Seismic Vulnerability Assessment of a strategic R/C building, gravity load designed

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Summary

The seismic vulnerability of a strategic frame structure site in Teramo (Abruzzo), typical of existing Reinforced Concrete buildings designed only to vertical loads, has been evaluated. They are representative of building types widely present in the Italian building stock of the last 50 years. A simulated design of the structures has been made with reference to the codes in force, the available handbooks and the current practice at the time of construction. The seismic response is calculated through non linear static analyses (pushover). The results show a high vulnerability for the building analyzed: in this case collapse can be considered likely also with not strong earthquakes.

Based on a purposely set up methodology, a simulated design of the structures has been made with reference to the codes in force, the available handbooks and the current practice at the time of construction.

Finally, the seismic behavior has been analyzed emphasizing the role of infills and, particularly, of the building age through a comparison with the results of previous studies on post-70s structures.

Keywords

assessment, existing buildings, frames, masonry infills, reinforced concrete, seismic vulnerability, simulated design

Theme

construction – analysis – earthquake – concrete

1. Introduction

Reinforced Concrete (R/C) buildings make up an increasing proportion of the building stock of many countries all over the world. Many of them were built before the advent of seismic codes or with the utilization of old and inadequate anti-seismic design criteria. During past earthquakes (for example L'Aquila 2009) R/C buildings often displayed unsatisfactory seismic behaviour, particularly when their design included only vertical loads and ductile detailing was not explicitly provided. Thus, the evaluation of seismic vulnerability of R/C building structures has a key role in the determination and reduction of earthquake impact.

Although the most dangerous consequences of earthquakes in the near future are likely to come from existing buildings, until now the attention of the community in its various components (researchers, professionals, policy makers, . . .) has largely focused on the seismic design of new buildings. There are a number of socio-economic reasons for this, including the high costs frequently requested by the retrofit of R/C buildings and the lack of community concern with safety in general, not only in the seismic field, as experience in other fields corroborates (see for example safety on job sites).

Along with socio-economic factors, there are also major technical problems. With the exception of some guidelines in Japan (JBDPA, 1977) and USA (FEMA, 1992), there is little guidance in the codes for the determination of the seismic resistance of R/C existing buildings. In Europe, the codes are just beginning to tackle the problem (see for instance Eurocode 8 - part 3 (CEN, 2002a)).

There is no doubt that the assessment of an existing R/C building is a much more difficult task than the design of a new one, because it requires work on structures of which only a limited knowledge can be obtained.

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There are difficulties in determining possible deterioration conditions as well as in obtaining sufficiently accurate knowledge of some structural data (e.g. amount and location of reinforcement) or completeness (e.g. materials strength), as appropriate technical documentation is rarely available. The specificity of the problem calls for the setting up of ad hoc methods to assess the seismic vulnerability of R/C buildings in a sufficiently reliable, as well as inexpensive way.

A crucial aspect of knowledge of the structural characteristics of a building is the period of its construction. When technical documentation is either not available or insufficient, most valuable data can be obtained through reference to the codes, the design methods and the typical current practice at the time of construction.

Using all the information obtainable from the sources above, and through an examination of the codes in force in the period, specifications on the prescribed values of loads and material strengths, the minimum values of the dimensions of structural elements and of reinforcement amounts can be drawn up. More difficulties occur for the evaluation of the values of internal forces actually used in the safety verifications, of the location of reinforcement and of the detailing solutions. For this reason, reference has to be made to the handbooks commonly adopted in the period and to the technical documentation of real buildings found in the archives of public administrations, building firms and professional offices. Finally, from the handbooks more accurate indications can be obtained regarding the design methods and the arrangement of reinforcement in the structural elements.

The present study aims at evaluating the seismic vulnerability of a strategic framed structural type representative of pre-1970 R/C existing buildings designed only to vertical loads. To obtain a realistic evaluation of their vulnerability, a great deal of work has been carried out to determine the most important structural characteristics of this building including a thorough examination of the technical documentation of the period.

2. Procedure

The evaluation of the seismic vulnerability of gravity-load designed R/C building has been carried out on the basis of a purposely set up procedure made up of five main steps (Masi, 2000; Masi et al., 2001a):

(1) the configuration of R/C buildings examined, typical of the period under examination, are studied;

(2) the structures, that are designed taking into account only vertical loads, are studied on the basis of the codes in force, of the available handbooks and of the current practice of the period;

(3) the seismic response is calculated through non linear static analyses;

(4) the seismic resistance is evaluated by means of fragility curves relevant to some structural and non-structural damage parameters (drift, ductility demands, etc.);

(5) the vulnerability class of each type, according to the European Macroseismic Scale 1998 (ESC, 1998), is defined taking into account the increasing damage degrees computed by applying increasing seismic intensities.

Operating procedure will be followed for the evaluation of seismic vulnerability:

STEP 1A: Data collection and preliminary investigations
STEP 1B: Use of structures and definition of design actions
STEP 1C: Investigations and tests on the structures
STEP 1D: Definition of the model and analysis method
STEP 2: Results of analysis and indicators of earthquake risk
STEP 3: Conclusions and proposals for action

3. STEP 1A: Data collection and preliminary investigations

The examination of the old seismic codes was made by considering aspects related to material properties, the

characteristics of structural elements important in assessing the capacity and resistance to actions. With regard to the material the R.D. 2229/39 prescribed cubic strength concrete with 28-day average of at least 120 kg/cm2 (160 for conglomerates to high resistance) and at least three times the safe load, up to a maximum of 180 kg/cm2. The allowable stress values, depending on the medium resistance, was 35, 45 and 60 kg/cm2 in the case of simple compression, and 40, 50 and 75 kg/cm2 in the case of bending or compression bending. The allowable value of shear stress tc0 was 4 kg/cm2 to 6 kg/cm2 for normal concrete and high strength concrete. Beyond these limits was no requirement for calculating a suitable shear reinforcement. For the steel bars were prescribed allowable stress of 1400 kg/cm2 to 2000 kg/cm2 for mild steel and steel semi-hard and hard, however, half of that yield. The possible use of more resistant steels was influenced by the strength class of concrete.
With regard to the information most authoritative of manuals, widely adopted by designers, were those of Santarella (1956) and Pagano (1963). The content of these texts was very extensive and detailed that, in order to highlight the most significant aspects, we set out below some brief outline in Table 1, which contains the information relating to the design and verification of pillars and beams, in addition to the regulatory requirements of the reporting period. It should be emphasized that the reference to the manuals have been decisive in some cases to fill gaps of RD 2229/39 in the performance of the simulated design.
With reference to the actions provided for the calculation of reinforced concrete structures, the first specific legislation is represented by DM 03.10.1978, issued under Law n. 64 of 2/2/1974 64. Actually a reference to the loads to be considered on reinforced concrete structures is already in R.D. 1939, which stipulated that the armed conglomerate taking a weight of 2500 kg/m3 and the live loads were to be set depending on the type, scale and purpose of use of the work to be carried out. An indication of actions to be taken was also given in Law n.1684 of 25/11/1962, which prescribed for the floors to house an accidental load of 200 kg/m2.
Then, The Italian Royal Decree 04/18/1909 n.193 which contained the technical standards for repairs, reconstruction and new construction of public and private buildings and the list of municipalities subject to compliance with the rules,

- was exclude the possibility of building on unsuitable sites (wetlands, landslide);
- was required compliance with detailed rules of construction (curbs, overhangs);
- Was Limited the height of buildings and the number of floors (depending on the technology);
- Was Required the adoption of horizontal and vertical static forces proportional to their weights;
- Was Defined the minimum width of streets and spaces between buildings.

Finally, The D.L. 1526 of 1916 has quantified the seismic forces and their distribution along the height of the building. The static actions, due to its own weight and to overload, was increased by 50% to simulate the effect of vibrating vibration (increase of vertical forces), and the dynamic effects due to seismic wave motion, was simulated by accelerations applied horizontally to the masses of the building in two directions (horizontal forces).
In conclusion:
At the time of construction, there were only codes of practice which set some criteria for seismic design, earthquake resistant, but the legislation was still not ripe because the structural design continued to rely primarily on linear static analysis, with design specifications that were held account the dynamic effects due to possible seismic shaking.
For the time period ranging from 1939 to 1973, therefore, the seismic design was based on technical rules of construction dictated by RD 2229 of 16/11/1939 [1]. The main aspects of this standard can be summarized, also for design parameters, in this way:

• The coefficient of homogeneity n, equal to the ratio between the elastic modulus of the steel and concrete, is considered to be equal to 10 if the concrete is Normal or 8 if the concrete is high strength.
• The elastic modulus of concrete must be determined experimentally in fact the value that is considered in the calculations is equal to 20000 MPa
• With regard to the minimum longitudinal reinforcement to be assigned to the columns shall be considered equal to 0.8% of the concrete if less than 2000 cm ² and is equal to 0.5% of the concrete, if it is greater than

8000 cm ². In all other cases we proceed by interpolation
• All calculations are performed using the method of the allowable stress

The calculation of the structure was carried out in a simplified manner. In particular, the structural scheme adopted was very simple and did not require the presence of a structural grid composed frames earthquake resistant in two directions.
The design of the columns was carried out for areas of influence. The beams was calculated using simplified schemes of continuous beams.
The outer transverse beams are calculated using the same scheme but with the loads of the cladding.

4. STEP 1B: Use of structures and definition of design actions

The original intended use was for residential and office work. Currently, the building has three floors underground not used for storage. From the ground floor upwards instead, the building is used as offices by the Local Health Office (ASL) of Teramo, with free public access.
As shown in Annex B1 of the DGR 438/2005, the building can be likened to building a strategic type A1, due to the presence of
- Local Health Office (at 5 th floor there is the leadership of the ASL Teramo)
- Health Presidio (1st floor and 2nd clinics are functioning regularly)

This involves the recruitment of Class IV of use.
To determine the extent and spatial distribution and temporal variables of overload, reference was made to the Italian Code according to the intended use.
The variable loads include the loads associated with the intended use of the work.
The overload variables currently envisaged for the structure are as follows:
1) scales: 400 kg / m
2) premises used for office: 400 kg / m
3) to the balconies and overhangs, the NTC 2008 require a variable overload of 400 kg / sq m, and it is likely that this same value was used in original calculation.

5. STEP 1C: Investigations and tests on the structures

As reported in Italian Circular 617/2009 (Instructions for the application of the Seismic Italian Code NTC 14/01/2008), for the identification of the structural geometry, the data collected include the following:
g) identification of the system resistant to horizontal forces in both directions;
h) texture of the floors;
i) geometric dimensions of beams, columns and walls;
j) width of the wings of the T-beams;
k) possible eccentricity between beams and columns to the nodes.
For the identification of structural details, the data collected must include the following:
l) amount of longitudinal reinforcement in beams, columns and walls;
m) number and details of transverse reinforcement in critical areas in the nodes and beam to column;
n) amount of longitudinal reinforcement in the floors that contributes to the negative moment of T beams;
o) lengths of support and constraint conditions of horizontal elements;
p) thickness of concrete cover;
q) length of the overlapping areas of the bars.
For the identification of materials, data collected include the following:
r) concrete strength;
s) yield strength, strength and ultimate strain of steel.

In first, was performed MASW seismic survey, which allowed us to reconstruct the sismostratigrafia subsoil from which to derive the velocity profile first and then the sub category; then, while referring to the works for possible details in the Annex, the wave velocity cutting of the various rock types can be summarized as follows:

- alluvial soils: VSH = 350 m / s
- altered substrate: VSH = 450 m / s
- substrate intact: VSH = 600 m / s

Applying this to the calculation methodology in the new NTC is a $VS_{,30}$ = 475 m / s and a corresponding category of subsoil B.

Then, the in situ and laboratory tests are aimed mainly at testing the properties of the materials used for the construction of structures.

As reported in the literature, in studies in the field and in the current regulations, an existing structure on the properties of materials in situ can be determined on the basis of the following sources of information:

- Usual values for the construction practice of the time;
- Details of the original project or original test certificates;
- Testing in situ (limited, extended, exhaustive).

The extent of in situ tests depends on the level of knowledge is sought and other information available.

In order to determine the compressive strength of concrete, the yield stress and tensile strength of steel, are required at least limited evidence.

Particularly important is the estimation of compressive strength of concrete, besides the role it has on the bearing capacity and durability of the structure, other properties of concrete such as modulus and tensile strength can be obtained indirectly or indirectly by it .

To allow the assessment to the building concerned, the structures have been used for such destructive methods of investigation (which involves the removal of local material) and non-destructive.

Among the first we must remember the drill, the second the covermeter.

For the purpose of vulnerability analysis, the choice of the type of analysis and values of the factors of trust, mentioned in C8.7.2.1 (Circular 617/2009 - Instructions for the application of the NTC in DM 14/01 / 2008), we distinguish the following three levels of knowledge:

- LC1: Knowledge Limited;
- LC2: adequate knowledge;
- LC3: accurate knowledge.

Aspects that define the levels of knowledge are:

- Geometry, ie the geometric characteristics of the structural elements,
- Structural details, namely the amount and arrangement of the reinforcement, including the step of the brackets and its closure, the AC, the connections for steel, the links between different structural elements, the consistency of non-cooperative structural,
- Materials, is the mechanical properties of materials.

The level of expertise determines the method of analysis and confidence factors to be applied to material properties.

The relationship between levels of knowledge, methods of analysis and confidence factors is illustrated in Table C8A.1.2 Circular 617/2009.

It was not possible to reach the levels of knowledge LC2 and LC3 to lack of necessary data (as shown in Table 2 of the Circular 617/2009).

According to the indications in the literature, the proof of the concrete core drilling belongs to the category of evidence destructive (PD) and is involved in the concrete samples to be tested.

The procedures for the extraction, processing of samples extracted to obtain the samples and the method of test compression are described in UNI EN 12504-1, UNI EN 12390-1 (Test on hardened concrete - Shape, size and other requirements for specimens and formwork), UNI EN 12390-2 (Test on hardened concrete -

Packaging and drying of specimens for strength tests ") and UNI EN 12390-3 ("Testing on hardened concrete - Compressive strength of test specimens").

Level of knowledge	*Geometry*	*Details Structural*	*Ownership of Materials*	*Methods Analysis*	*FC*
LC1	*From original drawings of structural work with visual survey or complete relief*	*Simulated project in accordance with the standards of the period and limited in-situ testing*	*Usual values for the construction practice of the time and limited in situ tests*	*Static or Dynamic linear analysis*	*1,35*
LC2		*Incomplete drawings with limited in situ testing or extended in situ testing*	*From the original specifications of the project or from the original test certificates with limited or from extended in situ tests*	*All*	*1,20*
LC3		*Incomplete drawings with limited in situ testing or extensive in situ testing*	*From the original specifications of the project or from the original test certificates with limited or from extensive in situ tests*	*All*	*1,00*

Table 1: Level of knowledge on the basis of available information and subsequent analysis methods and allowed values of the factors of confidence

Figure 1: carrots machine in action, and test of load on plates

The test of load on plates is intended to determine the carrying capacity of the foundations of the building in question and the deformation of the ground below the test plate to a depth of approximately the same width or diameter of the plate.
The test, in general, can be done in any kind of loosened soil, soft rock or rock itself (in the case of tests in loose soil, the maximum size of clasts below the plate must not be greater than 100 mm). In this case, the foundation soil consists of medium coarse gravel in sandy matrix.
Finally, was performed the Tensile test on steel bars.
The value of the average compressive strength tests on the results of 38.60 MPa; the cubic resistance took into account the following correction factors:
- slenderness H / d = 0.82 for resistance medium between 25-39 Mpa
- torment fTor = 1.15
- coring direction fdir = 1.075

Rcub = 38.60 x 0.82 x 1.15 x 1.075 = 39.12 MPa

The test of load on plates can then directly determine the deformation modulus or compressibility and, through the introduction of the Poisson's ratio, to evaluate the Young's modulus. Although the test is standardized in Italy, the Swiss standard SNV 670317 (1959) [5] is the most widespread. In this case it is obtained:

Md = 50 N/mm2 - Md / Md '= 0.150 N/mm2

The reaction module = 0.167 N/mm3

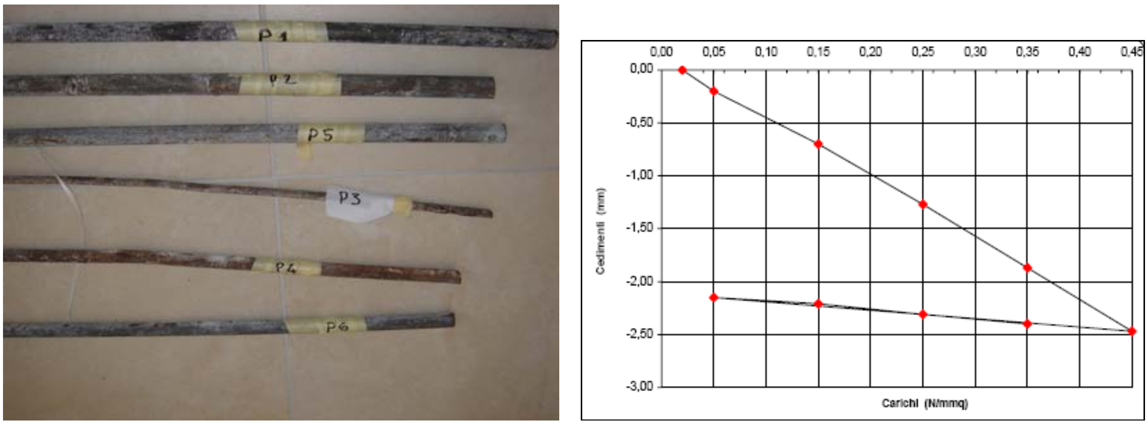

Figure 2: bars extracted and result of test of load on plates

Finally, from evidence traction on bars, performed on the samples, we have obtained the following values tensile strength:

Average value of yield strength = 358.62 [N / mm]

Average value of tensile strength = 510.33 [N / mm]

6. STEP 1D: Definition of the model and analysis method

The calculation is performed using the program automatically calculating CDS Win.

The spatial patterns that are then analyzed, taking into account the distribution of the masses and do not consider additional stiffness consist of non-structural elements.

The structure is outlined with frame resistant elements, in two main directions, connected by floors that serve as horizontal diaphragms.

The seismic vulnerability assessment is aimed at the definition of risk indicators. These indicators, called denominati α_u and α_e , are calculated by the relationship:

$$\alpha_u = \frac{PGA_{CO}}{S_T\ S\ \gamma_I\ PGA_{2\%}} \quad or \quad \alpha_u = \frac{PGA_{DS}}{S_T\ S\ \gamma_I\ PGA_{10\%}} \ , \tag{1.1}$$

depending on the state limit of reference

$$\alpha_e = \frac{PGA_{DL}}{S_T\ S\ \gamma_I\ PGA_{50\%}} \tag{1.2}$$

Where:

- PGA2% = horizontal acceleration on soil of category A with 2% probability of exceedance in 50 years;
- PGA10% = horizontal acceleration on soil of category A with 10% probability of exceedance in 50 years;
- PGA50% = horizontal acceleration on soil of category A with 50% probability of exceedance in 50 years
- PGA_{CO} = estimated PGA for collapse of the structure
- PGA_{DS} = estimated PGA for reaching the limit of severe damage
- PGA_{DL} = estimated PGA for reaching the limit of limited damage

For example, if

g = maximum horizontal acceleration on the ground, for soil category A = 0.25, we have:

PGA2%= ag = 0,375 - PGA10% = ag = 0,250 - PGA50% = ag / 2.5 = 0.100

Also:
S = stratigraphic amplification coefficient = 1,13
S_T = topographic amplification coefficient = 1,20
γ_I = coefficient of relevance = 1,40

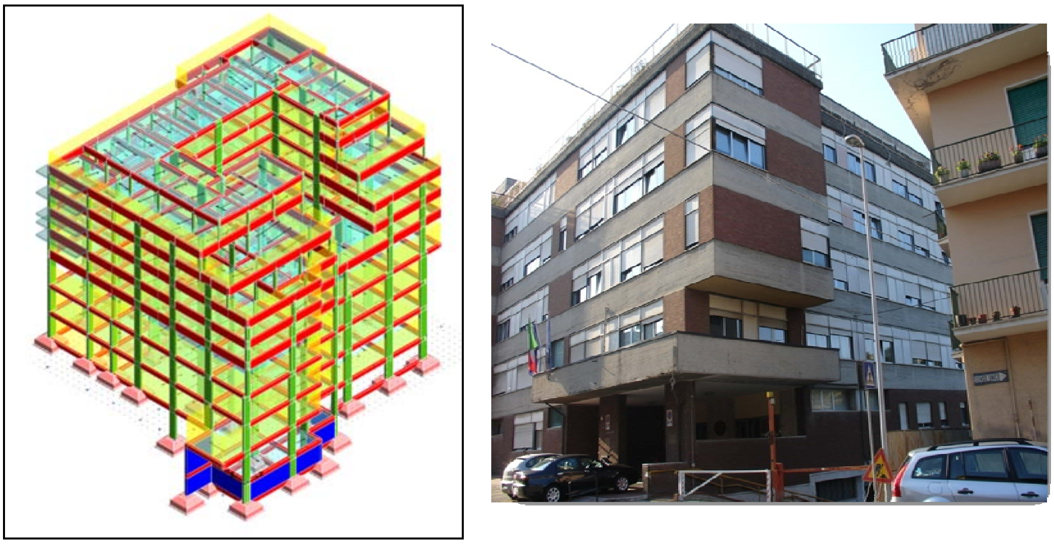

Figure 3: 3D Model and view of the building

The parameter α_u is considered an indicator of the risk of collapse, the parameter α_e is considered an indicator of the risk of unavailability. Values near or above unity characterize cases where the level of risk is close to that required by the rules; low values, close to zero, characterize high-risk cases.
The horizontal ground acceleration PGA10%, with expected exceedance probability of 10% in 50 years, in the absence of more accurate determinations, is to be taken as mentioned in par. 3.2.1 of the Ordinance (of Italy) N. 3431/2005 and shown here for convenience:
Zone 1: PGA10% -> 0,35 g
Zone 2: PGA10% -> 0,25 g
Zone 3: PGA10% -> 0,15 g
Zone 4: PGA10% -> 0,05 g
The values of the above may be altered by certain specifications and documents (eg INGV, 2006), but the differences can not be more than 20% of the acceleration for Zones 1 and 2 and the other 0.05g areas.
In the absence of more accurate determinations the ground acceleration PGA50% with expected exceedance probability of 50% in 50 years may be obtained by dividing the PGA10% for 2.5 of the table above (see Section 2.2 dell'OPCM N. 3431).
In the absence of more accurate determinations the ground acceleration PGA2% whit expected exceedance probability of 2% in 50 years may be obtained by multiplying by 1.5 the PGA10% of the above (see par. dell'OPCM 11.2.5.3 No 3431).
On the basis of what is mentioned in Ministerial Decree Infrastructure 2008, the risk indicators called α_u and α_e , are calculated with the following modified ratios:

$$\alpha_u = \frac{PGA_{SLV}}{S_T\ S\ \gamma_I\ PGA_{10\%}} \qquad \text{and} \qquad \alpha_e = \frac{PGA_{SDL}}{S_T\ S\ \gamma_I\ PGA_{50\%}}, \tag{1.3}$$

depending on the state limit of reference where,
- PGA10% = horizontal acceleration on soil of category A with 10% probability of exceedance in 50 years
- PGA63% = horizontal acceleration on soil of category A with 63% probability of exceedance in 50 years

- PGASLD = estimated PGA for reaching the limit state of damage (NTC 2008)
- PGASLV = = estimated PGA for reaching the limit state of Safety of Life (NTC 2008)

There are two levels of verification, Level 1 and Level 2, which differ for the different level of knowledge (LC) and the various tools of analysis and verification required. For all strategic and major works, will be carried out audits of level 1. The following list summarizes the operating modes corresponding to the two levels.

Level 1

The verification of Level 1 are those carried out by the method of linear analysis. In this case allowed a limited knowledge and LC1 is required to award a category of land described in DM 14/01/2008.

For buildings of reinforced concrete, the checks must be made using the limited knowledge level (LC1) according to DM 14/01/2008. Tested and verified in situ are as for that level of knowledge.

In the cases specified dallla NTC 2008, you can use the linear static analysis, in other cases will be used linear dynamic analysis.

The characteristics of the model, the seismic action and the audit finally, will be consistent with the method of analysis used in conformity with the NTC, 2008.

How to calculate PGACO , PGADS and PGADL

The calculation of the PGA corresponding to each limit state can be conducted with two methods.

The first is for multiple testing, performed in succession by increasing the ag and downstream control of the fulfillment of each analysis for each limit state verifications. The value of g corresponding to the last meeting of the checks is taken as the limit state related to PGA.

The second method requires a single analysis with PGA equal to unity. The PGA corresponding to the general limit state is the lowest among the values of the multiplier of seismic effect (stress or displacement) which, added to the effect produced by gravity loads, complies with the verification of the limit state considered.

Level 2

The verification of Level 2 are those carried out by the method of nonlinear analysis. is required to achieve at least an adequate level of knowledge LC2.

The checks of Level 2 are still preceded by Level 1 inspections, even with simplified methods.

You must then determine the ratios defined by (1.1) and (1.2) reported previously, with acceleration PGACO, and PGADS PGADL estimated through nonlinear analysis. In the case of non-linear static analysis, the goal is to establish a curve of global capacity of plan shear force vs moving a control node. The nonlinear static analysis should be diversified according to the different structural types.

It 'requires a deeper level of understanding of the type LC2 (adequate knowledge) or LC3 (detailed knowledge), as indicated in the 2008 NTC. It 'necessary to determine the range of soil and is permitted to consider separately the action in two main directions, but the building must be three-dimensional model.

How to calculate PGACO , PGADS and PGADL

Once calculated the capacity curve is possible to determine the period of the equivalent linear model to 1 degree of freedom, T*, and the movements corresponding to different limit states. The PGA corresponding to each limit state can be obtained by determining the elastic spectrum in moving of italian code $S_d = \omega^{-2} S_a$ (S_a =elastic spectrum in acceleration), that is obtained by inverting g at T* and the spectral displacement corresponding to travel so the structure reaches the limit state considered. Alternatively you can run more tests, increasing the ag sequence and downstream control of the fulfillment of each analysis for each limit state verifications. The value of g corresponding to the last meeting of the checks is taken as the limit state related to PGA.

Calculated resistances

For the calculate the capacity of ductile elements (checks in terms of deformation), we use the average values of the properties of existing materials, as obtained from in situ tests, divided by the Confidence Factor defined in relation to the level of knowledge achieved:

$$f_{cd} = f_{c,med} / FC$$

For calculating the capacity of fragile items (checks in terms of resistance) in tests at SLCO, we use the average values of the properties of existing materials, as obtained from in situ tests, divided by the Confidence Factor defined in relation to the level of knowledge achieved, and the the specific partial factor:

$$f_{cd} = f_{c,med} / FC \cdot \gamma_m$$

7. STEP 2: Results of analysis and indicators of earthquake risk

Was performed a nonlinear static analysis. From the findings was noted as about 30% of the longitudinal elements have the reinforcement bars lower than necessary to meet the tests of the 2008 NTC.

Is the same for shear-resistant armor, far from being sufficient in all the beams that run outside the building walls. The step is approximately 20 to 25 cm depending on the structural elements.

Even the internal beams of the type not to wall then, and the pillars of the whole building, have an insufficient armor for about 40% of the structural elements, as in this case the passage of the brackets is around 20 to 25 cm depending on whether you consider the area near the junction of the item or area of the center line.

The Push-over analysis, and the application of static forces that are gradually increased, was performed in both the X and Y, and in two lines + and -.

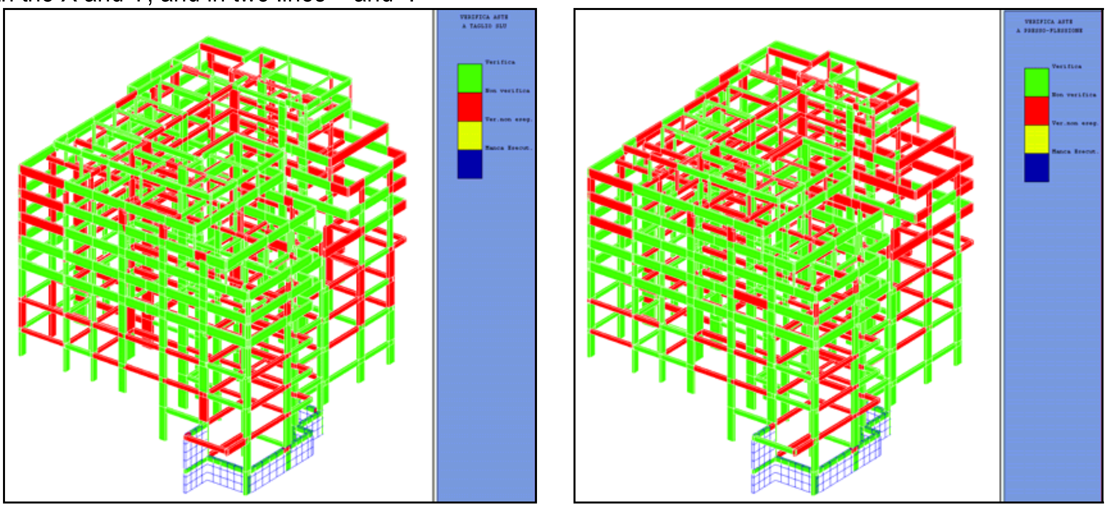

Figure 4: Shear and bending verify – SLU – pushover analisys

Then, it points out that, before the pushover analysis, was carried out a simulated design of the structure, using a design of structural elements that makes use only of the shares by the method of static allowable stress.

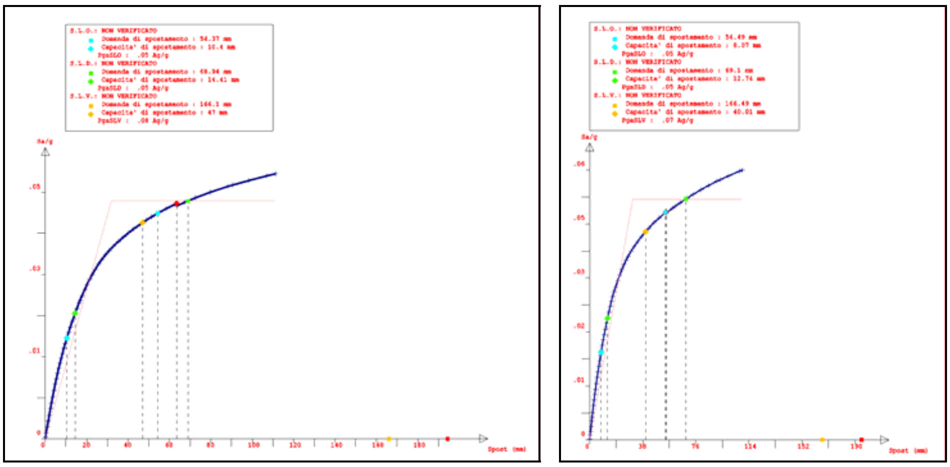

Figure 5: Capacity curve on ADSR spectrum , push n. 7 (Fy + & Fy -)

In the following pages are then given for the push over analysis, the deformation of the structure for each analysis with the ends of any individual auction, colored on hinges plastic, on the basis of their commitment.
Please note that the rotation is to give light to the yield point while the damage is severe for the last rotation of ¾ of the rotation for the collapse. On the right of the figure then, in addition to the color scale that identifies the level of damage associated with each hinge, giving the value of Displacement of the control point (center of gravity of the top floor of the building), the relationship αu/α1 of equivalent damping coefficient and structure factor q. Finally, we specify that the sequence of steps in the analysis was stopped at the achievement of the collapse of the structure.
In the figures above then shows the capacity curves, which describe the value of moving a control point of the structure (chosen at the top), with the increase of base shear.
From the spectrum shift on the limit state under consideration, according to the period T*, we get the request to move in the control point of building.
Then the comparison is made between the demand and the ability to move of the building in question, taht is assessed for the SLD that for the SLV, depending on the distribution of static forces applied to the structure.

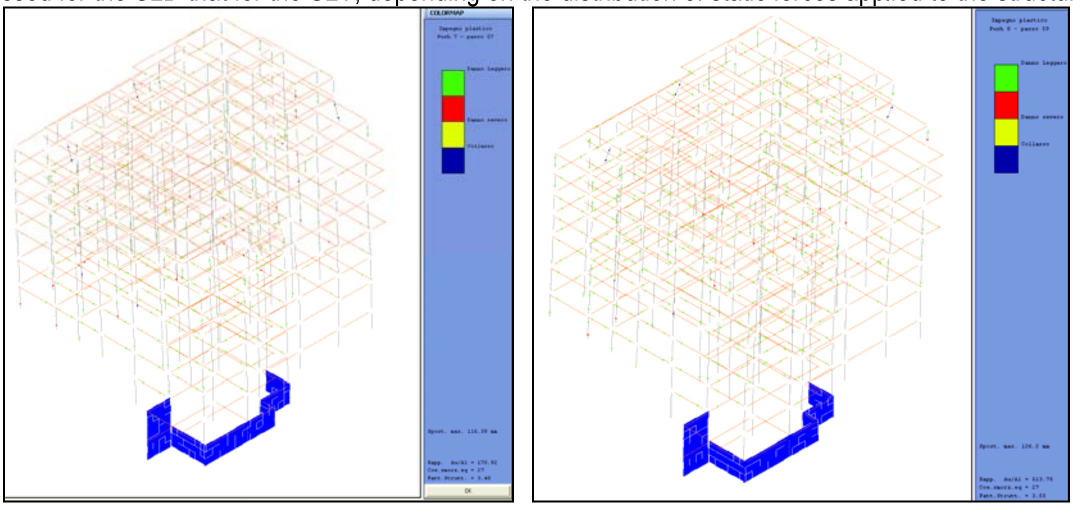

Figure 6: Commitment of plasticity of the structure, with plastic hinges in step 57- SLV, push n. 7 (Fy + & Fy -)

Indicators of seismic risk

The following indicators of seismic risk, referred to in par. 1.2, are obtained by pushover analysis.
Knowledge of the level of risk to the building is an essential prerogative for its preservation in time for use in safety.

N° push	*PGASLD / g*	*PGASLV / g*	*PGA10% / g*	*PGA63% / g*	α_u	α_e
5	*0,059*	*0,082*	*0,263*	*0,117*	*0,163*	*0,222*
6	*0,059*	*0,096*	*0,263*	*0,117*	*0,192*	*0,222*
7	*0,059*	*0,085*	*0,263*	*0,117*	*0,169*	*0,222*
8	*0,059*	*0,074*	*0,263*	*0,117*	*0,148*	*0,222*

The evaluation of seismic safety was conducted with reference to a method compatible with the level of knowledge attained, can estimate the acceleration of the soil corresponding to the attainment of the ultimate limit state. It should also be noted that this value of acceleration, compared to the acceleration peak feature of the site, only serves to define an index of earthquake safety (IS), useful to establish priorities for action, and action for improvement for seismic mitigation risk will eventually be made when the need arises, downstream from a more thorough assessment (LV2 or LV3). The seismic safety index IS of course, is defined with (1.1), (1.2) and (1.3).
Values of the seismic safety greater than 1 indicate that the article is suitable to withstand the seismic action planned in the area, on the contrary, if IS <1, the safety of the building is less than desirable, consistent with the requirements for appropriate development. The result of this evaluation is expressed in linguistic form,

through a level of vulnerability low, medium or high.

A way of illustration, it is possible to associate the quality levels of vulnerability to a range of acceleration values at ultimate limit state in the case of a product with importance factor of 1, placed on the foundation soil type A and in the absence of topographic amplification effects, we would get the following values of security depending on the seismic zone in which it is located:

- High vulnerability	IS	by	0.10	to	0.30
- Medium vulnerability	IS	by	0.30	to	1.00
- Low vulnerability	IS	by	1.00	to	2.00

Consequently, the seismic vulnerability of the building was high.

8. STEP 3: Conclusions and proposals for action

A simple and reliable procedure to evaluate the seismic resistance of some structural types of existing R/C buildings has been set up. It is mainly based on an accurate recognition of the main and widespread structural characteristics of the post-1970 building stock designed only to gravity loads. To this purpose, the codes in force, the handbook typically in use and, principally, a wide database of original design drawings relevant to real typical buildings of the period under consideration have been examined.

As reported in the new NTC 2008, the building in question was performed an analysis of vulnerability following an approach of performance, in terms of ultimate limit states. The results showed that the seismic vulnerability of the building was high.

In particular, it was determined the safety of the building in its current configuration, including providing proposals for measures to improve or adjustment seismic.

Have been proposed solution of improvement and adjustment seismic aimed at increasing the resilience of existing structures to the action concerned, even without reaching the required safety levels prescribed by the NTC, 2008.

The design and safety assessment will be extended to all parts of the structure could be affected by changes in behavior, and imminent danger of collapse and local plasticity (local or global).

It is estimated that 20% of the pillars and beams at various levels, should be adjusted with FRP or steel bands, to improve the ductility and strength, to achieve the seismic improvement (with total expenditure of about € 1.898.000,00).

then, It is estimated that 30% of the pillars and beams at various levels, should be adjusted with FRP or steel bands, to improve the ductility and strength, to achieve the seismic adjustment (with total expenditure of about € 3.806.000,00).

In addition, it has been recommended to reduce the mass of the plan and the reinforcement of the floors with lamps with high shear connectors.

Further studies aimed at a better evaluation of their vulnerability which take into account the interactions between the different plane types are currently in progress.

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