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STRUCTURAL ANALYSIS FOR AN HISTORICAL R.C. TALL BUILDING RESTORATION

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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 FEM models of the tower, 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.

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.
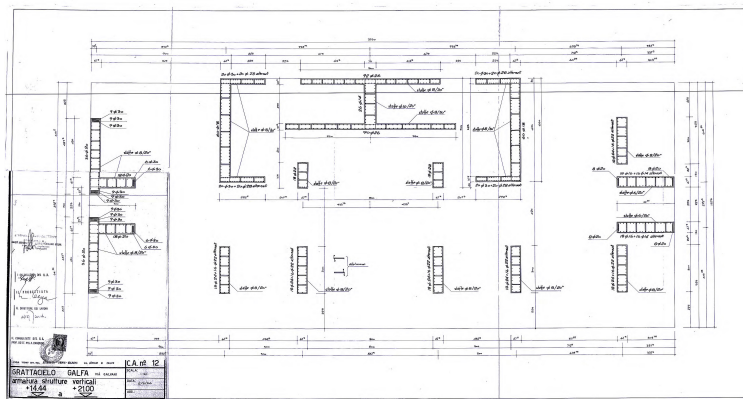

Figure 2: Example of an original design drawing.

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 f_c), 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 c_1 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.

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

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

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

FLOOR	TEST	D (mm)	H (mm)	H/D	Rc (Mpa)	c₁	c₂	Rc* (MPa)	Rc m* (MPa)
				H/D=1 – TOWER					
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, taking into account also 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 v_m*(m/S)	AVERAGE SPEED (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 ($I_{r,m}$) 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. 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) $R*_c = 7.695 \cdot 10^{-10} \cdot I_{r,m}^{1.450} \cdot v_m^{2.58}$, with $R*_c$ [daN/cm^2] and v_m [m/s]

- (GASPARIK 1992) $R*_c = 0.0286 \cdot I_{r,m}^{1.246} \cdot v_m^{1.85}$, with $R*_c$ [N/mm^2] and v_m [Km/s]

- (DI LEO, PASCALE 1994) $R*_c = 1.2 \cdot 10^{-9} \cdot I_{r,m}^{1.058} \cdot v_m^{2.446}$, 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 generaly 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.

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 tenisle strenght is $f_t \cong 530$ MPa.

3.   STRUCTURAL MODELING

Known the as built 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 = 180.000 kN/m^3.
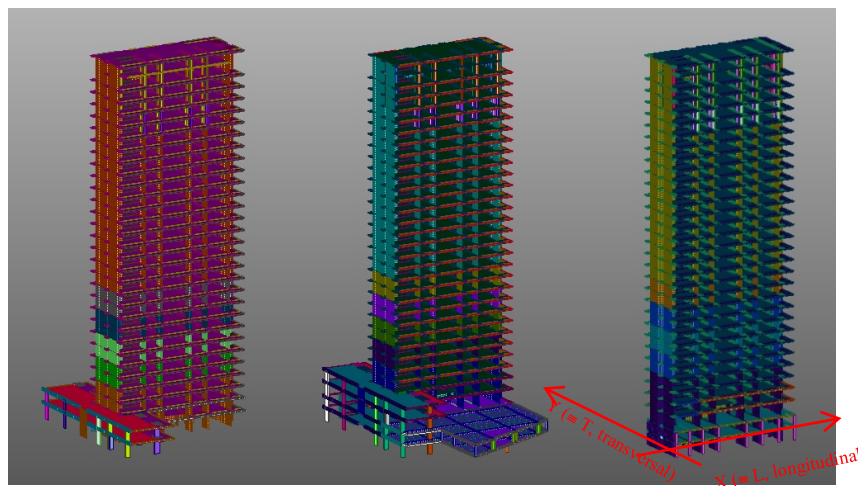

Figure 3: FEM of the Galfa Tower for all the cases

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 3: vibration modes, drift of floor and displacement into the top in direction (T) and (L)

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

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

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

In the FEM analysis the structural elements verified were distinctly noted by the direction T or L. In Figure 4 the ground floor and 25th plans for the transversal direction are shown.
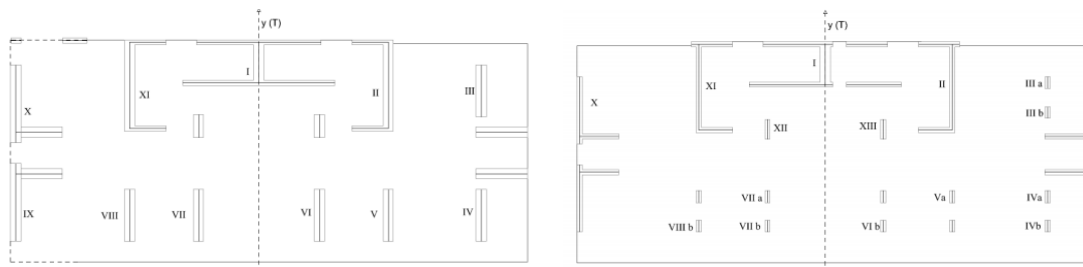

Figure 4: plans of ground floor and 25th floor with the numeration used for the transversal direction

4. 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 (μ_d) 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 (Tc) of the seismic spectrum with constant velocity, like it is in Galfa case). An example of capacity M-χ is shown in Figure 5, where the main points yield (noted Y_0), maximum (noted M) and ultimate (noted U) are pointed out.
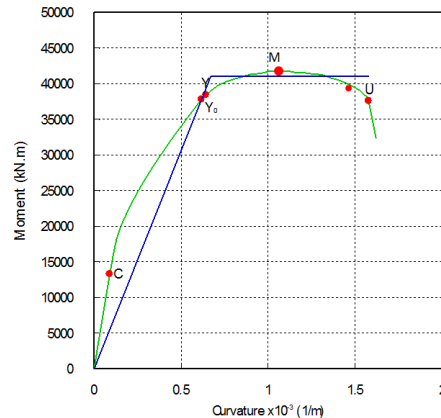

Figure 5: example of M-χ curve (wall n.° X, under the seismic action in transversal direction, see Figure 4)

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 $\eta_{top} = 14.1$ cm, due to the wind in the longitudinal direction; in relation to the total height of the tower (H =103 m), it corresponds to $\eta_{top} = H/730$, an acceptable value because lower than the limit top displacement $\eta_{top,1} \cong H/500$, 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 $h_i/500 \leq \Delta\eta_{lim} \leq h_i/400$ 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 3).

The resistance verifies done in terms of shear (V) and axial-bending (P_f) show an enough structural efficiency of many walls. In fact, considering acceptable (for example) in terms of V the "capacity / demand" ratio $\rho_R = V_s / V_r = 0.75$ (where V_s is the shear due to the earthquake or wind and V_r 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 ρ_R 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 $\rho_\mu = \mu_{\varphi C} / \mu_{\varphi D}$ results $\rho_\mu < 0.75$ and only one element shows $\rho_\mu < 0.50$ (element n.° X at the 18th floor under the transversal wind),
- for longitudinal (L) direction, there are 26 cases where the ductility ratio $\rho_\mu = \mu_{\varphi C} / \mu_{\varphi D}$ results $\rho_\mu < 0.75$, however these cases has a ρ_μ value very close to 0.75 ; also in L, only a single element shows $\rho_\mu < 0.50$ (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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