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Evaluation and considerations about fundamental periods of damaged reinforced concrete buildings

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Abstract. The aim of this paper is an empirical estimation of the fundamental period of reinforced concrete buildings and its variation due to structural and non-structural damage. The 2009 L'Aquila earthquake has highlighted the mismatch between experimental data and code provisions value not only for undamaged buildings but also for the damaged ones. The 6 April 2009 L'Aquila earthquake provided the first opportunity in Italy to estimate the fundamental period of reinforced concrete (RC) buildings after a strong seismic sequence. A total of 68 buildings with different characteristics, such as age, height and damage level, have been investigated by performing ambient vibration measurements that provided their fundamental translational period. Four different damage levels were considered according with the definitions by EMS 98 (European Macroseismic Scale), trying to regroup the estimated fundamental periods versus building heights according to damage. The fundamental period of RC buildings estimated for low damage level is equal to the previous relationship obtained in Italy and Europe for undamaged buildings, well below code provisions. When damage levels are higher, the fundamental periods increase, but again with values much lower than those provided by codes. Finally, the authors suggest a possible update of the code formula for the simplified estimation of the fundamental period of vibration for existing RC buildings, taking into account also the inelastic behaviour.

1 Introduction

The estimation of the fundamental period is a crucial aspect in response analysis for existing buildings and for their assessment and retrofitting. A reliable estimation of the fundamental period T is an important aspect both in classic (force-based design, FBD) and in more recent design procedures (e.g. pushover analysis, displacement-based design; see for a review Masi and Vona, 2010). In the linear static or dynamic method (FBD) the fundamental period (predicted in a simplified manner or calculated by analytical model) is the crucial parameter to define the spectral acceleration and thus the base shear force.

At present, concerning the evaluation of seismic actions on single structures, most design codes (e.g. ATC, 1978; BSSC, 2003; CEN, 2005; NZSEE, 2006) provide period-height empirical expressions, usually set up with an elastic force-based design in mind. The recent research into earthquake engineering shows that the fundamental periods estimated by numerical models are often significantly different than those obtained when using an experimental approach (Gallipoli et al., 2009, 2010; Oliveira and Navarro, 2010; Michel et al., 2010). These differences are probably due to the fact that the dynamic properties of buildings are evaluated using numerical analyses based on inaccurate FE (finite element) models, based on too simplified models which inadequately reproduce the dynamic behaviour of real structures. Although the mass properties can be easily assessed, the geometry and stiffness of structural and non-structural elements (when considered) are too simplified as well as the consequent damping properties of the structure considered. In fact, the infill panels are not generally included in this kind of numerical models. The inability to model correctly the structural damping linked to material properties and other physical characteristics (e.g. opening and closing of cracks, structural and non-structural elements interaction, etc.) makes numerical estimate of periods less realistic (Masi and Vona, 2010). There are recent works where special-purpose-element models have been developed in order to represent the actual seismic response of the RC (reinforced concrete) structures (Karayannis et al., 2011). Several key parameters influencing the dynamic characteristics of the buildings have been included, such as infills, beam-column RC joints, and boundary conditions such as structure to soil and/or adjacent structure interaction. Nevertheless the gap between numerical and experimental results exists, and the experimental estimations of fundamental period as well as its variation due to estimates damage should be an important step to update the models. The aim of this study is to estimate the fundamental period variation taking into account different levels of damage into account. To this purpose, we have collected several recordings during and after the L'Aquila (2009) seismic sequence (Mucciarelli et al., 2011; Picozzi et al., 2011). The fundamental periods of 68 RC buildings have been estimated for the first time in Italy after a strong seismic sequence. The fundamental periods of RC buildings with different typologies, structural characteristics, age, heights and damage levels have been investigated by means of ambient vibration measurements. Four different damage levels were considered with regard to the 5 damage levels defined by EMS 98 (Gruenthal, 1998). A new height–period relationship is proposed for the practical purposes such as the assessment and the retrofitting of the existing European RC moment-resistant frame (MRF) buildings.

2 Buildings data set and analysis

The investigated structures have been selected in L'Aquila and within the surrounding villages in order to cover a wide span of important characteristics such as

- seismic design;
- design/construction age;
- height;
- damage level.

The age of construction ranges from 1950 to 2000. After the 1915 Avezzano earthquake, L'Aquila has been classified as a seismic area, therefore all the RC buildings studied were designed according to the Italian seismic code enforced at the time of construction.

With regard to the seismic aspects, design details do not appear different to those of RC buildings designed only to vertical loads. Moreover, the surveyed damage showed that the behaviour of anti-seismic RC buildings in L'Aquila is not very different from those related to RC buildings designed only to vertical loads, as reported in Mucciarelli et al. (2004) and Masi and Vona (2010). This similarity is due to the use of old codes of seismic design. In fact, some typical characteristics have been observed:

- Beam–column joints without ties and ineffective anchorage of the steel bars (Fig. 1a).
- Open hoops (Fig. 1b).
- Structural elements with small section and few reinforcement bars (hoops and longitudinal bars, Fig. 1c).
- Structural system: strong frames mainly along one direction (typically the longitudinal direction) while in the orthogonal direction weak frames are generally present.

On the other hand, it is worth noting that the mechanisms of surveyed damage are not different to those surveyed in recent Italian earthquakes on existing from RC buildings designed to resist to vertical loads only (Fig. 2).

The 68 RC buildings where measurements were undertaken had heights ranging from 11 to 27 m, and different damage levels so that it is possible to study the real influence of strong ground shaking on fundamental period and to compare these results with those obtained with regard to the undamaged buildings. The damage levels (DL) range from DL = 1 (non-structural damage) to DL = 4 (heavy structural damage) accordingly with the EMS 98 (Gruenthal, 1998).

Figure 3 shows typical examples of the observed damage levels. Table 1 reports the distribution of buildings according to age classes and damage level. It has to be noted that the selected buildings are mainly grouped in the oldest age classes, and obviously the DL = 0 is mainly present in more recent buildings.

The main characteristics of the buildings studied are summarized in Table 2. We also considered some particular cases: two buildings with completely bare frames (Fig. 4b, number 36 and 37 in Table 2), two buildings without stiff

Table 1. Distribution of damage levels of surveyed RC buildings.

Age	No. Buildings	Damage level 0	1	2	3	4
1946–1961	9	1		7	1	
1962–1971	7			7		
1972–1981	18		1	11	6	
1982–1991	23	5	7	4	3	4
1992–2000	9	6	2		1	
2001–2009	2	2				
	68	14	10	29	11	4

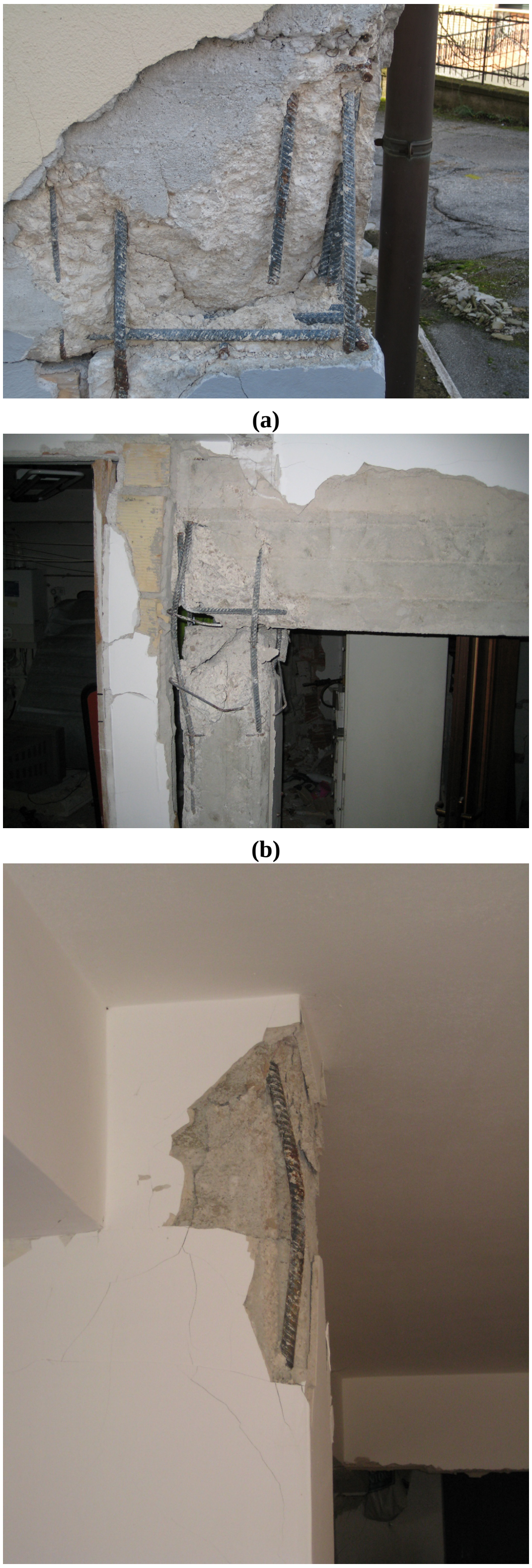
Table 2. Main characteristics of surveyed RC buildings.

ID	Age	H [m]	Period [s]	DL [EMS 98]
1	1992–2000	14.9	0.24	0
2	1992–2000	14.9	0.23	0
3	1982–1991	17.6	0.23	0
4	1982–1991	16.0	0.31	0
5	2001–2009	12.4	0.24	0
6	1982–1991	14.8	0.21	0
7	1992–2000	14.5	0.22	0
8	1992–2000	14.5	0.21	0
9	1992–2000	14.5	0.24	0
10	1992–2000	14.5	0.23	0
11	2001–2009	5.2	0.12	0
12	1946–1961	17.6	0.21	0
13	1982–1991	24.7	0.29	0
14	1982–1991	17.0	0.26	0
15	1992–2000	21.7	0.58	1
16	1982–1991	17.6	0.23	1
17	1972–1981	14.5	0.22	1
18	1982–1991	14.5	0.21	1
19	1982–1991	14.5	0.27	1
20	1982–1991	14.5	0.27	1
21	1982–1991	14.5	0.25	1
22	1982–1991	11.7	0.20	1
23	1982–1991	16.0	0.37	1
24	1992–2000	14.5	0.27	1
25	1982–1991	20.7	0.57	2
26	1982–1991	20.7	0.60	2
27	1982–1991	14.5	0.47	2
28	1972–1981	18.6	0.48	2
29	1972–1981	17.6	0.33	2
30	1972–1981	17.6	0.33	2
31	1972–1981	20.7	0.33	2
32	1982–1991	11.4	0.33	2
33	1962–1971	11.4	0.31	2
34	1972–1981	14.5	0.39	2
35	1972–1981	26.9	0.50	2
36	1972–1981	23.8	0.76	2
37	1972-1981	23.8	0.81	2
38	1962–1971	17.6	0.40	2
39	1962–1971	17.6	0.21	2
40	1962–1971	14.5	0.55	2
41	1962–1971	17.6	0.70	2
42	1972–1981	20.7	0.44	2
43	1946–1961	17.6	0.42	2
44	1946–1961	17.6	0.39	2
45	1946–1961	14.5	0.27	2
46	1946–1961	11.4	0.25	2
47	1946–1961	11.4	0.34	2
48	1946–1961	14.5	0.26	2
49	1946–1961	20.7	0.57	2
50	1972–1981	14.5	0.34	2
51	1972–1981	14.5	0.39	2
52	1962–1971	24.0	0.69	2
53	1962–1971	24.0	0.69	2
54	1982–1991	18.6	0.55	3
55	1982–1991	18.6	0.61	3
56	1972–1981	18.6	0.53	3
57	1972–1981	14.5	0.54	3
58	1972–1981	14.5	0.50	3
59	1982–1991	11.4	0.31	3
60	1972–1981	26.9	0.50	3
61	1992–2000	11.4	0.38	3
62	1946–1961	12.8	0.35	3
63	1972–1981	14.5	0.50	3
64	1972–1981	14.5	0.47	3
65	1982–1991	18.6	0.63	4
66	1982–1991	11.4	0.41	4
67	1982–1991	11.4	0.38	4
68	1982–1991	11.4	0.38	4

stair structures (number 40 and 41 in Table 2), and one building with just one story with completely bare frames (Fig. 4a number 44 in Table 2). The age classes have been derived both from the post-earthquake damage and the safety assessment inspection form (AeDES form) released by the Italian Department of Civil Protection after the 1997 Umbria–Marche and the 2002 Molise earthquakes (Baggio et al., 2007). Age classes have been selected according to different periods of enforcing of different building codes.

3 Data analysis and structural dynamic characterization

The best method to determine the dynamic parameters is, of course, that of recording earthquakes inside permanently monitored buildings, but this is a costly option restricted to a limited number of case studies. A possible alternative to permanent monitoring systems is to perform noise measurements using temporary stations. The use of a fast procedure and a portable instrument allows studying a large number of buildings during an earthquake-related emergency. Gallipoli et al. (2009, 2010) and Ditommaso et al. (2010, 2012) have compared several techniques for structural dynamic identification: ambient noise vibrations are very useful for the characterization of the fundamental frequency and the related modal shape (if more than one station has been used). In this work the frequencies have been estimated using ambient noise recordings by means of a portable three-directional tromometer (Micromed Tromino). In very few instances it was possible to perform more measurements within each building at different floors and at different points. In most cases, just one measurement has been carried out at the highest



(a)

(b)

(c)

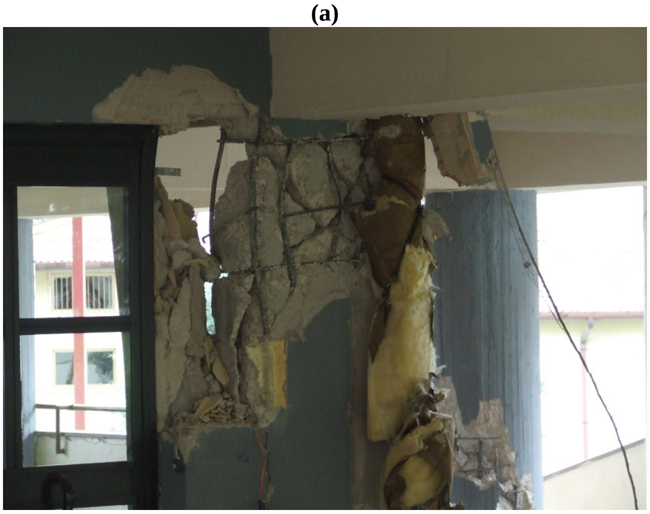
Fig. 1. Reinforcement details of an RC building designed taking into account the Italian seismic code (1975).

accessible floor in order to identify the fundamental period related to each instrumented structure. In all cases, the position was approximately central with respect to the plan configuration of the building. Measurements were performed using a time-window length variable from 6 to 10 min and a sampling frequency equal to 256 Hz. To estimate the building's fundamental period the horizontal to vertical spectral ratio (HVSR) technique has been used (Castro et al., 1998; Gallipoli et al., 2004; Di Giulio et al., 2005). Then, it was possible to estimate the building's fundamental period with a very good reliability. There are many reasons for the use of HVSR. Firstly, it was not possible to use the SSR (Standard Spectral Ratio) to estimate the transfer function of the buildings due to safety reasons. Most of the buildings were seriously damaged and the aftershock sequence was ongoing, so we wanted to minimize the number of measurements and the crew performing them; nowadays the restoration work often provides no space for free field measurements. Moreover, in a noisy environment with ongoing restoration and demolitions, HVSR has the same target in the building and in the soil: that of separating the true shear response of the investigated object from the propagating noise that can mask the true eigenfrequencies. In previous works, HVSR has proved to be a reliable proxy of SSR to estimate fundamental frequencies, as demonstrated on a large set of Italian buildings by Gallipoli et al. (2009), provided that the measurement points are carefully selected to avoid membrane modes of the floors and other possible problems as described in the appendix of the paper by Gallipoli et al. (2010).

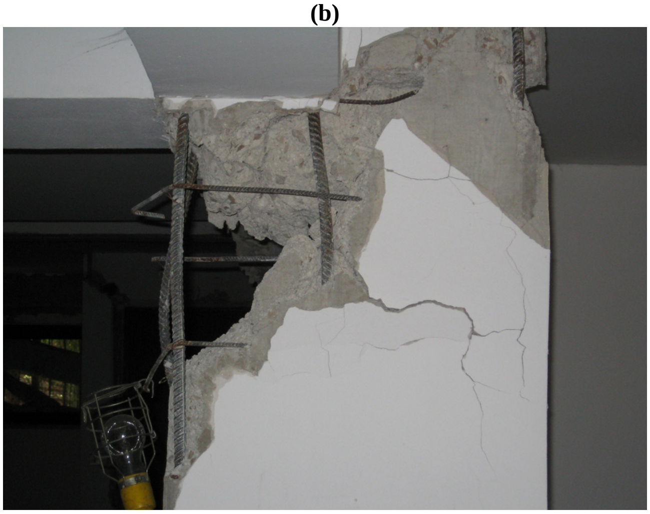
Table 2 reports the analysis results for each building, with age classes, damage levels and building height. Figure 5 shows the comparison between the periods of the damaged RC buildings with those obtained for undamaged RC Italian and European buildings (Gallipoli et al., 2009, 2010). It is worth noting that the theoretical equation provided by the codes returns an over-estimation of periods also for the highest damage level (DL = 4). Supposing that before the damage the frequencies of the buildings in L'Aquila were placed along the same relationship as for undamaged Italian buildings, the period increase due to damage never exceeds 100 % for the highest damage level (DL = 4), and the period increase is around 60 % when considering the lower damage levels (DL = 2 and DL = 3). This is in good agreement with the observation provided for the only Italian buildings whose dynamic behaviour was observed during a damaging earthquake which occurred in Molise, October 2002 (Mucciarelli et al., 2004), and as reported in Masi and Vona (2010) for damaged RC moment-resisting frames. In Mucciarelli et al. (2004) the damage state of an RC building in Bonefro (due to the 2002 seismic sequence in Molise, Italy) has been reported. During the second shock of the seismic sequence (1 November, $M_l = 5.3$, IMCS = VII), the building's damage increased (damage grade equal to 4 according to the EMS 98 scale) and the authors have observed the evolution of the fundamental period before, during and after the

Fig. 2. Comparison between damages from different Italian earthquakes. **(a)**, **(b)**, **(c)**: L'Aquila, Italy, L'Aquila earthquake 2009. **(d)**, San Giuliano, Italy, Molise earthquake 2002. **(e)**, Castelluccio Inferiore, Italy, Pollino earthquake 1998. **(f)**, Bonefro, Italy, Molise earthquake 2002.

(a)



(b)



(c)

Fig. 3. Damage level 3 **(a)** and damage level 4 **(b)**, **(c)**.

(a)

(b)

Fig. 4. Example of special case 1, top panel, **(a)**, and special case 2, bottom panel, **(b)**.

recorded earthquake using a seismic station placed within the building. The available data allow one to estimate the fundamental period of this building before the second shock and upon the shift of frequency due to damage. Moreover, the fundamental period of the undamaged structure (before the first strong motion) was estimated on the basis of a redefined numerical model (implementation of in situ and laboratory tests). Finally, rather low variations of the period values have

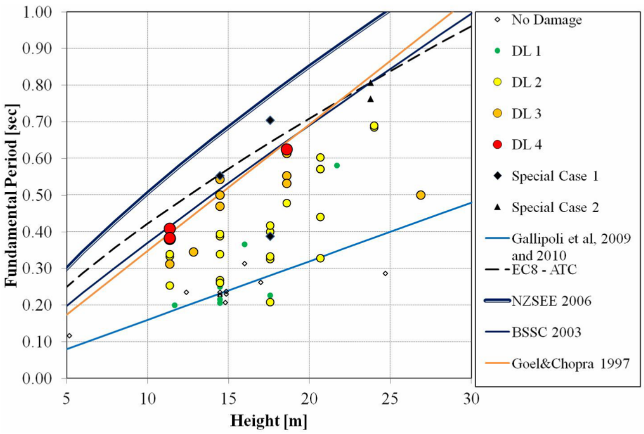


Fig. 5. Correlation between height, period and damage level, and comparison with code formulas.

been found (max value of the period elongation is around 30 %), in spite of the condition of incipient collapse after the second shock. Similarly, during the April 2009 sequence, Mucciarelli et al. (2011) studied the transient non-stationary behaviour of a damaged building carrying out ambient noise and strong motion measurements: the structural eigen-frequencies decrease during each aftershock, but then they come back to the starting values after each event. In addition, no damage evolution was found during the aftershock sequence because the building remains within the same damage mechanism caused by the mainshock.

In this study it is important to note that the highest damage levels (DL3, DL4) are often localized only within the first and second storeys and generally only on few columns or beam–column joints. These observations of localized damaged following strong earthquakes do not match the hypothesis of the codes of a diffuse damage on the frame. This is probably due to possible defects governing the damage evolution during earthquakes and confirms that old Italian seismic codes do not consider correctly the contribution of the infilled panels, staircases, and any other non-structural elements that contribute to the stiffness of buildings and their dynamic behaviour on damage to buildings during and after the earthquake.

The values of periods of the building showing the major damage level (DL4) could be considered as the upper limit for this typology of buildings (RC MRF). On the contrary, the theoretical equation provided by the codes returns periods that are an over-estimation even when this level of damage is observed. This disagreement has to be considered taking into account the construction practices used in each country, with the need to perform a similar investigation and to define a relationship based on typical Italian and European buildings, as showed in previous works by Gallipoli et al. (2009, 2010), Guler et al. (2008), and Oliveira et al. (2010) for undamaged RC buildings.

Table 3. Regression coefficient for different grouping of height/period data according to observed EMS 98 damage.

	DL2	DL3	DL2–DL3	DL0–DL1
alfa	0.026	0.028	0.026	0.016
lower 95 %	0.023	0.024	0.024	0.014
higher 95 %	0.028	0.033	0.028	0.018

4 Results and discussion

Starting from the experimental evidence, possible improvements to the current code provisions can be proposed. The analyses of the results reported in this paper show a significant difference between the code provision and the real dynamic behaviour of the existing RC buildings, both damaged and undamaged. We first explore a possible separation of more or less damaged buildings, dividing the database to consider the different damage levels surveyed.

The period vs. height data of the buildings with DL1 are very close to RC Italian and European relationships experimentally estimated by Gallipoli et al. (2009) and Gallipoli et al. (2010) for undamaged buildings. The first damage level (DL1) is defined in EMS 98 (Gruenthal, 1998) as negligible-to-slight damage (only non-structural damage) with fine cracks in plaster over frame members or in walls at their base, and fine cracks in partitions and infills. According to this definition, it is possible to consider DL1 as the damage limitation State defined in CEN (2005).

The DL2 and DL3 (cracks in columns and beam column joints of frames at the base and at joints of coupled walls, spalling of concrete cover, buckling of reinforced rods, large cracks in partition and infill walls, failure of individual infill panels. Cracks in columns and beams of frames and in structural walls, cracks in partition and infill walls, loss of brittle cladding and plaster, and falling mortar from the joints of wall panels.) could be considered as the ultimate limit state and the relevant periods representative of the yielding period. Based on the description of the SD (significant damage) limit state proposed by EC8-3 (CEN, 2005), damage levels 2 and 3 could be grouped because they represent the low and medium structural damage. Coherently with the paper's goals, damage levels 2 and 3 can be considered as corresponding to the life safety performance level.

Instead, based on the description of the NC (near collapse) limit state proposed by EC8-3 (CEN, 2005), the most similar to this definition is DL4, when the structure has suffered heavy, non-repairable damage with low residual lateral strength and stiffness.

Some statistical analyses have been carried out. The form $T = \alpha \cdot H$ has been considered. The constant α depends on the building properties and it is determined by regression analysis of the measurements by minimizing the square error

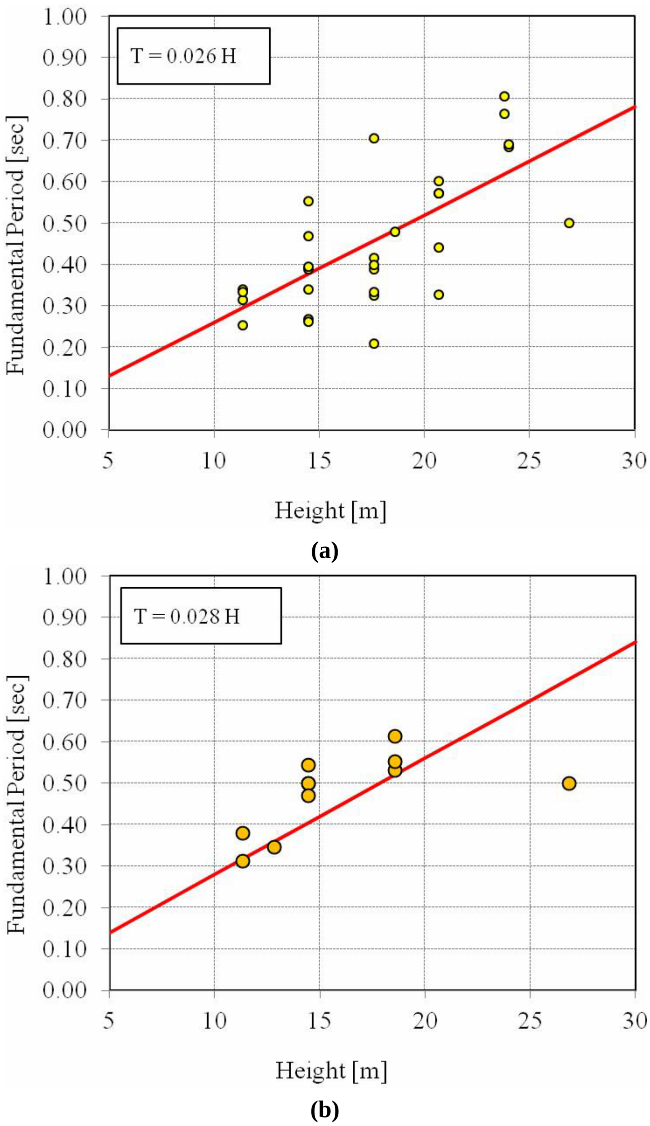


Fig. 6. Correlation between height and period for buildings with damage levels 2 **(a)** and 3 **(b)**.

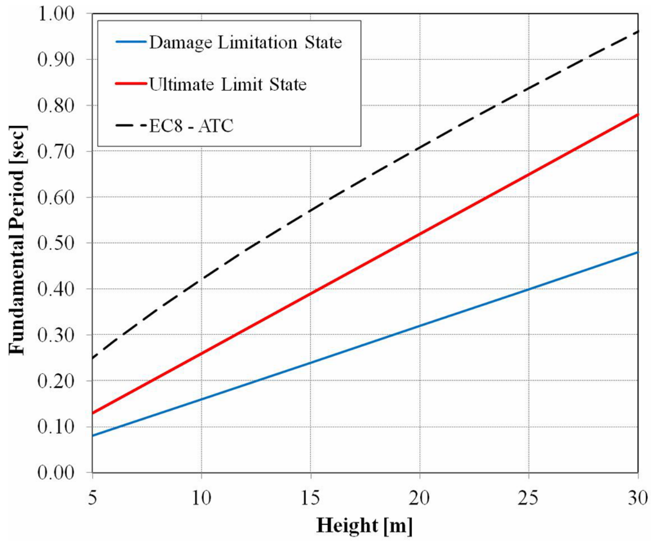


Fig. 7. Height–period relationship for different damage levels and comparison with code formula.

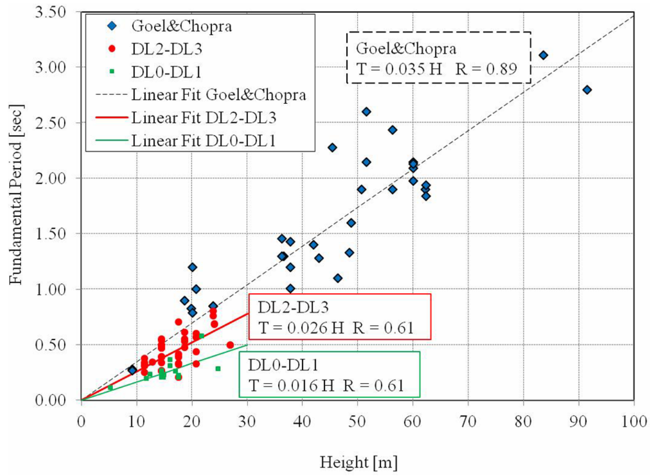


Fig. 8. Height vs. fundamental periods: comparison between 37 fundamental periods estimated by Goel and Chopra (1997), and 64 in this study (considering only the damage levels lower that DL4).

between measured and evaluated periods. Obviously, in this formula the intercept at $H = 0$ is taken equal to 0.

We consider separately the DL2 and DL3, and then both of them compared with the joint regression of DL0 and DL1. The results in Table 3 clearly show that it is possible to group DL2 and DL3, whose separate regressions fall within each others confidence bounds. The regression for DL0 and DL1, on the other hand, returns a coefficient that is significantly different from that of DL2 and DL3 taken together. The fundamental period–height relationships for damaged RC building are reported in Fig. 6, with equation $T = 0.026 \cdot H$ and $T = 0.028 \cdot H$, respectively. Finally, Fig. 7 reports the comparison between the EC8 relationship and the two experimental period–height relationships: one obtained for undamaged buildings considering it as a lower limit in elastic force-based design (DL1 and DL0), and the other related to buildings with DL2 and DL3, which represents the higher limit value for the assessment and retrofitting of existing RC buildings, in particular when the post-elastic behaviour of the structure is taken into account.

In order to try to understand why the proposed formulation in the European code is so different with respect to our empirical relationships both for damaged and undamaged buildings, it could be interesting to reappraise the data from which the EC8 derives. The EC8 height/period provisions, as well as those of most of the seismic codes in the world derive from the work of Goel and Chopra (1997). They proposed empirical formulae to estimate the fundamental vibration period of

RC MRF buildings, based on measurements carried out during several earthquakes in California, on mostly undamaged buildings.

In their work Goel and Chopra (1997) advise that "since these recommendations are developed based on data from buildings in California, they should be applied with discretion to buildings in less seismic regions of the US or other parts of the world where buildings design practice is significantly different than in California." Besides these mentioned design and construction differences between Californian and Italian RC buildings, one must take into account also the different composition of the two databases. Figure 8 shows the data (37 measurements on 27 RC MRF buildings) obtained in Goel and Chopra (1997) together with those described in this study. The two data sets barely overlap: in Goel and Chopra (1997) there are only 7 buildings with a height less than 35 m (9 floors), while the formula proposed here has been derived using buildings with heights in the range of 11–27 m.

5 Conclusions

The fundamental period of RC buildings and its elongation are relevant to earthquake engineering applications on existing buildings, and must be treated very carefully. In the present paper, a data set acquired after the 6 April 2009 L'Aquila earthquake has been analysed in order to investigate the dynamic behaviour of damaged buildings. The obtained experimental results have been compared with some codes and empirical relationships. This comparison shows a systematic over-estimation of period values by code provisions, not only for undamaged but even for damaged buildings. The code provisions need to be reviewed starting from the experimental evidence. The height–period relationship grouped for damage levels proposed in this paper can be used in code procedures for the assessment and the retrofitting of existing RC buildings, considering a grouping of damage according the five EMS classes that return a similarity with the three EC8 damage levels. According to our experimental results, in the future it will be important to consider also the contribution of damaged/undamaged non-structural elements in the evaluation of a building's fundamental period.

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References

ATC: Tentative provisions for the development of seismic regulations for buildings, Report N. ATC3-06, Applied Technology Council, California, 1978.

Baggio, C., Bernardini, A., Colozza, R., Corazza, L., Della Bella, M., Di Pasquale, G., Dolce, M., Goretti, A., Martinelli, A., Orsini, G., Papa, F., and Zuccaro, G.: Field manual for post-earthquake damage and safety assessment and short term countermeasures (AeDES), EUR 22868 EN – Joint Research Centre – Institute for the Protection and Security of the Citizen, edited by: Pinto, A. V. and Taucer, F., Luxembourg: Office for Official Publications of the European Communities 2007, EUR scientific and technical research series – ISSN 1018-5593, 2007.

BSSC: NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450), 2003 Edn., Building Seismic Safety Council, Washington DC, 2003.

Castro, R. R., Mucciarelli, M., Pacor, F., Federici, P., and Zaninetti, A.: Determination of the characteristic frequency of two dams located in the region of Calabria, Italy, B. Seismol. Soc. Am., 88, 503–511, 1998.

CEN: Eurocode 8: Design provisions for earthquake of structures – part 1–4: strengthening and repair of buildings, European Prestandard ENV 1998-1-4, Comité Européen de Normalisation, Brussels, 2005.

Di Giulio, G., Azzara, R. M., Cultrera, G., Giammarinaro, M. S., Vallone, P., and Rovelli, A.: Effect of local geology on ground motion in the city of Palermo, Italy, as inferred from aftershocks of the 6 September 2002 Mw 5.9 earthquake, Bull. Seismol. Soc. Am., 95, 2328–2341, 2005.

Ditommaso, R., Parolai, S., Mucciarelli, M., Eggert, S., Sobiesiak, M., and Zschau, J.: Monitoring the response and the back-radiated energy of a building subjected to ambient vibration and impulsive action: the Falkenhof Tower (Potsdam, Germany), Bull. Earthq. Eng., 8, 705–722, 2010.

Ditommaso, R., Mucciarelli, M., Parolai, S., and Picozzi, M.: Monitoring the structural dynamic response of a masonry tower: comparing classical and time-frequency analyses, Bull. Earthq. Eng., 10, 1221–1235, 2012.

Gallipoli, M. R., Mucciarelli, M., Castro, R. R., Monachesi, G., and Contri, P.: Structure, soil – structure response and effects of damage based on observations of horizontal-to-vertical spectral ratio of microtremors, Soil Dynam. Earthq. Eng., 24, 487–495, 2004.

Gallipoli, M. R., Mucciarelli, M., and Vona, M.: Empirical estimate of fundamental frequencies and damping for Italian buildings, Earthq. Eng. Struct. Dynam., 38, 973–988, 2009.

Gallipoli, M. R., Mucciarelli, M., Šket-Motnikar, B., Zupanćić, P., Gosar, A., Prevolnik, S., Herak, M., Stipčević, J., Herak, D., Milutinović, Z., and Olumćeva, T.: Empirical estimates of dynamic parameters on a large set of European buildings, Bull. Earthq. Eng., 8, 593–607, 2010.

Goel, R. K. and Chopra, A. K.: Vibration properties of buildings determined from recorded earthquake motions, Report No. UCB/EERC-97-14 December 1997, available at: http://nisee.berkeley.edu/elibrary/Text/320788, last access: 31 May 2013).

Gruenthal, G. (Ed.): European Macroseismic Scale 1998 (EMS-98), European Seismological Commission, sub commission on Engineering Seismology, working Group Macroseismic Scales. Conseil de l'Europe, Cahiers du Centre Européen de Géodynamique et de Séismologie, Volume 15, Luxembourg, 1998.

Guler, K., Yuksel, E., and Kocak, A.: Estimation of the fundamental vibration period of existing RC buildings in Turkey utilizing ambient vibration records, J. Earthq. Eng., 12, 140–150, 2008.

Karayannis, C. G., Favvata, M. J., and Kakaletsis, D. J: Seismic behaviour of infilled and pilotis RC frame structures with beam–column joint degradation effect, J. Eng. Struct., 33, 2821–2831, 2011.

Masi, A. and Vona, M.: Experimental and numerical evaluation of the fundamental period of undamaged and damaged RC buildings, Bull. Earthq. Eng., 8, 643–654, 2010.

Michel, C., Guéguen, P., Lestuzzi, P., and Bard, P. Y.: Comparison between seismic vulnerability models and experimental dynamic properties of existing buildings in France, Bull. Earthq. Eng., 8, 1295–1307, 2010.

Mucciarelli, M., Masi, A., Gallipoli, M. R., Harabaglia, P., Vona, M., Ponzo, F., and Dolce, M.: Analysis of RC Building Dynamic Response and Soil-Building Resonance Based on Data Recorded during a Damaging Earthquake (Molise, Italy, 2002), Bull. Seismol. Soc. Am., 94, 1943–1953, 2004.

Mucciarelli, M., Bianca, M., Ditommaso, R., Gallipoli, M. R., Masi, A., Milkereit, C., Parolai, S., Picozzi, M., and Vona, M.: Far field damage on RC buildings: the case study of Navelli during the L'Aquila (Italy) seismic sequence, 2009, Bull. Earthq. Eng., 9, 263–283, 2011.

NZSEE: Assessment and improvement of the Structural Performance of Buildings in Earthquakes, Recommendations of a NZSEE Study Group on Earthquake Risk Buildings, 2006.

Oliveira, C. S. and Navarro, M.: Fundamental periods of vibration of RC buildings in Portugal from in-situ experimental and numerical techniques, Bull. Earthq. Eng., 8, 609–642, 2010.

Picozzi, M., Parolai, S., Mucciarelli, M., Milkereit, C., Bindi, D., Ditommaso, R., Vona, M., Gallipoli, M. R., and Zschau, J.: Interferometric analysis of strong ground motion for structural health monitoring: the example of the L'Aquila (Italy) seismic sequence, 2009, Bull. Seismol. Soc. Am., 101, 2, 635–651, 2009.