8

THE STRUCTURAL INVESTIGATION

The constructive evaluation

Fundamental part to the survey of the artifact is the analysis of the constructive quality and the constituent materials, applied both to the structures as a whole and to the individual elements that constitute them, in order to identify the distinction of the individual parts, the identification of the supporting elements and, above all, the definition of the relations between the parts.
The **construction typology** of individual elements could be:

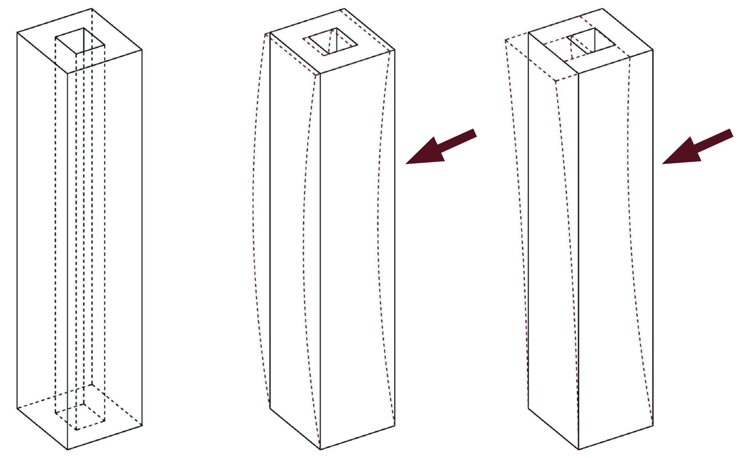
1. Foundations
2. Masonry structures
3. Vaulted structures
4. Connections between structural elements
5. Floor slabs
6. Stairs
7. Covers

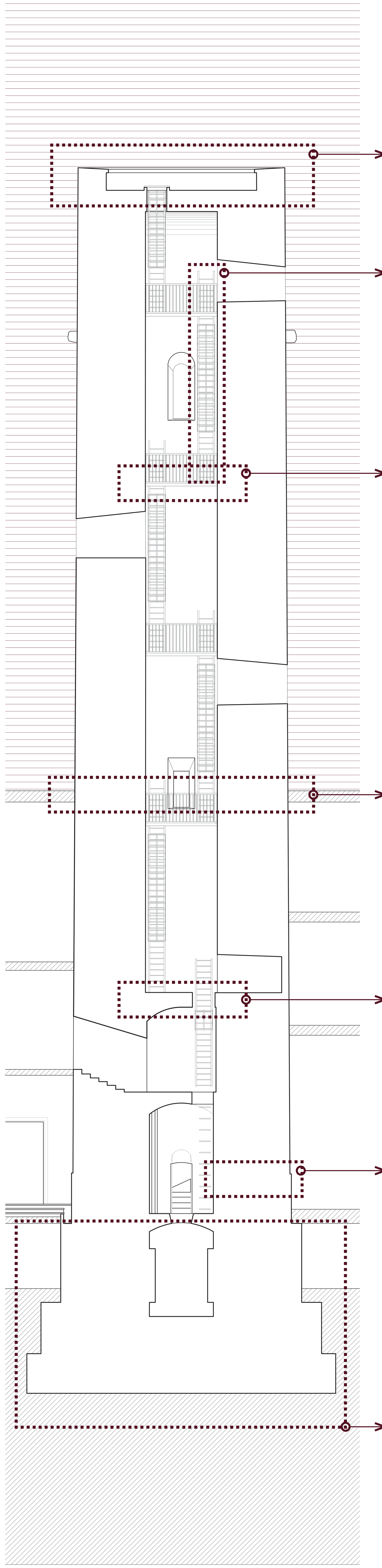
To these must be added the analysis of all the elements, not necessarily structural, with **high vulnerability**, and therefore greater sources of danger for the overall static safety of the building. In the case of the architectural typology of ***"Towers, bell towers and other structures with a predominantly vertical development"*** these are:

1. Slenderness of the structure
2. Presence in the upper part of architectural streamlined or more vulnerable elements
3. Possible presence of lower adjacent structures, capable of providing a horizontal constraint
4. Degree of amortization of the walls

THE BOX BEHEAVIOUR

Masonry, slabs and roofing, if considered stand alone, constitute a simple static scheme rather fragile. The connection of flat elements such as these, joined together by one-dimensional elements (such as slabs and arches) or spatial (such as the vault) instead to a box system characterized by a remarkable stiffness. The box behaviour of the building therefore depends not only on the quality of the individual elements that make up the structure (which is however essential) but above all on their collaboration: connections between vertical and horizontal elements play a key role in the building's response to stress.





The constructive elements

7. COVER

In the case of the Tower, there are no weakening elements typical of the prevailing vertical architectural typology, which has undoubtedly contributed to preserving a good structural stability even of the top portion. As mentioned above, the roof is made with a vaulted brick barrel structure. It is not clear whether originally the tower had a four-pitched roof, as represented in the historical engravings, as currently there are no visible signs of this possible modification.

6. STAIRS

The Tower has no masonry stairs that are part of the original structure and for the type and function of the building itself it is assumed that they did not even exist. Of these, in fact, there is not even a visual trace on the internal masonry. Today, the vertical connections are ensured only by a structure of metal ladders added retrospectively around the 70s to replace a system of old ladders and wooden landing stages.

5. FLOOR SLABS

Currently it has 4 floors made around the end of the 60's/early 70's.These are steel structures with a thickness of about 15cm that assume the function of landing. They are very rarely subjected to stress. In this specific case, the structural role of the floors is reduced in conformation, since not originally integrated into the masonry structure, although it contributes to use as an additional element of connection between the walls of the Tower and therefore constraint in the box behavior assumed by the whole in case of stress.

4. CONNECTION BETWEEN STRUCTURAL ELEMENTS

The Tower was built in a single construction phase and has not undergone substantial changes or additions over time to its original structure, it will therefore be interesting to investigate rather the connections between its walls and the buildings of the archive that have been attached to it over the years. These are in fact the points of greater stress to the movements to which the single manufactured ones, for conformation and class of use, are differently subject.

3. VAULTED STRUCTURES

The Tower has 4 vaulted ceilings, to divide the spaces of the basement, the ground floor, the first floor and the remaining free chimney. To these is also added the cover, also made with a barrel vault. From the relief of the floor of the first level, sectioned by the trap door from which you can access the free space above, you can deduce that these are brick vaults alternating with wooden joists, topped by a floor in steel, of the average variable overall thickness of about 50cm.

2. MASONRY STRUCTURES

The Tower consists of solid brick walls fixed with lime mortar 2.5m thick at the base and then gradually thinner as you climb upwards, with a loss of about 20cm on each side distributed over the entire height (37.4m) and not perceptible. The walls are made up of bricks almost regular characterized by mixtures and granularity substantially inhomogeneous, alternating with joints of mortar mostly homogeneous. For the response that the masonry in question has had to the stresses suffered over time it can be defined masonry of excellent quality.

1. FOUNDATIONS

The Tower presents (as can be deduced from the historical drawings of relief recovered and from the data collected during the geognostic surveys of the 90s) of the foundations with stone steps that extend below the floor for about 6m and widen with respect to the encumbrance of the tower above ground of about 1.6m.

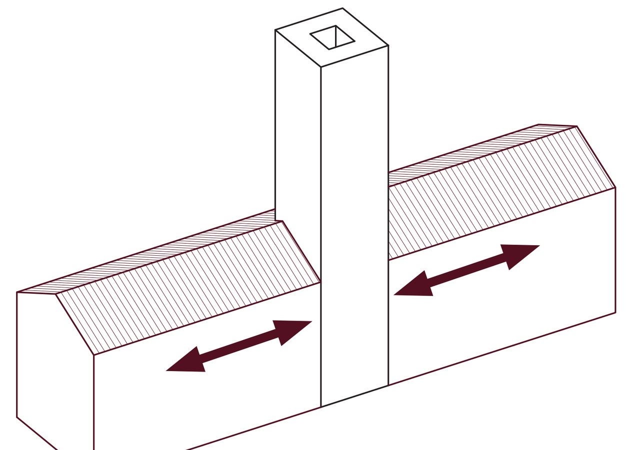
Criticality investigation

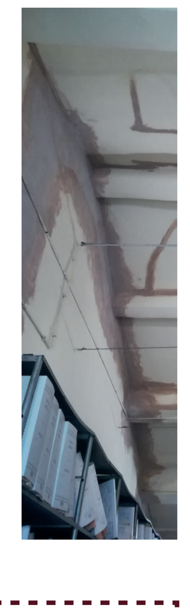
1. THE THRUST OF THE ADJACENT BUILDINGS

Injuries of disconnections due to the not perfect clamping of the two buildings walls

The presence of any adjacent buildings may provide the building with an additional degree of horizontal constraint that preserves at least the portions "boxed" from possible collapses but, at the same time, represent an additional weakening element in case of differentiated stresses associated with a low level of clamping.

CURRENT STATE

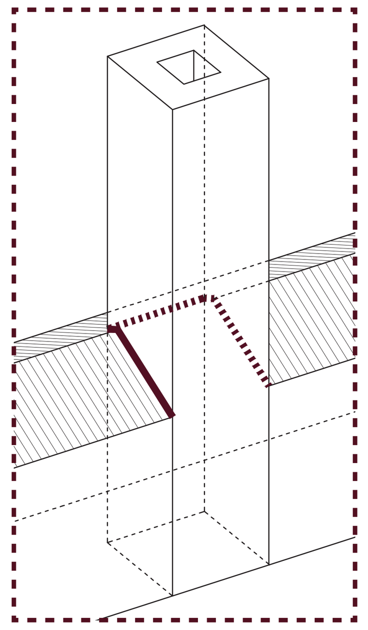




CONSOLIDATION PROJECT

Phisical cut through structural joints

Considering the reciprocal differential thrusts transmitted by the two bodies due to the different construction typology and the spatial configuration, an effective strategy to adopt could be to foresee the physical transverse cut between the two bodies realizing a separation joint to eliminate the compression current which transfers part of the load from one building to another

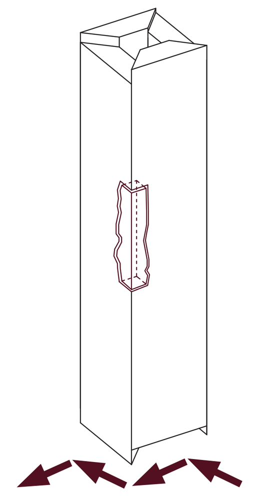


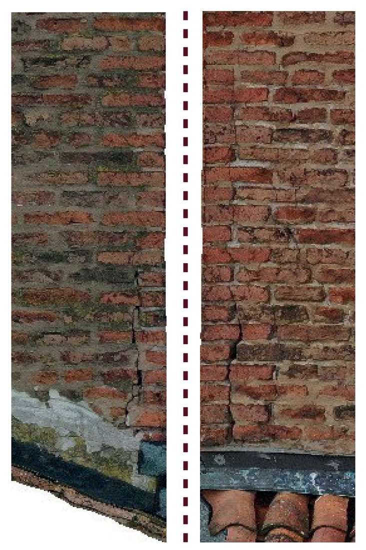
2. THE DEGREE OF CLAMPING OF THE WALLS

The edges vertical cracking

In the case of properly executed cantonal masonry, the possible collapse kinematics out of the plane can develop with the involvement of more or less large portions of masonry. It is a mechanism of rotation of portions of masonry around horizontal or vertical hinges, which usually results from differential subsidence in the foundation soil as a result, for example, of seismic phenomena. The reading of the cracking framework in this case can provide important indications on the type of activated mechanism, the response of the masonry and the possible consolidation interventions.

CURRENT STATE

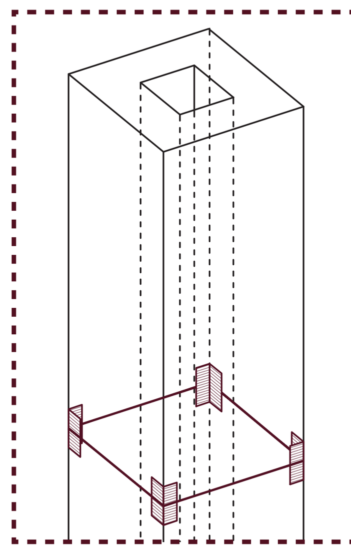




CONSOLIDATION PROJECT

External reinforcing rings

Made with metal elements or composite materials, they must be designed to avoid the onset of stress concentrations at the edges of the walls. They must be planned to be applied in different horizontal sections of the tower, properly designed depending on the particular local conditions detected on the structure.



Seismic vulnerability analysis

1. Determination of the seismic danger of the site

GEOG. COORDINATES:	Lat. 45.158 - Lon. 10.799
USE CLASS (Cu):	Class II buildings: *Buildings the use of which involves normal crowding*_Cu=1
NOMINAL LIFE (Vn):	Category 2 buildings: *Ordinary works, bridges, infrastructure works and dams of limited size or of regulatory importance_* Vn=50 years
RETURN PERIOD (Tr):	SLD_50 years | SLV_475 years

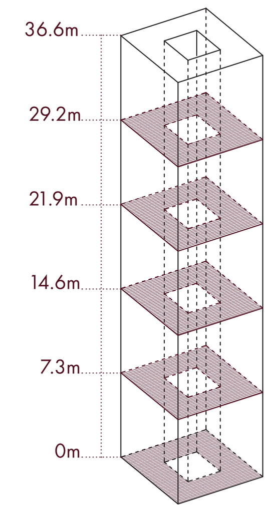
From Allegato B to the "Norme Tecniche delle Costruzioni" Lat. 45.155 - Lon. 10.798 :

	Tr	Ag	Fo	Tc*
SLD	50 anni	0.039	2.58	0.26
SLV	275 anni	0.0905	2.55	0.31
Vr=VnxCu		Vn	Cu	Vr
		50	1	50

2. Definition of the tower characteristics

			N		kg
H	36.6	V tot	1830		
B medio	7.5	M tot	3294000		
b	2.5	W tot	32314140		
Ps	1800	V aperture	0		
N° solai	0	A muro	50		
hS	0.3	V solai	0		
g	9.81	Vm1	365	M tot1	657000
λ	1	Vm2	365	M tot2	657000
		Vm3	365	M tot3	657000
h0	0	Vm4	365	M tot4	657000
h1	7.3	Vm5	370	M tot5	666000
h2	14.6		1830		3294000
h3	21.9				
h4	29.2				
h5	36.6				

NB: Calculations based on tdetector's critical approximation.
-The presence of holes was not considered
-Floors masses were not taken into account
-The cross-section side was considered to be an average 7.5
-Adjacent buildings were not taken into account



1. ACCORDING TO GUIDELINES

3. Definition of the mechanical properties of materials

In accordance with CIRC2019 and LG2011, in case of **historical masonry** the design resitances are expressed as: $fd = fm / FC \cdot \gamma M$
$\tau d = \tau 0 / FC \cdot \gamma M$

With: $fm = (2.4+4.0) / 2 = 3.2$
$\tau 0 = (0.06+0.092) / 2 = 0.076$
$Fc = 1 + \Sigma Fck = 1 + (0+0+0.06+0) = 1.06$
$\gamma M = 2$

Fc1= Complete geometric survey
Fc2= Complete historical and constructive knowledge
Fc3= Partial knowedge of the materials properties
Fc4= Complete soil and foundations knowledge

Determing a design resistance of:

$fd = fm / Fc \cdot \gamma M = 3.2 / 1.06 \cdot 2 = 1.51$
$\tau d = \tau 0 / Fc \cdot \gamma M = 0.076 / 1.06 \cdot 2 = 0.036$

4. Calculation of the last resisting moment

In accordance with LG2011, in the case of a **rectangular tower with hollow section** the last moment resistant to the base can be calculated as: $Mu = \sigma 0 \cdot Ab / 2 \cdot (bb - \sigma 0 \cdot Ab / 0.85 \cdot ab \cdot fd)$

The values from which depends the last resisting moments are the following, deriving from the geometry and mechanics of the tower:

σ0 [Mpa]	628.6	lato a [m]	7.5
W [kN]	32314	lato b [m]	7.5
Ab [m2]	51.41	fd [Mpa]	1.51

For a last resisting moment of:

$Mu = 628.6 \cdot 51,41 / 2 \cdot (7.5 - 628.6 \cdot 51.41 / 0.85 \cdot 7.5 \cdot 15$

5. Estimation of seismic actions

In order to determine the load pattern for the calculation to the Damage Limit State (SLD) and the Life Safeguard Limit State (SLV), stress in the x and y directions (which in our case, will be equal), the seismic action is modelled in the same way as an equivalent seismic force $Fh = Sd(T) \cdot W \cdot \lambda / g$

	Sd(T)	W	λ	g	Fh
SLD	0.094	32314	1	9.81	3034
SLV	0.087	32314	1	9.81	2796

The Equivalent Seismic Force is distributed linearly along the entire height of the building in many Fi forces. More precisely, for each mass of the construction placed on the i-th plane, the applied seismic force is equal to: $Fi = Fh \cdot Zi \cdot Wi / \Sigma Z_j \cdot W_j$

The partition coefficients Pi determines the fractions of the force Fh acting on the masses at different altitudes: $Pi = Zi \cdot Wi / \Sigma Z_j \cdot W_j$

Livello i	Zi (m)	Wi (kN)	ZixWi	Pi
1	7.3	6445	47050	0.08
2	14.6	6445	94099	0.16
3	21.9	6445	141149	0.24
4	29.2	6533	190777	0.32
5	36.6	3267	119562	0.20
		ΣWjxZj	592638	1.00

	Fi 1	Fi2	Fi3	Fi4	Fi5
SLD	241	482	723	977	612
SLV	222	444	666	900	564

6. Evaluation of the seismic safety index Is, SLV and the acceleration factor fa, SLV

From the value of the elastic spectrum ordinate Se, SLV leading to the limit state of the belfry, an iterative procedure is used to determine the return time T,SLV of the earthquake whose spectrum assumes this value at the period T1 of the structure.
Considering:
$Se,SLV = (q \cdot g \cdot Mu \cdot \Sigma W_j \cdot Z_j) / (0.85 \cdot W \cdot (\Sigma W_j \cdot Z_j)2 - (z \cdot \Sigma W_j \cdot Z_j)) \cdot Fc)$

q	2.8		
g	9.81		
Mu	66920		
ΣWjxZj	592638		
W	32314	T1	0.6844
Σ(WjxZj)^2	14755154	S	1
z	0	F0	2.55
Fc	1.06	Tc	0.31
Se,SLV	2.54	aSLV	2.20
Tr,SLV	2475	ag,SLV	0.89
IS,SLV	5.22	fa,SLV	2.47

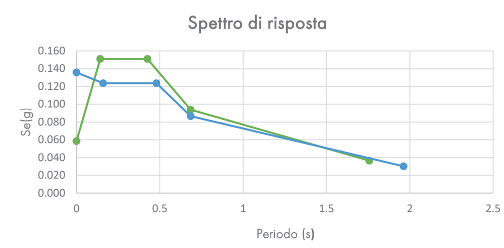
Is, SLV = Tr,SLV / TR,SLV
= 5.22>1
Fa, SLV = a,SLV / Ag,SLV
= 2.47>0.5

2. PRESS-FLEXION VERIFICATION

3. Definition of elastic response spectrum

BUILDING TYPOLOGY:	*Masonry historical towers*: T=0,0187·H=0.0187·36.6=**0.68s**
SUBSOIL CATEGORY (Mantova):	C: *"Deposits of medium-thickened coarse-grained soils or medium-consistent fine-grained soils"*_Ss=1.64/1.56 | Cc=1.64/1.55
TOPOGRAPHIC SURFACE:	T1: *"Flat surface, slopes and isolated reliefs with average inclination ≤ 15 °"*_St=1
STRUCTURE FACTOR:	*Towers and other predominantly vertical structures with adjacent structures in contact* _ q=2.8

	To	Tb	Tc	Td	S	Ag	Fo
SLD	0	0.14	0.43	1.76	1.5	0.039	2.58
SLV	0	0.16	0.48	1.96	1.5	0.0905	2.55



	0<T<Tb	Tb<T<Tc
SLD	0.059	0.151
SLV	0.136	0.124
	Tc<T<Td	**Td<T**
SLD	0.094	0.037
SLV	0.087	0.030

4. Estimation of the fundamental period

For Italian Legislation, *"Civil or industrial masonry buildings with a height of ≤ 40 meters and with a mass that can be considered as uniformly distributed along the entire height"*:
T= 0,0187·H = 0.0187·36.6 = **0.6844s**
F = 1/T = 1/0.6844s = **1.461Hz**

5. Estimation of seismic actions

Same procedure exposed for the ***"According to guidelines"*** method:

	Fi 1	Fi2	Fi3	Fi4	Fi5
SLD	241	482	723	977	612
SLV	222	444	666	900	564

6. Resolution of the isostatic structure and analysis of the M, N, T stresses

The values of N have been recalculated taking into account the weight of the floors considered individually weighing on each section considered (in our case 5: P0, P1, P2, P3, P4):

	N	kN
N0	6445170	6445
N1	6445170	6445
N2	6445170	6445
N3	6445170	6445
N4	6533460	6533
Nr		32314

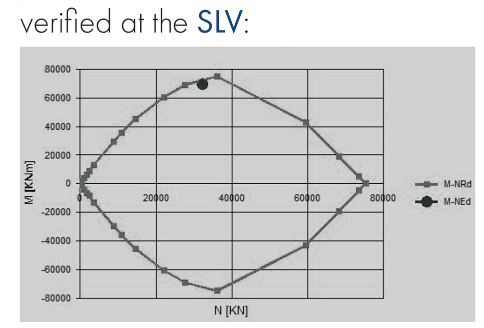
By imposing the equilibrium conditions on translation and rotation, the values of the constraint reactions were found with which to calculate the internal actions for the entire length of the cantilever.

In SLV:
Vertical traslation: Σ ΣN=0 Nr-N =0 Nr= 32314 kN
Horizontal traslation: ΣT=0 Tr+Fi =0 Tr = 2796 kN
Rotation: M=0 Mr-Fi · Zi =0 Mr = 69622 kNm

SLV				
0_1	N(z)	25869		
	T(z)	2796		
	M(z)	69622	Z=0	
		49209	Z=7.3	
1_2	N(z)	19424		
	T(z)	2574		
	M(z)	49209	Z=7.3	
		30416	Z=14.6	
2_3	N(z)	12979		
	T(z)	2130		
	M(z)	30416	Z=14.6	
		14864	Z=21.9	
3_4	N(z)	6533		
	T(z)	1464		
	M(z)	14864	Z=21.9	
		4175	Z=29.2	
4_5	N(z)	0		
	T(z)	564		
	M(z)	4175	Z=29.2	
		0	Z=36.6	

Sezione	N(kN)
0	32314
1	25869
2	19424
3	12979
4	6533
5	0

Considering a value **fd = 1.51** as calculated by a regulatory procedure, the most stressed section, i.e. the one at the base, is verified at the SLV:



N=32314kN
M=69622kNm