STRUCTURAL REPORT OF
KURCAJ BRIDGE

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1 INTRODUCTION

In this report is presented the structural engineering-seismological evaluation of the Kurcaj Bridge, a cultural heritage object. The project of reinforcement/adaptive rehabilitation interventions was realized in accordance to the architectonic and conservation projects and requirements.

Figure1: Location of the bridge

1.1 GENERAL PRINCIPLES

The need for structural rehabilitation
The main reasons for conducting a structural assessment of existing structures are:
Longevity of design/construction of structures (their age);
Current seismic risk assessments;
Changes in design codes from construction time to date;
Various damages that the structures have suffered over the years;
Planned conservation and architectural interventions;

Lifespan of building design (their age)

The bridge that is the subject of this assessment is very old. If we take into account the requirements of the old design conditions, or even those of today's conditions, the design life of structures with widespread use by the public is 50 to 100 years. This lifespan is defined as a period of time during which the structure is able to perform its function without the need for significant structural intervention. The structure is very old, so the need arises for its structural evaluation to judge whether it is necessary to carry out interventions to extend the time of use until future structural interventions. Historic monuments, among others, have been subjected to natural or artificial mechanical, chemical, physical, etc. mechanical actions. These long-term actions result in degradation of the material and building elements.

Today's seismic risk assessments

The first seismic map of Albania dates back to 1952. Since 1952, due to continuous enrichment with data, seismic risk has always been estimated to increase. Here it is important to mention the fact that for the buildings that were built before 1979, the technical conditions were old, but also the seismic zoning map had low values of the seismic intensities of the expected earthquakes.

Recent works (such as that of UNDP Albania and the Academy of Sciences of Albania) show a further increase in values representing seismic risk. Approximately, today's publications estimate the reference acceleration of about 0.25 g on solid ground in the area of the bridge's location for a return period of 475 years. Based on this fact, the bridge needs structural evaluation.

Codes adopted in this report

Eurocodes
Eurocode 0: Basis of structural design
Eurocode 1: Actions on structures
Eurocode 3: Design of steel structures
Eurocode 6: Design of masonry structures
Eurocode 7: Geotechnical design
Eurocode 8: Design of structures for earthquake resistance

European Standards for Structural Steel Product EN 10025

Albanian Standards for seismic design KTP-N.2-89
Standards for existing materials (in-situ testing)

Changes in design codes in Albania

In the field of civil engineering and that of earthquakes very great developments have been made since the time when the existing buildings were designed and implemented. The Technical Design Conditions in force in Albania (KTP-78 and KTP-N.2-89) date back to 1978, so they are already 42 and 31 years old. Although the last upgrade of the KTP-78 made in 1989 (KTP-N.2-89) is a design code that well reflects the requirements of its time, it stands relatively far short of many of the contemporary seismic design requirements. The rules contained in KTP-N.2-89 are more restrictive than those of the previous technical conditions, but the structural Eurocodes, the process for the adoption of which has already begun for our country, express even greater demands for buildings.

In these conditions, it is concluded that buildings that are designed with technical conditions that provide a limited security compared to the technical conditions in force, and even more reduced compared to contemporary requirements (e.g. those of the Structural Eurocodes) should necessarily undergo structural reassessment. Based on this fact, the bridge needs structural evaluation.

2 KURCAJ BRIDGE, STRUCTURE DESCRIPTION

2.1 GENERAL

The structure is a masonry object with stone retaining walls built in the XVIII century. Initially there were three circular vaults (according to the monument card). During the Second World War, the central vault also suffered damage which was restored after the war. The bridge is part of the short road Tiranë-Krujë. The bridge has suffered damage and restoration interventions.

The current situation presents only the central arch with numerous damages. The foundations from the visual observation (terrain part/surrounding layers) do not look damaged or cracked, but during floods the water affects the abutments of the bridge. We have carried out a structural measurement (update) for the whole building and have made an assessment of the degree of vulnerability of structures as well as simulation design and in-depth analysis.

Figure 2: Front photo of the bridge during the measurements bridge

The following figures show the plans of the structures.

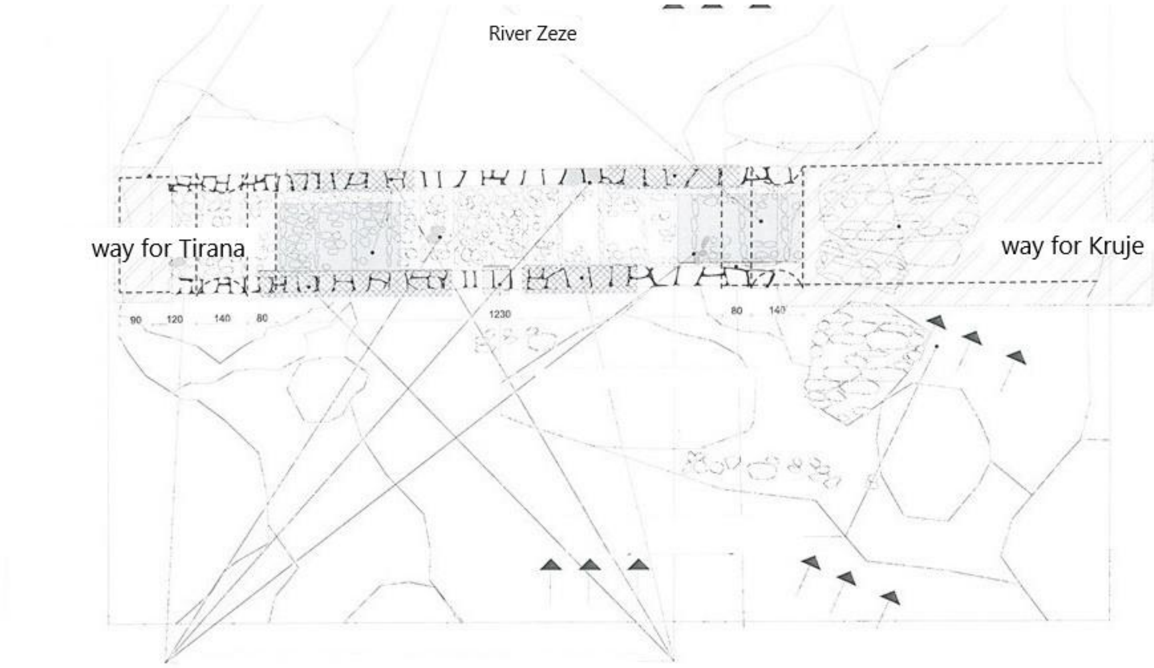

Figure3: Planimetry of previous interventions projects (IMK Archive)

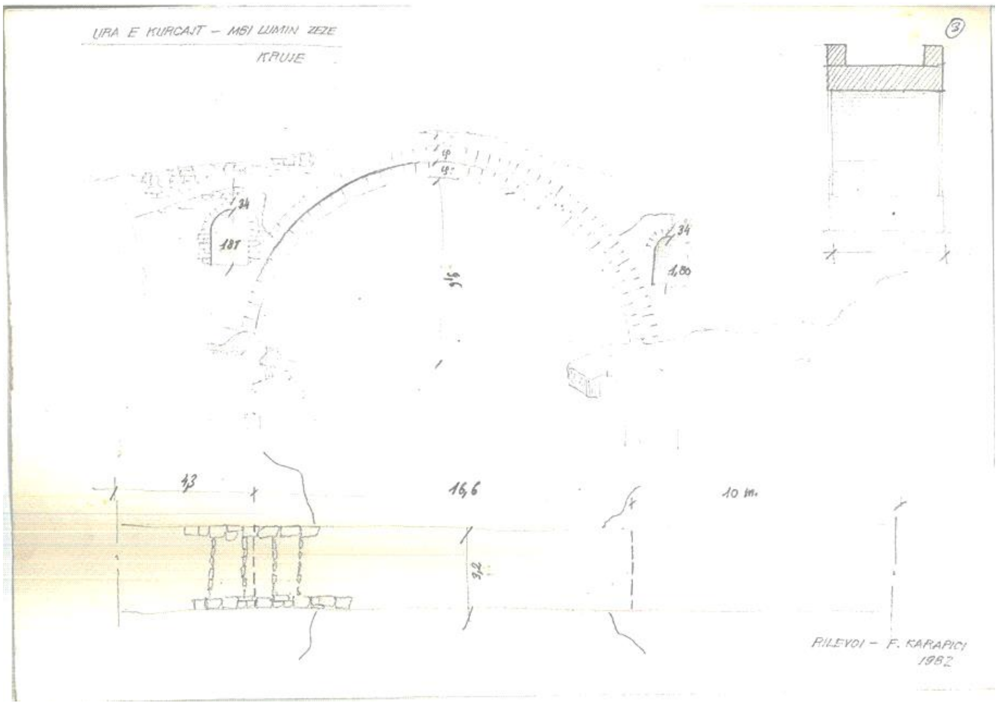

Figure 4: Existing planimetry (IMK Archive)

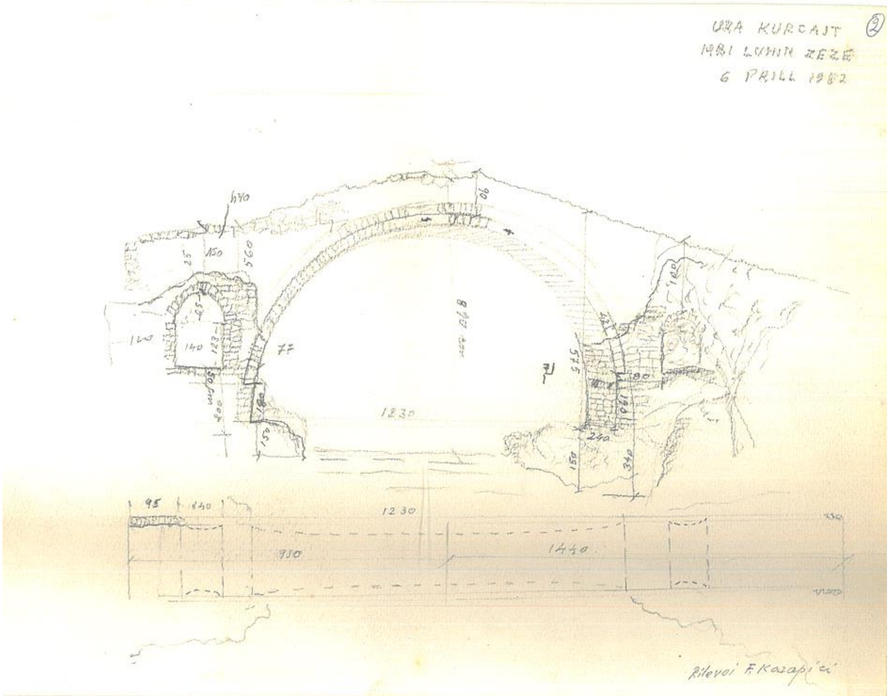

Figure 5: Structural section (IMK Archive)

Figure 6: Photo from the damages found in the project of 2018 (IMK Archive)

2.2 EXISTING SITUATION (FINDING ACT)

The existing condition of the materials and details of the building from observations, tests with half destruction and without destruction results very problematic.

There are cracks in the sides, arches, falling parts in the sides, detachment of the integrity of the elements.

Damage to the bridge is mainly due to temporal degradation and major events such as earthquakes, floods, etc. have contributed to the aggravation of the existing situation.

The following figures illustrate some of the verifications performed. More details of the existing situation are given in the summary tables and drawings of the project.

2.3 GEOMETRIC IDENTIFICATION AND RECOGNITION

The architectural plans were made available to us by the client. Based on it and detailed measurements on site, the planimetry and structural sections of the bridge were realized.
From the measurements in the object all the structural elements are identified. The structural elements were identified by surveillance and thermal cameras. These structural elements are:

- Continuous foundations as a continuation of bridge sides (cannot be verified)
- U-shaped stone masonry with bonded stone filling.
- The sides are irregular masonry with large irregular joints with lime mortar
- Lower arch regular masonry vault with small vertical and horizontal joints with lime mortar

2.4 IDENTIFICATION AND RECOGNITION OF MATERIAL CHARACTERISTIC

Identification

The materials used for the bridge are:

- Full limestone for structural masonry
- Lime mortar of different times. In separate parts it can be considered that it has remained from the initial construction.

Testing

In order to control the current condition of the materials used for the structural elements, tests were performed in the bridge. The tests performed in the site are:

- Sclerometric tests (stones, mortars)
- Sonic tests
- Teste 3D Scanner
- Tests of stone resistance, mortars in the laboratory.
- Chemical tests of stone, mortar in the laboratory.

From the analysis of test data, we have the following results:

Masonry

To determine the level of knowledge as we mentioned before we have to do verifications and tests since these verifications are done only in two positions inside, the values of the tests/recommended will be divided by a safety factor KS = 1.2

For the masonry of the facade, we have
Stone resistance, mortar resistance; f_b=500daN/cm^2, f_m=10-16daN/cm^2
Average masonry resistance; f_m=11 daN/cm^2 (γ_i=1.5)
Boundary resistance of masonry according to EC6; f_k=3.08 daN/cm^2
τ_o = 0.03 daN cm^2
E_m=8000 daN/cm^2
G_m=2900 daN/cm^2
Unit weight ρ=2000 kg/m^3

Correction coefficients are not taken into account because the condition of the mortar is very poor, the connections are not designed correctly and we don't have masonry reinforcement.

For the masonry of the arch, we have
Stone resistance, mortar resistance; f_b=800daN/cm^2, f_m=10-16daN/cm^2
Average masonry resistance; f_m=18 daN/cm^2(γ_i=1.5)
Boundary resistance of masonry according to EC6; f_k=100 daN/cm^2
τ_o=0.15 daN/cm^2
E_m=13000 daN/cm^2
G_m=4800 daN/cm^2
Unit weight ρ=2100 kg/m^3

For the interior (filler) masonry we have
Stone resistance, mortar resistance; f_b=100daN/cm^2, f_m=5-8daN/cm^2 (filling with soil mixture, plant roots, etc.)
Average masonry resistance f_m=5 daN/cm^2(γ_i=1.5)
Boundary resistance of masonry according to EC6 f_k=20 daN/cm^2
τ_o=0.015 daN/cm^2
E_m=1320 daN/cm^2
G_m=220 daN/cm^2
Self-weight, ρ=2000 kg/m^3

Structural Steel

Mechanical properties of steel used in the project is given in the table below.

Table 1. Mechanical properties of steel

Material	Density (kg/m^3)	Brinell Hardness Number	Modulus of Elasticity ($x10^3$ MPa)	Yielding stress (MPa)	Tensile strength (MPa)	Ultimate strain (%)	Coefficient of thermal expansion ($x10^{-6}$ °C)
Iron / Steel for RC elements							
Iron and mild steel	7870	115	196	195	390	35	12
Ductile stainless steel (Hot rolled)	7970	150	196	295	590	10	17
Steel for high strength cables							
High-strength stainless steel (Cold rolled)	7970	300	206	785	980	5	17
Iron / Steel for tie rods, profiles and accessories: Class S275							
Iron and mild steel	7870	115	200	265	410	23	12

2.5 SUMMARY OF STRUCTURAL RECOGNITION

The following tables and figures briefly illustrate structural recognition.

Table 2. Descriptive file of structural construction damage

Summary of Object data	
General	
Name	Kurcaj Bridge
Address	Nikël-Kurcaj-Krujë
Usage	Archeological
Typology	Bridge
Area	-

Date of declaration as monument	Thursday, 15 November 1984
Category of monument	1st Category
Structural system type	Stone masonry retaining walls
Year of construction	XVIII century, Turkish
Construction typology	Massive infrastructural constructions with stone arches
Regularity in plan	-
Regularity in elevation	-
Coverage	Damaged cobblestone
Material	Squared limestone, natural limestone, mortar
Details	Arch with regular stones. Regular stone box with irregular filling. According to the time of construction.
Foundation type	Irregular on the stones of the river bed, continuation of the walls.
Site category	B
Intervention file	
Materials	Existing stones, new mortar, mortar injections
Details	Very poor documentation
Additions from interventions	No info
Additions from the initial project (bracings etc.)	No info
Previous structural interventions	Restoration during years 1970-1980, 2015
Damage form the 26 November 2019 earthquake	
Previous damages	From documentations after major floods
Nonstructural damages	-
Structural damages	Wall and arch cracks, fall of parapet
Damage to the foundation	Do not result
Inclinations	Not noticed
Crash	There are no other buildings nearby
Analysis	
Empirical	Bearing capacity of walls, stability of mural blocks
Simplified Analytical/Numerical	Linear static/dynamic, slope stability
Detailed Numerical	Global nonlinear static/dynamic analysis of walls. Local nonlinear static/dynamic analysis of the work of the elements. Wall slope analysis.
Conclusions	The structure does not respond to seismic demand. The structure requires structural interventions: reinforcement, rehabilitation, adaption.

Table 3. File of restoration and structural interventions

Historical periods	Year of damage	Description of damage	Year of restoration	Description of restoration
Before XX Century				
From XX century to 1990	1940-1944	Arch damage	After 1945	Arch restoration
Actual time (1995-2020)	2018	Damage of the central arch, collapse of the side, river erosion		Unrealized project

Table 4. Summary table of the typology of damage levels

Nr.		Damage description	Yes/No	Origin	Local influence	Global Influence
1		Damage to the wall head closure, damaged joints, joint cavity, damaged stones	V	1, 2	V	V
2		Fallen/damaged parapet	V	1, 2, 3	V	
3		Wall surface detachment/stone movement in the outside/inside face.	V	1, 2, 3, 4, 5	V	V
4		Wall cracks according to joints	V	1, 2, 5	V	V
	4.1	Open crack	V	1, 2, 5	V	V
	4.2	Repaired cracks				
	4.3	Crack of wall panel according to joints	V	1, 2, 5	V	V
5		Cracks of stones	V	1, 4	V	V
6		Damage of arch, stone parapet/damage of French ceiling	V	1, 2, 3, 6	V	V
	6.1	Without repairs				
	6.2	With repairs	V	1, 2, 3, 6	V	V

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7		Arch damage				
8		Arch damage (door)	V	1, 2, 5	V	V
9		Wall crack, kinematic mechanism, out of plan	V	1,2,3,5	V	V
10		Mold, low plants	V	1,2	V	
11		Superficial joint damage	V	1,2	V	
12		Falling stones from the walls	V	1,2,4,5	V	V
	12.1	Falling stones from the inner face	V	1,2,4,5	V	V
	12.2	Falling stones from the outer face				
13		Wall panel destruction	V	1,2,4,5	V	V
	13.1	Partial destruction of wall panel elements	V	1,2,4,5	V	V
	13.2	Total destruction of wall panel elements	V	1,2,4,5	V	V
14		Calcification of plasters/joints	V	1,2	V	
15		Oxidation of the outer surface of stones	V	1,2	V	

Where indices represent

Nr.	Origin
1	Atmospheric conditions
2	Time degradation
3	Artificial mechanical damage
4	Natural mechanical damage
5	Earthquake damage
6	Wrong restoration
7	Other

3 STATIC AND SEISMIC RISK ASSESSMENT OF THE EXISTING STRUCTURE

For the evaluation of the structure, we have performed the following analyzes:

1. Static analytical analysis
2. 3D static and dynamic linear analysis
3. 3D static nonlinear analysis.

The following are the details of the analysis and the data needed to perform them.

3.1 REQUIRED INPUT DATA

Structural assessment information should include the following points.

a) Identification of the structural system and its compliance with the criteria given in the standards referred to above. Data on possible structural changes since the time of construction.
b) Identification of the type of foundations of the bridge.
c) Identification of land conditions according to the categorization made in EN 1998-1: 2004, 3.1.
d) Information on the overall dimensions and properties of the cross sections of the bridge elements as well as the mechanical properties and condition of the constituent materials.
e) Information about identifiable defects of materials and inadequate detailing.
f) Information on the seismic design criteria used in the initial design, including the value of the seismic force reduction factor (q), if used.
g) Description of the current and/or planned use of the bridge (identifying also its class of importance, as described in EN 1998-1: 2004, 4.2.5) and in the bridge documentation.
h) Reassess the actions taken taking into account the use of the bridge.
i) Information about the type and extent of structural damage in the past and present, if any, including previous repair measures.

3.2 LEVEL OF KNOWLEDGE

In order to select the acceptable type of analysis and the appropriate values of confidence factors, EC8/3 defines the following three levels of recognition:
KL1: Limited knowledge
KL2: Normal knowledge
KL3: Full knowledge

The relationship between levels of knowledge, usable methods of analysis as well as confidence factors is shown in the Table below.

Table 5. Knowledge level characterization according to EC8/3

Level of knowledge	Geometry	Details	Materials	Analysis	CF
KL1	From general construction drawings with partial visual survey (with samples) or from full survey	Simulated design in accordance with the practices of the time and from limited in-situ evidence	Values accepted in accordance with the standards of the time it was build and from limited in-situ tests	LF-MRS	CF_{KL1}
KL2	From incomplete initial construction drawings with limited in-situ inspection or from extended in-situ inspection	From original project specifications with limited in-situ testing or from enhanced in-situ testing	From the specifications of the original project with limited in-situ tests or from extended in-situ tests	All	CF_{KL2}
KL3	From detailed initial construction drawings with limited in-situ inspection or from comprehensive in-situ survey	From original evidence reports accompanied by limited in-situ evidence or from comprehensive in-situ testing	From original test reports accompanied by limited in-situ tests or from comprehensive in situ tests	All	CF_{KL3}

Based on the information obtained and tested as well as the fact that the structure is a historical monument, it results that the building corresponds to the second level of recognition, KL2 according to the definitions of Eurocode 8/3. For this level of knowledge, the reliability coefficient CF = 1.2 must be applied. the explanation below obtained from EC8/3 explains the selection criteria.

KL2 belongs to the following degree of recognition:

i) geometry: the general geometry of the structure and the dimensions of the elements are known either (a) from a detailed survey or (b) from the general drawings of the project, used for the initial construction and for changes that may have been made later
ii) details: structural details are known either from an extended in-situ inspection or from incomplete construction drawings. In the latter case, it is advisable to carry out limited in-situ inspections on the most critical elements, in order to verify that the available information corresponds to the factual situation.
iii) materials: information regarding the mechanical properties of building materials is found either from extended in-situ tests or from the specifications in the initial design. In the latter case, limited in-situ testing is advised. The required number of proofs of each type should be taken according to the table below.

Table 6. Number of tests for every inspection level

	Inspection of details	Tests of materials
	For each primary element (beam, column, wall)	
Level of inspection and test	The percentage of elements whose details are checked	Number of material samples per floor
Limited	20%	1
Extended	50%	2
Comprehensive	80%	3

3.3 DETERMINATION OF SEISMIC INPUT

Seismic input is obtained based on seismological study. The seismological study was designed for seismic parameters of the area and approaches according to seismic probabilistic risk assessments and PGA maps of the two groups of authors. (Aliaj et al. and Duni et al.) The design spectra derived from this study were used for software analysis.

3.4 STATIC ANALYTICAL ANALYSIS OF EXISTING STRUCTURE

In order to determine the evaluations of the elements according to the recommendations, we have carried out in-depth analytical analysis according to the definitions of the theory of solving the masonry arches. The following figures give the principle of calculation.

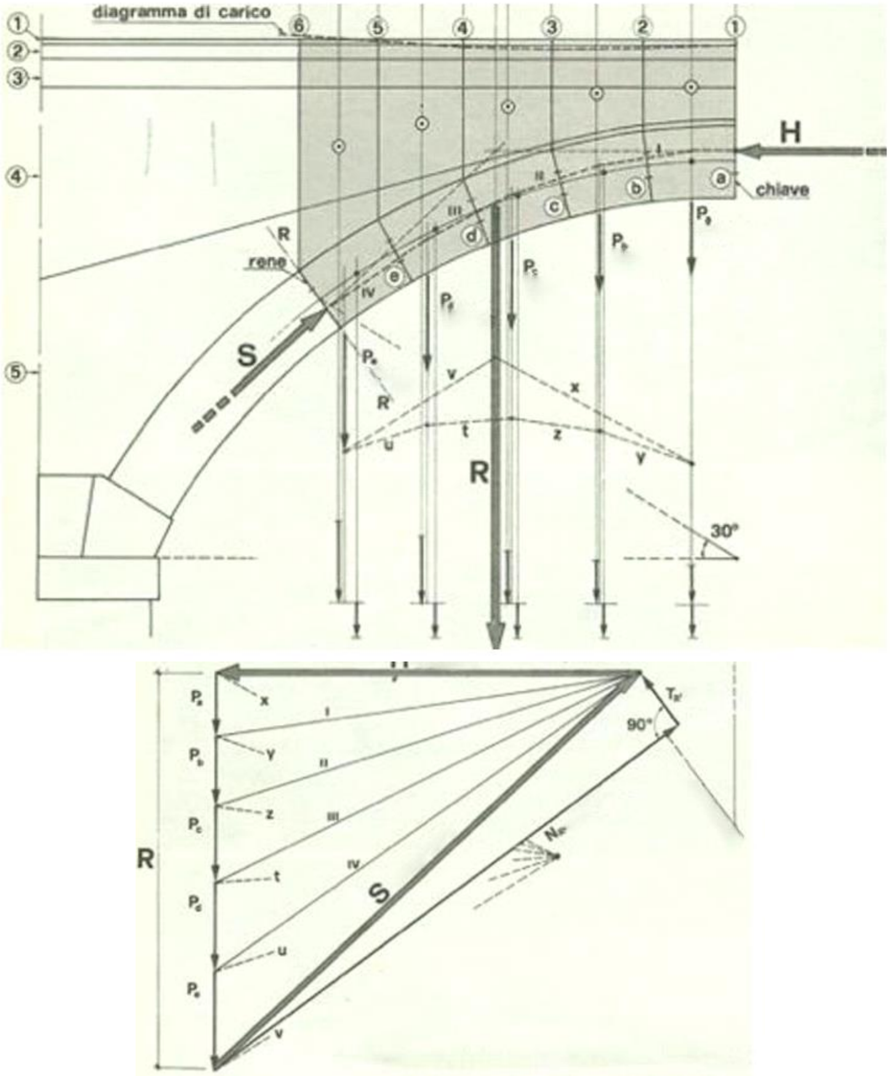


Figure 7: The principle of analytical solution of the masonry arches

3.5 LINEAR ANALYSIS OF EXISTING STRUCTURE

Linear analysis is performed by "Autodesk Robot Structural" and "limitstate Ring" software
In the following we are giving the main results of the analysis.

Loads and combinations

Dead Load: Self-weight of the elements of the bridge.
Live Load: Pedestrians and temporary loads, LL=5 kN/m^2.
Seismic Load: Design Earthquake with a return period of 475 years, input parameters from the Seismic Report.
Combinations: According to Eurocode 1.

Wind loads are not taken into consideration since the size of the structure is relatively small. Albanian Ant seismic Code KTP-2-89 which states that "In seismic design situation, wind loading is not being taken into account-except of " (Section 2.3.3/1, Page 18). Moreover, in the preliminary calculation of the wind load on the bridge surface, wind loading (considering the reference wind velocity as 30 m/s) is lower than the seismic force, therefore the seismic situation is the worst threating action to the existing bridge.
The object is a low-rise structure with a height of about 7m. And the span is also small. Thermal loads are not considered as critical loads to the structure. Therefore, thermal loads are not taken into account in structural calculations.

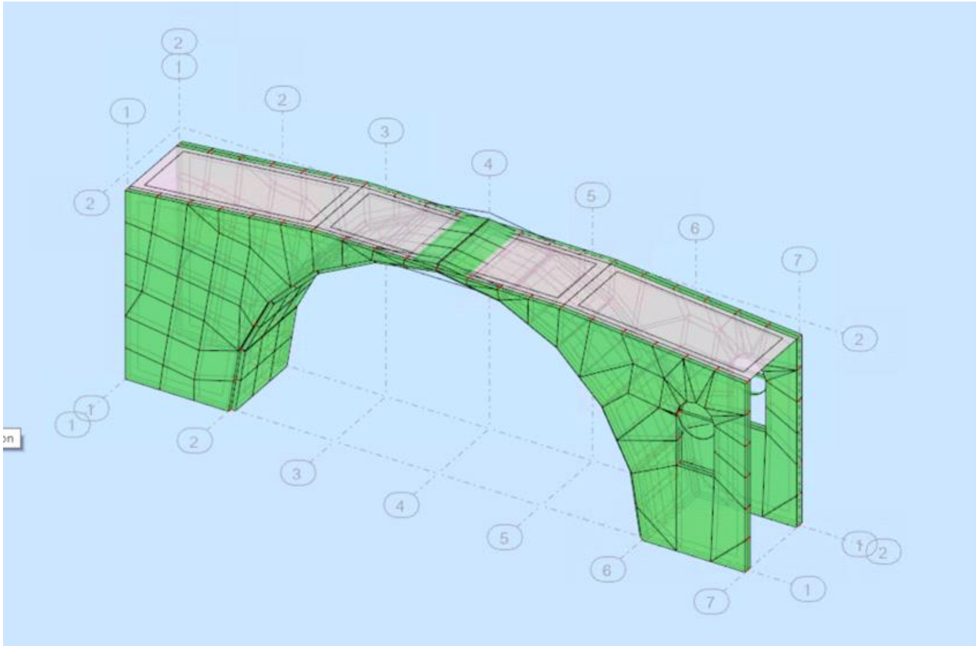


Figure 8: The principle of analytical solution of the masonry arches

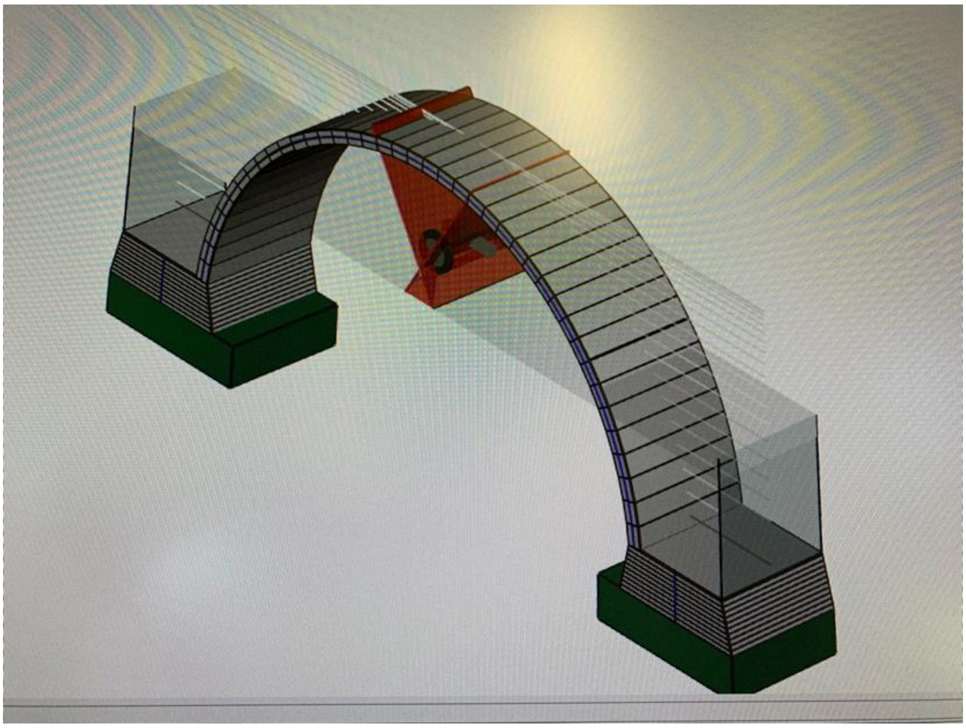


Figure 9: The principle of analytical solution of the masonry arches

Case	Load type	List									
1:DL1	self-weight	13to22 29to3	Whole structu	-Z	Factor=1.00	MEMO:					
2:DL2	(FE) uniform		PX=0.0	PY=0.0	PZ=-11.00	global	not project.	absolute	Limits	MEMO:	
2:DL2	(FE) uniform	40to49	PX=0.0	PY=0.0	PZ=-6.10	global	not project.	absolute	Limits	MEMO:	
2:DL2	(FE) uniform	43	PX=0.0	PY=0.0	PZ=-10.00	global	not project.	absolute	Limits	MEMO:	
2:DL2	(FE) uniform	43 48	PX=0.0	PY=0.0	PZ=-10.00	global	projected	absolute	Limits	MEMO:	
2:DL2	(FE) uniform	42 47	PX=0.0	PY=0.0	PZ=-20.00	global	projected	absolute	Limits	MEMO:	
2:DL2	(FE) uniform	40 41 45 46	PX=0.0	PY=0.0	PZ=-29.00	global	projected	absolute	Limits	MEMO:	
3:LL1	(FE) uniform		PX=0.0	PY=0.0	PZ=-5.00	global	projected	absolute	Limits	MEMO:	
3:LL1	(FE) uniform	60to62	PX=0.0	PY=0.0	PZ=-5.00	global	projected	absolute	Limits	MEMO:	
29:TEMP1	(FE) thermal load 3p	40to49 58 64	TX1=25.00	TZ1=0.0	TX2=0.0	TZ2=0.0	TX3=0.0	TZ3=0.0	N1X=0.0	N1Y=0.0	

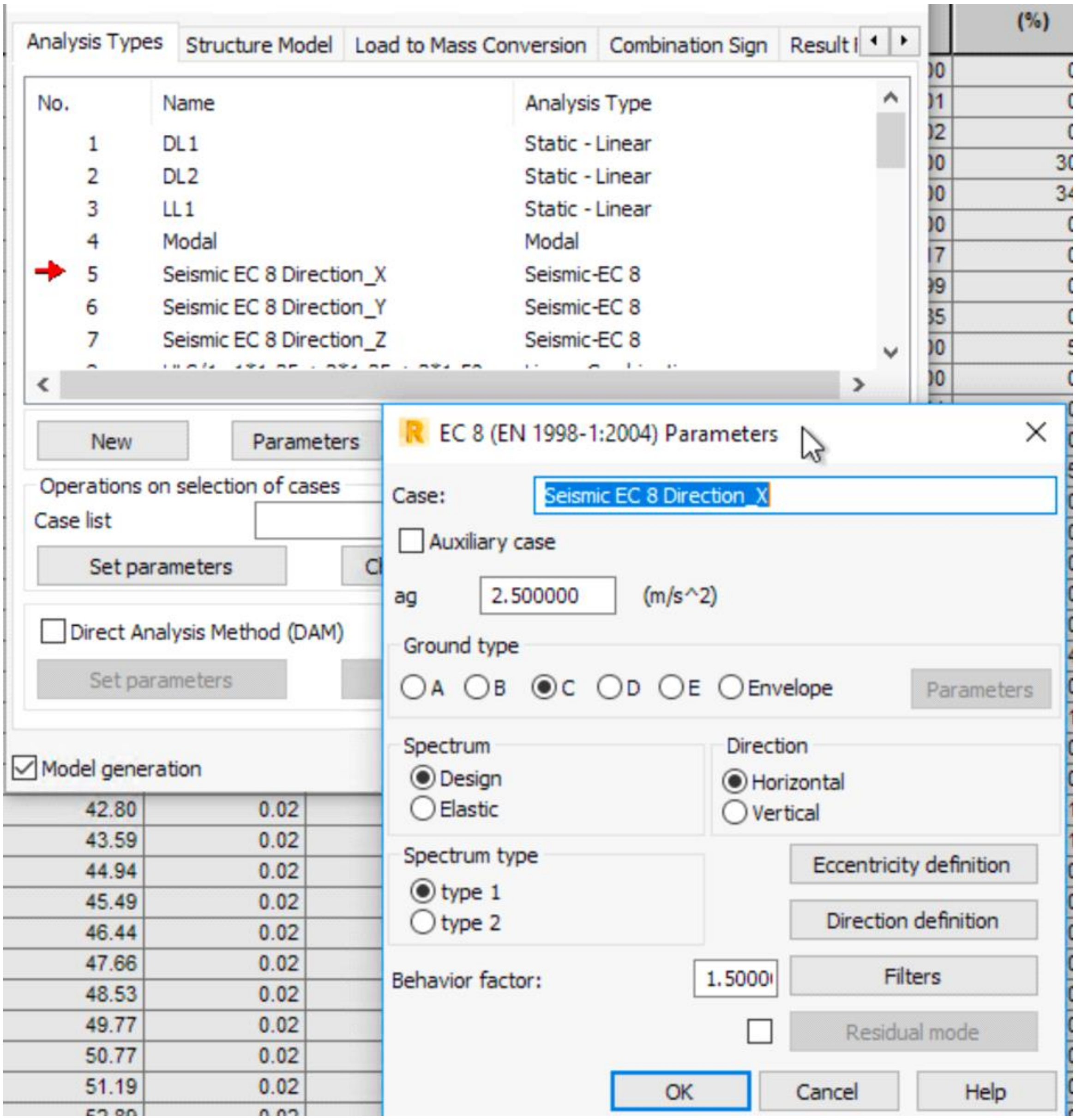


Figure 10: Static and seismic loads

Combinations	Name	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case
30 (C)	ULS/1=1*1.35 + 2*1.35 + 3*1.50 + 29*0.90	1	1.35	2	1.35	3	1.50	29	0.90	
31 (C)	ULS/2=1*1.35 + 2*1.35 + 3*1.50	1	1.35	2	1.35	3	1.50			
32 (C)	ULS/3=1*1.35 + 2*1.35	1	1.35	2	1.35					
33 (C)	ULS/4=1*1.00 + 2*1.00 + 3*1.50 + 29*0.90	1	1.00	2	1.00	3	1.50	29	0.90	
34 (C)	ULS/5=1*1.00 + 2*1.00 + 3*1.50	1	1.00	2	1.00	3	1.50			
35 (C)	ULS/6=1*1.00 + 2*1.00	1	1.00	2	1.00					
36 (C)	ULS/7=1*1.35 + 2*1.35 + 3*1.05 + 29*1.50	1	1.35	2	1.35	3	1.05	29	1.50	
37 (C)	ULS/8=1*1.35 + 2*1.35 + 29*1.50	1	1.35	2	1.35	29	1.50			
38 (C)	ULS/9=1*1.00 + 2*1.00 + 3*1.05 + 29*1.50	1	1.00	2	1.00	3	1.05	29	1.50	
39 (C)	ULS/10=1*1.00 + 2*1.00 + 29*1.50	1	1.00	2	1.00	29	1.50			
40 (C)	SLS:CHR/1=1*1.00 + 2*1.00 + 3*1.00 + 29*0.60	1	1.00	2	1.00	3	1.00	29	0.60	
41 (C)	SLS:CHR/2=1*1.00 + 2*1.00 + 3*1.00	1	1.00	2	1.00	3	1.00			
42 (C)	SLS:CHR/3=1*1.00 + 2*1.00	1	1.00	2	1.00					
43 (C)	SLS:CHR/4=1*1.00 + 2*1.00 + 3*0.70 + 29*1.00	1	1.00	2	1.00	3	0.70	29	1.00	
44 (C)	SLS:CHR/5=1*1.00 + 2*1.00 + 29*1.00	1	1.00	2	1.00	29	1.00			
45 (C)	SLS:FRE/6=1*1.00 + 2*1.00 + 3*0.50	1	1.00	2	1.00	3	0.50			
46 (C)	SLS:FRE/7=1*1.00 + 2*1.00	1	1.00	2	1.00					
47 (C)	SLS:FRE/8=1*1.00 + 2*1.00 + 3*0.30 + 29*0.50	1	1.00	2	1.00	3	0.30	29	0.50	
48 (C)	SLS:FRE/9=1*1.00 + 2*1.00 + 29*0.50	1	1.00	2	1.00	29	0.50			
49 (C) (CQC)	ACC:SEI/1=1*1.00 + 2*1.00 + 3*0.30 + 5*1.00	1	1.00	2	1.00	3	0.30	5	1.00	
50 (C) (CQC)	ACC:SEI/2=1*1.00 + 2*1.00 + 5*1.00	1	1.00	2	1.00	5	1.00			
51 (C)	ACC:SEI/3=1*1.00 + 2*1.00	1	1.00	2	1.00					
52 (C) (CQC)	ACC:SEI/4=1*1.00 + 2*1.00 + 3*0.30 + 6*1.00	1	1.00	2	1.00	3	0.30	6	1.00	
53 (C) (CQC)	ACC:SEI/5=1*1.00 + 2*1.00 + 6*1.00	1	1.00	2	1.00	6	1.00			
54 (C) (CQC)	ACC:SEI/6=1*1.00 + 2*1.00 + 3*0.30 + 7*1.00	1	1.00	2	1.00	3	0.30	7	1.00	
55 (C) (CQC)	ACC:SEI/7=1*1.00 + 2*1.00 + 7*1.00	1	1.00	2	1.00	7	1.00			
56 (C) (CQC)	ACC:SEI/8=1*1.00 + 2*1.00 + 3*0.30 + 5*-1.00	1	1.00	2	1.00	3	0.30	5	-1.00	
57 (C) (CQC)	ACC:SEI/9=1*1.00 + 2*1.00 + 5*-1.00	1	1.00	2	1.00	5	-1.00			
58 (C) (CQC)	ACC:SEI/10=1*1.00 + 2*1.00 + 3*0.30 + 6*-1.00	1	1.00	2	1.00	3	0.30	6	-1.00	
59 (C) (CQC)	ACC:SEI/11=1*1.00 + 2*1.00 + 6*-1.00	1	1.00	2	1.00	6	-1.00			
60 (C) (CQC)	ACC:SEI/12=1*1.00 + 2*1.00 + 3*0.30 + 7*-1.00	1	1.00	2	1.00	3	0.30	7	-1.00	
61 (C) (CQC)	ACC:SEI/13=1*1.00 + 2*1.00 + 7*-1.00	1	1.00	2	1.00	7	-1.00			
*										

Figure 11: Load combinations according to EC1

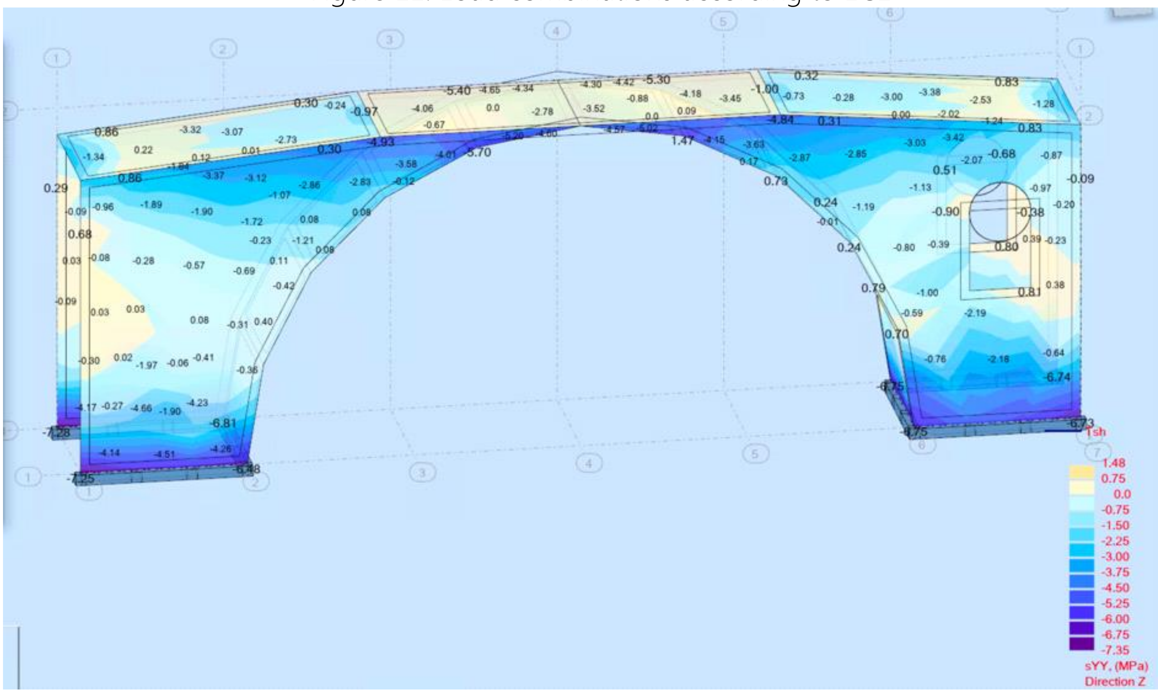


Figure 12: Vertical stress in the bridge masonry walls

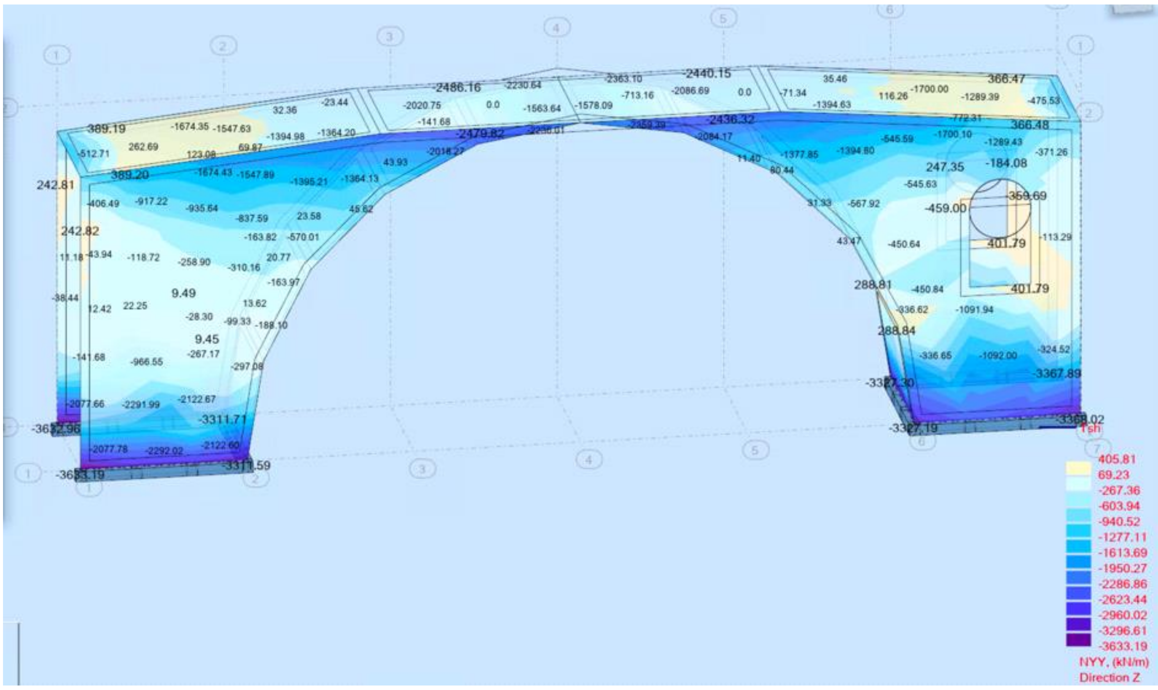


Figure 13: Axial forces in the masonry walls

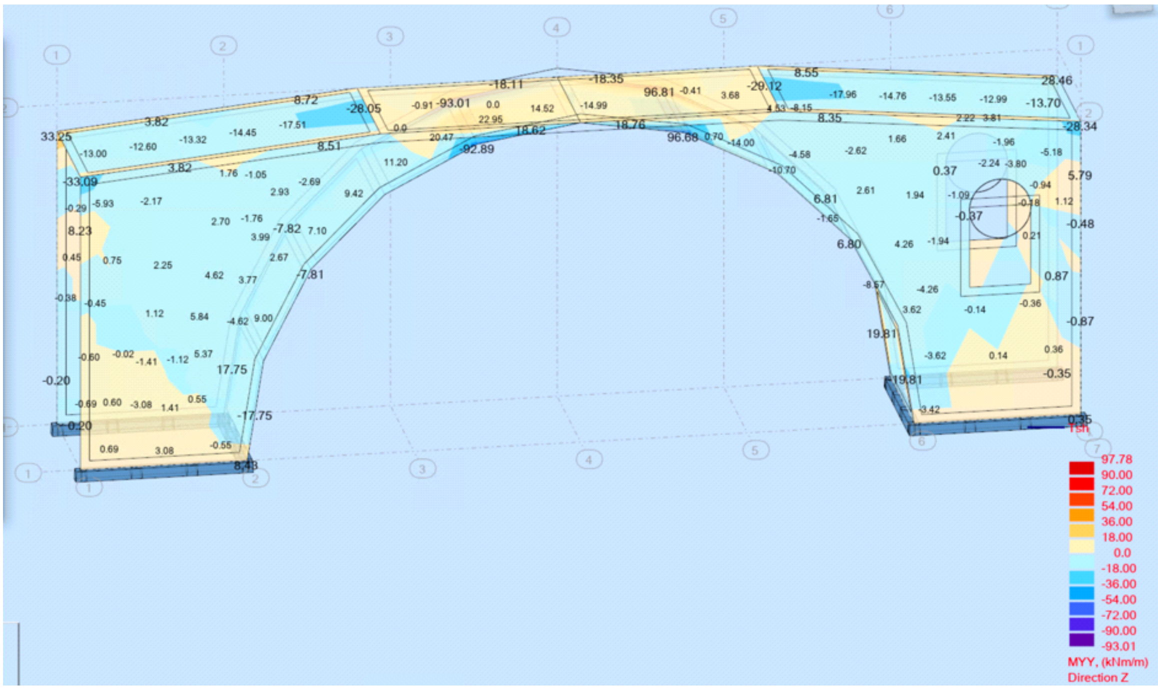


Figure 14: Bending moments in the masonry walls

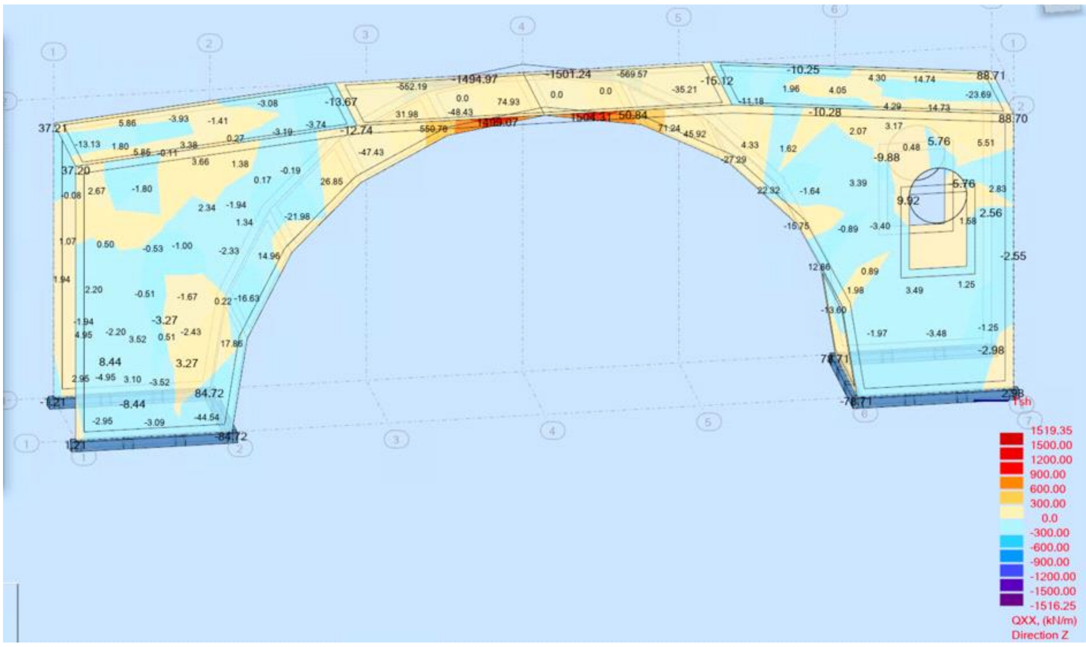


Figure 15: Shear force in the bridge masonry walls

Under seismic load smaller but asymmetric internal forces.

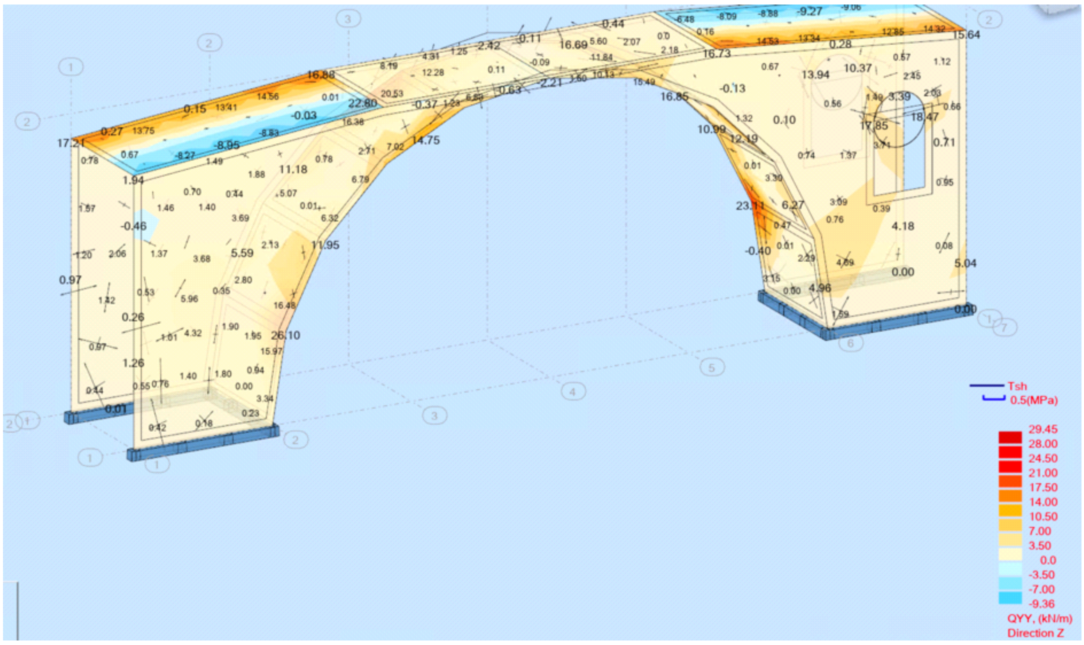


Figure 16: Shear force during seismic loading

3.6 NONLINEAR ANALYSIS OF EXISTING STRUCTURE

3.6.1 General

Nonlinear (NL) analyzes performed will be:

- NL with concentrated plasticity and macro elements
- NL analysis with concentrated plasticity and macro elements

Since we have performed linear analyzes and measurements in the object for comparison, we have realized nonlinear design by software "3Wall" according to the recommendations of EC2, EC6, EC8/1, EC8/3 and seismic loading according to the calculation procedures of EC8/1 and other European recommendations. The general scheme of the analysis procedure is given in the following figure.

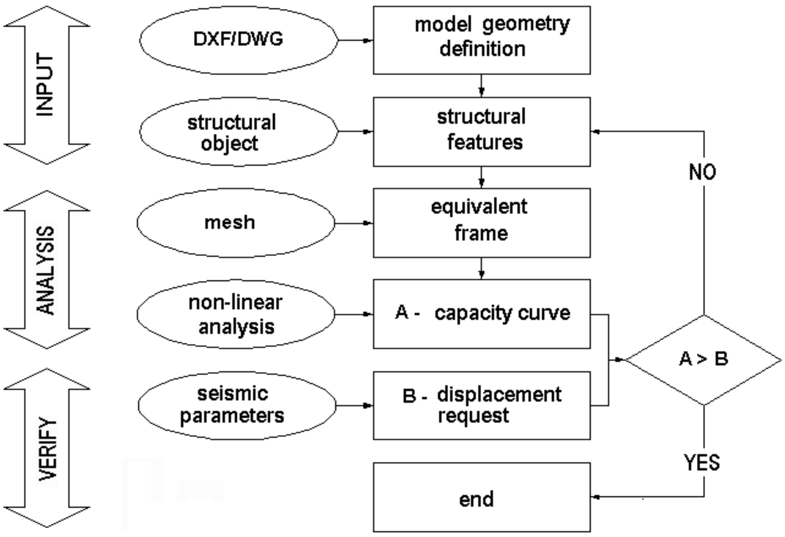


Figure 17: Nonlinear analysis chart

The analysis was done for three limit states. The structural imperfections according to the design codes have been considered when carrying the nonlinear analyses. For the limit state condition of the service with the return periods of 95 years, for the ultimate limit state condition with the return periods of 475 years.

Levels of performance for the structural elements

Ultimate limit State (ULS): theoretically large cracks and hinges are formed but structural integrity is still preserved.

Serviceability Limit State (SLS): spalling of mortar and after some repairs the bridge can be functional again.

About ULS: Since the structure has a very low level of ductility (behavior factor q=1.5), it has been assumed that it must be within the elastic range in the ULS, only very minor cracks are allowed. The results of the analysis according to Eurocodes are given below.

According to European design conditions (see previous chapter) the seismic load parameters and PGA = 0.25g have been selected.
The aspects of structural modeling and determination of seismic action given above are made in accordance with the conditions of Eurocode 8. The combination of seismic loads and other actions is made based on EC1.
The elements of the structure are modeled by frame elements, plane walls (macro-element) and rigid diaphragm soles in its plane. The contribution of non-structural elements has been neglected.

Due to the shape of the structure the 3D model is chosen under vertical loads and the action of the earthquake. The model for seismic calculation is selected modal analysis with reaction spectrum, with masses concentrated in the center of mass (concentrated by the program itself) and accidental eccentricity 5% in each direction.
NL analysis are performed for each orthogonal direction and distribution according to the predominant mode and according to the measures considering also the accidental eccentricities recommended by EC8. 24 NL analyzes were performed.

3.7 CONCLUSIONS FOR THE ANALYSIS OF THE EXISTING BRIDGE

Based on the results given above we conclude that:

The structure of the bridge does not withstand static loads and will suffer progressive damage such as disintegration of mortar, cracks, detachment of stones, up to the collapse of side walls (masonry blocks).

The structure of the bridge does not withstand seismic loads and will suffer immediate damage such as cracking according to the mortar joint, up to the collapse of sides (masonry blocks).

Floods with a frequency of 1 in 50 years will affect the body of the bridge, will disintegrate the mortar to the fall of stones or pieces of wall and will erode the gravel parts under the pieces of rock where the bridge is supported.

3.8 HYDRAULIC CALCULATIONS

The catchment basin is 27.88 km^2 with a perimeter of about 30km and a maximum rib length of 9.96 km. The absolute elevation (quota) at the bridge position is approximately 180.0m.

Inflow

Basic hydrological characteristics are:
Length of the flow line is about 9km, length of the flow line of the point 900- 950m
Duration of flow 1.15h
Intensity for 50-year return periods (bridges of special importance)
h_w=43.1$t^{0.405}$; I_{mes}=H*10^{-3}/Tc; I_{mes}=45.6 mm/h
The maximum inflow is respectively Q = 79.8 m^3/s
Average flow velocity through the bridge v = 3.95m/s
Maximum calculated flood elevation through the bridge h_{wp} = 180.4 (about 0.56m above the relative quota 0.00 of the foot of the bridge).
The magnitude of this average 0.60 m height of inflow is found to be relatively small in comparison to the seismic loads acting on the massive bridge structure.

Evaluation of erosions

At the place where the river protection of the bridge will be realized, as a result of the construction of the bridge abutments, a narrowing of the free water section is created during the floods with inflow Q = 79.8 m^3/s, for a repetition period T=50 years. Based on the geometry of the transverse section of the river, the light spaces between the sides, the geometric shape of the bridge and the hydraulic flow parameters in the section, before and after the bridge, calculations of erosion velocities and expected erosion depth were performed.
The minimum size of gravel populations, which would not move from the water during the fills, is as follows:

$$v_{cr} = 0.85 \times \sqrt{2 \times g \times d \times \frac{\gamma_s - \gamma_u}{\gamma_u}},$$

where:
γ_s = 27 000 N/m^3 is the volumetric weight of gravel in the river and γ_u = 9810 N/m^3 is volumetric weight of water
g = 9.81 m/s^2, is the acceleration of gravity
d = is the granulometry of the smallest particles or populations that do not move from water at the speed of the formula
v_{cr} = is the velocity of water which erodes particles of size d
The erosion rate should be <0.5 x v_0

In our case v_0 = 3.95 m/s and accepting v_{cr} = v_0/2 = 3.95/2 = 1.975 m/s> 1m/s.
The size of the smallest particle, which does not move from the water results d = 0.160 m = 16 cm.
Theoretical depth of erosion, in the case of bed composition with d_{50}<16cm, is calculated by the expression:

$$\frac{d_s}{s} = f_1\left(\frac{v_0}{v_{cr}}\right) \times \left[2 \times \tanh\left(\frac{y_0}{s}\right)\right] \times f_2\,(forma) \times f_3\left(\propto, \frac{l}{s}\right)$$

which is based on the graph and Shields method, and where: ds = erosion depth [m]
s = pile width
l = pile length
f_1 = coefficient which depends on the ratio of flow velocity behind the bridge to corrosion velocity
f_2 = coefficient, which depends on the shape of the pile
f_3 = 1.5, which is found in the corresponding graph and depends on the ratio of the length to the width of the pile and the angle of flow against the axis of the pile (in our case the flow angle is assumed to be 45 °, l = 3.5 m, s = 2.2 m)
y_0 = depth of water flow immediately after the bridge
Based on the above calculation and assuming one of the small values of the granulometry scale given by the geology (d_{50}<16 cm), it turns out that in this case the theoretical depth of erosion of the river bed would result ds = 1.30 m.
But, since according to geology it turns out that there are particles with granulometry from the smallest to 30 cm, we can say that the mixture of these cobbles and their natural compaction by water, does not allow complete erosion to the theoretically calculated depth. Also, the pit created during the floods should be filled with new material, brought from the river during flows with normal levels.

Erosion protection

Since the calculations result in erosion potentials that can influence the pieces of rock (scouring) on the river gravel where the bridge rests, these areas should be protected from erosion.
Under the pieces of rock in the gravel parts will be realized filling of stones connected with concrete under the bottom of the piece of rock up to a depth of 1m. Protection with gabion wall on the right side up to the river terrace, installation depth -1.3m from the relative height 0 and wall height 0.8m above this height.

4 DESIGN SCENARIOS PROPOSED BY THE PREVIOUS REPORT

Three scenarios were proposed in the previous structural report. These are summarized as;
1. Using steel elements (profiles) to strengthen the lower part of the bridge structure.
2. Using FRP elements to add tensile capacity to the bridge upper part of the arch.
3. Adding reinforced concrete elements to increase the tensile capacity to the upper part of the arch structure

The first option can be considered as an invasive solution (if no other better choice is available) and would have a great visual impact. The second scenario (Fiber Reinforced Polymers) is a relatively good solution as it increases the tensile capacity of the bridge arch at its upper part, but carries the burden of using a relatively new material that can have irreversible effects (due to the adhesion that the fabric of FRP requires with arch structure). The third scenario, although for decades it has been the most preferred one for arch consolidation in restoration practice, was considered not very suitable, due to the extra weight of the arch elements and due to its irreversibility.

Therefore, Atelier4 design team proposes to add post tensioned stainless-steel cables anchored to the base rock on both sides of the abutments, to add tensile capacity to the existing upper part of stone arch structure. The purpose of using cables is to increase the tensile capacity of the arch and to ensure the non-formation of the fourth hinge in the arch (which activates the mechanism of collapse of the bridge). The advantage of this intervention is not only the light weight of the steel cables but it is also a reversible technique in cases where the recommended solution is not effective (steel cables can be removed at any time without damaging the originality of the bridge).

4.1 CALCULATIONS OF THE STRUCTURE AFTER THE INTREVENTION

Based on the results of the model presented, the calculations of the mixed element were performed: stone masonry with internal reinforcement. The calculations were performed according to the EC8 Standards defined in the Design Task for the most unfavorable combination of vertical and horizontal loads. Live load values and load combinations are given in the previous chapters. (LL1 = 5 kPa)

Materials

Reinforcement Steel

Rebars of type B500C according to European standard EN 10080. They have the following characteristics:

Tensile strength
$f_{tk}> = 5500$ daN/cm^2
Yielding stress
$f_{yk} = 5000$ daN
Modulus of elasticity
$E_c = 2100000$ daN/cm^2
Characteristic strain under maximum load $\varepsilon_{uk} = 7.5\%$
$k = (f_t/f_y)k = 1.15$

Partial safety factor
$\gamma_S = 1.15$
Computational resistance

f_{yd} = 4347.8 daN/cm^2

Masonry (stone units connected with lime mortar)

Stone strength
f_b = 500daN/cm^2
Mortar strength
f_m = 30-32daN/cm^2
Average masonry compressive strength f_m = 31 daN/cm^2
Characteristic strength of masonry according to EC6 f_k = 137 daN/cm^2
τ_o = 0.15 daN/cm^2
E_m = 20,000 daN/cm^2
G_m = 8000 daN/cm^2
Self-weight ρ = 2200 kg/m^3

Reinforcement Calculation

This reinforcement serves to connect the two sides and withstand the vertical load during bridge visits, as well as to improve the seismic behavior during earthquakes.
$As_{longitudinal}$ = 2cm^2/m

4.2 REINFORCEMENT CALCULATIONS OF THE CONSOLIDATED AND RESTORED STRUCTURE

General for detailed reinforcement models

The existing structure after the processes of consolidation, restoration and replacement of new parts again needs reinforcing intervention. The reinforcement consists on the installation of steel cables in the part of the bridge arch.
Implementing the prestressing force will prevent the formation of the 4 hinges which can bring the arch to collapse. Prestressing force will serve to close cracks, therefore, improve the stiffness of the arch and to remain in the elastic range of response.

Calculations of bonding length and force

The anchorage length is closely associated to the design bond strength, f_{bd}, which is given as follows:
f_{bd}= 2,25·η_1·η_2·f_{ctd} (N/mm^2)
Where:
2,25 = basic value of the design bond strength (N/mm^2)
η_1 = coefficient related to the quality of the bond condition and the position of the bar during concrete/resine pouring.
η_1=1.0 stands for good bond conditions and η_1=0.7 is taken for all other cases.

η_2 = coefficient related to the rebar diameter: $\eta_2 = (132-\phi)/100 \le 1.0$ where ϕ is the nominal rebar diameter [mm] while η_2 = 1,0 for $\phi \le$ 32mm (-)
f_{ctd} = the design tensile strength of the concrete/rock
The basic required anchorage length $l_{b,rqd}$ is given as follows: $l_{b,rqd} = (\phi/4)/(\sigma_{sd}/f_{bd})$ (mm) Where:
ϕ = the reinforcing bar diameter (mm)
σ_{sd} = design steel stress at the beginning of the anchorage (N/mm^2)
f_{bd} = design value of the ultimate bond stress (N/mm^2)
The design anchorage length l_{bd} is calculated from the basic required anchorage length $l_{b,rqd}$ taking into account the influence of five parameters (α_1 to α_5) and it should not be less than a minimum anchorage length $l_{b,min}$. The design anchorage length l_{bd} is given as follows:
Rebar under tension: $l_{bd}= \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd} \ge l_{b,min}$ (mm)

$f_{bd}=2.25*\eta_1*\eta_2*\ f_{fctd}$ (η_1=0.7, η_2=1; poor bond conditions, ϕ= 20mm)
f_{ctd}= 1/10*f_{cd}=1/10*19.8 ≅ 2 MPa (design tensile strength)
f_{bd}=2.25*0.7*1.0*2=3.15 MPa≅ 3 MPa

The basic required anchorage length, (σ_{sd} = 3800daN/cm^2)
$l_{b,rqd}$ = (20/4)/(3800/300000/10000)= 633mm ≅ 64 cm

Design anchorage length (α_1 =1, α_2 =1, α_3 =1, α_4 =1, α_5 =1, for straight ones)
$l_{bd}= \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ *l_{b,rqd}$= 64 cm

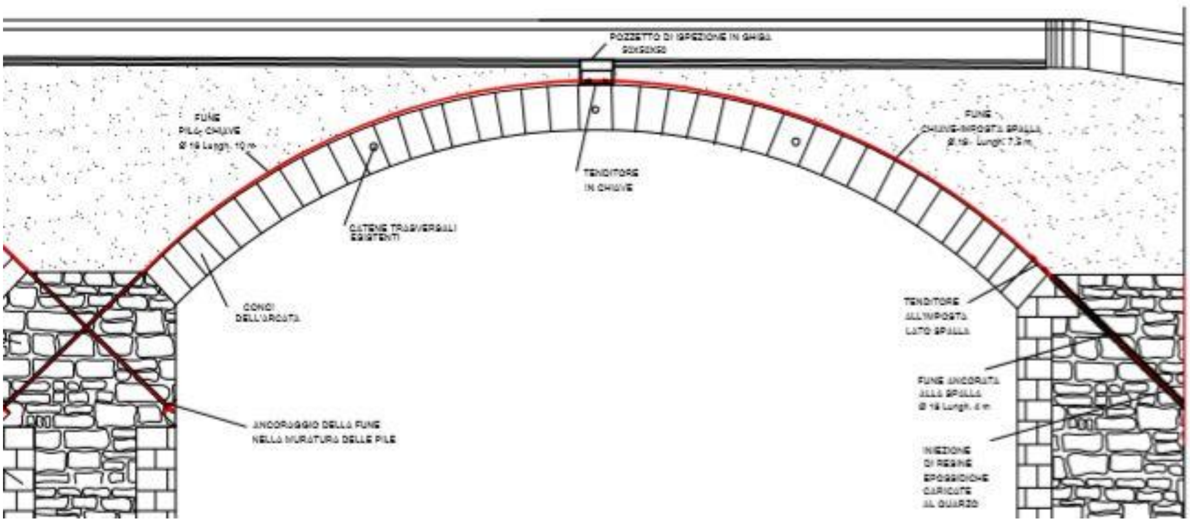
Allowable anchorage length
60$\phi \ge l_b \ge$ max(0.3$_{lbrqd}$; 10ϕ; 100mm)
120 cm $\ge l_b \ge$ max(0.3*64=19.2 cm; 20 cm; 10cm)
120 cm $\ge l_b \ge$ 20 cm
It is selected as l_b =100 cm

Controls for load transfer of cables through spacers

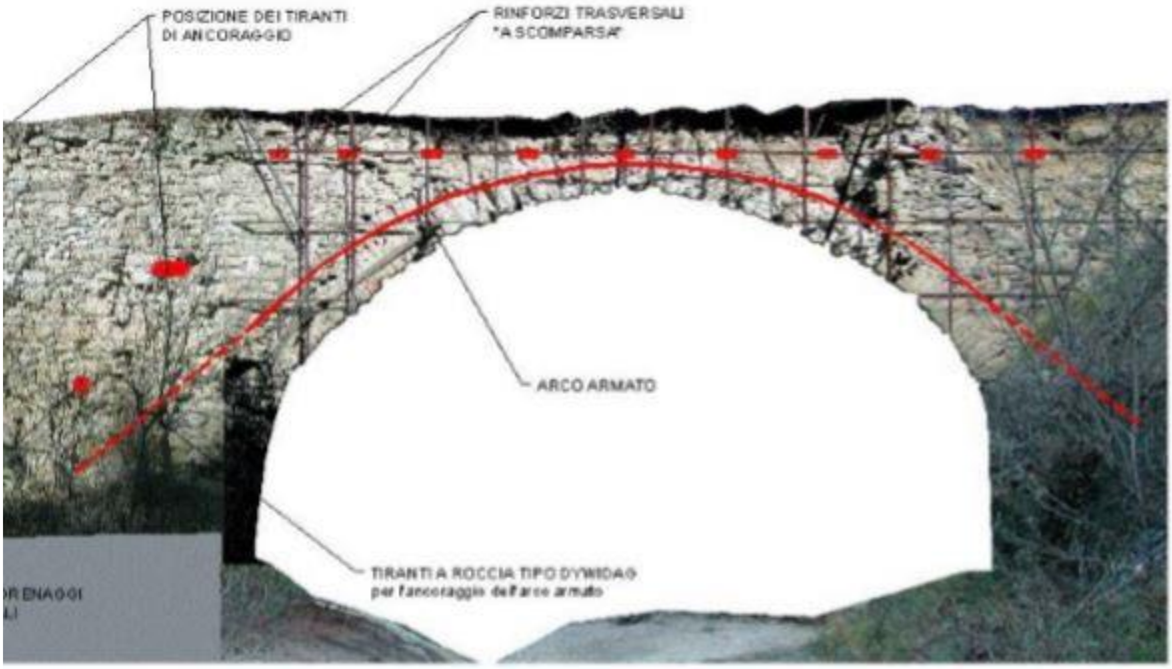
Design tensile force on one cable, F_d= 30 kN

Load shared on each spacer, P_s=30/12=2.5 kN

Stress applied by each spacer, σ_{avg}= 4.2 kPa << Average masonry compressive strength, f_m = 3100kPa



(a)



(b)

Figure 18: Location examples of the steel cables in the ach

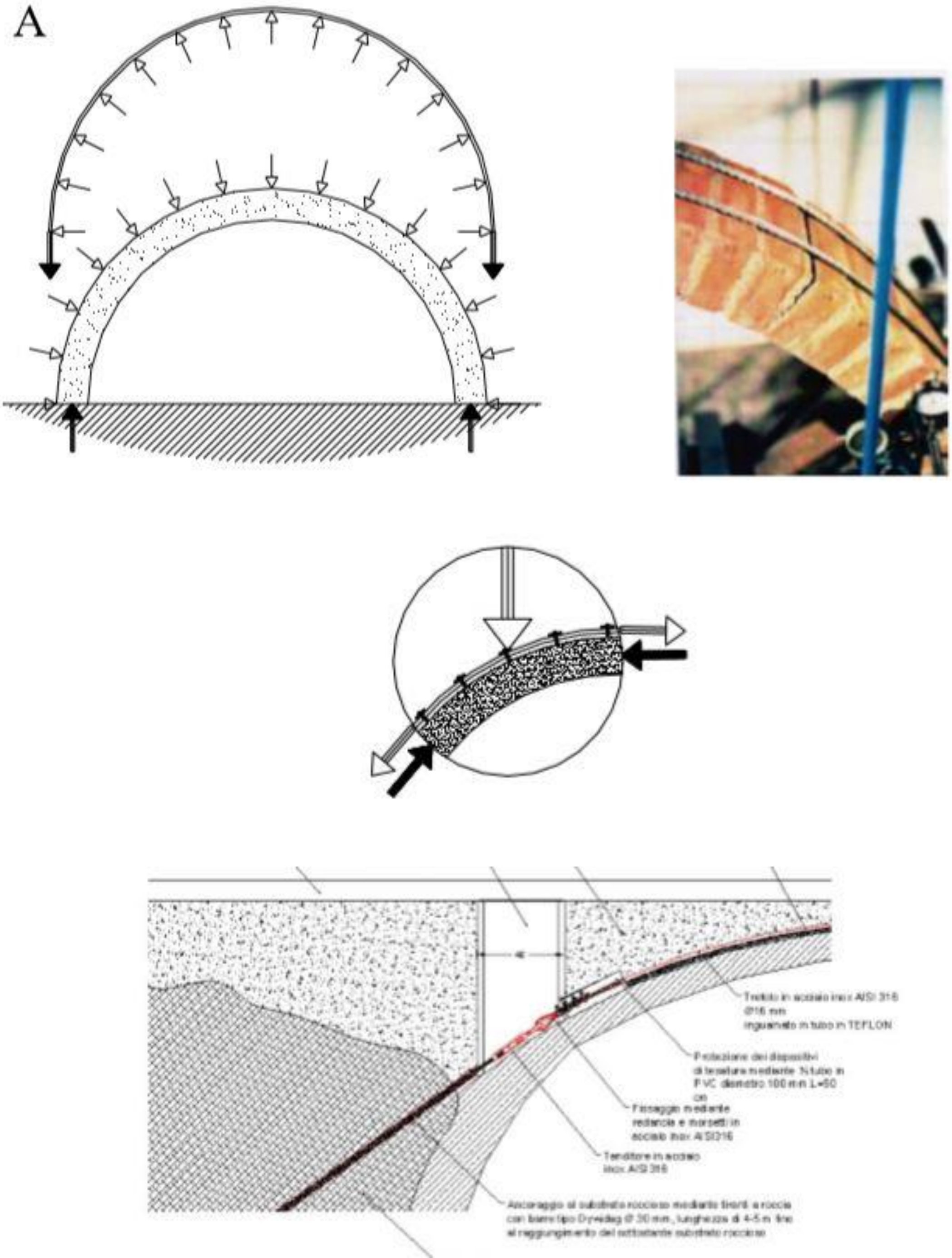


Figure 19: Reinforcement of the ach with steel cables

For the control of the vulnerability of the reinforced structure we have realized two detailed models.

Model I. This model consists of a nonlinear masonry arch with equivalent section characteristics.

Models II. This model consists of nonlinear masonry panels connected by nonlinear "link" elements that simulate the work of filling between walls.

The following figures give details of the modeling and the main results.

Models I.

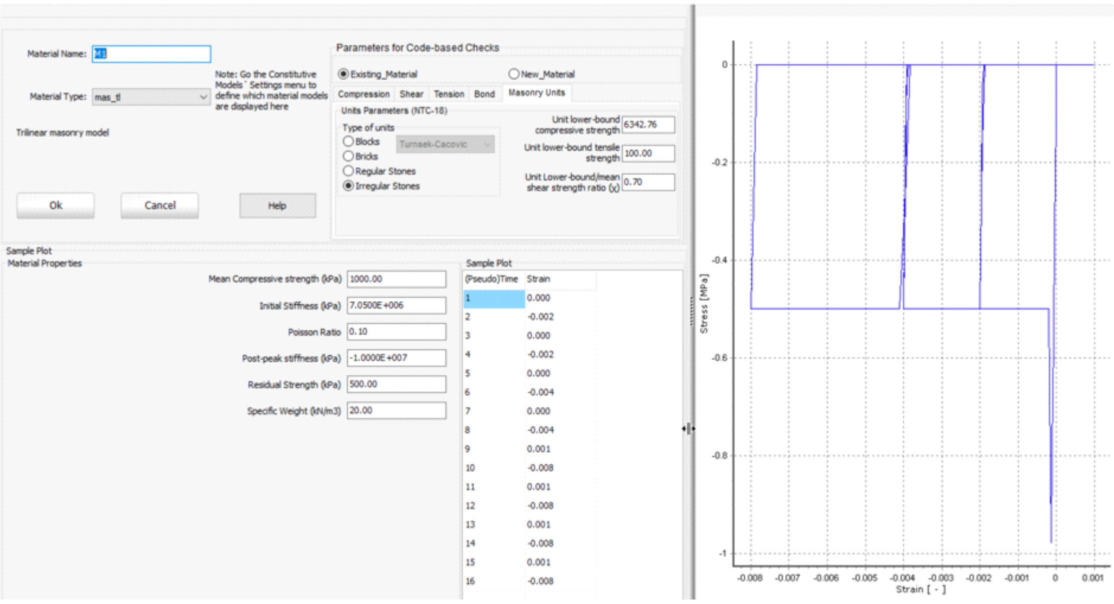


Figure 20: Nonlinear behavior of the reinforced material

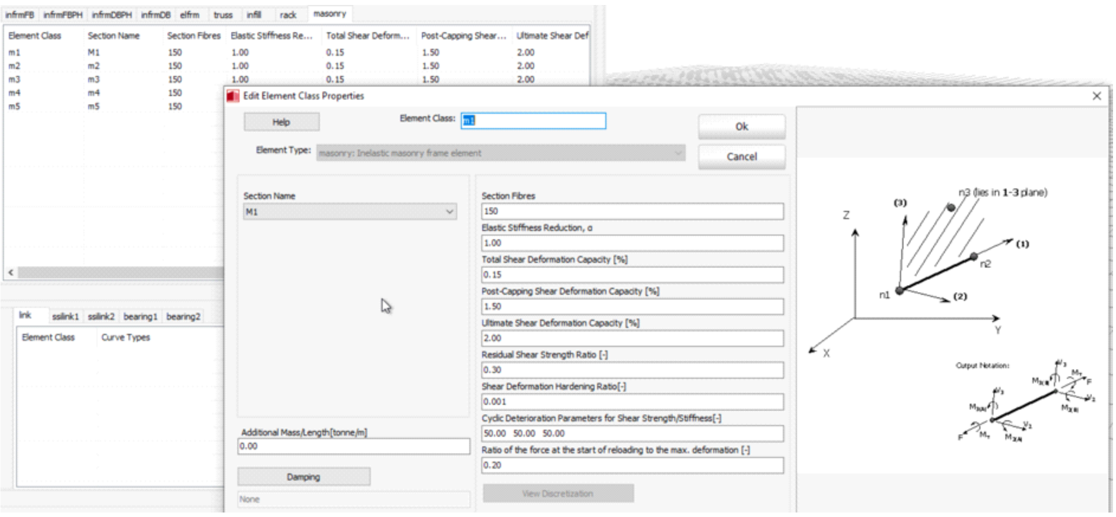


Figure 21: Nonlinear characteristics of the reinforced material

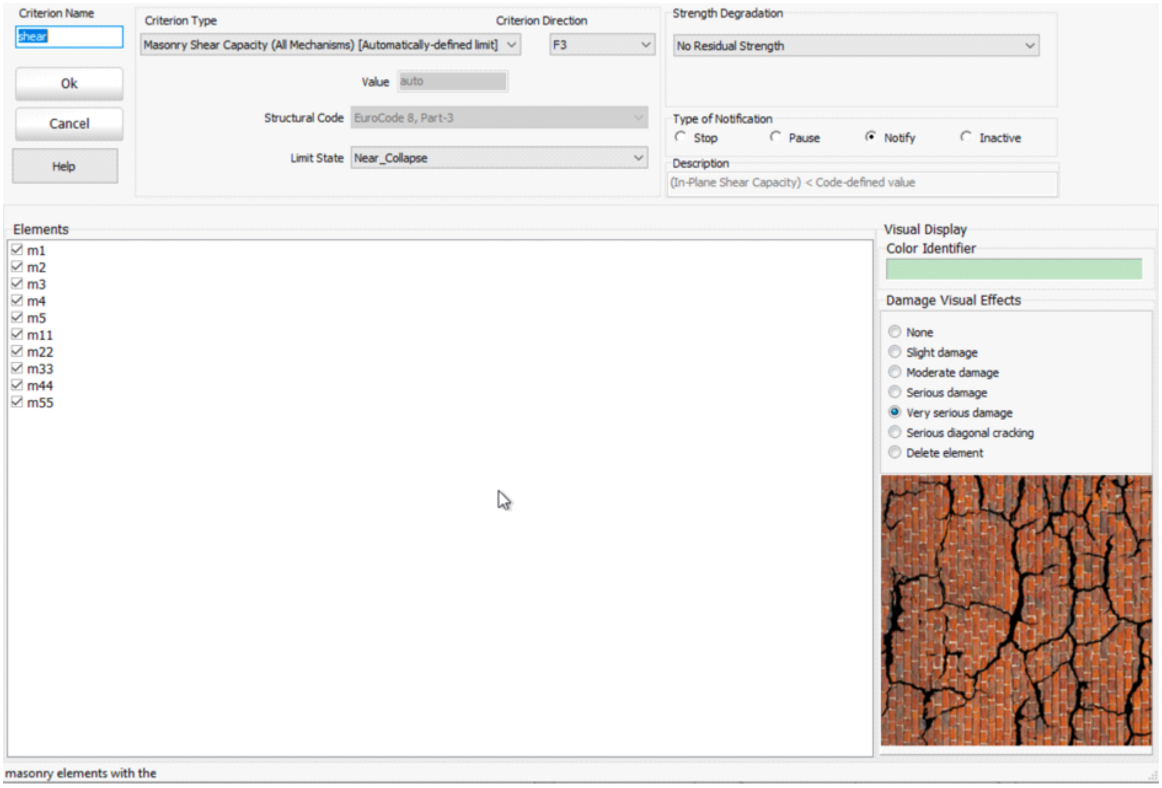


Figure 22: Performance check of elements

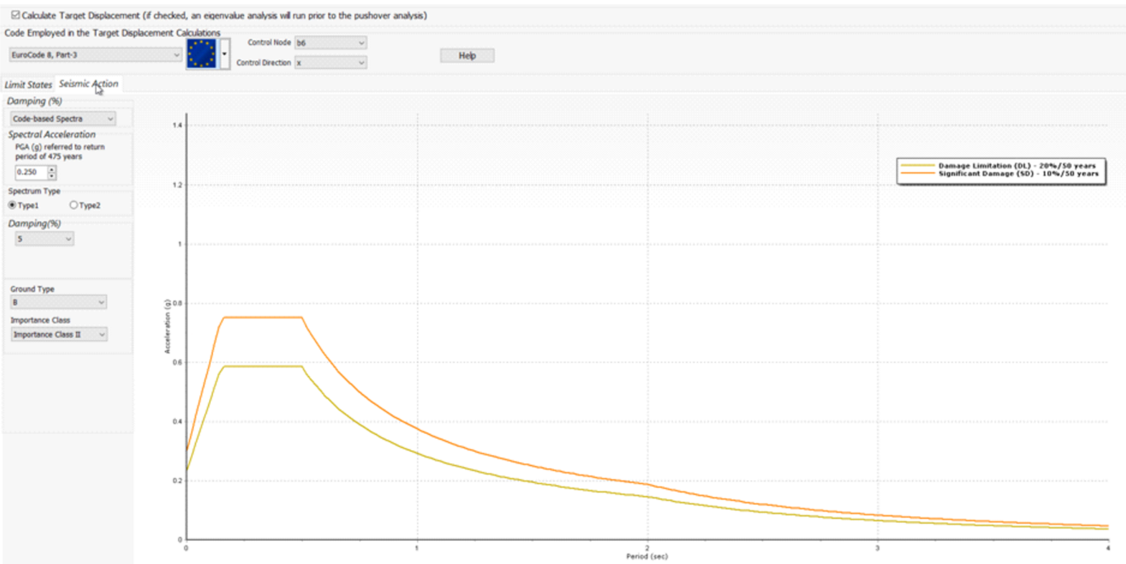


Figure 23: Response spectrums according to performance levels

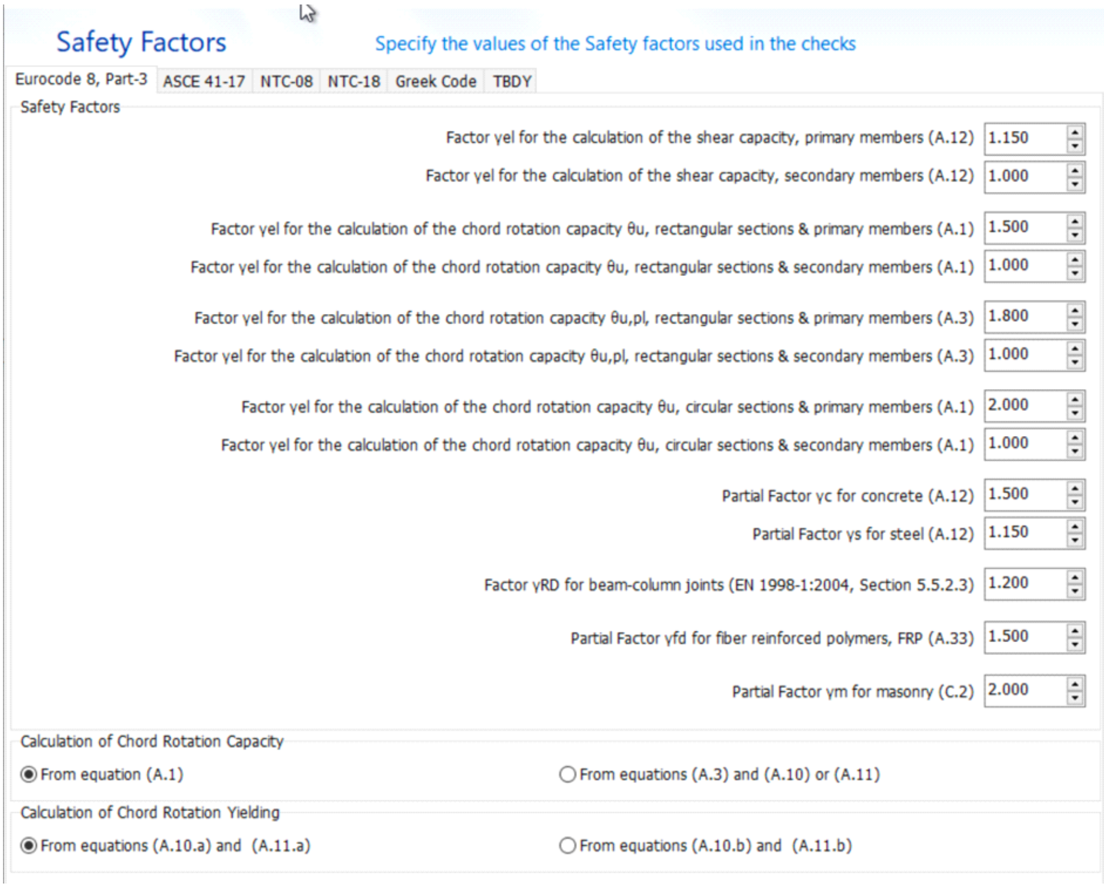
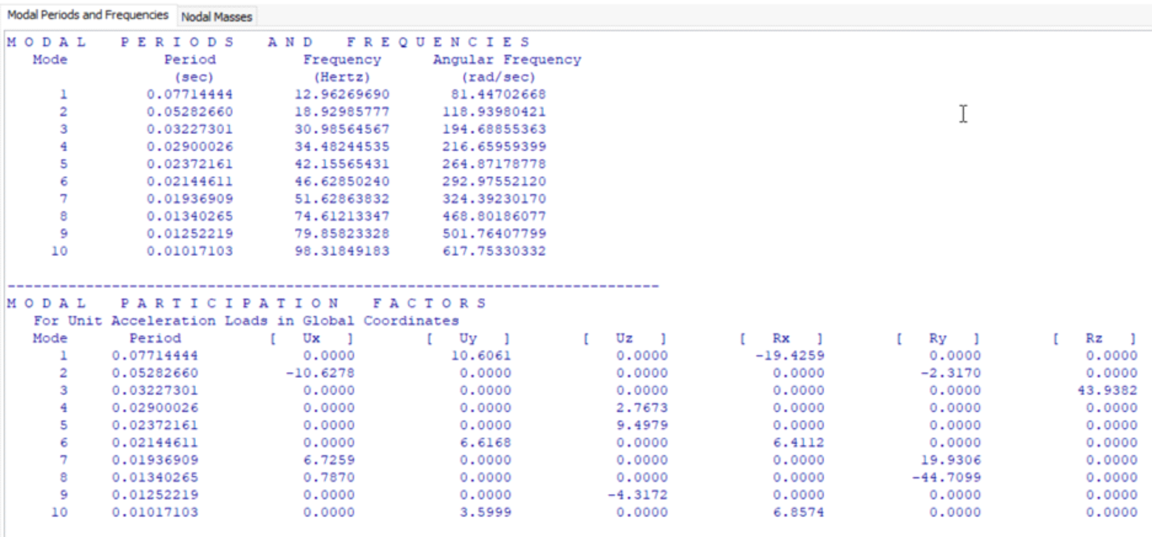


Figure 24: Safety factor values and check equation indexes



Modal Periods and Frequencies | Nodal Masses

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

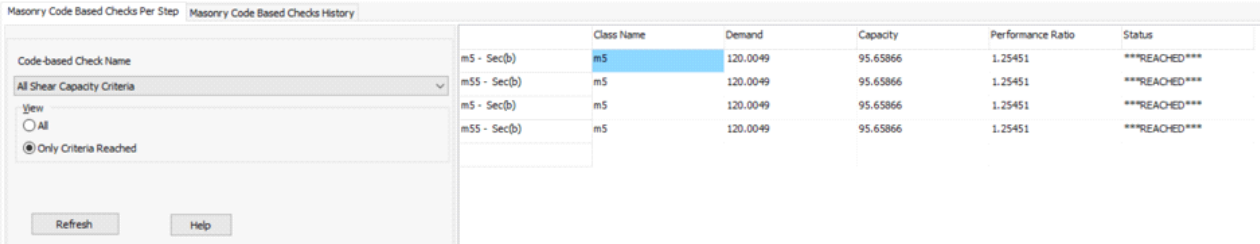
Mode	Period (sec)	Frequency (Hertz)	Angular Frequency (rad/sec)
1	0.07714444	12.96269690	81.44702668
2	0.05282660	18.92985777	118.93980421
3	0.03227301	30.98564567	194.68855363
4	0.02900026	34.48244535	216.65959399
5	0.02372161	42.15565431	264.87178778
6	0.02144611	46.62850240	292.97552120
7	0.01936909	51.62863832	324.39230170
8	0.01340265	74.61213347	468.80186077
9	0.01252219	79.85823328	501.76407799
10	0.01017103	98.31849183	617.75330332

M O D A L   P A R T I C I P A T I O N   F A C T O R S
For Unit Acceleration Loads in Global Coordinates

Mode	Period	[Ux]	[Uy]	[Uz]	[Rx]	[Ry]	[Rz]
1	0.07714444	0.0000	10.6061	0.0000	-19.4259	0.0000	0.0000
2	0.05282660	-10.6278	0.0000	0.0000	0.0000	-2.3170	0.0000
3	0.03227301	0.0000	0.0000	0.0000	0.0000	0.0000	43.9382
4	0.02900026	0.0000	0.0000	2.7673	0.0000	0.0000	0.0000
5	0.02372161	0.0000	0.0000	9.4979	0.0000	0.0000	0.0000
6	0.02144611	0.0000	6.6168	0.0000	6.4112	0.0000	0.0000
7	0.01936909	6.7259	0.0000	0.0000	0.0000	19.9306	0.0000
8	0.01340265	0.7870	0.0000	0.0000	0.0000	-44.7099	0.0000
9	0.01252219	0.0000	0.0000	-4.3172	0.0000	0.0000	0.0000
10	0.01017103	0.0000	3.5999	0.0000	6.8574	0.0000	0.0000

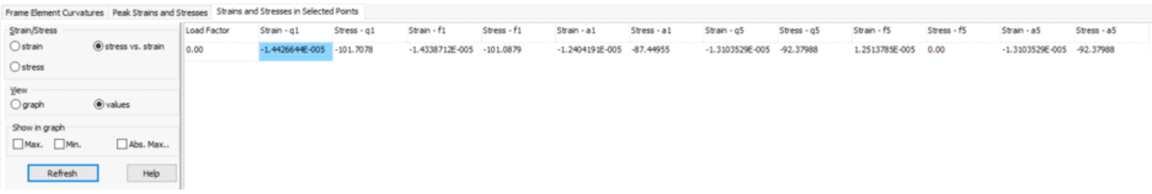
Figure 25: Period values and modal participation

As seen from the comparison with previous models. the structure is stiff and we will have reduction of deformation.



	Class Name	Demand	Capacity	Performance Ratio	Status
m5 - Sec(b)	m5	120.0049	95.65866	1.25451	***REACHED***
m55 - Sec(b)	m5	120.0049	95.65866	1.25451	***REACHED***
m5 - Sec(b)	m5	120.0049	95.65866	1.25451	***REACHED***
m55 - Sec(b)	m5	120.0049	95.65866	1.25451	***REACHED***

Figure 26: Performance ratio for the seismic loading



Load Factor	Strain - q1	Stress - q1	Strain - f1	Stress - f1	Strain - a1	Stress - a1	Strain - q5	Stress - q5	Strain - f5	Stress - f5	Strain - a5	Stress - a5
0.00	-1.4426644E-005	-101.7078	-1.4338712E-005	-101.0879	-1.2404191E-005	-87.44955	-1.3103529E-005	-92.37988	1.2513785E-005	0.00	-1.3103529E-005	-92.37988

Figure 27: Stress and strain for the reinforced masonry element

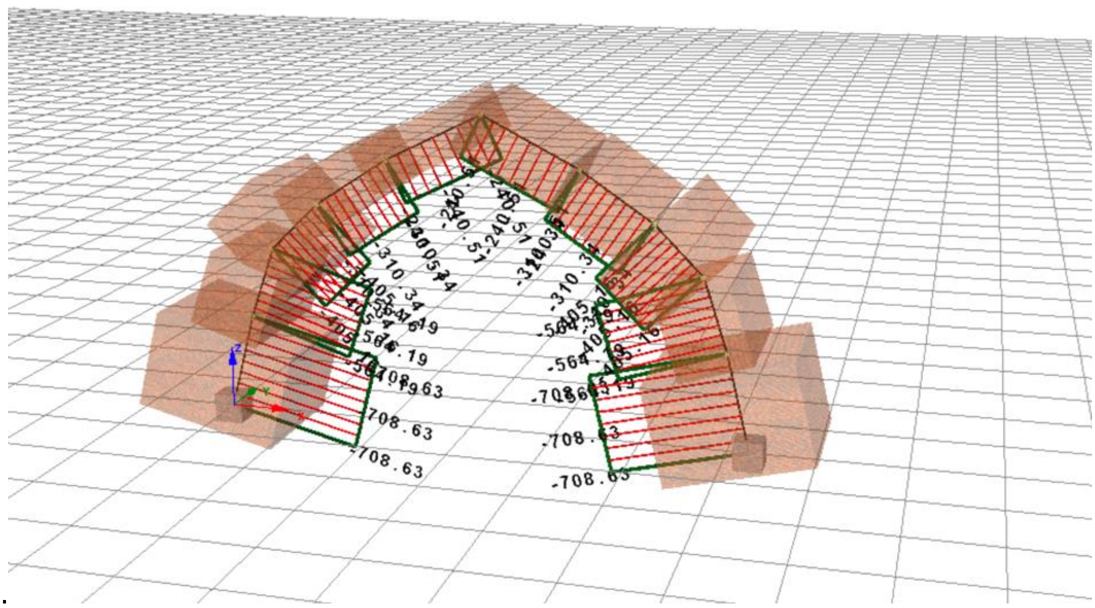


Figure 28: Axial force diagram, [kN/m]

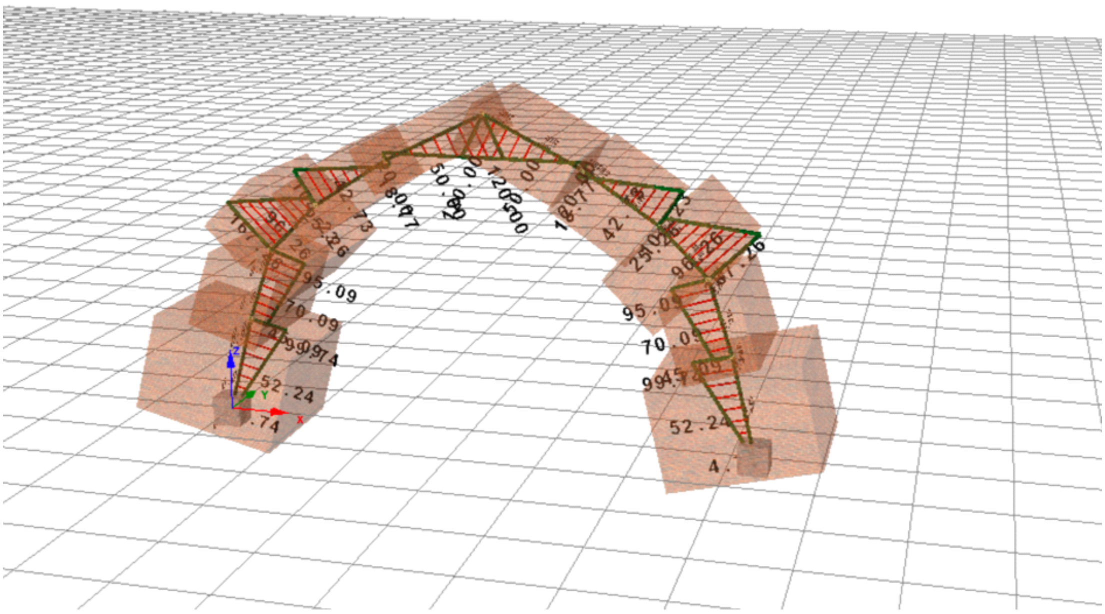


Figure 29: Shear force diagram, [kN/m]

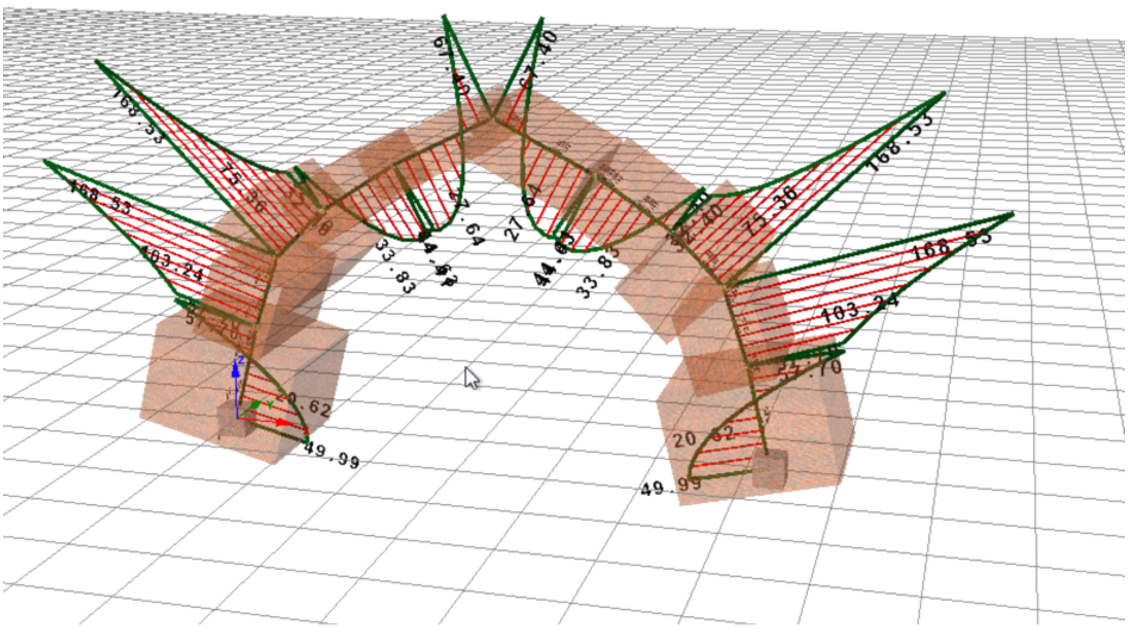


Figure 30: Bending moment diagram, [kN-m/m]

The stress analysis of the bridge after the reinforcement at the extrados (stresses envelope which takes into consideration the maximum values of all combinations for Ultimate Limit State), reveals that the normal stresses and shear stresses are below the allowable values, while tensile stresses in the intrados become zero, due to the modification of the pressure line. Stresses have been calculated at three positions: abutment, ringstone, keystone (see table below).

Table 6: Stresses calculated for the arch bridge

Normal stresses σ_{avg} (kPa) vs. design compressive strength f_d		
Abutment	Ringstone	Keystone
σ_{avg} = 340 kPa	σ_{avg} = 340 kPa	σ_{avg} = 115 kPa
f_d = 3100 kPa	f_d = 3100 kPa	f_d = 3100 kPa
Shear stresses τ_{lim} (kPa) vs. shear capacity v_{Rd}		
Abutment	Ringstone	Keystone
τ_{lim} = 360 kPa	τ_{lim} = 360 kPa	τ_{lim} = 360 kPa
v_{Rd} = 1080 kPa	v_{Rd} = 1080 kPa	v_{Rd} = 1080 kPa

The effect of the application of the stainless-steel cables, because of the rising of the compressive stresses/axial force, reduces the eccentricity of the arch and modifies the pressure curve towards the ideal compressive stress distribution of the arch. Below are presented two P-M interaction diagrams: without and with stainless steel reinforcement. As it can be seen, the application of reinforcement greatly improves the capacity of the sections of the arch to withstand loads/actions.

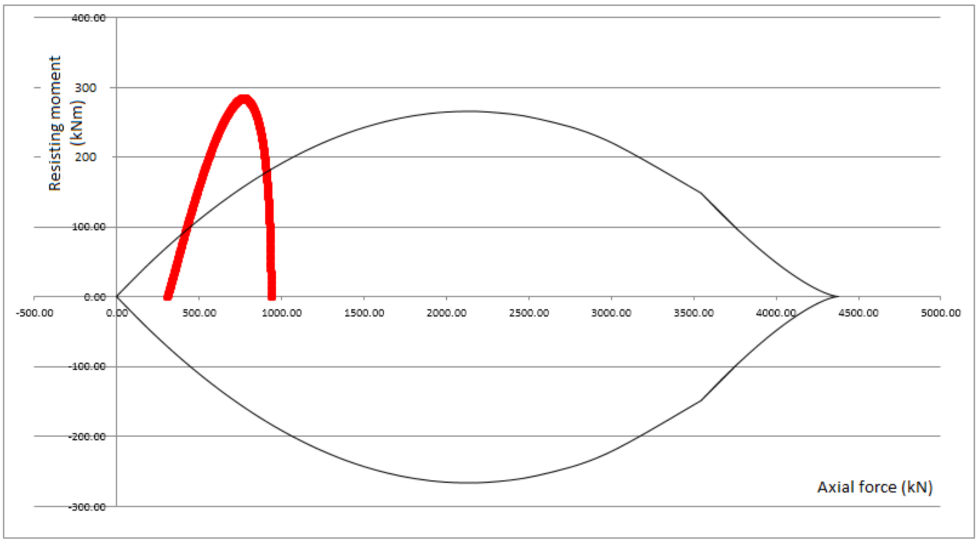


Figure 31: P-M interaction curve of the cross section of the arch without reinforcement.

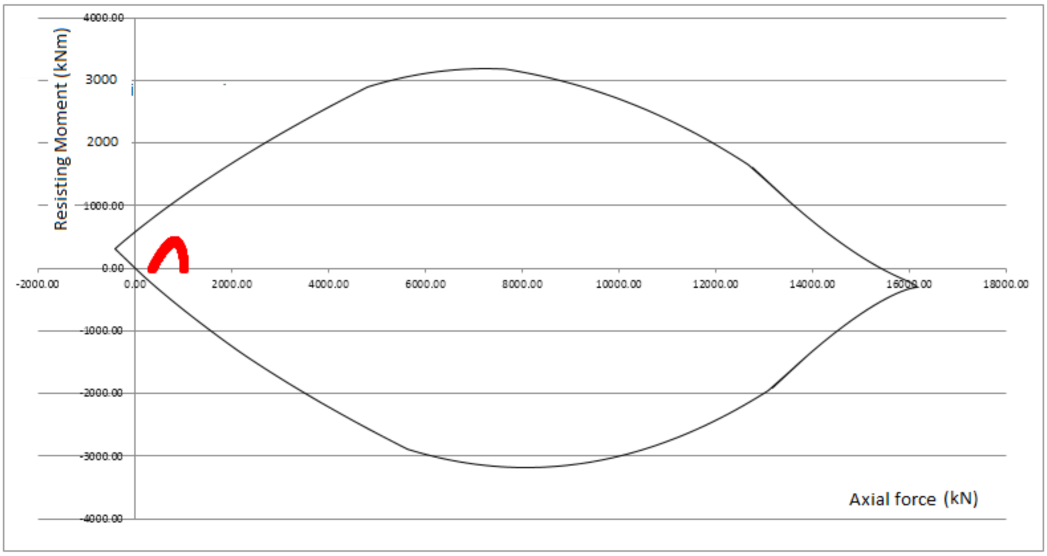


Figure 31: P-M interaction curve of the cross section of the arch with reinforcement.

Stability Checks for Bridge Foundations and Walls

Under storm conditions

- *Hydrostatic force of 3.3 m (height of the chamber)*

$P_{ws} = \frac{1}{2}\rho h^2 = \frac{1}{2} * 9.81*3.3^2 \cong 53$ kN/m

-Drag force due to water flow (v= 2.5 m/s, c_d = 2 worst case scenario)

$F_{drag} = \frac{1}{2}\, c_d\, \rho\, v^2\, A \cong 6.23$ kN/m

Σ Pw (total hydro forces) ≅ 60 kN/ m (storm conditions)

Check against sliding

(γ_{stone} = 22 kN/m^3 , H=6m, Φ=30°, c=10 kPa)

$FS_{slid} = \frac{\Sigma\, V*\tan\Phi + B*C}{\Sigma\, P_w} = \frac{6*22*\tan 30+3*10}{60} = 1.77 > 1.25$ ✓

Check against overturning

(M_o = 60$\frac{3.3}{3}$ = 66 kN*m/m)*

$FS_{over} = \frac{\Sigma\, V * X_M}{M_O} = \frac{6*22*1.5}{66} = 3 > 1.5$ ✓

Bearing Capacity check (Φ=30°, N_c = 37.2, N_g = 22.5, N_γ = 19.7)

q_{ult} = c*Nc + γ* D_f* N_g + 0.5* γ*B*N_γ (B= 3m, strip)
q_{ult} = 10*37.2 +0 + 0.5*20*3*19.7 ≅ 961 kN/m^2
σ_v=6*22=132 kN/ m^2
$FS_{bearing} = \frac{961}{132}$ = 7.28 > 3 ✓

5 CONCLUSION FOR THE REINFORCED MODEL.

As can be seen from the results presented: Under seismic computational load for transverse direction the model reaches performance points and may suffer damage. For longitudinal direction the bridge is provided according to the requirements of EC8.
For the selected type of interventions (see the decision of the Technical Council 4.1 and 4.2) the bridge is provided according to the requirements of non-damage for a seismic event with a return period of 425 years.

5.1 INTERVENTION METHODOLOGY

Reinforcement intervention in the structure will follow the following stages:

1) Clearing the square of vegetation and debris
2) Realization of river protection
3) Realization of sub-foundations in the pieces of rock where the bridge is supported.
4) Temporary protective/insulating coating of the arch and sides.
5) Molding of the supporting structure of the whole bridge, the arch from the bottom and the side faces.
6) Cleaning the filling masonry of the bridge without touching the sides and the lower arch.
7) Consolidation and Restoration of arch and sites. Structural filling of cracks with mortar and joints with mortar. The work will be realized by making partial openings of the formworks and its restoration by realizing structural scaffolding to work under the arch and on the side faces.
8) Cleaning of joints and stones from calcification and oxidation.
9) Application of steel rods with partial upper opening, drilling and injecting in the stone material at the side face.
10) Realization of reinforcement with cables in the part of the arch

11) Realization of the opening of the closed chamber by combining techniques "Stitching". Existing stones by lime mortar and Suture cracks by internal injection of Mortar from the window sides.
12) Realization of fallen walls and closing sides of the existing bridge with existing stones and mortar.
13) Removal of structural scaffolding
14) Realization of systems and protection from slope erosion on the support (A) of the bridge.
15) Realization of new connecting parts of external stone masonry with stones and joints similar to the existing one.
16) Realization of the upper cobblestone and the parapets of the bridge with existing stones and mortar.
17) Installation of monitoring benchmarks according to the project
18) Removal of the construction site and return of the work area to its natural state according to the architectural landscape.