ARKIMADE

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TECHNICAL REPORT

FOR THE CONTRACT:
THE PREPARATION OF DESIGN AND SUPERVISION FOR THE REPAIR AND CONSOLIDATION WORKS OF HIGH UNIFIED SCHOOL "ISMET NANUSHI"

1st LOT

DURRES MUNICIPALITY

CLIENT **CONSULTANT**

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➢ ***Codes and References***

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12. *``Earthquake-Resistant Concrete Structures``, E&FN SPON (George G. Penelis, Andreas J. Kappos).*
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The project in question is calculated based on the linear state hypothesis of materials (concrete – reinforcing steel) and on the theory of small displacements, for constructions linear reinforced concrete.
During the unification of the physical construction with the static calculation model, the different elements of the construction "translate" into the Mechanical Model and the Loading Model, in such a way as to create and subsequently solve a Unique Mathematical Model.
The program used for the calculations is holoBIM10.

CONTENTS

1 MATERIALS

The strength of concrete foreseen in the project for the foundations (Section 1, Section 2 and Section 3 of the massive type with 60cm of height, meanwhile Section 4 with continuing beams of dimensions 150x40x50x120), is C25/30. For all other elements of the superstructure - columns, r/c walls, slabs and beams - is C30/37. Based on EC8, in the structures of mean ductility DCM, the primary seismic concrete elements cannot be of a strength inferior than C16/20.

The reinforcing steel used in the construction is brought from abroad and has a yield strength fy = 500 Mpa and tensile strength fy' = 5000kg/cm2. This type of reinforcing steel is foreseen for every case where reinforcement is to be used. Moreover, it must show satisfactory qualities in strength and deformation (ductility). In the primary seismic elements, as steel reinforcement will be used steel of the type B or C, as per table C1 in the normative annex C of Eurocode 2, EN 1992. With reference to Eurocodes, reinforcing bars must be of periodic type, that is to say having indentations instead of a smooth surface.
For the metallic cover of Section 4, is used steel S275.

The tensile strengths (used in the design) for concrete and steel reinforcement are taken from the reduction of characteristic strengths according to concrete (or steel) strength used with the relevant safety factor as below:

$$f_{cd} = f_{ck} / \gamma_c \qquad \text{(EC22.3.3.2)}$$

- *f_{cd} - calculatitg value of compressive strength in a cylindrical specimen*
- *fck - characteristic compressive strength of a cylindrical specimen after 28 days of curing.*
- *γ_c = partial factor of safety for concrete, equal of 1.5 (EC22.3.3.2)*

$$f_{ctm} = f_{ctk} / \gamma_c \qquad \text{(EC22.3.3.2)}$$

- *f_{ctm} - Mean value of axial tensile strength of concrete*
- *f_{ctk} – characteristic axial tensile strength of concrete*
- *γ_c = partial factor of safety for concrete, equal of 1.5 (EC22.3.3.2)*

$$f_{yd} = f_{yk} / \gamma_s \qquad \text{(EC22.3.3.2)}$$

- f_{yd} *- calculating yield stress of normal steel reinforcement*
- f_{yk} *- characteristic yield stress of normal steel reinforcement*
- γ_s *= partial factor for steel reinforcement, which equals to 1.15, (EC22.3.3.2)*
-

$$f_{ywd} = f_{ywk} / \gamma_s \qquad \text{EC22.3.3.2)}$$

- f_{ywd} *- calculating yield stress of stirrups*
- f_{ywk} *- characteristic yield stress of normal steel reinforcement*
- γ_s *= partial factor for steel reinforcement, which equals to 1.15, (EC22.3.3.2)*

The used materials are tabulated below:

MATERIALS			
Columns Concrete Strength (Class):	C30/37	Wall Concrete Strength (Class):	C30/37
Slabs Concrete Strength:	C25/30	Concrete Beam Strength:	C30/37
Foundation Beams Concrete Strength:	C25/30	Concrete Footings Strength:	C25/30
Columns Main Steel Reinforcement Class:	B500C:	Wall Main Steel Reinforcement Class:	B500C
Slabs Main Steel Reinforcement Class:	B500C:	Beams Main Steel Reinforcement Class:	B500C
Foundation Beams Main Steel Reinf. Class:	B500C:	Concrete Footings Main Steel Reinf. Class:	B500C
Columns Stirrups Steel Reinforcement Class:	B500C	Wall Stirrups Steel Reinforcement Class:	B500C
Slabs Stirrups Steel Reinforcement Class:	B500C	Beams Stirrups Steel Reinforcement Class:	B500C
Foundation Beams Stirrups St. Reinf. Class:	B500C	Concrete Footings Stirrups St. Reinf. Class:	B500C

As far as the frame structures are concerned, reinforced concrete nodes are the most important elements. Such nodes must not deteriorate and stay intact in the event of potent earthquakes. The nodes are foreseen to avoid reaching the plastic state whether in beams or in columns. Therefore, during the concreting of beams, special care should be exercised in the mounting/fitting of the main steel bars in the vicinity of the node itself, strictly taking into account each and every detail of the project.

2 DESCRIPTION OF THE STRUCTURE

In seismic zones, the frame structures are designed in such a way that their comprising elements (beams, columns) be able to resist bending moments M and the rest of the internal forces (N, Q) that are generated during the action of the external loads. In normal conditions, the complex structures must be concieved in such a manner that their functioning could take place along the three axes. But in special cases, mostly in symetric buildings, in an approximation, their funcion could be seen as being along a single plane. If this is the case, we say that we are dealing with a planar complex structure. However, in every bearing complex structure, in order to withstand the seismic effect, a good enough spatial interaction of its structural elements should be achieved.

To carry out a construction project, presupposing a reasonable seismic safety, design criteria are applied such that, for any expected situation, the building is guaranteed acceptable seismic responses. Depending on the intensity of the earthquake considered, these reactions are differentiated between them. The earthquake can be moderate, ie not strong (with high probability of falling), but more important is the evaluation of a possible strong and very strong earthquake (with high probability of falling). In accordance with the intensity of the earthquakes, the so-called "Basic Requirements" are defined as well as the corresponding design criteria and the respective boundary conditions. So, between them we distinguish:

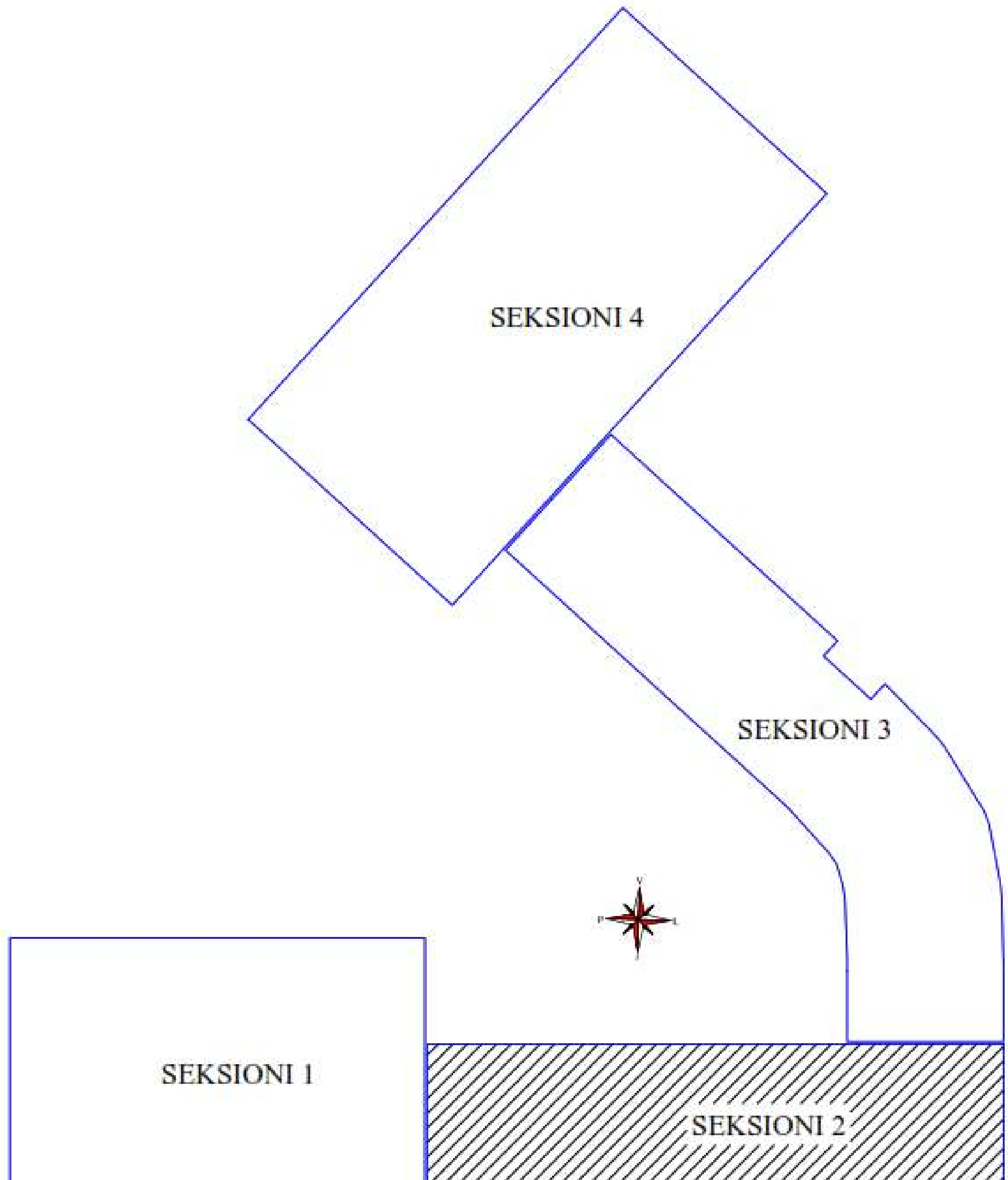
1) Request for limitation of damages

As a design criterion that responds to this requirement would be the coping with "moderate" earthquakes, that is not strong and relatively frequent, in such a way that only some very limited deformations and damages are allowed, which do not compromise the specific requirements of the building function. The design that refers to the above criteria is known as "Design according to the boundary condition of use".

2) Non-demolition requirement

As a design criterion for this requirement is this: to be able to withstand a strong and relatively rare earthquake, which can occur during the life of the building, in such a way that there is no such structural damage that causes collapse, local destruction or global (collapse) of the building which would be dangerous for the safety of people. Adequate seismic design must ensure that, after the earthquake, the structure still retains a structural integrity and considerable bearing capacity.

The design that refers to the above criteria is known as "Design according to the ultimate limit state".



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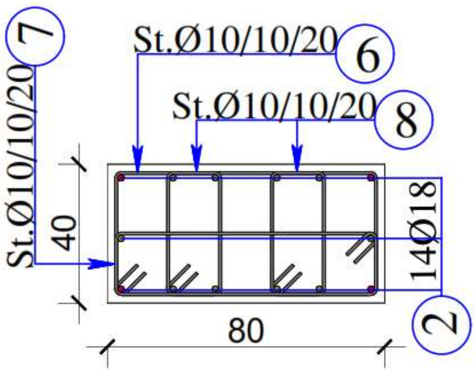


The columns have a cross-sectional shape mainly square and rectangular with different dimensions, generally (40x80) and (40x60). The variable section according to the height is used only in three columns located on the inner axes and are also reflected in the drawings. Jointing of column bars will be done at different levels according to the drawings. The distribution of the walls is done in such a way that we do not have significant distortion in the structure. So we have approximated the center of gravity of the structure to the center of its rigidity. In addition to the requirement of resistance to external loads according to the first boundary condition for the calculation of internal forces M, Q, N of horizontal elements, the control was performed according to the second boundary condition. This control (deformation control is done automatically by the program and all displacements result within the limit according to Eurocode. Thus, $[U_k] = 0.0033 \cdot h$.

The slabs are of two-way type, with thickness t = 30 cm. Their selection is aimed at a better distribution of loads acting on them, through the beams of the building and to better ensure their role as a horizontal diaphragm. The width of the rib with travertine concrete is accepted 20 cm and the thickness of the slab 5 cm. As a lightening filler material are used bricks with specific weight δ> 700 daN / m3.

Coverage beams are selected of different types, with considerable and reduced height with different dimensions, mainly (30x60) and (60x30) in function of the spaces and the intersection. In the longitudinal and transverse direction (but not perimetrically) of the building are used mainly beams of reduced height. The placement of such beams in the building is conditioned by the architectural requirement to have a flat ceiling surface in all rooms and to allow the installation of heating - cooling, ventilation, etc. pipes.

In the calculation of the beams are placed the trapezoidal or triangular loads coming from the slabs as well as the uniform load coming from the walls. The brick masonry in the building is provided with a thickness of 12, 20 and 25 cm made with horizontal holes (lightened bricks).

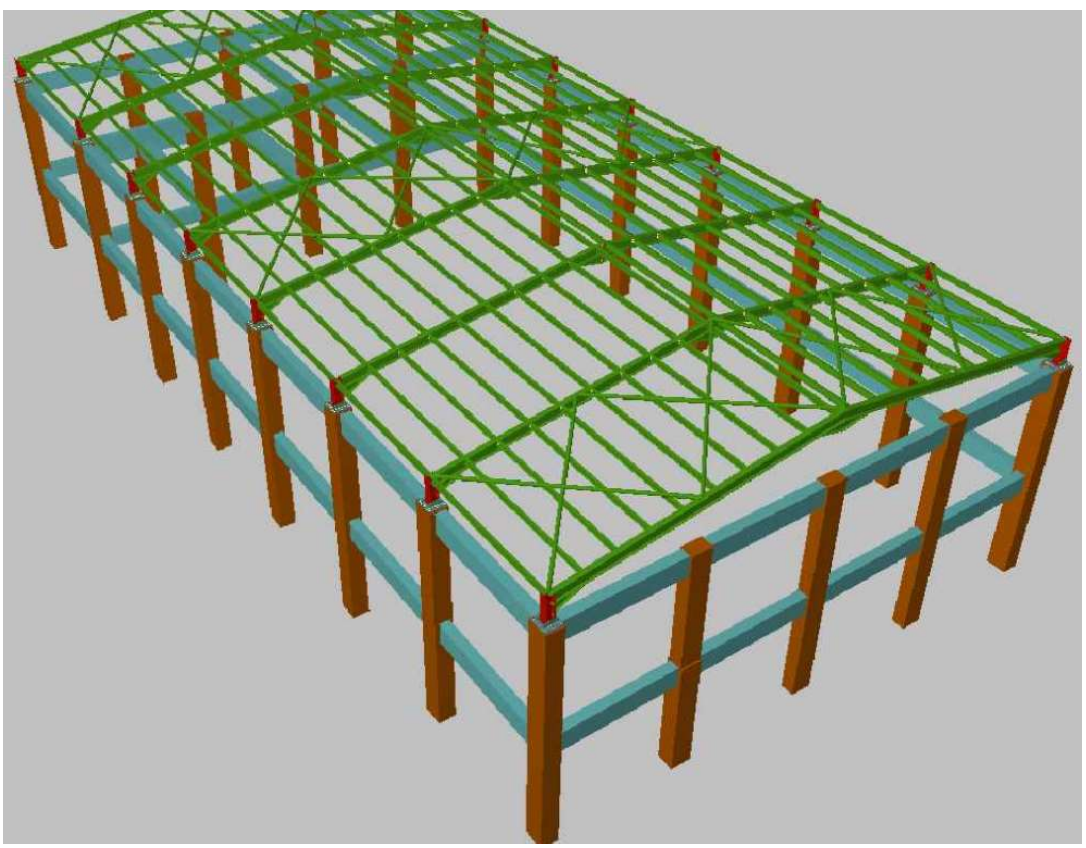
For the horizontal elements, in addition to the control of the first limit state, the control was performed according to the second condition. Generally the loading scheme for these elements is taken fixed connection to fixed connection, with distributed load q. The saggings are calculated using the following formula:

$$U = \frac{1}{384} \cdot \frac{q \cdot l^4}{E \cdot I} \leq [U] = \frac{l}{200}$$

Regarding Section 4, it is conceived with reinforced concrete columns and metal cover S275, made of metal beams and covered with sandwich panels.

3 THE ANALYSIS AND COMPUTER CALCULATION

Static and dynamic analysis to determine the response of the structure to different types of loading of the structure was performed with the program "holoBIM 10.".
The modeling of the structure as a whole and of each element is done on the basis of the finite element methodology (Finite Element Method - FEM).
Dynamic analysis is based on modal analysis with the reaction spectrum method.



The calculated dynamic (seismic) loads are accepted as equivalent static loads and are applied in place of concentrated masses. The basis for the method of dynamic calculations with the method of reaction spectrum is the analysis of its own values and its own vectors. By this method the forms of self-oscillations and the frequencies of free oscillations are determined. The values and vectors themselves undoubtedly give a clear and complete picture of the behavior of the structure under the action of dynamic loads. The program automatically searches for modes with lower circular frequencies (higher periods) as more contributing to the absorption of seismic loads from the structure. The maximum number of modes required by the program is conditioned by the constructor himself in n = 18 modes, while the floor measures of this building are considered with three degrees of freedom, in which 2 rotating and one translational according to the plan of the slab itself. The cyclic frequency f (cycles / sec), the circular frequency ω (rad / sec) and the period T (sec) are related to each other through the relations: $T = 1 / f$ and $f = \omega / 2\pi$. As a result of the analysis, the displacements, internal forces (M, Q, N,) and the stresses σ in each element of the structure are taken.

The analysis by the reaction spectrum method was performed using the modal superposition. (According to Wilson & Button 1982).

Geometric analysis is based on the geometric construction of the constituent elements of the structure, in planimetry and height.

Geometry in height. The application of dual walls (vertical diaphragms) are placed in such a way as to absorb horizontal loads (seismic loads, wind loads, etc.) and work as vertical brackets. The distribution of shear forces in wall-mounted systems as above becomes proportional to the moments of inertia of the transverse sections of the system walls, based on the same character of deformations and displacements of the component walls of the system. To frame the structures, whose flexibility is great (horizontal displacements are large), mixed systems with frames and walls are also used.

Dual systems are characterized by fundamental changes in the separation and deformation of their constituent components. Under the action of lateral loads, the global deformation of a frame in a dual system is similar to that of a vertical element working in shear, while the walls behave like sloping vertical consoles. The deformations of the frames are less in the upper floors, which is compensated by a greater displacement of the walls there. The forces of interaction that result in them give a favorable result, greatly modifying the inclinations of moments and shear forces, both in the walls and in the frames. In short, we say that the application of the walls in this building was done because:

1) reduction of deformations in the elements of the building

2) reducing the period of the structure as a whole

3) reduction of armaments and dimensions of the elements in general.

Geometry in planimetry. The application of the above walls has a logical sense of their placement in the building. Their placement should be done in such a way as to reduce translational and rotational displacements.

In the case of asymmetric structures, interactions between translational and rotational displacements are expected. Consequently, three-dimensional analysis becomes necessary for them, which must reflect the spatial interaction of vertical and horizontal structural elements. In these cases a special importance is given to the definition, for each level or floor, of the so-called "center of rigidity" or (Q. N.). This is the point where the resultant of the resistance forces of the structural elements of the respective floor passes, during the assumed horizontal seismic actions. Around this center, at the level under consideration, rotational motion or twisting of the structure may occur. Meanwhile, given the concept of seismic forces as inertial forces, it is understood that these forces are applied passing through the center of gravity of each floor (Q. G.). Physically it is also understandable that the discrepancy between the centers QG and QN would create a situation where for the floor under consideration, in addition to the seismic force acting on that floor, we will have the appearance of a torsional moment, which is taken as a product of seismic force with eccentricity between QG and QN.This phenomenon appears as an additional adverse effect on buildings that have structural irregularities. In practice it is attempted to avoid the twisting phenomenon or to reduce it as much as possible.

3D modeling of the structure was done, using the holoBIM 10 program. The structures were calculated according to the boundary condition, and the static scheme of the building was taken for Section 1 "Frame System" according to x-x axis and "Wall System" according to y-y axis, Section 2 and Section 3 " Wall system, while for Section 4 it is "Frame system".

The tiles are equated with equivalent rectangular tiles and are selected according to Czerny. The skeleton (ramat) is identified with joints, rods, rigid bodies and diaphragms, in space. Each node generally has 6 degrees of freedom, if the node belongs to the diaphragm it has 3 degrees of its own freedom and 3 degrees of freedom of the diaphragm. In the respective place of each column and beam, a '3d beam element' is created with the respective inertial characteristics, which is subject to deformations in

bending and cutting. The ends of the beam support on the columns are considered to be rigid bodies as long as the length of the beam support on the column.
The beams and foundations are aligned with a linear beam element on an elastic base. The frame in space, in general, rests flexibly on the foundation, thus producing, after the solution, the computational values and the stresses on the foundation.

4 CALCULATION LOADS

4.1 Dead Loads-DL

Permanent loads include: The weight of the permanent parts of buildings or works, including the part of supporting structures, coatings, fillers and partitions; soil weight and pressure (fills), mountain pressures, prestressing action on structures, weight of some parts of buildings or works, the position of which during the exploitation process, may change (eg partition walls that bear only their own weight). The permanent loads include: The own weight of all the supporting elements of the reinforced concrete structure (foundations, beams, columns, the own weight of the slats, floor layers, self-supporting partition walls with pointed bricks, and the railings of balconies, stairs, etc.).
The standardized loads that have been considered for the above structure, for all 4 Sections, are presented in the following table:

DEAD LOADS:					
Volumetric weight of concrete:	25.00	kN/m^3	Volumetric weight of brick partitions (single row):	2.10	kN/m^2
Brick Wall Load (double row):	3.60	kN/m^2	Terrace layers:	1.50	kN/m^2
Tile layers:	1.00	kN/m^2	Layers on the stairs:	1.30	kN/m^2
Volumetric weight of backfilling:	18.00	kN/m^3	Other dead loads:	0.00	

4.2 Live Loads-LL

Live Loads are divided in two categories:

1. *Live Loads with a prolonged period of action:*

The weight of stationary equipment, including the weight of their filling with solid or liquid material during the use of the work, the load on the floor of bookstores, archives, libraries and similar buildings or facilities ; prolonged action of temperature by stationary equipment; the weight of the water layer on the flat covers that are filled with water, the temporary loads on the residential and social buildings, where the weight of the equipment prevails or where there is a possibility of frequent gathering of people, etc.

2. *Live loads with short period of action:*
   Loads from mobile lifting-transport equipment (such as cable cars, cranes, etc.), which are used both during construction and during the use of buildings and works; loads in the residential or social buildings due to the weight of people, furniture and light equipment, the weight of people, details, repair materials in the service areas of equipment (entrances, spaces and all other parts that are free of equipment); snow load; wind load; climatic temperature actions; etc.

For each of the Sections, as temporary loads in the structure are calculated the loads of using the floors in the offices, stairs, balconies, terraces, etc., which in summary are also presented in the following table:

LIVE LOADS :					
Auditorium floorings:	2.00	kN/m2	Social building floorings:	2.00	kN/m2
Balcony floorings:	5.00	kN/m2	Secondary staircase floorings:	3.50	kN/m2
Service facilities floorings:	5.00	kN/m2	Primary staircase floorings:	5.00	kN/m2
Other live load 1:	0.00		Other live load 2:	0.00	

The above loads are nominal and depending on the combination for which the structure will be checked, permanent loads (DL) or temporary loads (LL) are multiplied by the relevant safety factor, according to the relevant design codes (conditions).

Loads that have been considered for the cover with metal construction:

- (**Dead Loads-DL) 0.2 kN/m2** (the cover is intended to be made with sandwich panels with a thickness of 4-5cm)
- (**Live Loads-LL) 0.6 kN/m2**

4.2.1 Modeling of the Loading

The distribution of loads on the edges of the slabs that rest on the beams of the structural walls, is done according to the law (1/3, 1/2 and 2/3) * φ (when φ = 90deg, the law becomes 30deg, 45deg, 60deg). The real load of each slab edge is equalized and added to the other loads of the beams, thus making up the total load of the beams. For dynamic analysis the mass of each plate is considered to be distributed at the level of the diaphragm. The beam mass is considered to be distributed either along the length of the beam or at the level of the diaphragm to which it belongs. The mass of the columns is considered to be distributed either in the upper and lower nodes or in the diaphragms belonging to the nodes of the element.

4.3 *Other Loads-OL*

Such loads are comprised of:
Seismic actions; actions of foundation subsidence caused by demolition of soil structure, soil compaction (reduction of fills), etc.

4.4 Earthquake Loads-EL

The area where the facility will be built is part of areas with an intensity of about 8 points according to the modified Merkel scale. The maximum acceleration of the base formation is obtained a = 0.26g;

4.4.1 Calculating the Coefficient of the Seismic Reaction *[EC8 §5.2.2.2]*

<u>Simbole:</u>

q coefficient of seismic reaction
q_o base value of seismic reaction coefficient
k_W the coefficient referring to the predominant mechanism of destruction in the system construction with structural walls
α_1 determines the formation of the first plastic hinge in the structure
α_u determines the limit of loss of overall stability.

4.4.2 Determining the Design Spectrum *[EC8 §3.2.2]*

<u>Symbols:</u>

α_{gR}	Ground acceleration (PGA)
□i	coefficient of importance
q	coefficient of seismic reaction
S	ground coefficient
T	the oscillation period of the oscillator by one degree
ξ	viscous quenching
□	Lower limit in the horizontal action spectrum of the calculation
$S_d(T)$	Design Spectrum
g	Gravitational acceleration

Let us give some notions (assumptions) on the theoretical basis of the calculation of buildings for seismic resistance.

- The seismic shape is evaluated in the so called 'ballë unit' according to the division of the 12-point system.
- The seismic force has any direction in space but during the calculation the seismic force is taken as acting horizontally.
- In the calculations of the resistance of constructions to seismic forces, the following is not taken into account: the influence of the dynamic action of the equipment, the lateral braking forces of the cranes and the inertial forces of the weights raised by the wing cranes
- Horizontal seismic load Sk where according to the construction calculation scheme the mass Qk is concentrated is determined by the formula:

$$S_k = Q_k \cdot K_c \cdot \beta \cdot m_k$$

where:

Q_k: vertical load that causes the force of inertia consisting of the weight of the structure itself, cranes, temporary weight such as snow load, etc. For the determination of Qk the weight of construction elements and cranes is taken according to the standardized loads, while the temporary loads are taken in full.

K_c: seismic coefficient equal to:

for 7 'ballë', 1/40,
per 8 'ballë', 1/20,
per 9 'ballë', 1/10.

β: the dynamic coefficient that depends on the period of free oscillations of the object and is determined by the formula:

$$\beta = \frac{0.9}{T}$$

- **Calculation Spectrum for horizontal seismic action:**

For horizontal results of seismic action, the calculation spectrum Sd (T) is determined by the following relations:

$0 \leq T < T_B$: $S_d(T) = \gamma i\, a_{gR} \cdot S \cdot (2/3 \cdot T/T_B \cdot (2.5/q - 2/3))$
$T_B \leq T < T_C$: $S_d(T) = \gamma i\, a_{gR} \cdot S \cdot 2.5/q$
$T_C \leq T < T_D$: $S_d(T) = \max(\gamma i\, a_{gR} \cdot S \cdot 2.5/q \cdot T_c/T, \beta \cdot \gamma i \cdot a_{gR})$
$T_D \leq T$: $S_d(T) = \max(\gamma i\, a_{gR} \cdot S \cdot 2.5/q \cdot T_c \cdot T_D/T^2, \beta \cdot \gamma i \cdot a_{gR})$
Where: β=0.20 is the recommended value according to E8

- **Calculation Spectrum for vertical seismic action:**

For the vertical results of seismic action, the calculation spectrum Sd (T) is determined from the above relations taking into account the following changes:
$a_{vg} = 0.90 \cdot a_g$
S = 1.00

Regarding the characteristics of the building, the frame type building made of reinforced concrete with filling walls, in the case when during the calculations the frame-wall interaction is taken into account, the formulas for calculating the period of the first tone of personal oscillations are determined by the formula:

$$T_1 = \frac{0.09 \cdot h}{\sqrt{b}}$$

where:
h: building height; b: planimetric dimension of the building along the direction of calculation.

m_k: coefficient that depends on the shape of the deformation of the structure during its free oscillations as well as the position of weight placement Qk.

- For calculations of ordinary buildings (industrial and social), only the first form of free oscillations is taken into account.
- Balconies, shelters above the ports, to a considerable extent compared to the building, are calculated as if the seismic forces acted vertically, taking the product of coefficients β equal to 5.

Detail of seismic load calculation according to the above approximations based on KTP. - N.2 - 89:
For the calculation of buildings and various engineering works with the reaction spectrum method, in the case of horizontal seismic operations, the calculated (design) values of the acceleration reaction spectrum Eki to be taken from the expression:

$$E_{ki} = k_E \cdot k_r \cdot \psi \cdot \beta_i \cdot \eta_i \cdot Q_k$$

where:

E_{ki}: horizontal seismic force, which is exerted at the point (level) "k" and corresponds to the tone "i" of its own oscillations
K_e: seismicity coefficient, e.g. for ground category B and terms with intensity 8 'ballë', Ke = 0.22

K_r: the coefficient of importance of the construction object, e.g. for works and buildings of special importance such as our case, Kr = 1.2 (3rd category)

Ψ: the reaction coefficient of the structure under seismic action, e.g. for reinforced concrete frame structures, when frame-wall interaction is not considered for:

h/b<15, ψ=0.25
h/b>25, ψ=0.38

15<h/b<25, by interpolation method, (h: column height, b: cross-secitonal dimension of the column along the direction of action of the seismic force).

β_i: the dynamic coefficient that is determined according to the following formulas as a function of the period Ti of the personal oscillations of the construction and the category of the land of the construction site.

When for the calculation of different constructions to the vertical seismic action is accepted the calculation scheme in the form of a horizontal rod with concentrated masses, the vertical seismic force, which is exerted at point "k" and which corresponds to the tone "i" of the self-oscillations, is calculated at the same way provided that the value of the coefficient βi is multiplied by the coefficient 2/3.

- For grounds of first category:

$$0.65 \le \beta_i = \frac{0.7}{T_i} \le 2.3$$

- For grounds of second category:

$$0.65 \le \beta_i = \frac{0.8}{T_i} \le 2.0$$

- For grounds of third category:

$$0.65 \le \beta_i = \frac{1.1}{T_i} \le 1.7$$

η_{ki}: is the coefficient of distribution of the seismic load of the calculation, which corresponds to the form "i" of the personal oscillations of the structure at the point (level) "k" which simplified is calculated by the formula:

$\eta_k = \frac{3 \cdot k}{2 \cdot n + 1}$, (where: 'k' is the relevant level (floor), n: number of the floors in the building).

Q_k: is the weight of the part of the building or engineering work that is concentrated in point (level) "k" and that is determined on the basis of calculated loads (permanent and temporary) reduced by the coefficients of combination as follows: DL: 0.9; LL1 with long action: 0.8; Short-acting LL2: 0.4.

5 LOAD COMBINATIONS

The calculation of building structures is carried out by taking into account the possible most unfavorable combinations of loads for individual elements, as well as for the building as a whole, which can act at the same time during serviceability period or even during construction.

As far as the combination of loads is concerned, according to KTP. 6 – 1978 there are three groups:

1. The Main Combination.
2. The Extra Combination.
3. The Specific Combination.

Lets take a closer look at each one of them:

I. The main combination is comprised of:

1) Dead loads;
2) Live loads acting for a prolonged period of time;
3) Live loads acting for a short period of time, such as:

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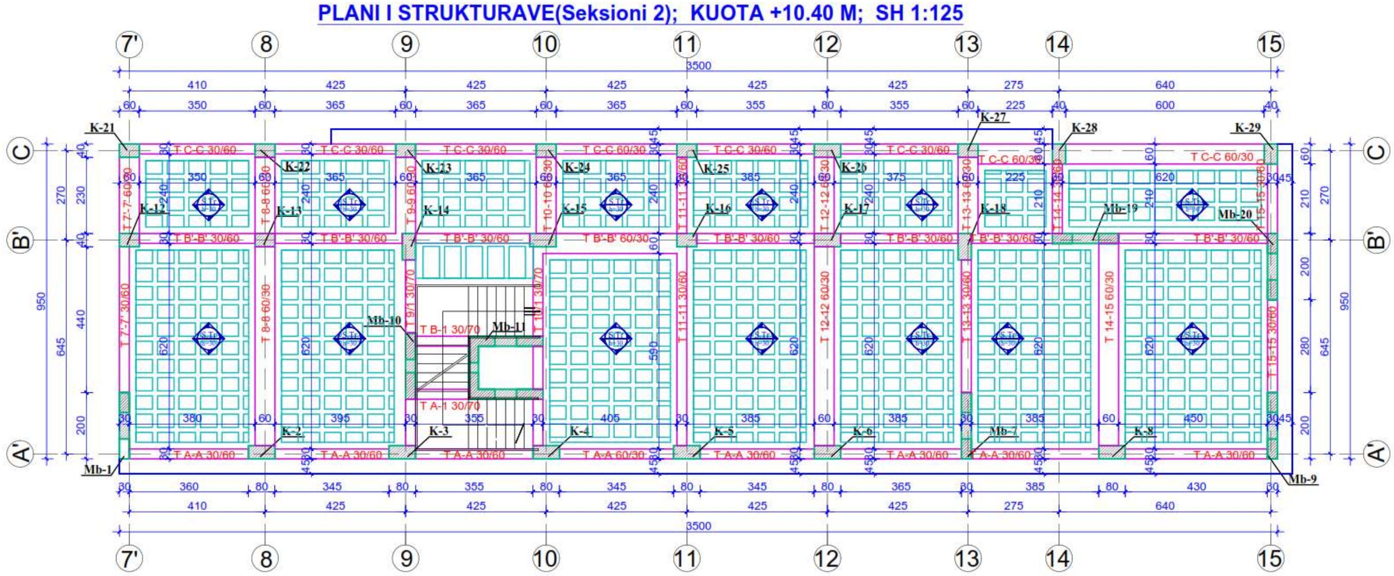


a) Loads from the mobile equipment for lifting-transporting (cable car, elevator, crane, etc.) which are used both during the construction and the serviceablilty phase of buildings and works.

b) Intermediate loads in residential or social buildings due to the weight of people, furniture and light equipment;
c) Snow load;
d) Loads during transport and assembly of construction structures.

II. The extra combinations are comprised of:

1) Dead loads;
2) Live loads acting for a prolonged period of time;
3) All the live loads acting for a short period of time, for no less than two of them. Such loads are to be multiplied by the factor of 0,9.

III. The specific combinations are comprised of:

1) Dead loads;
2) Live loads acting for a prolonged period of time;
3) Live loads acting for a short period of time;
4) One of the specific loads. In this case, live loads acting for a short period of time, are to be multiplied by the factor 0,8.

The determination of the bearing capacity of our structure (ULS) is performed by combining the acting loads in the structure according to the Eurocode combinations as follows:

A 1.30G + 1.50Q	
1B 1.00G + 0.30Q + 1.00Ex+eccy + 0.30Ey+eccx	1C 1.00G + 0.30Q + 1.00Ex+eccy - 0.30Ey+eccx
1D 1.00G + 0.30Q + 0.30Ex+eccy + 1.00Ey+eccx	1E 1.00G + 0.30Q - 0.30Ex+eccy + 1.00Ey+eccx
1F 1.00G + 0.30Q - 1.00Ex+eccy - 0.30Ey+eccx	1G 1.00G + 0.30Q - 1.00Ex+eccy + 0.30Ey+eccx
1H 1.00G + 0.30Q - 0.30Ex+eccy - 1.00Ey+eccx	1I 1.00G + 0.30Q + 0.30Ex+eccy - 1.00Ey+eccx
2B 1.00G + 0.30Q + 1.00Ex-eccy + 0.30Ey+eccx	2C 1.00G + 0.30Q + 1.00Ex-eccy - 0.30Ey+eccx
2D 1.00G + 0.30Q + 0.30Ex-eccy + 1.00Ey+eccx	2E 1.00G + 0.30Q - 0.30Ex-eccy + 1.00Ey+eccx
2F 1.00G + 0.30Q - 1.00Ex-eccy - 0.30Ey+eccx	2G 1.00G + 0.30Q - 1.00Ex-eccy + 0.30Ey+eccx
2H 1.00G + 0.30Q - 0.30Ex-eccy - 1.00Ey+eccx	2I 1.00G + 0.30Q + 0.30Ex-eccy - 1.00Ey+eccx
3B 1.00G + 0.30Q + 1.00Ex+eccy + 0.30Ey-eccx	3C 1.00G + 0.30Q + 1.00Ex+eccy - 0.30Ey-eccx
3D 1.00G + 0.30Q + 0.30Ex+eccy + 1.00Ey-eccx	3E 1.00G + 0.30Q - 0.30Ex+eccy + 1.00Ey-eccx
3F 1.00G + 0.30Q - 1.00Ex+eccy - 0.30Ey-eccx	3G 1.00G + 0.30Q - 1.00Ex+eccy + 0.30Ey-eccx
3H 1.00G + 0.30Q - 0.30Ex+eccy - 1.00Ey-eccx	3I 1.00G + 0.30Q + 0.30Ex+eccy - 1.00Ey-eccx
4B 1.00G + 0.30Q + 1.00Ex-eccy + 0.30Ey-eccx	4C 1.00G + 0.30Q + 1.00Ex-eccy - 0.30Ey-eccx
4D 1.00G + 0.30Q + 0.30Ex-eccy + 1.00Ey-eccx	4E 1.00G + 0.30Q - 0.30Ex-eccy + 1.00Ey-eccx
4F 1.00G + 0.30Q - 1.00Ex-eccy - 0.30Ey-eccx	4G 1.00G + 0.30Q - 1.00Ex-eccy + 0.30Ey-eccx
4H 1.00G + 0.30Q - 0.30Ex-eccy - 1.00Ey-eccx	4I 1.00G + 0.30Q + 0.30Ex-eccy - 1.00Ey-eccx

The elements of the structure are also checked in accordance with the allowable deformations caused in them by the action of normative loads. In these combinations the load combination coefficients are accepted equal to 1 unit.

6 NUMERICAL MODELING – CALCULATION METHODS

Based on the local matrices of the structural elements - after their conversion in the unique coordinate system has been done - the formulation of the general matrix of inertia of the structure is done. In parallel, the matrices of measures and the matrices of acting forces are formulated for each combination of loads.

The program initially calculates the structure according to G and Q loads in order to create any possible combination of G and Q loads, e.g. $\gamma gG + \gamma qQ$ and $1.0G + \psi 2Q$. Continuing with 1 + 4 positions of the mass (1 in its physical position and 4 displaced in the eccentricities of the case ± eccx and ± eccy) performs 1 + 4 analysis and calculates 1 + 4 additions of idiomorphic displacement values. The overlap of idiomorphs is done according to the law of complete quadratic overlap, CQC.

Verifications

In addition to the usual controls, the following controls are performed:

a. Control according to the superconducting capacity in bending and according to the shear force
b. Column rigidity control.
c. Avoidance control of plastic hinges in columns.
d. Control according to the boundary condition of the foundations
e. Characterization of structural walls
f. Control of short columns

7 DETERMINING THE DEFORMATIONS AND SEISMICAL VERIFICATION

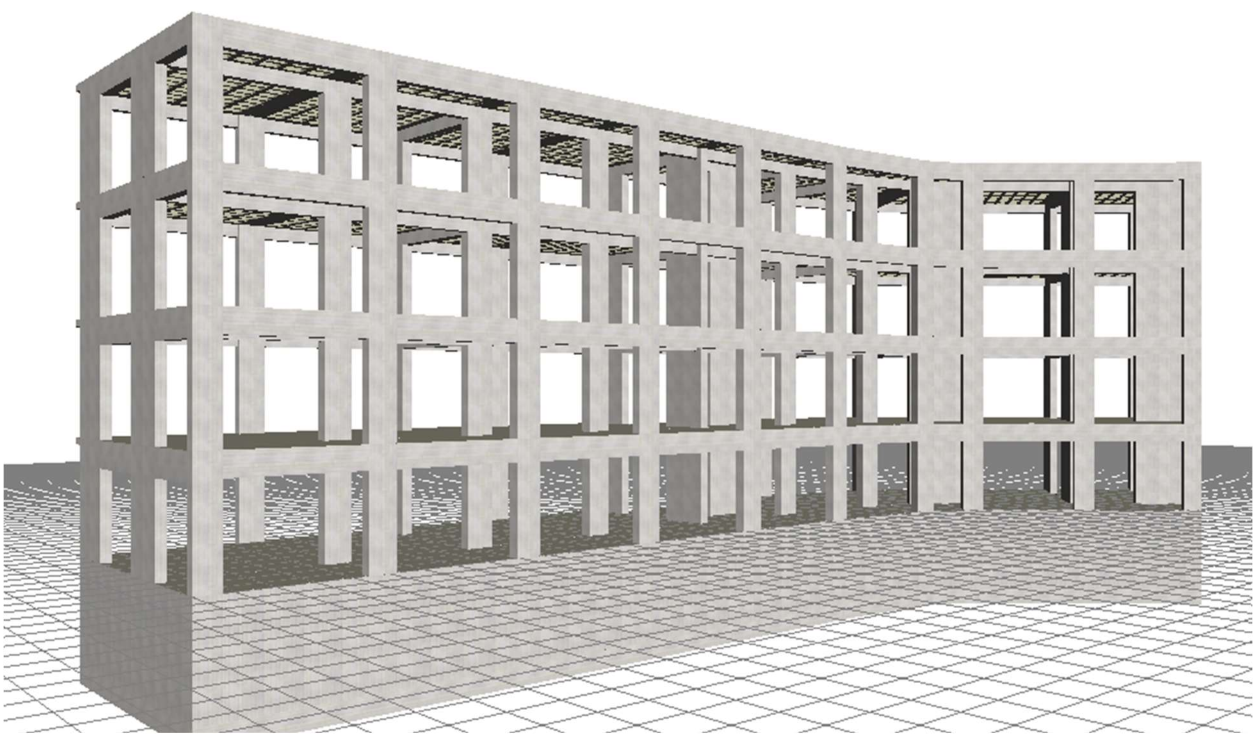
Traditionally, the measurement and construction of structural elements have relied on the construction as the primary partition. According to this, deformations or displacements of structural elements are considered as secondary to the role of loads, however, even for displacements controls are applied to satisfy certain conditions. Passing as the primary design criterion from cracks and resistances to deformations and displacements,

it is argued with the indisputable fact that structural damage during an earthquake is related, first of all, to imposed deformations.

To determine the deformations due to seismic action, for the form "i" of the self-oscillations, the Ukiel elastic displacement of the point "k" of a structure, can be determined:

$$U_{ki}^{el} = k_E \cdot k_r \cdot \psi \cdot \beta_i \cdot \eta_{ki} \cdot g \cdot \left(\frac{T_i}{2 \cdot \pi} \right)^2$$

For other calculation schemes the determination of Ukiel is done in accordance with the seismic loads corresponding to those schemes. Ukiel computational elastic displacements are determined by making the combination for different forms of oscillations.



The total maximum displacement Uk that the point "k" of a structure undergoes due to seismic action, taking into account the effect of plastic deformations, can be determined approximately by the formula:

$$U_k = \frac{U_k^{el}}{\psi}$$

For different buildings (engineering works), the maximum total displacements are limited based on the functional requirements of those buildings (engineering works). The displacement of the interlayer (drift) according to the two directions of excitation of the structure have resulted within the limits defined in EC8 for structures, the non-structural elements of which will not be ductile. For these structures the allowable limit for interstitial displacements results in the order 0.0033. The maximum allowable displacement is approx:

$$\left[U_k\right] = 0.0033 \cdot h$$

where: 'h' is the floor height of which the displacement is calculated.

The elastic behavior spectrum for the horizontal oscillation of the ground is determined according to KTP. N.2 - 89 for lands of the second category where the dynamic coefficient β is taken 0.65 ≤β = 0.8 / T1 ≤ 2. In accordance with the recommendations of KTP. N.2 - 89, for vertical oscillations βv = 2 / 3β is accepted.
The calculation spectrum is obtained by factorizing the elastic behavior spectrum with factors that take into account the dynamic reaction of the structure. These spectral scaling factors from the calculations have resulted (formulas 3.8-3.11 EK8):

7.1 Verification of the Displacements and Drifts

- **Displacement Calculation**

The effect of accidental torsion is included in the calculation of the building being automatically incorporated into the level of seismic forces.
From the calculations, the maximum displacements of the joints for each section, according to both directions of excitation have resulted:

1. <u>SECTION 1</u>

- **Displacements**

Max elastic deformation along direction x-x: sx,max = 21.52mm
Max absolute deformation along direction x-x: 2.88*21.52 = 61.98mm
Displacement along direction x-x = 6.2 cm
Max elastic deformation along direction y-y: sy,max = 26.00mm
Max absolute deformation along direction y-y: 2.88*26.00= 74.87mm
Displacement along direction y-y = 7.5cm.

- **Relative seismic deformations of 3rd floor**

Drift limit allowed between floors: d_r*v/h = 1.20‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 2.79mm, dyp = 6.01mm
Max elastic drift: dx,max = 2.99mm, dy,max = 6.29mm
$N_{tot,x}$ = 3421.4kN $V_{tot,x}$ = 1214.2kN $N_{tot,y}$ = 3421.4kN $V_{tot,y}$ = 1346.7kN

PDelta Effects:
□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 0.69% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.31% < 10%

- **Relative seismic deformations of 2nd floor**

Drift limit allowed between floors: d_r*v/h = 0.87‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 4.53mm, d_{yp} = 7.38mm
Max elastic drift: dx,max = 5.02mm, dy,max = 7.65mm
$N_{tot,x}$ = 9681.6kN $V_{tot,x}$ = 2941.7kN $N_{tot,y}$ = 9681.6kN $V_{tot,y}$ = 3136.6kN

PDelta Effects:
□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.36% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.94% < 10%

- **Relative seismic deformations of 1st floor**

Drift limit allowed between floors: d_r*v/h = 0.88‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 6.55mm, d_{yp} = 7.52mm
Max elastic drift: dx,max = 7.03mm, dy,max = 7.72mm
$N_{tot,x}$ = 16095.7kN $V_{tot,x}$ = 4326.5kN $N_{tot,y}$ = 16095.7kN $V_{tot,y}$ = 4416.7kN

PDelta Effects:
□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 2.15% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 2.31% < 10%

- **Relative seismic deformations of ground floor**

Drift limit allowed between floors: d_r*v/h = 0.73‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 6.15mm, dyp = 4.30mm
Max elastic drift: dx,max = 6.40mm, dy,max = 4.43mm
$N_{tot,x}$ = 22519.0kN $V_{tot,x}$ = 5187.7kN $N_{tot,y}$ = 22519.0kN $V_{tot,y}$ = 5142.0kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 2.29% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.60% < 10%

2. SECTION 2

- **Displacements**

Max elastic deformation along direction x-x: sx,max = 9.78mm
Max absolute deformation along direction x-x: 3.60*9.78 = 35.21mm
Displacement along direction x-x = 3.5 cm
Max elastic deformation along direction y-y: sy,max = 12.10mm
Max absolute deformation along direction y-y: 3.60*12.10 = 43.55mm
Displacement along direction y-y = 4.4cm.

Between Section 1 and Section 2, we live a construction joint having a width of 15cm

- **Relative seismic deformations of 3rd floor**

Drift limit allowed between floors: d_r*v/h = 0.31‰ < 5‰, v=0.4
Max elastic drift in the center: dxp = 1.69mm, dyp = 2.17mm
Max elastic drift: dx,max = 1.75mm, dy,max = 2.68mm
Ntot,x = 4934.1kN Vtot,x = 994.1kN Ntot,y = 4934.1kN Vtot,y = 975.6kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 0.89% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.40% < 10%

- **Relative seismic deformations of 2nd floor**

Drift limit allowed between floors: d_r*v/h = 0.36‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 2.35mm, d_{yp} = 2.72mm
Max elastic drift: dx,max = 2.43mm, dy,max =3.18mm
$N_{tot,x}$ = 10627.9kN $V_{tot,x}$ = 1856.4kN $N_{tot,y}$ = 10627.9kN $V_{tot,y}$ = 1805.8kN

PDelta Effects:

□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.43% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.92% < 10%

- **Relative seismic deformations of 1st floor**

Drift limit allowed between floors: d_r*v/h = 0.39‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 2.81mm, d_{yp} = 3.02mm
Max elastic drift: dx,max = 2.90mm, dy,max = 3.41mm
$N_{tot,x}$ = 16289.2kN $V_{tot,x}$ = 2616.1kN $N_{tot,y}$ = 16289.2kN $V_{tot,y}$ = 2561.5kN

PDelta Effects:
□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.85% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 2.23% < 10%

- **Relative seismic deformations of ground floor**

Drift limit allowed between floors: d_r*v/h = 0.30‰ < 5‰, v=0.4
Max elastic drift in the center: dxp = 2.60mm, dyp = 2.22mm
Max elastic drift: dx,max = 2.65mm, dy,max = 2.52mm
$N_{tot,x}$ = 21953.1kN $V_{tot,x}$ = 3217.0kN $N_{tot,y}$ = 21953.1kN $V_{tot,y}$ = 3201.1kN

PDelta Effects:
□$_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.86% < 10%
□$_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.78% < 10%

3. SECTION 3

- **Displacements**

Max elastic deformation along direction x-x: sx,max = 10.19mm
Max absolute deformation along direction x-x: 3.50*10.19 = 35.66mm
Displacement along direction x-x = 3.6 cm
Max elastic deformation along direction y-y: sy,max = 14.28mm
Max absolute deformation along direction y-y: 3.50*14.28 = 49.97mm
Displacement along direction y-y = 5.0cm.

Between Section 2 and Section 3, we live a construciton joint having a width of 15cm

- **Relative seismic deformations of 3rd floor**

Drift limit allowed between floors: d_r*v/h = 1.10‰ < 5‰, v=0.4
Max elastic drift in the center: dxp = 1.79mm, dyp = 1.91mm
Max elastic drift: dx,max = 1.90mm, dy,max = 2.76mm
Ntot,x = 5580.6kN Vtot,x = 1675.3kN Ntot,y = 5580.6kN Vtot,y = 1486.2kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 0.63% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.03% < 10%

- **Relative seismic deformations of 2nd floor**

Drift limit allowed between floors: d_r*v/h = 1.52‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 2.46mm, d_{yp} = 2.72mm46
Max elastic drift: dx,max = 2.465m, dy,max =3.79mm
Ntot,x = 12113.9kN Vtot,x = 3194.7kN Ntot,y = 12113.9kN Vtot,y = 2821.6kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.00% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.63% < 10%

- **Relative seismic deformations of 1st floor**

Drift limit allowed between floors: d_r*v/h = 1.73‰ < 5‰, v=0.4
Max elastic drift in the center: d_{xp} = 2.83mm, d_{yp} = 2.68mm
Max elastic drift: dx,max = 3.06mm, dy,max = 4.33mm
Ntot,x = 18670.8kN Vtot,x = 4399.8kN Ntot,y = 18670.8kN Vtot,y = 3884.4kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.30% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 2.08% < 10%

- **Relative seismic deformations of ground floor**

Drift limit allowed between floors: d_r*v/h = 1.27‰ < 5‰, v=0.4
Max elastic drift in the center: dxp = 2.34mm, dyp = 1.83mm
Max elastic drift: dx,max = 2.45mm, dy,max = 3.17mm
Ntot,x = 25231.7kN Vtot,x = 5283.1kN Ntot,y = 25231.7kN Vtot,y = 4734.9kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.17% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 1.69% < 10%

4. <u>SECTION 4</u>

- **Displacements**

Max elastic deformation along direction x-x: sx,max = 5.42mm
Max absolute deformation along direction x-x: 3.50*5.42 = 18.97mm
Displacement along direction x-x = 1.9 cm

Max elastic deformation along direction y-y: sy,max = 6.06mm
Max absolute deformation along direction y-y: 3.50*6.06 = 21.22mm
Displacement along direction y-y = 2.1cm.

Between Section 2 and Section 3, we live a construciton joint having a width of 15cm

- **Relative seismic deformations of 1st floor**

Drift limit allowed between floors: d_r*v/h = 2.46‰ < 5‰, v=0.4
Max elastic drift limit: d_{xp} = 2.73mm, d_{yp} = 3.01mm
Max elastic drift: dx,max = 3.43mm, dy,max = 6.14mm
$N_{tot,x}$ = 3964.2kN $V_{tot,x}$ = 1012.4kN $N_{tot,y}$ = 3964.2kN $V_{tot,y}$ = 1012.5kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.34% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 2.41% < 10%

- **Relative seismic deformations of ground floor**

Drift limit allowed between floors: d_r*v/h = 1.93‰ < 5‰, v=0.4
Max elastic drift in the center: dxp = 2.37mm, dyp = 2.27mm
Max elastic drift: dx,max = 2.67mm, dy,max = 4.84mm
Ntot,x = 6807.3kN Vtot,x = 1394.5kN Ntot,y = 6807.3kN Vtot,y = 1368.6kN

PDelta Effects:
$\square_x$ = (Ntot,x/Vtot,x)*q*(dx,max/h) = 1.30% < 10%
$\square_y$ = (Ntot,y/Vtot,y)*q*(dy,max/h) = 2.41% < 10%

7.2 Object Classification according to *[EC8 §5.2.2.1]*

7.2.1 Regularity on the horizontal plane *[EC8 §4.2.3.2]*

Data:
All the floors are equipped with rigid diphragms

Result: Criterion fulfilled : YES

7.2.2 Regularity on vertical plane *[EC8 §4.2.3.3]*

Verifications:
All the columns are of countinuous type.

Results:
Criterion fulfilled: YES

Result:
Regularity on horizontal plane : YES
Regularity on vertical plane: YES

7.3 Plastic Node Avoidance Control in the Columns *[EC8 §4.3.3.3]*

7.3.1 Autosufficiency Verification of Structural Walls

SECTION 1

- Direction X-X : V_t = 0.0 kN, V_{tot} = 5066.54 kN, n_V = 0.0 < 0.65
- Direction Y-Y : V_t = 4187.65 kN, V_{tot} = 5001.65 kN, n_V = 0.837 > 0.65

SECTION 2

- Direction X-X : V_t = 1899.48 kN, V_{tot} = 3025.70 kN, n_V = 0.628 ≤ 0.65
- Direction Y-Y : V_t = 2153.89 kN, V_{tot} = 2637.17 kN, n_V = 0.817 > 0.65

SECTION 3

- Direction X-X : Vt = 2278.97 kN, Vtot = 4793.06 kN, nv = 0.4759 ≤ 0.65
- Direction Y-Y : Vt = 2737.49 kN, Vtot = 3970.61 kN, nv = 0.689 > 0.65

SECTION 4

- Direction X-X : V_t = 0.0 kN, V_{tot} = 1011.75 kN, n_V = 0.0 < 0.65
- Direction Y-Y : V_t = 0.0 kN, V_{tot} = 1247.83 kN, n_V = 0.0 < 0.65

7.4 Calculation of seismic reaction koefficient *[EC8 §5.2.2.2]*

Symbols:

q	koefficient of seismic reaction
qo	base value of coefficient of seismic reaction
kw	the coefficient referring to the predominant mechanism of destruction in the construction system with structural walls
α1	constant of multiplication of horizontal seismic computational actions when

the first plastic node of the system is created
αu constant multiplication of horizontal seismic actions of the calculation when the plastic mechanism is created

- Data "SECTION 1":

Object Category Direction x-x	Frame System
Object Category Direction y-y	Wall System
Ductility Category	Mean Ductility
Regularity on horizontal plane:	YES
Regularity on vertical plane:	YES

	αu/α1	αqo	qo	kw	q
Direction x-x	1.30	3.00	3.12	1.00	3.12
Direction y-y	1.20	3.00	2.88	1.00	2.88

- q = 0.80 x αqo x (1/2x(1+ αu/α1)) *[EC8 §3.2.2]*

Results:
Coefficient of seismic reaction *q*: **2.88**

- Data **"SECTION 2"**:

Object Category Direction x-x	Wall System
Object Category Direction y-y	Wall System
Ductility Category	Mean Ductility
Regularity on horizontal plane:	YES
Regularity on vertical plane:	YES

	αu/α1	αqo	qo	kw	q
Direction x-x	1.20	3.00	3.60	1.00	3.60
Direction y-y	1.20	3.00	3.60	1.00	3.60

- q = 0.80 x αqo x (1/2x(1+ αu/α1)) *[EC8 §3.2.2]*

Results:
Koefficien of seismic reaciton *q*: **3.60**

- Data for **"SECTION 3"**:

Object Category Direction x-x	Wall System
Object Category Direction y-y	Wall System
Ductility Category	Mean Ductility

Regularity on horizontal plane:	YES
Regularity on vertical plane:	YES

	αu/α1	αqo	qo	kw	q
Direction x-x	1.20	1.20	1.20	1.20	1.20
Direction y-y	3.00	3.00	3.00	3.00	3.00

- q = 0.80 x αqo x (1/2x(1+ αu/α1)) *[EC8 §3.2.2]*

Results:
Koefficient of seismic reaciton *q*: **3.60**

7.5 Determining of the Design Specter *[EC8 §3.2.2]*

<u>Symbols:</u>

α_{gR} maximal acceleration with reference to ground of category A
□$_i$ koefficient of importance
q koefficient of seismic reaction
S ground coefficient
T the oscillation period of the oscillator by one degree
ξ viscous quenching
□ The lower limit in the horizontal spectrum of the calculation
$S_d(T)$ Design spectrum
g Gravitational acceleration

<u>SECTION 1</u>

<u>Data:</u>

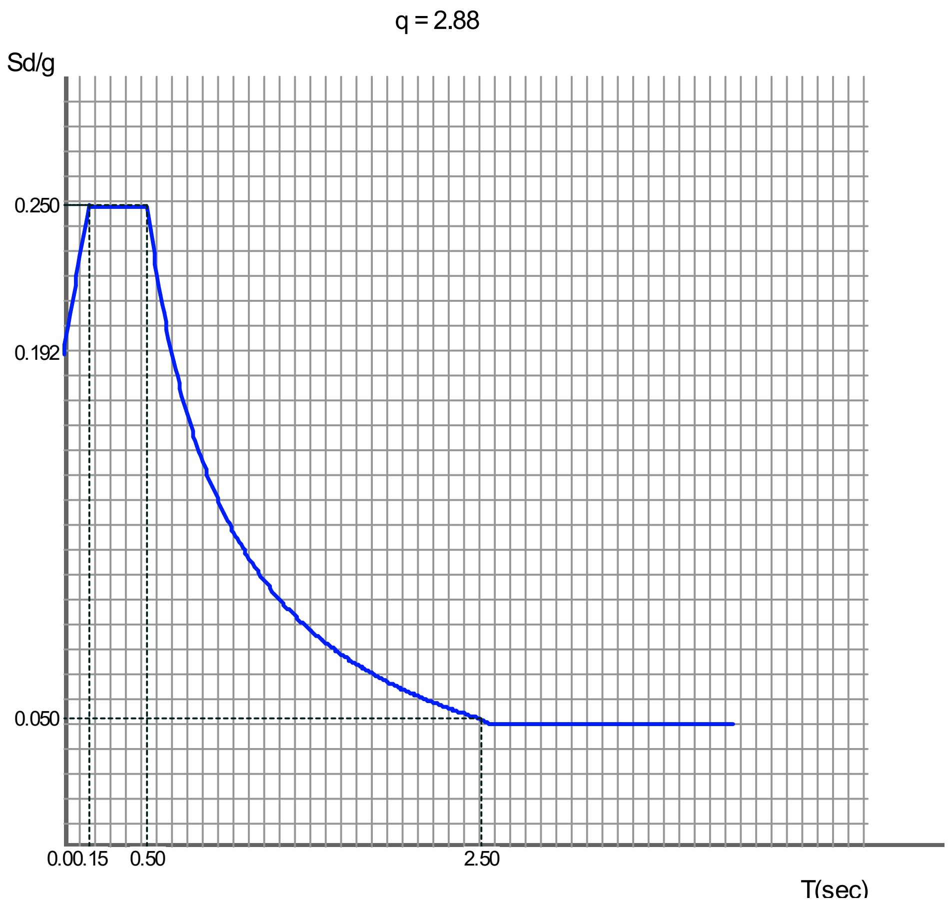
□$_i$ 1.20 (III)
ξ 5 %
□ 0.20

Spectrum Type: Type 1
Ground Type: C

	αgr (II)	q	S	TB (s)	TC (s)	TD (s)
horizontal	0.26	2.88	1.20	0.15	0.50	2.50
vertical	0.22	1.50	1.00	0.05	0.15	1.00

Result:
SD/g

	0	TB	TC	TD	4
horizontal	0.19	0.25	0.25	0.05	0.05
vertical	0.14	0.19	0.06	0.04	0.04



SECTION 2

Data:

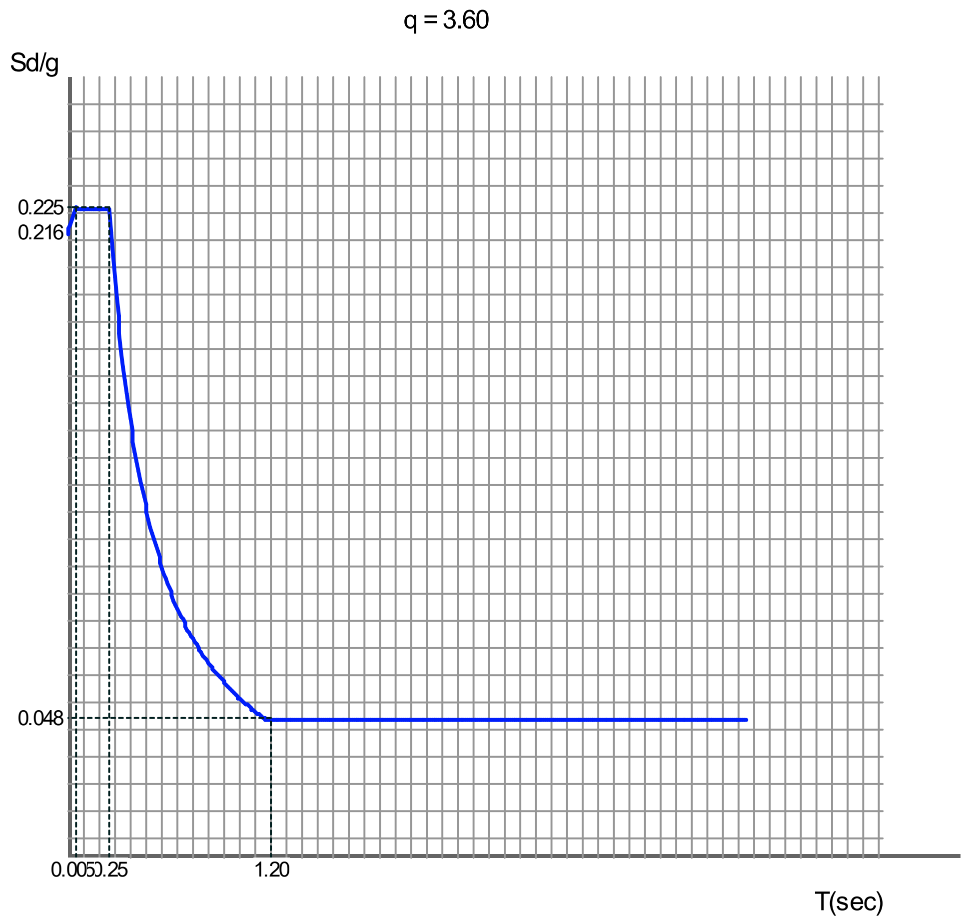
□i 1.20 (III)
ξ 5 %
□ 0.20

Spectrum Type: Type 2
Ground Category: C

	αgr (II)	q	S	TB (s)	TC (s)	TD (s)
horizontal	0.26	3.60	1.35	0.05	0.25	1.20
vertical	0.22	1.50	1.00	0.05	0.15	1.00

Result:
SD/g

	0	TB	TC	TD	4
horizontal	0.18	0.19	0.19	0.06	0.05
vertical	0.14	0.11	0.04	0.04	0.04



SECTION 3

Data:

□$_i$ 1.20 (III)

ξ 5 %

□ 0.20

Spectrum Type: Type 2

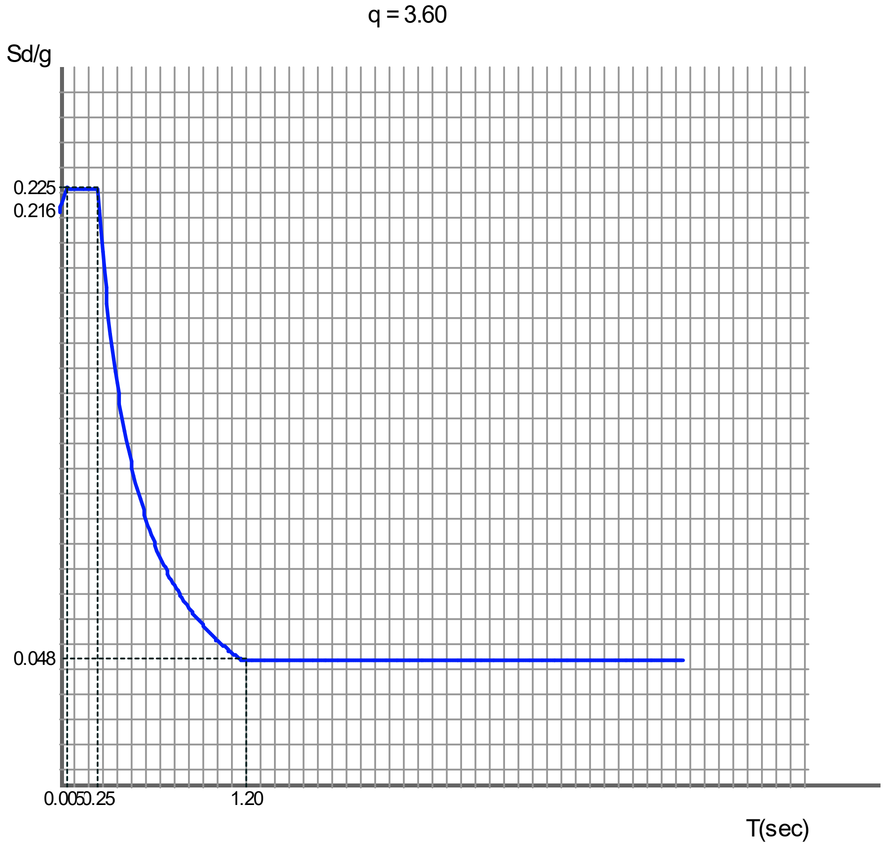
Ground Category: C

	αgr (II)	q	S	TB (s)	TC (s)	TD (s)
horizontal	0.26	0.24	0.24	0.24	0.24	0.24
vertical	3.60	3.60	3.60	3.60	3.60	3.60

Result:

SD/g

	0	TB	TC	TD	4
horizontal	0.18	0.35	0.35	0.10	0.05
vertical	0.14	0.20	0.07	0.04	0.04



SECTION 4

□i 1.20 (III)
ξ 5 %
□ 0.20

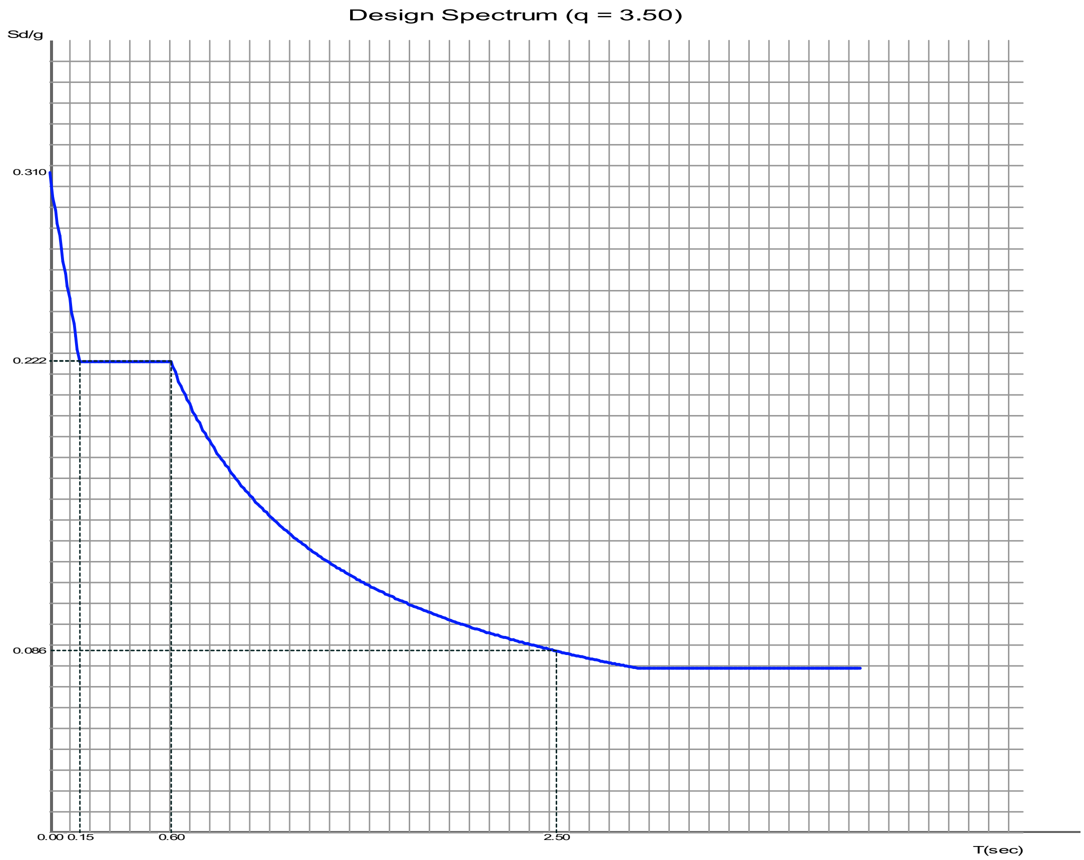
Spectrum Type: Type 2
Ground Category: C

	αgr (II)	q	S	TB (s)	TC (s)	TD (s)
horizontal	0.26	0.24	0.24	0.24	0.24	0.24
vertical	3.60	3.60	3.60	3.60	3.60	3.60

Result:
SD/g

	0	TB	TC	TD	4
horizontal	0.18	0.35	0.35	0.10	0.05

vertical	0.14	0.20	0.07	0.04	0.04



7.6 Modal Analysis of Design Specter *[EC8 §4.3.3.3]*

7.6.1 Modal form analysis

<u>***"SECTION 1"***</u>

Table of modal forms:

Mode	□ (rad/sec)	T (sec)	S_d	$□_x$	C_x (%)	$□_y$	C_y (%)	$□_z$	C_z (%)
1	12.56	0.500319	2.50	-1.74	0.10	-42.07	61.11	0.00	0.00
2	12.99	0.483794	2.50	43.21	64.47	-1.87	0.12	0.00	0.00
3	16.28	0.385871	2.50	-9.81	3.32	-0.79	0.02	0.00	0.00
4	38.95	0.161329	2.50	-14.49	7.25	-0.11	0.00	0.00	0.00
5	41.82	0.150252	2.50	0.12	0.00	-17.32	10.36	0.01	0.00
6	52.10	0.120604	2.39	-0.93	0.03	-1.19	0.05	0.00	0.00
7	61.73	0.101784	2.31	8.33	2.40	-0.17	0.00	0.00	0.00

8	76.81	0.081803	2.24	-0.20	0.00	-13.20	6.02	-0.03	0.00
9	83.15	0.075561	2.21	4.52	0.71	-0.50	0.01	0.00	0.00
Sum					**78.29**		**77.69**		

Participation criterion of 90% of mass in modal forms

Dir.	k		$3n^{0.5}$	$T_k \leq 0.20s$
x	9	≥	6	0.076 ≤ 0.20
y	9	≥	6	0.076 ≤ 0.20

k, Number of modes that take part in the analysis.
n, Number or floor above ground level.
Tk, Period of *k* Mode

a_{CM} = 1.7261
Kontrolli: □□□

"SECTION 2"

Table of modal forms:

Mode	□ (rad/sec)	T (sec)	S_d	$□_x$	C_x (%)	$□_y$	C_y (%)	$□_z$	C_z (%)
1	14.26	0.440642	1.28	-9.29	3.09	-41.39	61.38	-0.02	0.00
2	15.34	0.409600	1.37	42.10	63.50	-9.97	3.56	-0.01	0.00
3	19.46	0.322817	1.74	-4.06	0.59	-7.53	2.03	0.02	0.00
4	50.86	0.123543	2.25	-15.55	8.66	-4.13	0.61	0.00	0.00
5	54.38	0.115550	2.25	4.01	0.58	-17.06	10.43	0.00	0.00
6	67.07	0.093686	2.25	1.73	0.11	3.72	0.50	0.00	0.00
7	96.08	0.065392	2.25	-8.27	2.45	-1.93	0.13	0.00	0.00
8	101.34	0.061999	2.25	-1.46	0.08	11.04	4.37	0.02	0.00
9	131.23	0.047878	2.25	1.22	0.05	3.07	0.34	-0.02	0.00

Sum					**79.12**		**83.35**		

Participation criterion of 90% of mass in modal forms

Dir.	k		$3n^{0.5}$	$T_k \leq 0.20s$
x	9	≥	6	0.048 ≤ 0.20
y	9	≥	6	0.048 ≤ 0.20

k, Number of modes that take part in the analysis.
n, Number or floor above ground level.
Tk, Period of *k* Mode

a_{CM} = 1.0819
Control: □□

“SECTION 3”

Table of modal forms:

Mode	□ (rad/sec)	T (sec)	S_d	□x	C_x (%)	□y	C_y (%)	□z	C_z (%)
1	17.37	0.361718	2.22	-16.81	8.51	-36.23	39.52	0.03	0.00
2	18.98	0.331065	2.22	42.59	54.62	-10.60	3.38	-0.01	0.00
3	20.59	0.305144	2.22	-6.21	1.16	26.14	20.58	-0.02	0.00
4	59.11	0.106295	2.48	6.82	1.40	15.03	6.80	0.03	0.00
5	65.27	0.096266	2.54	16.87	8.57	-2.97	0.27	0.00	0.00
6	68.91	0.091180	2.57	-3.33	0.33	13.94	5.85	0.03	0.00
7	114.13	0.055054	2.78	-4.42	0.59	-10.12	3.08	0.01	0.00
8	127.44	0.049304	2.81	-9.66	2.81	2.40	0.17	0.01	0.00
9	134.80	0.046610	2.83	1.62	0.08	-10.04	3.04	-0.01	0.00
Sum					**78.07**		**82.69**		

Participation criterion of 90% of mass in modal forms

Dir.	k		$3n^{0.5}$	$T_k \leq 0.20s$
x	9	≥	6	0.049 ≤ 0.20
y	9	≥	6	0.049 ≤ 0.20

k, Number of modes that take part in the analysis.
n, Number or floor above ground level.
Tk, Period of *k* Mode

a_{CM} = 1.2100
Control: □□

"SECTION 4"

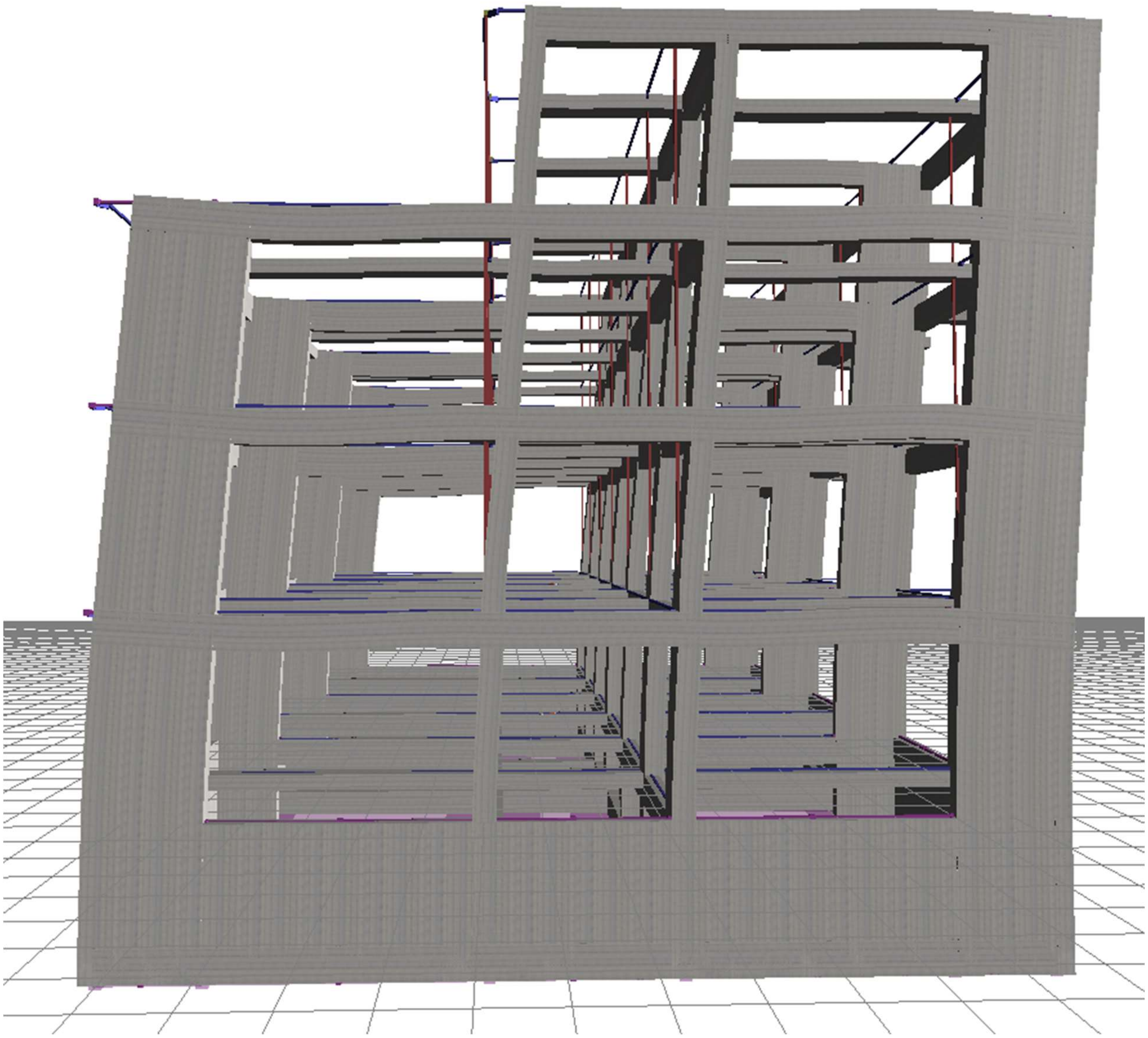
Table of modal forms:

Mode	□ (rad/sec)	T (sec)	S_d	□$_x$	C_x (%)	□$_y$	C_y (%)	□$_z$	C_z (%)
1	22.75	0.276128	2.22	2.96	1.08	-24.34	73.17	-0.02	0.00
2	22.83	0.275169	2.22	24.49	74.10	2.99	1.11	0.01	0.00
3	29.63	0.212077	2.22	-0.84	0.09	1.64	0.33	-0.01	0.00
4	68.18	0.092151	2.56	2.01	0.50	-0.11	0.00	0.02	0.00
5	68.74	0.091403	2.56	0.68	0.06	0.44	0.02	0.03	0.00
6	71.15	0.088310	2.58	-0.60	0.04	-0.04	0.00	0.00	0.00
7	73.21	0.085827	2.60	0.08	0.00	-0.25	0.01	-0.01	0.00
8	75.75	0.082943	2.61	-0.65	0.05	-0.18	0.00	0.00	0.00
9	77.67	0.080898	2.63	-0.48	0.03	2.56	0.81	-0.01	0.00
Sum					**75.94**		**75.46**		

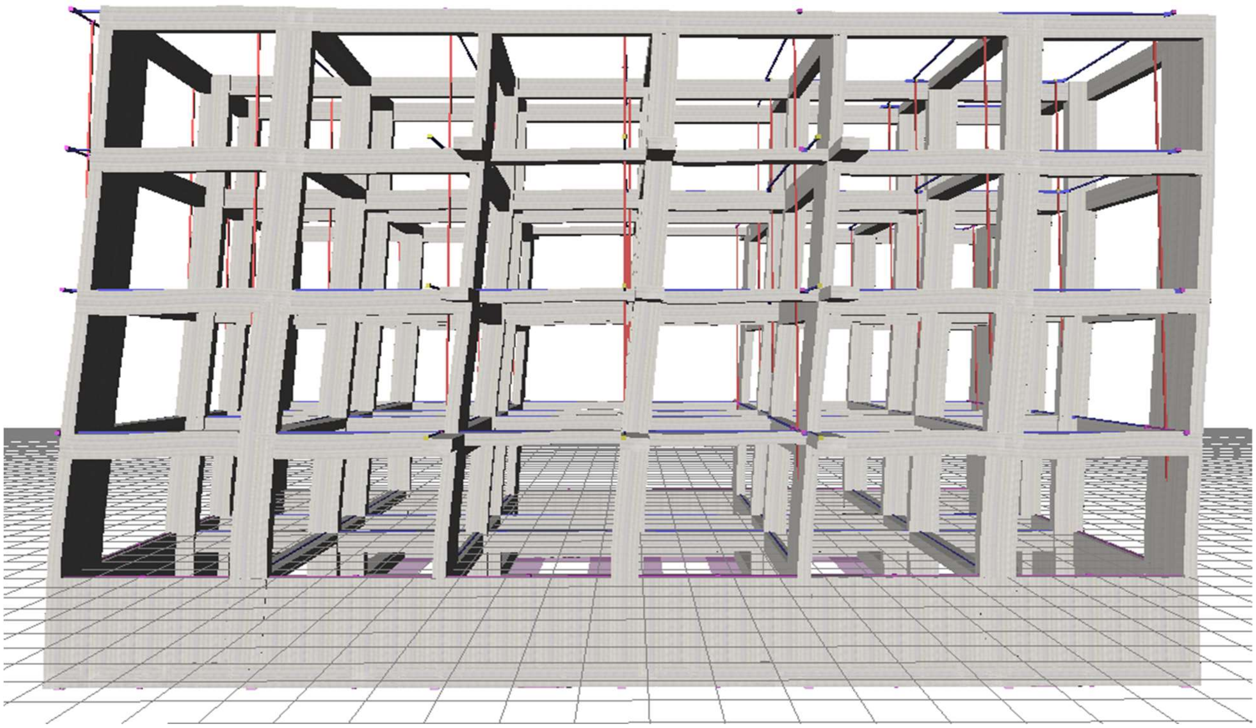
7.7 Dynamic Analysis of Structure

In order to reflect as accurately as possible the dynamic characteristics of the structure, 18 basic oscillation shapes have been considered. This has resulted in the oscillation of almost OVER 80% of the mass of the building. The period of the first oscillation tone has resulted T = 0.45 sec. PGA (Pick Ground Acceleration) is 0.26 g corresponding to 8 magnitude earthquake, Category C ground.
For all 4 sections, the first and second modes resulted in translational displacement, while the third mode resulted in rotational displacement. For the sake of simplicity, we are graphically representing the displacements for Sections 1 and 2.

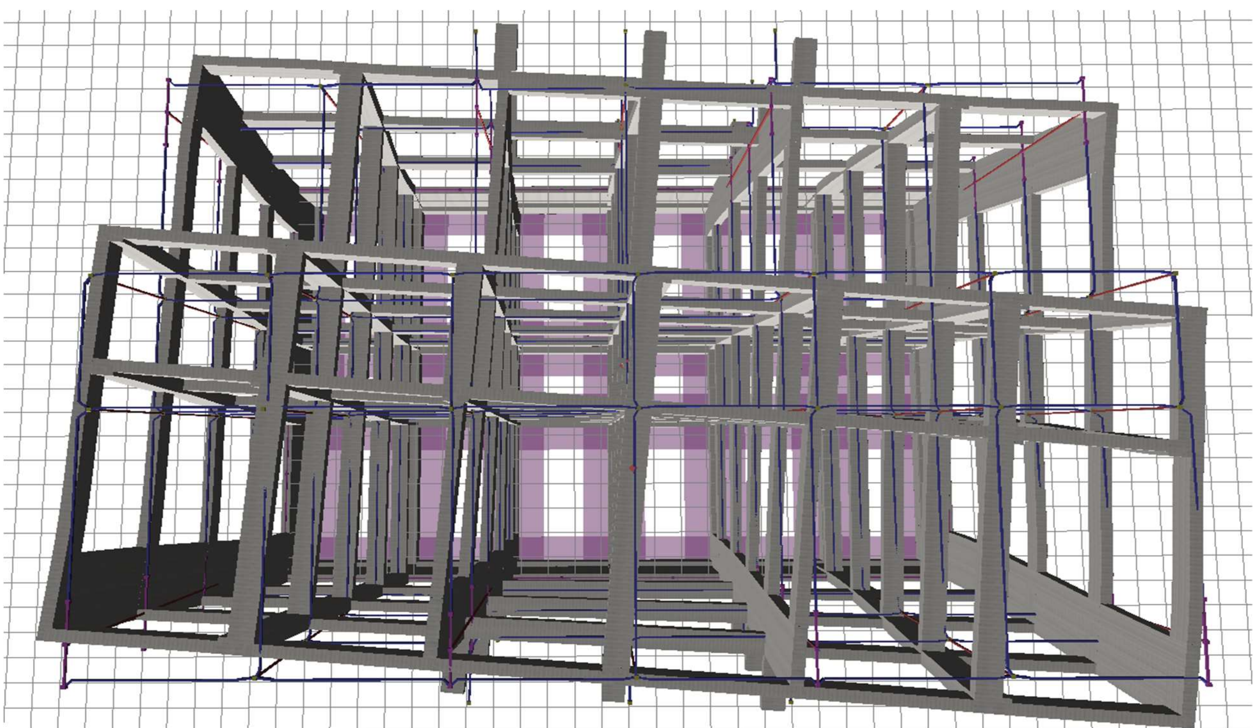
SECTION 1



First mode of oscillation

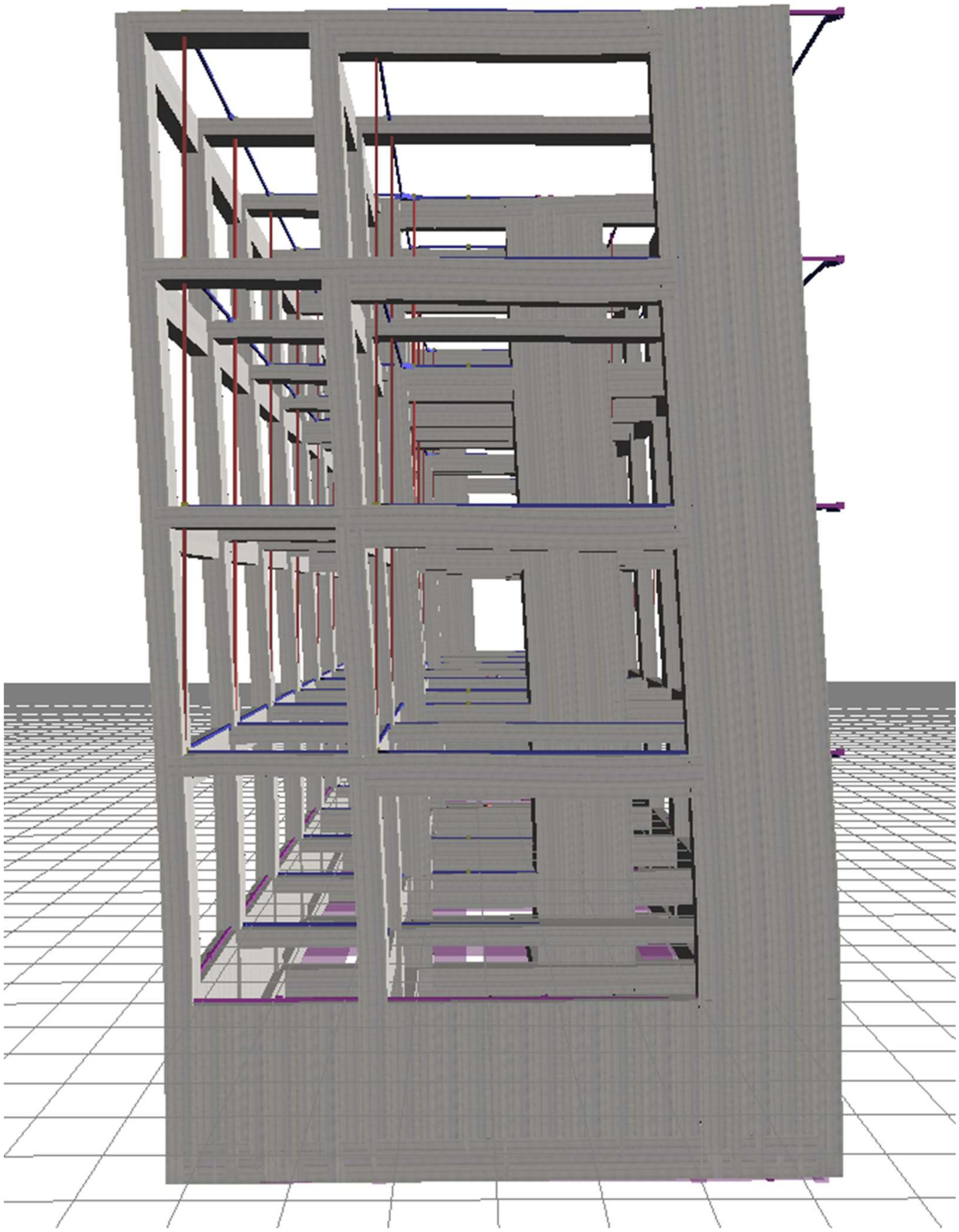


Second mode of oscillation

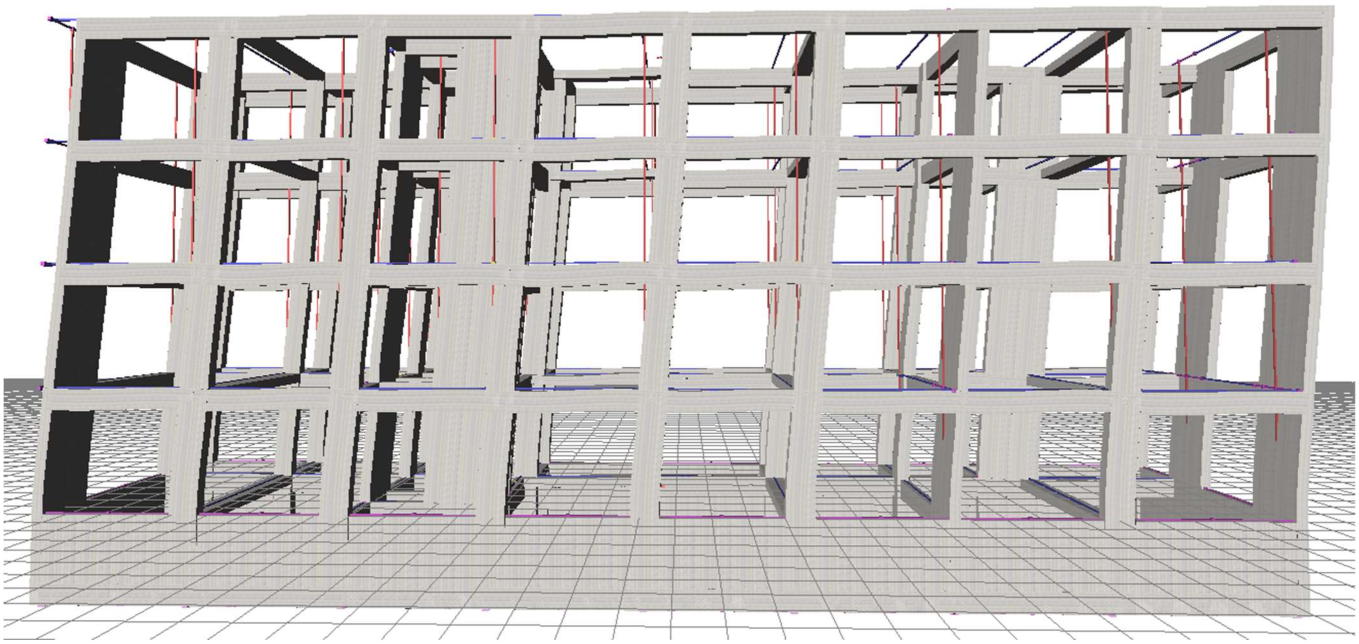


Third mode of oscillation

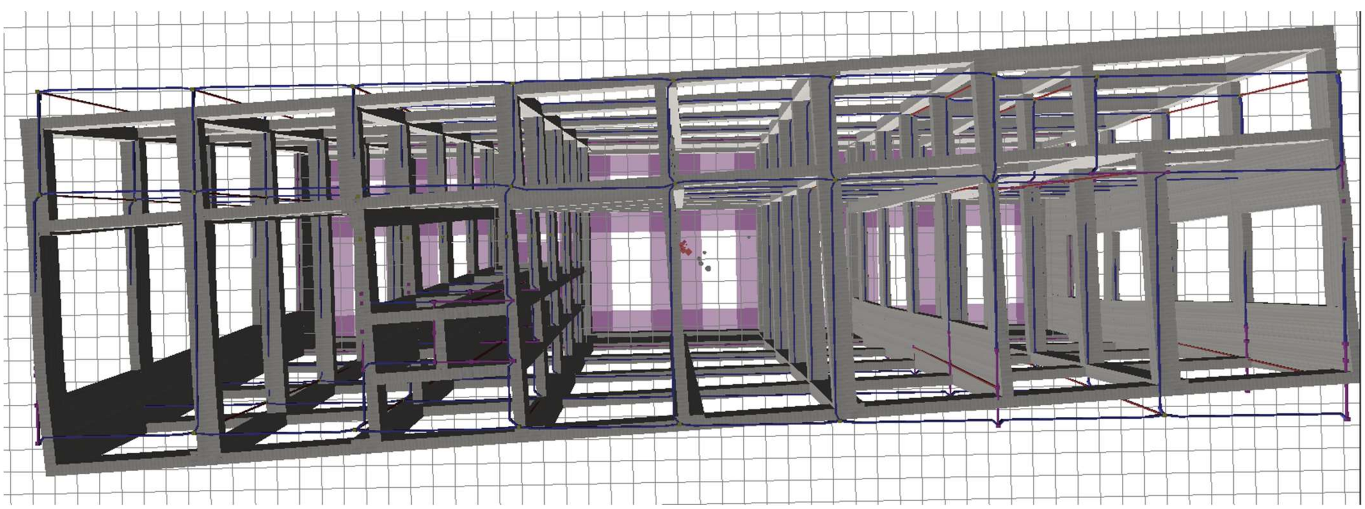
SECTION 2



First mode of oscillation



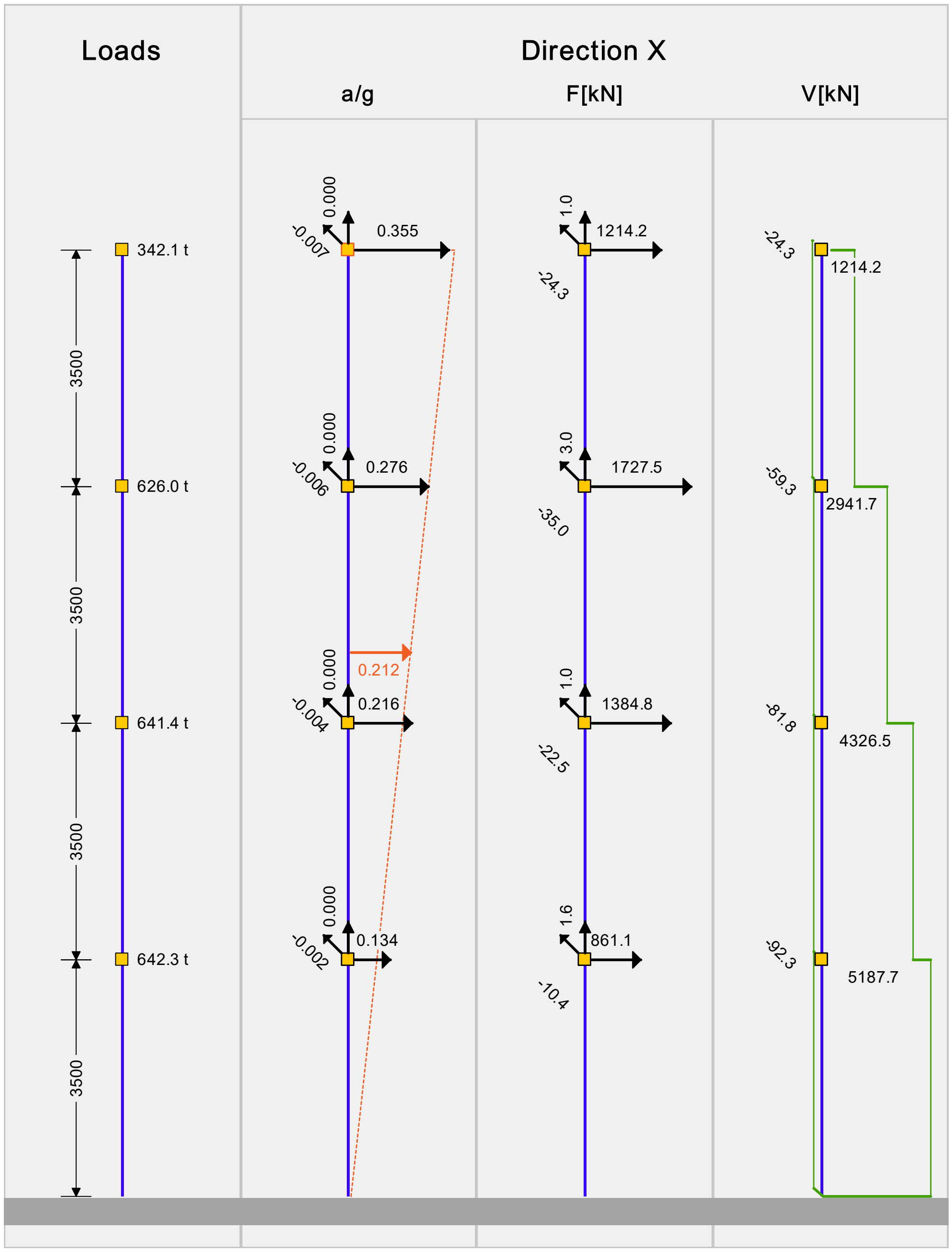
Second mode of oscillation

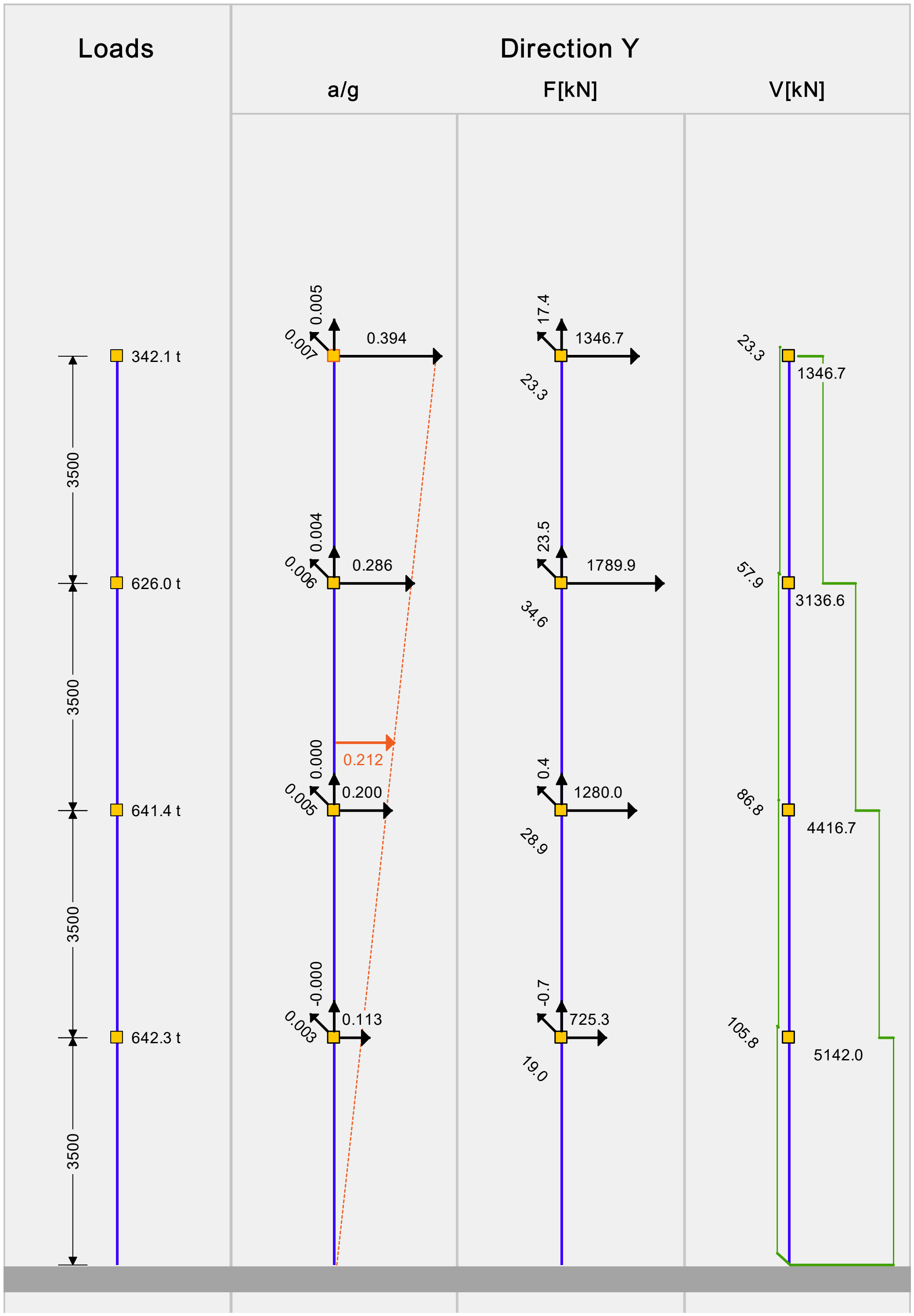


Third mode of oscillation

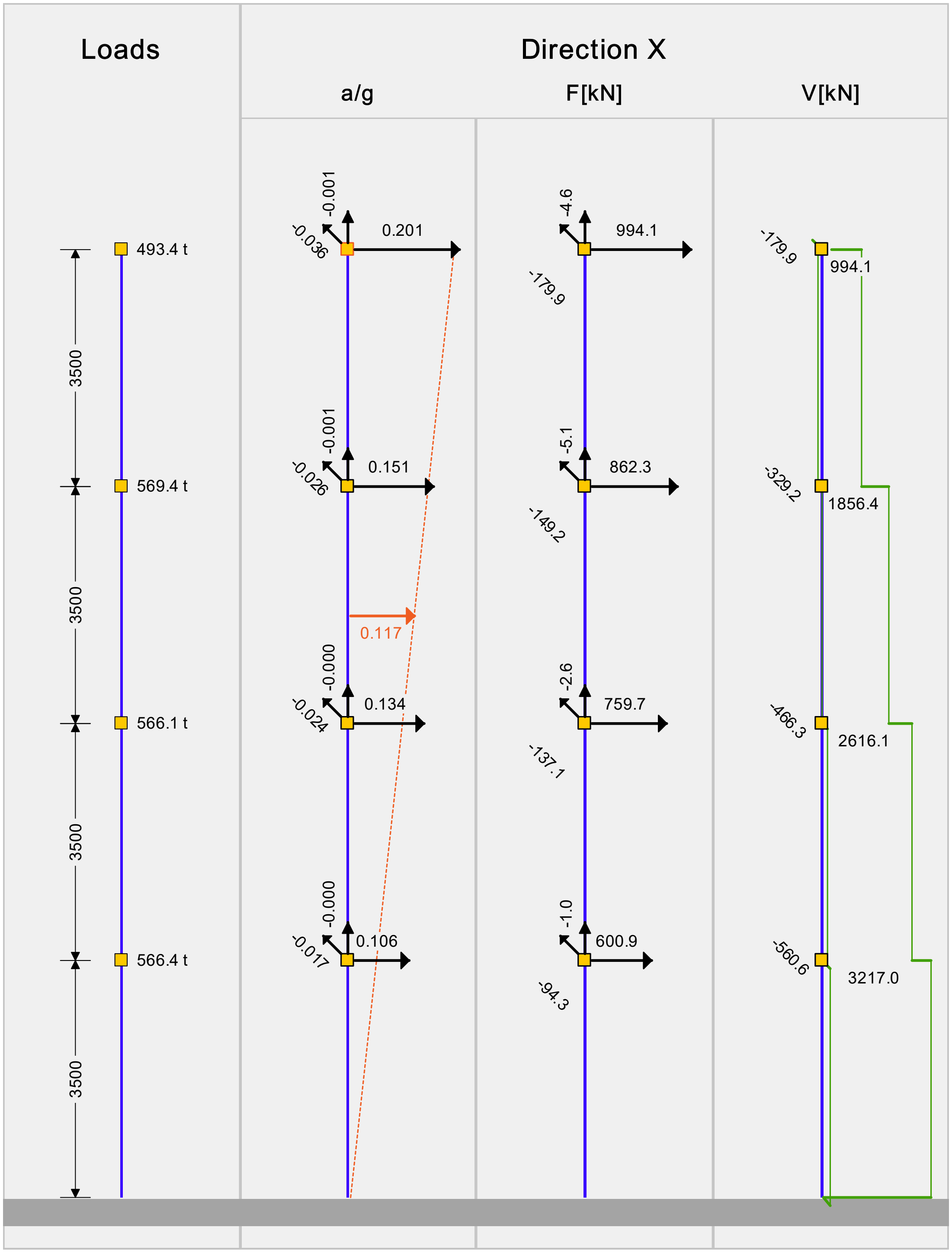
7.8 Acceleration and Seismic Forces

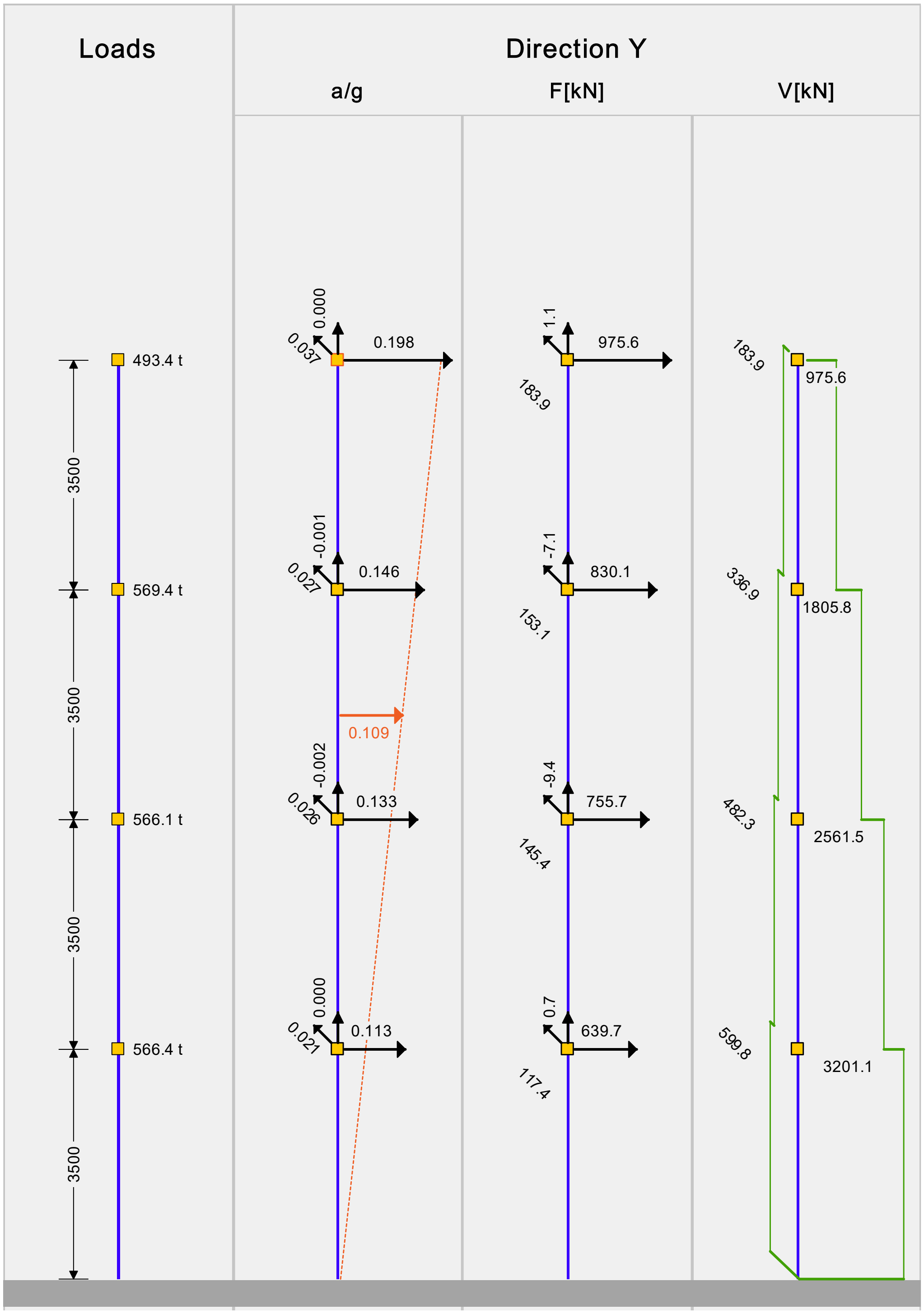
SECTION 1



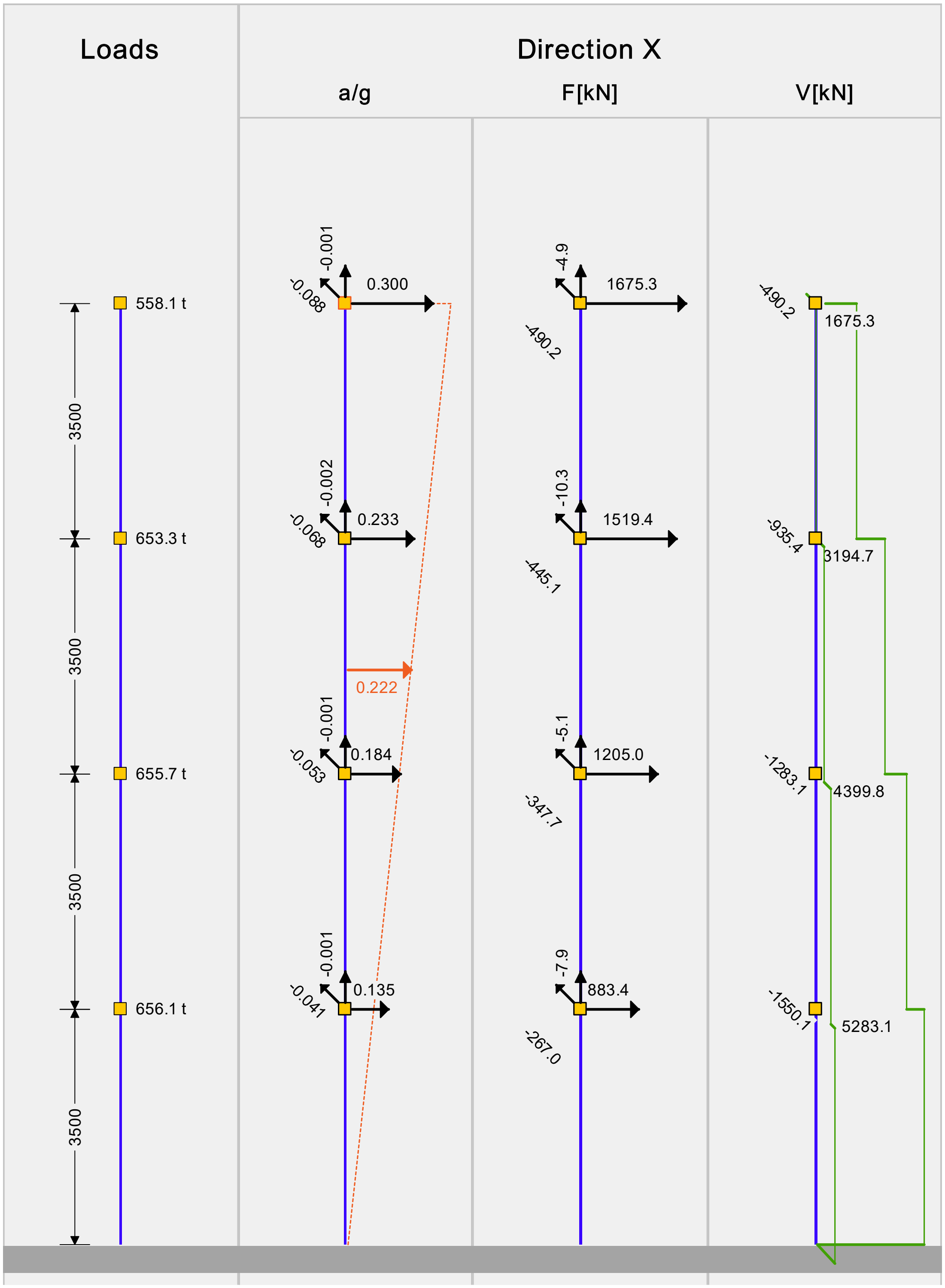

Loads
Direction Y
a/g
F[kN]
V[kN]
342.1 t
626.0 t
641.4 t
642.3 t
3500
3500
3500
3500
0.005
0.007
0.394
0.004
0.006
0.286
0.000
0.212
0.005
0.200
-0.000
0.003
0.113
17.4
1346.7
23.3
23.5
1789.9
34.6
0.4
1280.0
28.9
-0.7
725.3
19.0
23.3
1346.7
57.9
3136.6
86.8
4416.7
105.8
5142.0

SECTION 2


Loads
Direction X
a/g
F[kN]
V[kN]
493.4 t
569.4 t
566.1 t
566.4 t
3500
3500
3500
3500
-0.001
-0.036
0.201
-0.001
-0.026
0.151
0.117
-0.000
-0.024
0.134
-0.000
-0.017
0.106
-4.6
994.1
-179.9
-5.1
862.3
-149.2
-2.6
759.7
-137.1
-1.0
600.9
-94.3
-179.9
994.1
-329.2
1856.4
-466.3
2616.1
-560.6
3217.0


Loads
Direction Y
a/g
F[kN]
V[kN]
493.4 t
569.4 t
566.1 t
566.4 t
3500
3500
3500
3500
0.000
0.037
0.198
-0.001
0.027
0.146
0.109
-0.002
0.026
0.133
0.000
0.021
0.113
1.1
975.6
183.9
-7.1
830.1
153.1
-9.4
755.7
145.4
0.7
639.7
117.4
183.9
975.6
336.9
1805.8
482.3
2561.5
599.8
3201.1

SECTION 3



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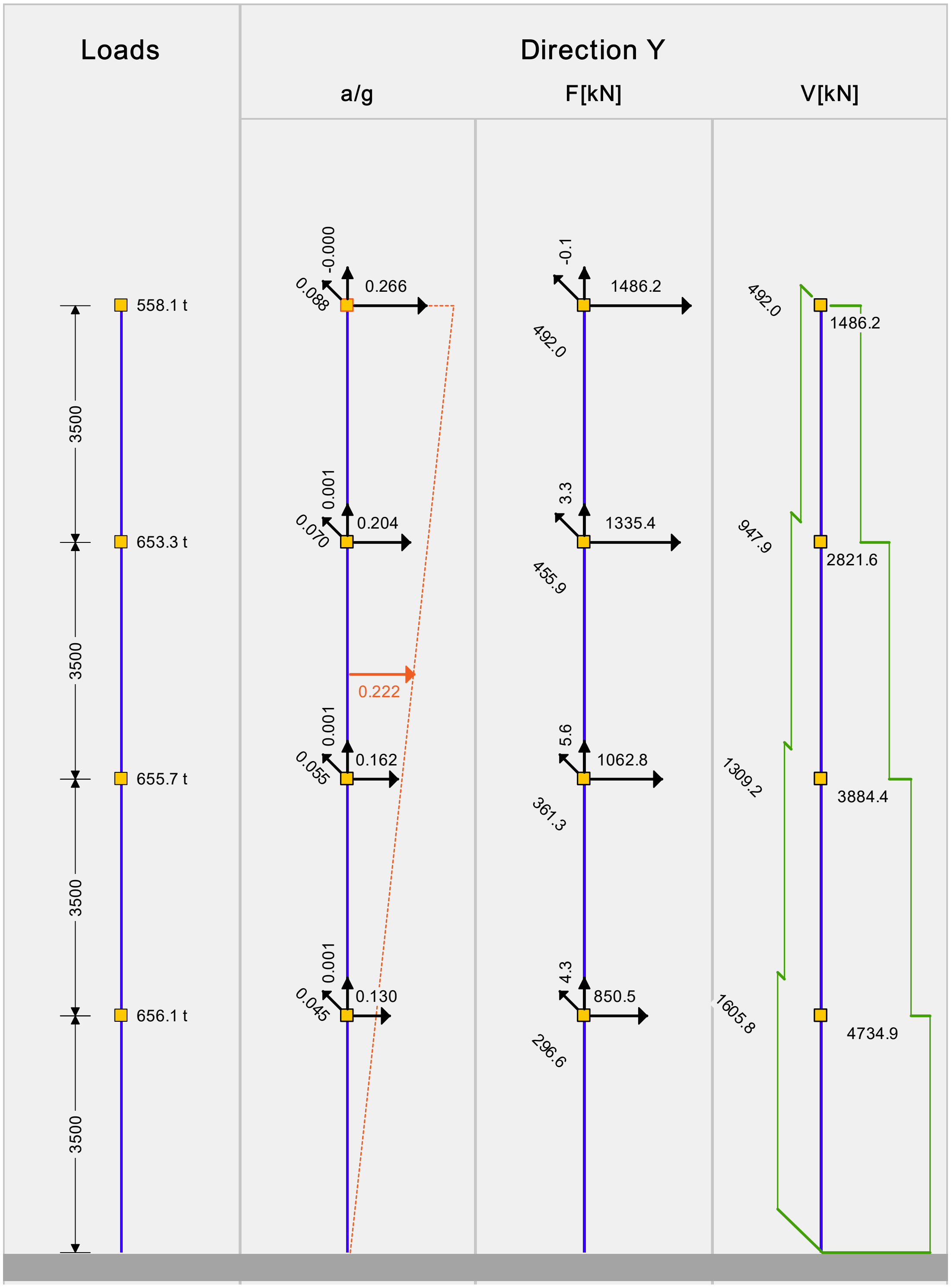
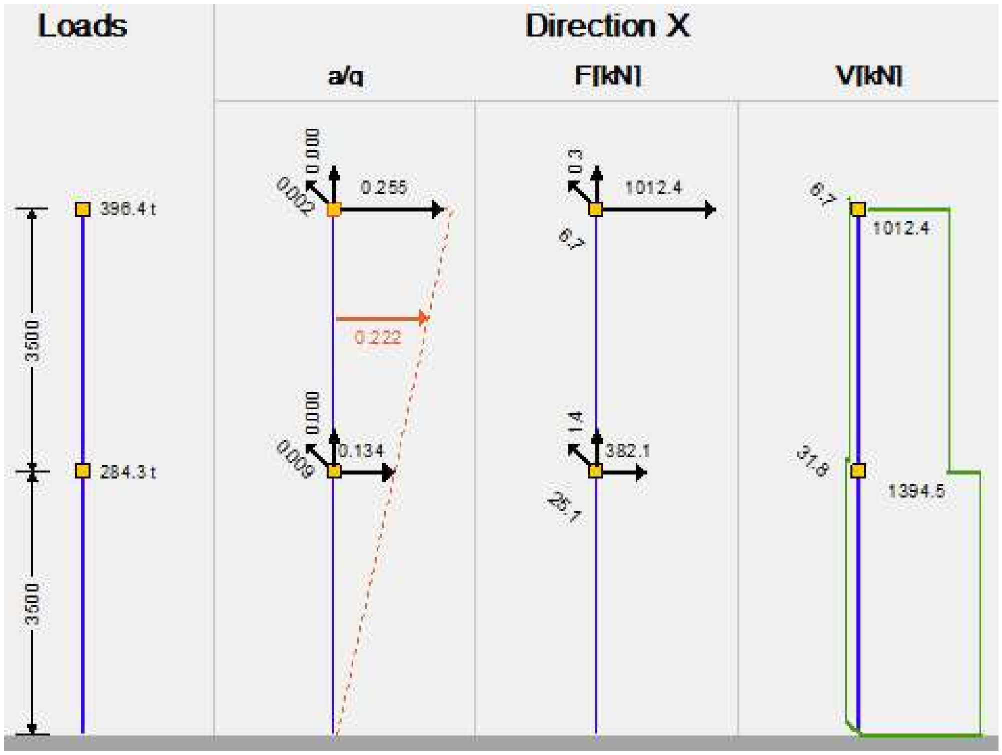
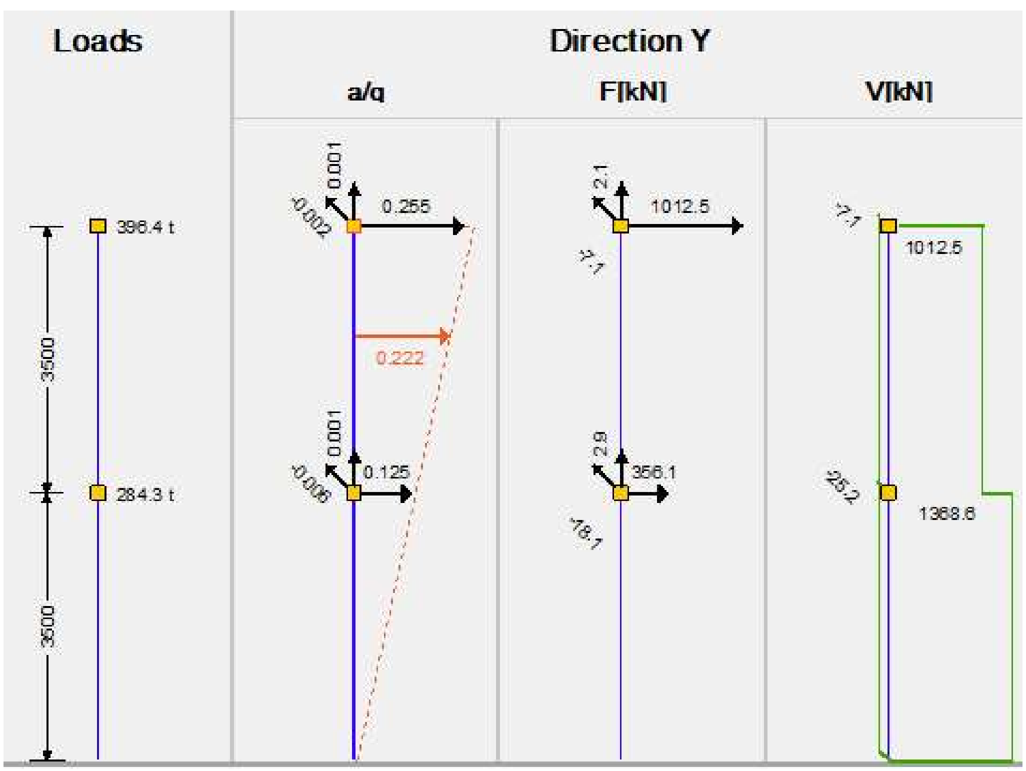
E-mail: arkimade@yahoo.com
website: www.arkimade.com


Loads
Direction Y
a/g
F[kN]
V[kN]
558.1 t
653.3 t
655.7 t
656.1 t
3500
3500
3500
3500
-0.000
0.088
0.266
0.001
0.070
0.204
0.222
0.001
0.055
0.162
0.001
0.045
0.130
-0.1
1486.2
492.0
3.3
1335.4
455.9
5.6
1062.8
361.3
4.3
850.5
296.6
492.0
1486.2
947.9
2821.6
1309.2
3884.4
1605.8
4734.9

SECTION 4


Loads
Direction X
a/q
F[kN]
V[kN]
396.4 t
284.3 t
3500
3500
0.000
0.002
0.255
0.222
0.000
0.009
0.134
0.3
1012.4
6.7
1.4
382.1
25.1
6.7
1012.4
31.8
1394.5


Loads
Direction Y
a/q
F[kN]
V[kN]
396.4 t
284.3 t
3500
3500
0.001
-0.002
0.255
0.222
0.001
-0.006
0.125
2.1
1012.5
-7.1
2.9
356.1
-18.1
-7.1
1012.5
-25.2
1368.6

8 CONCLUSIONS

- From the study of the constructive project of the above data that refer to the Technical Report, as well as from the static and dynamic calculation of the 3D model with the program HoloBIM 10, we conclude that:
- The building is regular in the horizontal and vertical plane.
- All loads, dead, live and special (seismic) are obtained in accordance with KTP-N2-89 and Eurocode 8.
- Load combinations are made in accordance with EC2 and EC8.
- The design with beams of considerable height combined with beams with reduced height makes the object have sufficient stiffness/rigidity in both planar directions.
- Classification of Structures: Section 1 "Frame System" according to direction x-x and "Wall System" according to direction y-y, Section 2 and Section 3 "Wall System, while for Section 4 is "Frame System".
- Deformations result within the norms set by the respective Eurocodes.
- Deformation schemes are translational (first 2) and rotational (third)
- The construction of all structures was done respecting KTP-N2-89 but also Eurocodes 2 and 8.
- The percentages of steel reinforcement of the structural elements are within the norms defined by EC2 and EC8.
- The structures are designed with high quality material suitable for constructions of this type and for areas with considerable seismic likelyhood.