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STRUCTURAL REPORT
NATIONAL HISTORICAL MUSEUM

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1. INTRODUCTION

The building "National Historical Museum" (NHM) has the status of Cultural Monument of the second category and is located within the declared area of the "Cultural Monument Ensemble of the main axis and the historic center of the city of Tirana." The building was built in the early 1980s. The structure of the building consists of 4 structures separated by joints between them respectively buildings A, B, C and D.

Due to a period of approximately 41 years from the time of design and construction, accompanied by changes in technical design conditions and changes in local parameters, in particular seismic parameters, the assessment and rehabilitation of existing structures is necessary to be based on Eurocodes , Terms of Reference [ToR] requirement. The level of knowledge of the structure is based on the graphic materials of the initial project, the visual evaluation during the visits to the building of the National Historical Museum and the evaluation of the issues raised in the accompanying reports of the ToR.

The evaluation of the structures of the existing building is based on the graphic material made available by the Ministry of Culture, the evaluation of the structural materials through non-destructive tests and on-site observations of the issues raised. The assessment is based on the assessment of the time period in which the project is realized, technical conditions of design and implementation of works in the design year, geological study report of the existing facility, existing architectural projects, structural, mechanical, electrical and requirements of the ToR.

The evaluation of the existing structure was carried out according to the following phases:

1. In the first phase the existing structure is analyzed referring to the measurement of the existing structural elements with the real scheme according to the parameters of resistance of materials, namely steel and concrete, based on the existing structural design, ground parameters according to the existing geological ratio, loads according to Eurocodes, presenting the recommendations for the project implementation phase. The calculation at this stage for the structure is based on the linear work phase of the materials. The reason for carrying out the above analysis is related to the fact that the facility will be certified according to Eurocodes, request of ToR.

2. In the second phase, based on the architectural, mechanical, electrical, fire protection project and the geological, seismic, etc. report. the structure is calculated with the exploitation loads according to the new exploitation scheme proposed in the architectural implementation project.

According to the ToR, the project of rehabilitation of objects part of the "National Historical Museum" must be certified according to European design norms, Eurocodes. The evaluation and design path follows the following steps:

a) Analysis of the existing condition of the stability of the structure of the objects part of the building of the National Historical Museum.

b) Structural reinforcements

c) Rehabilitation of the structure should be carried out according to Eurocodes.

Based on the requirements of the ToR, the evaluation of the structure and the intervention should be carried out based on the design of the structure of the existing buildings and laboratory tests of concrete and steel part of the reinforced concrete structures of the supporting skeleton and brick masonry.

Due to the importance of the facility the structure rehabilitation project should provide solutions to improve the bearing capacity for the whole structure globally and the specific elements in locally defined areas. The repair and reinforcement of structural elements will be carried out based on the requirements of Euronorms.

2. DESCRIPTION OF THE STRUCTURE

2.1 GENERAL

The building of the National Historical Museum is made of reinforced concrete structure. The constructive scheme is a 3D frame type with separate plinth type foundations, the beams are monolithic and prefabricated reinforced concrete and the soles are prefabricated and some are monolithic reinforced concrete. The structure of the building consists of 4 structures separated by joints between them respectively the buildings A, B, C and D.

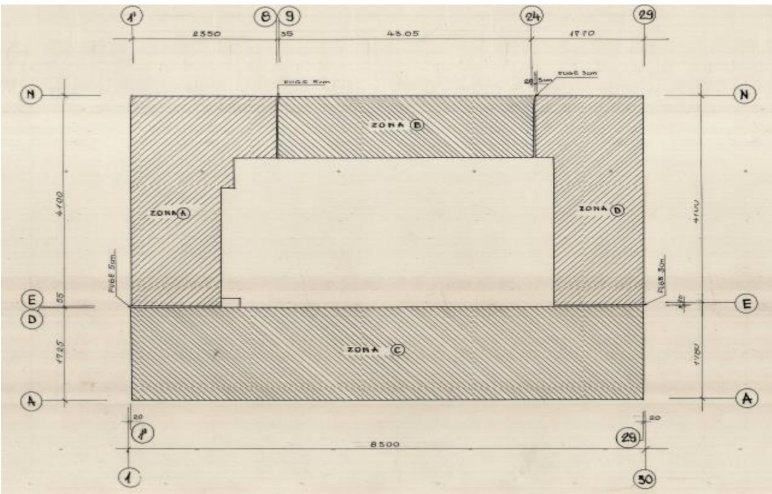


Figure 1. Separation of structures, part of the original project

Structural systems

The structural system consists of the foundation, the vertical structure where the columns belong, the horizontal structure where the beams and floors belong, and the stairs which play not only the connecting role of the floors for the use of the building but are also the only structural connections between the floors as vertical diaphragms are missing. The evaluation of the existing building is presented in the form of a global evaluation of the structure and elements in particular.

2.2 STRUCTURAL SUPPORTING MATERIALS

In the existing structural design, the concrete is M300. Based on Eurocode 2, the concrete of the existing structures evaluated according to the existing project corresponds to the concrete of class C25 / 30. The armature in the existing project has a calculated resistance of 2100daN/cm^2. Referring to the requirements of EC2, the characteristics of structural steels are higher than the steel used in the structure.

2.3 FOUNDATIONS AND GEOTECHNICAL CONDITIONS

The foundations of all buildings are conceived as separate individual (plinth) type foundations connected partly with foundation beams and partially connected with lean concrete walls, in the areas of buildings A and partly C. The bases of the individual footings are positioned in several elevations. Building A and building C partially have a basement. The perimeter walls of the basement are made of lean concrete.

The positions of the individual type foundations are dictated by the geological study and the architectural project. According to the geological study, it is noticed that 3 boreholes with a depth of up to 10.0m were made from the natural ground level. Geological strata are uniform.

In the general assessment of the existing structure, the impact of the soil-structure interaction has been taken into account referring to the parameters of the geological study carried out for the initial project.

The individual footings of the existing buildings are placed in different elevations, namely:

Zone A: The individual footings rest on the elevation -	8.45m, -7.70m.
Zone B: The individual footings rest on the elevation	-5.50m.
Area C: The individual footings rest on the elevation	-8.45m, -7.70m, -4.70m.
Area D: The individual footings rest on the elevation of	4.70m.

It should be noted that the connection with the foundation beam in the existing building does not ensure the connection of the individual footings according to the technical design norms where the connecting beam of the foundation should connect the individual footings at the level of their base. This requirement is also necessary for the fact that the plinth soles are placed in different elevations.

2.4 VERTICAL SUPPORTING ELEMENTS OF THE STRUCTURE

The vertical structure is realized through reinforced concrete columns with cross-sectional dimensions 40x80 cm, 40x100cm and 40x60cm. The reinforcement of the columns is realized with longitudinal rebars and transverse reinforcement, stirrups of Φ8.

The ratio of working longitudinal reinforcement and transverse reinforcement does not meet the requirements for column reinforcement. The construction of cross sections of columns does not meet the requirements for the construction of columns in anti-seismic facilities according to Eurocodes. There are no reinforced concrete shear walls in the structural scheme. The vertical stiffness of the structure is realized through the columns and stairs that connect the floors functionally and structurally.

The construction of the columns does not meet the requirements according to Eurocodes. The combination of longitudinal rebars, the diameter of the stirrups and the dimensioning of the columns do not meet the requirements according to the Eurocodes and based on the structural calculations in some cases the reinforcement is deficient. Referring to the structural calculation, the longitudinal reinforcement of the columns in some cases is deficient, which dictates a need in improvement of the rigidity of the elements and an increase of the longitudinal reinforcement.

2.5 HORIZONTAL SUPPORTING ELEMENTS OF THE STRUCTURE

The horizontal constructive supporting elements of the structure is realized by slabs and beams. The beams are regularly positioned creating simple structural schemes in general for all objects of the building of the National Historical Museum. The beams are monolithic reinforced concrete and the slabs are prefabricated and some are monolithic reinforced concrete.

Beam reinforcement: The evaluation of beams is done based on the technical rules of reinforcement of beam type elements according to the requirements of Eurocode 2 and Eurocode 8.

Analyzing the technical drawings of the beams, it is noticed that in general the percentage of beam reinforcement is within the limits recommended by the European norms for the longitudinal working reinforcement. Constructive reinforcement (plain rebars) in most beams and the combination of rebars does not meet the constructive rules according to Eurocodes. Critical areas are unidentified and stirrups have a uniform distribution along the entire length of the beams. The demand to withstand the shear forces at the beams in their end zones is realized through the bent up longitudinal rebars.

Prefabricated and monolithic slabs generally meet constructive requirements.

The plans of the structures of the existing building are presented referring to the constructive project.

3. TECHNICAL CONDITIONS AND NORMS ON WHICH THE STRUCTURES ARE EVALUATED

The calculation and structural design of the facility is based on the technical conditions of the design according to European design norms, required by the ToR. The design according to European norms is considered necessary as the technical conditions for the design of structures according to Albanian norms have not been renovated and fulfilled for a very long period of time, it is a requirement of ToR where the certification of the facility will be done according to Eurocodes. The technical design conditions according to the Albanian standard will be used only for reference in relation to local values such as. parameters for seismic calculation, wind, snow, environmental conditions, etc. References for constructive calculation are:

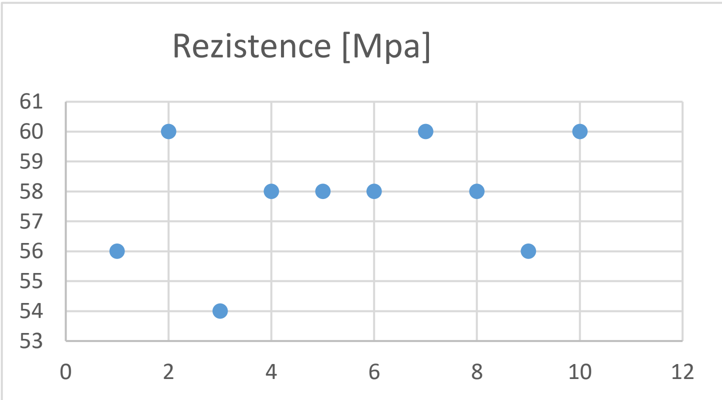
- Eurocode 0, ENV 1991-1: 1994
- Eurocode 1, ENV 1991-2-1: 1995
- Eurocode 2, EN 1992-1-1: 2004 (E)
- Eurocode 7, EN 1997-1
- Eurocode 8, EN 1998-1 (2003)

The evaluation of the existing structures is based on the requirements of Eurocode 8 prEN 1998-3: 200X-2003. The required performance criteria of the structures are evaluated based on the basic requirements of point 2.1. and EC8 prEN 1998-3: 200X-2003. Based on this classification, the 4 structures part of the building of the National Museum are classified as DL. The assessment is based on visual assessment and calculations of existing structures for existing loads. The verification of the structural elements is done based on the classification of the elements as "ductile" and "brittle", point 2.2.1 and 2.2.4 EC8 prEN 1998-3: 200X-2003.

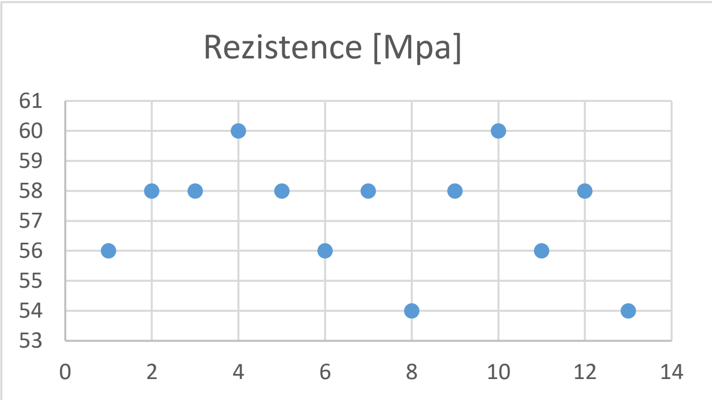
According to the requirements of point 2.2.4 the analytical method of structural controls can be based on linear or non-linear analysis. Capacities are based on the flow limit for all ductile and brittle structural elements. The coefficient of behavior of the structures is determined based on the reduced seismic action by evaluating the presence of masonry. According to EC8 prEN 1998-3: 200X-2003, the level of recognition of structures according to tab.3.1 and point 3.3.2 is KL2, recognition at normal level.

In-situ tests were performed through non-destructive concrete tests. Non-destructive testing is performed by obtaining information about the structure without damaging it. The concrete evaluation was previously performed for the structural elements through tests with the Schmidt hammer. This test is called non-destructive testing and is performed directly on the structure. In these tests, at least 8-10 strokes are performed in an area measuring 15x15cm where the assessment is based on the Gauss_ curve. The evaluation is presented in tabular and graphical form. Specifically, the values obtained from the tests without destroying mainly the columns in random positions distributed in the structure are presented below.

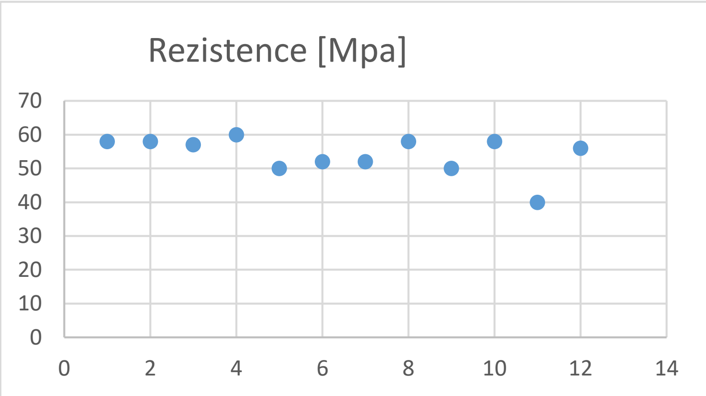
Basement			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 1	1	56	57.80
	2	60	
	3	54	
	4	58	
	5	58	
	6	58	
	7	60	
	8	58	
	9	56	
	10	60	



Basement			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 2	1	56	**57.23**
	2	58	
	3	58	
	4	60	
	5	58	
	6	56	
	7	58	
	8	54	
	9	58	
	10	60	
	11	56	
	12	58	
	13	54	



Basement			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 3	1	58	**54.08**
	2	58	
	3	57	
	4	60	
	5	50	
	6	52	
	7	52	
	8	58	
	9	50	
	10	58	
	11	40	
	12	56	



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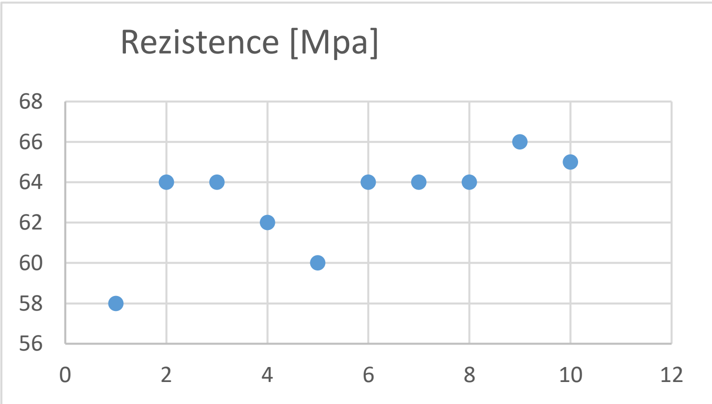
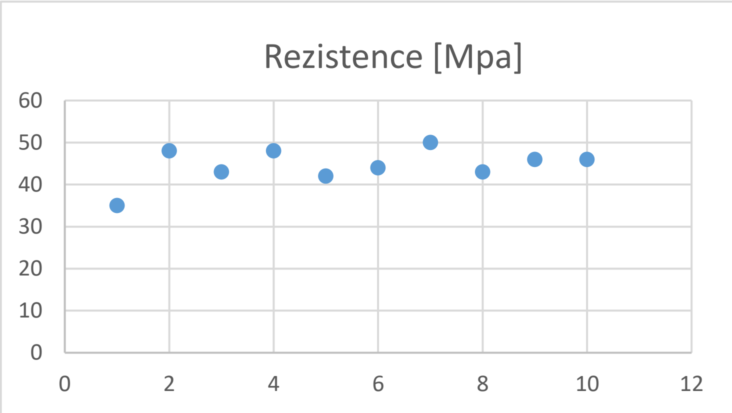
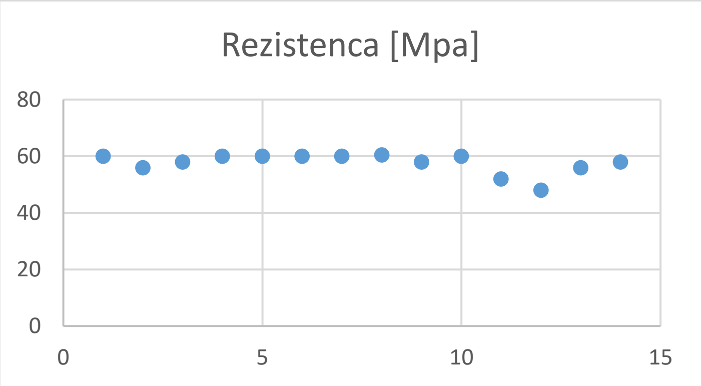
Basement			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 4	1	58	**63.10**
	2	64	
	3	64	
	4	62	
	5	60	
	6	64	
	7	64	
	8	64	
	9	66	
	10	65	



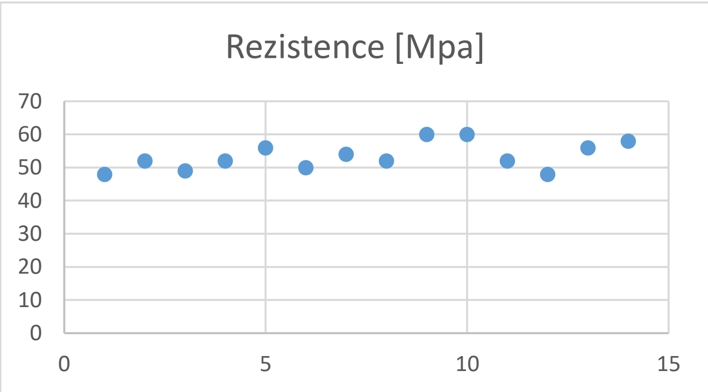
Floor 0.00			
Element	No measurement	Rezistence [MPa]	Average [MPa]
Stairs	1	35	**44.50**
	2	48	
	3	43	
	4	48	
	5	42	
	6	44	
	7	50	
	8	43	
	9	46	
	10	46	



Floor 0.00			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 1	1	60	**57.61**
	2	56	
	3	58	
	4	60	
	5	60	
	6	60	
	7	60	
	8	60.5	
	9	58	
	10	60	
	11	52	
	12	48	
	13	56	
	14	58	

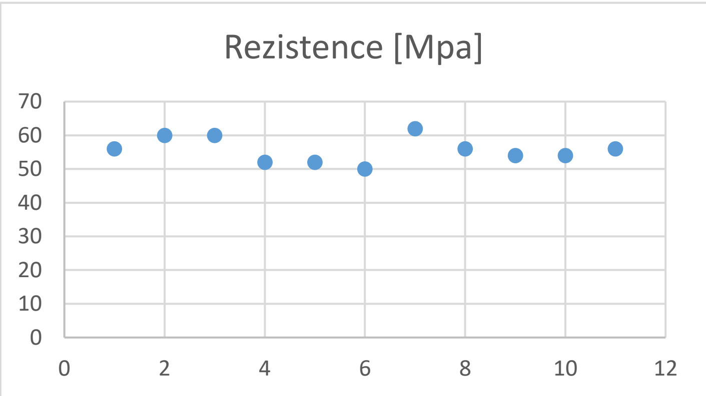


Floor 2			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 1	1	48	**53.36**
	2	52	
	3	49	
	4	52	
	5	56	
	6	50	
	7	54	
	8	52	
	9	60	
	10	60	
	11	52	
	12	48	
	13	56	
	14	58	

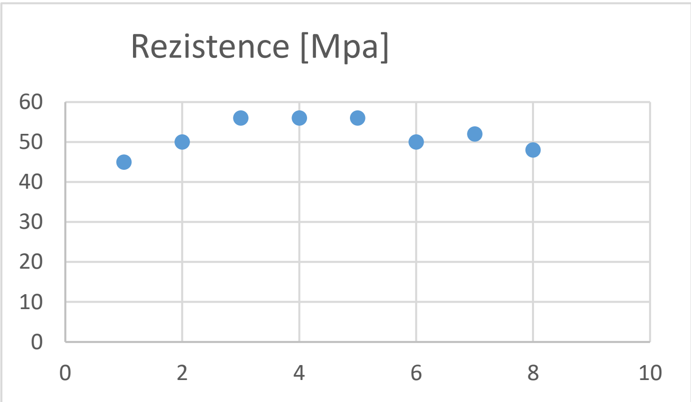


Floor 2			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 2	1	56	**44.00**
	2	44	
	3	44	
	4	44	

Floor 2			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 3	1	56	**55.64**
	2	60	
	3	60	
	4	52	
	5	52	
	6	50	
	7	62	
	8	56	
	9	54	
	10	54	
	11	56	



Floor 3			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 1	1	45	**51.63**
	2	50	
	3	56	
	4	56	
	5	56	
	6	50	
	7	52	
	8	48	



Floor 3			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 2	1	58	**54.64**
	2	58	
	3	56	
	4	54	
	5	60	
	6	56	
	7	49	
	8	50	
	9	48	
	10	56	
	11	56	

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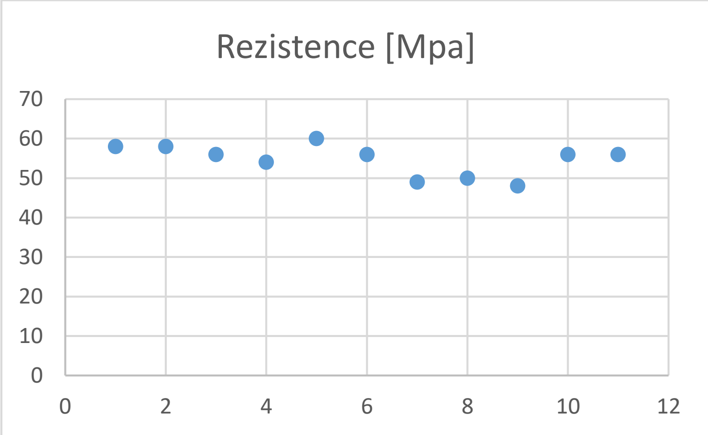
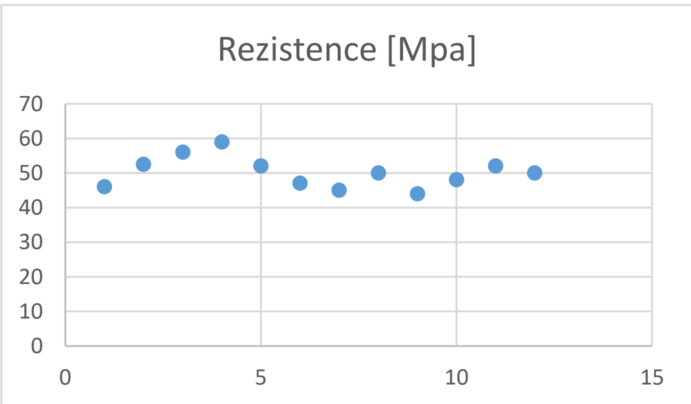
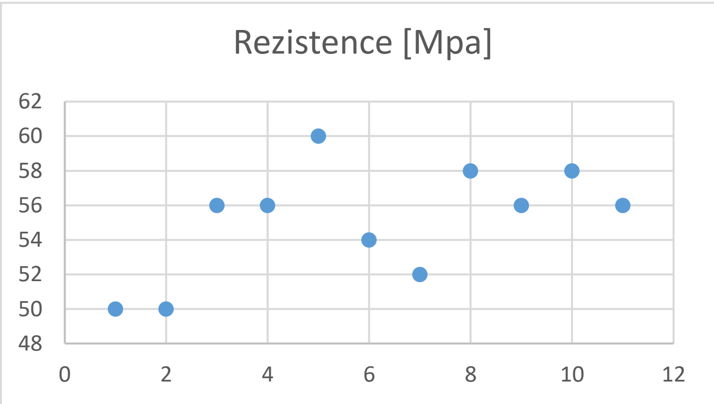


Floor 3			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 3	1	46	**50.13**
	2	52.5	
	3	56	
	4	59	
	5	52	
	6	47	
	7	45	
	8	50	
	9	44	
	10	48	
	11	52	
	12	50	



Floor 3			
Element	**No measurement**	**Rezistence [MPa]**	**Average [MPa]**
Column 4	1	50	**55.09**
	2	50	
	3	56	
	4	56	
	5	60	
	6	54	
	7	52	
	8	58	
	9	56	
	10	58	
	11	56	



No of test	Floor/ Element	Average values
1	Kati 3/Column 1	51.63
2	Kati 3/Column 2	54.64
3	Kati 3/Column 3	50.13
4	Kati 3/Column 4	55.09
5	Kati 2/Column 1	53.36
6	Kati 2/Column 2	44.00
7	Kati 2/Column 3	55.64
8	Kati 0.00/Shkalla	44.50
9	Kati 0.00/Column 1	57.61
10	Basement/Column 1	57.80
11	Basement/Column 2	57.23
12	Basement/Column 3	54.08
13	Basement/Column 4	63.10
Overall average		**53.75**

According to EC8 prEN 1998-3: 200X-2003, the level of recognition of structures according to Tab.3.1 and point 3.3.2 is KL2, recognition at normal level.

Security coefficients for verification of structures according to Tab. 3.3 jane γ_m as EN 1998-1.

4. STRUCTURAL MODELING

The analytical methods recommended in point 4.4 of EC8 prEN 1998-3: 200X-2003 are four. For the evaluation of structures, calculations were performed with two different methods, the multi-modal method of spectral analysis (linear) and the non-linear static method (pushover).

Calculation according to the multi-modal elastic method

The "Tower calculation program" was used for the structural calculation. Structure calculation schemes are three dimensional (3D) which allows the spatial calculation of the structure and taking into account all the factors that actually operate in them. Through the calculation the impact of all vertical and horizontal loads that currently act on building structures is taken into consideration, where we can mention the impact of horizontal wind forces, earthquake forces, temperature change, bending (lowering) of foundations, the impact of vertical forces from different loads (permanent, temporary, special), etc.

Each element in the structure is modeled as a linear prismatic element.

a) The stiffness of the structural elements in bending and shear is taken into account as much as 50% of the stiffness of the uncracked concrete element. The elastic stiffness of the elements in the torsion is as much as 10% of the stiffness of the uncracked element.

b) The vertical elements have larger cross-sectional dimensions in value than the cross-sectional dimensions of the beams in this way the beam-column connection is considered rigid.

c) The width of the slab in the T-shaped elements is taken into account according to EC 2 according to directions X and Y respectively and is at least 20% of the span of the beam.

d) Slabs are considered rigid diaphragms.

e) The individual footings are modeled with Winkler springs evaluating the soil according to the parameters of the geological study.

f) The masses are calculated for gravitational loads with the formula G + QEQ, where $\Psi_E = \varphi\psi_2$, and ψ_2 is determined depending on the category of temporary loads according to Table A1.1. EC0 in EN 1990: 2001

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
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 ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

The calculation of the structure is divided into the following sub-phases:

4.1 Determining the geometry of the structure.

4.2 Determination of calculation parameters of concrete and steel.

4.3 Determination of loads.

4.4 Soil modeling.

4.5 Determination of seismic coefficients.

4.6 Determination of load combination coefficients.

4.7 The calculation of the structure is performed referring to:

a) Calculation according to the first border condition (calculation in bearing capacity) (ULS).

b) Calculation according to the second boundary condition (calculation in the exploitation phase (SLS), determination of deformations and size of cracks opening).

4.1 DETERMINING THE GEOMETRY OF STRUCTURES.

To assess the condition of the existing building, the structure is modeled and dimensioned with the dimensions and real orientation of the structural elements.

The elements in the structural scheme are positioned based on the existing constructive design.

a) The individual footings are positioned referring to the elevation of their base with real dimensions. The foundations are positioned in several elevations. Part of building C is in the basement. Building A has a basement. Buildings B and D do not have a basement.

b) Slabs are of two types, monolithic and pre-fabricated. Pre- fabricated slabs are mainly used. Monolithic slabs are used only in limited areas.

The geometry of the structures is determined based on the constructive design of the existing building. The structural schemes are modeled according to the current condition and positioning of the structural elements. It is noticed that according to the chosen scheme the structure is classified as dual system according to EC8 5.12. (1) prEN 1998-1 (2003) for structural classification.

4.2 DETERMINATION OF CALCULATION PARAMETERS OF CONCRETE AND STEEL.

Structures according to EC0_ENV 1991-1: 1994 (2001) according to Tab.2.1 and EC2_EN 1992-1-1: 2004 (E) according to 4.4.1.2. (5) are classified for design life 100 years of class S5 as they are monumental buildings.

Table 2.1 - Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

4.2.1 Characteristics of concrete

The selection of the concrete class is based on the exposure class of the structure.

a) The exposure class required for plinth evaluation refers to Tab. 4.1, EN 1992-1-1: 2004 (E) according to EC2 and geological report requirements. The foundation exposure class is class XC2 according to EC2.

Table 4.1: Exposure classes related to environmental conditions in accordance with EN 206-1

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

Concrete grade selected for foundation referred to Tab. 4.3N, and Tab. 4.E.1N according to EC2_EN 1992-1-1: 2004 (E) according to the exposure class should be C25 / 30. The concrete grade selected in the structural design of the foundations is M200.

The evaluation for the existing structures is done based on the existing constructive project. The grade of concrete in the existing constructive design for the superstructure is M300 which is converted by estimating the life of the structure to C25 / 30. The compressive strength of concrete produced by in-situ tests indicates higher values than those projected. In the control and calculation, the concrete will be classified C25 / 30 for all of the structures.

Table E.1N: Indicative strength classes

	Exposure Classes according to Table 4.1									
Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage to Concrete							
	No risk	Freeze/Thaw Attack			Chemical Attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45

b) Superstructure exposure class refers to Tab. 4.1, EN 1992-1-1: 2004 (E) according to EC2.

Superstructure exposure class is selected class XC3.

EN 1992-1-1:2004 (E)

Table 4.1: Exposure classes related to environmental conditions in accordance with EN 206-1

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

The concrete class according to the exposure class for the superstructure refers to Tab. 4.3N, and Tab. 4.E.1N according to EC2_EN 1992-1-1: 2004 (E) for the recommended values.

Table E.1N: Indicative strength classes

	Exposure Classes according to Table 4.1									
Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage to Concrete							
	No risk	Freeze/Thaw Attack			Chemical Attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45

It is noted that the concrete grade for the existing structure is M300 according to the existing construction design. Calculations for the evaluation of the structure are made with concrete grade C25 / 30 according to the classification for concrete grades based on EC2_EN 1992-1-1: 2004 (E).

The concrete grade of the superstructure for the calculations of the evaluation phase is C25 / 30 for all objects. Referring to Tab. 4.E.1N according to EC2_EN 1992-1-1: 2004 (E) the recommended grade of concrete should be C30 / 37. Reinforcements will be made of C30 / 37 concrete.

Tab.3.1 Characteristics of concrete according to EC2_EN 1992-1-1: 2004 (E)

	Strength classes for concrete														Analytical relation / Explanation
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30\times f_{ck}^{(2/3)} \leq$C50/60 $f_{ctm}=2,12\cdot\ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7\times f_{ctm}$ 5% fractile
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3\times f_{ctm}$ 95% fractile
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	see Figure 3.2 ε_{c1} (‰) $= 0,7 f_{cm}^{0,31} < 2,8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	see Figure 3.2 for $f_{ck} \geq 50$ Mpa ε_{cu1}(‰)$=2,8+27[(98-f_{cm})/100]^4$
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{c2}(‰)$=2,0+0,085(f_{ck}-50)^{0,53}$
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.3 for $f_{ck} \geq 50$ Mpa ε_{cu2}(‰)$=2,6+35[(90-f_{ck})/100]^4$
n	2,0									1,75	1,6	1,45	1,4	1,4	for $f_{ck} \geq 50$ Mpa $n=1,4+23,4[(90-f_{ck})/100]^4$
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{c3}(‰)$=1,75+0,55[(f_{ck}-50)/40]$
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	see Figure 3.4 for $f_{ck} \geq 50$ Mpa ε_{cu3}(‰)$=2,6+35[(90-f_{ck})/100]^4$

Table 3.1 Strength and deformation characteristics for concrete

EN 1992-1-1:2004 (E)

There are no fire protection areas in the existing building project. Fire protection areas in the reinforcement project will be assessed according to the protection time classification based on EC2 recommendations.

Concrete protective layers:

According to EC2 the protective layers of the reinforcement in the structural elements are calculated:

$c_{nom} = c_{min} + \Delta c_{dev}$

The minimum protective layer price must provide:

-transmission of forces

-protection of steel from corrosion

-fire resistance

Protective layers for superstructure elements must meet the requirements referred to in Tab. 4.2. and Tab. 4.4N according to EC2_EN 1992-1-1: 2004 (E) for minimum values

Table 4.2: Minimum cover, $c_{min,b}$, requirements with regard to bond

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

EN 1992-1-1:2004 (E)

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

c_{min} = max {$c_{min,b}$; $c_{min,dur}$ + $\Delta c_{min,\gamma}$ - $\Delta c_{min,st}$ - $\Delta c_{min,add}$; 10mm}

c_{min} = max {30; 25+ 0- 0 - 0; 10mm}; c_{min} = max{30; 25; 10mm}

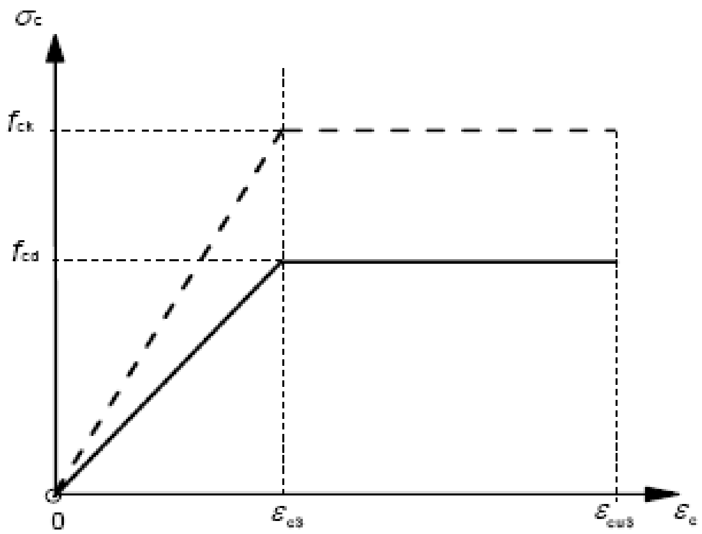
Δc_{dev} = 0mm [national annex],

according to EC2_EN 1992-1-1: 2004 (E) ref. 4.4.1.3. The value is calculated as 10mm if there are no other recommendations:

c_{nom} = c_{min} + Δc_{dev} = 30 + 0 = 30mm

Protective layers should be 30mm for the superstructure. However, it is noticed that the protective layers of the structural elements are generally 25mm, even for the foundations they are 25mm.

The idealized behavior diagrams of the material according to EC2 for concrete according to the requirements of Eurocode 2 are:



Ideal diagram for ordinary concrete

4.2.2 Characteristics of steel

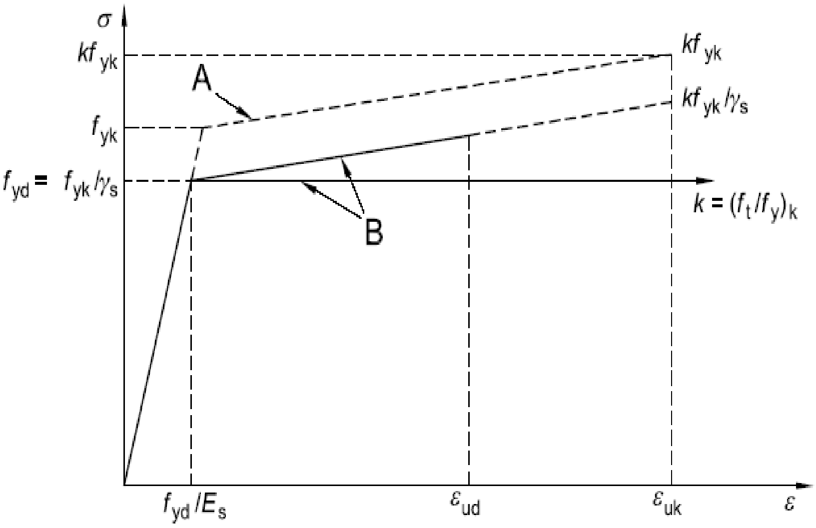
Steel reinforcement is evaluated according to the classification of tab: C.1 EC2. The existing constructive design of the structures presents the reinforcement with calculated resistance 2100kg/cm^2. Referring to the norms of the Eurocodes, the steel reinforcement is classified according to the classes based on "Table C.1: Properties of reinforcement", EC2.

Table C.1: Properties of reinforcement

Product form	Bars and de-coiled rods			Wire Fabrics			Requirement or quantile value (%)
Class	A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600						5,0
Minimum value of $k = (f_t/f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic strain at maximum force, ε_{uk} (%)	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0
Bendability	Bend/Rebend test			-			
Shear strength	-			0,3 A f_{yk} (A is area of wire)			Minimum
Maximum deviation from nominal mass (individual bar or wire) (%) — Nominal bar size (mm) ≤ 8 / > 8	± 6,0 ± 4,5						5,0

It is noted that the reinforcement of existing structures based on the requirements of tab. C.1. EC2 does not meet EC2 recommendations (Referring to Tab. 4.1, En 1992-1-1: 2004 (E))

The idealized behavior diagrams of EC2 material for steel according to the requirements of Eurocode 2 are:



Ideal diagram for ordinary steels

The partial safety coefficients for the materials for the calculation at the finite boundary condition are estimated based on the recommendations of Tab.2.1N, EC2_EN 1992-1-1: 2004 (E), according to EC2.

Table 2.1N: Partial factors for materials for ultimate limit states

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

The partial safety coefficients for the materials for the calculation in the state of service are estimated based on the recommendations of point 2.4.2.4 (2) note according to EC2_EN 1992-1-1: 2004 (E) where their values are taken into calculations equal to 1.0. Based on the existing design the characteristics of the steel reinforcement used are presented only with the value of the resistance at the flow limit.

4.3 DETERMINATION OF LOADS.

For the assessment of the existing situation the loads are determined based on the classification of loads according to Eurocodes EC1 and their combination according to EC0.

a) The values of the dead loads are determined referring to the parameters according to EC1 for the preliminary evaluation.

• Reinforced concrete is taken into account with a volume weight of 25kN/m^3.

• The layer load in the soles is taken into account 3 kN/m^3.

• The load of the walls as a load distributed in the soles is taken into account 2 kN/m^2.

• The load of the walls on the beam is calculated 15 kN/m and the terrace parapets 5 kN/m with horizontal trust load 1 kN/m.

b) Temporary loads in the technical report are presented with their values referring to the load uniformly distributed per 1m^2 horizontal surface. Loads refer to Tab.6.1, 6.7, 6.2, 6.8. according to EC1 prEN 1991-1-1: 2001. Temporary load (live load) is taken for structural calculations as:

Service facilities, category C3 for museum objects, qk = 3 kN/m^2 and Qk = 4 kN for the minimum values recommended for the evaluation of the existing structure. The parameters are selected from the following tables:

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Table 6.1 - Categories of use

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	C1: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. C2: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. C3: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. C4: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. C5: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	D1: Areas in general retail shops D2: Areas in department stores

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m²]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

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Table 6.12 - Horizontal loads on partition walls and parapets

Loaded areas	q_k [kN/m]
Category A	q_k
Category B and C1	q_k
Categories C2 –to C4 and D	q_k
Category C5	q_k
Category E	q_k
Category F	See Annex B
Category G	See Annex B

NOTE 1 For categories A, B and C1, q_k may be selected within the range 0,2 to 1,0 (0,5).
NOTE 2 For categories C2 to C4 and D q_k may be selected within the range 0,8 kN/m –to 1,0 kN/m.
NOTE 3 For category C5 q_k may be selected within the range 3,0 kN/m to 5,0 kN/m.
NOTE 4 For category E q_k may be selected within the range 0,8 kN/m to 2,0 kN/m. For areas of category E the horizontal loads depend on the occupancy. Therefore the value of q_k is defined as a minimum value and should be checked for the specific occupancy.
NOTE 5 Where a range of values is given in Notes 1, 2, 3 and 4, the value may be set by the National Annex. The recommended value is underlined.
NOTE 6 The National Annex may prescribe additional point loads Q_k and/or hard or soft body impact specifications for analytical or experimental verification.

4.4 SITE MODELING.

The foundation is made in the form of individual footings (plinths). The following parameters are evaluated in the site modeling:

a) Engineering geological conditions

b) The surface of the base of the foundation

c) Estimation of floor stresses from structural calculations

d) The architectural project of the use of the building

e) Distribution of stresses in the slabs

The site modeling parameters are presented in the technical report by determining the value of the source coefficient according to the Winkler model. The site modeling parameters have been calculated referring to the geological-engineering study referring to the parameters of the supporting layer of the floor.

4.5 DETERMINATION OF SEISMIC COEFFICIENTS.

a) Referring to the seismological study, the maximum acceleration (PGA) is a_{gR} = 0.248g for the horizontal acceleration for a probability of 10% / 50 according to "Seismicity, seismotechnics and seismic risk assessment in Albania" Sh. Aliaj etj. , according to EC8 prEN 1998-1 (2003) 4.2.5 point 5 (P) $\gamma I = 1.2$.

$a_g = \gamma I \cdot a_{gR} = 1.2 \cdot 0.248g = 0.2976g$ for the horizontal design acceleration for the type C ground, (seismological ratio reference), we select the average ductility degree, DCM.

b) The calculation uses the elastic spectrum of type 1 according to EC8 prEN 1998-1 (2003).

The following expressions are used to define the horizontal elastic spectrum:

$0 \le T \le T_B$ $S_e(T) = a_g \cdot S \cdot [1 + T \cdot (\eta \cdot 2.5 - 1) / T_B]$

$T_B \le T \le T_C$ $S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$

$T_C \le T \le T_D$ $S_e(T) = a_g \bullet S \bullet \eta \bullet 2.5 \bullet [TC / T]$

$T_D \le T \le 4s$ $S_e(T) = a_g \bullet S \bullet \eta \bullet 2.5 \bullet [T_C \bullet T_D / T^2]$

Where a_g is the designed ground acceleration of type A ($a_g = \gamma I \cdot a_{gR}$) for quenching η = 5%. Referring to the geological report, the land can be classified as category C land according to EC8.

According to the recommendations of EC8 prEN1998_1_dec2003 tab.3.2 the calculation parameters for type 1 of the elastic spectrum are:

• The site in the calculation is of type C, (PGA) a_{gR} = 0.2976g type 1 of the elastic spectrum of EC 8 with S = 1.15 for the type C plot for the horizontal design spectrum and T_B (s) = 0.20sec, T_C (s) = 0.60sec, T_D (s) = 2.0sec η = 1.0 for 5% damping according to the technical report of the seismological study.

• The vertical component is not considered as the conditions required by EC8 are not met.

• Seismic design according to Eurocode 8 is for DC M (Medium).

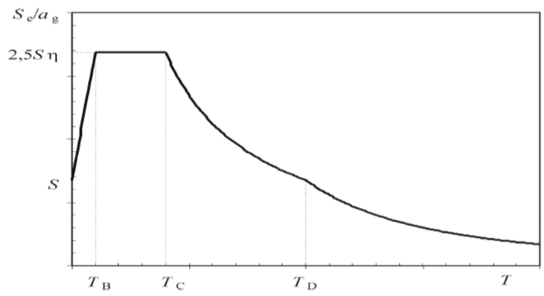


Figure 1 The shape of the elastic reaction spectrum of the structure, according to EC 8

Table 3.2: Values of the parameters describing the recommended Type 1 elastic response spectra

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

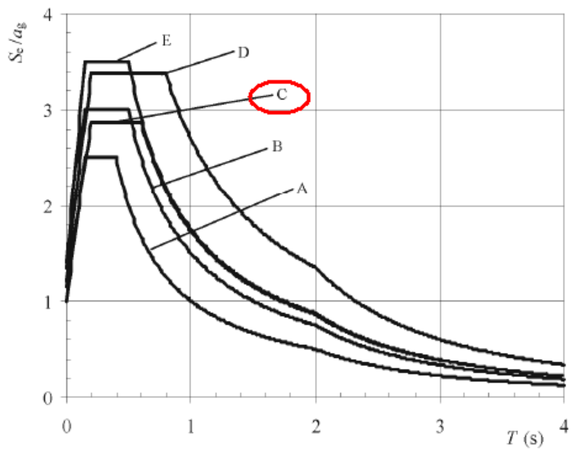


Figure 3.2: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

According to Fig 3.2 EC8 the design spectrum according to EC 8 for horizontal components, is calculated with the expressions:

$0 \leq T \leq T_B$ $\qquad S_d(T) = a_g \cdot S \cdot [2/3 + (T/T_B)(2.5/q - 2/3)]$

$T_B \leq T \leq T_C$ $\qquad S_d(T) = a_g \cdot S \cdot 2.5/q$

$T_C \leq T \leq T_D$ $\qquad S_d(T) = a_g \cdot S \cdot (2.5/q) \cdot [T_C / T]$

$S_d(T) \geq \beta \cdot a_g$

$T_D \leq T$ $\qquad S_d(T) = a_g \cdot S \cdot (2.5/q) \cdot [T_C \cdot T_D / T^2]$

$S_d(T) \geq \beta \cdot a_g$

β = 0.2 according to the value recommended by EC8 for 1998-1 (2003)

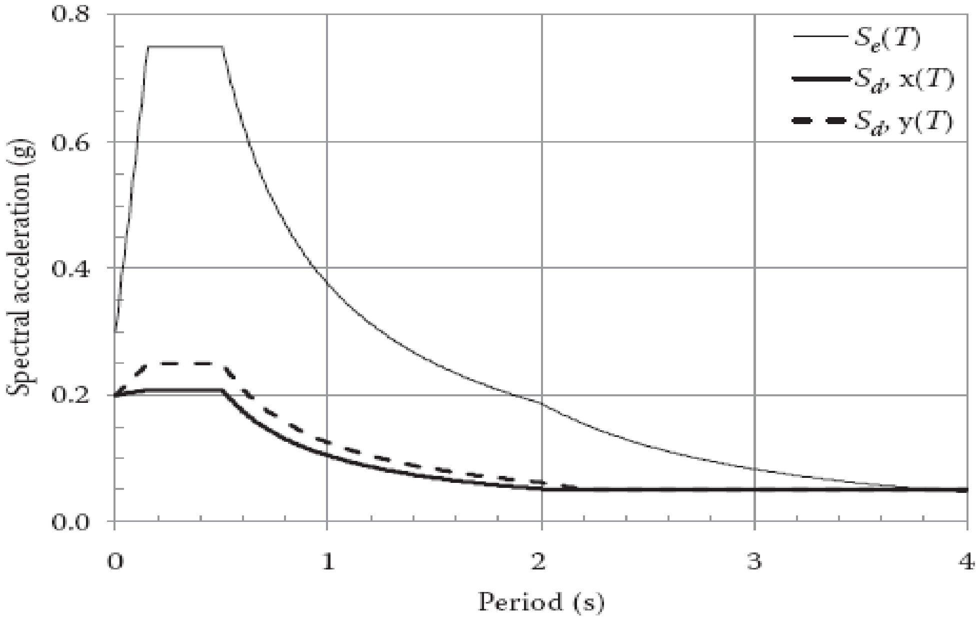


Figure 5 Projection spectrum in X and Y with 5% damping

According to tab.4.3 EC8 prEN1998_1_dec2003, the object is classified in the class of importance III, the coefficient of importance of the object is recommended in the value γ_I = 1.2 according to EC8 prEN 1998-1 (2003) 4.2.5 point 5 (P).

prEN 1998-1:2003 (E)

Table 4.3 Importance classes for buildings

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

NOTE Importance classes I, II and III or IV correspond roughly to consequences classes CC1, CC2 and CC3, respectively, defined in EN 1990:2002, Annex B.

Structure behavior coefficient

The coefficient of behavior of the structure according to 5.2.2.2 EC8 prEN 1998-1 (2003) is based on the classification of the type of structure. Static calculation is performed with the method of lateral forces according to EC8 to determine the ratios in % of the shear force that the wall withstands at the base in relation to the total according to the two orthogonal directions.

V_{wall}, X / V_b, X according to X and V_{wall}, Y / V_b, Y according to Y.

The structure as a whole is classified into frames structures according to section 5.2.2.1. "Structural type" of EC8 prEN1998_1_dec2003. The coefficient of behavior of the structure is:

$q = q_0 k_w \geq 1.5$

The ductility class is DCM.

$q_0 = 3.0\ \alpha_u / \alpha_1$

$\alpha_u / \alpha_1 = 1.3$ according to the X direction

$\alpha_u / \alpha_1 = 1.3$ according to the Y direction

The structures are not regular in plan, they are regular in height according to EC8.

α_u / α_1 = average (1 and 1.3) = 1.15. [Due to irregularity in the plan]. The value $\alpha_u / \alpha_1 = 1.15$ was selected referring to the recommended limit values.

Referring to Fig.4.1. EC8 prEN1998_1_dec2003 criteria (b) and (d) are checked and evaluated according to the two main orthogonal directions of the structure plan.

The structure is considered rather irregular in height as assessed by the recommendations of Fig.4.1. EC8 prEN1998_1_dec2003. Due to this the coefficient of behavior of the structure is reduced.

$q_0 = 3.0 * 1.15 = 3.45$

kw = 1.00 according to 5.2.2.2. point 11 (P) EC8 prEN 1998-1 (2003).

$q = 1 * 3.45 = 3.45$

In judging the structure in the calculation, the coefficient of behavior is taken q = 3.15 for all structures.

Accidental torsional effects

Accidental torsional effects are estimated according to EC 8 in the measure eai = 0.05Li where Li is the plane dimension of the perpendicular object with the direction of seismic action.

4.6 DETERMINATION OF LOAD COMBINATION COEFFICIENTS.

The calculation and design of structures is carried out according to the finite boundary condition (ULS), static equilibrium (EQU), structural element design (STR), ground structure interaction and ground strength (GEO) and service phase calculation (SLS). Reference EC0 (A1.3.) (2001)

Load combination coefficients refer to Tab.A1.1 EC0 EN 1990_FinalDraft_July2001.

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
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 ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

The results are presented in the following computer calculation format.

4.7 STRUCTURAL CALCULATIONS

The structural calculations were performed for control in:

I) Structural calculations in the linear work phase

a) Calculation according to the Ultimate Limit State (ULS) capacity. Reference EC0 (A1.3.) (2001).

b) Calculation according to Serviceability Limit State, SLS (determination of deformations and size of crack openings). Reference EC0 (A1.4.) (2001) for each structural members.

The limit values in the calculation in the service phase are presented in Tab.7.1.N of EC2.

The calculation in the phase of use or service is performed by meeting the criteria of 4.3.1. and EC8 prEN1998_1_dec2003.

Note: The value of w_{max} for use in a Country may be found in its National Annex. The recommended values for relevant exposure classes are given in Table 7.1N.

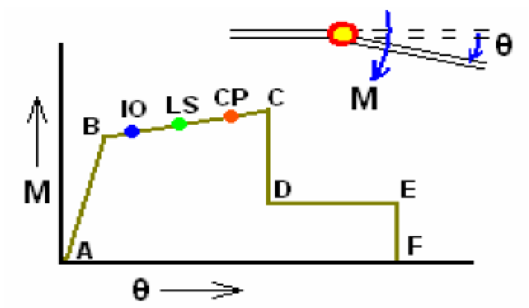
Table 7.1N Recommended values of w_{max} (mm)

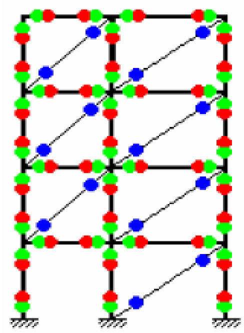
Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XS1, XS2, XS3		Decompression

II) Calculation of structures in the non-linear work phase with the static pushover method for global structure analysis.

Pushover calculation of the structure consists in non-linear calculation of the structure by determining the positions and characteristics of possible plastic hinges by determining their characteristics in relation to the controls against the internal forces of the structure. The calculations are performed with the Tower calculation program which takes into account the structure in the linear phase and then the calculation is performed in the non-linear phase, pushover method.

Pushover analysis of the structure is performed by classifying plastic hinges for beams as [●] hinges from the action of bending moment, for columns as [● + ●] hinges from the action of normal force and bending moment graphically expressing the relationship between the roof top deformations of the object and the base shear force of the structure. For all structural elements, plastic hinges [●] have been evaluated by the action of shear force.

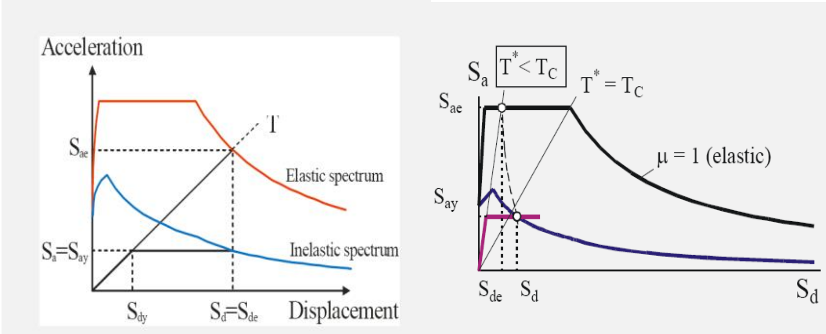


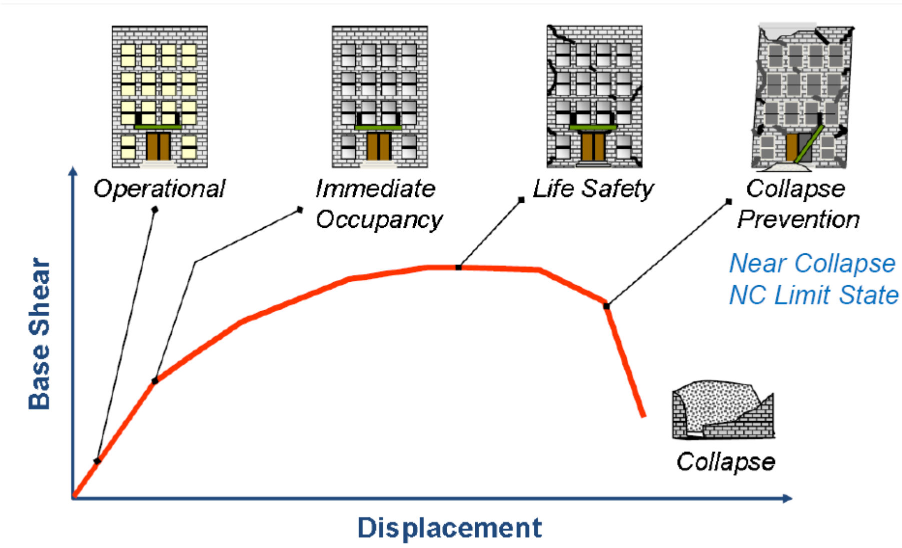


Characteristics of plastic hinges in positions IO (immediate occupancy), LS (life safety), CP (collapse prevention)

- ● Hinge from bending moment action
- ● Hinge by action of shear force
- ● Hinge by action of axial force

The characteristics of plastic hinges are based on the values according to FEMA 356, ATC-40 and FEMA 440.



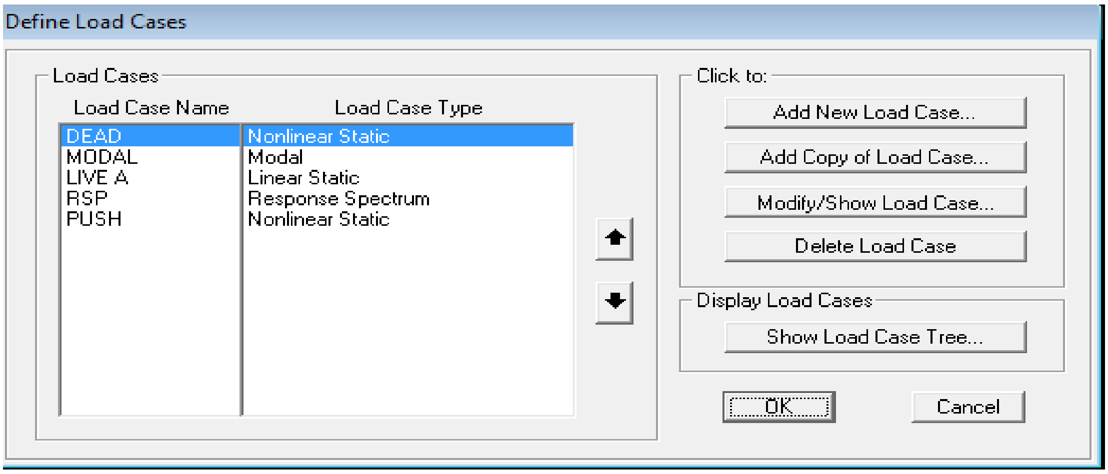


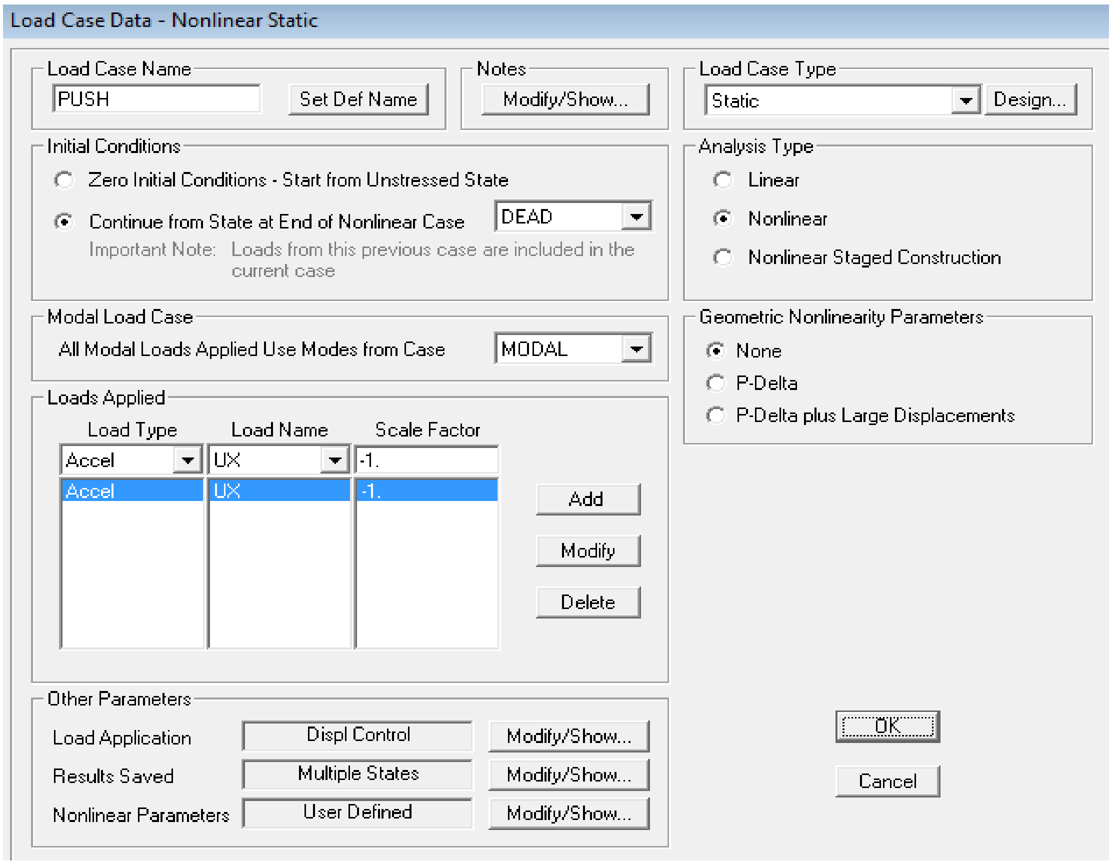
Global capacity of the structure depending on the top displacements and the base shear force.

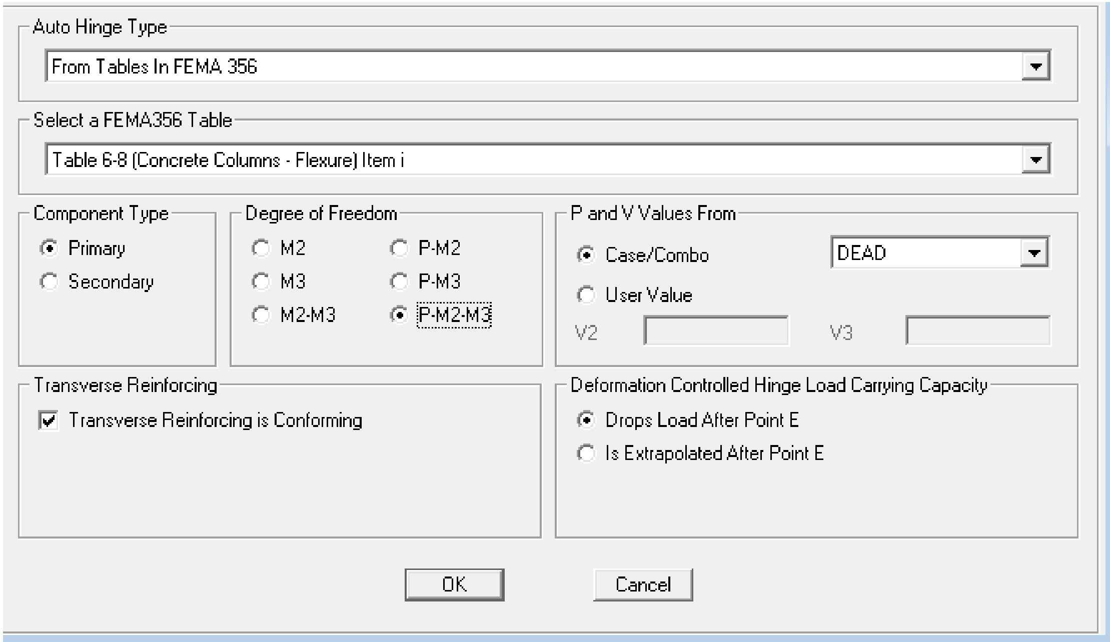
Performance point determination is based on ATC-40.

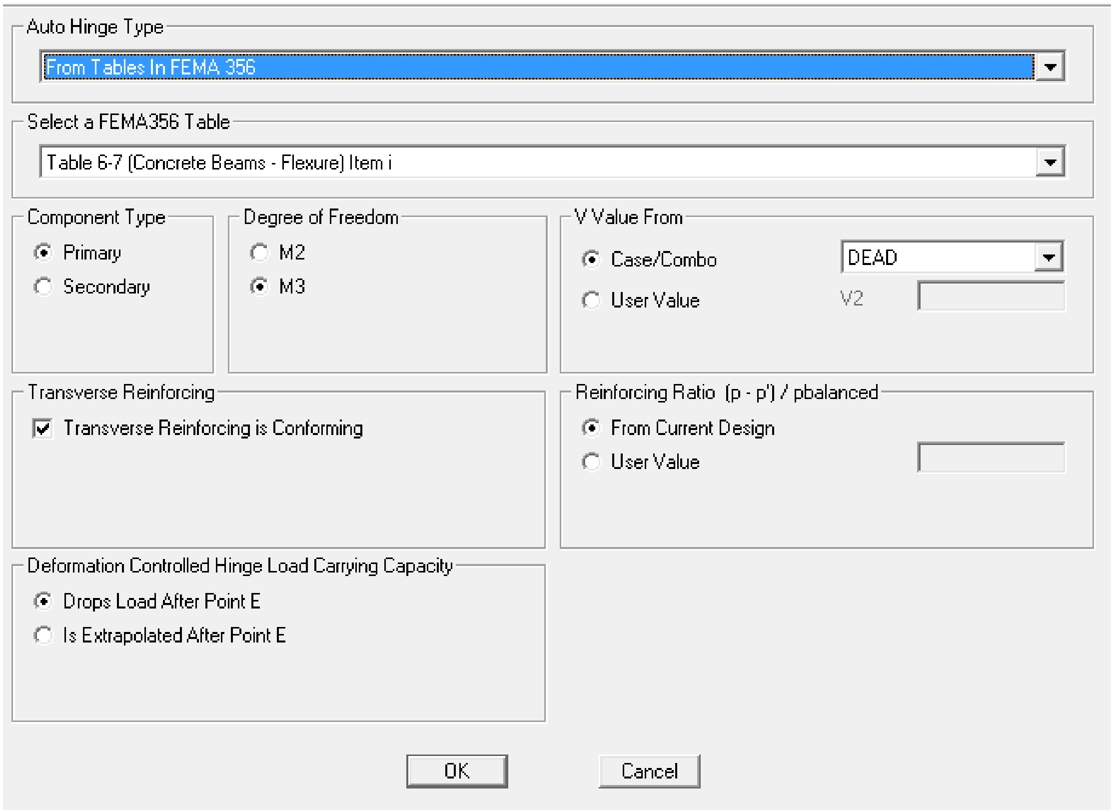
Detailed calculation with the pushover method precisely defines the points of intervention according to the exploitation scheme based on the creation of plastic hinges.

The parameters for structural calculations in the nonlinear phase are presented in the following tables.


Define Load Cases
Load Cases
Load Case Name
Load Case Type
DEAD Nonlinear Static
MODAL Modal
LIVE A Linear Static
RSP Response Spectrum
PUSH Nonlinear Static
Click to:
Add New Load Case...
Add Copy of Load Case...
Modify/Show Load Case...
Delete Load Case
Display Load Cases
Show Load Case Tree...
OK
Cancel


Load Case Data - Nonlinear Static
Load Case Name
PUSH
Set Def Name
Notes
Modify/Show...
Load Case Type
Static
Design...
Initial Conditions
Zero Initial Conditions - Start from Unstressed State
Continue from State at End of Nonlinear Case
DEAD
Important Note: Loads from this previous case are included in the current case
Analysis Type
Linear
Nonlinear
Nonlinear Staged Construction
Modal Load Case
All Modal Loads Applied Use Modes from Case
MODAL
Geometric Nonlinearity Parameters
None
P-Delta
P-Delta plus Large Displacements
Loads Applied
Load Type
Load Name
Scale Factor
Accel UX -1.
Accel UX -1.
Add
Modify
Delete
Other Parameters
Load Application
Displ Control
Modify/Show...
Results Saved
Multiple States
Modify/Show...
Nonlinear Parameters
User Defined
Modify/Show...
OK
Cancel





A minimum of 500 push steps have been performed for each structure in the directions of the global horizontal axes and the creation of plastic hinges has been evaluated in function of the calculation parameters selected for the Tirana area. It is noticed that LS plastic hinges are created in the floor columns -1 and 0. For this reason it is necessary to reinforce with reinforced concrete jacketing of the columns in these positions. On the other floors the reinforcement and dimensioning of the columns is sufficient for the calculation phase.

Evaluation of existing structures .:

Existing structures are analyzed referring to:

1. Measurement of the existing structural elements with the real scheme according to the parameters of the existing project.

2 Site parameters are taken into account according to the existing geological report.

3. The facilities are loaded with operating loads according to EC1 and combinations of loads according to EC0, the class of concrete and steel is taken into account according to the characteristics presented in the existing technical design. The building on the ground will be certified according to Eurocodes, request of ToR.

4. All the details of the connecting joints, the characteristics of the materials, the classification of the mechanisms as ductile and brittle or (the shear mechanism of the columnar joints), the deformed stirrups by checking all the angles of rotation in the joints of the elements have been checked. for all loading cases according to the scheme of point A.2. of EC8 prEN 1998-3: 200X-2003 as well as point A.3.1.3. for verification in the second border situation.

Evaluating the results of the calculation for the current state of the structure, it is noticed that the structures need to be strengthened.

a) Reinforcement of structures is an intervention in the structural system of a building to increase the resistance to seismic loads by optimizing the stresses and ductility under the action of seismic loads according to the technical conditions at the time of certification.

b) Reinforcement of structures is necessary as the structural capacity to withstand loads is insufficient.

The factors that are considered in the decision to strengthen the structures are:

- The technical aspect related to the testing of the structure through the evaluation of the existing project for the current state of use.

- The importance of the building. The "National Historical Museum" has the status of Cultural Monument of the second category.

For this reason:

• The class of concrete in the existing construction project is M300, which is converted by estimating the life of the structure, to C25 / 30 according to table 4.E.1N referring to EC2_EN 1992-1-1: 2004 (E).

Referring to the exposure class according to Tab. 4.E.1N, EC2_EN 1992-1-1: 2004 (E) The recommended concrete grade should be C30 / 37 for the building.

Reinforcement of structures will be realized with concrete of class C30 / 37 minimally.

• It is noticed that the reinforcement of the existing structures is smooth unribbed rebars with tensile strenght of 2100 kg/cm^2. Based on the requirements of tab. C.1. EC2 steel reinforcement does not meet the recommendations of EC2 (Referring to Tab. 4.1, En 1992-1-1: 2004 (E)). The reinforcement of the structures will be realized with steel of class S500B.

• Protective cover of existing structural elements do not meet EC2 requirements based on exposure classes. During the process of repairing the structures in the structural elements, the protective layers of concrete will be completed to ensure the defined longevity of the structure.

• Existing columns have insufficient dimensions to ensure the stiffness of the building, a phenomenon observed in the basement floor and floor 0. Plastic hinges in some cases are created earlier in columns than in the beam, endangering the safety of the structure.

• The construction of the existing columns does not meet the requirements according to Eurocodes. The combination of longitudinal working rebars, the diameter of the staffs and the dimensioning of the columns do not meet the requirements according to the Eurocodes and based on the constructive calculations the armature is deficient. Referring to the calculation, the longitudinal reinforcement of the columns in some cases is deficient, which dictates their reinforcement through the weighting and completion of the working longitudinal reinforcement.

The calculation in the redesign is performed based on the exploitation scheme according to the project requirements of all disciplines. For this it is necessary to intervene in the structures of the building according to the following scheme:

• Reinforcement of columns in basement floors and 0 will be carried out by increasing the cross-sectional size of columns, according to the recommendations of point A.4.1 of EC8 prEN 1998-3: 200X-2003 by ensuring the connection of existing concrete with new concrete of reinforcement. This intervention is necessary because the armature of the columns is insufficient according to the requirements of Eurocodes and the details of their armament do not meet the requirements of EC8.

• Total reinforcement as reinforcement surface on floors 1 and 2 completes the calculations. There are shortcomings in staff positioning and rebar lengths. For this and due to the high cost, interventions have been decided as jacketing with metal profiles according to A.4.2. EC8 prEN 1998-3: 200X-2003

• Beam reinforcement will generally be carried out at the joints connecting the columns ensuring the stiffness of the joints and in some special cases in the beam spaces. Beam reinforcement at the joints of the columns does not meet the insertion lengths. Reinforcement of the beam joints will ensure the condition of inserting the beam reinforcement in the columns and stiffening the joints. The critical areas of the beams are unidentified and the stirrups have a uniform distribution along the entire length of the beams. The request for reinforcement of the beams in their end zones is realized through the bent up reinforcement.

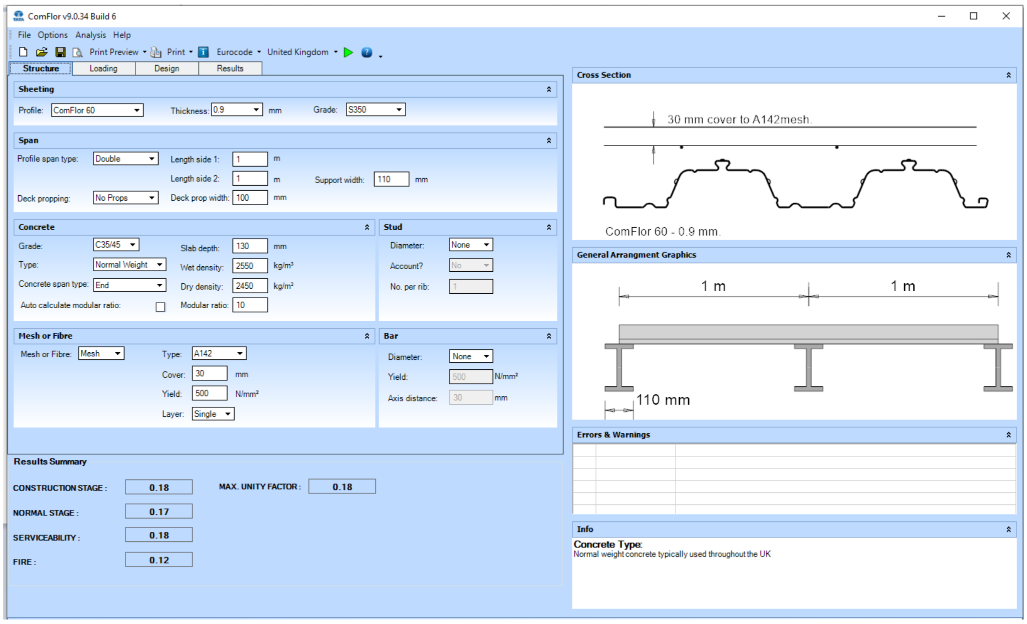
The insertion length (anchorage) of the beam rebars in some cases is insufficient according to the recommendations of the Eurocodes. Enlarging columns by increasing the size of the joints helps. The beams will be inspected and evaluated step by step in place at the joints of the reinforced columns.

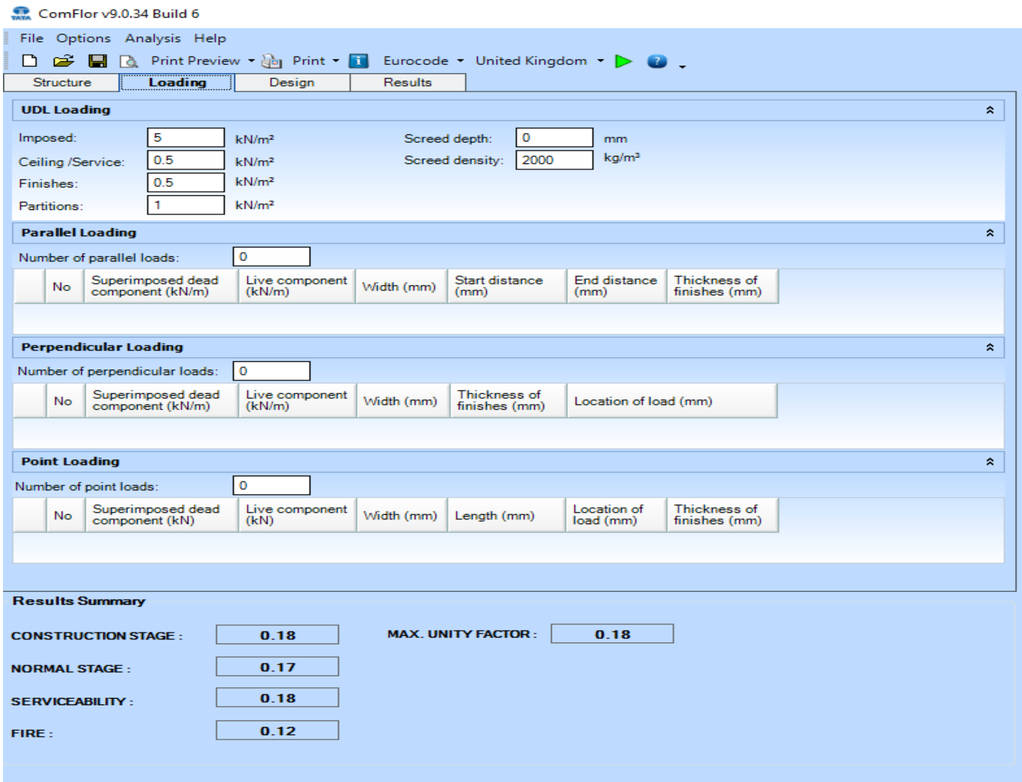
• Pre-prepared slabs should be monolithic in the joints between them. Differentiated reductions of columns and beams have affected the deformed scheme of the slabs.

The following are the calculations of the structures

ANNEX : SECTION NENTOKA SHTESE

Llogaritja e soletave kompozite bazohet ne kerkesat e Eurokodit 4, 1-1-vers 17-11-2004. Llogaritja e soletave kompozite eshte realizuar me programin ComFlor per percatimin e tipit te struktures se llamarines se valezuar ku eshte perzgjedhur tipi ComFlor 60, me trashesi 0.9, me grade S350 per mbrojtje kunder zjarrit 90min. Armimi eshte me rrjete A142.


ComFlor v9.0.34 Build 6
File Options Analysis Help
Print Preview Print Eurocode United Kingdom
Structure Loading Design Results
Sheeting
Profile: ComFlor 60 Thickness: 0.9 mm Grade: S350
Span
Profile span type: Double Length side 1: 1 m
Length side 2: 1 m Support width: 110 mm
Deck propping: No Props Deck prop width: 100 mm
Concrete
Grade: C35/45 Slab depth: 130 mm
Type: Normal Weight Wet density: 2550 kg/m³
Concrete span type: End Dry density: 2450 kg/m³
Auto calculate modular ratio: Modular ratio: 10
Stud
Diameter: None
Account? No
No. per rib: 1
Mesh or Fibre
Mesh or Fibre: Mesh Type: A142
Cover: 30 mm
Yield: 500 N/mm²
Layer: Single
Bar
Diameter: None
Yield: 500 N/mm²
Axis distance: 30 mm
Results Summary
CONSTRUCTION STAGE : 0.18
MAX. UNITY FACTOR : 0.18
NORMAL STAGE : 0.17
SERVICEABILITY : 0.18
FIRE : 0.12
Cross Section
30 mm cover to A142mesh
ComFlor 60 - 0.9 mm
General Arrangment Graphics
1 m
1 m
110 mm
Errors & Warnings
Info
Concrete Type:
Normal weight concrete typically used throughout the UK


ComFlor v9.0.34 Build 6
File Options Analysis Help
Print Preview Print Eurocode United Kingdom
Structure Loading Design Results
UDL Loading
Imposed: 5 kN/m²
Ceiling /Service: 0.5 kN/m²
Finishes: 0.5 kN/m²
Partitions: 1 kN/m²
Screed depth: 0 mm
Screed density: 2000 kg/m³
Parallel Loading
Number of parallel loads: 0
No
Superimposed dead component (kN/m)
Live component (kN/m)
Width (mm)
Start distance (mm)
End distance (mm)
Thickness of finishes (mm)
Perpendicular Loading
Number of perpendicular loads: 0
No
Superimposed dead component (kN/m)
Live component (kN/m)
Width (mm)
Thickness of finishes (mm)
Location of load (mm)
Point Loading
Number of point loads: 0
No
Superimposed dead component (kN)
Live component (kN)
Width (mm)
Length (mm)
Location of load (mm)
Thickness of finishes (mm)
Results Summary
CONSTRUCTION STAGE : 0.18
MAX. UNITY FACTOR : 0.18
NORMAL STAGE : 0.17
SERVICEABILITY : 0.18
FIRE : 0.12

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