Seismic Improvement of an Existing Building with Relevant Vertical Slope due to Foundations

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SUMMARY:
Due to L'Aquila Earthquake, an existing RC building, at about 70 kms from the epicentral area, suffered relevant non-structural damage; the building was designed in the 80's according to an old seismic code, so the damage is an unexpected and an unacceptable event considering the low seismic effect given the distance from the earthquake site. It appears a very interesting case study to clarify damage reasons and define a retrofitting procedure. According to the new Italian Code the building was investigated in terms of material characteristics and structural detailing while a comprehensive study on site and foundation characteristics is in progress.
The building was realized with poor reinforcement details (without capacity-design criteria, incompliance with the old seismic code) and with low strength concrete (lower than design provisions due to incorrect casting in situ). It is placed on sloping and sliding ground and relevant differential vertical displacements have been detected too. The L'Aquila Earthquake effect evaluation on the study site, the in-situ and laboratory test results analysis and a strategy for retrofitting according to new Seismic Code are the topics of this paper.

Keywords: in-situ non destructive tests, RC existing building, sliding and sloping ground, structural retrofitting

1. THE CASE STUDY: A RC BUILDING DESIGNED WITH OLD SEISMIC CODE

The study building is a RC structure, designed in the '80s and built in the '90s, according to the old seismic Italian Code. It is located in Offida. Offida was classified as a seismic area until 1962 and the study site is located at 73 kms from L'Aquila Earthquake MainShock Epicenter (01:32:39 UTC 06/04/09 ML = 5.80). The building site is very close to a housing area formed by similar RC buildings, built in the 80's and 90's. Some of these buildings were heavily damaged by a relevant landslides in the '90s, therefore two of these were evacuated in the 2000s and finally they were demolished in 2010. This landslide is well known and reported in the Marche Region Geological Risk Map. However, irrespectively, an urban plan for social housing was approved in 1983 and study building was built.

The study building was designed in 1984, construction started in 1985, it was stopped twice for structural problems (i.e. for unexpected vertical settlements) and it was completed in 1994. After the L'Aquila MainShock, the study building suffered extended cracking in both external and internal masonry infilling panels. Due to a significant horizontal deformability of the structure, the inhabitants suffered from panic and they left the building immediately after the shock. All of these arguments are summarized in Biondi, S., (2011).

In the area, in 1984, there was an urban plan for three RC buildings (twin buildings "A_1-A_2", where A_1 is the study building, and two single buildings "B"-"C") and according to this provision two different geological surveys were carried out with five different drilling positions and, more specifically, one in situ shear wave velocity measurement station (S_s), in order to obtain a geological section for design, Figure 1. In the geological section 3 different layers can be detected: an upper layer of soft clay with high water content (type "a"), a median layer of medium clay (type "b") with a thin layer of sensitive clay (sliding layer, black area in Figure 1), a lower layer of stiff clay (type "c" assumed as bed rock).

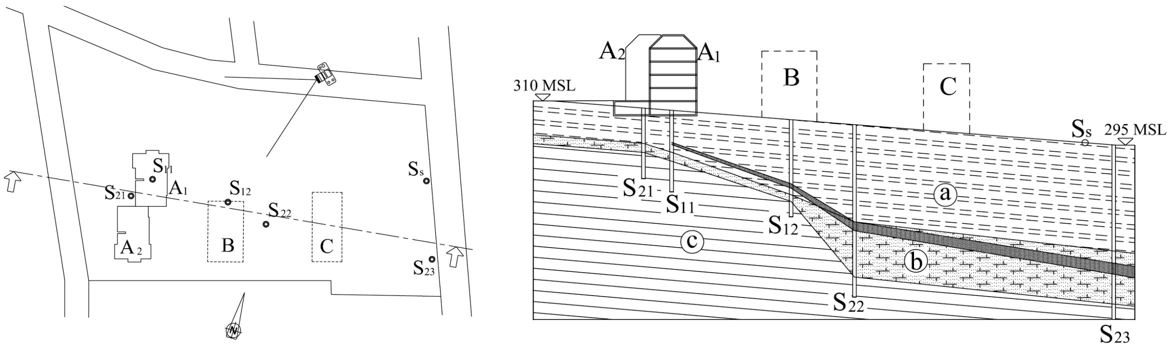

Figure 1. Geological survey's results at design time (1984): housing area and drilling positions (first series S_{1i}, second series S_{2i}), geological section (W-E) with stratigraphical profile and natural ground slope (≈ 12.5%)

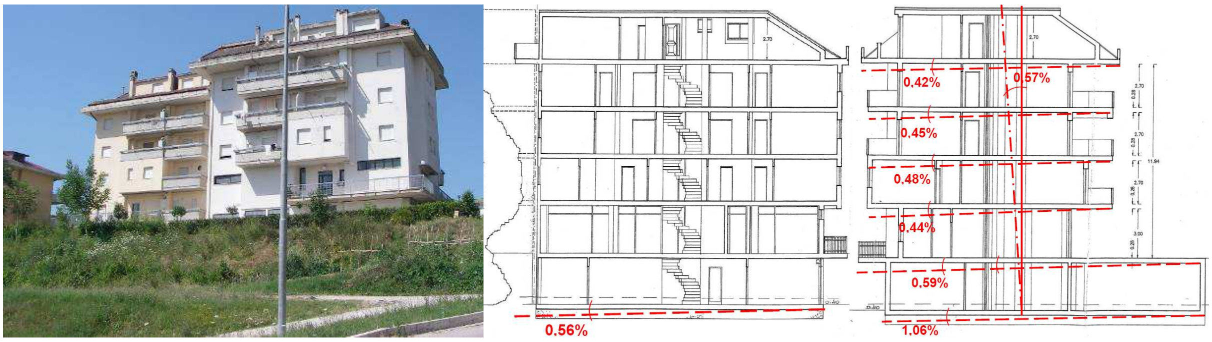

Figure 2. View of the study building (right side building) with the twin one (left side); longitudinal (S-N) and transverse (W-E, like geological one) sections of building with vertical sloping

Table 1. Average values of soil characteristics

Soil	Layer thickness - h_i [m]					γ	ϕ	N_{SPT}	C_u	S_S	V_S
	S_{11}	S_{12}	S_{21}	S_{22}	S_{23}	[kN/m^3]	[°]	[-]	[kPa]	h_i [m]	[m/s]
"a"	9.50	17.00	8.00	22.00	24.00	18.75	17	15	92	4.00	170
"b"	2.00	1.50	2.00	12.50	12.00	19.60	18	18	133	5.00	583
"c"	> 7.00	> 3.50	> 6.00	> 4.50	> 3.50	20.40	26	-	170	21.00	1087

In Table 1 both layer thicknesses (for 2 S_{1i} and 3 S_{2i} drilling positions) and average values of specific weight, γ, angle of internal friction, ϕ, SPT resistance, N_{SPT}, and undrained shear strength, C_u, are shown. Average values of shear wave velocity, V_S, are shown too, together with layer thickness for the wave velocity measurement station (S_S). Due to poor soil quality, only the first twin building was built (building "A_1-A_2") while the others weren't built ("B", "C"). The study building ["A_1" - photo right side in Figure 2] was started in 1984 while the twin building "A_2" was built after 1994. Due to its position the "A_2" building has lower values of thickness of both type "a" and "b" soils and its foundation doesn't trespass the thin layer of sensitive clay (sliding layer), i.e. the two buildings have different soil stability conditions. The "A_1" study building is a six floor RC building: base level partially underground, four levels above ground level but under the roof, 6th level at roof level.

The building suffered a relevant phenomenon of vertical settlements at the time and, above all, differential settlements both in longitudinal and in transverse directions. In Figure 2 the longitudinal and transversal slopes of the building are shown. It is possible to note that the longitudinal slope is constant at each floor (almost equal to 0.56%) while the transversal slope varies at each floor level. In particular while the average transversal slope is equal to 0.57% (quite similar to the longitudinal one), the base floor gradient is equal to 1.06%, the first floor gradient is 0.59% and, finally, the gradients of the other floors are in the following range: 0.42%-0.48%. This situation is clearly related to the construction history. In fact after the construction of both base floor (in 1985) and first floor (in 1988) the construction works were stopped on both occasions, due to these phenomena, for three years every time. Finally the other four floors were built all together in the 1991-1993 period.

Assuming that at each in situ casting the extrados of each floor was perfectly horizontal, it is possible to conclude that the base floor had a transversal gradient equal to 0.47% in the space of three years [0.157 % per year], the first floor had a transversal gradient equal to 0.15% in three years [0.05 % per year], the other floors had a transversal gradient equal to 0.45% in seventeen years [0.026 % per year].

For all these reasons the proprietor of the building decided to take legal action against the building firm; in particular they brought to attention three issues: 1. the compliance of the aftershock building behavior as concerns design provisions, 2. the real material characteristics at both building time (i.e. standard 28 days age after building) and present time, 3. the real possibility of a seismic retrofitting of the building according to actual seismic code. Subsequently a seismic assessment evaluation was carried out after L'Aquila Earthquake while the retrofitting activity hasn't started yet, because of the necessity to control the ground slope evolution. Therefore, a complex activity (including pulse echo method tests on RC pile, slope piezometers laying and controlling for almost three years, high precision topographical monitoring of building facades verticality) will be started in 2012.

2. THE COMPLIANCE OF THE AFTERSHOCK BUILDING BEHAVIOR AS CONCERNS DESIGN PROVISIONS

The original design was carried out using a static linear elastic approach considering bare (in-plane) frames. Again considering original design reports almost three uncorrected hypotheses can be pointed out: **1.** the building was considered 5 storey instead of a 6 storey building; **2.** the floor global seismic weight was underestimated; **3.** the base storey clear height was underestimated too. All of these mistakes had an univocal result: the base shear was underestimated, i.e. the global horizontal force on foundation pile top was underestimated too. For this reason maximum peak ground acceleration in the study site due to L'Aquila Mainshock, $a_g/g\,|_{AQ\max}$, is assumed as control parameter.

In order to determine the peak ground acceleration in Offida due to L'Aquila Earthquake sequence, data of ITACA – the ITalian Accelerometric Archive are used (see http://itaca.mi.ingv.it/ItacaNet/). As discussed in Biondi, S., (2011), 14 earthquakes were selected starting from L'Aquila Earthquake Mainshock. For each recorded earthquake epicentral location (Latitude N, Longitude E), local magnitude (M_L), hypocentral depth and epicentral distance from study site in Offida (km) are considered. An average distance of 65.50 kms from the study site is obtained. The maximum recorded, $\alpha|_{AQi} = a_g/g\,|_{AQi}$, near fault peak ground acceleration is used to estimate the Offida peak ground acceleration, $\alpha|_i = a_g/g\,|_i$, for the i^{th} earthquake. In this aim an original attenuation relationship, [Biondi, S., Fabietti, V., Sigismondo, S. and Vanzi, I. (2012)], in terms of epicentre distance x (km) is used. The result for L'Aquila Earthquake MainShock is thus obtained: $\alpha|_1 = a_g/g\,|_1 = 0.0297$.

With the aim of controlling this result, in the range of Offida distance from epicentre, two different literature attenuation relationships [Sabetta, F. and Pugliese, A. (1987), S-P] and [Zonno, G. and Montaldo, V. (2002), Z-M] are considered too, based on Italian earthquake data for local magnitude and epicentral distance similar to that of the present paper. With these two relationships maximum values in Offida $\alpha|_{1(S\text{-}P)} = 0.0327$ and $\alpha|_{1(Z\text{-}M)} = 0.0187$ are obtained respectively for L'Aquila MainShock. According to this result it is possible to assume that, due L'Aquila Mainshock, the study site suffered a peak ground acceleration ($a_g/g \approx 0.03$) that is about 43% of the design pga provision for the old II category seismic zone ($a_g/g = 0.070$, this value was used in 1984 for RC seismic design). With the same design pga, $a_g/g = 0.070$, the companion building A_2 was designed. This building, in quite same ground condition of A_1 building, Figure 1, didn't suffer any phenomenon of vertical settlements and of differential settlements nor any post-earthquake damage.

So it is possible to conclude that there isn't any compliance between the aftershock building behavior with the design provisions: it is evident that the study building shows unexpected seismic, and static too, behavior. Or the structural materials are incorrect, or foundation system is insufficient. Surely the original design did not maintain safety goal and building has to be retrofitted or evacuated.

3. MATERIAL CHARACTERISTICS AT BOTH BUILDING TIME AND PRESENT TIME

As in any structural assessment of an existing building it is necessary to select the target Knowledge Level (KL) [Biondi, S. (2008), Federal Emergency Management Agency (2000)]. This factor is used to express the confidence with which the characteristics of the building components are known, when calculating component capacities. The value of the factor is established from the knowledge obtained based on original design documents and on destructive or nondestructive testing of representative components. According to both Eurocode 8 [Comité Européen de Normalisation (2004), Comité Européen de Normalisation (2005)] and Italian Seismic Code [Ministero Infrastrutture (2008)], 3 different Knowledge Levels (KL1, KL2, KL3) can be defined depending on 3 different states of knowledge (I., II., III.). In order to have both the possibility of using any type of structural analysis [Linear lateral force analysis, Multi-modal response spectrum analysis, Nonlinear static analysis, Non-linear time-history analysis,] and to use the lower confidence factor $CF = 1.00$ the KL3 Full Knowledge was selected as target KL for this paper. So if the target Knowledge Level (KL) is attained, the average value, f_m, can be assumed as design value, f_d, as shown in (3.1).

$$f_d = \frac{f_m}{CF} \tag{3.1}$$

This goal is theoretically possible because both original construction drawings and material test reports are available and according to this objective the in-site and laboratory tests quantity was defined for the study building [Biondi, S. (2011)]. According to the Italian Code the minimum requirements for geometry, details and materials data can be selected as:
<u>I. Geometry</u>: Original construction drawings (O.C.D.) plus in-situ visual survey
<u>II. Details</u>: Complete O.C.D. plus limited in-situ inspection (I.S.I.)
<u>III. Materials</u>: Original test report plus limited in-situ tests (I.S.T.) or comprehensive I.S.T.
With regard to materials, the original test report is available but unfortunately this original test report for concrete is quite surprising. The tests were carried out in 1994 (ten years after the first building phase) considering 6 cubic specimens only. The design cubic characteristic strength value (R_{ck} = 30 MPa) wasn't detected. In fact 5 specimens show higher average values ($R_{cm1\text{-}5}$ = 57.07 MPa - 1.90 times R_{ck}) while one specimen shows a low specific weight (γ_c = 23.11 kNm^{-3}) and an unacceptable compressive strength value (R_{c6} = 25.78 MPa). For this reason the original test report data for concrete and steel were completely disregarded and comprehensive in situ tests (I.S.T.) were carried out by Scam Structural Laboratory of Chieti-Pescara University, under the author's supervision. Both actual material characteristics have to be defined in order for building retrofitting procedure and real material characteristics at building time (i.e. standard 28 days after building) are fundamental information for the building proprietor to take legal action against the building firm.

On this basis, assuming that the 2008 Italian Seismic Code permits the substitution of some destructive tests with non-destructive tests, the global activity carried out is:
<u>I. Geometry</u>: in-situ full survey with almost 1,450 m^2 of survey real plans. In these plans an overlapping of original drawings and test positions was carried out;
<u>II. Details</u>: **15** geometrical and reinforcement inspections of columns; **15** beams; **6** tile-lintel and precast RC floors; **5** geometrical reinforcement inspections of beam-column joints; **38** pacometer surveys for reinforcement controlling in RC elements;
<u>III. Materials</u>: **6** drilled out cores (**17** laboratory ultrasonic and **11** destructive compressive tests on cores) and **24** in-situ SonReb tests for concrete; **14** extractions of rebars (**14** tensile and bend tests and **6** elastic modulus evaluations on steel specimens); **2** vertical load tests on tile-lintel horizontal floors.

Test results were not encouraging both for steel reinforcement and for concrete compressive strength on drilled cores. In Figure 3 steel reinforcement after tensile tests (left & centre) and bent test (right) are shown. This reinforcement was extracted from both a beam at first floor (D_s = 14 *mm*) and a column at partial underground floor (D_s = 16 *mm*): it is possible to note a low extension of restriction length in the fracture zone and a fragile fracture due to bend test on mandrel for D_s = 16 *mm* rebars. All these facts reveal low ductility in these bars, while D_s = 14 *mm* show a good ductility.

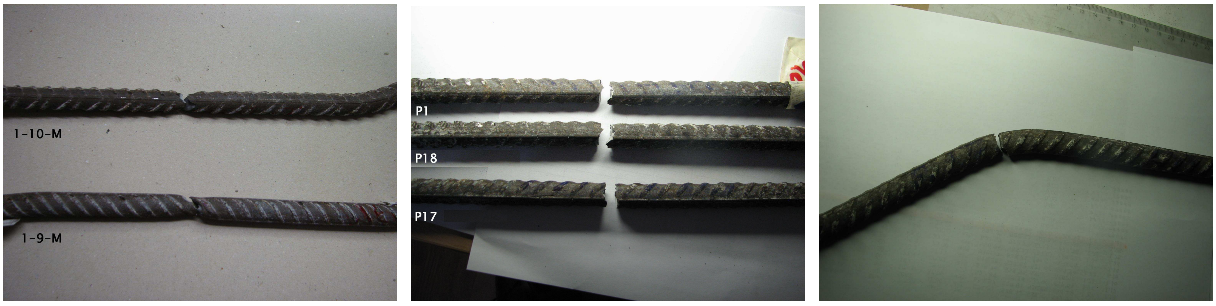

Figure 3. Steel reinforcement (D_s = 14 *mm* left) after tensile tests and bend test: low ductility is detected in D_s = 16 *mm* rebars (centre and right). Good ductility in D_s = 14 *mm* rebars

In Figure 4 concrete drilled cores after compressive tests are shown. In these cases great dispersion in compressive strength are detected (f_c = 11.40 MPa for S 8-16 M-a [beam at partially underground level]; f_c = 12.10 MPa for 4-8 M-b [column at 4th level]; f_c = 21.30 MPa for T 6 M-a [column at ground level]; f_c = 29.90 MPa for 2-3 B [column at 2nd level]). In Table 2 in-situ and laboratory test results (average values) for structural materials are shown: f_{ym}, f_{tm} yielding and tensile strength, A_{5m} ultimate strain in 5 diameters length in the fracture zone, $(f_t / f_y)_m$ steel over-strength; γ_{cm} specific weight, f_{cm} and R_{cm} cylindrical and cubic compressive strength, R_{cBm} real compressive strength taking into account in-situ drilling out. It is possible to note a lower value of actual cubic compressive strength R_{cBm} with respect to the design value R_{ck} = 30 MPa, nineteen years after the last concrete cast-in-place in 1993!

Figure 4. Concrete drilled cores after compressive tests: lower strength cores (S 8-16 M-a & 4-8 M-b [left]), medium strength core (T6 M-a [centre]) and highest strength core (2-3 B [right])

Table 2. In situ and laboratory test results for structural materials (steel and concrete) of the study building

laboratory test results							
f_{ym}	f_{tm}	A_{5m}	$(f_t / f_y)_m$	γ_{cm}	f_{cm}	R_{cm}	R_{cBm}
[MPa]	[MPa]	[%]	[-]	[kN/m^3]	[MPa]	[MPa]	[MPa]
529.01	682.10	23.74	1.30	21.78	18.61	20.72	23.39
in situ test results and actual values							
R_{cRm}	R_{cVm}	R_{cSm}	$R_{cSmCORE}$	R^*_{cm}	f^*_{cm}	E^*_c	
[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	
46.62	16.98	24.63	23.62	24.39	20.24	27183	

These results on concrete core are, unfortunately, confirmed by means of the non-destructive tests as shown in the second part of Table 2. In this case R_{cRm}, R_{cVm} and R_{cSm} are respectively the equivalent rebound, the pulse velocity and the SonReb strengths while $R_{cSmCORE}$ is SonReb strength calculated on the drilled concrete cores (note that $R_{cSmCORE} \approx R_{cBm}$). Using this value it is possible to define the actual values of cubic, R^*_{cm}, and cylindrical compressive strength, $f^*_{cm} = 0.83\, R^*_{cm}$, and elastic modulus, $E^*_c = 22000\,(0.10\, f^*_{cm})^{0.30}$, basing on in-situ SonReb compressive strength results as shown in (3.2).

$$R^*_{cm} = \left(\frac{R_{cBm}}{R_{cSmCORE}} \right) R_{cSm} \tag{3.2}$$

All these actual values are summarized in Table 2: both cubic compressive strength, R^*_{cm}, and elastic modulus, E^*_c, are lower than design values. As stated above the building proprietor requests to know the real material characteristics at building time (i.e. standard 28 days after building completion). For this scope aging and load level influences [according to International Federation for Structural Concrete FIB (2010)] are considered. According to this document the mean concrete compressive strength after t days, $f_{cm}(t)$, can be expressed in terms of mean compressive strength after 28 days, f_{cm28}, considering a coefficient, $\beta_{cc}(t)$, which depends on the age of concrete t and a coefficient s, (s = 0.25 for normal hardening cement) and a coefficient, $\beta_{cl}(t,t_o)$, which depends on the time under high sustained loads, t - t_o (days), where t_o is the age of the concrete at loading:

$$f_{cm}(t) = f_{cm28}\beta_{cc}(t)\beta_{cl}(t,t_o) \tag{3.3}$$

$$\beta_{cc}(t) = e^{s\left(1-\sqrt{\frac{28}{t}}\right)} \tag{3.4}$$

$$\beta_{cc}(t,t_o) = 0{,}96 - 0{,}12\sqrt[4]{\ln[72(t-t_o)]} \tag{3.5}$$

In this case the mean concrete compressive strength, $f_{cm}(t)$ is known at the time of laboratory tests on concrete cores while the compressive strength after 28 days, f_{cm28}, has to be defined. On the basis of other test experiences and considering the natural variation of live loads on a residential building as the study building, an average influence is taken into account for the high sustained loads coefficient $\beta_{cl}(t,t_o)$ in (3.3). In this manner the compressive strength after 28 days, f_{cm28}, can be defined as in (3.6). Using the value on cylindrical concrete cores, $f_{cm}(t)$ = 18.61 MPa as base value, the at-the-time cubic compressive strength R_{ck28} can be determined considering the statistical sample size and core slenderness. According to the Italian Code the characteristic cubic value can be determined as in (3.7) [in MPa]. This value is R_{ck28} = 16.88 MPa = 0.56 × 30 Mpa. So it is possible to conclude that the building firm built a structure that had a concrete strength loss of 44% with respect to design value. Inadequate original designs and material assumptions can be observed in Figure 5 where ratios between analysis moment to ultimate moment for different material assumptions are shown: it is possible to note that for 1984 Design assumptions there some columns that don't respect admissible moment also. This limit is dramatically overpassed if actual material and seismic code are considered.

$$f_{cm28} = \frac{2 f_{cm}(t)}{\beta_{cc}(t)[1+\beta_{cl}(t,t_o)]} \tag{3.6}$$

$$R_{ck28} = \min\{R_{cm28} - 3.50; R_{ci28\min} + 3.50\} \tag{3.7}$$

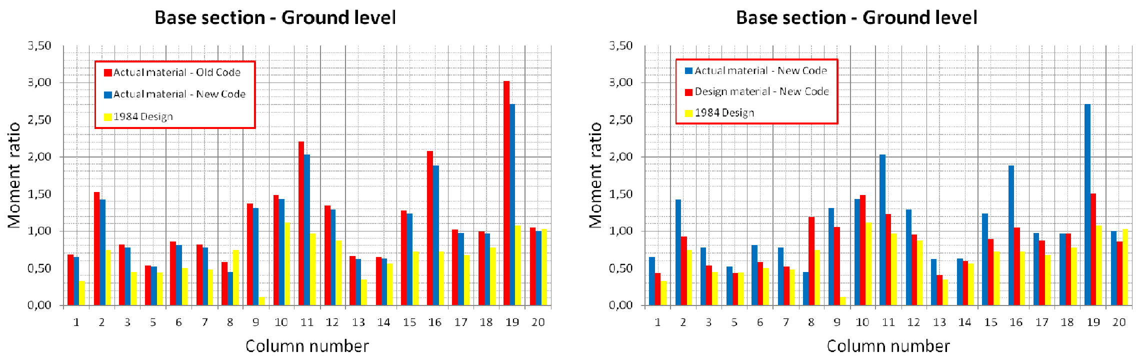

Figure 5. Ratio between analysis moment to ultimate one (for calculated axial load) for each column considering different material assumptions. 1984 Design refers to elastic ratio between analysis moment to admissible one

Finally results of vertical load tests on tile-lintel horizontal floors are used in order to confirm elastic modulus value. In particular two static load tests are carried out: the first at the second floor and the second at the fourth floor; water was used to load slowly the tile-lintel horizontal floors. In the first case a load area of 7.68 sqm is considered with a nominal maximum vertical load q_{vmax} = 3.22 kNm^{-2}, in the second a load area of 10.69 sqm with a nominal maximum vertical load q_{vmax} = 3.07 kNm^{-2}. Results in terms of vertical displacement [mm] versus vertical load [kNm^{-2}] are shown in Figure 6. It is possible to note quite elastic behaviour without any residual displacements at the end of each test. Maximum vertical displacement are δ_{vmax} = 0.14 mm^{-2} and δ_{vmax} = 0.12 mm^{-2} respectively for second and fourth floor (note that 0.01 mm is the precision of the mechanical displacement gauges used during the tests). In Figure 7 the linear Fem models for second (left) and fourth (right) tile-lintel horizontal floors are shown. Beams and slabs are modelled considering the actual elastic modulus E^*_c = 27183 MPa while the influence of the non structural concrete slab is considered too. With this model maximum vertical displacements δ_{vmax} = 0.135 mm and δ_{vmax} = 0.126 mm are obtained for second and fourth floor respectively. The actual elastic modulus provision is thus confirmed and this value can be used for retrofitting analysis.

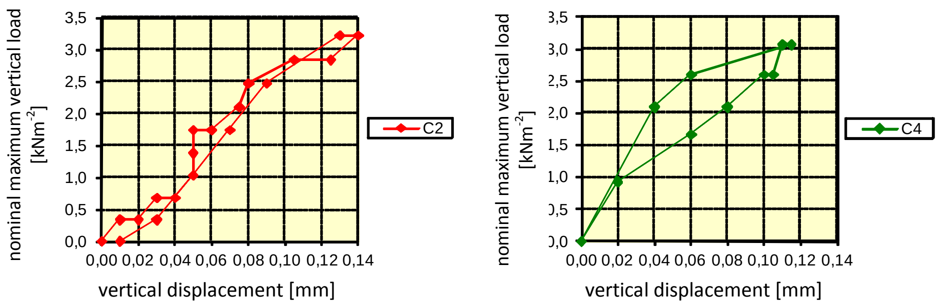

Figure 6. Vertical load-displacement results for second floor (left) and fourth floor (right) static load tests

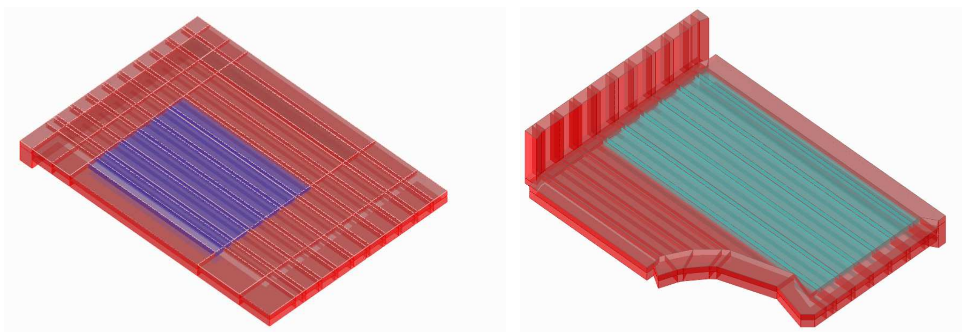

Figure 7. Linear Fem models for second (left) and fourth (right) tile-lintel horizontal floors

4. STRUCTURAL RETROFITTING APPROACH

It is clear that the study building has to be retrofitted due to design mistakes, poor material quality and local geological situation. In this paper a retrofitting procedure is summarized, considering that an operative plan can't be adopted until the test campaign on ground and foundation is not completed. After L'Aquila Earthquake, in July 2009 a new Structural Code [Ministero Infrastrutture (2008)] was adopted. Seismic input for assessment and retrofit has to be defined considering site location, nominal building life (V_N = 50 years), category use (category II use for private building with use coefficient C_U = 1.00), working building life ($V_R = C_U\ V_N$ = 50 years). Spectral parameters for different limit states (SLO Functionality Limit State & SLD Damage Limit State for Serviceability Limit States; SLV Life Safety Limit State & SLC Collapse Limit State for Ultimate Limit States) have to be determined according to a basic return period T_R = 50. Assuming ground type E, spectral parameters for each different limit state can be determined. In Figure 8 design spectra for study building at serviceability limit state (SLD) and at ultimate limit state (SLV) are shown assuming a viscous damping ratio ξ = 5% for SLD and a behaviour factor q = 2.25, as recommended by Code for existing buildings.

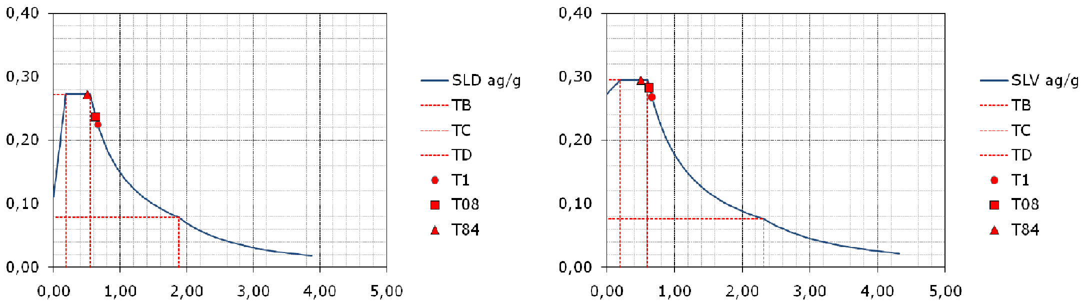

Figure 8. Design spectra for study building at SLD (elastic, left) and at SLV (q = 2.25, right)

In Figure 8 the vibration period of study structure is shown for different hypotheses (T_1 = 0.662 s is the average value calculated via a modal linear analysis for an infilled frame structure, T_{84} = 0.503 s was the design period, T_{08} = 0.603 s is the simplified actual Italian Code provision). It is possible to note that in both cases (SLD & SLV) these vibration periods are quite similar to the corner period at the upper limit of the constant acceleration, T_C. In particular if the SLD pga (a_g/g = 0.0698) is assumed, the SLD design action is $a_g(T_1)$ = 0.225 g that is 3.21 times the original design action $a_{gd}(T_{84})$ = 0.07 g. Also, considering the calculated peak ground acceleration due L'Aquila Mainshock a_g/g = 0.0297, the equivalent value for linear spectrum is $a^*_g(T_1)$ = 0.096 g. This equivalent acceleration is probably the local acceleration that the study structure suffered in April 2009.

The idea of the retrofitting procedure is that this value, $a^*_g(T_1)$ = 0.096 g, can be assumed as target value in the elastic range for the retrofitted structure. In other words the retrofitting design has to take up an exceeding spectral acceleration equal to $\Delta a^*_g = a_g(T_1) - a^*_g(T_1)$ = 0.129 g. Assuming that it isn't possible to provide a base isolation system due to lack of structural joint between "A_1" and "A_2" buildings, a first possibility to retrofit the structure is to stiffen the RC structure and to strengthen the structural elements (beam, columns, infilling masonry, slabs). As discussed in the previous paper there aren't problems in terms of spectral response but, for the sake of capacity design, a stiffening and strengthening in superstructure could cause the foundation failure. Given this, we have absolutely to avoid it considering the site geological situation. In fact if we assume that pile foundation has to behave in the elastic range during the earthquake, then the elastic design actions on foundation, $V_{ed,p}$ and $M_{ed,p}$, have to be equal (or lower) the ultimate design actions, $V_{Rd,c}$ and $M_{Rd,c}$, on superstructure elements (columns). Considering the detected reinforcement disposal and material characteristics and assuming a compressive stress in first level columns equal to 75% of design allowable compressive stress these values are obtained for shear: $V_{ed,p}$ = 204,3 kN, $V_{Rd,c}$ = min {234.7;604.5} kN, $V_{Rd,c} = V_{Rwd,c}$ = 234,7 kN. Similarly for bending moment; if we consider zero axial load flexural strengths $M_{ed,p}$ = 91.25 kNm and $M_{Rd,c}$ = 98.89 kNm are obtained. In both cases the ultimate limit values in the column are greater than the maximum elastic strength in pile.

For this, the only possibility to retrofit the study structure is to provide some dissipating systems in order to amplify the viscous damping and to reduce spectral acceleration for the serviceability limit state and to improve the structural behaviour for the ultimate limit state. A proposal of this kind of design procedure, based on the Capacity Spectrum Method, is discussed in previous papers [Biondi, S. (2011), Nuti, C., Biondi, S., Bergami, A.V. and Pierucci, D. (2010)]. The basic hypothesis is that the effective damping of a braced structure is expressed in terms of equivalent viscous damping $\nu_{eq,S+B}$ as a linear combination of the equivalent damping of the structure, $\nu_{eq,S}$, the equivalent damping of the bracing system $\nu_{eq,B}$ and the inherent structural damping ν_c (5% for a RC frame), (4.1).

$$\nu_{eq,S+B} = \nu_{eq,S} + \nu_{eq,B} + \nu_c \tag{4.1}$$

This approach starts with the definition of an acceptable limit state, corresponding to a displacement configuration of the structure and, therefore, to a single point in the Capacity Curve. This assumed displacement value is the target displacement δ^* of the design procedure; the designer has to define a dissipative system able to provide a combination of stiffness and dissipation in order to match the target displacement (i.e. the performance point) by means of the retrofitted structure. For the study structure the following hypotheses have been assumed for target displacements: SLD: the target displacement is L'Aquila mainshock equivalent displacement $\delta^{*}_{SLD} = \delta_e(\alpha^{*}_g(T_1))$; SLV: the target displacement is displacement corresponding to a strength decay of 20%.

$$\nu^{*}_{eq,B} = \nu^{*}_{eq,S+B} - \nu^{*}_{eq,S} - \nu_c \tag{4.2}$$

With these target displacements, dissipative brace characteristics have to be determined, using (4.2), to guarantee the required additional damping; in this formula the equivalent viscous damping $\nu_{eq,S+B}$ has to be calculated preliminarily and an iterative procedure has to be carried out. Therefore if the retrofitted structure response is able to guarantee selected equivalent damping $\nu^{*}_{eq,S+B}$ (i.e. if the bracing system is able to have a relevant yielding and energy dissipation for low displacements) the design pga can be determined. In this procedure a new vibration period $T_{S+B} < T_1$ has to be calculated due to stiffening effect of bracing system and consequently the expected spectral acceleration can be determined $a_{gSLD}\ (T_{S+B}) = 0.157\ g$. As shown in a previous paper [Biondi, S. (2011)] this value is greater than target acceleration $a_{gSLD}\ (T_{S+B}) = 1.64\ a^{*}_g(T_1)$ but it is significantly smaller than the actual SLD value $a_g(T_1) = 0.225\ g$: $a_{gSLD}\ (T_{S+B}) = 0.60\ a_g(T_1)$. In conclusion if a retrofitting strategy based on dissipative bracing is selected for the structure, the exceeding acceleration $\Delta a^{*}_g = 0.129\ g$ can be absorbed by a half by the bracing system, {(0.225 - 0.157) = 0.53 × (0.225 - 0.096)}.

Figure 9. Mass removing hypotheses for East (left) and North building fronts (right) of the study building

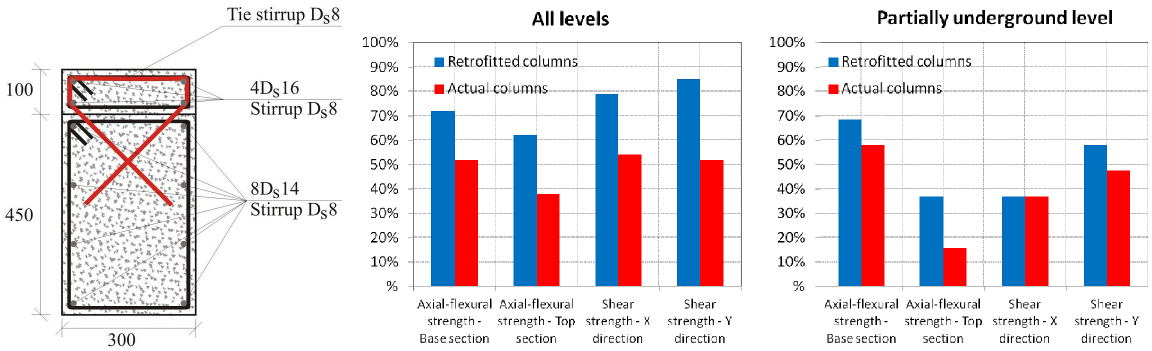

Figure 10. Inadequacy of r.c. structure in terms of safety ratio (i.e. percentage of column that verifies safety without dissipative elements). Red actual configuration, blue upper floors strengthening and mass reduction.

Therefore the dissipative approach has to be supported by a classical structural strengthening. In order to respect Code conditions for pile foundation, i.e. elastic foundation behavior in respect to superstructure plastic behaviour, the structural strengthening has to have a minimum impact on base columns. Furthermore, together with structural strengthening at upper floors, a seismic demand reducing can be carried out in order to complete the dissipative approach. In particular, seismic demand can be reduced by removing upper floors or other mass from the structure. Assuming that the first alternative isn't possible, mass removing can be usefully planned too. Heavy concrete parapets, external infilling masonry portions, cantilever element for overhanging masonry facades without external bracing can be removed, Figure 9. With these simple actions a reduction of the global base shear almost greater than 7% is obtained. If this mass reduction is combined with partial jacketing in upper floor columns, (as in Figure 10 left for a 300 × 450 column retrofitted up to a 300 × 550 mm^2 gross section), flexural and shear behaviour of columns is enhanced, above all at first partially underground level, Figure 10. Just another component of the retrofitting strategy.

5. CONCLUSIONS

An interesting case study of seismic retrofitting of an existing RC building with pile foundation is presented in this paper. The structure was built in a zone classified as seismic and in accordance with Seismic Code; in spite of this, it shows a very impressive series of structural deficiencies: poor material strength, significant landslide movements and foundation settlements, appreciable vertical slope gradient in two orthogonal directions, insufficient seismic joint and obviously poor reinforcement detailing. The study building suffered the effects of a recent earthquake (L'Aquila Earthquake 2009) with significant nonstructural damages. A comprehensive test campaign on structural details and material characteristics was conducted in order to take legal action against the building firm. Irrefutable data are pointed out on structural inadequacy. So, according to Italian Code, the study building has to be retrofitted or has to be evacuated definitively.

The principal difficulty for seismic retrofitting is related to the foundation system: RC piles are difficult to investigate and, above all, are practically impossible to retrofit in an homogeneous manner (and we have to take in mind how relevant is regularity in earthquake engineering). Again, above all considering local geological situation, the foundation system has to be stressed in the elastic range in order to guarantee structural safety. If both base isolation (due to lack of an adequate seismic joint with a twin building) and exclusive superstructure strengthening and stiffening (due to pile foundation fragility) have to be rejected, the only retrofitting strategy is to combine a dissipative approach with mass reduction and columns strengthening by means of partial jacketing in upper floors.

The paper discusses this procedure and shows encouraging results; providing the geological survey investigation confirms global stability of the ground slope, the building can be strengthened and an acceptable safety level can be assured to building owners.

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