THE KNOWLEDGE LEVEL IN EXISTING BUILDINGS ASSESSMENT

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ABSTRACT:

In this paper a relevant topic in the new Italian Seismic Code is discussed: the knowledge level definition for existing buildings assessment. In the moment in which in a seismic Nation, as Italy in 2003, the Code modifies seismic zoning or it introduces new and more accurate methods for structural analysis, the topic of seismic safety evaluation for existing buildings assumes a relevant role. Particularly in the case of a strategic building that has to be sure during earthquake (avoiding severe structural damage) and to be efficient immediately after the seismic event (avoiding component damage).
For these reasons the assessment of an existing building has to be preceded by a phase in which performance objectives, seismic hazard and determination of deficiencies has to be taken into account. Obviously first phases are pertaining to Codes while the latter is devoted to structural engineer. For this aim Code introduces knowledge level as crucial aspect able to define design coefficients and assessment strategies.
The aspects that define the levels of knowledge are: global geometry, geometric characteristics of structural elements, structural details, reinforcement ratio and arrangements (in RC structures), structural connections (in steel structures), and connections among different structural elements, non structural element disposal and acceptable constitutive relationships for design. While the first ones of these matters are linked to a geometrical survey, the last one it is connected with the definition of a correct procedure for structural tests.
In the present paper the knowledge factor in a case of a strategic building, an important school in a seismic region, Marche Region, is discussed. The case-study building is a more complex one: portions of historical and recent masonry with RC floors are connected with a prevalent infilled RC frame. Results of a wide experimental tests campaign will be discussed and some indications for material qualification will be pointed out.

KEYWORDS: existing frames, knowledge level, original design data, in-situ tests, mechanical characterization

1. INTRODUCTION

In this paper the state of the art of Italian Seismic Code is discussed particularly regarding the actual approach to the evaluation of existing buildings. On this topic a case study is presented particularly regarding to laboratory tests on materials extracted from a complex strategic building located in a seismic region: Marche Region in the Central part of Italy.
Earthquake engineering had a strong evolution in the last Century, in spite of that a great effort has to be done by both Scientific Community to deep theoretical knowledge of seismic phenomenon and Government in order to control design activity for new buildings and to funding a correct assessment of existing infrastructural facilities and housing network. Thus in the aim to avoid catastrophic events such more recent 2008 Sichuan Earthquake.
Starting from a linear elastic approach to structural design and considering practical impossibility to guarantee this regime for just medium earthquake, in 20th Century a widely-accepted philosophy was carried out that permits nonlinear response of a building when subjected to a ground motion that is representative of the design earthquake. The concept was to allow calculating a reduced equivalent base shear to produce a rough approximation of the internal forces during a design earthquake. This equivalent base shear is highly lower than theoretical elastic shear. In spite of simplicity of this philosophy, the in-field experience shows that if an uncorrected detailing of structures is carried out the effects of an earthquake could be dramatically negative. Due to a lack of knowledge on earthquake demand and building capacity this design procedure has to be considered as an unsafety one.

For this reason a refinement of this non elastic approach was carried out with the so-called Capacity Design procedure: the equivalent base shear can be reduced but the structural design has to guarantee a large, stable and energy consuming post-elastic behaviour. According to this philosophy a large amount of current design of 70's & 80's has been disavowed, above all if it has been carried out, as in Italy, by means of allowable stress criteria for structural element check.

Particularly a check between reducing base shear factor, generally named as response modification factor R, and structural response has to be focused. On this basis the structure force-displacement is assumed as control parameter and by means of different approximation criteria (equal energy for stiff structures or equal displacement for deformable structures) a relationship between response modification factor and structural ductility, μ, can be derived ($R=\sqrt{2\mu-1}$ or $R=\mu$ respectively, ATC 19 (1995)). So structural design has to guarantee a target ductility in global response and, consequently, for each element and connection. In this aim a control of strength hierarchy and structural redundancy has to be carried out in order to obtain an optimum seismic design.

Recent earthquake observation (as 1994 Northridge Earthquake in California) has shown a more complex situation, in particular the building displacements provided using this base shear are significantly lower than the displacements that a building will really experience during a design earthquake, even if a correct capacity design was carried out (ATC 40 (1996)). A new trend (Displacement Based or Performance Based) was put on discussion in scientific community and a new design approach was defined. The topic is not to define structural ductility in order to define conventional seismic input (or vice versa) but a more complex procedure has to be carried out: to determine inelastic earthquake demand based on inelastic capacity of building, to solve demand versus capacity problem in order to determine performance point, to design structure in terms of an imposed displacement.

This debate involved not only scientific community but, obviously, Government department and standard commission (FEMA 356 (2000)) and formed decisions in terms of Code promulgation. This debate involved the Italian community too, and some related arguments are discussed in this paper.

So if after 70's and 80's very destructive earthquake (such as 1976 Friuli and 1980 Irpinia Earthquakes that caused more than 3700 deaths) a long period (up to 1996) had to pass in order to introduce Capacity Design and Ultimate Limit State design approaches in Codes. But, on the other hand, the allowable stress criterion was permitted too. After the less destructive 1997 Umbria-Marche Region Earthquake, Code hadn't changed even if new criteria were introduced by Regional Governments in post-earthquake repairing, particularly devoted to masonry historical heritage, De Sortis et al. (1998). Thereafter a great part of national territory wasn't again classified as seismic.

At long last, after the 2002 Molise Region Earthquake that caused some victims in a territory classified as not seismic (even if historically earthquake prone), the seismic zoning was upgraded and all national territory has been classified as seismic with 4 seismic input levels (OPCM 3274 (2003)). This Code wasn't a Performance Based Code but it provided that an extensive program of seismic analysis could be carried in order to define a global strategy for seismic risk reduction. In particular strategic existing buildings, as schools, hospitals, fire-stations, had to be involved in order to obtain not only a Life Safety Performance Level but also an Operational or Immediate Occupancy Level. More recently another Code (D.M. 14.01.2008) confirmed these objectives and redefined seismic zoning introducing a grid of 10751 nodes of seismic spectral parameters for all National territory. In each node a different value for peak ground acceleration (a_g), local amplification factor (F_o) and control period (T_C, upper limit of constant spectral acceleration branch) is defined. Thus seismic input isn't a large scale parameter (as necessarily if only four seismic zones are defined for all national territory) but it is a local provision.

Regarding to existing buildings OPCM 3274 considered the well-know necessity of a preliminary building evaluation (FEMA 310 (1998)) and, according to Eurocode 8 part 3 (CEN (2005)), introduces some criteria to collect information for structural assessment. A definition of Knowledge Level (KL in this paper) is carried out and some KL identification criteria are defined particularly devoted to the definition of levels of inspection and testing. On the basis of this National Code and of the necessity to coordinate local interventions some Regional Governments prepared Technical Documents on this topic (Ferrini, M. (Ed.) (2004), Dolce et al. (2005), Marche Region (2005)); these documents generally suggest some modifications and additions with respect to National Code, Biondi et al. (2008). On this topic it is to note that the more recent Italian Code (D.M. 14.01.2008) considers a global strategy for existing buildings (Chapter 8) even if the KL it isn't defined in this document.

For these reasons, basing on an interesting case study that is a strategic building (an important primary and secondary school) in a seismic region, a discussion of KL relevance in existing building assessment is the topic of the paper. Above of all in the awareness that KL is not the state-of-the-art for structural engineers knowledge.

2. KNOWLEDGE LEVEL DEFINITION

In case of an existing building, it has not to be designed but it has to be evaluated; building evaluation involves many substantial difficulties. One is the matter of uncovering the structure since plans and design calculations often are not available, or it is not sure that design provisions had to be applied correctly during construction phases. In many buildings the structure is concealed by architectural finishes or, as in the case study, different quality in non-structural finishing can deceive structural engineer.
Some intrusive testing may be necessary to determine material quality and to verify geometry and reinforcement provision conformance with the plans. The extent of this investigation is a crucial question in existing building evaluation and it has been defined by means a target Knowledge Level, FEMA 310 (1998) - FEMA 356 (2000).
According to both European Code (Eurocode 8 Part 3, CEN (2005)) and Italian Code (OPCM 3274), three different Knowledge Levels (KL1, KL2, KL3) can be defined depending on three different states of knowledge (I., II., III.) and permit to choose two different classes of structural parameters (A., B.):

KL1: Limited Knowledge, KL2: Normal Knowledge, KL3: Full Knowledge

I. Geometry, II. Details, III. Materials

A. Type of Analysis, B. Material partial factors [γ_m for OPCM 3274 - Confidence factor CF for EC8-3]

Each category can be subdivided as:
I. Geometry: **I.A. Original** construction drawings (O.C.D.) **plus** in-situ **visual** survey – **I.B.** in-situ **full** survey
II. Details: **II.A. Simulated** design as in relevant practice **plus limited** in-situ inspection (I.S.I.) - **II.B**, **Incomplete** O.C.D. **plus limited** I.S.I. **or extended** I.S.I. - **II.C. Complete** O.C.D. **plus limited** I.S.I. **or extended** I.S.I.
III. Materials: **III.A. Standard** values of the time of construction **plus limited** in-situ tests (I.S.T.), **III.B. Original** design specifications **plus limited** I.S.T. **or extended** I.S.T., **III.C. Original** test report **plus limited** I.S.T. **or comprehensive** I.S.T.
A. Type of Analysis: **A.I.** Linear lateral force analysis, **A.II.** Multi-modal response spectrum analysis, **A.III.** Nonlinear static analysis, **A.IV.** Non-linear time-history analysis.

Table 1. Knowledge Levels and corresponding state of knowledge and structural parameters

Knowledge Level	Geometry	Details	Materials	Type of Analysis	OPCM 3274	EC8-3
					γ_m	CF
KL1	I.A. or I.B.	II.A.	III.A.	A.I. or A.II.	$\gamma_m = 1.25\gamma_c$ - $\gamma_m = 1.15\gamma_s$	$CF = 1.35$
KL2		II.B.	III.B.	All	$\gamma_m = \gamma_c$ - $\gamma_m = \gamma_s$	$CF = 1.20$
KL3		II.C.	III.C.	All	$\gamma_m = 0.80\gamma_c$ - $\gamma_m = 0.85\gamma_s$	$CF = 1.00$

In Table 1 Knowledge Levels and corresponding state of knowledge and structural parameters are shown, in this Table 1 two different material partial factors are considered: γ_c for concrete and γ_s for steel rebar, while EC8-3 CF are those recommended by EC8-3. Different values can be defined in National Annex. As shown in Table 1, the two Codes define similarly KLs, apart that for partial coefficients. Just again this different parameters have to be applied to different reference values: for OPCM 3274 they have to be applied to new building values (i.e. have to be applied to characteristic values) while for EC8-3 they have to be applied to "*... the mean values obtained from in-situ tests and from the additional sources of information …*" So different design strength can be defined for concrete, f_{cdIC}, f_{cdEC}, and steel, f_{sdIC}, f_{sdEC}, for both OPCM 3274 and EC8-3:

$$f_{cdIC} = \frac{f_{ck}}{\gamma_m} \qquad f_{sdIC} = \frac{f_{sk}}{\gamma_m} \qquad f_{cdEC} = \frac{f_{cm}}{\gamma_c}\frac{1}{CF} \qquad f_{sdEC} = \frac{f_{sm}}{\gamma_s}\frac{1}{CF} \tag{2.1}$$

If the same values of the partial factors adopted for the persistent and transient design load situations are applied for two Codes, $\gamma_c = 1.50$ and $\gamma_s = 1.15$, the values of Table 2 are obtained.

Table 2. Material partial factors for different KL for OPCM 3274 and EC8-3

Knowledge Level	OPCM 3274		EC8-3	
	Concrete	Steel	Concrete	Steel
KL1	$\gamma_m = 1.25\gamma_c = 1.875$	$\gamma_m = 1.15\gamma_s = 1.323$	$\gamma_m = CF\gamma_c = 2.025$	$\gamma_m = CF\gamma_s = 1.553$
KL2	$\gamma_m = \gamma_c = 1.50$	$\gamma_m = \gamma_s = 1.15$	$\gamma_m = CF\gamma_c = 1.80$	$\gamma_m = CF\gamma_s = 1.38$
KL3	$\gamma_m = 0.80\gamma_c = 1.20$	$\gamma_m = 0.85\gamma_s = 0.978$	$\gamma_m = CF\gamma_c = 1.50$	$\gamma_m = CF\gamma_s = 1.15$

Table 3. Recommended minimum requirements for different levels of inspection and testing

Knowledge Level	OPCM 3274		EC8-3	
	Elements	Samples	Elements	Samples
KL1	15%	1	20%	1
KL2	35%	2	50%	2
KL3	50%	3	80%	3

The average ratio between γ_{mIC} and γ_{mEC} is $\gamma_{mIC} / \gamma_{mEC} \approx 0.85$ for both concrete and steel; this value is often characteristic to average strength ratio for new materials: is this assumption correct for existing buildings? We will discuss it later on.

Again the two Codes have same differences in recommended minimum requirements for different levels of inspection and testing; in Table 3 these differences are shown in terms of percentage of elements (beam and column) that have to be checked for details and material (concrete and steel) samples per floor.

It is possible to note an evident greater request in terms of elements in elements check in EC8-3 in respect to OPCM 3274; thereafter while EC8-3 states " *... there might be cases requiring modifications to increase some of them. These cases will be indicated in the National Annex ...*" the OPCM 3274 was integrated by another Code (OPCM 3431 (2005)) that permits to take into account the repetitiveness of structural elements in order to reduce population in percentage calculus. The same OPCM 3431 (2005) clarified that in the case of concrete specimens, the limit of Table 3 has to be intended as samples per each 300 sqm of a floor and permitted to substitute destructive material tests with non-destructive in the limit of 50% of global requirement and considering that one destructive tests is equivalent to three in situ non destructive tests.

A discussion of the possibility to use in-situ test for KL evaluation is in the companion paper Biondi et al. (2008), where some Regional Government provisions are outlined too [Ferrini (Ed.) (2004), Dolce et al. (2005), Marche Region (2005)]. In the next chapter consequences in KL definition of II. Details provisions and OPCM 3274 and OPCM 3431 to EC8-3 differences in Table 2 will be briefly discussed.

3. THE CASE STUDY: A SCHOOL BUILDING IN MARCHE REGION

According to OPCM 3274 a wide program of structural analysis was carried out in Italy, particularly referring to strategic buildings. According to Marche Regional Government Bylaw No. 1616 of December 12, 2005, the seismic assessment of IPSIA – Sant'Anna School in Corridonia (Macerata province) was assigned to a Temporary Association of Engineering Societies (Integra Ltd Rome, Iskra Sas Fermo, Stin Ltd Rome). Structural detail analysis was carried out by a private company (Cnd Ltd. Rome) while destructive tests on in-situ samples were carried out by Scam Structural Laboratory of Chieti-Pescara University, under the Author supervision.

Corridonia is a little town (about 15,000 inhabitants) in Marche Region located at 60 km away for Epicentral Area of 1997 Umbria-Marche Earthquake (September 27, 1997; $M_L = 5.80$). In 1997 Corridonia was classified as 2nd category seismic zone and this zoning was confirmed in OPCM 3274 with seismic input parameters peak ground acceleration ($a_g / g = 0.25$), local amplification factor ($S = 1.25$) and control period ($T_C = 0.50s$).

These parameters are changed in $\left[a_g / g = 0.11 \quad F_o = 2.43 \quad T_C = 0.31\right]$, $\left[a_g / g = 0.35 \quad F_o = 2.45 \quad T_C = 0.34\right]$ for Damage Limit State ($T_R = 140$ years) and Ultimate Limit State ($T_R = 2475$ years) respectively, due to (D.M. 14.01.2008). In Figure 1 a general plan of IPSIA – Sant'Anna School complex in Corridonia and a partial sketch of structural details analysis at second floor are shown.

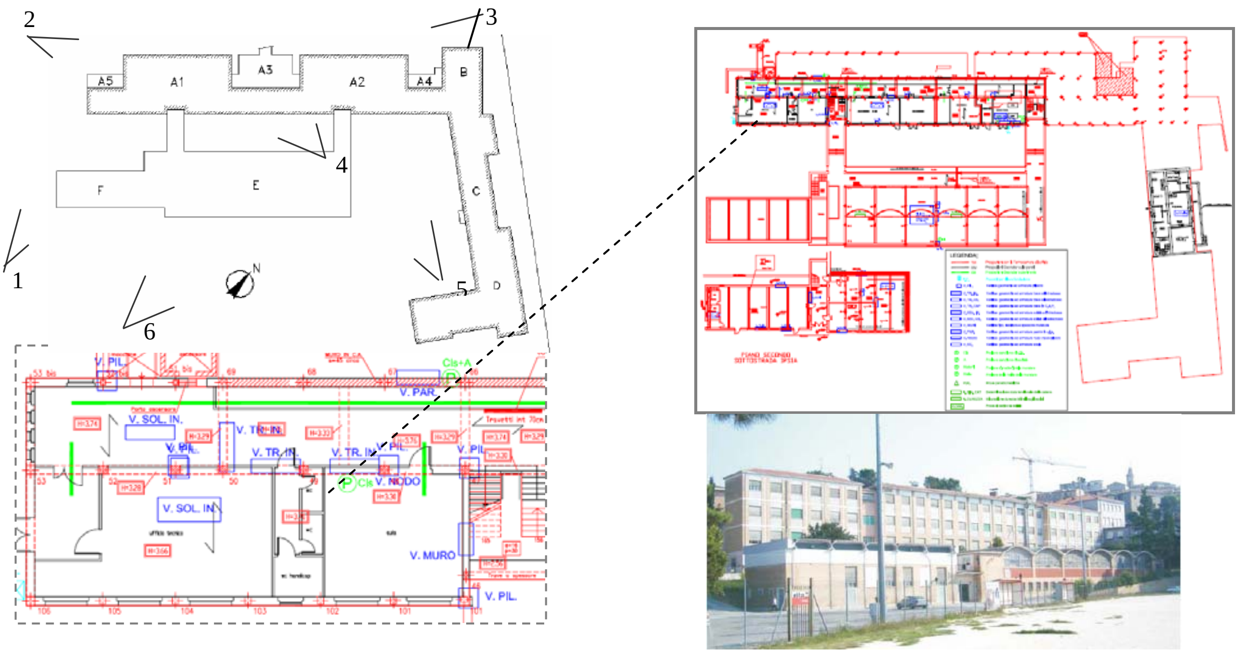

Figure 1. IPSIA – Sant'Anna School complex in Corridonia general plan with photo points of view and in details analysis (up side), partial sketch of structural details analysis at ground floor and general view (down)

Figure 2. Views of IPSIA – Sant'Anna School complex in Corridonia in clockwise order from top left according to points of view in Figure 1

The complex is dived in two different administrative boards: the IPSIA Secondary School (A1, partially A2, A3, E, and F; for a total consistency of 6,500 m^2 and 26,500 m^3) and the Sant'Anna Primary School (partially A2, A4, B, C, D for a total consistency of 4,650 m^2 and 18,500 m^3).
Briefly A1-A2 is the principal building of the complex, it is a mixed structure built in 1952 with external confined masonry, internal cast in situ RC frame, tile-lintel slender floors without concrete slabs; A3, A4 & A5 a recent (80's) RC or steel frame for stairs and lift, designed without seismic gaps. B & C buildings are traditional masonry structure (C was a XVIII Century Church) that are heavily rearranged from a structural point of view. In particular inadequate (too stiff) precast RC floors were built in 70's. D is more recent masonry structure with tile-lintel floors, E is a 50's one floor masonry laboratory building with vaulted cylindrical floors and steel tie braces for horizontal load components. Finally F is a sport hall, built in 70's, with a partial prestressed floor beams.

3.1. Knowledge level determination
According to Owner request the Temporary Association of Engineering Societies had to guarantee a KL2: for this aim, according to Table 1, the following states of knowledge and classes of structural parameters were selected:

I. Geometry: **I.B.** in-situ **full** survey **plus limited** O.C.D.

II. Details: **II.B**, **Incomplete** O.C.D. **plus limited** I.S.I.

III. Materials: **III.A.-III.B. Standard** values of the time of construction and (partially) **Original** design specifications **plus limited** I.S.T.,

A. Type of Analysis: **A.II.** Multi-modal response spectrum analysis, **A.III.** Nonlinear static analysis.

In particular the attention was focused on A1-A2 principal building; for this building the Engineering team had partially at disposal original (1952) construction drawings, construction drawings regarding to 1977 tile-lintel floor assessment and retaining RC wall construction with pile foundation, structural design of 1982 RC frame. In Figure 3 original construction drawings or technical handbook and in-situ inspection results are shown.
It is possible to note a good accuracy between original construction drawings or technical handbook excepted for retaining RC wall. In this case just the in-situ visual survey showed a suspicious rotation of top of the wall (see photo in Figure 3), a subsequent excavation at foundation level confirmed this suspicion: foundation piles weren't built (may be for lack of money?) and retaining wall is practically a gravity wall. So this fact forced the Engineering team to design a pile wall in order to guarantee global slope stability in presence of a seismic event.

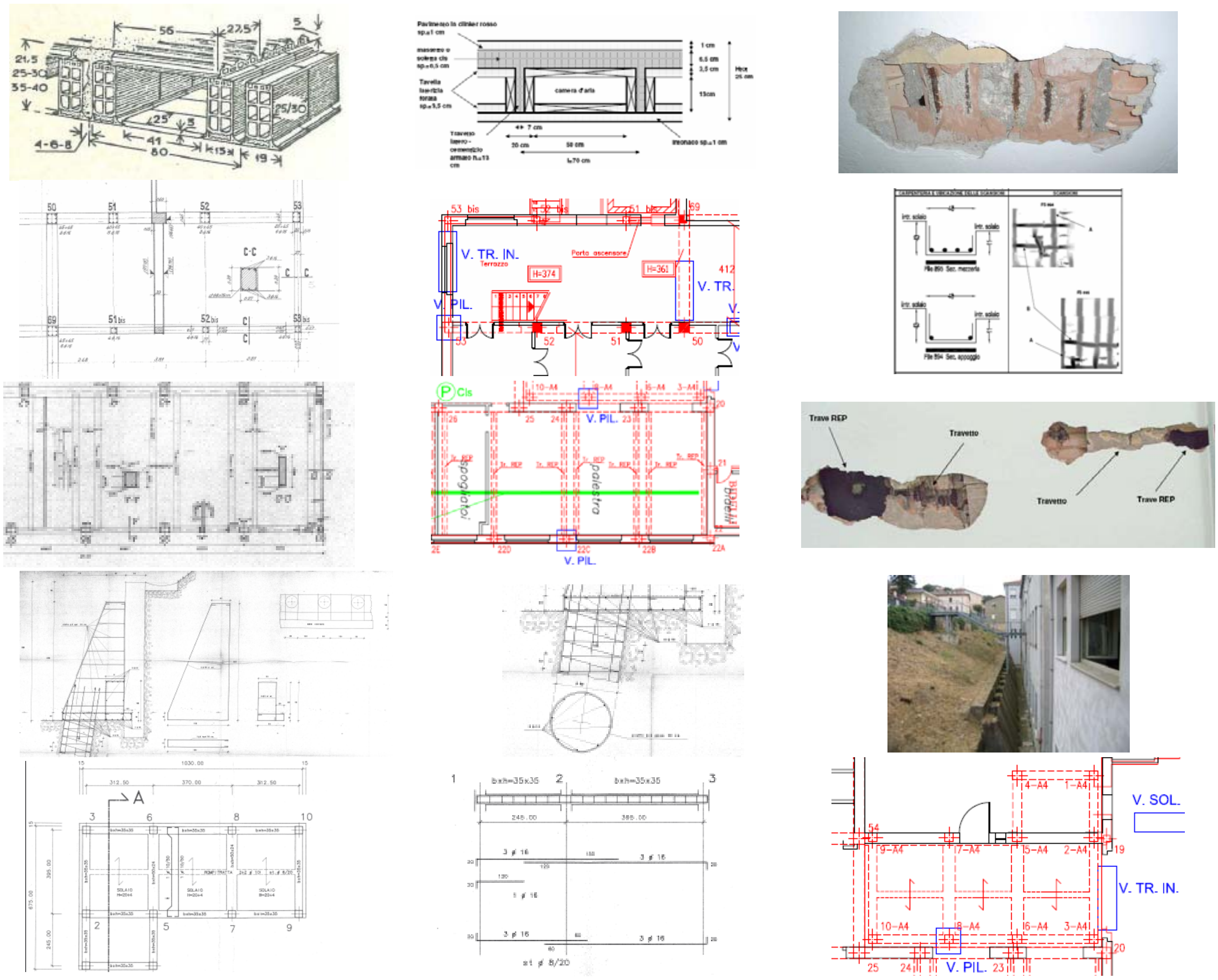

Figure 3. Original construction drawings or technical handbook (left side) and in-situ inspection results (right side)

In terms of state of knowledge, the situation was the following:

I. Geometry: in-situ full survey with almost 11,000 m^2 of survey real plans. In these plans an overlapping of original drawings and of test positions was carried out, Figure 1;

II. Details: **48** geometrical and reinforcement inspections of columns; **45** of beams; **18** of tile-lintel and precast RC floors; **3** geometrical reinforcement inspections of beam-column joints; **6** of shear walls; **1** relay of both RC stairs and prestressed RC beam; **18** geometrical and dimensional inspections of structural masonry; **97** pacometer surveys for reinforcement disposal controlling in RC elements; **1** complete daily set of Ground Penetrating radar and infrared thermal camera inspection;

III. Materials: **19** drilled out cores and **10** SonReb tests for concrete; **15** brick extractions for compressive and three point flexural tests; **4** chemical tests and **18** non-destructive penetrometer tests for hardness of mortar; **4** extractions of rebars; **5** Brinell hardness test and **2** steel tie braces stress evaluation by means of instrumented impact hammer for steel; **1** load test on tile-lintel slender floors without concrete slab (A-2 build.).

It is possible to note a frequency of 1 destructive test per 293 floor m^2 and 1 non destructive test per 398 floor m^2 for concrete and masonry: this option is in coherence with repetitiveness criteria of OPCM 3431 (2005) and with what discussed in Biondi et al. (2008). On the other hand rebar extraction and load test on floors have been limited due to Owner request. Concrete cores and bricks were subject to Laboratory destructive tests. In Figure 4 some photos of compressive tests on concrete and bricks are shown. Tests results are summarized in Table 4. In this Table 4 average, standard deviation, minimum, maximum and characteristic values are collected. It has to note that for concrete the conventional cubic value is considered in two different cases: R_{cc} for whole population and R^*_{cc} for those cores that didn't shown problems (as rebar presence, micro-cracking due to in-situ drilling out, large aggregate diameter presence).

In Figure 4 lowest (C13 $f_c = 2.90MPa$) and highest strength cores (C5 $f_c = 19.70MPa$) are shown. As above said, test results has to "engineering" controlled: in fact if all results are considered unacceptable values are obtained no more in terms of average value $R_{ccm} = 8.48Mpa$ but in terms of characteristic value $R_{cck} = R_{ccm} - k\sigma = 0.76Mpa$ due to large standard deviation $\sigma = 5.51MPa$. Thereafter also considering R^*_{cc} values (particularly $R_{cc} \geq 6.00Mpa$) a very high standard deviation $\sigma = 4.80MPa$. For these reasons seems that OPCM 3274 hypothesis to use material partial factor in respect to characteristics values [Eqn. (2.1)] could be misleading from a structural point of view. More stable values are obtained both for reinforcement steel and for brick both in flexural (B6 $f_{bf} = 2.70MPa$, A1 build.) and perpendicular to mortar bed (B10 $f_{bc} = 26.40MPa$, C build.) and normal to mortar bed (B14 $f_{bc} = 27.30MPa$, D build.) compressive tests on bricks. It is to note that brick B10 is an old solid bricks and its strengths is about 30% higher than strongest core ($R_{cc\max} = 20.68Mpa$). In spite of this good brick performance, masonry has a global low strength due to very poor mortar strength (it is to note that a mortar that has $f_{mm} = 0.49MPa$ can be refused according to Codes. Finally in Table 4 values of equivalent rebound, R_{ccR}, and in-laboratory pulse velocity, R^*_{ccV}, compressive strengths are shown for complete population. With these values a simulated SonReb value, R_{ccS}, are determinate according to Basilicata Region provisions (Dolce et al. (2005)).

Figure 4. Compressive tests on concrete cores in clockwise order from up left from lower strength to higher strength (C5 $f_c = 19.70MPa$); brick flexural and compressive (perpendicular or normal to mortar bed) tests

Table 4. Test results on concrete cores, steel rebars, bricks and mortar

	concrete cores				in situ tests			steel rebars		bricks		mortar
	γ	R_{cc}	R^*_{cc}	E_c	R_{ccR}	R^*_{ccV}	R_{ccS}	f_y	E_s	f_b	f_{bf}	f_m
	[kN/m^3]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]	[Mpa]
average	21.09	8.48	12.12	12733	33.03	8.62	14.39	317.22	178131	21.67	2.56	0.49
standard deviation	0.43	5.51	4.80		11.46	3.44		33.52		4.57	0.73	0.11
minimum	20.55	2.89	6.35		19.64	4.23		283.15		13.20	1.42	0.35
maximum	21.91	20.68	20.68		63.59	17.39		360.04		30.10	3.39	0.74
characteristic		0.76	5.40		7.89	3.81	6.68	159.18		15.98	1.62	0.36

As discussed in Biondi et al. (2008) this method overestimates destructive test results, also is in-laboratory pulse velocity, R^*_{ccV}, is taken into account.

4. CONCLUSIONS

In the paper an evaluation of Knowledge Level relevance on seismic analysis of existing buildings is carried out. Different provisions of Italian Code (OPCM 3274) and Eurocode 8 are discussed basing on an interesting case study. It is possible to remark that KL procedure has to be refined again in order to obtain stable results and above all in order to become a procedure easy to use for structural engineers.

In particular a KL approach has to put a great attention in control original construction drawings in order to verify the correspondence of what designed with reality.

From material point of view a refinement of testing procedure has desirable in order to avoid to overestimate structural element quality if in-situ non destructive tests are used.

So the Eurocode 8 provision, that only destructive tests could be accepted and that material partial factors have to be applied in reference to average test values, appears more on safety side respect to OPCM 3274.

Particularly for this class of existing buildings probably the concept of characteristic strength has no sense and the in-situ non destructive test has to be used by means of different correlation formulas than those for new buildings.

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