit value of the behaviour factor q, shall be derived by expression 27.

$$q = q_o k_w \geq 1,5 \tag{26}$$

Where q_o is the basic value of the behaviour factor for buildings presenting regularity in elevation, see Table 2.8, k_w is the factor reflecting the prevailing failure mode in structural systems with walls shall be taken according expression (27).

$$k_{\mathrm{w}} = \left\{ \begin{array}{c} 1{,}00, \mathit{for\ frame\ and\ frame - equivalent\ dual\ systems} \\ (1+\alpha_{\mathrm{o}})/3 \leq 1, \text{but not less than } 0{,}5, \text{for wall, wall} - \text{equivalent} \\ \text{and torsionally flexible systems} \end{array} \right\} \tag{27}$$

Table 2.8 – Basic value of behaviour factor. q_o, for systems regular in elevation

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

In case building do not present regularity in elevation, behaviour factor q_o, should be reduced by 20%.

For buildings which presents regularity in plan values of multiplication factor α_u/α_1, shown in Table 2.9, may be applied.

Table 2.9: Multiplication factor for regular in plan buildings

Types of structural systems	Description	α_u/α_1

Frames or frame-equivalent dual systems	One-storey buildings	1,1
	Multi-storey, one bay frames	1,2
	Multi-storey, multi-bay frames or frame-equivalent dual structures	1,3
Wall or wall-equivalent dual systems	Wall systems with only two uncoupled walls per horizontal direction	1,0
	Other uncoupled wall systems	1,1
	Wall-equivalent dual, or coupled wall systems	1,2

where

α_u/α_1 is the overstrength ratio

2.3. Seismic analysis according to AzDTN 2.3-1

2.3.1. Requirements and Limit States

Azerbaijan code consider two different limit states [21]. The limit state that makes the operation of structures completely unusable named “first stage of limit state”. The limit state that complicates the normal operation of the structure or reduces the longevity of buildings and structures in relation to their service life is named “second stage of limit state”.

First stage limit state includes:

- Strength design;
- Durability design (thin wall structures);
- Stability design (overturning, slipping).

Second stage limit state includes:

- Crack formation design;
- Crack opening design;
- Deformation design.

All types of concrete and reinforced concrete structures should obey the following requirements:

- In terms of safety requirement;
- In terms of operational suitability;
- In terms of durability;

To meet safety requirements, structures must have such initial characteristics that with a proper degree of reliability under various design impacts in the process of construction and operation of buildings and structures, the destruction of any nature or impairment of usability, related to harm to life or health of citizens, property and environment have to be excluded.

To meet operational requirements the design must have such initial characteristics that with the appropriate degree of reliability for various design, the formation or excessive opening of cracks do not occur, as well as excessive movement, vibrations and other damage occurred hindering normal operation.

To meet the requirements of durability, the structure must have such initial characteristics that within the established time, structure would satisfy the safety requirements and serviceability, considering impacts of geometric structural characteristics and mechanical characteristics of materials.

Azerbaijan seismic code requires the installation of engineering seismic observation stations in order to obtain reliable information during earthquake in high level responsibility buildings and structures as well as buildings and structures which height exceeds 75 meters and 16 storeys.

Actions in the structures of buildings and constructions designed for construction in seismic areas, as well as in their elements, should be determined taking into account at least three shapes of natural vibrations, in case of the periods of the first (lowest) shape of natural vibrations T_1 are more than 0,4 second, and taking into account only the first shape, if T_1 is equal to or less than 0,4 second.

2.3.2. Seismic Action and Soil Parameters

This code has four types of soil, I, II, III and IV (see Table 2.10). Based on their seismic characteristics soil classified by: Standard Penetration Test (N_{SPT}); average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5}

or less ($v_{s,30}$); and bearing resistance of soil. It is not allowed to construct high rise buildings in soil type IV.

Table 2.10: Ground types according to AzDTN 2-3-1 [6]

Ground type	Description of stratigraphic profile	Parameters	
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30 sm)
I	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	–
II	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	>50
III	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 – 50
IV	Deposits of loose-to-medium cohesionless soil, or of predominantly soft-to-firm cohesive soil.	<180	<15

For other types of soils not considered in this classification the ground on the construction site the average shear wave velocity in width and number of blows in the standard penetration test should be determined by expression below:

$$V_i = \frac{30}{\sum_{i=1}^{n} \frac{h_i}{V_i}} \tag{28}$$

$$N_{SPT} = \frac{30}{\sum_{i=1}^{n} \frac{h_i}{N_{SPTi}}} \tag{29}$$

where

h_i is the thickness in meters;

V_i is the velocity of the seismic waves;

N_{SPTi} is the number of blows of SPT;

n is the number of layers of soil in 30 meters depth.

Another factor which is used in computation of base shear is soil factor, k_q, and it related for each ground type described in Table 2.10. Values of soil factor corresponded to each ground type shown in Table 2.11.

Table 2.11: Soil factor according to AzDTN 2.3-1 [6]

Ground type	Soil factor k_q
I	0,7
II	1,0
III	1,3
IV	1,6

Seismic zones

According to AzDTN 2.3-1 [6], Azerbaijan is divided into 5 seismic zones rated by earthquake intensity and probability of earthquake occurrence once every 100, 1000 or 10000 years. The map of seismic zones of Azerbaijan Republic presented in Figure 2.6.

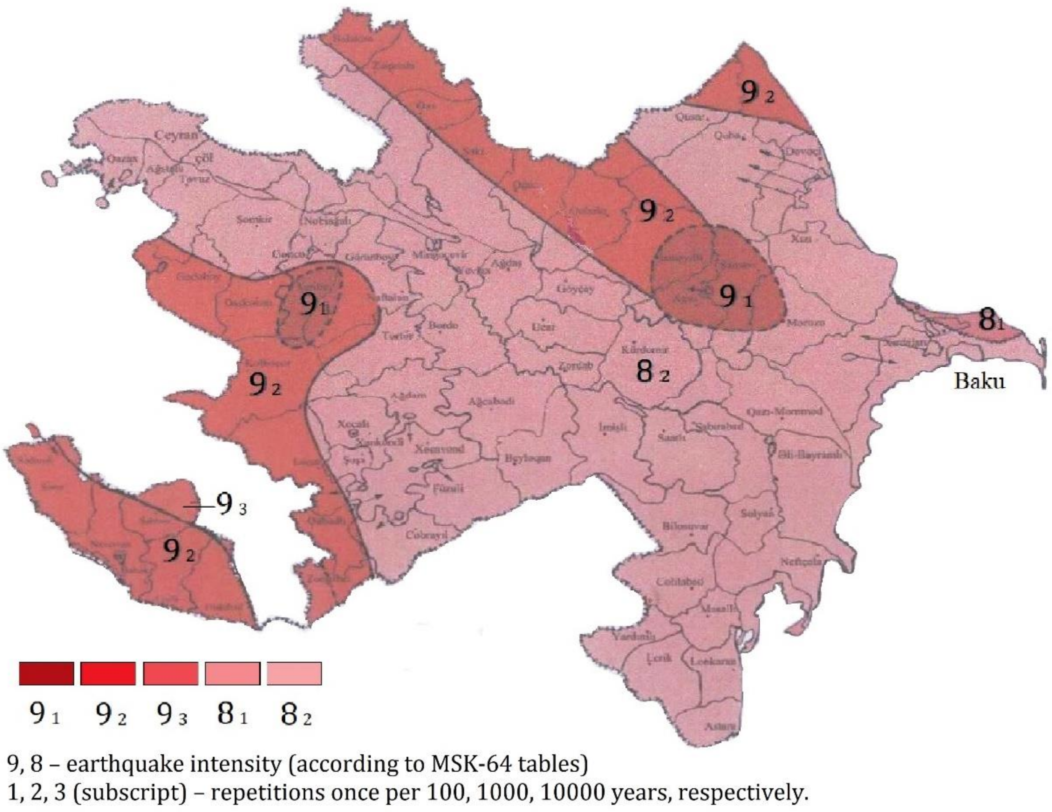


Figure 2.6: Seismic zones of Azerbaijan Republic according to AzDTN 2.3-1

Design ground acceleration A_0, should be determining according to equation (30):

$$A_0 = k_q \cdot a_0 \tag{30}$$

where

k_q is the soil factor (see Table 2.11)

a_0 is the factor of design ground acceleration, (see Table 2.12).

Table 2.12: Reference peak ground acceleration according to AzDTN 2.3-1 [6]

Peak Ground Acceleration		
Seismic intensity	m/s^2	a_0
7	0,125	0,125
8	0,25	0,25
9	0,5	0,50

Horizontal elastic response spectrum

In order to define horizontal components of the seismic action, the expressions of elastic response spectrum β_i, presented in expressions (31), (32) and (33).

$$0 \le T_i \le T_A : \beta_i = 1 + 1,5 \frac{T_i}{T_A} \tag{31}$$

$$T_A < T_i \le T_B : \beta_i = 2,5 \times k_q \tag{32}$$

$$T_B < T_i : \beta_i = 2,5 \left(\frac{T_B}{T_i} \right)^{0,5} \tag{33}$$

where

β_i is the elastic response spectrum;

T_A is the lower limit of the period of the constant spectral acceleration branch;

T_B is the upper limit of the period of the constant spectral acceleration branch;

k_q is the soil factor, presented in Table 2.11;

T_i is the vibration period of SDOF system.

Table 2.13: Values for parameters for elastic response spectra

Ground Type	$\beta(T_0)$	T_A (sec)	T_B (sec)
I	1,08	0,10	0,40
II	1,15	0,10	0,40
III	1,23	0,10	0,60
IV	1,30	0,10	0,80

The value of elastic response spectrum β_i, should not be accepted less than 1,0 for ground types I and II while for ground types III and IV should not be accepted less than 1,2 (see Table 2.13).

For the four ground types I, II, III and IV (see Table 2.10) presented in Azerbaijan seismic code the values of the elastic response factor β_i, corresponding to period (i) of structure and vibration period on a point A (T_A), B (T_B) are given in Table 2.13.

The basic spectral shape is composed by two branches presented below in Figure 2.7.

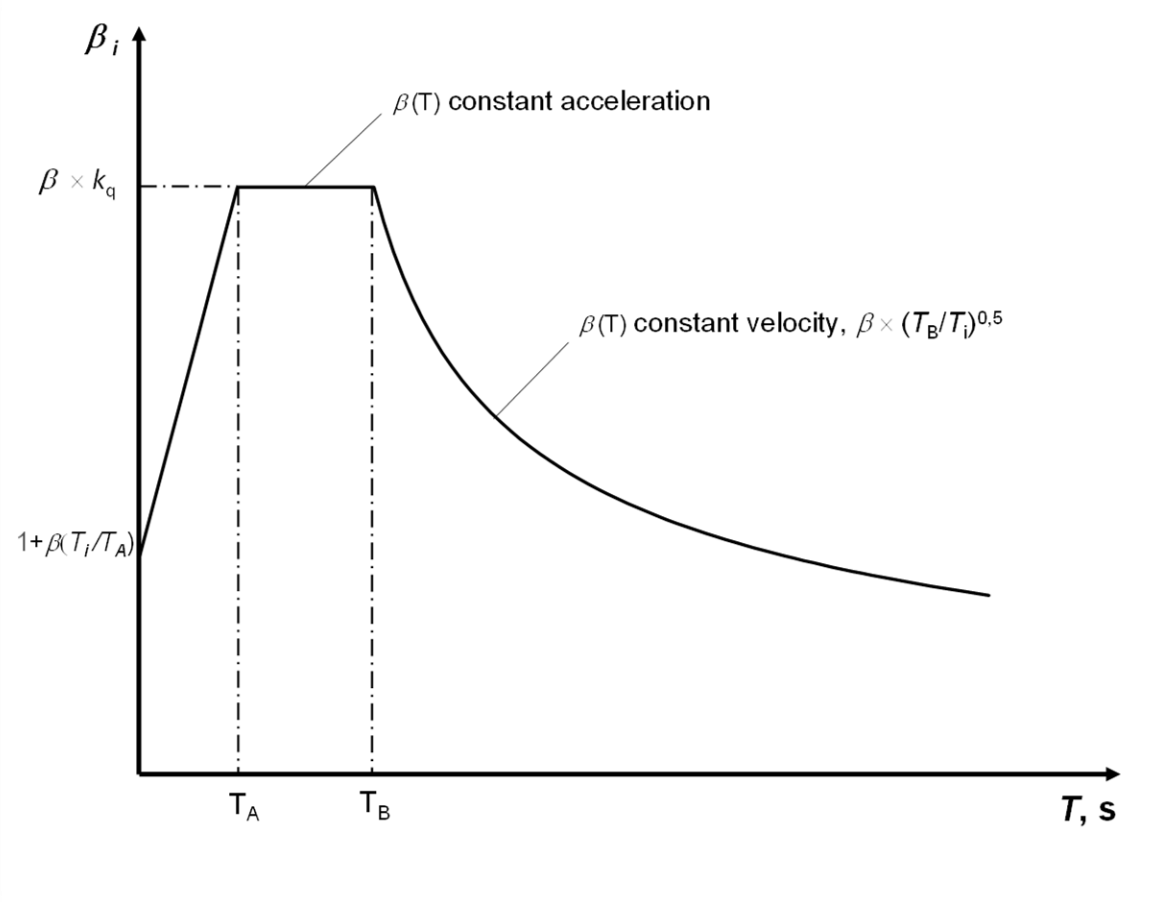


Figure 2.7: Basic shape of the elastic response spectrum of AzDTN 2.3-1 [6]

The short period range continues from T_A to T_B period and represents branch of constant spectral response acceleration.

Elastic response spectra for all ground types are presented in Figure 2.8.

The horizontal seismic action is described by two orthogonal components, assumed as independent and being represented by the same response spectrum. The horizontal elastic response spectra according to AzDTN 2.3-1 [6] is much more conservative in poor soil types (IV). The range of constant acceleration branch for soil type IV lasts from 0,1 to 0,8 s, and after turns to constant velocity brunch. All soil types reach the lower limit of the period of constant spectral acceleration branch in period of 0,1 s. The maximum value of spectral response acceleration for constant spectral acceleration branch for soil type I, II, III and IV are 1,75, 2,5, 3,25 and 4 respectively.

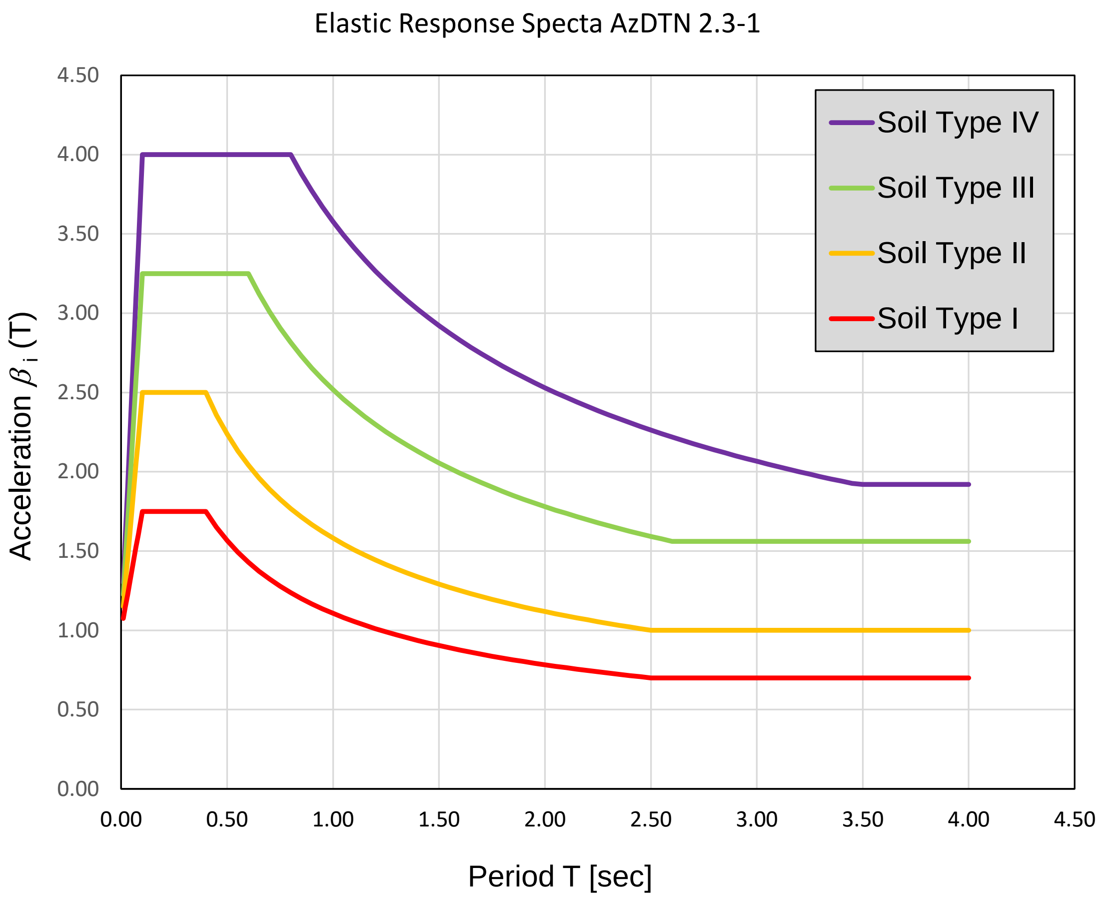
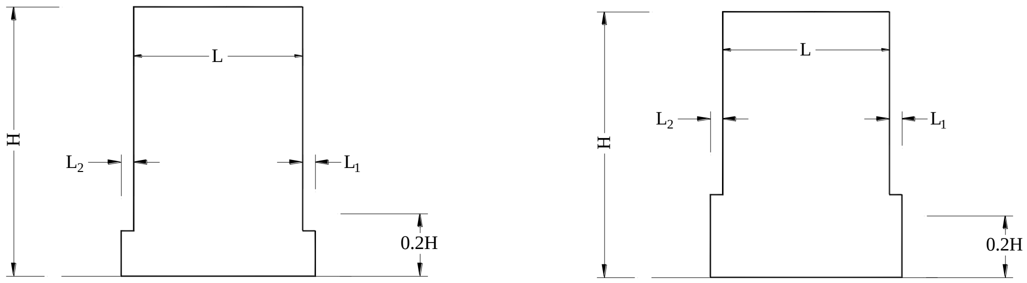


Figure 2.8: Elastic response spectra for ground types I to IV

2.3.3. Design Buildings under Seismic Action

The criterion of evenly spread setbacks in elevation of structure according to AzDTN 2.3-1 [6] presented below:



(a) Criterion: $\frac{L_2 + L_1}{L} \leq 0.50$ (b) Criterion: $\frac{L_2 + L_1}{L} \leq 0.20$

$$\frac{L_2}{L_1} = 0.4 \div 0.6$$

Figure 2.9: Criteria for regularity of buildings with setbacks AzDTN 2.3-1 [6]

If setbacks appear in lower than 20% of total height of structure, criterion (a) presented in Figure 2.9 shall be covered. If setbacks take place in higher place than 20% of total height of structure as shown in Figure 2.9 (b) the corresponding expression must be accepted.

Load combinations

The design of structures and foundations in seismic areas should be performed using special load combinations taking into account seismic effects.

Calculating buildings and structures for special load combination, design load values should be multiplied by combination coefficients taken according to the Table 2.14.

Table 2.14: Coefficients for special load combinations

Type of load	Values of coefficients of combinations
Permanent	0,9
Quasi permanent	0,8
Variable	0,5

Depending on the composition load combination is distinguished by:

Main load combination (a) includes permanent load P_d, quasi permanent load P_l, variable load P_t which determined by expression (34).

$$C_m = P_\mathrm{d} + \sum \psi_\mathrm{li} P_\mathrm{li} "+" \sum \psi_\mathrm{ti} P_\mathrm{ti} \tag{34}$$

where

C_m is the main load combination;

P_d is the permanent load;

P_l is the quasi permanent load;

P_t is the variable load;

ψ_{li} is the coefficient for quasi permanent combination (i=1, 2, 3, …);

ψ_{ti} is the coefficient for variable combination (i=1, 2, 3, …).

Special load combination (b) includes permanent load P_d, quasi permanent load P_l, quasi permanent load P_t and one of the special load P_s, and should be obtained by expression (35).

$$C_s = C_m + P_s \tag{35}$$

where

C_s is the special load combination;

P_s is the special loads.

Special loads P_s, include:

– Seismic loads;

– Explosion loads;

– Loads caused by severe technological process interruption, temporary malfunction or break of equipment;

– Loads caused by fire;

– Loads caused by car accident with members of structure.

Coefficients for main loads combination (a), presented in Table 2.15.

Table 2.15: Coefficients for main load combination

Characteristic of load	Designation	Value
First important variable load	ψ_{t1}	1,00
Second important variable load	ψ_{t2}	0,90
Third and subsequent important variable loads	$\psi_{t3...}$	0,70

Uniformly distributed quasi permanent load	ψ_l	0,95
Remain quasi permanent loads	ψ_l	1,00

Coefficients for special loads combination (b), presented in Table 2.16.

Table 2.16: Coefficients for specific load combination

Charactestic of load	Designation	Value
Permanent	ψ_d	0,9
Quasi permanent loads	ψ_l	0,8
Variable loads	ψ_t	0,5
Seismic load	ψ_s	0,8

Importance classes

According to AzDTN 2.3-1 [6] seven importance classes of buildings are distinguished. Each of importance type of buildings correspond to importance factor k_1, shown in Table 2.17.

Table 2.17: Importance factor according to AzDTN 2.3-1 [6]

Type of building or structure	k_1 factor
Especially critical facilities whose failure associated with severe consequences for the environment and population	2,0
Especially important buildings	1,5
Crowded buildings with 300 people or more in same time, stadiums, theatres, museums, shopping malls, undergrounds, train stations and etc.	1,4
Buildings and facilities, whose operation is necessary for emergency response during earthquakes such as electrical stations, water stations, fire station, communications facilities, ambulances and etc.	1,2
Schools, kindergartens, hospitals, nursing homes, dormitories, soldier's barracks.	1,2
Residential, public and civil buildings which is not mentioned above	1,0
Single storey agricultural and storage facilities, temporary one-storey buildings, whose destruction is not accompanied by loss of lifes.	0,5

Design seismic load

The design seismic load S_{ik}, in selected direction should be determined by expression (36).

$$S_{ik} = k_1 \cdot k_2 \cdot k_3 \cdot S_{oik} \tag{36}$$

where

k_1 is the importance factor, shown in Table 2.17;

k_2 is the behaviour factor, shown in Table 2.18;

k_3 is the factor determined by expression below:

$$1,0 \leq k_3 \leq 1,25 : k_3 = 1 + 0,02(n-5) \tag{37}$$

where:

n is the number of storeys of structure

S_{oik} is the horizontal seismic load which should determine by expression below:

$$S_{oik} = k_\psi \cdot Q_k \cdot A_0 \cdot \beta_i \cdot \eta_{ik} \tag{38}$$

where:

k_ψ is the structural type values of which presented in Table 2.19;

Q_k is the weight of storey corresponding to k point, design seismic loads should be in accordance with Table 2.14;

β_i is the value of acceleration (m/s^2) corresponding to structure frequency;

η_{ik} is the coefficient depending on the shape of the deformation of the building or structure with its own fluctuations in the i-th shape and on the location of the load;

A_0 is the design seismic factor which should be determined by expression below:

where

The coefficient η_{ik} is determined by the following expression.

$$\eta_{ik} = \frac{X_i(x_k)\sum_{j=1}^{n} Q_j X_i(x_j)}{\sum_{j=1}^{n} Q_j X_i^2(x_j)} \tag{39}$$

where

$X_i(x_k), X_i(x_j)$ are the displacements of a building or structure with its own vibrations in the i-th shape at the considered point k and at all points j see Figure 2.10, where in accordance with the calculation scheme its weight is assumed concentrated;

Q_j is the weight of structure referred to point j, calculated in accordance with Table 2.14.

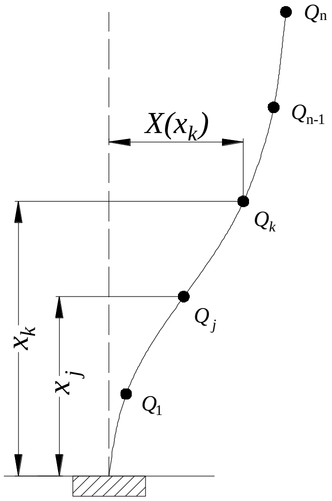


Figure 2.10: Displacement of the structure under its own vibration

In case of structures with uniformity in elevation up to 5 storeys high inclusive, insignificantly changes in mass in elevation and rigidness of joints in case period T is equal to or less 0,4 s, the following expression can be used to calculate η_k.

$$\eta_k = \frac{x_k \sum_{j=1}^{n} Q_j (x_j)}{\sum_{j=1}^{n} Q_j x_j^2} \tag{40}$$

where

x_k, x_j are the distance from top of the foundations to k and j point

2.3.4. Particular Factors and Rules

According to AzDTN 2.3-1 [6] there is behaviour factor k_2, which included to the computation of seismic load S_{ik}, corresponding to different types of structure. Table 2.18 illustrate structural system and values of behaviour factor k_2, corresponded to it.

Table 2.18: Behaviour factor according AzDTN 2.3-1 [6]

Types of structural system	***k*₂ factor**	***q****
1. Structures where inelastic deformation and damage is not allowed	1.00	1.00
2. Buildings and structures in the construction of which residual deformations may be allowed and damage that impede normal operation, while ensuring safety of people and safety of equipment erected by:		

– Steel frame	0.25	4.00
– Concrete frame without vertical diaphragms or connections	0.35	2.86
– Concrete frame with vertical diaphragms or connections	0.30	3.33
–	0.25	4.00
– Reinforced concrete panels and monolithic reinforced concrete walls in large dimensions	0.40	2.50
– Brick or masonry	0.45	2.22
– Pillars of the seismic systems	0.60	1.67
– Regardless of the design, all buildings till 5 storeys	0.25	4.00
3. Buildings and structures in the construction of which may be allowed significant residuals deformation, cracks, damage of individual elements, temporarily stopping normal operation while ensuring safety people	0.15	6.67
*– equivalent to behavior factor in EN 1998-1 [6]		

Structural types

Factor k_{ψ}, included in expression (38), is a factor that takes into account the energy dissipation capacity of buildings, corresponding to each type of structure presented below in Table 2.19. This factor is essential in calculating horizontal seismic load S_{oik}, see 2.3.3.

Table 2.19: Description of structural types and factors

Characterization of structure and buildings	k_{ψ}
1.Buildings which has small dimensions in plan such as towers, chimneys and freestanding elevator shafts	1,3
2.Buildings with height to width ratio greater than 4, bridges longer than 50 meters and buildings with spans more than 24 meters	1,2
3.Buildings with frame systems in which wall fillings does not affect building's deformability and the ratio between design seismic load in the direction of the columns height (h) to it width (b) equal or more than 25	1,3
4.Buildings with the ratio between design seismic load in the direction of the column's height (h) to it width (b) equal or less than 15	1,0
5.Buildings which does not mentioned above	1,0
Note: In case h/b ratio between 15 to 25, factor k_{ψ} must be determined by interpolation.	

2.4.Comparison Between Eurocode 8 and AzDTN 2.3-1

All aspects of seismic design of both seismic codes are quite similar. In most aspects AzDTN 2.3-1 is more conservative rather than Eurocode 8.

The description of limit states of both codes is similar as well. The Ultimate limit states (ULS) and Serviceability limit states (SLS) of EN 1998-1 [3] and First Stage Limit State (FSLS) and Second Stage Limit State (SSLS) of AzDTN 2.3-1 [6] are presented in Figure 2.11.

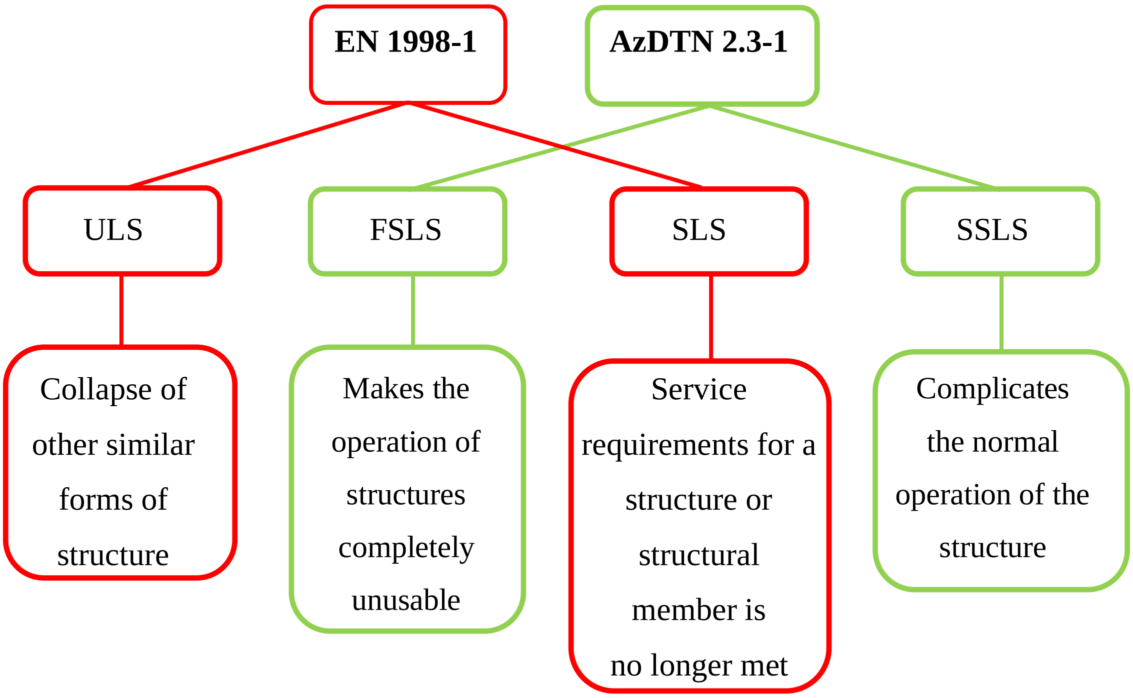


Figure 2.11: Limit states of EN 1998-1 & AzDTN 2.3-1

The seismic design according to AzDTN 2.3-1 [6] considers structural types of building k_ψ (see Table 2.19 in section 2.3.3), while EN 1998-1 [3] does not include factors for each structural type, however describes several types of structure (see Table 2.8 in section 2.2.4).

Both codes present ground classifications based on soil characteristic $v_{s,30}$ and N_{SPT} (see Table 2.1 in section 2.2.2, Table 2.10 in section 2.3.2).

The importance classes for buildings for both codes are quite different. The Azerbaijan seismic code includes seven importance classes for building (see Table 2.17 in section 2.3.3) while European code only four (see Table 2.2 in section 2.2.2).

The first important class described in Eurocode 8 [3], is similar to class seven described in AzDTN 2.3-1 [6] corresponding to buildings with one storey and minor importance to public safety such as agricultural buildings. The importance factor for the same class is 0,5 in case Azerbaijan code and 0,8 for European code for classes described above. The reference importance class with importance factor 1,0, for both codes are described residential and ordinary buildings which not mentioned in other classes. The importance class II fits the description of sixth importance class described in Azerbaijan seismic code. According to description of third importance class described in Eurocode 8 [3] the third, fourth and fifth classes described in AzDTN 2.3-1 [6] matched by descriptions. The first two importance classes in AzDTN 2.3-1 [6] included in IV importance factor presented in EC8 [3]. Table 2.20 matches two importance classes and factors for more explicit representation.

Table 2.20: Comparison of importance classes and factors according to EN 1998-1 [3] and AzDTN 2.3-1 [6]

<table>
<tr><th colspan="2">EN 1998-1</th><th colspan="2">AzDTN 2.3-1</th></tr>
<tr><th>Description of Importance class</th><th>Importance factor γ</th><th>Importance factor k1</th><th>Description of Importance class</th></tr>
<tr><td>I. Buildings of minor importance for public safety.</td><td>0,8</td><td>0,5</td><td>7. Low-priority buildings with priority for human safety.</td></tr>
<tr><td>II. Reference importance class, not mentioned in other classes.</td><td>1,0</td><td>1,0</td><td>6. Reference importance class, not mentioned in the other classes.</td></tr>
<tr><td rowspan="2">III. Buildings whose importance in view of the consequences associated with a collapse.</td><td rowspan="2">1,2</td><td>1,2</td><td>5. Hospitals with 100 and more beds, dormitory with 250 and more places, educational institution and etc.</td></tr>
<tr><td>1,2</td><td>4. Oil tanks, energy and water supply, sewage pipelines, fire, security, systems.</td></tr>
<tr><td rowspan="3">IV. Structures with vital importance.</td><td rowspan="3">1,4</td><td>1,4</td><td>3. Crowded buildings with 300 and more people in same time, such as stadiums, theatres, railway stations, shopping malls, metros.</td></tr>
<tr><td>1,5</td><td>2. A number of state-important administrative buildings.</td></tr>
<tr><td>2,0</td><td>1. Damage to the environment and the possibility of creating severe consequences for public safety and structures that can produce results.</td></tr>
</table>

The comparison of factors of importance classes is presented in Figure 2.12.

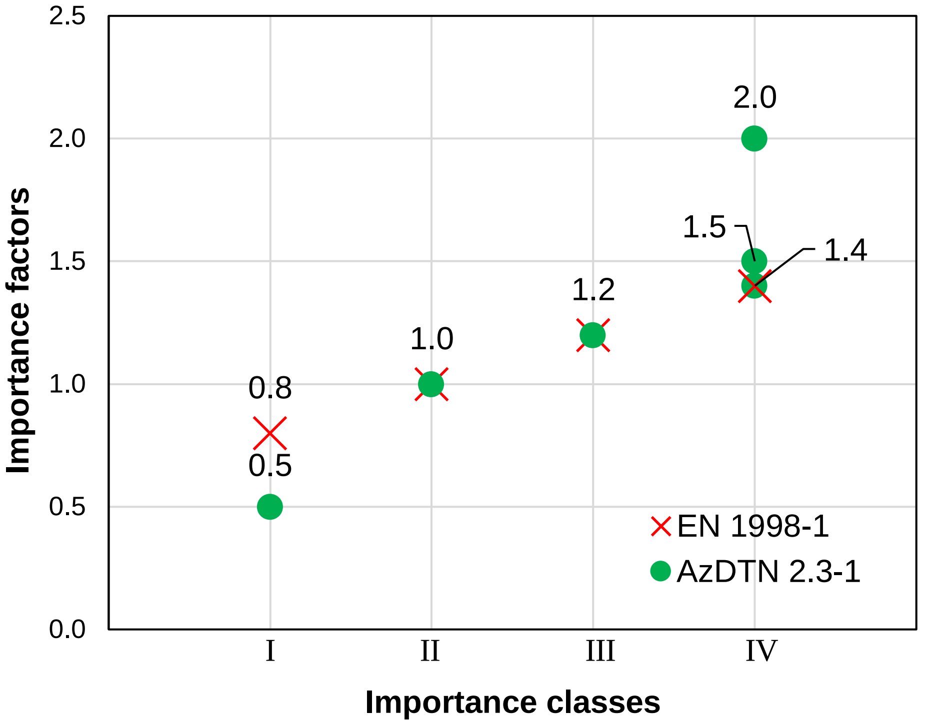


Figure 2.12: Importance classes and factors of EN1998-1 & AzDTN 2.3-1

The Azerbaijan code has differences in factors in first and fourth importance classes with Eurocode one. More wide range shows AzDTN 2.3-1 [6] in importance class IV.

To compare horizontal elastic response spectrum, four type of soil will be taken into account. As far as characteristic of soils I, II, III, IV for each code is same, those soil will be compared.

As shown in Figure 2.7 in section 2.3.2, response spectrum shape according to Azerbaijan code present two branches, while European seismic code (see Figure 2.3 in section 2.2.2) present three branches.

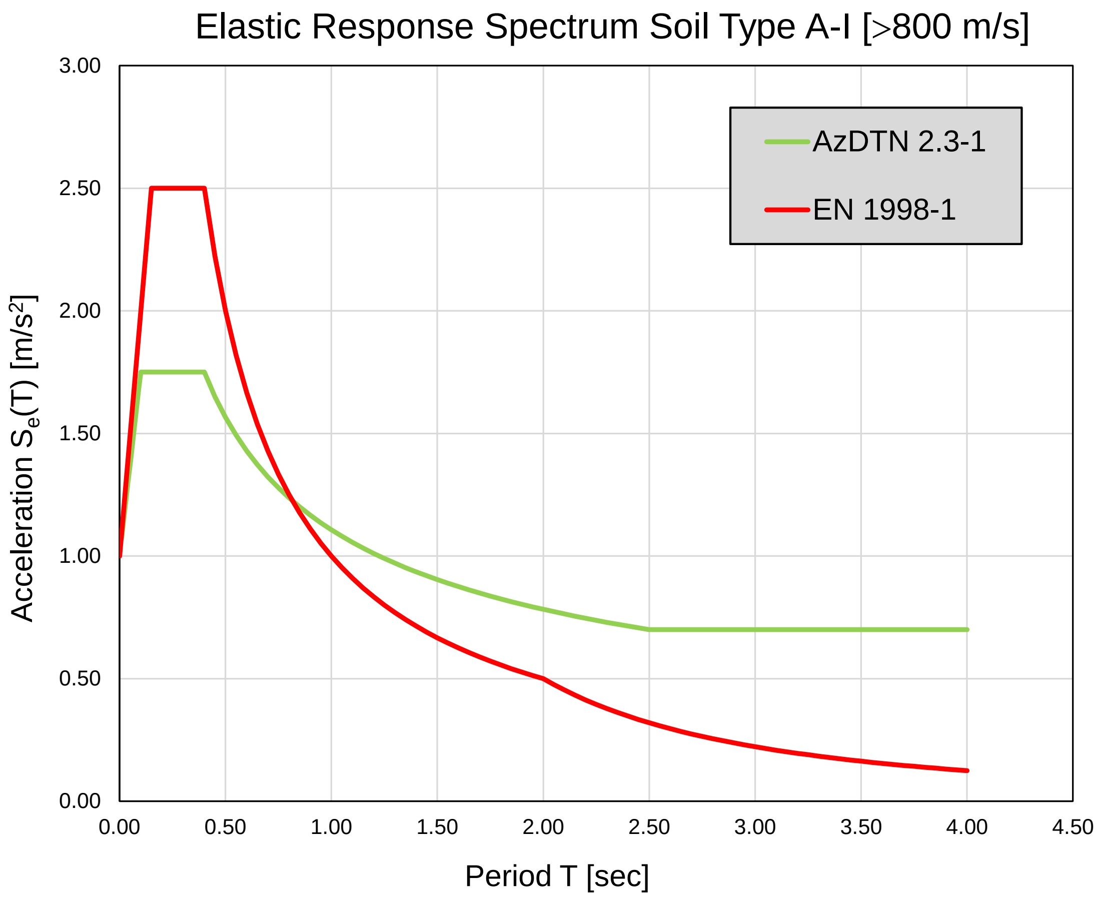


Figure 2.13: Elastic response spectrums for soil type A – I, with average shear wave velocity $v_s > 800$m/s

For reference soil type such as rock the structures with higher period of vibration AzDTN 2.3-1 [6], is more conservative than EN 1998-1 [3]. For structures with natural frequency close to 2 Hz ground accelerations for AzDTN 2.3-1 [6] and EN 1998-1 [3] are 1.57 m/s^2 and 2.00 m/s^2, respectively, which represent 78 % difference. For structure's vibration period of 2.5 s the value of ground accelerations for AzDTN 2.3-1 [6] and EN 1998-1 [3] are 0.70 m/s^2 and 0.32 m/s^2, respectively, which represent 46 % difference.

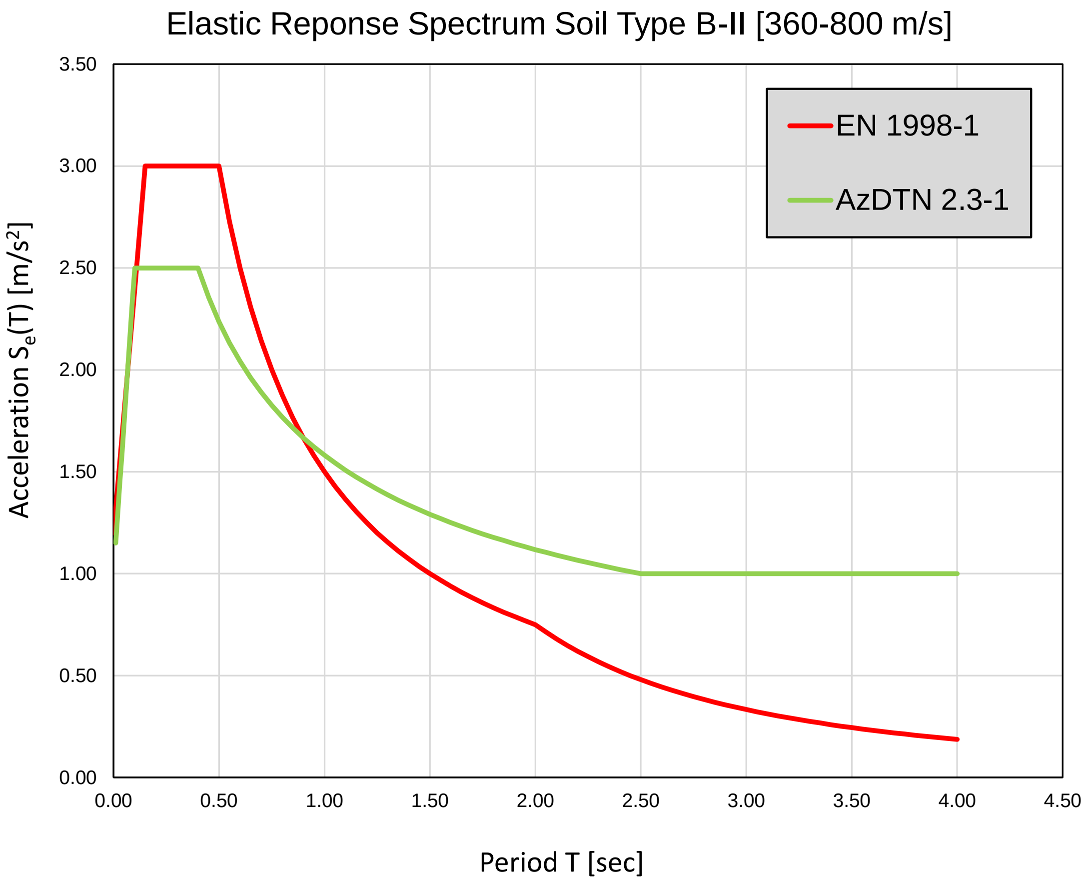


Figure 2.14: Elastic response spectrums for soil type B – II, with average shear wave velocity v_s 360 – 800m/s

Elastic response spectrums for ground type B according to Eurocode 8 [3] and ground type II according to Azerbaijan seismic code [6] represent similar shape as for rock and rock type ground. The vibration period of structure more than 1.0 s according Azerbaijan code is much more conservative than Eurocode. The ground accelerations for 0.5 s vibration period for AzDTN 2.3-1 [6] and EN 1998-1 [3] are 2.24 m/s^2 and 3.0 m/s^2, respectively, which represent 74.5 % difference. For higher periods such as 2.5 s values of ground accelerations are 1.0 m/s^2 for Azerbaijan seismic code and 0.48 m/s^2 for Eurocode 8, which represent 48 % difference.

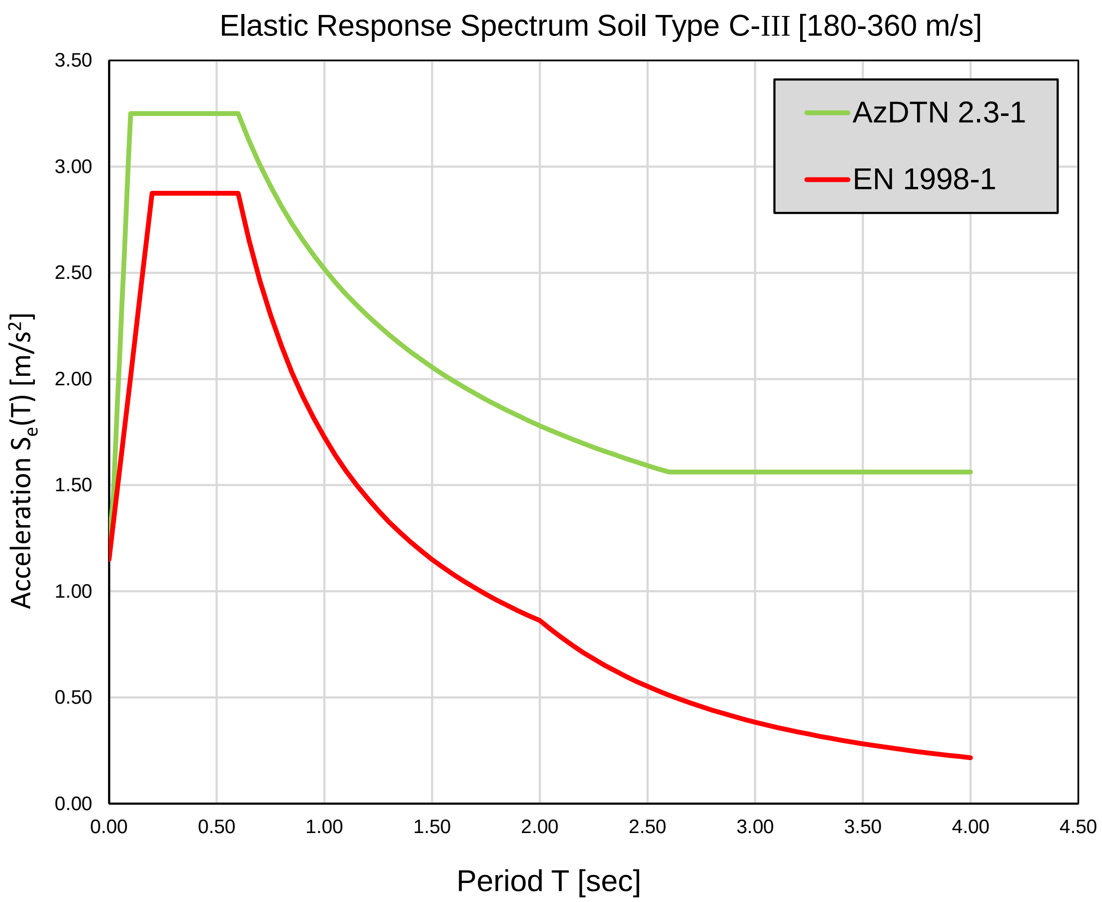


Figure 2.15: Elastic response spectrums for soil type C – III, with average shear wave velocity v_s 180 – 360m/s

In case of ground type C (see Table 2.1 in section 2.2.2) and ground type III (see Table 2.10 in section 2.3.2), for structures with any vibration period AzDTN 2.3-1 [6] present higher ground acceleration than EN 1998-1 [3]. For period of 0.5 s 88 % deference in ground acceleration mentioned. For higher period such as 2.5 s Azerbaijan code present 1.59 m/s^2 while Eurocode is only 0.55 m/s^2, which makes AzDTN 2.3-1 [6] extremely conservative in comparison to Eurocode 8 [3].

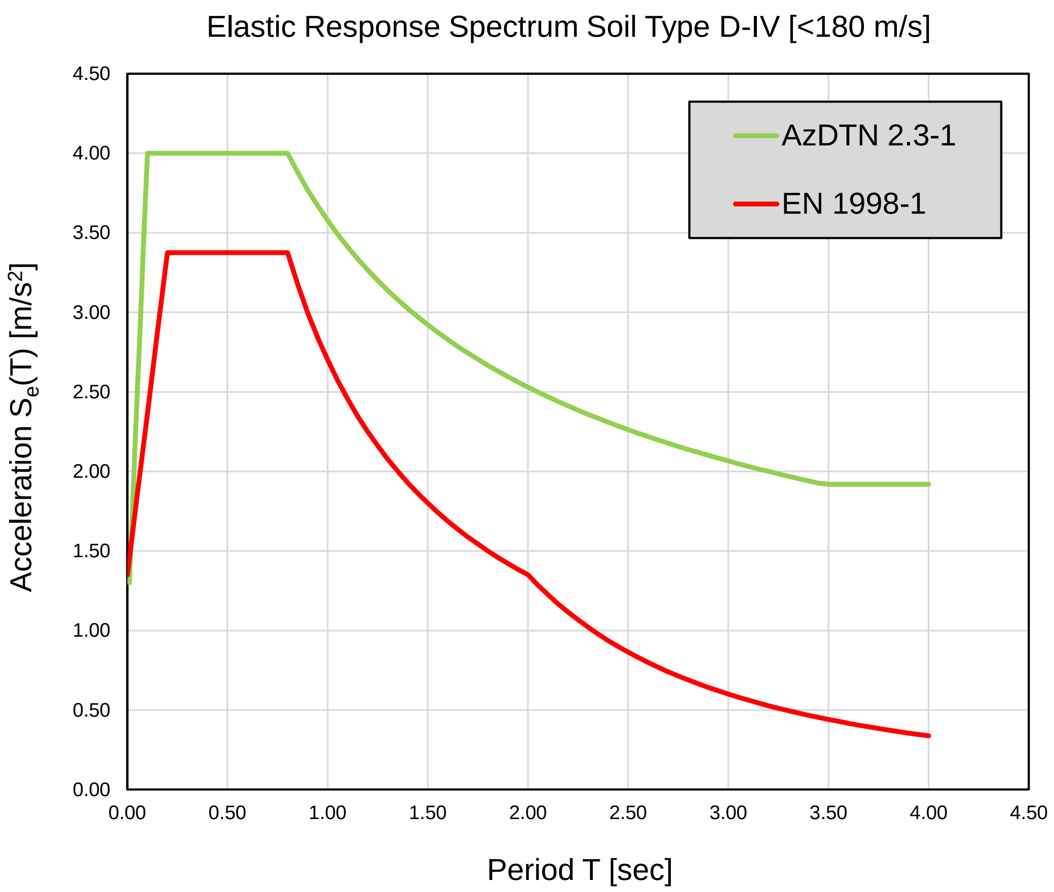


Figure 2.16: Elastic response spectrums for soil type D – IV, with average shear wave velocity v_s < 180 m/s

For the weak ground types such as type D according EN 1998-1 [3] and type IV according AzDTN 2.3-1 [6], values for ground acceleration in elastic response spectrum are similar in terms of percentage deference, to shape of response spectrum for previous soil type. The percentage difference in vibration period 0.5 s represent 84 %, while for 2.5 s vibration period 2.26 m/s^2 and 0.86 m/s^2 for AzDTN 2.3-1 [6] and EN 1998-1 [3], respectively.

Behaviour factor k_2, presented in AzDTN 2.3-1 [6] used in calculation of base shear multiplied with other factor described (see chapter 2.3.4), while in EN 1998-1 [3] in order to obtain base shear behaviour factor q, is subject to division.

2.5. Final remarks

Both seismic codes represent similar approaches in most areas, except in calculation of base shear. The prescription of EN 1998-1 [3] allows to compute base shear in structure directly, see expression (21), while prescriptions of AzDTN 2.3-1 [6] calculate seismic load applied to each storey with further determination of base shear.

Seismic hazard maps for use with Eurocode 8 are meant to be provided by individual nations.

Values, factors, coefficients are presented in paragraph 2.3 regarding AzDTN 2.3-1 take into account 1st and 2nd reissue of Azerbaijan Codes.

3. Case Study

In this section the author presents the case study, a RC multistorey building with eleven storeys used as case study to compare the two codes. All the aspects concerning the concept and the modelling of the structure and the modelling of the action take into account the prescriptions of the Eurocodes [1-5] and regarding seismic action also the prescriptions of the Azerbaijan Code [6].

3.1. Introduction

An existing building was selected for seismic evaluation case study. The construction of the building was done in January 2020 (see Figure 3.1). It is a residential building, with eleven storeys, consisting of 114 apartments, located in the capital of the Azerbaijan Republic, Baku, on “Absheron” Peninsula, 13 kilometres from the coastline, “Hokmali” district.

Figure 3.1: Building studied

The building includes one underground storey of basement and ten elevated storeys and roof. The site ground level is 81.05 meters above sea level. Occupancy of areas are shown in Table 3.1.

Table 3.1: Occupancy of areas in m^2

Total building area	10490.00
Residential area	6007.10
Non-residential area	884.04
Construction site	1454.88

3.2. Geotechnical Investigation

Due to the lack of sufficient data concerning the ground and the ground-water conditions at construction site, based on ground foundations around building (see Figure 3.1), medium dense sand was considered for soil. According to EN 1997-1 [4], this soil has a bearing resistance 2.0 kgf/cm^2 (200 kN/m^2), 18 kN/m^3 of specific weight and the angle of shearing resistance 32°. The soil on site classified according to Table 2.1: Ground types according Eurocode 8 in section Seismic Action and Soil Parameters 2.2.2, and presented below in Table 3.2.

Table 3.2: Ground type

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$	N_{SPT}	c_u (kPa)
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 – 50	70 – 250

where

N_{SPT} is the number of blows for Standard Penetration Test;

c_u is the undrained shear strength of soil;

$v_{s,30}$ is the average value of propagation velocity.

3.3. Structural System

The frame of the structural system of the building is a dual system, composed by frames and walls, with solid slabs. The structure presents simplicity and regularity in plan and elevation. The structural elements are regularly distributed. The building's structure is symmetrical in plan with respect to one orthogonal axis. The frame of structure includes columns, beams, slabs and shear walls. Geometrical data of entire structure is given in Table 3.3.

Table 3.3: Geometrical data of structure in meters

Total height of structure	42.70
Height of Basement storey	3.89
Height of Apartments storeys	3.14
Total width of building	19.00
Total length of building	52.80

Architectural schemes of facades are presented in Appendix A.

Structural plans have been done by the author based on architectural sketches presented by Azerbaijan Architecture and Construction University, Department of Reinforced Concrete Structures. Structural elements are defined according to Eurocode 2 [5]. Due to whole structure, columns present regularity in plans and throughout the height of building have tended to section reducing on 0, 4, 6 and 8 storeys by 0.1 m of deep at certain columns. Section geometry of reinforced concrete columns are presented in Table 3.4.

The structure has two types of reinforced concrete beams, with next parameters shown hereafter in Table 3.5. The project includes two-way and one-way solid slabs as show in Table 3.6. The structure includes several floor openings for stairwell and elevator shaft. The structure includes few types of shear walls with different thicknesses. Beginning from the basement until 3^{rd} storey inclusive 0.30 m wall applied, after and until the ceiling of the 10^{th} storey inclusive 0.20 m shear wall applied. Openings in shear wall with 1-meter width and 2-meter height take place in the basement storey.

The list of shear walls is shown in Table 3.7. Structure includes two reinforced concrete elevator shafts symmetrically presented on plan, with 0.20 m width, due to whole height of structure.

For the whole structure either column or shear walls applied one uniform reinforced concrete foundation slab with 0.80 m depth on a compacted soil. Retaining walls performed in basement storey with 0.25-meter depth, without openings for vehicle approaching, are presented to transfer loads to foundation slab.

The structural plan is presented in Figure 3.2.

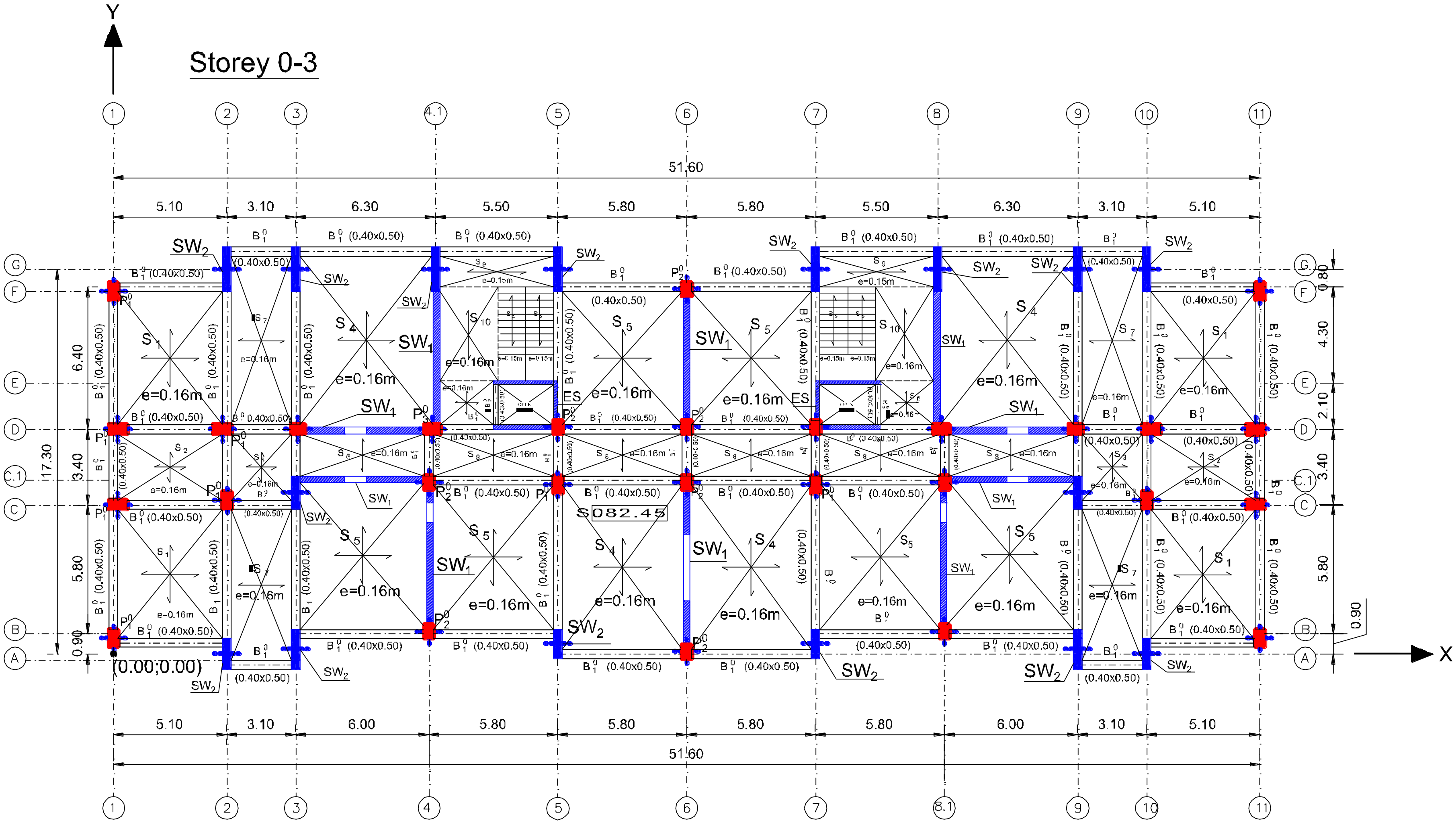


Figure 3.2: Structural plan of storeys 0 to 3

The building frame includes six different types of columns, see Table 3.4, according to their cross-sectional geometry. The spans between columns in X direction is about 6 meters, while in Y direction about 6 meters and also 3.4 meters in middle span.

Table 3.4: Reinforced concrete columns

Designation	Section properties	
	b [m]	h [m]
P_1	0.50	0.80
P_2	0.40	0.80
P_3	0.40	0.70
P_4	0.40	0.60
P_5	0.40	0.50
P_6	0.40	0.40

Structure has two types of beams. Cross-sectional geometry of beams presented in Table 3.5. Most beams' span is roughly about 6 meters long. Beams with 1.8 meters long included in elevator shaft, see Figure 3.2.

Table 3.5: Reinforced concrete beams

Sign	Section properties	
	b [m]	h [m]
B_1	0.40	0,50
B_2	0.20	0,40

The reinforced concrete two-way solid slab applied to foundation with 0.80 m depth. Slabs applied from 0 to 10th storey have 0.16 m width. Stairwell solid slab applied with 0.15 m thickness. Table 3.6 present slabs applied to structure.

The shear walls applied symmetrically with respect to Y axis. As far as shear walls placed asymmetrical with respect to X axis, centre of stiffness displaced from the center of mass. The asymmetrical behaviour of structure with respect to X axis is visible in shape modes, see section 3.8. The shear wall begins with 0.3 m width, and decrease in geometry in 4th storey in 0.1 m. The reason of displacement of center of stiffness is elevation shaft located in one-half of building, see Figure 3.2: Structural plan of storeys 0 to 3. Table 3.7 shows shear wall applied to structure.

Table 3.6: Reinforced concrete slabs

Structural element	Structural model	Type	Designation on the plan	Thickness [m]
Floor Slab	Two-way	Reinforced Concrete	S1	0,80
			S2	0,16
			S3	0,16
			S4	0,16
			S5	0,16
			S6	0,16
	One-way		S7	0,16
			S8	0,16
			S9	0,16
			S10	0,16
Stairwell Slab	One-way		SS	0,15
Roof Slab	Two-way		SR	0,16

Table 3.7: Shear Walls

Structural Element	Designation on the plan	Thickness of wall [m]
Shear Wall	SW1	0,30
Elevation Shaft	ES	0,20
Shear Wall	SW2	0,40
Shear Wall	SW3	0,20

The retaining structure take place in basement storey, with 3.5 m height and 0.25 m thickness. The retaining structure round building from all sides.

Table 3.8: Retaining structure

Structural Element	Designation on the plan	Thickness of wall
Retaining Structure	RS	0,25 m

3.4. Materials

As far as that project built in Baku, with respect to Azerbaijan code, materials used in project vary slightly with Eurocodes ones. Due to that issue based on properties of actual materials, the following materials were used for modelling.

❖ **CONCRETE**

– Concrete C25/30 applied for columns, beams, slabs including foundation slab, retaining wall and etc., in accordance with EN 206-1 [7].

– Concrete C16/20 used for floor screeding, in accordance with EN 206-1 [7].

Based on environmental and ground conditions in accordance with EN 206-1 [7] XC1 exposure classes obtained for whole structural members, which correspond to concrete inside buildings with low air humidity or concrete permanently submerged in water according to EN 1992-1 [5].Cover of structural elements presented in Table 3.9.

Table 3.9: Cover applied for elements in mm

Foundation	Other Structural Members
50	30

❖ **STEEL**

Steel for concrete reinforcing, in accordance with EN 10080 [10].

– Steel A500 NR SD with ordinary reinforcement B class.

3.5. Loads

3.5.1. Self-weight

Specific weight of building materials taken according to EN 1991-1-1:2002 [2]. Self-weigh of structural elements calculated based on their geometrical properties and specific weight, which described below in Table 3.10.

Table 3.10: Specific weight of materials

Material	Specific Weight [kN/m^3]
Concrete	24.0
Reinforced concrete	25.0
Earth	18.0
Water	10.0
Steel	77.0
Screed	12.0

3.5.2. Permanent Loads

Dead loads such as, interior wall, exterior wall and floor screed were applied. Dead load of walls computed according to their materials, height of particular wall, width and specific weight and consider as knife load and applied to their specific place on plane and direction.

According to architectural drawings, dead load of floor screed was determined. Dead load applied in Table 3.11.

Table 3.11: Permanent loads

Storey	Description	Applied Load
0	Floor screed	1.20 [kN/m^2]
1 to 9	Interior hollow brick wall [0.1]m	2.80 [kN/m]
	Interior hollow brick wall [0.2]m	5.60 [kN/m]
	Floor screed	1.20 [kN/m^2]
	Exterior hollow brick wall [0.3]m	5.10 [kN/m]
10	Floor screed	1.20 [kN/m^2]
	Shell rock wall [0.4]m	13.27 [kN/m]

3.5.3. Variable Loads

In order to determine the variable loads, the use of the structure, particular qualities and geometry taken into account. The amount of load applied was taken based on EN 1991-1 [2] and shown in Table 3.12.

Table 3.12: Imposed loads

Category	Structural Element	Imposed Load [kN/m^2]
A	Floors	2.00
	Stairs	4.00
	Balconies	4.00
H	Roof	0.40

3.5.4. Earth Load

The effects of the earth were calculated according to Rankine theory. Main properties and geometry of the basement are shown in Figure 3.3.

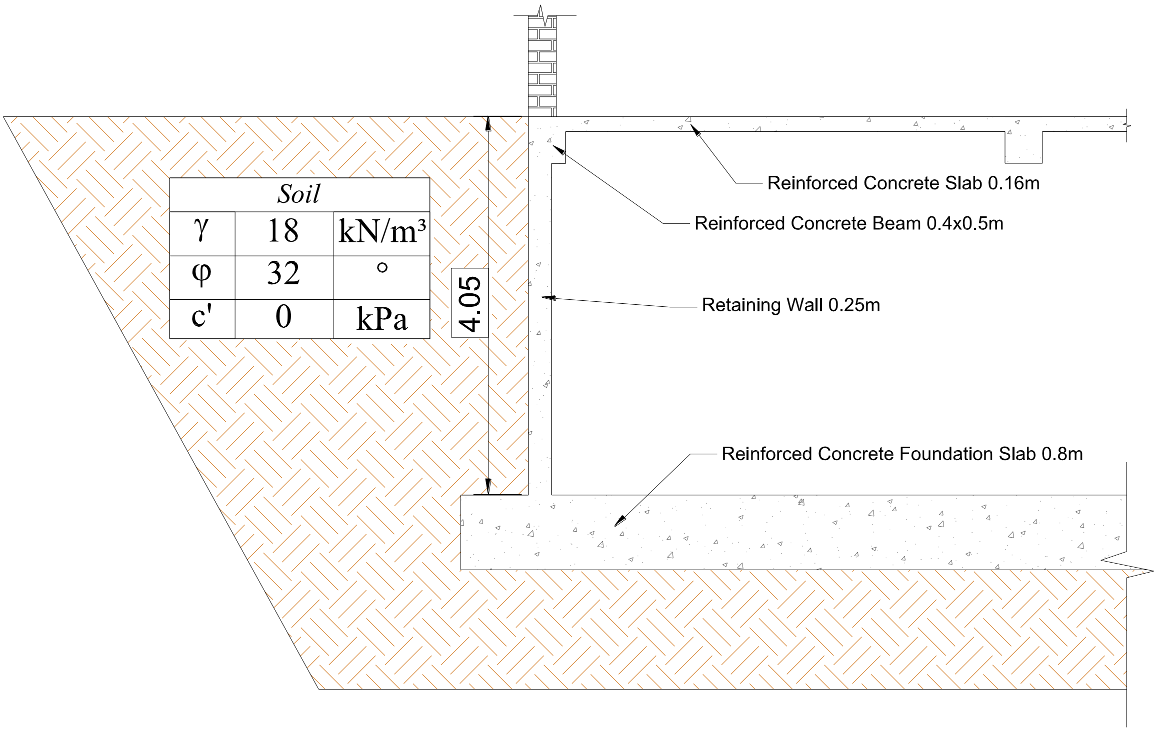


Figure 3.3: Scheme of retaining structure

where

γ weight density;

φ' angle of shearing resistance;

c' cohesion intercept.

1. Main expression

Imposed earth pressure to the wall computed according to EN 1997-1 [4], and presented in expression (41).

$$\sigma_a(z) = K_a\left[\gamma \cdot z + q\right] - 2c\sqrt{K_a} \tag{41}$$

where

$\sigma_a(z)$ is the normal stress to the wall at depth z (active limit state);

K_a is the coefficient of horizontal active earth pressure;

γ is the weight density of retained soil;

z is the distance down the face of the wall;

q is the vertical surface loads;

c is the ground cohesion.

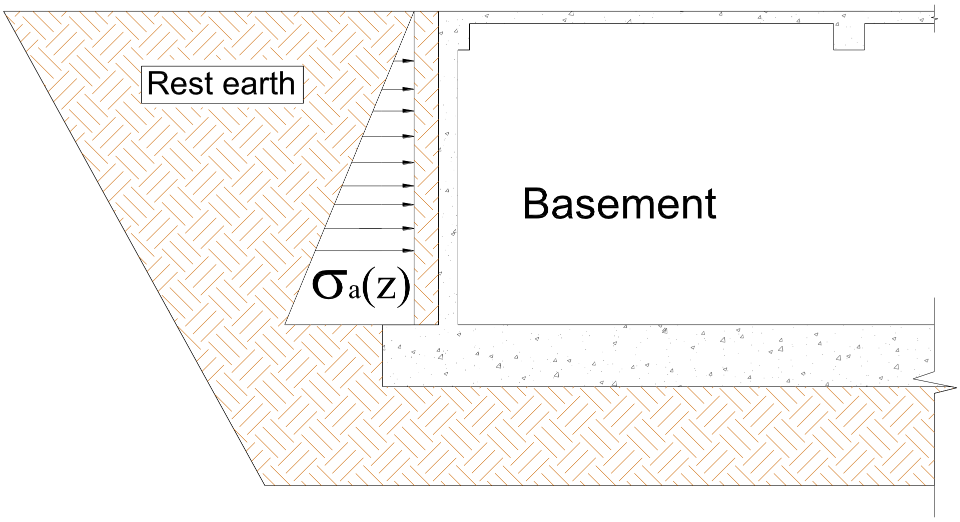


Figure 3.4: Diagram of Stress Imposed to Wall

2. At rest earth pressure coefficient (K_0)

In order to determine at rest earth pressure coefficient K_0, for "Combination 1" and "Combination 2", following expressions used:

$$K_0 = (1 - \sin\varphi') \times \sqrt{\mathrm{OCR}} \quad (*) \tag{42}$$

$$K_0^2 = (1 - \sin\varphi'_d) \times \sqrt{\mathrm{OCR}} \quad (**) \tag{43}$$

where:

OCR over-consolidation ratio which is equal to 1 for that specific case;

$\varphi_d^{'}$ design value of $\varphi^{'}$.

Taken into account partial factors for combinations according to EN 1997-1 [4].

3. Design Approaches and Combinations

In order to compute earth pressure to retaining wall Design Approach 1 taken into account according EN 1997-1 [4]. Combinations described hereafter taken into account:

*Combination 1: $A1$ "+" $M1$ "+" $R1$ [STR];

**Combination 2: $A2$ "+" $M2$ "+" $R1$ [GEO].

where "+" implies: "to be combined with".

Values of partial factors γ, presented in Appendix D.

4. Angle of shearing resistance

Expressions presented below used in order to determine angle of shearing resistance φ' and design value of angle of shearing resistance φ'_{d}.

$$\varphi' = \arctan\left[\frac{\tan\varphi'_k}{\gamma_{\varphi'}}\right] = \arctan\left[\frac{\tan 32}{1}\right] = 32° \tag{44}$$

$$\varphi'_{\mathrm{d}} = \arctan\left[\frac{\tan\varphi'_k}{\gamma_{\varphi'}}\right] = \arctan\left[\frac{\tan 32}{1,25}\right] = 26.56° \tag{45}$$

where

$\gamma_{\varphi'}$ is partial factor for the angle of shearing resistance (tan φ'), see Appendix D.

Consequently,

$$K_0^1 = 0,47 \text{ (*)} \tag{46}$$

$$K_0^2 = 0,55 \text{ (**)} \tag{47}$$

5. Stress imposed to retaining structure

According to expression (41), stresses applied to wall calculated below.

$$\sigma_{\mathrm{a}}(z) = 0,47\times\left[18\times 4,05+0\right]-2\times 0\sqrt{0,47} = 34kN/m^2 \text{ (*)} \tag{48}$$

$$\sigma_{\mathrm{a}}(z) = 0,55\times\left[18\times 4,05+0\right]-2\times 0\sqrt{0,55} = 41kN/m^2 \text{ (**)} \tag{49}$$

3.6. Combinations

According to Eurocode 8 [3] two different limit states are considered. The combinations were used to verify Ultimate limit state (ULS), see combination (50). Seismic combinations presented in current section according to EN 1990 [1].

3.6.1. EN 1990

Ultimate Limit State

- Combinations of actions for seismic design situations:

$$E_d = \sum_{j\geq 1} G_{k,j} \text{"+"} A_{\mathrm{Ed}} \text{"+"} \sum_{\mathrm{i}\geq 1} \psi_{2,\mathrm{i}} Q_{\mathrm{k,i}} \tag{50}$$

where:

"+"	implies "to be combined with";
$\gamma_{G,j}$	is the partial factor for permanent action *j*;
$G_{k,j}$	is the characteristic value of permanent action *j;*
$\gamma_{Q,1}$	is the partial factor for variable actions *1;*
$Q_{k,1}$	is the characteristic value of the leading variable action *1;*
$\gamma_{Q,I}$	is the partial factor for variable action *I;*
$\psi_{0,I}$	is the factor for combination value of a variable action *i*;
$Q_{k,I}$	is the characteristic value accompanying variable action *i*;
A_{Ed}	is the design value of seismic action $A_{Ed} = \gamma_I A_{Ek}$;
$\psi_{2,I}$	is the factor for quasi-permanent value of a variable action *i*.

In order to obtain loads for security verification of Ultimate Limit State, most unfavourable load assumption taken into account.

The partial factors γ_{Gi} and γ_Q considered for permanent and variable load, respectively, according to their status either favourable or unfavourable values shown in Table 3.13.

Table 3.13: Partial factors

Loads	Favourable	Unfavourable
Self-weight of materials	1.35	1.00
Other permanent loads	1.35	1.00
Variable loads	1.50	0.00

3.6.2. AzDTN 2.1-1

Load combinations according to AzDTN 2.1-1 [17], represent two types of following combinations, depending on the composition of the load:

a. Main load combination includes permanent load P_d, quasi permanent load P_l, variable load P_t which determined by expression (51).

$$C_m = P_d + \sum \psi_{li} P_{li} \text{"+"} \sum \psi_{ti} P_{ti} \tag{51}$$

where

C_m Main load combination;

P_d Permanent load;

P_l Quasi permanent load;

P_t Variable load;

ψ_{li} Coefficient for quasi permanent combination (i=1, 2, 3, …);

ψ_{ti} Coefficient for variable combination (i=1, 2, 3, …).

b. Special load combination includes permanent load P_d, quasi permanent load P_l, variable load P_t and one of the special load P_s, and expressed by expression (52).

$$C_s = C_m + P_s \tag{52}$$

where

C_s Special load combination

P_s Special loads

Special loads P_s, include:

1.Seismic Loads

2.Explosion Loads

3.Loads caused by severe technological process interruption, temporary malfunction or break of equipment.

4.Loads caused by fire

5.Loads caused by car accident with members of structure.

Coefficients for main loads combination (a) presented in Table 3.14.

Table 3.14: Coefficients for main load combination

First important variable load	ψ_{t1}	1,0
Second important variable load	ψ_{t2}	0,9
Third important variable load	$\psi_{t3\ldots}$	0,7
Uniformly distributed quasi permanent load	ψ_{l}	0,95
Remain quasi permanent load	ψ_{l}	1,0

Coefficients for special load combinations (b) presented in Table 3.15.

Table 3.15: Coefficients for special load combination

Permanent	–	0.9
Quasi permanent load	ψ_{l}	0,8
Variable loads	ψ_{t}	0,5
Seismic Load	–	0,8

3.6.3. Horizontal Components of Seismic Action

Horizontal combination of the seismic combinations taken into account consider seismic action in direction X and Y, also considered seismic actions in direction – and +. According EN 1998-1 [3], in one direction of seismic actions should be considered 30% of other simultaneously, as show in expression (16) and (17).

Horizontal components of seismic combinations for Azerbaijan code considered actions in both directions X and Y, also include – and + directions. Seismic actions according AzDTN 2.3-1 [6] should be considered separately.

Seismic combinations used in order to analyse results according two codes presented in Table 3.16.

Table 3.16: Seismic combinations used

EN 1998-1		AzDTN 2.3-1	
1	+X+0.3Y	1	+X
2	–X+0.3Y	2	–X
3	+X–0.3Y	3	+Y
4	–X–0.3Y	4	–Y
5	+0.3X+Y		

6	–0.3X+Y	
7	+0.3X–Y	
8	–0.3X–Y	

3.7.Structural Model

The performance of the structure and its analysis was carried out with a numerical model consisting of bar elements, in the case of beams and columns, and by “finite element” (FEM) in the case of slabs and walls. The structure includes columns which section depth exceed 4 times its width and the height are three times its section depth. According to Eurocode 2 [5], it considered as a concrete wall, and applied in model by two-dimensional plates with further material definition, and analysed by finite element method (see Figure 3.5). Shear walls due to whole height include medium size [14] openings and applied to structural model with maximum precision

The Robot Structural Analysis [44] program was used for evaluation of studied structure. It was considered a three-dimensional model. The model is shown in Figure 3.5. The effects produced by actions on structural elements, considering the various scenarios of loading, were quantified. A linear elastic behaviour of materials involved were considered and in case of the seismic analysis, nonlinear behaviour of materials was taken into account. The structural model fulfils all requirements of EN 1998-1 [3] and AzDTN 2.3-1 [6].

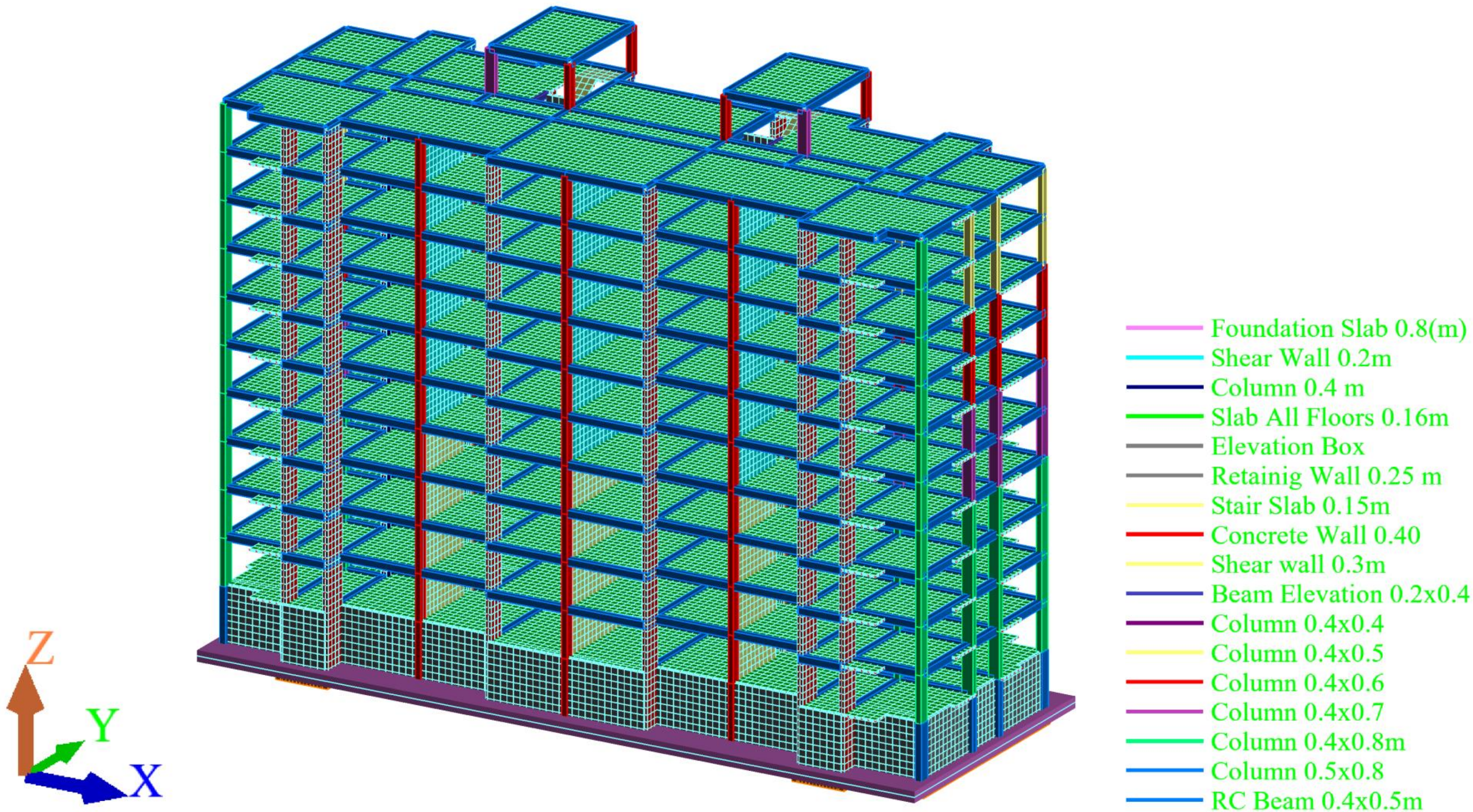
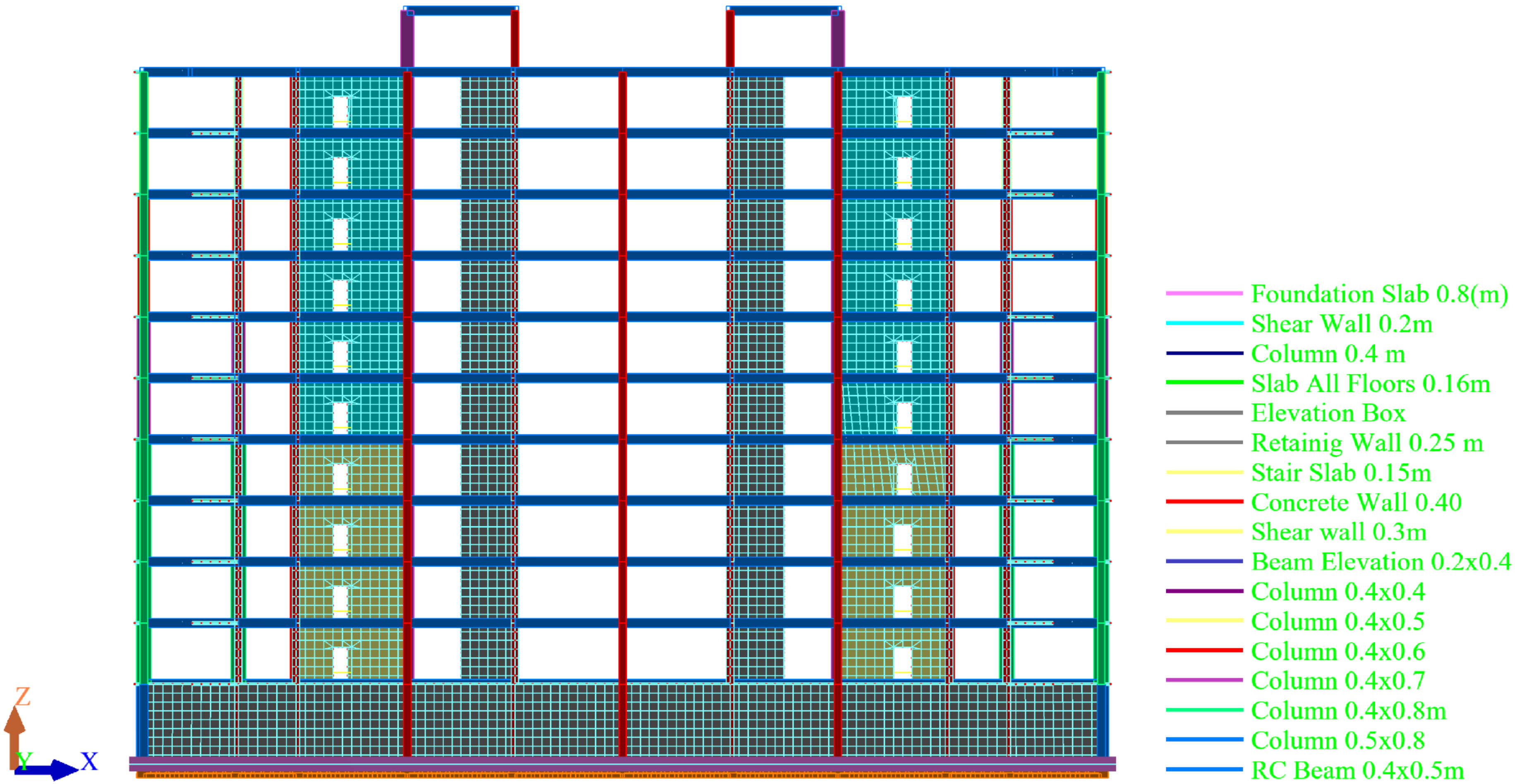
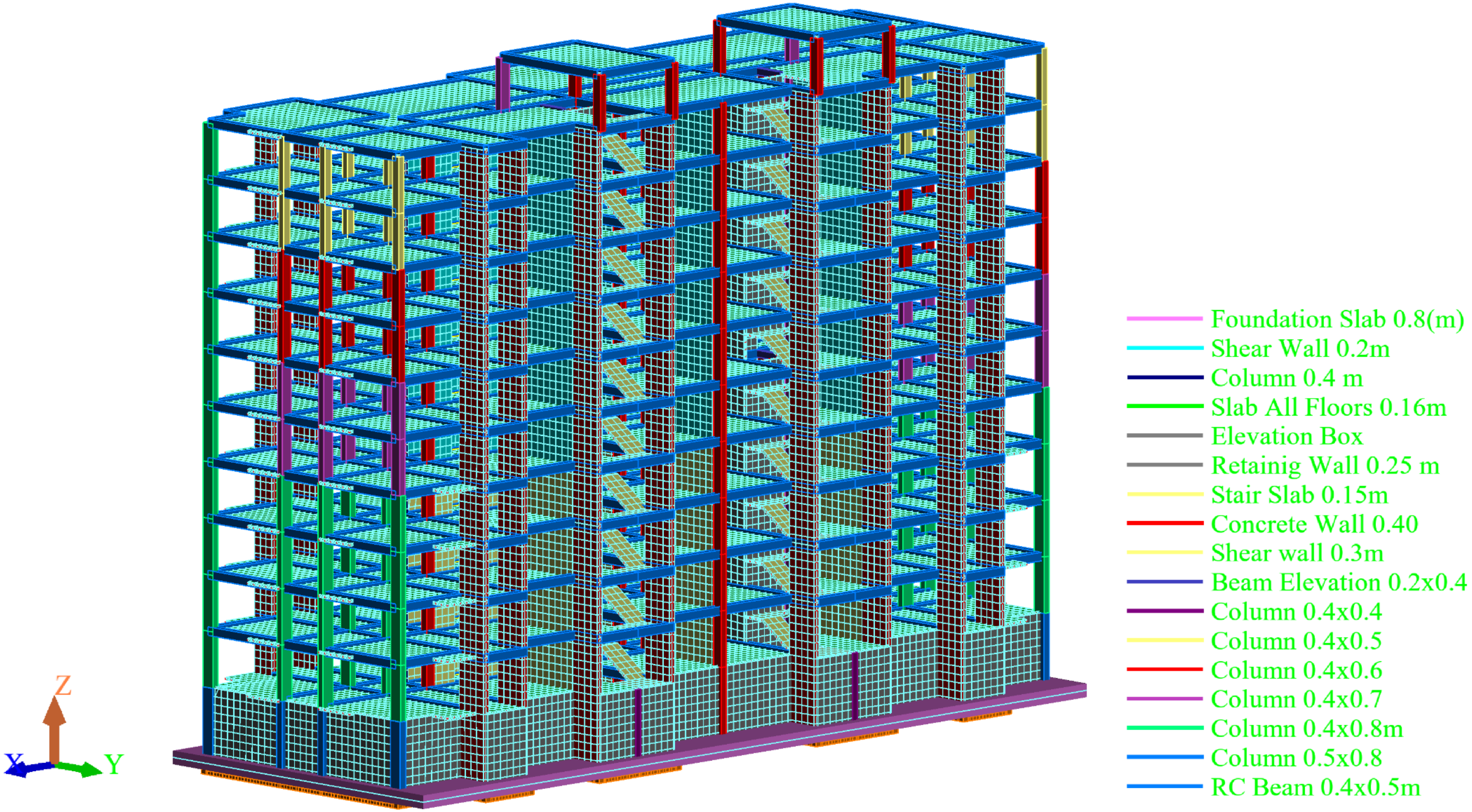


Figure 3.5: Structural model (a)



Structural model front view (b)



Structural model back view (c)

More figures of three-dimensional model are presented in Appendix C.

3.8.Frequencies and Mode Shapes

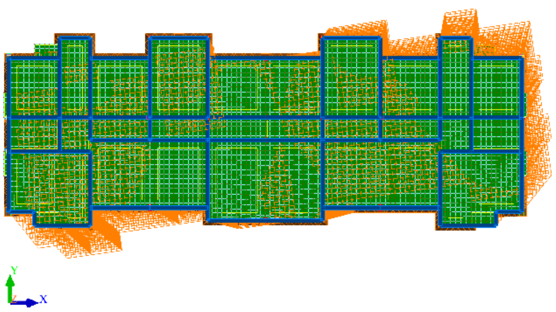
The effect of asymmetry in one direction, as mentioned in section 3.3, clearly visible on fundamental modes of fully non-symmetric structure, building studied is confirmation of it. As far as shear walls distributed non-uniformly in the plan with respect to Y axis, the distribution of stiffness varies thought the plan of structure, which affect to mode shapes (see Figure 3.6).

The values of frequencies corresponding to the first three vibration modes and the modal participation factors for model computed according EN 1998-1 [3], are shown in Table 3.17.

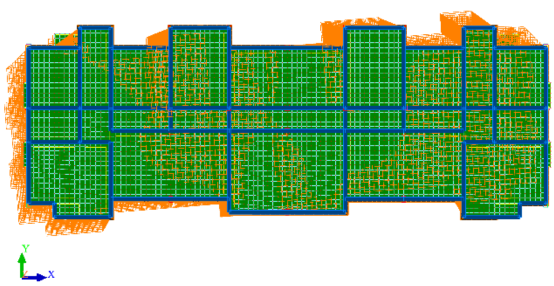
Table 3.17: Results of first three vibration modes according to EN 1998-1 [3]

Vibration modes		1	2	3
Frequency, f [Hz]		1.00	1.07	1.34
Period, T (sec)		1.00	0.93	0.75
Participation factor (%)	X – X	22.41	57.40	57.40
	Y – Y	0.00	0.00	56.29
	Z – Z	0.00	0.00	0.00
Current mass (%)	X – X	22.41	34.99	0.00
	Y – Y	0.00	0.00	56.29
	Z – Z	0.00	0.00	0.00

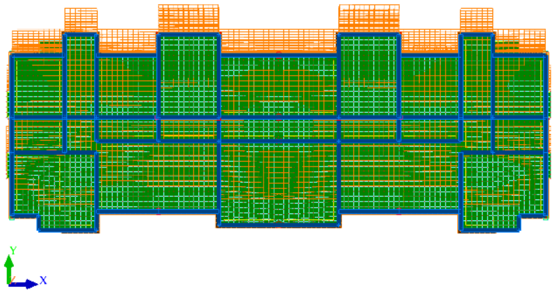
Figure 3.6 (a, b, c) illustrates the results of the first three vibration modes according to the model performed with EN 1998-1 [3] prescriptions.



First mode, $f_1 = 1.00$ Hz



Second mode, $f_2 = 1.07$ Hz



c) Third mode, f_3 = 1.34 Hz

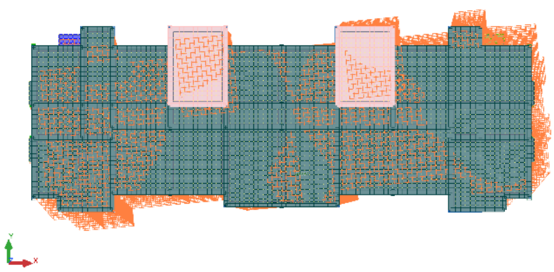
Figure 3.6: Three fundamental vibration modes according to prescriptions of EN 1998- 1 [3]

The values of frequencies corresponding to the first three vibration modes and the modal participation factors for model computed according AzDTN 2.3-1 [6], are presented in Table 3.18.

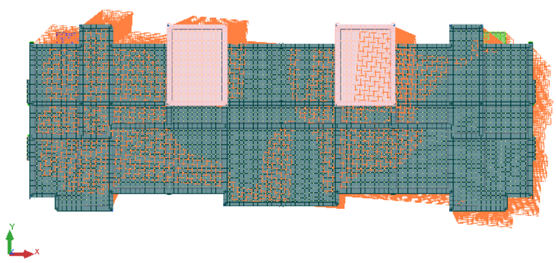
Table 3.18: Results of first three vibration modes according to AzDTN 2.3-1 [6]

Vibration modes		1	2	3
Frequency, f [Hz]		1.44	1.54	1.93
Period, T (sec)		0.70	0.65	0.52
Participation factor (%)	X – X	29.16	66.41	66.41
	Y – Y	0.00	0.00	65.03
	Z – Z	0.00	0.00	0.00
Current mass (%)	X – X	29.16	37.25	0.00
	Y – Y	0.00	0.00	65.03
	Z – Z	0.00	0.00	0.00

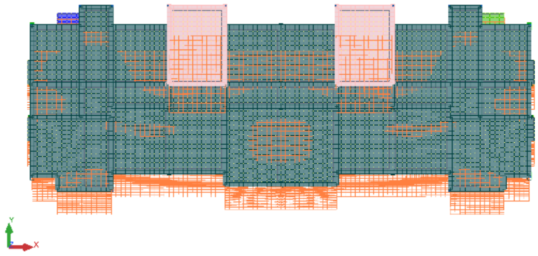
Figure 3.7 (a, b, c) illustrates the results of the three first vibration modes for analysed model according to AzDTN 2.3-1 [6].



First mode, f_1 = 1.44 Hz



Second mode, f_2 = 1.54 Hz



c) Third mode, f_3 = 1.93 Hz

Figure 3.7: Three fundamental vibrations modes according to prescriptions of AzDTN 2.3- 1 [6]

4. Analysis and Interpretation of Results

4.1. Introduction

According to European and Azerbaijan seismic codes the most important parameters for seismic analysis are base shear, response of structural members on seismic load and overall displacement drift, which are presented in this chapter. The results taken from a three-dimensional model computed under combinations described in part 2.2.3 for EN 1998-1 [3] and in part 2.3.3 for AzDTN 2.3-1 [6].

The following aspects will be compared:

- Base shear of structure;
- Displacement drift;
- Forces due to seismic action in several structural members.

The horizontal elastic response spectrum for both seismic codes, for assumed ground type, are presented in Figure 4.1.

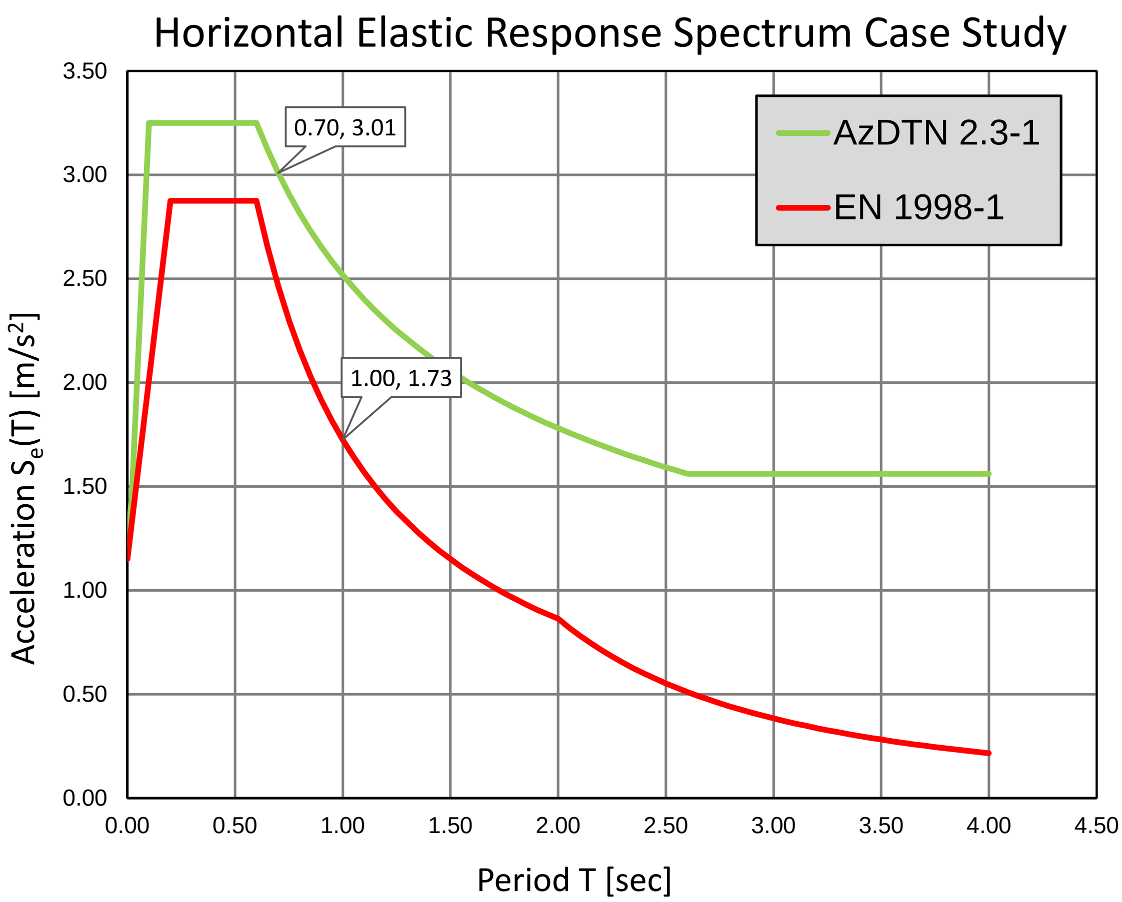


Figure 4.1: Horizontal elastic response spectrum for ground type C and III

Ground acceleration for studied structure, with natural frequency 1.00 Hz with prescriptions of EN 1990 [1], is 1.73m/s^2, while acceleration with prescriptions of AzDTN 2.1-1 [17] is 3.01 m/s^2 with natural frequency of 1.43 Hz. The values of ground accelerations with prescriptions of both seismic cods are 74%.

4.2.Base Shear

Structure's base shear computed according to European seismic code as well as Azerbaijan seismic code are presented in Table 4.1.

Table 4.1: Base shear under seismic combinations

	EN 1998-1		AzDTN 2.3-1		
№	Direction	Base shear [kN]	Direction	Base shear [kN]	Difference (%)
1	+X+0.3Y	15144,24	+X	23774,17	63,70
2	–X–0.3Y	–15144,24	–X	–23774,17	63,70

3	+0.3Y+X	20352.95	+Y	26711,42	76,20
4	–0.3X–Y	–20352.95	–Y	–26711,42	76,20

The result presented in Table 4.1 shows a difference of about 70% between computed models, which is close to the difference in ground acceleration presented in Figure 4.1.

4.3.Displacements and Drifts

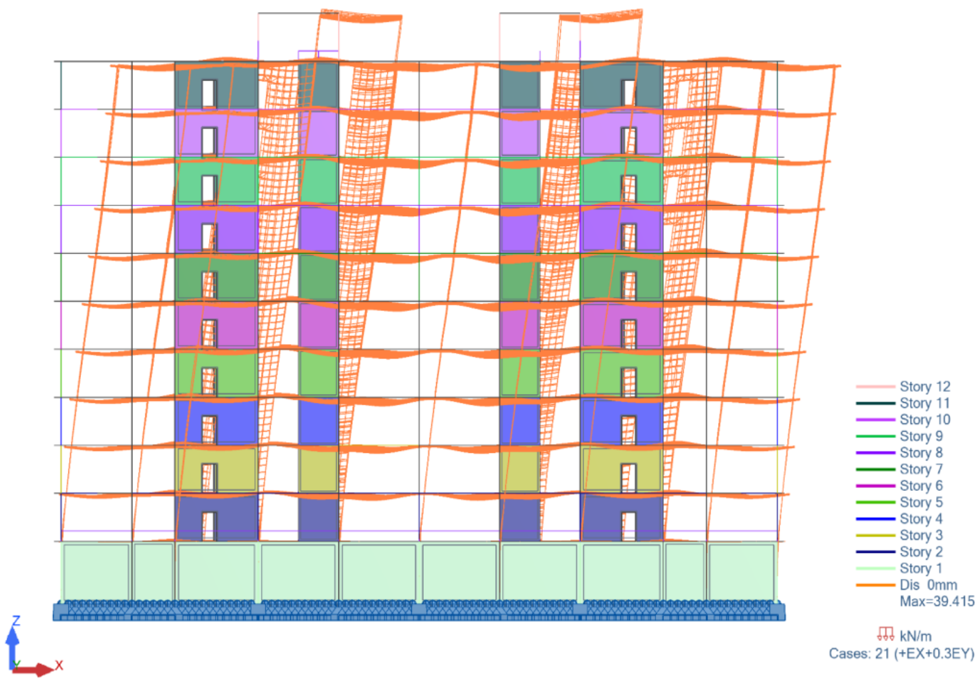
To compare displacements and the drifts, four seismic combinations were used, and are mentioned in Table 4.1. Results of displacement in elevation of structure are straight output from computed model (see section 3.7). The response spectrum analysis was considered to analyse the structure behaviour. For that purpose, two horizontal elastic response spectra (presented in Figure 4.1) have been involved with the computed model.

The drift displacement was compared for of each storey. Displacement drift under seismic combination 1 (see Table 4.1) for EN 1998-1 [3] and AzDTN 2.3-1 [6] and also deference in percentage presented in Table 4.2.

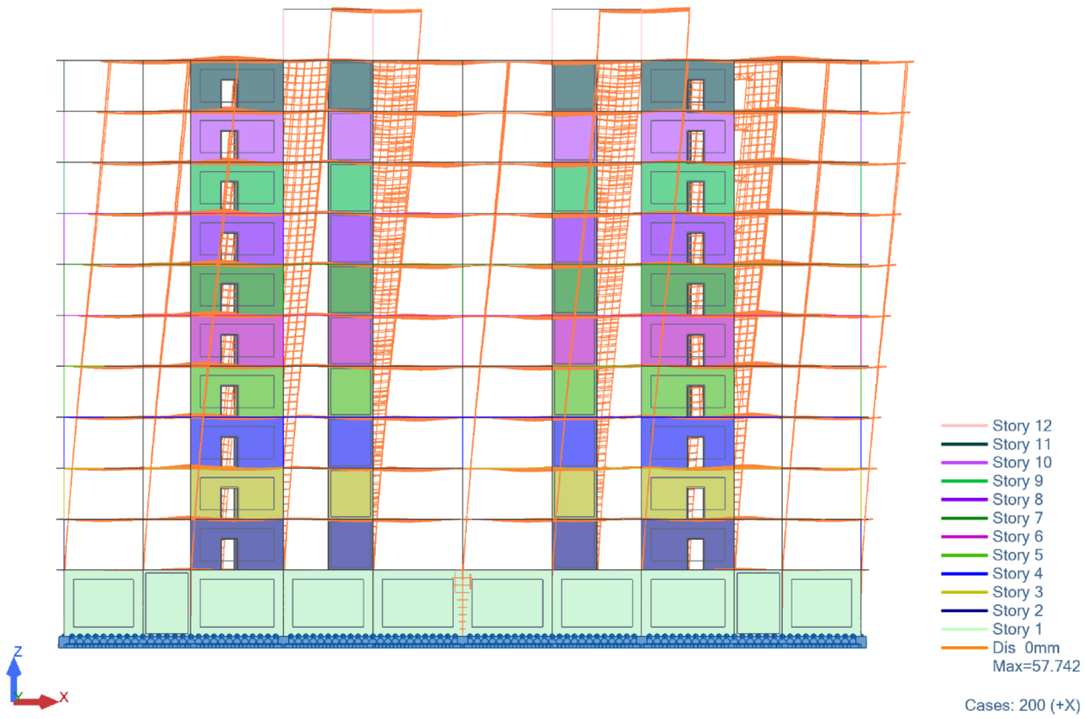
Table 4.2: Displacement drift under seismic combination 1 (see Table 4.1)

EN 1998-1 [+ X + 0.3 Y]			AzDTN 2.3-1 [+ X]			
Storey	Displacement (mm)	Drift (mm)	Storey	Displacement (mm)	Drift (mm)	Difference in drift (%)
11	40.12	3.35	11	57.74 mm	4.67	71.69
10	36.77	3.88	10	53.08 mm	5.51	70.50
9	32.89	4.17	9	47.57 mm	5.99	69.54
8	28.72	4.19	8	41.58 mm	6.41	65.39
7	24.53	4.55	7	35.17 mm	6.60	68.89
6	19.98	4.76	6	28.57 mm	6.63	71.77
5	15.22	4.05	5	21.93 mm	6.24	64.91
4	11.17	4.00	4	15.70 mm	5.63	71.04
3	7.17	3.56	3	10.06 mm	5.07	70.29
2	3.61	2.97	2	5.00 mm	4.00	74.09
1	0.64		1	0.99 mm		

The deformation shapes under seismic combination 1 (see Table 4.1), presented below in Figure 4.2.



a) Deformation shape under +X + 0.3 Y combination according EN 1998-1 [3]



b) Deformation shape under +X combination according AzDTN 2.3-1 [6]

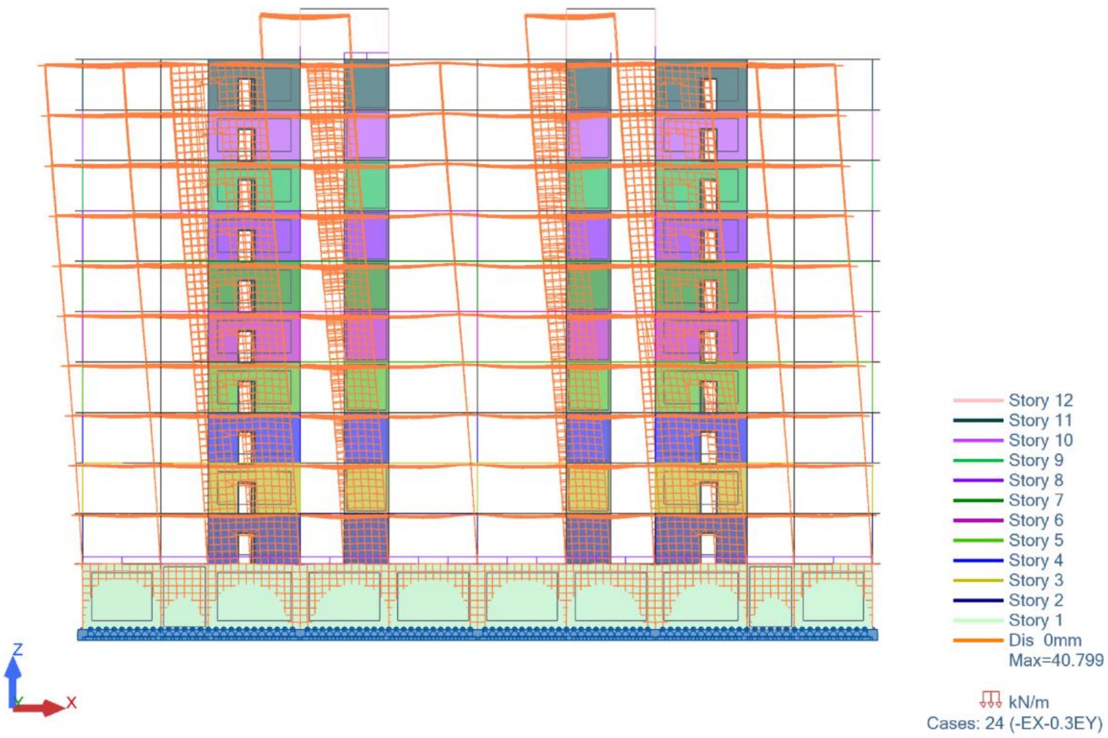
Figure 4.2: Deformation shapes under seismic combination 1 (see Table 4.1)

Table below present values of displacement and drift for each storey under seismic combination 2 (see Table 4.1)

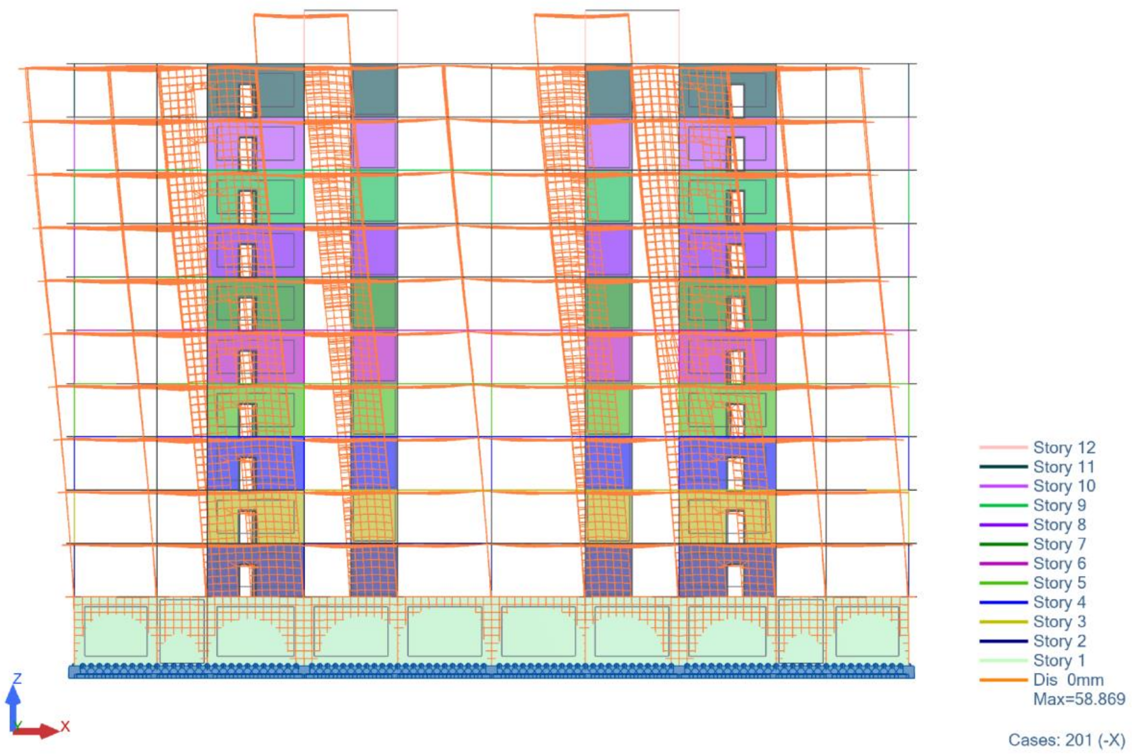
Table 4.3: Displacement drift under seismic combination 2 (see Table 4.1)

EN 1998-1 [– X – 0.3 Y]			AzDTN 2.3-1 [– X]			
Storey	Displacement (mm)	Drift (mm)	Storey	Displacement (mm)	Drift (mm)	Difference in drift (%)
11	40.79	3.31	11	58.80	4.79	69.19
10	37.47	3.92	10	54.01	5.64	69.45
9	33.56	4.22	9	48.37	6.12	68.96
8	29.34	4.50	8	42.25	6.53	68.86
7	24.84	4.57	7	35.72	7.46	61.17
6	20.28	4.55	6	28.26	6.45	70.57
5	15.73	4.26	5	21.81	5.63	75.59
4	11.47	4.00	4	16.17	5.71	70.04
3	7.47	3.56	3	10.47	5.07	70.12
2	3.92	3.12	2	5.40	4.23	73.78
1	0.80		1	1.17		

The deformation shapes under seismic combination 2 (see Table 4.1), presented below in Figure 4.3.



a) Deformation shape under –X – 0.3 Y combination according EN 1998-1 [3]



b) Deformation shape under –X combination according AzDTN 2.3-1 [6]

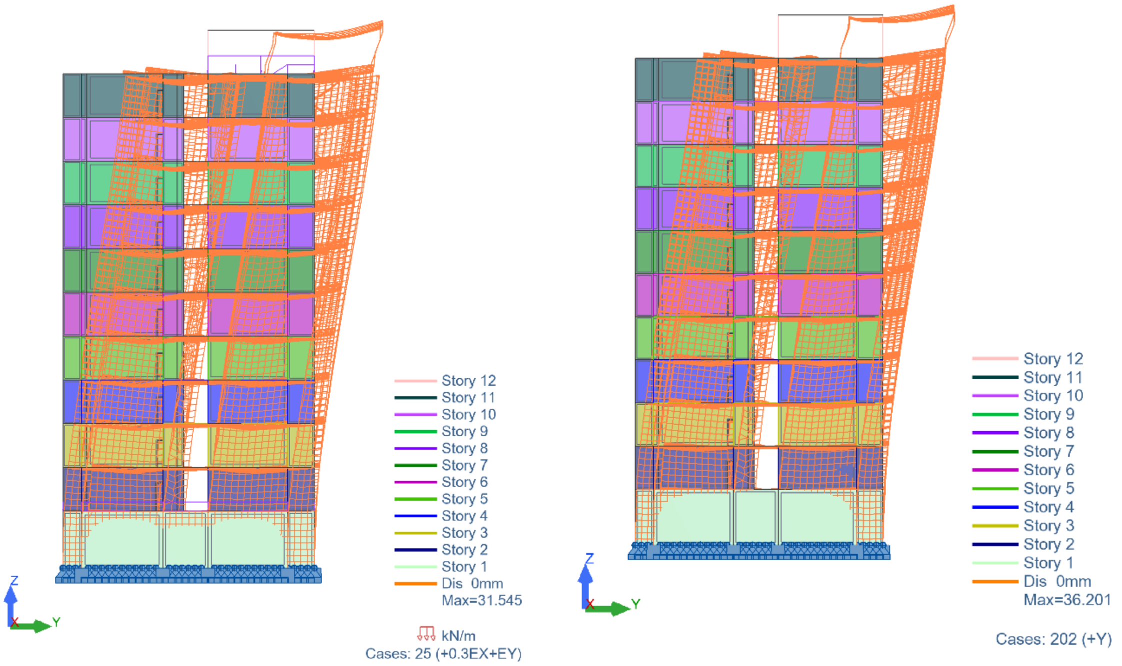
Figure 4.3: Deformation shapes under seismic combination 2 (see Table 4.1)

Table 4.4 present values of displacement and drift for each storey under seismic combination 3 (see Table 4.1).

Table 4.4: Displacement drift under seismic combination 3 (see Table 4.1)

EN 1998-1 [+ 0.3X + Y]			AzDTN 2.3-1 [+ Y]			
Storey	Displacement (mm)	Drift (mm)	Storey	Displacement (mm)	Drift (mm)	Difference in drift (%)
11	29.54	2.52	11	31.70	3.04	82.84
10	27.03	2.87	10	28.67	3.12	91.84
9	24.16	3.15	9	25.54	3.42	92.04
8	21.01	3.31	8	22.12	3.58	92.40
7	17.70	3.34	7	18.54	3.60	92.86
6	14.36	3.30	6	14.94	3.50	94.23
5	11.06	3.10	5	11.44	3.25	79.83
4	7.96	2.90	4	8.97	2.73	94.23
3	5.06	1.77	3	6.24	2.41	73.37
2	3.30	2.24	2	3.83	2.58	86.70
1	1.06		1	1.26		

The deformation shapes under seismic combination 3 (see Table 4.1), presented below in Figure 4.4.



a) Deformation shape under +0.3X + Y combination according EN 1998-1 [3]

b) Deformation shape under +Y combination according AzDTN 2.3-1 [6]

Figure 4.4: Deformation shapes under seismic combination 3 (see Table 4.1)

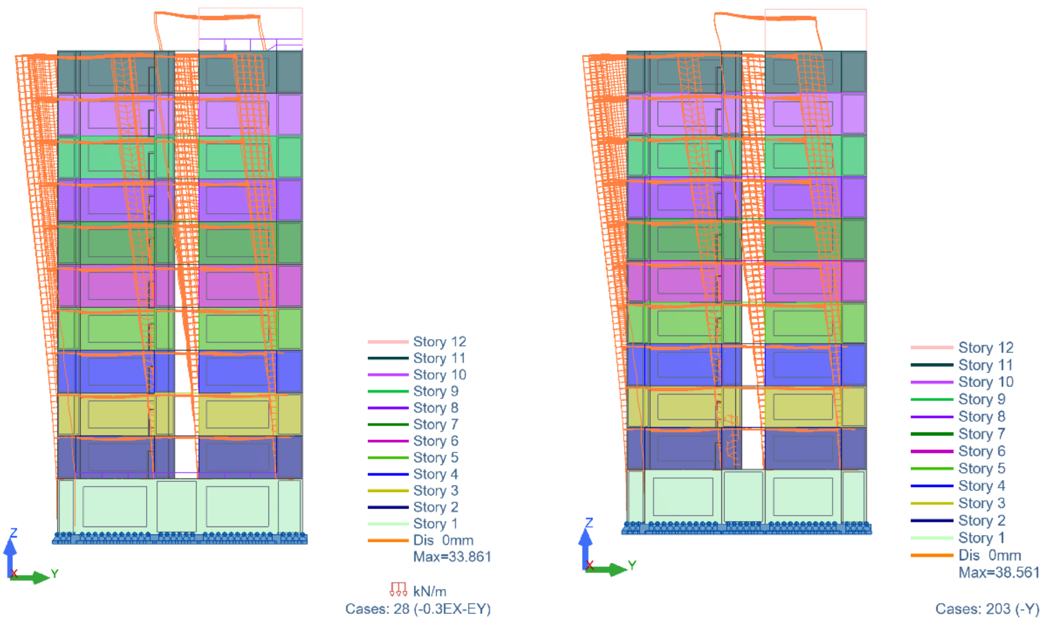
Table below present values of displacement and drift for each storey under seismic combination 4 (see Table 4.1).

Table 4.5: Displacement drift under seismic combination 4 (see Table 4.1)

EN 1998-1 [– 0.3X – Y]			AzDTN 2.3-1 [– Y]			
Storey	Displacement (mm)	Drift (mm)	Storey	Displacement (mm)	Drift (mm)	Difference in drift (%)
11	28.94	2.50	11	31.49	2.82	88.81
10	26.44	2.81	10	28.67	3.13	89.53
9	23.64	3.05	9	25.54	3.39	89.75
8	20.59	3.19	8	22.15	3.54	90.18
7	17.40	3.22	7	18.61	2.80	86.90
6	14.17	2.41	6	15.81	2.70	89.07
5	11.77	3.24	5	13.11	3.19	98.55

4	8.53	2.27	4	9.91	2.89	78.58
3	6.26	2.27	3	7.02	2.60	87.15
2	3.99	2.96	2	4.42	3.19	92.61
1	1.03		1	1.23		

The deformation shapes under seismic combination 4 (see Table 4.1), presented below in Figure 4.5.



a) Deformation shape under –0.3X – Y combination according EN 1998-1 [3]

b) Deformation shape under –Y combination according AzDTN 2.3-1 [6]

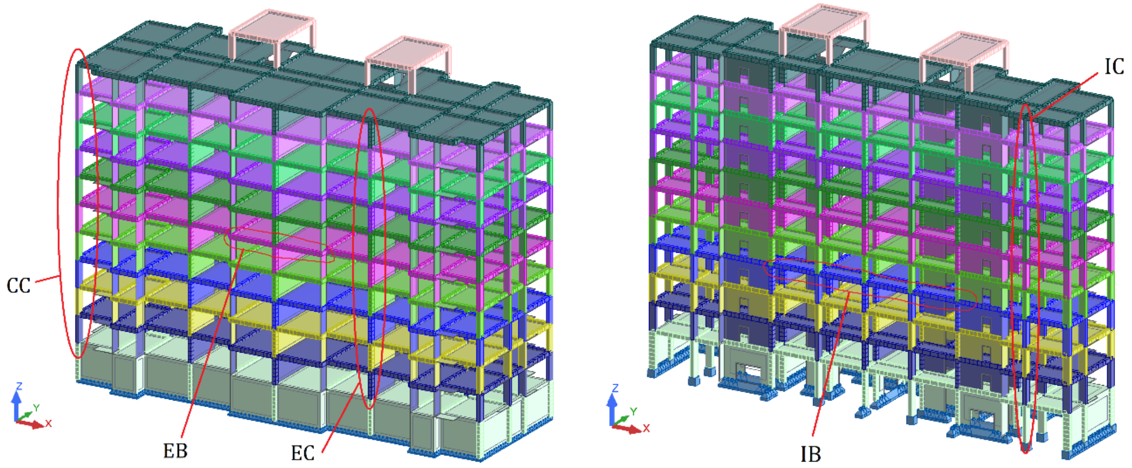
Figure 4.5: Deformation shapes under seismic combination 4 (see Table 4.1)

More shape modes presented in Appendix E.

4.4.Forces in Structural Members

In this section the forces in different structural members are presented. For columns: corner column (CC), edge column (EC) and inner column (IC) with cross section shown in Figure 4.7, were selected to compare results. For beam: edge beam (EB), inner beam (IB) and also shear walls were taken into comparison. The beams considered in this section have cross-sectional geometry of 0.4 m width and 0.5 m depth. In order to get results horizontal components of combinations presented in Table 3.16 are considered.

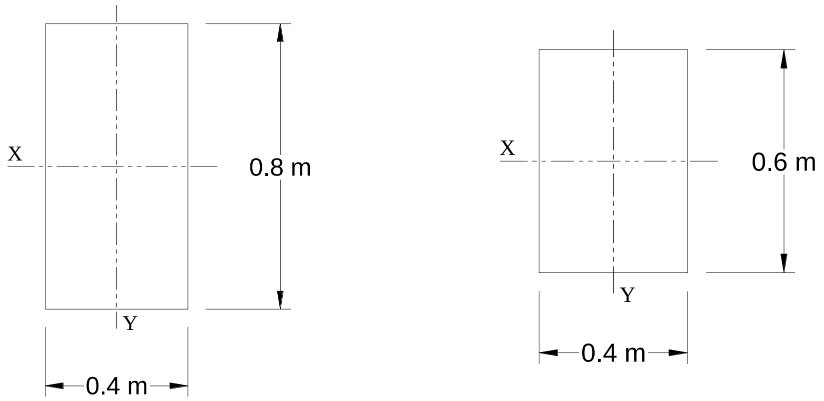
The Figure 4.6 shows members selected to compare the results.



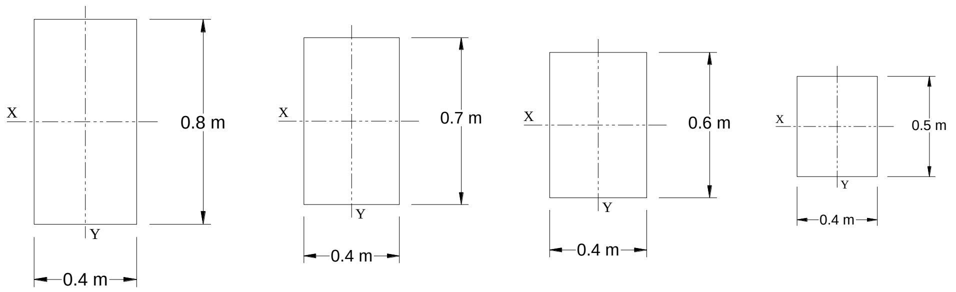
a) Corner column, edge beam and edge column b) Inner column and inner beam

Figure 4.6: Structural members considered to comparison

Cross-sectional geometry of columns taken into comparison presented in Figure 4.7.



a) Corner column (CC) geometry b) Edge column (EC) geometry



c.1) Cross section 0-3 storeys

c.2) Cross section 4-5 storeys

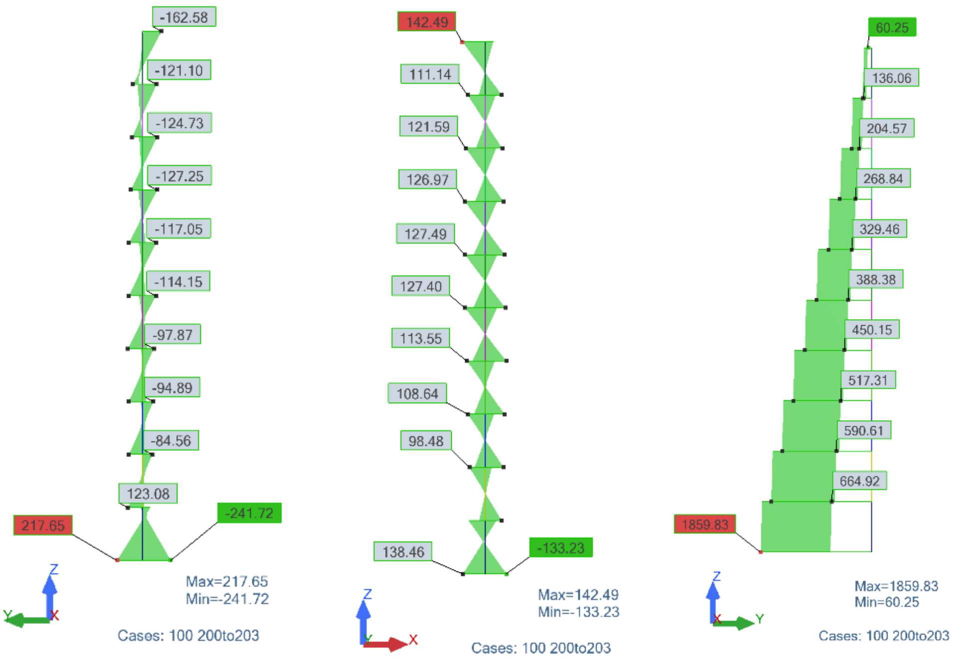
c.3) Cross section 6-7 storeys

c.4) Cross section 8-9 storeys

c) Inner column (IC)

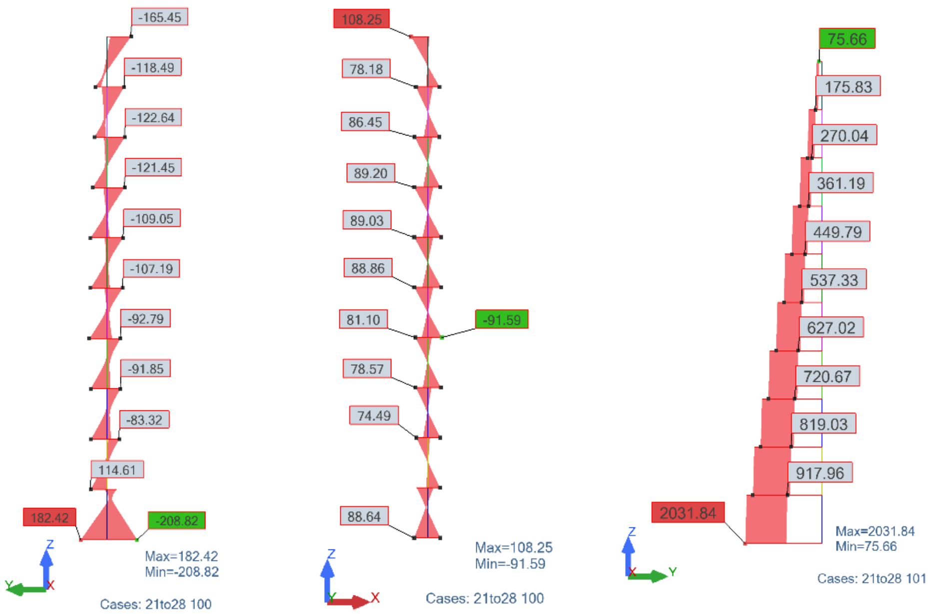
Figure 4.7: Geometry of analysed columns

Forces in corner column (CC) are presented in Figure 4.8 and Figure 4.9.



a) Moment (Y) in CC b) Moment (Z) in CC c) Axial force in CC

Figure 4.8: Forces for corner column according AzDTN 2.3-1 [6]



a) Moment (Y) in CC b) Moment (Z) in CC c) Axial forces in CC

Figure 4.9: Forces for corner column according EN 1998-1 [3]

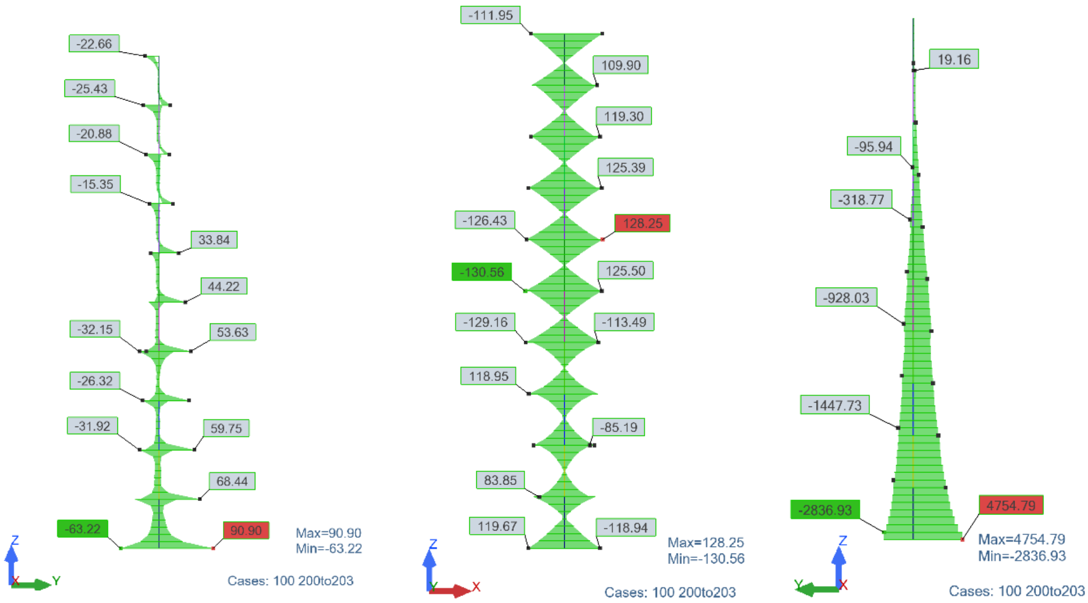
Forces in corner column with prescriptions of both seismic codes are compared in Table 4.6.

Table 4.6: Forces observed in corner column (CC)

Forces	AzDTN 2.3-1 [6]		EN 1998-1 [3]		Difference (%)	
	Min	Max	Min	Max	Min	Max
Moment (Y) in kN/m	–241.72	217.65	–208.82	182.42	86.39	83.81
Moment (Z) in kN/m	–133.23	142.49	–91.59	108.25	68.74	75.97
Axial force in kN	60.25	1859.83	75.66	2031.84	84.37	91.53

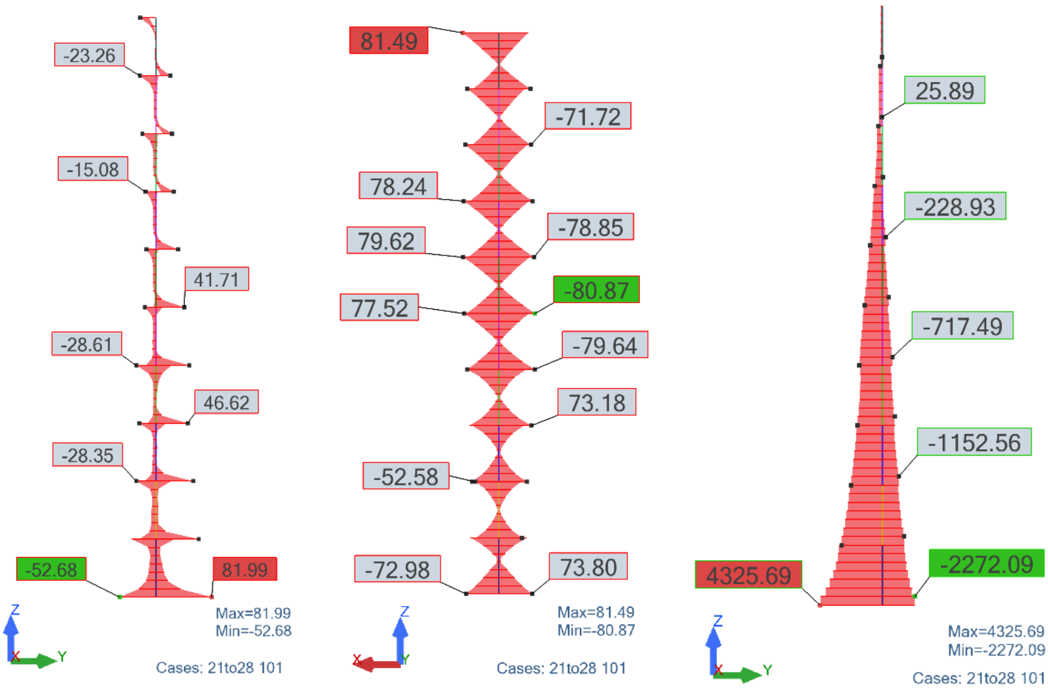
Moments in corner column (CC) as well as axial force have an average of about 81% difference between models computed according prescriptions of AzDTN 2.3-1 [6] and EN 1998-1 [3] (see Table 4.6), considering higher and lower value. The difference is expected as far as base shear computed presents roughly the same value of difference.

Forces in edge column (EC) are presented in Figure 4.10 and Figure 4.11.



a) Moment (Y) in EC b) Moment (Z) in EC c) Axial force in EC

Figure 4.10: Forces for edge column according AzDTN 2.3-1 [6]



a) Moment (Y) in EC b) Moment (Z) in EC c) Axial forces in EC

Figure 4.11: Forces for edge column according EN 1998-1 [3]

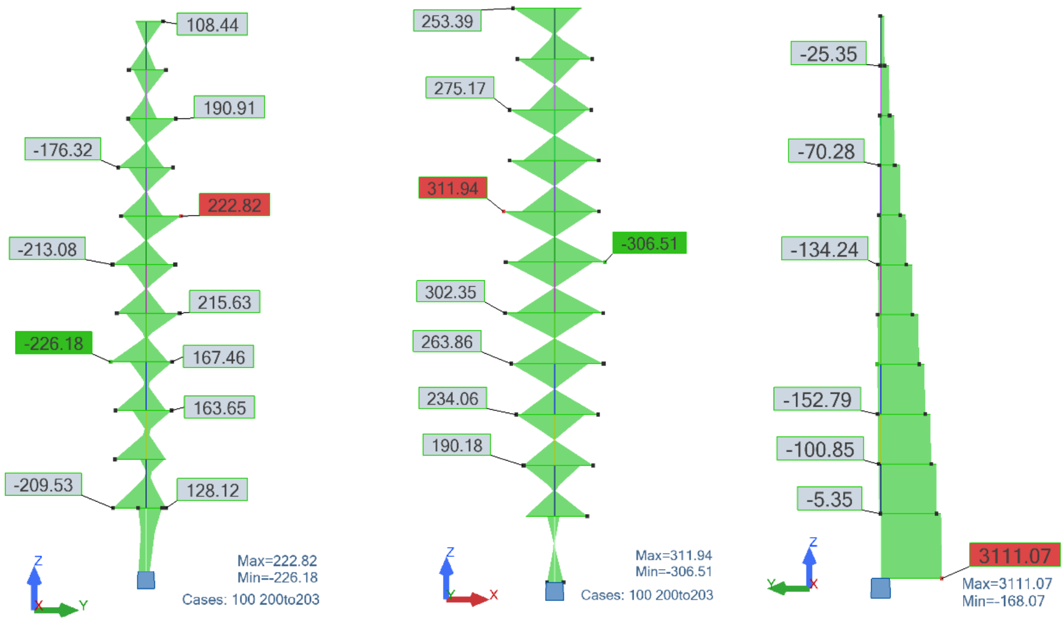
Forces in edge column with prescriptions of both seismic codes are compared in Table 4.7.

Table 4.7: Forces observed in edge column (EC)

Forces	AzDTN 2.3-1 [6]		EN 1998-1 [3]		Difference (%)	
	Min	Max	Min	Max	Min	Max
Moment (Y) in kN/m	–63.22	90.90	–52.68	81.99	83.32	90.19
Moment (Z) in kN/m	–130.56	128.25	–80.87	81.49	63.53	61.94
Axial force in kN	–2836.93	4754.79	–2272.09	4325.69	80.08	90.09

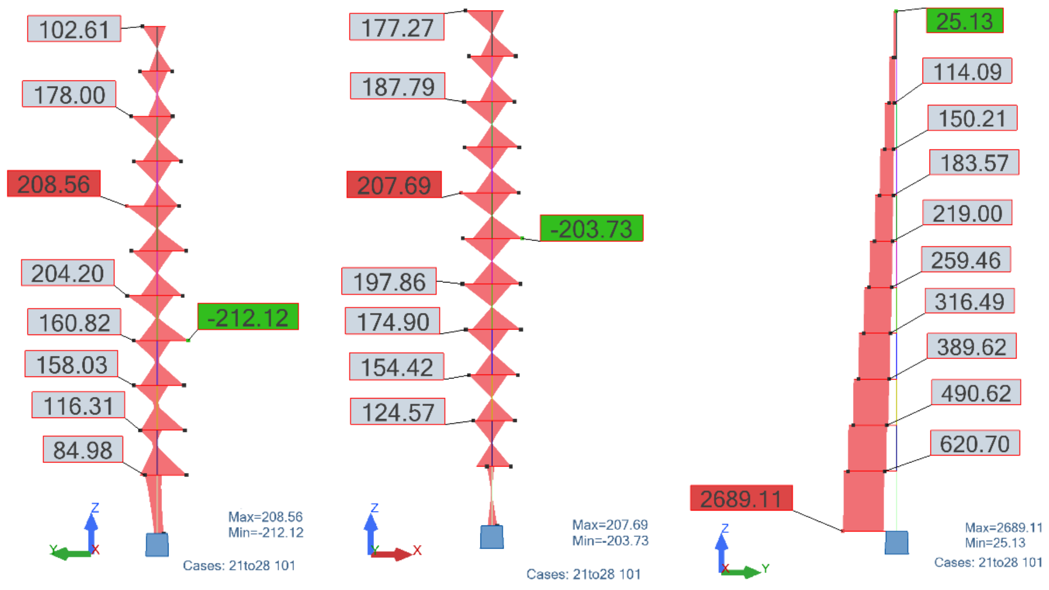
Moments in edge column (EC) as well as axial force have an average of about 78% difference between models computed (see Table 4.7), considering higher and lower value.

Forces in inner column (IC) are presented in Figure 4.12 and Figure 4.13.



a) Moment (Y) in IC b) Moment (Z) in IC c) Axial force in IC

Figure 4.12: Forces for inner column according AzDTN 2.3-1 [6]



a) Moment (Y) in IC   b) Moment (Z) in IC   c) Axial forces in IC

Figure 4.13: Forces for inner column according EN 1998-1 [3]

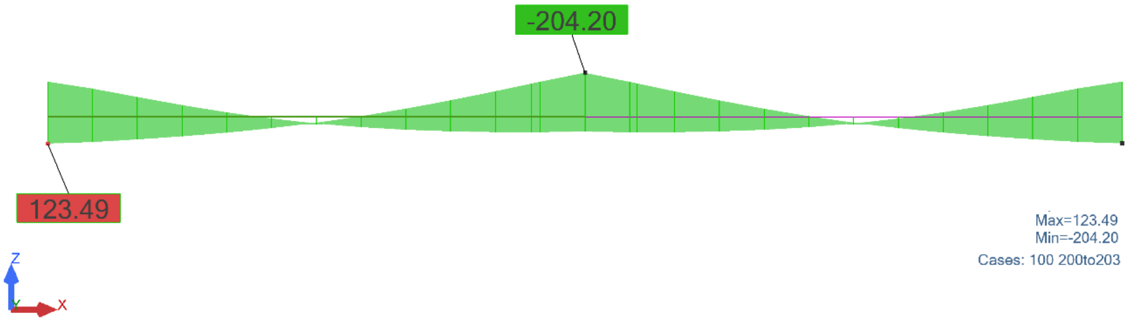
Forces in inner column with prescriptions of both seismic codes are presented in Table 4.8.

Table 4.8: Forces observed in inner column (IC)

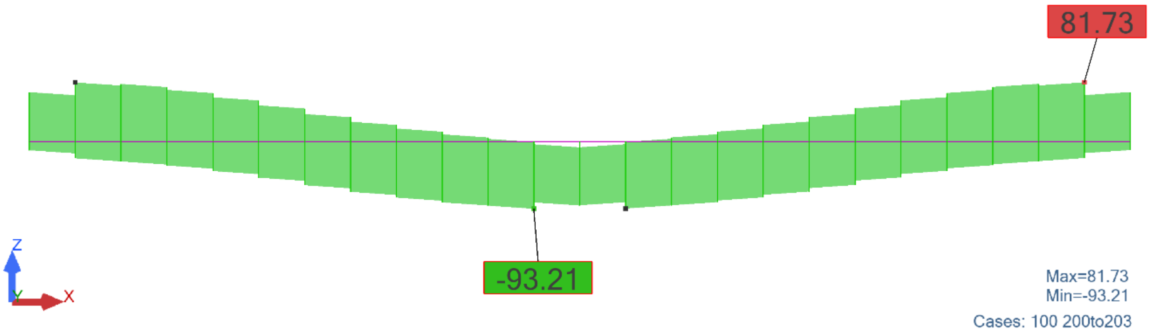
Forces	AzDTN 2.3-1		EN 1998-1		Difference (%)	
	Min	Max	Min	Max	Min	Max
Moment (Y) in kN/m	–226.18	222.82	–212.12	208.56	93.78	93.60
Moment (Z) in kN/m	–306.15	311.94	–203.73	207.69	66.54	66.58
Axial force in kN	–25.35	3111.07	25.13	2689.11	99.13	86.43

Moments in inner column (IC) as well as axial force have an average difference of about 83% between models computed (see Table 4.8), considering higher and lower value. The difference is expected as far as base shear (see Table 4.1) computed presents roughly the same value of difference.

Forces in edge beam (EB) with two spans presented in Figure 4.14 and Figure 4.15.

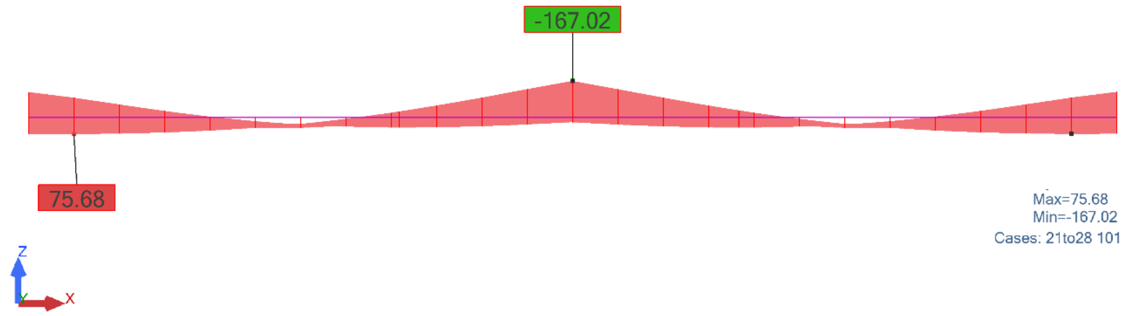


a) Moment (Y) in EB

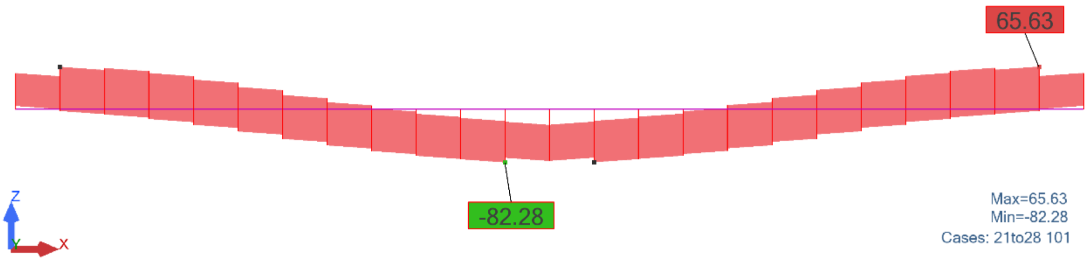


b) Shear force in EB

Figure 4.14: Forces for edge beam according AzDTN 2.3-1



a) Moment (Y) in EB



b) Shear force in IB

Figure 4.15: Forces for edge beam according EN 1998-1

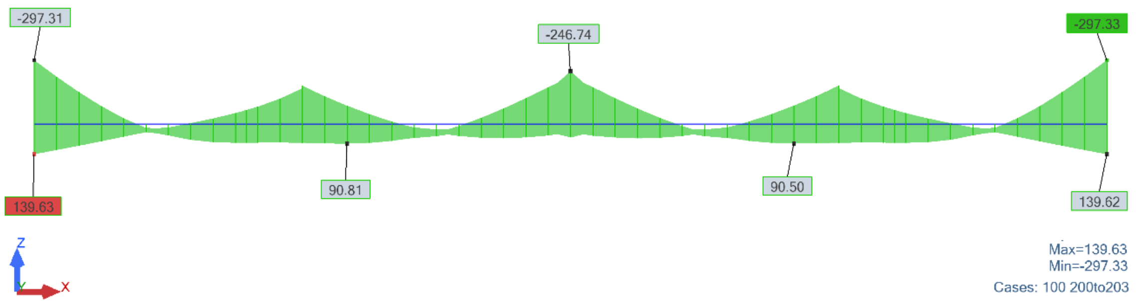
Forces in edge beam with prescriptions of both seismic codes are compared in Table 4.9.

Table 4.9: Forces observed in edge beam (EB)

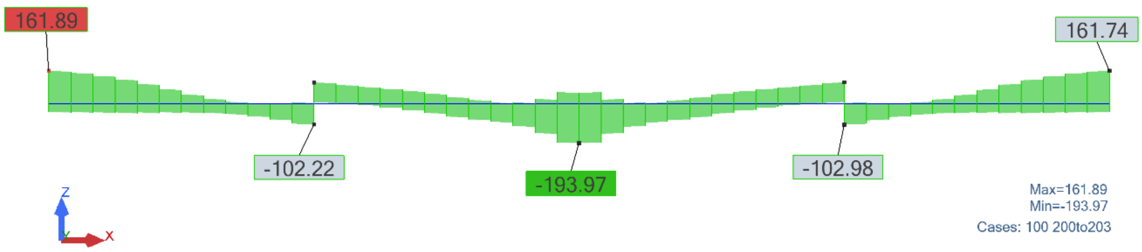
Forces	AzDTN 2.3-1 [6]		EN 1998-1 [3]		Difference (%)	
	Min	Max	Min	Max	Min	Max
Moment (Y) in kN/m	–204.20	123.49	–167.02	75.68	81.79	61.28
Shear force in kN	–93.21	81.73	–82.28	65.63	88.27	80.30

Moment in edge beam (EB) as well as shear force is about 78% difference between models computed (see Table 4.9) as it shows all columns presented above ,see Table 4.6, Table 4.7 and Table 4.8.

Forces in inner beam (IB) with three spans presented in Figure 4.16 and Figure 4.17.

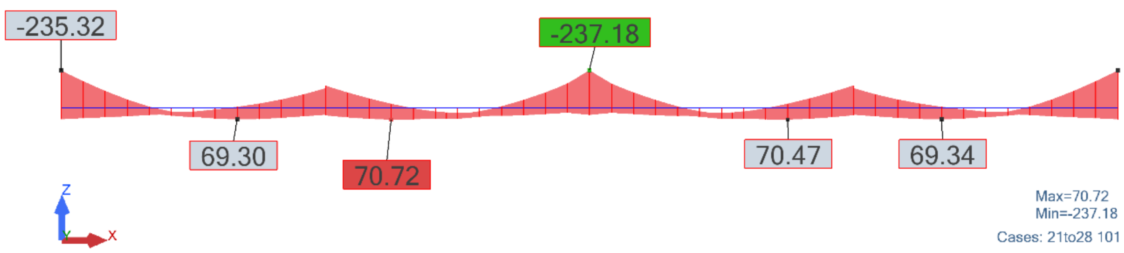


a) Moment (Y) in IB

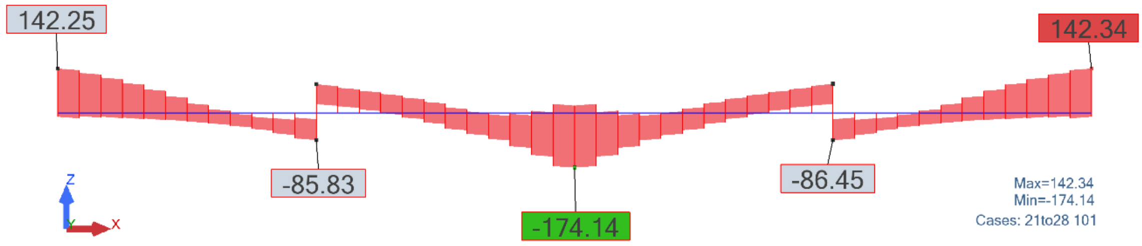


b) Shear force in IB

Figure 4.16: Forces for inner beam according AzDTN 2.3-1



a) Moment (Y) in EB



b) Shear force in IB
Figure 4.17: Forces for edge beam according EN 1998-1

Forces in edge beam with prescriptions of both seismic codes are compared in Table 4.10.

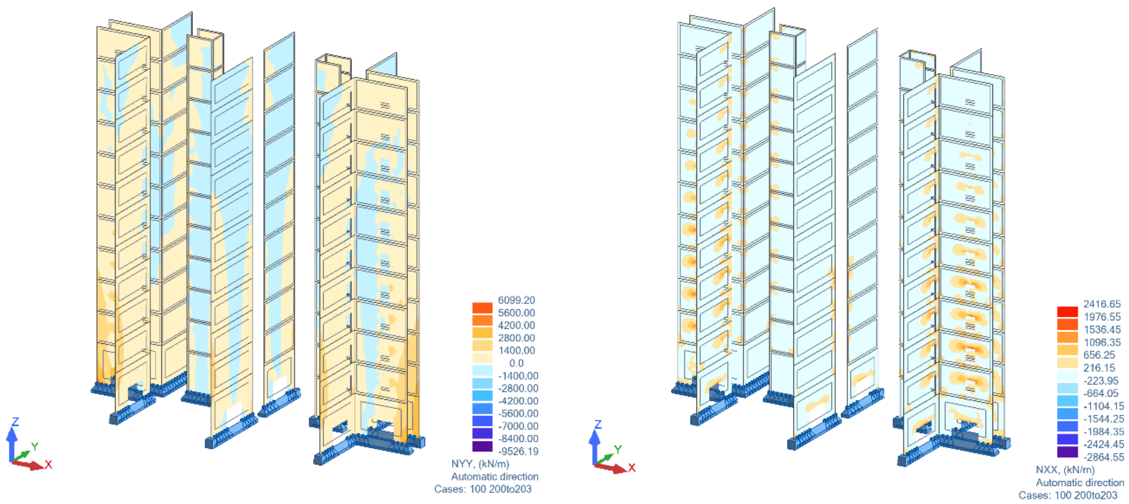
Table 4.10: Forces observed in inner beam (IB)

Forces	AzDTN 2.3-1		EN 1998-1		Difference (%)	
	Min	Max	Min	Max	Min	Max
Moment (Y) in kN/m	–297.33	139.63	–237.18	70.72	79.79	50.64
Shear force kN	–193.97	161.89	–174.14	142.34	89.77	87.92

Moment in inner beam (IB) as well as shear force present 76% average difference between models computed (see Table 4.10) as it shows all columns presented above.

In order to compare the results of response of shear walls, membrane forces are taken into account. The results consider horizontal components of combinations presented in Table 3.16.

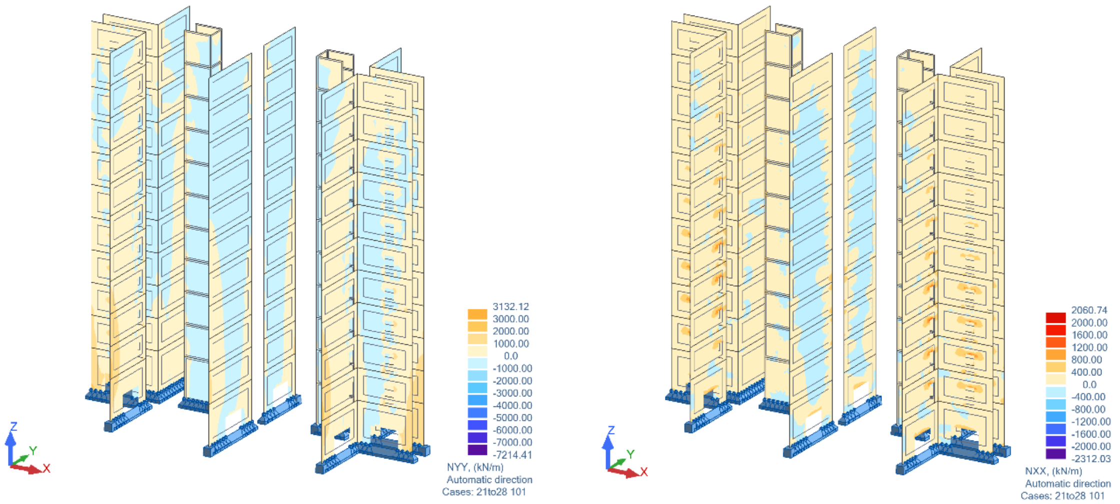
Membrane forces in two directions for calculated model according to AzDTN 2.3-1 [6] are presented in Figure 4.18.



a) Y direction b) X direction

Figure 4.18: Membrane forces in shear walls according to AzDTN 2.3-1

Membrane forces in two directions for calculated model according to EN 1998-1 [3] are presented in Figure 4.19.



a) Y direction b) X direction

Figure 4.19: Membrane forces in shear walls according to EN 1998-1

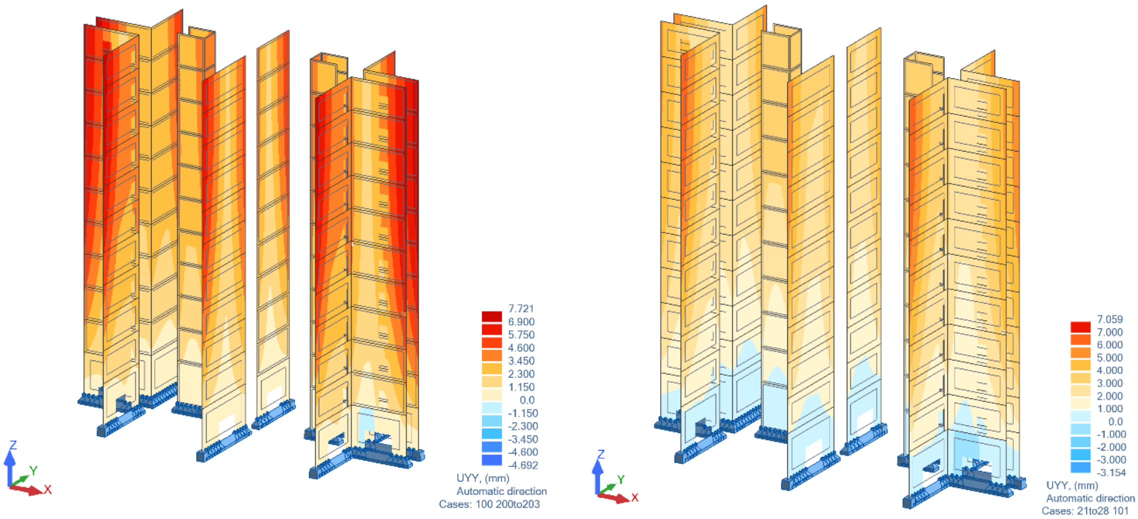
Forces in shear walls with prescriptions of both seismic codes are compared in Table 4.11.

Table 4.11: Forces observed in shear walls

Forces (kN/m)	AzDTN 2.3-1		EN 1998-1		Difference (%)	
	Min	Max	Min	Max	Min	Max
Membrane force (Y)	–9526.19	6099.20	–7214.41	3132.12	75.73	51.35
Membrane force (X)	–2864.55	2416.65	–2312.03	2060.74	80.71	71.93

Membrane forces in shear walls have an average of about 69% difference between models computed (see Table 4.11), considering higher and lower value. The difference is expected as far as base shear computed (see Table 4.1) presents roughly the same value of difference.

Displacement in shear walls under horizontal components of seismic combination (see Table 3.16 in section 3.6.3), presented in Figure 4.20.



a) AzDTN 2.3-1 b) EN 1998-1

Figure 4.20: Displacement in shear walls in Y direction

Displacements in shear walls with prescriptions of both seismic codes are compared in Table 4.12.

Table 4.12: Displacement observed in shear walls

Displacement (mm)	AzDTN 2.3-1		EN 1998-1		Difference (%)	
	Min	Max	Min	Max	Min	Max
Direction Y	–4.692	7.721	–3.154	7.059	67.22	91.42

Displacement in shear walls have an average of about 79% difference between models computed (see Table 4.12), considering higher and lower value. The difference is expected as far as base shear computed (see Table 4.1) presents roughly the same value of difference.

5. Conclusions and Future works

In this Chapter the main conclusion will be presented based on comparison of the two codes analysed and the results obtained with the case study chosen for this purpose.

5.1. Summary of Conclusions

As the results show, the biggest statement is that Azerbaijan seismic code presents more conservativity in all important aspects of seismic analyses than Eurocode 8. For aspects presented in the fourth Chapter the differences in results of the different parameters analyzed vary between 60 to 80 %.

In terms of economy, prescriptions of EN 1998-1 are more affordable than AzDTN 2.3-1, as far as results show, higher forces in structural members lead to using stiffer elements with higher geometry and consequently higher construction costs.

As mentioned in the first Chapter Azerbaijan Republic is moving towards European standards. The conclusion drawn by the author based on case study says is that the Azerbaijan Republic could accept European Construction Standards without major changes.

One of the main differences can be related with seismic combinations, as far as AzDTN 2.3-1 does not consider seismic action in two directions simultaneously (see Table 3.16 in chapter 3.6.3), as it present EN 1998-1. Another, not less important issue is that EN 1998-1 considers “accidental torsional effects”, which does not exist in AzDTN 2.3-1 prescriptions. These two aspects must be accepted by Azerbaijan Construction Standards in order to enter to European Standards. The first steps have already been taken in 2011 in the 1st reissue of AzDTN 2.3-1, while classification and parameters of ground type were adopted from EN 1998-1.

The structural members considered in section 4.4 show differences in forces between 70% to 80%, which meets expectations, as far as close differences observed in horizontal elastic response spectrum.

5.2. Future Developments

In this study the author analyzes a multistorey building with eleven storeys. Based on the conclusion of this work it will be important in the future to analyze different situations taken into account different types of buildings.

This project could be developed further, for example:

- Comparison of buildings with different structural types, such as “frame system” or ductile “wall system”;
- Comparison of structure with other ground types;
- Comparison of similar buildings with different behaviour factors;
- Comparison with totally asymmetric and irregular in elevation and in plan structures (situations not allowed in the studied codes).

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Appendices

In this Chapter the author present figures and tables which are not included to the main text.

Appendix A presents architectural drawings, which is north and east facades of the studied building. Drawings presented by Azerbaijan Architecture and Construction University, Department of Reinforced Concrete Structure.

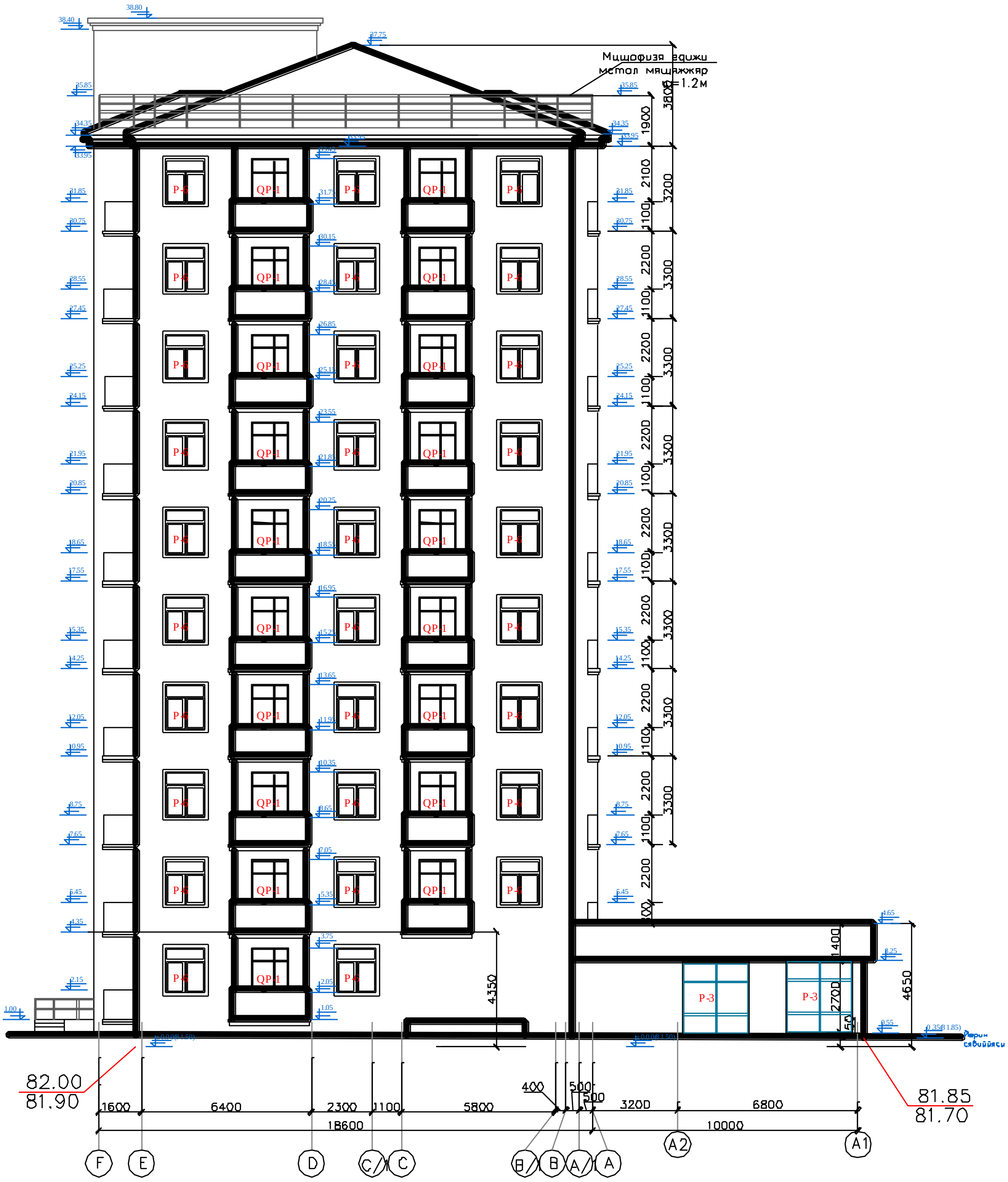
Appendix B presents structural plan which drawn by the author, based on architectural drawings provided.

Appendix C presents a numerical three-dimensional model.

Appendix D presents partial factors used to determining stresses applied to retaining structure.

Appendix E presents displacement shapes under seismic combinations.

Appendix A



Appendix B

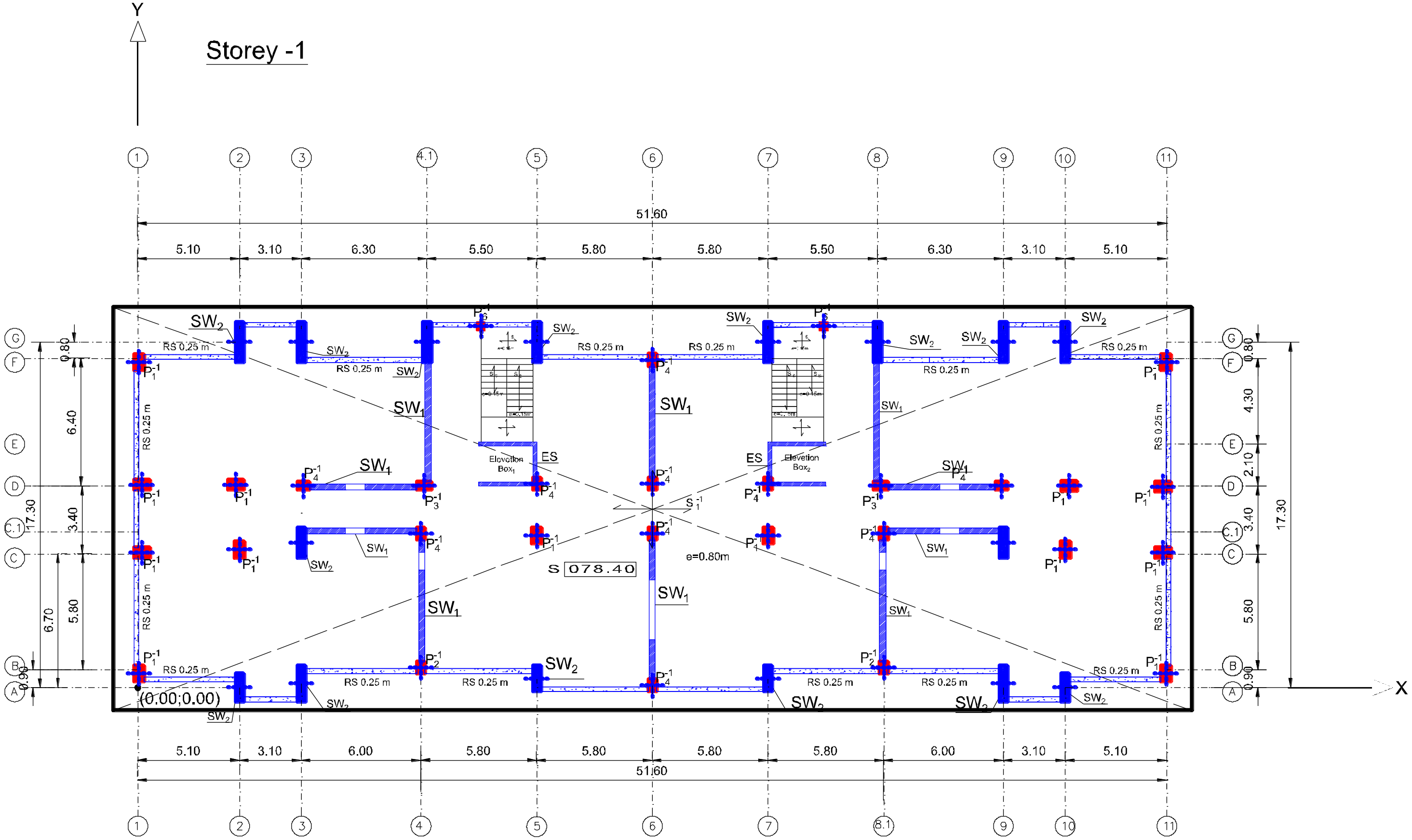


Figure B.0.1: Structural plan of basement storey

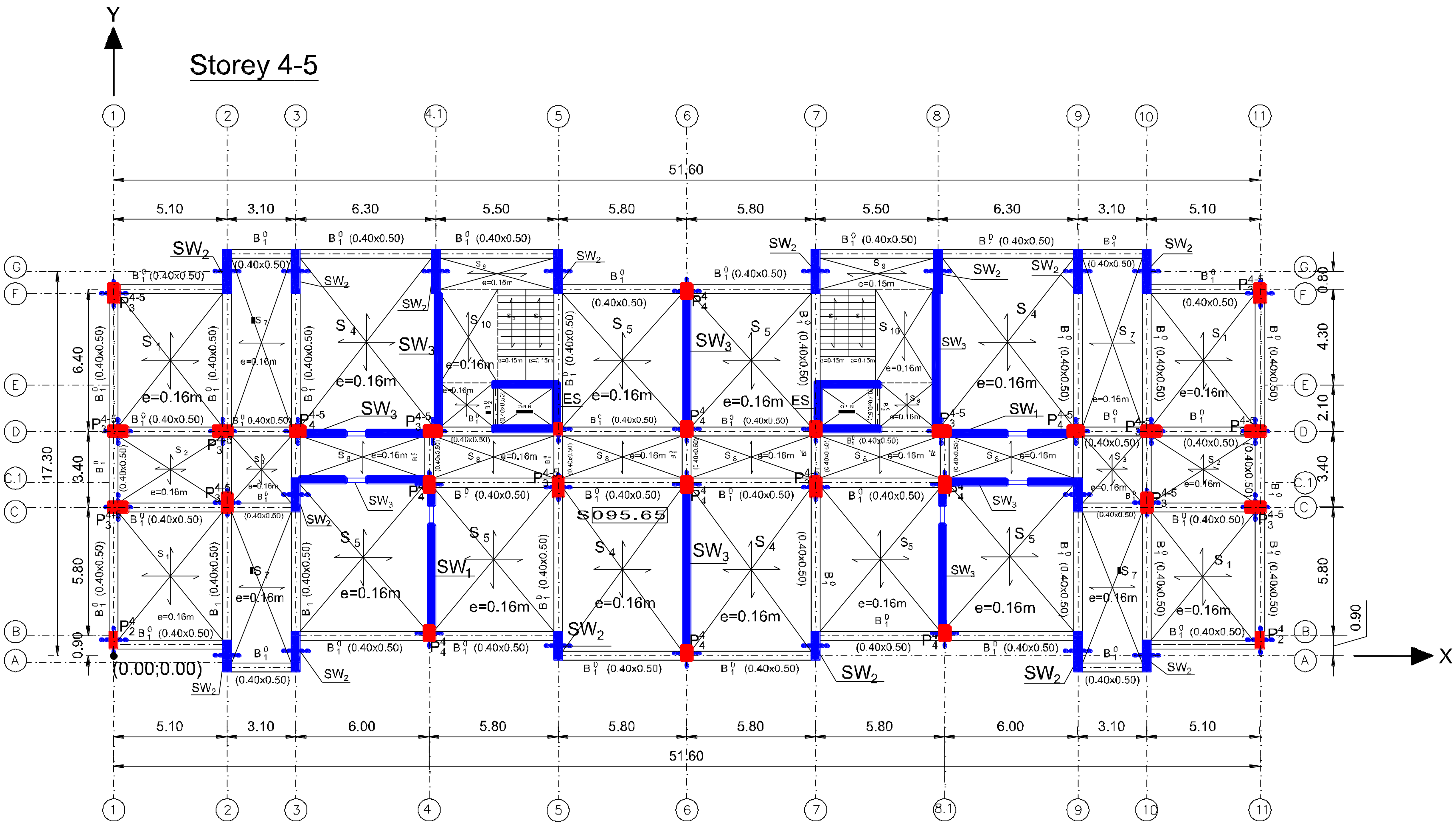


Figure B.0.2: Structural plan of 4th and 5th storeys

Appendix C

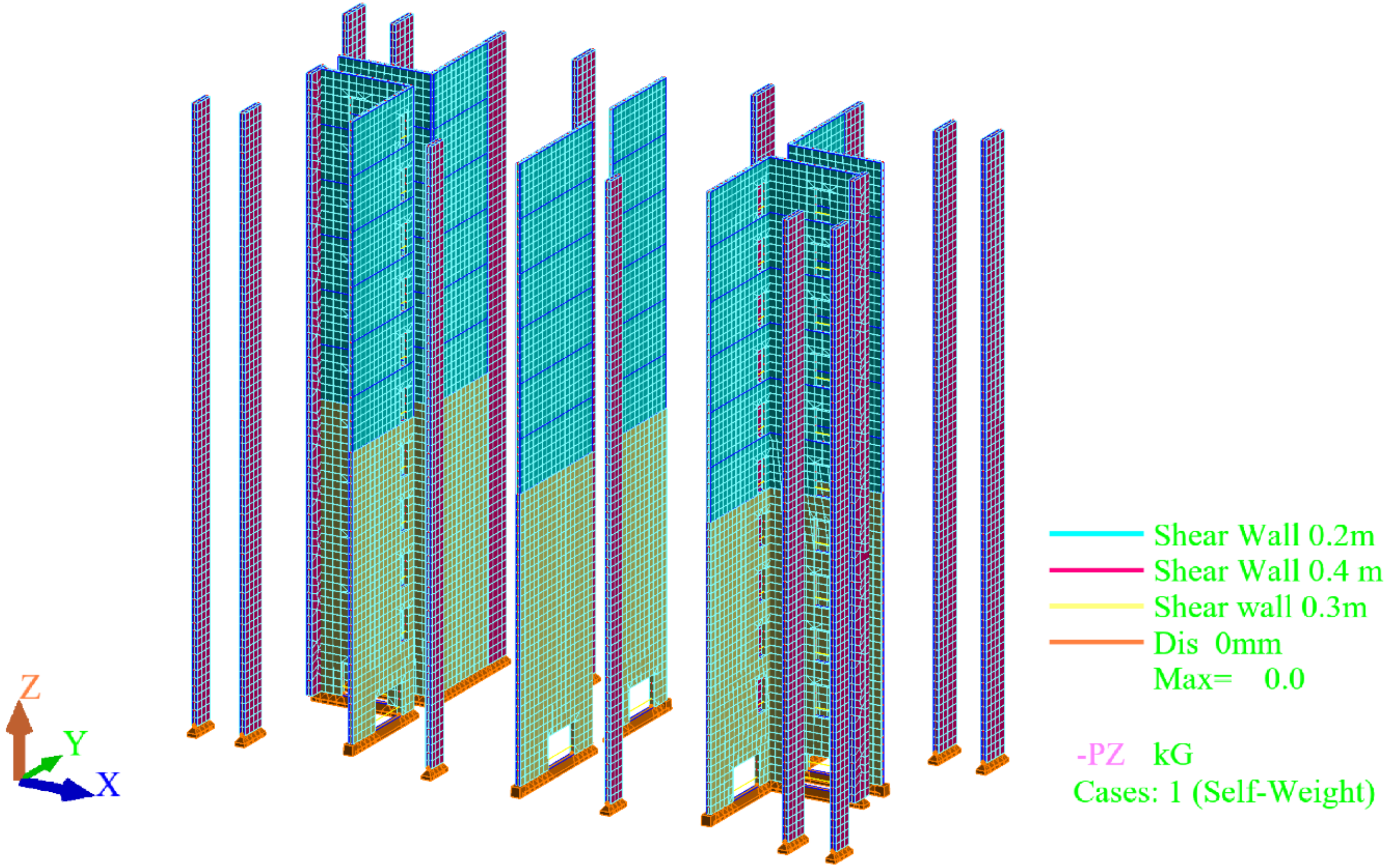


Figure C.0.1: Structure's shear walls

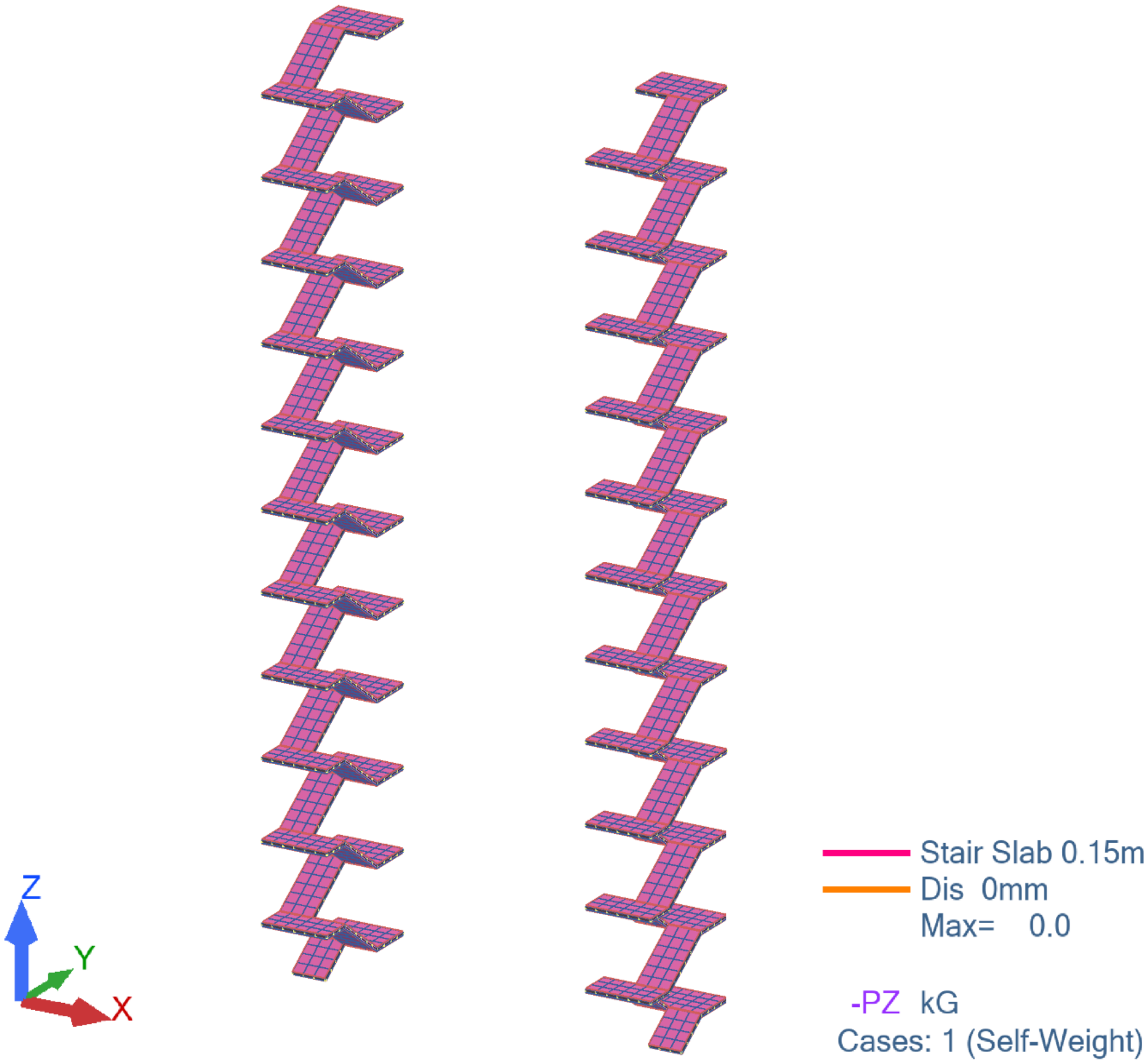


Figure C.0.2: Structure’s stairwell

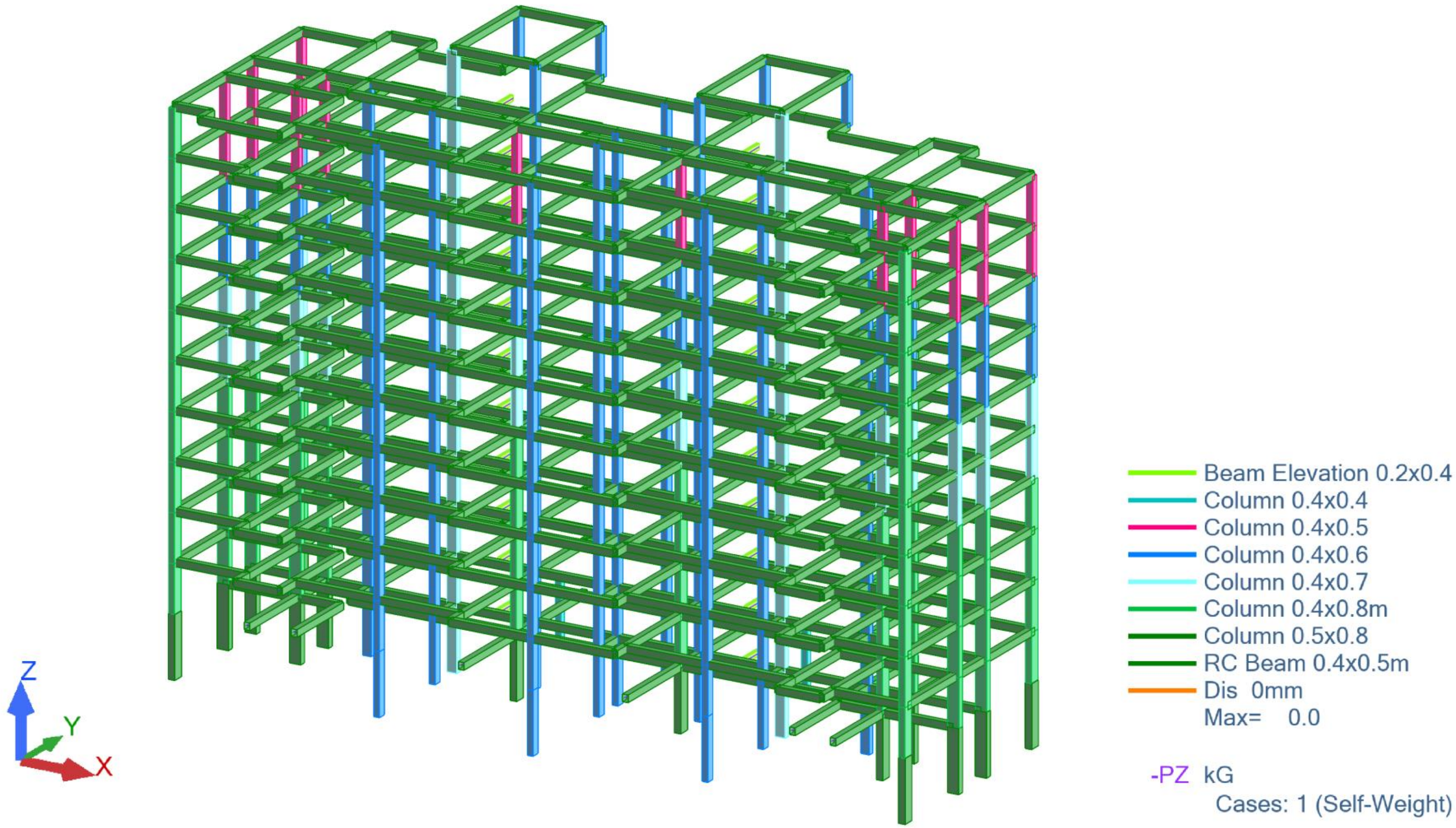


Figure C.0.3: Structure’s frame

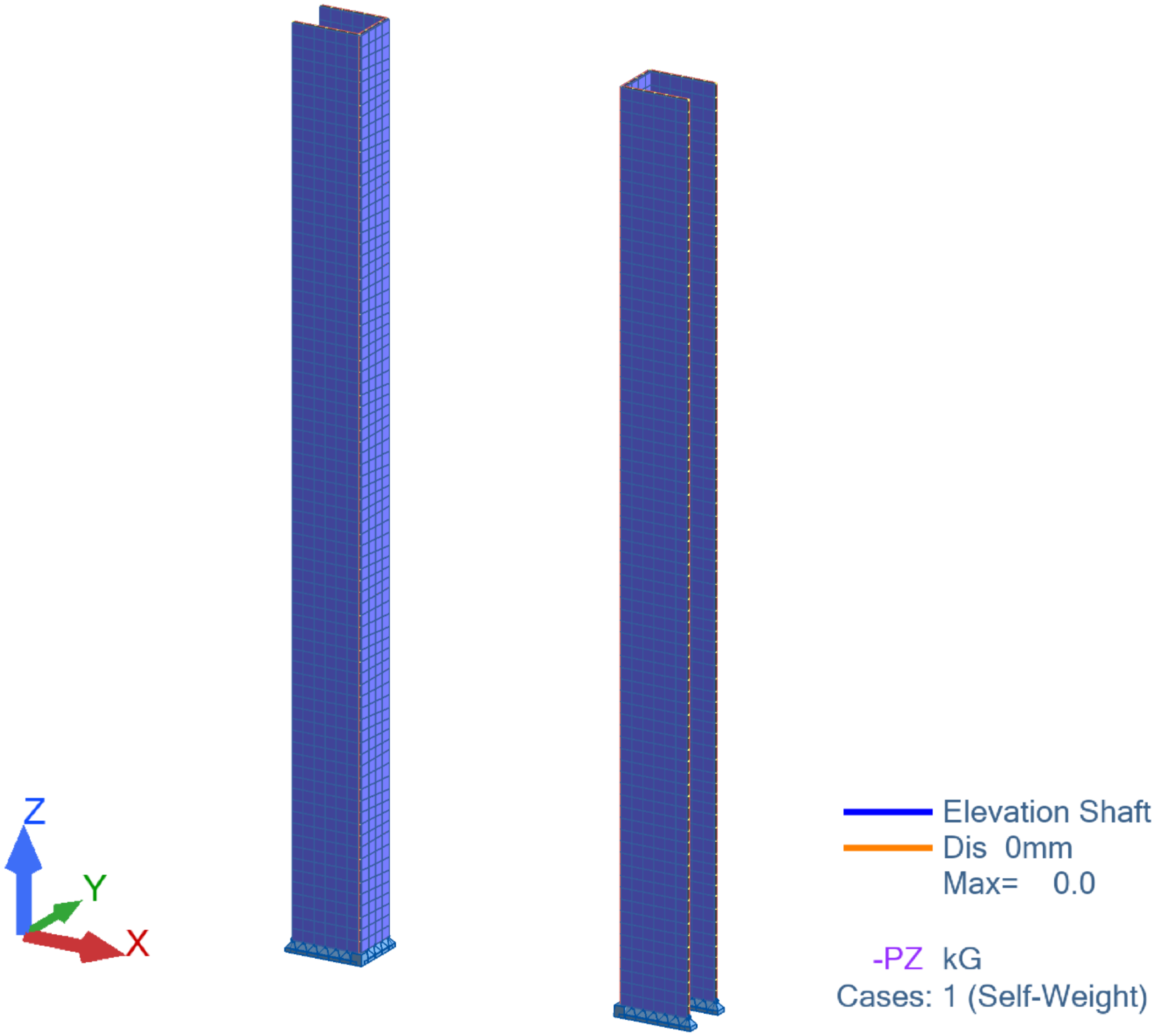


Figure C.0.4: Structure's elevation shaft

Appendix D

Table D.0.1: Partial factors on actions (γ_F)

Action	Symbol	Value
Permanent		
Unfovorable	$\gamma_{G;dst}$	1,1
Favorable	$\gamma_{G;stb}$	0,9
Variable		
Unfovourable	$\gamma_{Q;dst}$	1,5
Favourable	$\gamma_{Q;stb}$	0

Table D.0.2: Partial factors for soil parameters (γ_M)

Soil parameter	Symbol	Value
Angle of shearing resistance	$\gamma_{\varphi'}$	1,25
Effective cohesion	$\gamma_{c'}$	1,25

Undrained shear strength	γ_{cu}	1,4
Unconfined Strength	γ_{qu}	1,4
Weight density	γ_{γ}	1,0

Table D.0.3: Partial factors on actions (γ_F) or the effects of actions (γ_E)

Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

Table D.0.4: Partial factors for soil parameters (γ_M)

Soil parameter	**Symbol**	**Set**	
		M1	*M2*

Angle of shearing resistace	$\gamma_{\varphi'}$	1,0	1,25
Effective cohesion	$\gamma_{c'}$	1,0	1,25
Undrained shear strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Weight density	γ_{γ}	1,0	1,0

Table D.0.5: Partial resistance factors (γ_R) for spread foundations

Resistance	Symbol	Set		
		R1	R2	R3
Bearing	$\gamma_{R;v}$	1,0	1,4	1,0
Sliding	$\gamma_{R;v}$	1,0	1,1	1,0

Appendix E

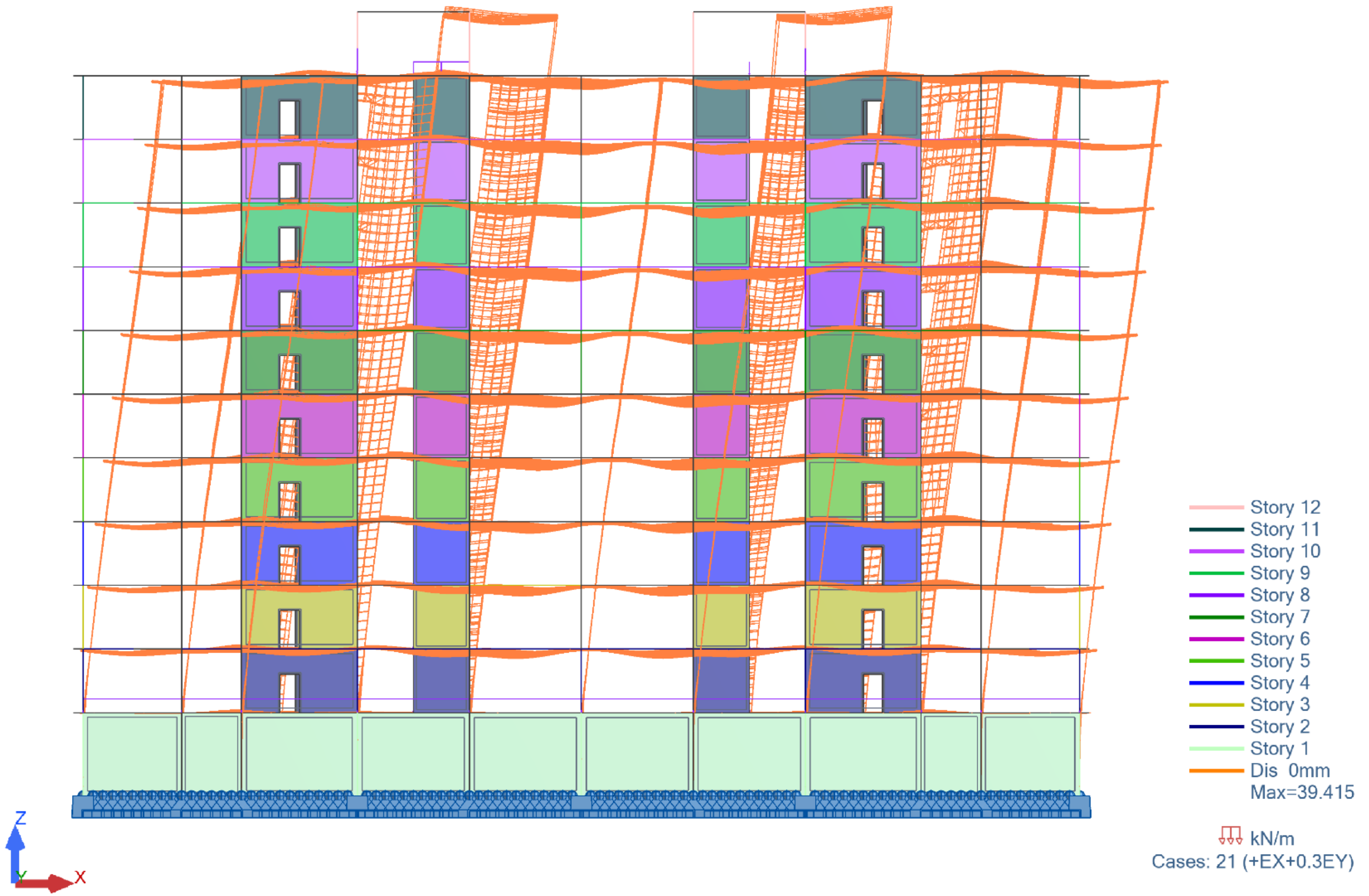


Figure E.1: Displacement shape under seismic combination 1 (see Table 4.1) for EN 1998-1

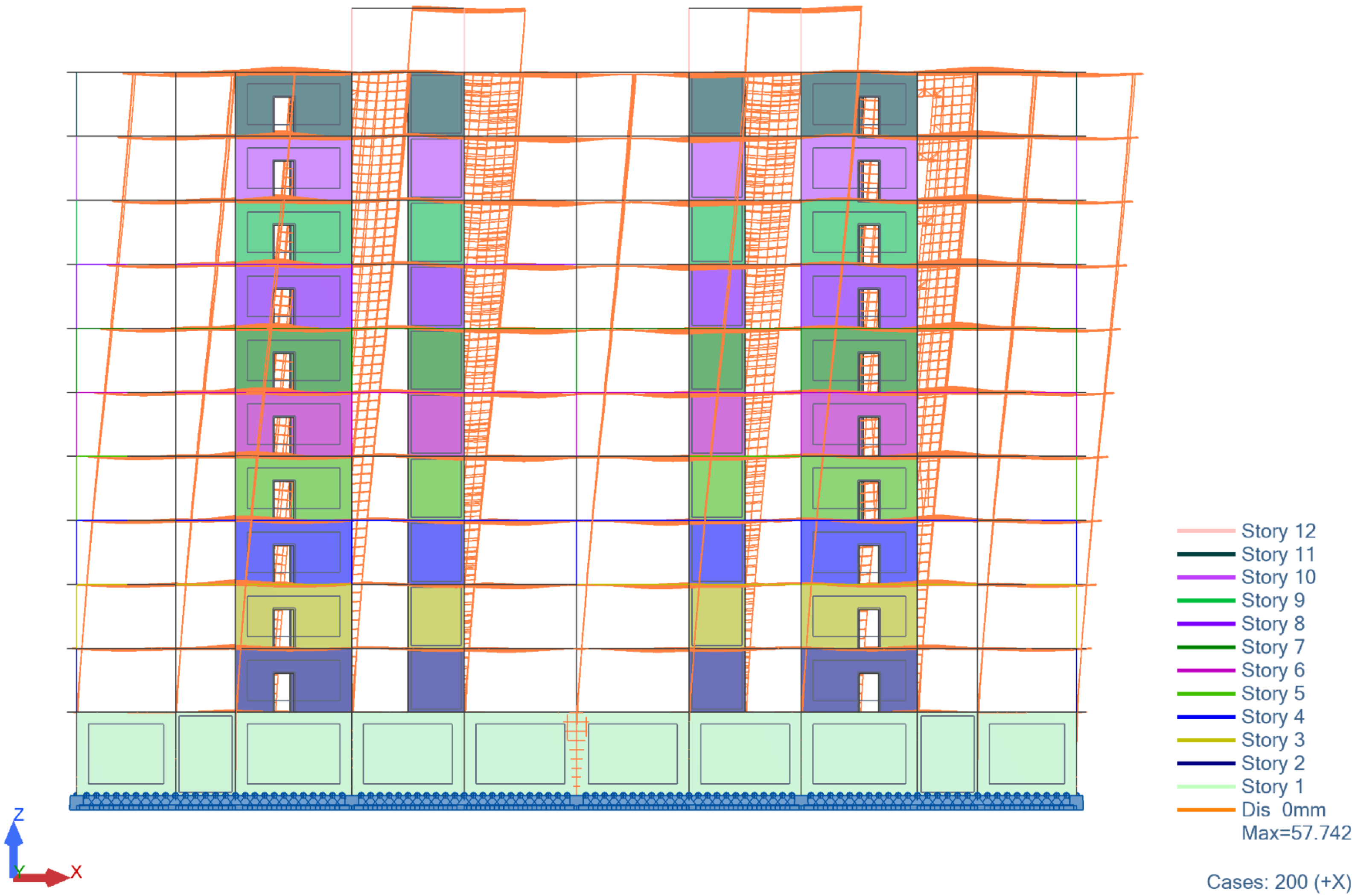


Figure E.2: Displacement shape under seismic combination 1 (see Table 4.1) for AzDTN 2.3-1

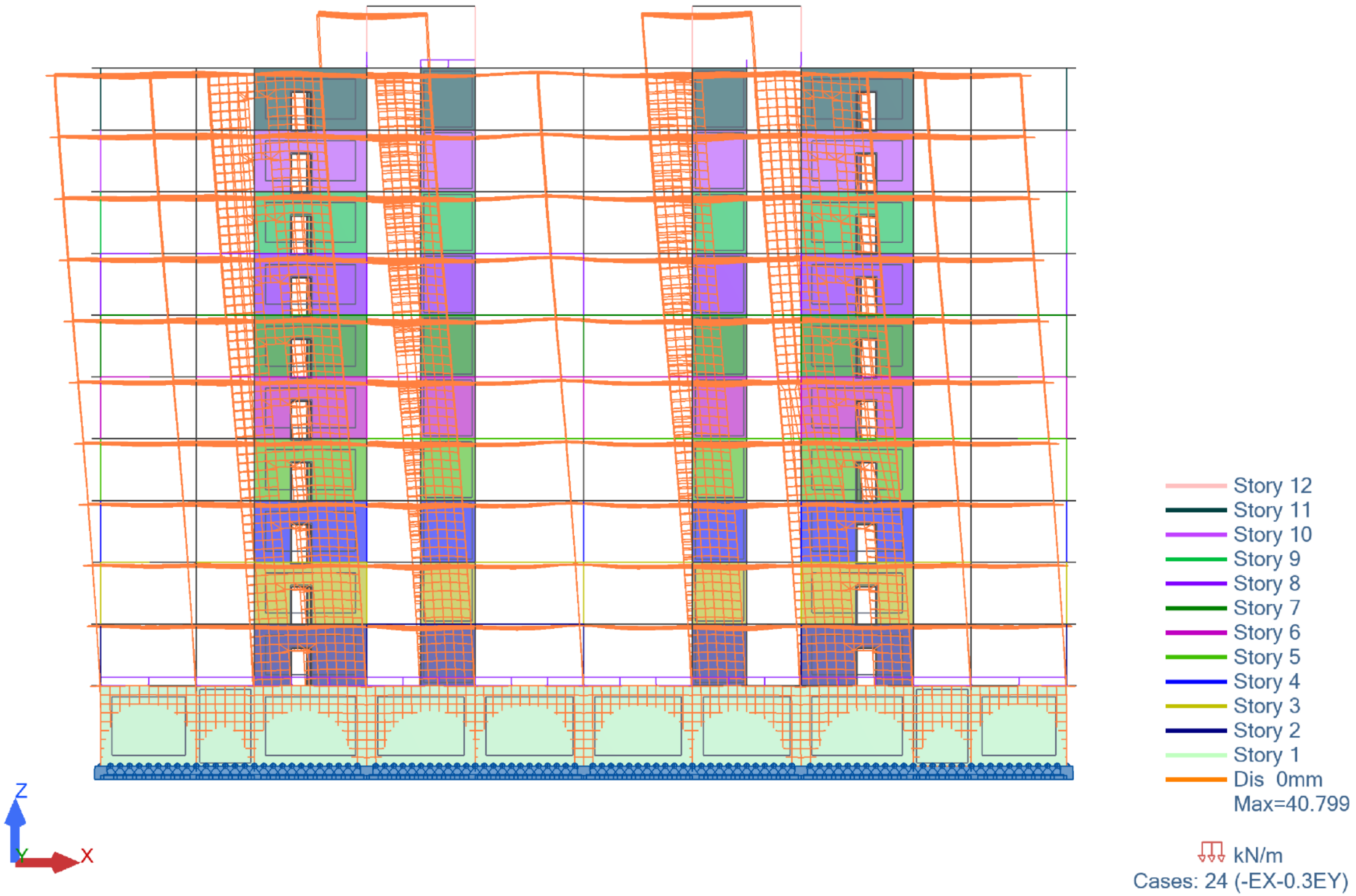


Figure E.3: Displacement shape under seismic combination 2 (see Table 4.1) for EN 1998-1

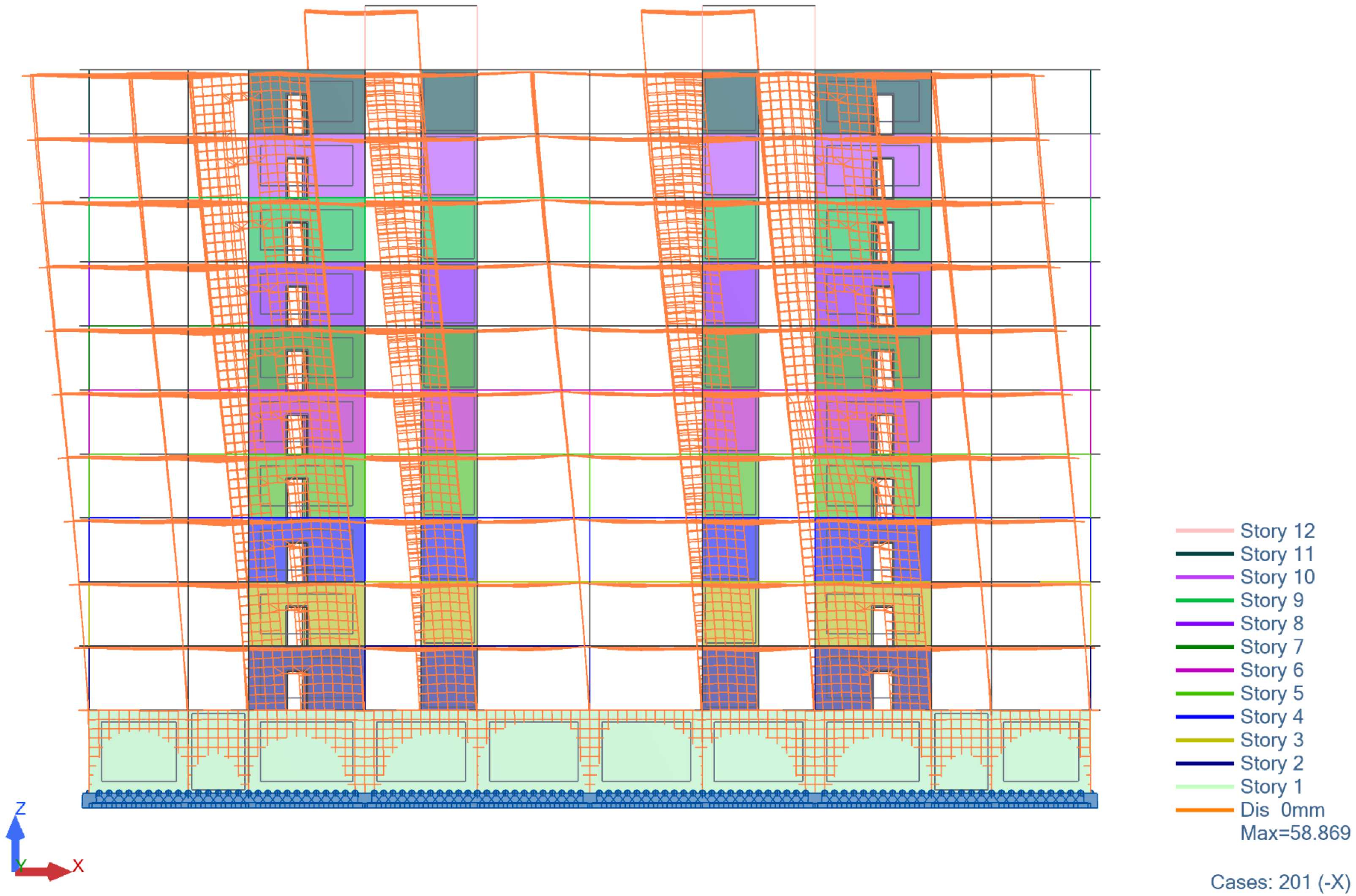


Figure E.4: Displacement shape under seismic combination 2 (see Table 4.1) for AzDTN 2.3-1

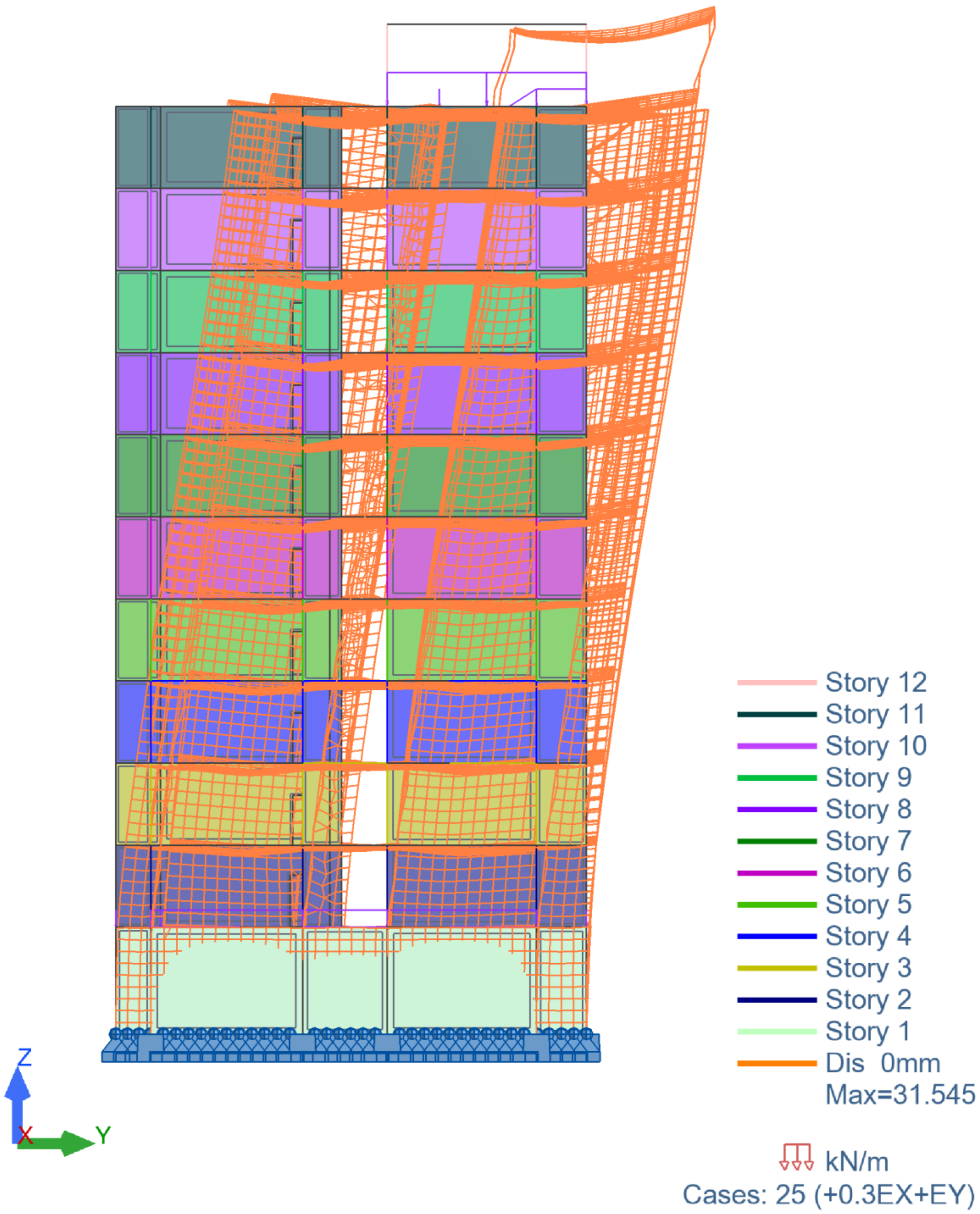


Figure E.5: Displacement shape under seismic combination 3 (see Table 4.1) for EN 1998- 1

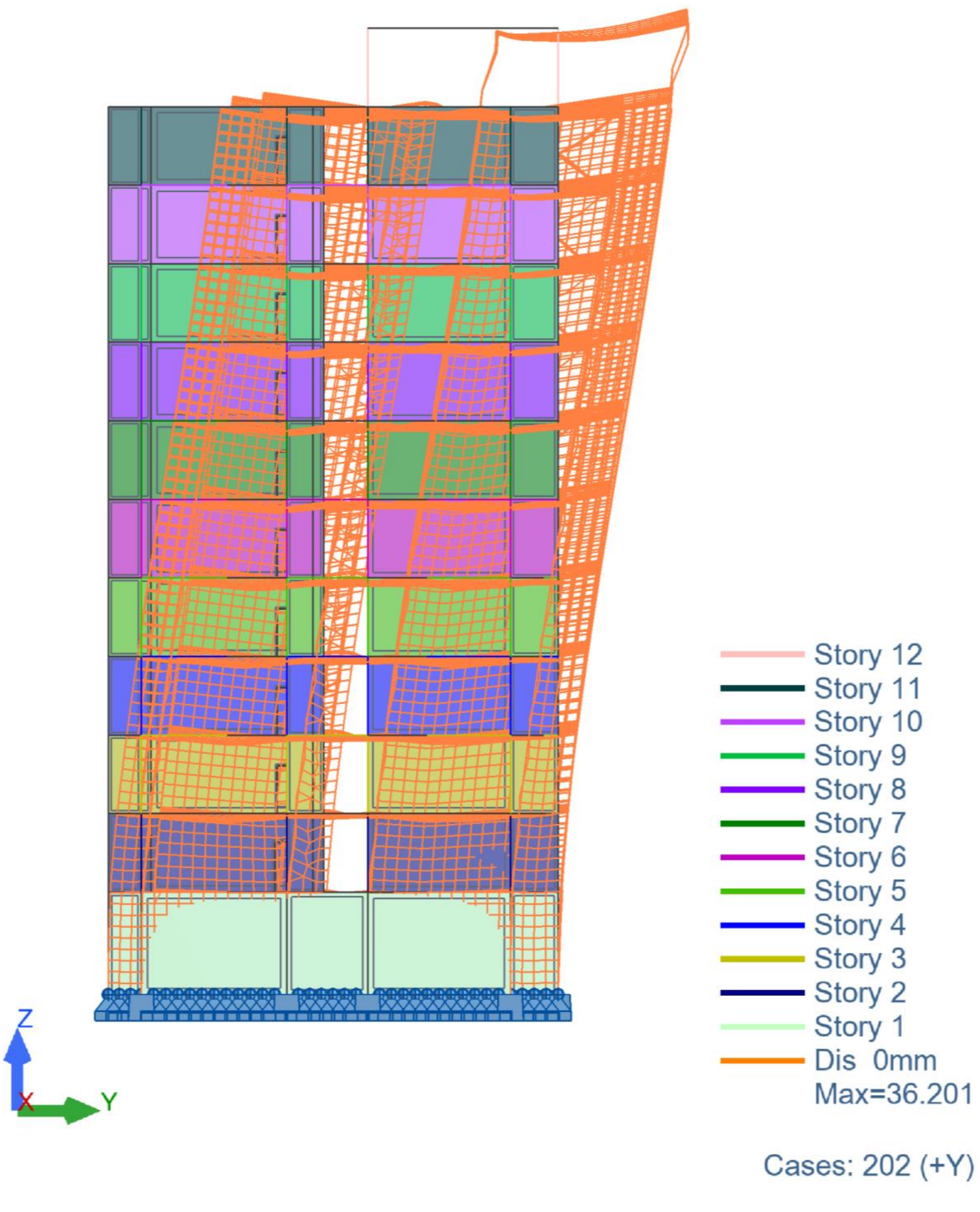


Figure E.0.6: Displacement shape under seismic combination 3 (see Table 4.1) for AzDTN 2.3-1

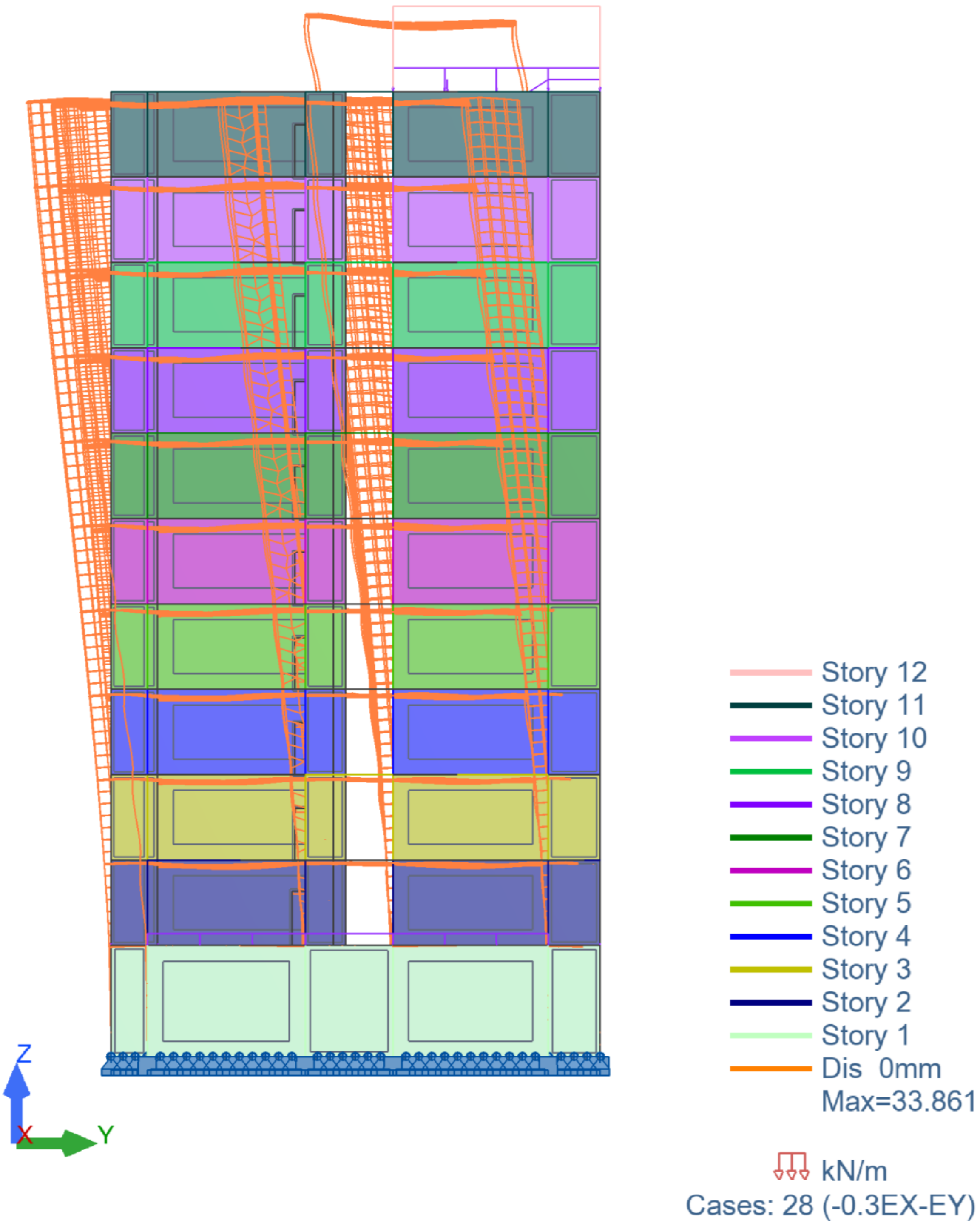


Figure E.7: Displacement shape under seismic combination 4 (see Table 4.1) for EN 1998- 1

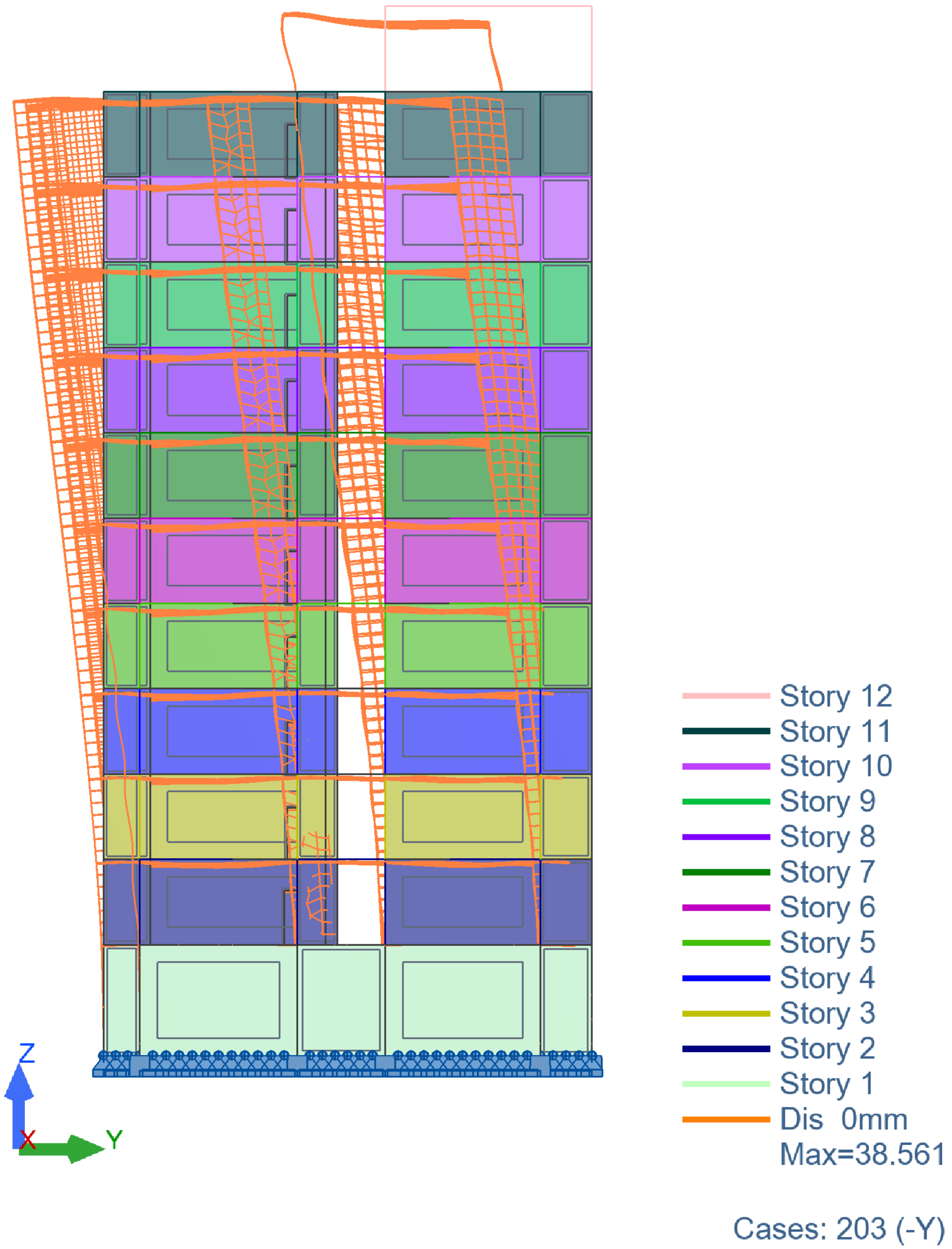


Figure E.8: Displacement shape under seismic combination 4 (see Table 4.1) for AzDTN 2.3-1