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REDEVELOPMENT OF EXISTING BUILDINGS: THE CASE OF THE "GALFA" TOWER IN MILAN

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ABSTRACT

A detailed structural analysis (under seismic and wind loads) of an historical tall building in Milan is carried out in this paper. Galfa tower is one of the first tall building (109 m height) built in Italy during the '50 years. Nowadays, it is interested by an important restoration process involving also a change in its intended use (from office to luxury hotel and residences).

Several destructive, non-destructive and combined tests were performed in order to investigate the on-site characteristics of concrete. Moreover, additional mechanical and chemical tests on the steel reinforcement are performed too. Some finite elements models (FEMs) of the tower are implemented by using beam and plate elements and considering two different boundary conditions (fully constrained at foundation level and elastic soil support according to Winkler's model). The interaction with the close existing lower buildings is considered as well.

In all of the FEMS the materials characteristics are assigned on the basis of the statistical interpretation of the on-site test results. The seismic and wind loads are applied according to the Italian Design Code (NTC).

The structural safety verifications are carried out in terms of shear and combined compressive-bending actions, whereas further ductility verifications are conducted considering suitable nonlinear behaviours of concrete and steel rebars.

Keywords: Tall Building, Structural Analysis, Restoration

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1. INTRODUCTION

Galfa Tower is an historical tall building in Milan (Italy), designed by the architect M. Bega in 1956 and built in 1959. The name “Galfa” comes from the location of the building, which is placed at the intersection of Galvani street and Fara street (the name is the acronym made of the first syllable of the two streets). The tower is 102.5 m height (from the ground level to the 31st floor) and its typical floor has a rectangular plan (dimensions: 37.5 × 15.75 m). The underground floors reach 10 m below the ground level. Around the tower, there are lower buildings (in the following called CB “Corpi Bassi”) characterized by two aboveground levels and two underground levels.

Figure 1. Historical view of the tower.

2. TESTS AND SURVEYS

An adequate knowledge level of an existing building is necessary in order to evaluate the structural safety of the building and to perform the structural analysis under earthquake and wind loads. The knowledge level of the building is based on the awareness of its history (by means of an historical analysis), its geometry (from survey of the structural elements in the “as built” configuration) and its materials (through mechanical tests). Regarding the mechanical characteristics of materials, the evaluation of the concrete compressive strength, concrete elastic modulus and characteristics of the steel reinforcement (number and diameter of bars and stirrups, detailing of reinforcement and its mechanical properties) are fundamental. For this reason, specific tests have to be carried out on representative elements of the building.

For instance, in a generic column, the evaluation of the reinforcement (longitudinal bars and stirrups) has to be performed in mid-height section and in the critical sections (base and top of the column). On the other hand, in a generic beam, the evaluation could regard the lower bars located at the half span of the beam and the stirrups placed close to the ends of the beam. About the steel reinforcement, the mechanical tests are performed on few samples taken from the structure, so the obtained information about the bars and stirrups could be considered as limited. Thus, a comparative analysis between the test results and the material prescriptions included in the original drawings has to be done to find the necessary correspondence.

In the case of Galfa tower, the original structural drawings were available and the checks between them and the detected information about the "as built" structure were positive both in terms of dimensions of the concrete elements and detailing of reinforcement.

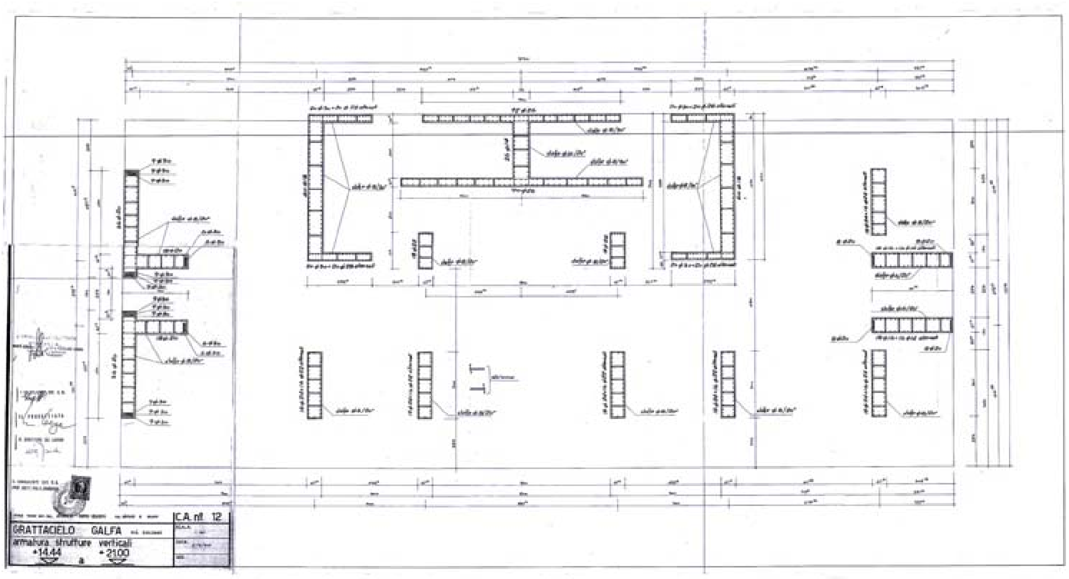

Figure 2. Example of an original design drawing.

The evaluation of the position and diameter of the reinforcement bars in the structures was carried out through an instrument called "pachometer". The tool consists of a feeler that emits magnetic field. This feeler is connected to a unit of digital and acoustics processing. The feeler is made to slide along the surface of the concrete to be investigated and, by the absorption of the magnetic field, the position of the bars surface, the thickness of the concrete cover and the diameter of the bars can be detected with good approximation.


Figure 3. Survey after pachometer investigation.

2.1 Concrete compression tests

Concrete compression tests allow the determination of either concrete compressive strength (by using sample characterized by the ratio H/D = 1, where H is the sample height and D is the diameter) and elastic modulus E (by using sample with H/D = 2). About the concrete strength, knowing the compression test results (either in terms of cubic strength R_c or cylindrical strength fc), the "in situ" compression strength (in the following: "in situ" cubic strength R_c* or "in situ" cylindrical strength f_c*) could be evaluated by means of the following coefficients c_1 and c_2. The coefficient c1 depends on the geometry of the sample whereas the coefficient c_2 considers the disturbance due to the extraction of the sample (concrete core) from the generic concrete element.

$c_1 = 1 + 0.25 \cdot [(H/D) - \alpha]$ with $\alpha = 1$ or 2;

$c_2 = 1/0.85$ (according to Malhotra [8]); or $c_2 = 1/0.94$ (according to ACI94)

By the application of the above coefficients c_1 and c_2 to the compression test results, the "in situ" average cubic strength R_{cm}* of concrete of the Galfa tower are obtained (the calculated values are listed in the following Table 1.

Table 1. R_{cm}* concrete strength of Galfa tower.

H/D=1 – TOWER									
FLOOR	TEST	D (mm)	H (mm)	H/D	Rc (Mpa)	c_1	c_2	Rc* (MPa)	Rc m* (MPa)
FONDATIONS	C(F)7	99	99	1,00	45,34	1,000	1/0,94	48,2	47,5
	C(F)10	99	97	0,98	44,17	0,995	1/0,94	46,8	
-2	C(-2)9	99	97	0,98	38,97	0,995	1/0,94	41,3	41,3
0	C(0)0A	99	100	1,01	27,93	1,003	1/0,94	29,8	35,8
	C(0)CB	99	93	0,94	35,34	0,985	1/0,94	37,0	
	C(0)4	94	107	1,14	30,55	1,035	1/0,94	33,6	
	C(0)5A	99	88	0,89	40,66	0,972	1/0,94	42,1	
	C(0)5B	99	99	1,00	34,56	1,000	1/0,94	36,8	
6	C(6)2A	99	104	1,05	30,66	1,013	1/0,94	33,0	36,5
	C(6)2B	99	109	1,10	26,76	1,025	1/0,94	29,2	
	C(6)3	99	106	1,07	27,93	1,018	1/0,94	30,2	
	C(6)5B	99	105	1,06	45,08	1,015	1/0,94	48,7	
	C(6)6	99	99	1,00	35,47	1,000	1/0,94	37,7	
	C(6)7A	99	100	1,01	33,00	1,003	1/0,94	35,2	
	C(6)7B	99	100	1,01	38,58	1,003	1/0,94	41,1	
12	C(12)1	99	101	1,02	32,48	1,005	1/0,94	34,7	29,9
	C(12)3A	99	97	0,98	26,50	0,995	1/0,94	28,1	
	C(12)5	99	101	1,02	25,07	1,005	1/0,94	26,8	
18	C(18)3	99	97	0,98	25,59	0,995	1/0,94	27,1	27,1
24	C(24)2A	99	103	1,04	24,68	1,010	1/0,94	26,5	30,1
	C(24)2B	99	100	1,01	28,97	1,003	1/0,94	30,9	
	C(24)3	99	102	1,03	30,66	1,008	1/0,94	32,9	
29	C(29)1	99	102	1,03	30,01	1,008	1/0,94	32,2	31,9
	C(29)2bis	99	100	1,01	32,61	1,003	1/0,94	34,8	
	C(29)3	99	103	1,04	26,89	1,01	1/0,94	28,9	

2.2 Concrete elastic modulus

The determination of the concrete elastic modulus E is an essential task because, in a tall building, the dynamic response due to seismic or wind loads could largely depend on E.

For the case of Galfa tower, the experimental values of E obtained from the test campaign gave 15 values included in the acceptable range 20000-30000 MPa and some unusual values lower than 20000 MPa or higher than 30000 MPa (unusual if compared to the concrete strength values shown in the previous Table 1).

In the concrete elastic modulus estimation process, the unusual values of E were discarded and, from the uniform distribution, a clear values clustering included in the range 24.000 to 27.000 MPa come out. At the end of the statistical interpretation, also taking into account the correlation with the above mentioned compressive strength, a concrete elastic modulus equal to E = 27000 MPa was determined.

2.3 SonReb method

In order to complete the evaluation of the concrete resistance by the combined SonReb method, the ultra-sonic and sclerometric tests are carried out. For the sonic tests, the evaluation of the resistance has been performed either by the direct waves transmission and the indirect transmission, depending on the accessibility from the two opposite “faces” of the wall tested. The (“in situ) average velocities

(v_m*) are shown in the next Table 2. The sonic tests were carried out on n.°5 floors, for each floor the average velocity is always indicated in Table 2 with the Leslie and Cheesman "judgment" [9] in relation to the average velocities.

Table 2. Values of concrete quality according to literature.

JUDGMENT	AVERAGE SPEED OF FLOOR	AVERAGE SPEED
	v_m*(m/S)	(m/S)
terrible	<2.135	2.135,00
poor	2.135-3.050	2.595,50
discreet	3.050-3.660	3.355,00
good	3.660-4.575	4.117,50
excellent	>4.575	4.575,00

The sclerometric tests were performed testing from n.°2 to n.°5 elements for each floors (exept at the 29th floor where only one element was tested). From the average rebound indices (Ir,m) the corresponding ("in situ") average compression resistance was valued $R_{c,\,m}$* = 41.2 MPa and the variation coefficient c* = 0.086 is calculated too.

Figure 4. Example of ultra-sonic and sclerometric tests.

Finally, combining the ultra-sonic velocities (v_m) with the rebound indices ($I_{r,m}$) by the following relations (where I_{rm} is the average value of I_r and v^*_m is the average of v*), the SonReb method gives an "in situ" average resistance : $R_{c,m}$* = 27 MPa for the floors in elevation and $R_{c,m}$* = 36 MPa for the underground floors.

(RILEM 1993) $R^*_c = 7.695 \cdot 10^{-10} \cdot I_{r,m}1.450 \cdot v_m2.58$, with R^*_c [daN/cm^2] and v_m [m/s]

(GASPARIK 1992) $R^*_c = 0.0286 \cdot I_{r,m}1.246 \cdot v_m1.85$, with R^*_c [N/mm^2] and v_m [Km/s]

(DI LEO, PASCALE 1994) $R^*_c = 1.2 \cdot 10^{-9} \cdot I_{r,m}1.058 \cdot v_m2.446$, with R^*_c [N/mm^2] and v_m [m/s] .

2.4 Carbonation tests

To investigate the durability of the concrete, carbonation tests were performed on all concrete samples extracted from the vertical structural elements. The carbonation test consists in a phenolphthalein solution sprinkling on the external sample's surface. The absence of carbonation is signaled by fuchsia color reaction, otherwise the concrete is considered carbonated if the color of the sample doesn't change. The portion of the generic sample without the fuchsia coloring is generally measured in (linear) mm and it is called carbonation depth (u). For the Galfa case, u was averagly lower than 20 mm except in a sample extracted from the basament for which u was about 35 mm.

Figure 5. Example of carbonation test.

2.5 Tensile tests on steel

Some tensile tests were performed on the steel bars; the average yield stress is $f_y \cong 360$ MPa and consequently the tensile strenght is $f_t \cong 530$ MPa.

3 STRUCTURAL MODELING

After the knowledge of geometries and the materials characteristics, some FEMs have been implemented to perform the seismic and the wind structure analysis (referred to the Italian Structural Code, NTC 2008). In the n.°4 implemented FEMs the influence of the CB (like mentioned, the lower construction close the Tower) and the interaction of the structure with the soil are analysed like it is explained in the following cases noted 1a) , 1b), 2a) and 2b).

case 1a), Tower (fixed at the ground floor) with CB;
case 1b), Tower (with soil deformations at the ground floor) without CB;
case 2a), Tower (fixed at the base; the base is two level underground) without CB;
case 2b), Tower (with soil deformations at the base; the base is at two levels underground) without CB.

In the cases 1a) and 2a), the deformable soil is represented by Winkler springs having the coefficient k = 180000 kN/m^3.

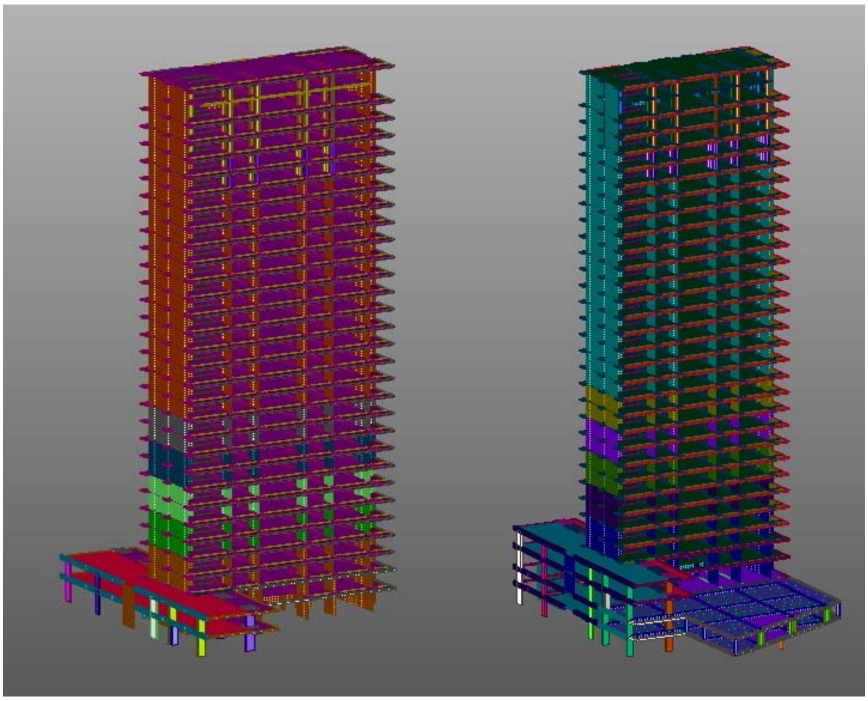

Figure 6. FEMs of the Galfa Tower.

Starting from the drawing in DXF of the geometric survey, the new drawing, cleansed of superfluous elements for the purposes of structural modelling has been obtained. It was then imported into MidasGen, forming the basis for the modelling of the structural elements.

- Beams and columns were modelled by Beam elements (Figure 7):

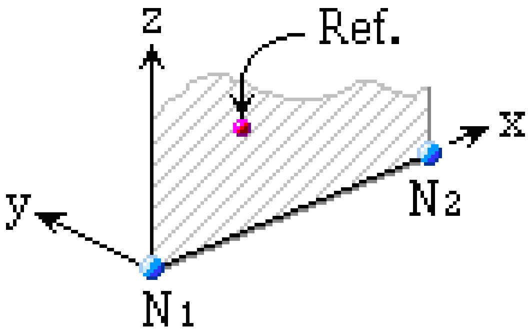

Figure 7. Representation beam element FEM.

- Floor and walls were modelled by Plate elements (Figure 8):

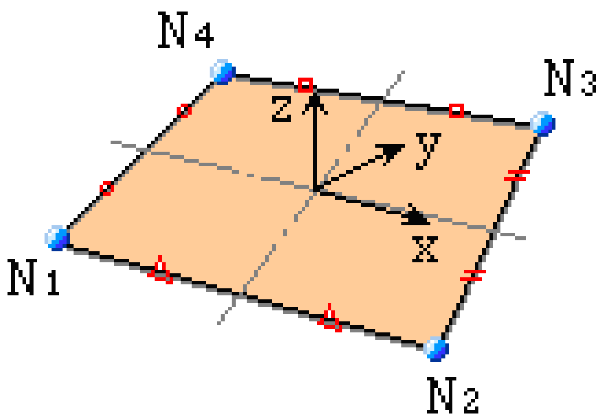

Figure 8. Representation plate element FEM.

The deck slab (thickness 20 cm, characterizing the floors -1 and -2) and all the walls are implemented with bidirectional plate elements, the others floors of the Tower (realized in reinforced concrete and hollow tiles mixed floor of 25+7 cm) were implemented by "one-way" plate elements and "stiffened" function. By the function, once the single plate has been designed, it is possible to insert the geometries of the floor in a different way according to its local plans xz and yz (x, y and z are the local axes of the plate element representatives of the floor).

Then, Materials, Sections and Thickness have been defined in the "Properties" section.

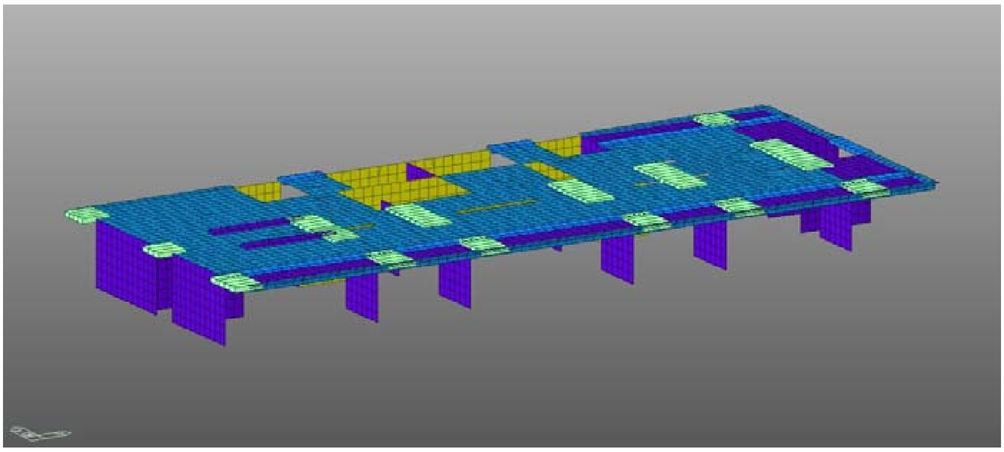

Figure 9. Representation of the floor type.

3.1 Material Properties

The characteristics of the materials (concrete and steel) concerning the existing structures have been obtained from the results of tests, as reported in the above paragraphs. Given the large number of tests carried out, an appropriate level of knowledge of the structure has been reached. Confidence factors equal to 1 can be correlated (that is, you have a Confidence level LC3 such as not to require any reduction of the values of the characteristics of the materials obtained from the tests).

As regards the concrete the following values have been implemented in the FEM:

- for the compressive strength, in reference to the characteristic values
  Floors in elevation above the 6th, R_{ck} * = 26 MPa;
  Floors in elevation below the 6th, R_{ck} * = 28 MPa;

- for the elastic modulus E_c * (* = E_{cm}) = 26,000 MPa;

- for the Poisson's ratio for all plans v = 0.125.

As regards the steel of the reinforcing bars :

- for the yield stress, in reference to the characteristic value , f_{yk} * = 310 MPa;

- for the ultimate stress, in reference to the characteristic value, f_{tk} * = 482MPa;

- for the elongation at rupture, in reference to the characteristic value of the elevation floors, A_{gk} * = 13%.

Finally, for the steel ladder of new construction the characteristics of class S275 steel have been considered, inserted automatically by the software:

- Elastic modulus, E = 2.1 ·108 kN / m^2;

- Poisson's ratio, v = 0.3;

- Density, ρS275 = 76.98 kN / m^3.

3.2 Applied loads

The vertical loads, in addition to the own weight of the structural elements (calculated automatically by the software through the geometry and density), are: the permanent loads , non-structural loads , the live loads and snow load . All these loads were included as pressure loads on the floors.

In particular, the non-structural loads, applied in the different "areas" of the floors, are reported in the table 3.

The live loads (Q) are:

1) live load applied in all floors, Q1 = 200 kg / m (= 2 kN / m,in according to Table 3.2.2 of DM 2008, cat. A);
2) live load applied in the roof , Q2 = 50 kg / m (= 0.5 kN / m, in according to Table 3.2.2 of DM 2008, cat. H1).
3) snow load, in according to DM 2008 is:

μ_i	q_{sk} [kN/m^2]	C_E	C_t	q_s [kN/m^2]
0,8	1,50	1	1	1,20

where:

μ_i is the form coefficient of the roof;

$q_{sk}$is the characteristic value of the snow load;

c_E is the exposure factor;

C_t is the thermal coefficient.

Table 3. Example of non- structural loads.

PARETE IN CARTONGESSO				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Lastra in cartongesso	900	0,0125	4	45
Lana di roccia	30	0,075	1	2,25
			TOT	47,25
			Incr. 10%	**51,98**
DOPPIA PARETE IN CARTONGESSO				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Lastra in cartongesso	900	0,0125	5	56,25
Lana di roccia	30	0,05	2	3
			TOT	59,25
			Incr. 10%	**65,18**
PLACCAGGIO IN CARTONGESSO PARETE IN C.A.				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Lastra in cartongesso	900	0,0125	2	22,5
Lana di roccia	30	0,05	1	1,5
			TOT	24
			Incr. 10%	**26,40**
PAVIMENTO IN MARMO				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Massetto	1200	0,08	1	96
Strato 1	2800	0,02	1	56
			TOT	152
			Incr. 10%	**167,20**
MOQUETTE				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Massetto	1200	0,055	1	66
Strato 1	1800	0,005	1	9
Strato 2	800	0,02	1	16
			TOT	91
			Incr. 10%	**100,10**
PAVIMENTO VINILICO (COME SOLUZIONE RESINA)				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Massetto	1200	0,095	1	114
Strato 1	800	0,005	1	4
			TOT	118
			Incr. 10%	**129,80**
PAVIMENTO GRES PORCELLANATO				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Massetto	1200	0,085	1	102
Strato 1	2200	0,005	1	11
Strato 2	2800	0,01	1	28
			TOT	141
			Incr. 10%	**155,10**
PAVIMENTO IN LEGNO				
	PESO SPECIFICO (kg/mc)	**SPESSORE (m)**	**N**	**PESO PER UNITA' DI SUPERFICIE (kg/mq)**
Massetto	1200	0,055	1	66
Strato 1	1800	0,005	1	9
Strato 2	800	0,02	1	16
			TOT	91
			Incr. 10%	**100,10**
FACCIATA				
	Perimetro	**PESO/m**	**TOT (Kg)**	**TOT (t)**
	70,84	330	23377,2	23,3772
IMPIANTI				
				PESO PER UNITA' DI SUPERFICIE (kg/mq)
				150

The horizontal loads are wind and earthquake.

For the wind, the load was applied both longitudinally, in according to the global axis x and transversely, in according to the global axis y, in the model.

For the earthquake, the structure factor q is calculated as:

$$q = q_0 \bullet K_R \bullet K_W$$

where:

q_0 is the maximum value of the structure factor. It depends by the ductility class provided;

K_R is a reduction factor . It depends by the characteristics of regularity in height of the building;

K_W is a reduction factor for the walls stubby, this is used to reduce the risk of brittle fracture.

The structure factor is:

$$q = q_0 \cdot K_R \cdot K_W = 3 \cdot 1 \cdot 0{,}65 = 1{,}95 .$$

To implement the response spectrum earthquake, the seismic input data are:

Nominal life, 50 years;

Use Class II, with use coefficient Cu = 1,

low ductility class;

soil category ,type C;

topographic conditions, type S1 ;

Reference period of the structure , 50 years.

With these data, the spectrum, implemented in the software, is as follows:

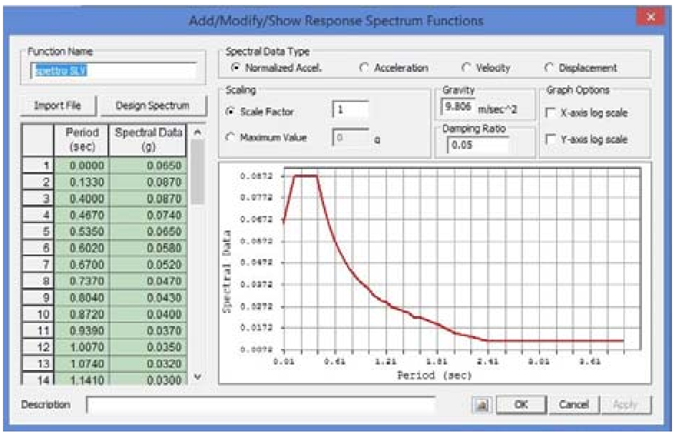

Figure 10. Representation of the SLV response spectrum earthquake.

4 RESULTS OF THE ANALYSIS

The structural analysis consisted in:

- the identification of the characteristics of the structure (in particular: the modes of vibration (periods) and mode shapes), for each of the four models;
- the study of the response of the structure, in terms of displacement of the top and maximum displacements of inter-floor (floor drift), under vertical loads and horizontal actions (wind and earthquake); the action is applied both transversely and laterally and in the two directions;
- The definition of efficiency ("capacity / demand") of walls and pillars, in terms of resistance (ρ) and deformation (δ).

For each FEM, the eigenvalue analysis is performed and the vibrational mode shapes are valued in terms of frequencies, percentages of the mass and deformed shapes. Moreover, the vertical dead and live loads are applied like pressure or linear loads on the plate and beam elements. Whereas the seismic action is implemented by the site spectrum . The wind action is represented by an horizontal force vertically distributed. All the vertical and lateral loads are estimates like following the Italian Construction Code NTC 2008 and are themselves combined for the Serviciability Limit States (SLS), Ultimate Limit State (ULS) and Life Safety Limit State (LSS). For all the mentioned cases (1a)÷2b)), the results of the eigenvalue analysis, the displacements and the inter-storey drifts due to the earthquake and wind actions are shown in Table 3. The index L and T in the Table 3 respectively means if the horizontal loads are applied longitudinally (L) or transversely (T) like it is also drawn in the previous Figure 3.

Table 4: Vibration modes, drift of floor and displacement into the top in direction (T) and (L)

FLOOR	TRANSVERSAL ACTION	MODEL			
		drift of floor (mm)			
		impeded movements		free movements	
		1a)	1b)	2a)	2b)
-2	earthquake	-	1,2	1,2	1,3
	wind	-	1,5	1,5	1,5
0	earthquake	1,6	2,5	2,5	3,0
	wind	0,5	1,5	2,0	2,0
6°	earthquake	2,2	3,0	3,0	3,0
	wind	1,6	2,0	2,0	2,0
14°	earthquake	1,3	2,0	2,0	2,0
	wind	1,6	2,0	1,5	1,0
18°	earthquake	1,2	1,0	1,0	1,0
	wind	1,8	2,0	2,0	2,0
25°	earthquake	3,4	2,0	2,0	2,0
	wind	3,0	3,0	3,0	3,0
29°	earthquake	3,6	2,0	3,0	3,0
	wind	2,7	3,0	3,0	3,0

Model	Mode n.°	Period sec.	TRANSLATION X Mass %	TRASNSLATION Y Mass%	ROTATION Z Mass %
1a)	1	3,93	16,64	30,99	16,05
	2	3,54	24,37	29,05	5,22
	3	2,61	25,11	0,69	33,66
	4	1,09	5,51	3,67	5,49
	5	0,93	3,85	11,62	0,24
	6	0,72	3,80	1,04	7,80
	7	0,51	2,64	0,93	2,65
	8	0,42	1,81	4,37	0,03
	9	0,34	1,44	1,30	3,71
	10	0,31	1,52	0,34	2,05
2a)	1	4,15	16,76	28,37	13,26
	2	3,77	24,33	27,84	4,97
	3	2,76	22,72	0,69	30,33
	4	1,16	5,53	3,72	5,61
	5	1,00	3,65	12,57	0,58
	6	0,78	3,87	0,88	9,10
	7	0,55	2,57	1,06	2,61
	8	0,45	1,64	4,58	0,00
	9	0,37	1,36	1,03	3,81
	10	0,34	1,32	0,45	1,61
1b)	1	4,14	15,67	27,86	10,11
	2	3,74	23,24	25,65	2,35
	3	2,73	20,63	0,68	26,89
	4	1,15	5,29	3,49	4,08
	5	0,99	3,64	11,17	0,18
	6	0,76	3,49	1,03	6,56
	7	0,54	2,96	0,76	2,82
	8	0,45	1,78	4,98	0,00
	9	0,36	1,77	1,64	3,91
	10	0,34	1,94	0,13	3,51
2b)	1	4,16	19,42	30,49	15,31
	2	3,79	25,19	31,68	6,78
	3	2,79	24,57	0,71	41,61
	4	1,17	5,75	3,82	5,86
	5	1,00	3,65	12,58	0,58
	6	0,78	3,88	0,89	9,10
	7	0,56	2,58	1,06	2,70
	8	0,46	1,64	4,58	0,00
	9	0,37	1,36	1,03	3,91
	10	0,33	1,32	0,45	1,81

TRANSVERSAL ACTION	MODEL			
	displacements at the top (cm)			
	impeded movements		free movements	
	1a)	1b)	2a)	2b)
earthquake	7,0	6,0	6,2	6,0
wind	7,5	7,0	7,3	7,3

LONGITUDINAL ACTION	MODEL			
	displacements at the top (cm)			
	impeded movements		free movements	
	1a)	1b)	2a)	2b)
earthquake	9,7	10,0	10,0	10,0
wind	13,2	14,1	14,0	14,0

FLOOR	LONGITUDINAL ACTION	MODEL			
		drift of floor (mm)			
		impeded movements		free movements	
		1a)	1b)	2a)	2b)
-2	earthquake	-	1,2	1,3	1,3
	wind	-	1,3	1,4	1,4
0	earthquake	1,7	2,4	2,7	3,0
	wind	0,4	1,6	1,0	1,0
6°	earthquake	2,5	3,0	3,0	3,0
	wind	2,5	3,0	3,0	3,0
14°	earthquake	3,2	3,0	3,2	4,0
	wind	4,2	4,0	4,3	5,0
18°	earthquake	2,9	3,0	3,0	3,0
	wind	4,9	5,0	4,0	4,0
25°	earthquake	2,5	2,0	2,5	3,0
	wind	6,0	6,0	6,0	6,0
29°	earthquake	2,0	2,0	2,0	2,0
	wind	6,0	6,0	6,0	6,0

The analysis of results allow us to make some conclusions:

- the main vibration modes , regardless of the modelling, have a flexural behavior associated with a rotational component (the modes are "flexural-torsional" modes);

- the maximum displacement (η top) is 14.1 cm; this value is found under the action of the wind in the longitudinal direction (+ x) and, considering the total height (H) of the Tower, about 103m, this corresponds to η_{top} = H / 730 (the value is lower than the displacement limit, $\eta_{top, max}$, often referred to in literature for high buildings, estimated at $\eta_{top, max} \cong$ H / 500);

- the maximum drift of the floor ($\Delta\eta$) is 6.0 mm; this value is found under the action of the wind in the longitudinal direction (+ x), with values virtually identical for all modelings, at the upper floors (from the 25th floor where there is the splitting of the wall in pillars); considering the inter-floor extent (h) with a value of 3.28 m, this drift $\Delta\eta$ = 6.0 mm corresponds to $\Delta\eta$ = h / 546 (a lower value than the drift limit, $\Delta\eta$lim, often recommended, both in literature and in practice h/500 $\leq \Delta\eta$lim $\leq$ h/400).

In the FEM analysis the structural elements verified were distinctly noted by the direction T or L. In Figure 11 the ground floor and 25 th plans for the transversal direction are shown.

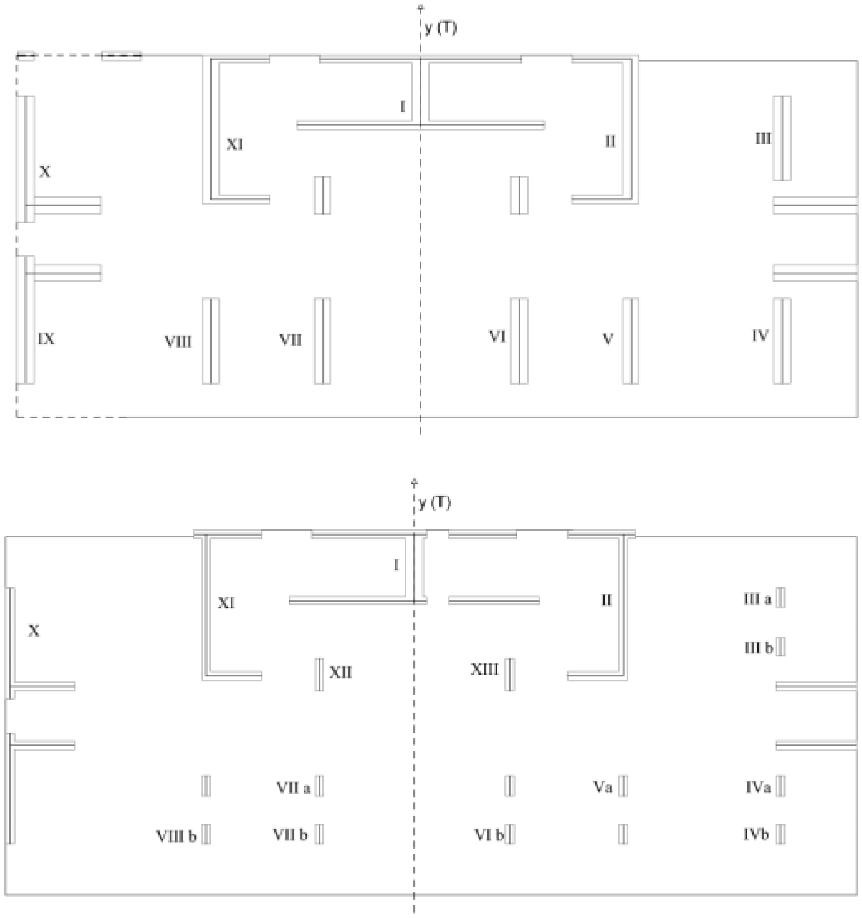

Figure 11. Plans of ground floor and 25 th floor.

4.1 Resistance Analysis

As required by NTC, shear resistance “V_{Rd}” of structural elements with specific shear reinforcement must be assessed on the basis of an adequate grid schematization.

The calculation of the efficiency in terms of resistance of a wall in the transverse direction of the Ground Floor, model 1b, is reported as an example. The resistant elements are: the transverse reinforcement, the longitudinal reinforcement, the concrete compressed stringer and the inclined core struts. The inclination “q” of the concrete stringer in relation to the axis of the beam must meet the following limits:

$$1 \leq \mathrm{ctg}\,\theta \leq 2{,}5$$

The verification of resistance (SLU) is:

$$V_{Rd} \geq V_{Ed}$$

where V_{Ed} is the design value of the agent shear stress.

For the transverse armature, the "shear-torsion" resistance is calculated with:

$$V_{Rsd} = 0{,}9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_{yd} \cdot (ctg\alpha + ctg\theta) \cdot \sin\alpha$$

For the concrete core, the "shear-compression" resistance is calculated with:

$$V_{Rcd} = 0{,}9 \cdot d \cdot b_w \cdot \alpha_c \cdot f'_{cd} \cdot (ctg\alpha + ctg\theta)/(1 + ctg^2\theta)$$

The shear resistance of the beam is the smallest of the two defined above:

$$V_{Rd} = \min\,(V_{Rsd}, V_{Rcd})$$

Moreover, as reported in paragraph §7.4.4.5.1 of NTC, for structures in Low ductility CD "B" and in High ductility CD "A", the possible increase of the shear forces due to the formation of the plastic hinge at the base of the wall must be taken into account. For the structure in low ductility this requirement is satisfied if the shear resulting from the analysis is increased by 50%.

SECTION AND MATERIAL				
B [cm]	37,00	Axial lood [kN]		12964
H[cm]	900,00	reinforcement long diameter [mm]		28
Concrete cover [cm]	4,00	f_{ck} [N/mm^2]		25,00
f_{yk} [N/mm^2]	310,00	f_{cd}[N/mm^2]		14,17
TRANSVERSE REINFORCEMENT				
bracket diameter [mm]	8,00	pitch [cm]		30,00
Area bracket [mm^2]		101	n.°	2
CHECK				
θ per V_{Rsd} =V_{Rsd}	cotgθ	α_c	V_{Rsd} [kN]	V_{Rcd} [kN]
5,9386	1,0000	1,25	726,65	13176,57
V_{Rd} [kN]				**726,65**
V_{Rd} [kN] (increment of 15% boost by the contribution of the perpendicular walls)				**835,65**
	Efficiency	0,75	NOT VERIFIED	

The efficiency ρ (T) of the wall concerned is equal to:

$$\rho(T) = \frac{V_{Rd}}{V'_{Ed}} \cdot 100 = 75\%$$

After the examination, the following considerations can be made:

Improving of longitudinal efficiency in height in relation to the transverse direction, except for some situations (for example, as regards elements 6 and 7 on the ground floor and in the foundations);

In the case of the walls with efficiency <75%, some seismic improvement interventions (with traditional or innovative techniques) are needed in order to increase efficiencies.

4.2 Ductility Analysis

A ductility analysis is carried out starting from the definition of the global ductility factor q = 1.96 and tacking into account the case 1b) because considered the most representative; the global structural ductility is achieved only if the local elements (walls and columns), under the lateral actions, have a sufficient local ductility in terms of the plastic rotation requested. To evaluate the structural efficiency in terms of local ductility, the ratio "capacity / demand" is calculated for each walls and columns starting from their capacity curves M-χ (where M represents the bending moment and χ is the curvature).

The curves are obtained by assigning the Kent-Park constitutive laws to the concrete and the Park Strain Hardening constituive laws to the reinforced steel bars and also by applying the horizontal forces (earthquake and wind) to the generic reinforced concrete wall (or column) axially loaded by a constant force (N).In the M-χ curves three main points are detected: the yield (initial), the maximum and the ultimate. The ratio between the ultimate curvature (χ_U) and the yield curvature (χ_{PL}) represents the ductility capacity ($\mu_{\varphi C}$) of the generic considered element ($\mu_{\varphi C} = \chi_U / \chi_{PL}$). The value of the ductility capacity ($\mu_{\varphi C}$) was compared to the ductility demand ($\mu_{\varphi D}$) which depends on the ductility factor q.

In the Eurocode (UNI EN 1998-1:2005 par. 5.2.3.4) the capacity in terms of curvature (μ_φ) depends on the structural capacity in terms of the deformations (μd) by the relation: $\mu_\varphi = 2\ \mu_\delta - 1$ (where $\mu\delta = q$ if the period of the main vibrational mode (T_1) is higher than the period corresponding to the part (T_c) of the seismic spectrum with constant velocity, like it is in Galfa case). An example of capacity M-χ is shown in Figure12, where the main points yield (noted Y0), maximum (noted M) and ultimate (noted U) are pointed out.

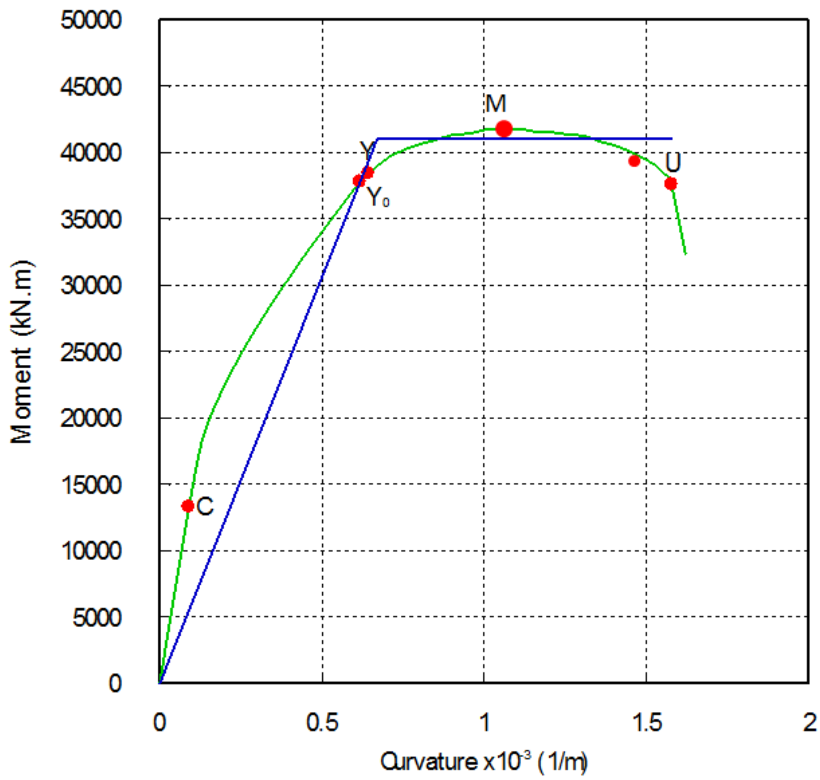

Figure 12. Example of M-χ curve (wall n.° X, under the seismic action in transversal direction, see Figure 11).

5 CONCLUSIONS

The several tests on the materials have provided an adequate knowledge about the mechanical performances of the concrete and the steel reinforcements. The characteristics of the materials are used in n.°4 "as built" finite element models representing the cases mentioned in the paragraph 3. By these different cases the structural response under the seismic and wind action are investigated in relation to the influence of the CB and the interaction structure-soil. For each cases eigenvalue analysis are carried out showing the main vibration modes have a bending-rotational deforms .

Under the horizontal loads, the maximum valued displacement is η_{top} = 14.1 cm, due to the wind in the longitudinal direction; in relation to the total height of the tower (H =103 m), it corresponds to η_{top} = H/730 , an acceptable value because lower than the limit top displacement ηtop,= H/500, in many cases considered acceptable for the tall buildings.

Moreover, the maximum valued inter-storey drift is $\Delta\eta$ = 6.0 mm, due to the wind in the longitudinal direction; in relation to the inter-storey height h_i = 3.28 m, it corresponds to $\Delta\eta = h_i/546$, an acceptable value because included in the limit range of the inter-storey drift $hi/500 \leq \Delta\eta_{lim} \leq h_i/400$ generally considered acceptable. From the different analysed case, the case 1b) was considered the most representative even if the effects on the structure due to the earthquake and wind are very similar in all the considered cases (i.e., for the displacements see the previous Table 4).

The resistance verifies done in terms of shear (V) and axial-bending (Pf) show an enough structural efficiency of many walls. In fact, considering acceptable (for example) in terms of V the "capacity / demand" ratio ρ_R = Vs /Vr = 0.75 (where Vs is the shear due to the earthquake or wind and Vr is the resistance shear due to the stirrups and walls thickness) the following considerations could be underlined:

for transversal (T) direction, all the walls located from the ground floor to the 18th floor show $\rho_R < 0.75$, even if from the 14th floor some for a lot of walls ρR is very close to the predicted limit $\rho_R = 0.75$ but around the 25th floor, where the walls are split in columns, ρ_R largely backs $\rho_R < 0.75$;

for longitudinal (L) direction, the walls located at the ground floor largely shows $\rho_R < 0.75$ whereas the walls from the 6th to the 18th floor has practically $\rho_R = 0.75$ (even if lower than the limit value 0.75); like in transversal case, where the walls are split in columns, ρ_R largely backs $\rho_R < 0.75$.

The ductility verifies are largely satisfied, in fact:

for transversal (T) direction, there are only three cases where the ductility ratio ρμ= μφC / μφD results ρμ < 0.75 and only one element shows ρμ < 0.50 (element n.° X at the 18th floor under the transversal wind),

for longitudinal (L) direction, there are 26 cases where the ductility ratio ρμ= μφC / μφD results ρμ < 0.75, however these cases has a ρμ value very close to 0.75 ; also in L, only a single element shows ρμ < 0.50 (element n.°7 at the at the 18th floor under the longitudinal wind).

That means the structural improvements have to mainly interest the resistance of the elements rather the ductility. All the hypothesis to improve the structural behaviour under the lateral loads have to be proposed by considering cost-benefit analysis and the invasiveness in relation to the architectural design carried out to change the intended use of the Tower.

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