

Simplified models for seismic vulnerability analysis of bell towers

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ABSTRACT: In order to evaluate the seismic vulnerability of ancient bell towers, the simplified mechanical model proposed into Italian document “Guidelines for evaluation and mitigation of seismic risk to cultural heritage” was applied to a sample of 31 bell towers damaged by the 1976 Friuli (Italy) earthquake. The seismic safety level has been evaluated taking into account the seismic input of the 1976 seismic event, in order to compare the forecast obtained by this simplified model with the observed damage. The comparison has highlighted some limits of the proposed methodology, based on the hypothesis of a tower with cantilever behaviour, constrained at the base that collapsed to axial compression and bending action. Such behaviour is not often confirmed by the damage observation. The crack patterns put in evidence the development of local collapse mechanisms ruled mainly by the equilibrium loss of masonry portion instead of crushing phenomena. The bell towers are not frequently able to develop an overall behaviour for the lack of interlocking corners or of steel tie-rods or of well-connected diaphragms. In the paper the authors proposed a new simplified method to evaluate the seismic risk of towers, based on the analysis of the constructive characteristics of the structure.

1 INTRODUCTION

In the “Guidelines for evaluation and mitigation of seismic risk to cultural heritage” (called from now on *Guidelines*) different simplified mechanical models (LV1 level) have been identified which are useful for the most diffuse types of historic structures: *buildings, villas and other structures with bearing walls and horizontal diaphragms; churches and other structures with large halls, without intermediate diaphragms; towers, bell towers, and other tall and slender structures*. The adoption of these models though affected by uncertainties, has the ability of supplying a homogeneous evaluation on a territorial scale and, thus, is significant for the aim of designing future strengthening interventions.

The LV1 level allows the evaluation of collapse acceleration by means of a simplified method based on a limited number of geometric and mechanical parameters or which utilises qualitative tools (analysis of the construction characteristics, critical and stratigraphical surveys).

Seismic behaviour of the towers depends on certain specific factors: the slenderness of the structure, the degree of connection between the walls, the eventual presence of adjacent structures in the lower portions, which may create horizontal constraints, the presence of slender architectural elements at the top of the structure (steeple, towering gables, battlements, etc.) or in any case belfries.

Quite frequently, towers and bell towers are in contact with other lower structures. Some usual cases are towers built as part of or next to churches, towers incorporated in various ways within the urban setting and towers built into city walls. The presence of horizontal constraints at differing heights can deeply modify the behaviour of the structure. On the one hand, by limiting the actual slenderness, and on the other by creating localised stiffening elements and points where stress-concentration is possible. Survey damage has generally demonstrated that these situations are often characterised by noteworthy vulnerability.

In bell towers, the belfry can be a particularly vulnerable element, due to the wide openings that lead the pillars to be slender and due to top masses. Further, the amplification of seismic motion has an even more critical effect in the higher parts of the construction. The observation of damage has in fact demonstrated how similar belfries can behave in very different ways, even when the seismic action at the base of the tower was the same, and this is due to the diverse interaction between earthquakes, foundation soil, structure and superstructure. Analogous considerations must be made for slender and towering elements at the top of towers. Their vulnerability is due first to their modest vertical-load bearing (related only to dead weight) that guarantees a poor stabilising effect with respect to overturning.

Damage and collapse mechanisms for the towers are, therefore, various and depend on geometric

variables like slenderness as well as constructive characteristics (masonry quality and clamping).

The development of simplified mechanical models capable of analysing these mechanisms at territorial level is not possible. Therefore, for the towers, the *Guidelines* proposed a simplified model based on hypothesis of a failure due to combined axial force and bending moment. The model considers towers as cantilevers, solicited by lateral forces, in addition to their dead loads, which may be subject to crises in a generic section for crushing in the compressed zone, after the reduction of the effective un-cracked area due to non-tensile-strength.

31 bell towers damaged by 1976 Friuli earthquake have been studied applying the LV1 level proposed in the *Guidelines*.

2 BELL TOWERS OF FRIULI DAMAGE BY THE 1976 SEISMIC EVENTS

In order to apply the LV1 model proposed by the *Guidelines* for the towers a wide database was assembled. In particular we have focused our attention on the bell towers damaged by the Friuli (Italy) earthquake (6th May and 15th September 1976). This choice has been due to the concomitancy of three different factors: the high magnitude of the seismic events (equal to 6.4 for the first main shock); the wide presence of isolated bell towers, and the detailed data available.

The high intensity of the earthquake has allowed to highlight clearly the damage pattern occurred and the collapse mechanisms activated. The high number of isolated bell towers has permitted to simplified the analysis in order to do not take into account the dynamic interaction of the bell tower with the church and, therefore, increasing the reliability of the adopted simplified model. This particular situation, if it could limit a generalization of the results for other kind of towers, is necessary in order to decrease the variable number that characterized the seismic behaviour.

The 1976 Friuli earthquake represents the first Italian seismic event deeply studied and analyzed. The first studies of the seismic behaviour of monumental buildings are, in fact, grown up as from this event. For this reason, the wide documentation of the seismic behaviour of cultural heritage has been used to obtain the information able to adopted the LV1 model proposed in *Guidelines*. The data in the “The churches and the earthquake” (Doglioni *et al.*, 1994, in Italian) include figures, the geometric and crack patterns surveyed after the seismic event of many bell towers, and a preliminary interpretation of the more recurrent collapse mechanisms. Moreover the recent formation of the “Documentation Centre on the Earthquake and Cultural Heritage of Venzone” has allowed the authors to obtain for different bell-towers substantial

Table 1. Bell tower sample.

Code	Village	Church
Ch 190	Arba	—
Ch 192	Buia	B.V. ad Melotum
Ch 18	Cavazzo Carnico	S. Daniele
Ch 39	Cavazzo Carnico	S. Valentino
Ch 41	Cavazzo Carnico	S. Stefano
Ch 200	Colloredo M. A.	Ognissanti
Ch 31	Forgaria del Friuli	S. Lorenzo
Ch 45	Forgaria del Friuli	S. Giuliana
Ch 201	Forgaria del Friuli	S. Nicolò
Ch 12	Gemona	S. M. Assunta
Ch 202	Gemona	Santo Spirito
Ch 207	Majano	SS. Pietro e Paolo
Ch 209	Majano	S. Elena Imperatrice
Ch 16	Moggio	S. Spirito
Ch 212	Nimis	SS. Gervasio e Protasio
Ch 213	Nimis	S. Stefano
Ch 216	Osoppo	S. Maria ad Nives
Ch 21	Pinzano	S. Stefano
Ch 33	Raveo	S. Maria
Ch 147	Resiutta	S. Martino
Ch 222	S. Leonardo	S. Leonardo Abate
Ch 226	Spilimbergo	—
Ch 227	Spilimbergo	S. Marco
Ch 228	Taiapana	SS. Trinità
Ch 229	Tarcento	S. Pietro Apostolo
Ch 231	Tarcento	S. Biagio V. M.
Ch 233	Tarcento	S. Antonio Abate
Ch 235	Trasaghis	S. Bartolomeo Apostolo
Ch 236	Trasaghis	S. Michele Arcangelo
Ch 181	Villa Santina	S. Maria Maddalena
Ch 240	Vito d’Asio	—

photographic documentation and more detailed surveys (regarding both geometrical data and the material, the kind of intermediate floors, the description of some constructive details).

On the basis of the catalogued data a sample composed by 31 bell towers (16 isolated and 15 with only one or two sides shared with the church) has been identified. In Table 1 the list of the 31 bell towers analyzed is reported. Each bell tower is identified by the code adopted in the book “The churches and the earthquake” (Doglioni *et al.*, 1994).

The damage level surveyed after the 1976 seismic events has highlighted the high vulnerability of such kind of buildings. In many cases, in fact, the structures were completely compromised by the earthquake, causing total collapse (Figure 1) or the demolition of the existing structure during the reconstruction phase (Figure 2).

The 1976 seismic sequence was characterized by two main shocks with analogous magnitude (May 1976 – Ml 6.4; September 1976 – 6.1). This features increased damage, of which the authors have taken into account in the comparison between the safety index (I_s) obtained by LV1 level and the damage observation.

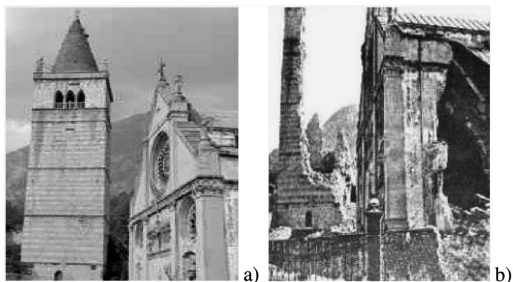


Figure 1. S. M. Assunta Church – Gemona (UD): a) after 1976 earthquake, b) before 1976 earthquake – (Doglioni *et al.*, 1994).

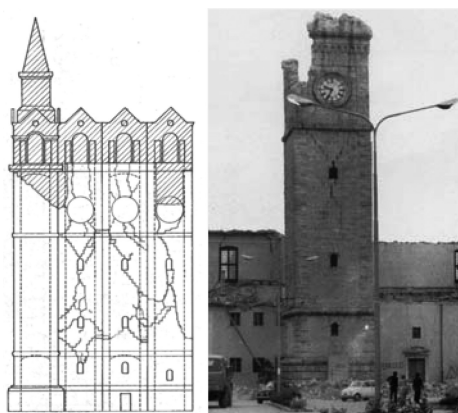


Figure 2. SS. Pietro e Paolo Church – Maiano (UD): demolished after the seismic event – (Doglioni *et al.*, 1994).



Figure 3. S. Stefano Church – Cavazzo Carnico (UD): a) damage after 6th May 1976 earthquake; b) damage after 15th September 1976 earthquake – (Doglioni *et al.*, 1994).

For the well researched bell towers, the information about the damage increase caused by the second main shock has been deducted through the analysis of the damage patterns or about the photographic documentation (Figure 3).

3 SIMPLIFIED MECHANICAL MODELS FOR SEISMIC VULNERABILITY ANALYSIS OF BELL TOWERS

The simplified model proposed by *Guidelines* is based on the assumption that the towers are structures with cantilever-like behaviour. The investigation of 31 bell towers has been performed comparing the design moment with the ultimate moment and assuming both the masonry as non tensile-strength material and an opportune nonlinear compression distribution.

The different structures analysed were divided into n sectors with uniform geometric characteristics and the checks were performed in correspondence to each section change. When structures have squared rectangular sections, in the hypothesis that normal forces are not superior to $0.85 \cdot f_d \cdot a \cdot s$, the ultimate resistant moment at the base of the same sector may be calculated as:

$$M_{u,i} = \frac{\sigma_{0i} A_i}{2} \left(b_i - \frac{\sigma_{0i} A_i}{0.85 a_i f_d} \right) \quad (1)$$

where:

- $M_{u,i}$ is the moment which corresponds to collapse due to combined forces of the section being analysed;
- a_i is the perpendicular side of the direction of the seismic action considered in the same section being analysed, depurated of any eventual openings;
- b_i is the side parallel to the direction of the seismic action considered in the analysed section;
- A_i is the total area of the section under analysis
- σ_{0i} is the average normal tension of the section being analysed;
- f_d is the design compression strength of the masonry opportunely reduced in relation to the knowledge level reached. In this analysis, the impossibility to obtain an adequate level of knowledge has determined the adoption of confidence factor equal to 1.35 (maximum value proposed in *Guidelines*)

For each section, on the basis of the ultimate moment value (capacity) estimated, the adopted model provides the relative peak ground acceleration (demand) that caused this strain. The minimum value of peak ground acceleration among those obtained for different sections analysed represents the acceleration corresponding to ultimate limit state for the bell tower (a_{ULS}). Known the a_{ULS} , it is possible to obtain a safety index (I_s) for the tower, calculated in terms of the site hazard (a_g):

$$I_s = \frac{a_{ULS}}{\gamma_1 \cdot S \cdot a_g} \quad (2)$$

where: a_{ULS} is the value of peak ground acceleration which corresponds to the ultimate limit state; γ_1 is the

significance coefficient; S is the factor which takes into account the stratigraphic profile of the soil and any eventual morphological effects; a_g is the reference peak ground acceleration of the site. A safety index (I_s) greater than 1 indicates that the building is able to sustain the seismic action forecast for that area.

Although the safety index should be calculated taking into account the cultural relevance (factor of importance γ_1) of the building (different probability of exceeding the limit in 50 years), such factor has not been used, since the aim was to evaluate the effect of a previous seismic event. For the same reason the factor S was considered equal to 1 for all the towers analysed.

In this study the value of acceleration corresponding to the seismic intensity assigned to each Municipality after the earthquake is defined using the intensity-PGA correlation proposed by Guagenti and Petrini (1989). This equation has been developed from Italian data and makes reference to MCS intensity.

$$\ln a_g = 0.602 \cdot I_{MCS} - 7.073 \tag{3}$$

where a_g [g] is the peak ground acceleration and I_{MCS} the macroseismic intensity.

In the Table 2, the list of safety index (I_s), in ascending order, for the analyzed sample is reported. In the last two columns, moreover, it is shown the damage level observed after 1976 earthquakes for the bell towers and the belfries, according to the European Macroseismic Scale (Grunthal *et al.*, 1998). As previously mentioned the evaluation of the damage level has been performed through the analysis of the available data, in order to deduct the damage increase after the second main shock. In the cases in which this information was not clearly appraised, the damage level was not defined.

The result analysis allows some immediate remarks. Firstly, we can notice that the safety index, in all the bell towers analysed is greater than 1. This result is in contrast with the observed damage. The comparison with the damage level highlights how the safety index is greater than 1 although many structures are collapsed (i.e. Ch. 12, Ch. 21, Ch. 213).

This evident incongruence is implicit in the simplified method proposed by the *Guidelines*, that assimilates the seismic behaviour of a bell tower to a cantilever constrained at the base. The observation of the collapse mechanisms puts in evidence different kinds of kinematism associated, mainly, to local structural vulnerabilities that rarely are connected to an axial compression and bending phenomenon.

Finally, for all the 31 bell-towers, the section with the minimum ultimate moment is always at the base of towers, although in same situations (i.e. Ch. 200, Ch. 227) the belfry is the more severe damaged part of the tower.

Table 2. Value of peak ground acceleration which corresponds to the ultimate limit state, seismic safety index I_s for each bell towers and damage level observed after 1976 earthquake.

Code	I_{MCS}	a_g	a_{ULS}	I_s	Damage level	
		m/s ²	m/s ²		Bell tower	Belfry
Ch228	8.5	1.39	1.42	1.02	—	—
Ch16	9	1.87	1.94	1.04	3	3
Ch216	9.5	2.53	3.07	1.21	—	—
Ch190	8.5	1.39	1.90	1.37	4	0
Ch31	9.5	2.53	3.52	1.39	2	1
Ch192	9.5	2.53	3.53	1.39	—	—
Ch147	9	1.87	2.70	1.44	4	0
Ch201	9.5	2.53	3.70	1.46	3	3
Ch202	8.5	1.39	2.23	1.61	5	5
Ch33	8	1.03	1.67	1.63	3	4
Ch12	9.5	2.53	4.20	1.66	5	5
Ch21	8.5	1.39	2.43	1.75	5	5
Ch213	8.5	1.39	2.49	1.79	4	5
Ch236	8.5	1.39	2.55	1.84	3	0
Ch240	9	1.87	3.52	1.88	—	—
Ch200	9	1.87	3.63	1.94	2	5
Ch207	9	1.87	3.71	1.98	5	5
Ch231	8	1.03	2.03	1.98	—	—
Ch41	8.5	1.39	3.00	2.16	4	5
Ch212	8.5	1.39	3.01	2.17	3	4
Ch235	8.5	1.39	3.05	2.20	3	3
Ch181	8	1.03	2.33	2.27	—	—
Ch45	8.5	1.39	3.34	2.41	—	—
Ch18	8.5	1.39	3.95	2.85	1	2
Ch209	9	1.87	5.85	3.12	3	2
Ch226	7.5	0.76	2.56	3.38	—	—
Ch233	8	1.03	3.55	3.45	2	1
Ch39	7.5	0.76	2.72	3.58	1	0
Ch229	8.5	1.39	5.24	3.78	3	1
Ch227	7.5	0.76	2.93	3.86	2	4
Ch222	6.5	0.42	2.68	6.44	—	—

4 VULNERABILITY ANALYSIS FOR THE BELL TOWER AND BELFRY MACRO-ELEMENTS

In order to define a new safety index model, a new methodology based on a macroseismic approach, is proposed, similarly to the LV1 model adopted by the *Guidelines* for the churches. The approach is based on a detailed analysis of the structure, focusing the attention on those typological and constructive characteristics that can affect the seismic behaviour (positively or negatively). In particular, in reference both to the specific vulnerability of the tower (poor masonry quality; lack of well-connected diaphragms, the presence of pushing elements, etc.) and to the presence of structural element that can be considered a-seismic measure (tie-rods, buttresses, etc) a vulnerability index can be calculated. This index has

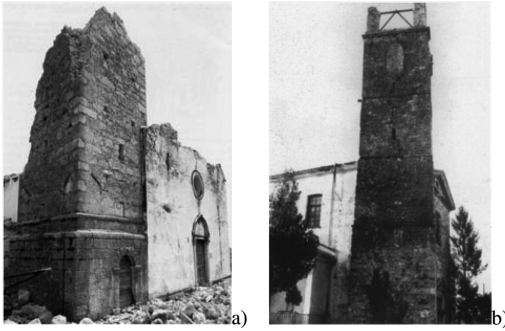


Figure 4. a) S. Stefano Church (Ch21) – Pinzano (PN); b) S. Stefano Church (Ch213) – Nimis (UD) – (Doglioni *et al.*, 1994).

only qualitative meaning; nevertheless wide accessibility (1997 Umbria and The Marches earthquake; 2002 Molise earthquake; 2004 Lombardy earthquake) has allowed us to define a correlation between the observed damage and the macroseismic intensity for different value of vulnerability index.

The vulnerability curve, defined for the churches by Lagomarsino and Podestà (2004b), is represented by the following function:

$$\mu_D = 2.5 \cdot \left[1 + \tanh \left(\frac{I + 3.4375 \cdot i_v - 8.9125}{Q} \right) \right] \quad (4)$$

where μ_D is the mean damage grade of damage distribution, I is the macroseismic intensity i_v the vulnerability score and Q (equal to 3 for churches) is connected to the function slope and represents a ductility factor. This function is analogous to the one proposed by Giovinazzi and Lagomarsino (2004) for ordinary buildings with reference to the vulnerability index (V_I). An analogous function was studied by Curti *et al.*, (2006) in reference to bell-towers and belfries through a statistical analysis of the information gathered following the earthquakes in Friuli (1976), Umbria and the Marches (1997), Molise (2002) and Lombardy (2004).

In general, the free-parameters of the vulnerability curves are V_I and Q , as it is possible to notice:

$$\mu_D = 2.5 \cdot \left[1 + \tanh \left(\frac{I + 6.25 \cdot V_I - 13.1}{Q} \right) \right] \quad (5)$$

where V_I is the vulnerability index.

In order to use the same general expression for all the typology a correlation between the two measures of vulnerability is then proposed (Lagomarsino and Podestà, 2004b).

$$V_I = 0.67 + 0.55 \cdot i_v \quad (6)$$

Table 3. Vulnerability curve parameters for belfries and bell towers.

Macroelement	V_I	Q
Tower	0.89	2
Belfry	0.94	1.49

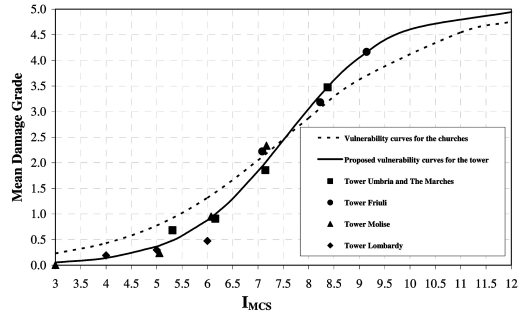


Figure 5. Vulnerability curve of bell towers.

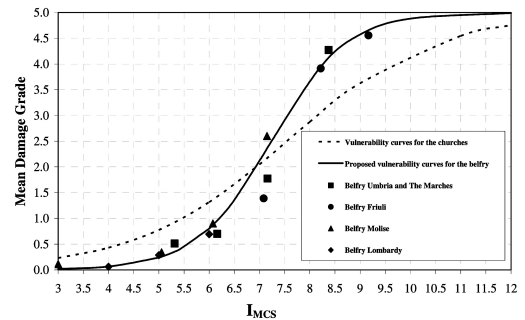


Figure 6. Vulnerability curve of belfry

In Table 3 the parameter values, that distinguish the vulnerability curves for the bell-towers and belfries, are reported (Curti *et al.*, 2006). In Figure 5 both the vulnerability curves are shown.

The use of the vulnerability curve allows, known the vulnerability index and a prefixed damage limit state, to obtain the correspondence macroseismic intensity value and therefore the peak ground acceleration.

In reference to the 31 bell towers analyzed, the vulnerability of the towers and of the belfries was studied using the Appendix C of *Guidelines* (Form for evaluating the seismic vulnerability in religious building): damage mechanisms 27 and 28.

The vulnerability has been defined (for bell tower and belfry) adopting the same function proposed for the churches:

$$i_v = \frac{1}{6} \cdot (v_{ki} - v_{kp}) + \frac{1}{2} \quad (7)$$

Table 4. Vulnerability index, value of peak ground acceleration which corresponds to the ultimate limit state, seismic safety index and damage level observed after 1976 earthquakes.

Code	Bell tower			Belfry			Damage	
	V_i	a_{ULS}	Is (macro)	V_i	a_{ULS}	Is (macro)	Bell Tower	Belfry
Ch201	1.04	0.09	0.354	1.04	0.08	0.296	3	3
Ch41	1.13	0.06	0.457	1.04	0.08	0.541	4	5
Ch190	1.13	0.06	0.457	0.76	0.22	1.523	4	0
Ch202	1.13	0.06	0.457	1.04	0.08	0.541	5	5
Ch212	1.13	0.06	0.457	1.04	0.08	0.541	3	4
Ch213	1.13	0.06	0.457	1.04	0.08	0.541	4	5
Ch235	1.13	0.06	0.457	1.04	0.08	0.541	3	3
Ch236	1.13	0.06	0.457	0.85	0.15	1.078	3	0
Ch16	1.04	0.09	0.478	0.95	0.11	0.565	3	3
Ch147	1.04	0.09	0.478	0.76	0.22	1.127	4	0
Ch207	1.04	0.09	0.478	1.13	0.05	0.284	5	5
Ch12	0.95	0.13	0.499	1.04	0.08	0.296	5	5
Ch216	0.95	0.13	0.499	1.13	0.05	0.210	—	—
Ch21	1.09	0.08	0.538	0.99	0.09	0.637	5	5
Ch31	0.85	0.18	0.705	1.13	0.05	0.210	2	1
Ch192	0.85	0.18	0.705	0.76	0.22	0.834	—	—
Ch33	1.04	0.09	0.873	1.04	0.08	0.731	3	4
Ch181	1.04	0.09	0.873	0.67	0.30	2.905	—	—
Ch231	1.04	0.09	0.873	1.13	0.05	0.518	—	—
Ch18	0.95	0.13	0.912	1.04	0.08	0.541	1	2
Ch45	0.95	0.13	0.912	1.22	0.04	0.271	—	—
Ch209	0.85	0.18	0.953	1.04	0.08	0.400	3	2
Ch240	0.85	0.18	0.953	0.95	0.11	0.565	—	—
Ch39	1.04	0.09	1.179	0.85	0.15	1.969	1	0
Ch226	1.04	0.09	1.179	0.95	0.11	1.395	—	—
Ch228	0.85	0.18	1.287	1.04	0.08	0.541	—	—
Ch229	0.85	0.18	1.287	0.85	0.15	1.078	3	1
Ch200	0.76	0.26	1.345	1.22	0.04	0.201	2	5
Ch233	0.85	0.18	1.739	0.76	0.22	2.057	2	1
Ch227	0.85	0.18	2.350	0.95	0.11	1.395	2	4
Ch222	0.85	0.18	4.291	0.85	0.15	3.595	—	—

where for both mechanisms: v_{ki} and v_{kp} are respectively the points obtained by the vulnerability survey of the vulnerability indicators and by a seismic measures.

Taking into account a mean damage grade equal to 3.8 representative of the ultimate limit state, the corresponding value of macroseismic intensity has been calculated.

Through the correlation between intensity and peak ground acceleration (Guagenti and Petrini 1989), it is possible to define a direct correlation between seismic input and vulnerability. This allows the calculation of ground acceleration value for each macroelement which corresponds to the damage ultimate limit state. The proposed relationships are shown below, respectively for bell towers and belfries:

$$\text{Bell Towers } a_{SLU} = 2.255 \cdot 10^{(0.30-1.63 \cdot V_i)} \quad (8)$$

$$\text{Belfries } a_{SLU} = 2.255 \cdot 10^{(0.22-1.63 \cdot V_i)} \quad (9)$$

In Table 4 the vulnerability indexes, the values of the acceleration corresponding to the ultimate limit state and the safety factors are reported, in reference to the 1976 seismic event.

The correlation between the new seismic safety index and the damage level is more coherent with the collapse mechanisms occurred after the 1976 earthquake. In reference to the tower, it is possible to notice, in fact, that the minor safety indexes are connected to the highest damage level.

The same conclusion is not possible to deduct also for the belfries. In this case there are different trend between damage level and seismic safety index.

This aspect highlights how a macroseismic approach can not be sufficient to completely understand (and, therefore, forecast) the seismic behaviour of belfries, pointing out the need to take into consideration the dynamic parameters (seismic input at the base, filter effect of the structure, dynamic features of the belfry, etc.).

In addition, the macroseismic approach allows to define a correlation between the vulnerability and value of the ground acceleration, which corresponds to the damage limit state (DLS). Considering a mean damage grade equal to 1.5 as representative of the damage limit state, the corresponding value of acceleration has been calculated. The proposed relationship is shown below only for towers:

$$\text{Bell Towers } a_{\text{DLS}} = 2.255 \cdot 10^{(-0.22 - 1.63 \cdot V_i)} \quad (10)$$

In reference to damage limit state, the safety index is less than 1 for all the bell towers analysed.

5 CONCLUSION

The analyses carried out have highlighted how the bell towers (or tall and slender structures) do not show the overall behaviour used to defined the simplified mechanical method proposed by the *Guidelines* for the evaluation of the seismic risk. In these structures, in fact, the wall clamping was, often, limited and in many cases, elements able to create a ring effect (tie-rods, well-connected diaphragms) were not present. The intermediate floors (in general simple wooden structures) had an awful state of maintenance, without tie beams (or punctual connections) able to create a link among the different masonry walls. The steel tie-rods, normally used in the historical buildings of a seismic area were not systematically adopted, but localized principally only in the belfry. The lack of an overall behaviour determines damage patterns ascribable, on the one hand to the activation local collapse mechanisms and, on the other hand, not referable to crushing collapse. These aspect underlines the need to use mechanical model based on the limit equilibrium analysis for the seismic risk evaluation of the belfries. The advantages of this approach are, however, decreased in order to make a seismic risk analysis at territorial level. A reliable appraisal should, in fact, take into account, for the definition of the seismic input, of the dynamic interaction between the tower and the belfry.

In reference to the towers, the seismic risk evaluation, when the overall behaviour is not guaranteed, can be obtained with the methodology proposed in the paper, analogous to the LV1 method proposed in the *Guidelines* for the churches. The methodology is based on the survey of those constructive and typological characteristics that deeply affected the seismic response of the historical buildings. Through the vulnerability index (directly obtainable from the survey form) and the peak ground acceleration it is possible to calculate a different value of the safety index. This method, applied to the sample of 31 bell towers damaged by 1976 earthquake, has shown, for

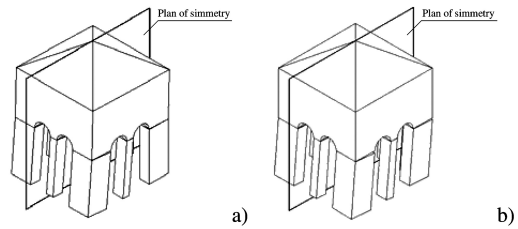


Figure 7. Damage mechanisms collapse (Curti, 2007).

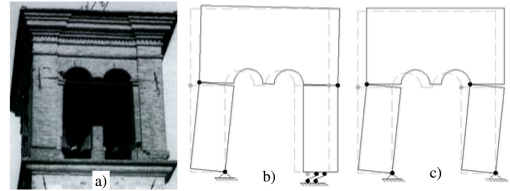


Figure 8. a) damage after 1976 earthquake; b) and c) damage mechanisms collapse (Curti, 2007).

the macroelement tower, a more reliable and realistic prevision of the vulnerability that is, generally, in agreement with the observed damage. For such aspects the authors propose to adopt this approach for these situations in which is not possible to verify the overall behaviour of the tower.

It is worth noticing that a correct evaluation of the seismic safety index has to take into consideration also the vulnerability of the belfry (when present). In these situations, the analysis of the collapse mechanisms can be performed through the equilibrium limit analysis although the methodology is difficult to implement in a territorial risk evaluation.

In reference to this aspect, the case of the S. Marco Church in Spilimbergo (Ch227) is representative. In this instance, the damage level is concentrated, mainly, in the belfry (see Table 3). Although the architectural conformation of the belfry was common to many other similar structures, a priori individuation of the local damage mechanism it was difficult (Figures 7 and 8). This feature, as already mentioned, makes difficult to use this approach in a simplified evaluation procedure.

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