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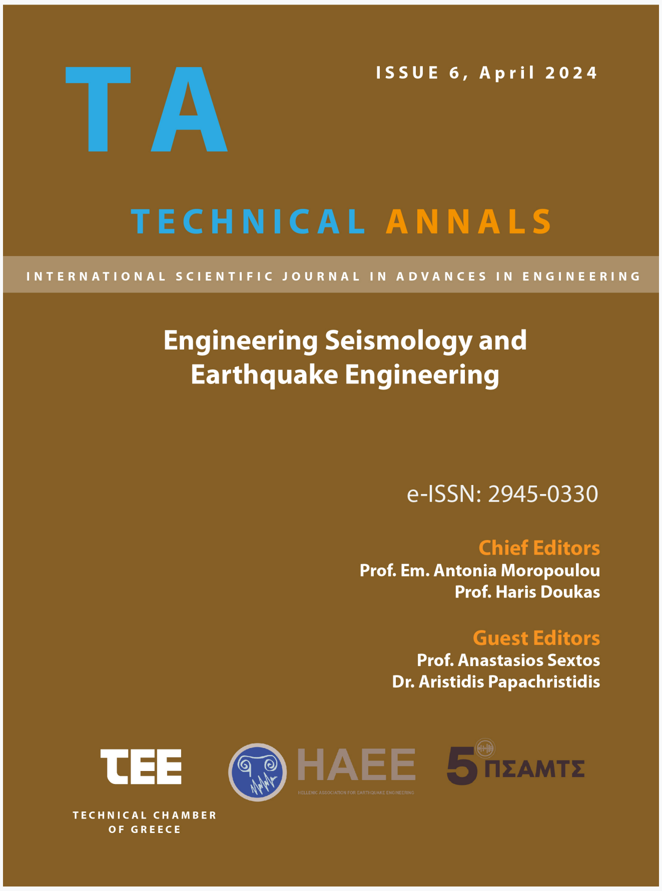

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An approximate method to assess the seismic capacity of existing RC buildings

Lamprini D. Zochiou[1[0009-0006-7182-5837]], Maria D. Panagopoulou[1[0009-0002-2522-6375]] and Stefanos E. Dritsos[1]

[1]University of Patras, Department of Civil Engineering, Aristotelous 26500, Rion, Greece
lamprini.zochiou@gmail.com

Abstract. An approximate method to assess the seismic capacity of existing RC buildings is demonstrated, which is applied to the methodology of the Second-level pre-earthquake inspection according to its recent first revision (2022). This method is validated by comparing the obtained results with the ones of a non-linear static analysis. In particular, the following points are being examined: (a) the effect of masonry infill walls on the seismic resistance of the building and its failure index, taking into account their construction detailing, (b) the effect of unknown information regarding steel reinforcement amounts and (c) the difference in results when the prior to the revision methodology is applied (version of 2018). The results obtained from the examination, show that the values of the failure indices of a building, for the case of known amounts of reinforcement, provided by the Second-level pre-earthquake inspection, were in great convergence with the corresponding ones in terms of acceleration obtained by the non-linear static analysis, for reasonable geometry and location of openings on infill walls. When the maximum failure index of the column elements was considered as the main failure index for the non-linear static analysis, differences in results were observed in some cases.

Keywords: Second-level pre-earthquake inspection, Approximate method, Preliminary seismic analysis, Non-linear static analysis, Building assessment, Reinforced concrete

1 Introduction

For a country like Greece, which faces frequent and often intense earthquake events, it is of great importance for a comprehensive framework of regulations to apply when assessing the seismic capacity of existing buildings before or after a seismic event or other severe destructive causes [9]. This necessity becomes more obvious considering the fact that most of the existing buildings in Greece were constructed before 1984, when the National Earthquake Codes were updated. Therefore, not only are they designed based on older regulations or even with a very low level of seismic design, but in the majority of them, they have already exceeded the intended life span, equal to 50 years for ordinary structures [8].

The assessment of the seismic capacity of an existing structure is a complex and time-consuming process, especially when there are no reliable data on the design and reinforcement amounts and details of the RC structural members as well as the strength of the materials used [11]. Besides, it is important to have a way of prioritizing structures with an, even approximate, assessment of their seismic vulnerability [2].

There are several techniques developed for the damage detection and assessment of existing structures as well as for its structural control. Structural Health Monitoring (SHM) enables for the real-time and early detection, localization and evaluation of damage points or structural degradation, with its non-destructive character and proven accuracy, making it suitable for the assessment and control of various types of structures [1]. Active, passive or hybrid control techniques are designed to improve the performance and stability of structures subjected to earthquake excitation, altering their dynamic response by applying direct or indirect control forces [10]. When combined with advanced sensing and data acquisition systems they can be valuable for assessment purposes, giving real-time information. Destructive testing techniques are often necessary to gain insight on the structure's material properties and failure mechanisms, whereas, numerical modeling and simulations is the most common method to accurately predict damage patterns and progression under various load scenarios.

The seismic assessment is structured into several levels, to avoid time-consuming and often high-cost advanced analysis methods for all potentially vulnerable structures and to allow for a way to prioritize the structures, by classifying them according to their structural vulnerability. The first level refers to a preliminary evaluation of the seismic safety of a building and the determination of those who are in need of a more detailed examination [1]. The Rapid Visual Screening (RVS) procedure as defined in the FEMA P-154 Report, is such a first level methodology, developed to identify potential seismic hazard and to classify the structures into those with acceptable expected seismic performance and those that need to be further investigated [7].

Similarly, the second-level pre-earthquake inspection [5-6] that is applied in the current study, was created specifically for this reason. For existing RC structures, an approximate estimation of their failure index is used as the main criterion, based on the seismic demand, as defined in current assessment provisions. The proposed methodology includes a series of approximate calculations, that can provide the failure index of the structure, without the need to create a detailed numerical model or to use a specific analysis software. It is possible, in fact, for this methodology to be applied, but with less reliability, for the case where there are insufficient data on the structure's reinforcement amount and details as well as the material properties.

In order to validate the reliability of the proposed method of the Second-level pre-earthquake inspection, in this current work, a comparison is made between the results of this method and the corresponding results of a non-linear static analysis. The comparison is made using the revised version of the Second-level pre-earthquake inspection [5] and in particular for the following points:

- The effect of the masonry infill walls
- The lack of sufficient information on the reinforcement amounts and details of column elements

It is noted that in this aforementioned revision of the Second-level pre-earthquake inspection regulation (v.2022) [5], the main points that differentiate it from the pre-revised version (v.2018) are the following:

- The addition of the contribution of infill walls to the seismic resistance of the structure
- The use of higher values for the behavior factor q

2 Description of the case study RC buildings

Both of the structures that are being examilned consist of frame RC load-bearing structural system and were built before 1984. The first structure has a symmetrical rectangular floor-plan (building A) and the second one is non-symmetrical Γ-shaped (building Γ) with structural floor-plans as shown in Fig.1. Both buildings are three-storey with a floor height of 3.00m and accessible roof. In order to examine the effect of masonry infill walls, different conditions of their participation are considered. More specifically, they are being categorized according to the quality of their construction detailing and wedging and the existence or not of openings on them. The presence of openings is considered for the case that their size and location reduce the contribution of the infill walls in earthquake resistance to 50%. Table 1 and 2 briefly present the cases to be further examined.

Table 1. Different cases of structural systems and masonry infill walls contribution

Building A	Building Γ	Description of case study RC buildings
A_n	Γ_n	Reference buildings without taking into account the contribution of masonry infill walls
A_1	Γ_1	Buildings considering presence and contribution of masonry infill walls in all levels without openings
A_0	Γ_0	Buildings considering presence and contribution of masonry infill walls in all levels with openings

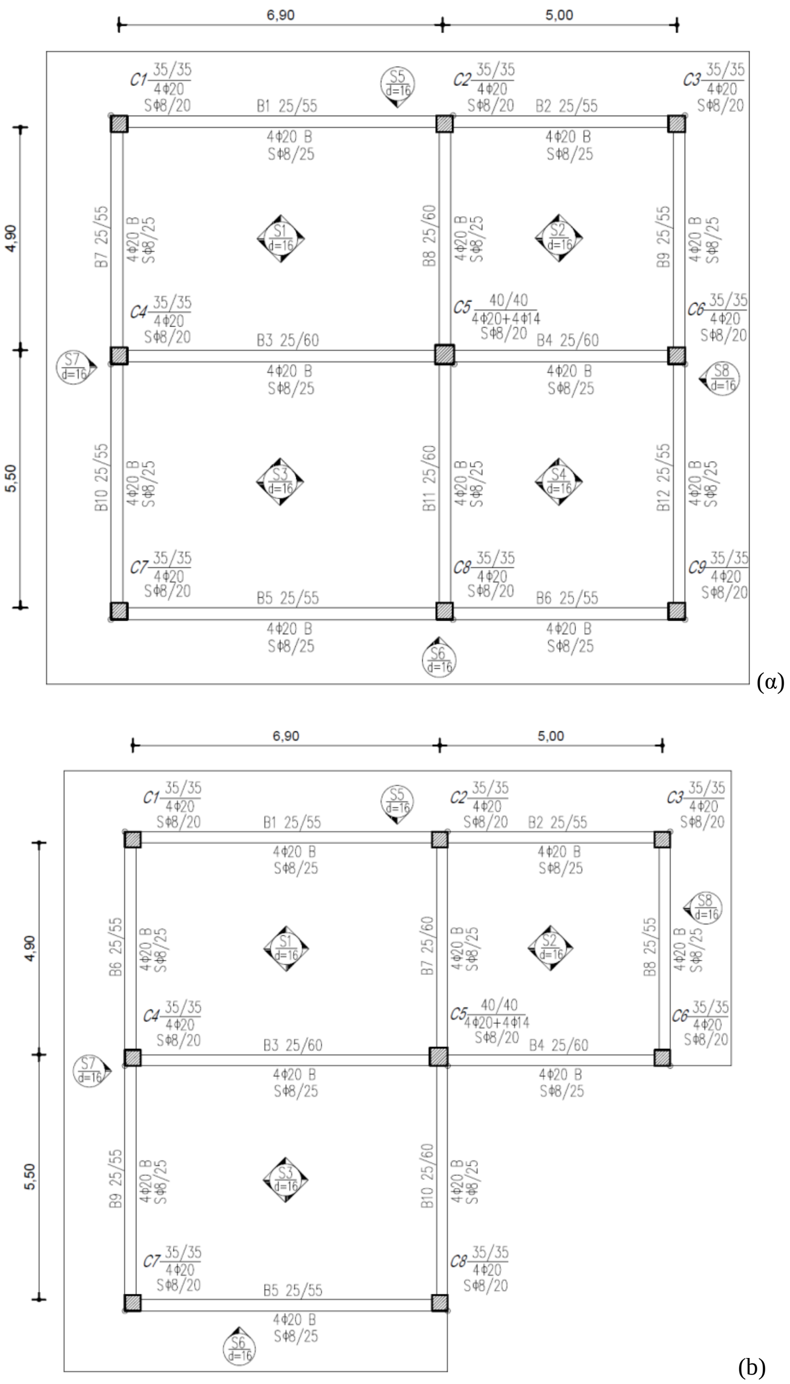


Fig. 1. Structural floor-plans of the two buildings A (a) and Γ (b)

Table 2. Different categories of masonry infill walls

Building A	Description of case study RC buildings
Inf.Wall.1	Good construction detailing and wedging of masonry infill walls without considering the presence of openings
Inf.Wall.2	Good construction detailing and wedging of infill walls considering the presence of openings
Inf.Wall.3	Poor construction detailing and wedging of masonry infill walls without considering the presence of openings
Inf.Wall.4	Poor construction detailing and wedging of infill walls considering the presence of openings

2.1 Reinforcement amount and details, Loads and Design Spectrum

The columns located on the perimeter of the building have a cross-section of 35x35cm and the one located at the center has a cross-section of 40x40cm. External beam elements have a cross-section of 25/55 and have 4Φ16 bottom reinforcement at mid-span from which 2Φ16 are bent at the supports. Internal beam elements have a cross-section of 25/60 and have 4Φ20 bottom reinforcement at mid-span from which 2Φ20 are bent at the supports. All beam elements have 2Φ8 top reinforcement which does not participate in moment resistance at the supports, due to insufficient anchorage length. Columns located on the perimeter are reinforced with 4Φ20 at the corners and the one located at the center of the building is reinforced with 4Φ20 at the corners and 4Φ14 (1Φ14 in the middle of each side). Ties are rectangular Φ8/20 in all column elements and Φ8/25 in all beam elements with adequate anchorage. The thickness of the slabs is taken equal to 16cm.

Material properties are considered as follows assuming Data Reliability Level (DRL) to be "Sufficient" [4]: the average value for the compressive strength of concrete is considered 18 MPa and characteristic value 14 MPa, whereas the corresponding values of the tensile strength of reinforcement bars and ties are 460 MPa and 400 MPa respectively. The dead loads (G) of the structure include the self-weight of the RC elements ($25\ \mathrm{kN/m^3}$), the floor toppings ($1.3\ \mathrm{kN/m^2}$) and the outer and inner masonry walls ($3.6\ \mathrm{kN/m^2}$ and $2.0\ \mathrm{kN/m^2}$ respectively). The live loads (Q) include the ones in the floors and roof ($2.0\ \mathrm{kN/m^2}$). Axial loads at the base of the ground floor columns were calculated by considering the effective slab areas for each column, dividing the slabs in triangular and trapezoidal subareas, for the $\mathrm{G} + 0.3\mathrm{Q}$ loading combination.

The seismic loads (E) were calculated in accordance with the EC8 design spectrum [3], with a ground acceleration equal to $\mathrm{a_g} = 0.24\mathrm{g}$, (where g denotes the acceleration due to gravity, 9.81 m/sec2), soil type B (medium dense sand or stiff clay) and seismic zone II. For the Second-level pre-earthquake inspection the soil index is taken equal to 1.00 and the design spectrum is used with behavior factor q equal to 2.00. For the non-linear static analysis the elastic spectrum is used and the soil index is considered equal to 1.20.

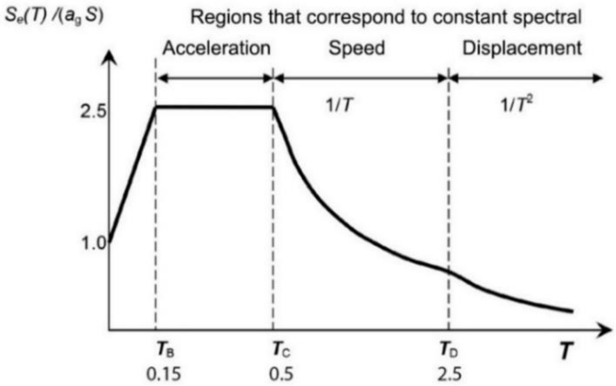


Fig. 2. Elastic spectrum

2.2 Dynamic characteristics

When applying the methodology of the Second-level pre-earthquake inspection, the period T of the building was determined according to the approximate equation from the Greek Code of Structural Interventions - *KANEPE* [4] as following:

$$T = C_t\, H^{0.9} \tag{1}$$

where C_t is equal to 0,052 and H is the height of the building equal to 9.90 m.

When conducting the non-linear static analysis, the period T was determined based on the direction and distribution of the seismic loading and the structural properties of the structures. Table 3 presents the results that were obtained by the two methods.

Table 3. Empirical and analytical periods T

	Second-level pre-earthquake inspection	Non-linear static analysis	
		A	Γ
No infill walls	0.41	1.36	1.32
Inf.Wall.1	0.41	0.61	0.58
Inf.Wall.2	**0.41**	**0.71**	**0.75**
Inf.Wall.3	0.41	0.72	0.68
Inf.Wall.4	**0.41**	**1.06**	**0.78**

It is worth mentioning that the period T obtained by the approximate equation from *KANEPE* [4], is equal for both structures with values much smaller than those resulting from the non-linear static analysis. In the analysis, periods in all cases correspond to values higher than $T_c = 0.50s$, i.e. they are in the descending branch of the spectrum. Consequently, a lower demand is expected compared to the one Second-level pre-earthquake inspection predicts, where period values correspond to the plateau of the spectrum ($T < T_c$) which means that the seismic demand will be higher.

3 Application of the approximate method

In the Second-level pre-earthquake inspection, the seismic demand, V_{req}, is determined for each direction of the earthquake according to the design spectrum as follows:

$$V_{req} = M\,S_d(T) \tag{2}$$

where, M is the total mass of the structure and is calculated by the sum of the axial loads at the base of each column:

$$M = N_{tot}/g \tag{3}$$

and $S_d(T)$ is the design spectrum acceleration at period T:

$$S_d(T) = a_g\,S\left(\frac{2.5}{q}\right) \tag{4}$$

Analysis is performed for the “Significant Damage Performance” level (Level B) [4] and the behavior factor q is taken equal to 2.00 according to Table 4 of the methodology of the Second-level pre-earthquake inspection [5]. As observed, proposed values for the factor q, for structures built before 1984, are slightly higher than the corresponding ones proposed by the methodology of *KANEPE* (Table S4.4 [4]).

Table 4. Proposed values of the behavior factor q for “performance level B – Severe damage” [5] according to the Second-level pre-earthquake inspection.

Standards applied for design (and construction)	Favourable presence or absence of infill walls (1)		Generally unfavourable presence of infill walls (1)	
	Substantial damage in primary elements		Substantial damage in primary elements	
	No	Yes	No	Yes
1995<…	3.0	2.3	2.3	1.7
1985<…<1995(2)	2.3	1.7	1.7	1.3
…<1985	1.7	1.3	1.3	1.1

For each earthquake direction, the value of the basic seismic resistance (V_{R0}) of the members of the critical floor (generally the ground floor) is determined using the following Eq.5:

$$V_{R0} = \alpha_1 \sum V_{Ri}^{RC\ col.} + \alpha_2 \sum V_{Ri}^{RC\ walls} + \alpha_3 \sum V_{Ri}^{RC\ short.col.} + \sum V_{Ri}^{infill\ walls} \quad (5)$$

where for this particular case (frame structural system without RC walls and without short columns) the values are considered according to the proposed method as $\alpha_1 = 0.85$ and $\alpha_2 = \alpha_3 = 0$.

According to the proposed methodology, the maximum shear force that elements can carry is determined by checking their failure mechanism (shear or flexural) as defined at Eq.6 in which V_{Rd} is the shear strength and V_M is the flexural strength of the element. When data for reinforcement amounts are not available, the method is applied by assuming that $V_{R,i} = V_{Rd}$. In this paper, results are given for both cases (Second-level pre-earthquake inspection with and without reinforcement information).

$$V_{R,i} = \min\left(V_{Rd}, V_M\right) \quad (6)$$

The value of the shear force V_{Rd} is determined using the equations from Appendix 7C of *KANEPE* [4]. When no data for reinforcement amounts are available, values of μ_θ^{pl} and x, are being calculated in an approximate way. In this paper, according to the data presented in §2.1 the value $\mu_\theta^{pl} = 2.5$ was considered (Second-level pre-earthquake inspection [5]). The height of the compressive zone was taken equal to $x = 0.35d$ which results from the approximate equation of the curvature adopted by *KANEPE* (Appendix 7A [4]) as follows:

$$\varphi_y = \frac{1.52\ f_y}{E_S\ d} \quad (7)$$

and the mechanics of materials equation:

$$\varphi_y = \frac{\varepsilon_S}{d - x} = \frac{f_y}{E_S\ (d - x)} \quad (8)$$

Resulting to:

$$x = d - \frac{d}{1.52} \cong 0.35\ d \quad (9)$$

In order to examine the effect of the aforementioned estimations regarding x and μ_θ^{pl}, a comparison was made between the shear resistance values of the column elements and the corresponding results of the non-linear static analysis (with known data about reinforcement amounts and details). The results are presented in Table 5 and it is worth mentioning that their convergence is great.

Table 5. Shear resistance of RC columns – Approximate method and analysis

Second-level pre-earthquake inspection				
	BUILDING A		BUILDING Γ	
	Reinf. data	No reinf. data	Reinf. data	No reinf. data
COLUMNS	V_R (KN)	V_R (KN) $x = 0.35d$, $\mu_\theta^{pl} = 2.5$	V_R (KN)	V_R (KN) $x = 0.35d$, $\mu_\theta^{pl} = 2.5$
1	107.31	108.35	107.31	108.35
2	126.11	133.52	126.50	134.32
3	100.40	101.94	104.75	105.94
4	126.22	133.76	126.70	134.73
5	185.46	173.30	177.57	181.45
6	122.10	126.27	98.06	99.84
7	109.16	110.12	115.09	115.93
8	127.11	135.58	108.05	109.06
9	102.13	103.52	-	-

The shear resistance of the masonry infill walls $V_{Ri}^{\text{infill walls}}$ is calculated by the following Eq.10:

$$V_{Ri}^{\text{infill walls}} = 0.3\, f_{wc,s}\ t_w\ b_w \left(\frac{l}{L}\right) \qquad (10)$$

where $f_{wc,s}$ is the compressive strength of the infill walls in the diagonal direction and can be obtained from Table 3 of Appendix D [6], whereas t_w b_w stands for the thickness and the effective width of the infill wall respectively. §7.4.1. describes how to take into account the contribution of infill walls in the resistance of the structure. Approximately, b_w can be considered as $b_w \approx L \cdot (f_{wv}/f_{wc,s})$, where f_{wv} is the strength in diagonal cracking.

The final seismic resistance, V_R , is defined for each main direction by the following Eq.11:

$$V_{R,x} = \beta_x \times V_{R0,x} \quad , \quad V_{R,y} = \beta_y \times V_{R0,y} \qquad (11)$$

The reduction factor β is determined based on 13 criteria, each one of which participates with a weight corresponding to its influence on reducing the seismic capacity of the structure and is evaluated with a value of 1 to 5, where 1 corresponds to the highest reduction.

For the case of the two structures that are being examined on this paper, criterion 3 (normalized axial load) was graded equal to $\beta_3 = 3$ for both buildings A and Γ. These values were obtained because in building A, the maximum value of the axial load in a column element was $0.40 \leq v_d^i = 0.402 < 0.50$, whereas in building Γ, the average value of the normalized axial load of column elements was $0.25 \leq v_d = 0.275 < 0.35$. Criterion 5 (the stiffness distribution in plan-torsion), for building Γ only, was graded with $\beta_5 = 4$, given the fact that the normalized eccentricity for x direction was $\varepsilon_x = 0.14 > 0.05$, which corresponds to grade 4 and for y direction was $\varepsilon_y = 0.03 < 0.05$, which corresponds to grade 5. In all other cases the criteria were graded with $\beta_i = 5$. As a result of the above, values of the reduction factor for buildings A and Γ were $\beta_x = \beta_y = 0.98$ and $\beta_x = 0.96$, $\beta_y = 0.98$ respectively.

The failure index λ of the structure for each main direction x and y is calculated by the available seismic resistance and the seismic demand according to Eq.12 as follows:

$$\lambda_x = \frac{V_{req,x} + 0.30V_{req,y}}{V_{R,x} + 0.30V_{R,y}} \ , \lambda_y = \frac{V_{req,y} + 0.30V_{req,x}}{V_{R,y} + 0.30V_{R,x}} \tag{12}$$

4 Application of the non-linear static analysis method

A non-linear static analysis is also employed, in accordance with the provisions of *KANEPE* [4] for a "performance level B".

In this paper, the failure indices of the buildings are being defined in two ways. In the first way, based on the minimum horizontal ground acceleration for which first failure occurs for an acceptable level performance B and for all possible loading combinations. The failure index is calculated by the following Eq.13 as:

$$\lambda_{ag} = \frac{a_{g,ref}}{\alpha_g} \tag{13}$$

where $a_{g,ref}$ is the reference horizontal ground acceleration, which in this case equals to 0.24g, with a probability of exceeding the seismic action of 10% in the structure's intended life span, which equals to 50 years for ordinary structures.

In the second way, through the maximum failure index of column elements for a "performance level B1" and for all possible loading combinations.

5 Results comparison

5.1 Seismic Resistance obtained by Second-level pre-earthquake inspection and Non-linear static analysis

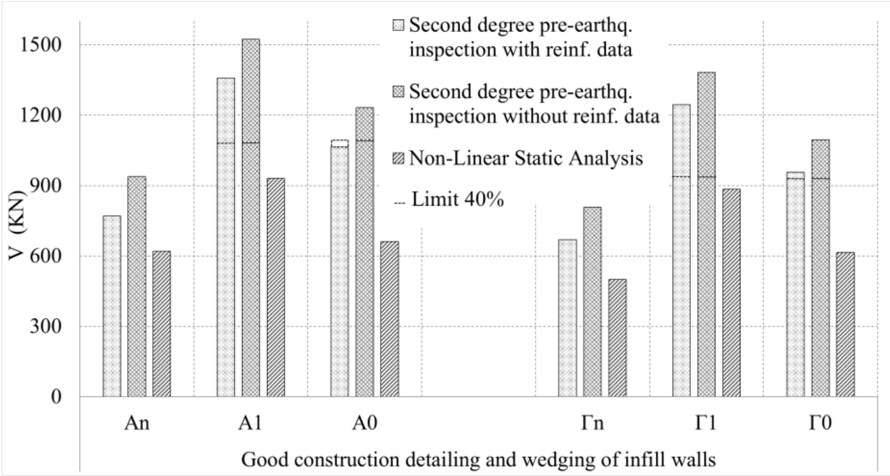
Table 6 shows the results of the seismic resistance in terms of base shear force of the buildings examined in this paper. The results obtained by applying the methodology of Second-level pre-earthquake inspection are presented for both cases mentioned above, i.e. for the case that there are available data about reinforcement amounts of the column elements and for the case that they are not available. Regarding the non-linear static analysis, the value of the shear force presented on Table 6 is the maximum value obtained by the capacity curve of the structure, which appears before or during the point when the structure reaches "performance level B".

Table 6. Maximum Seismic Resistance.

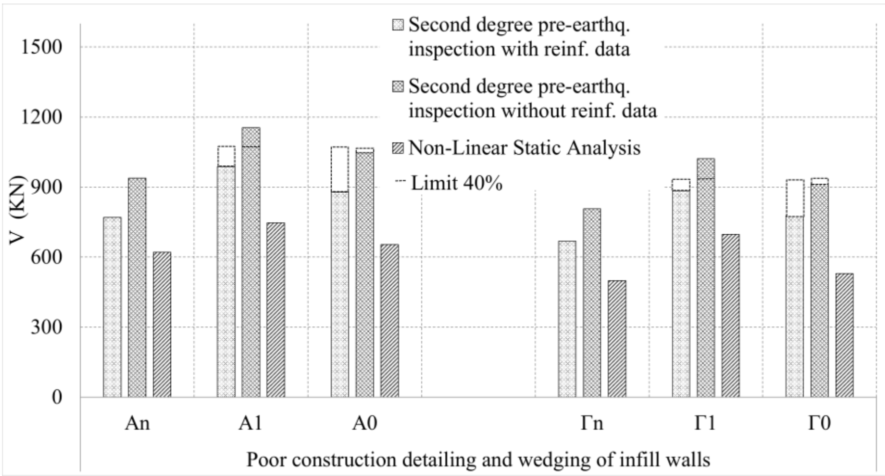
BUILDING A			
	Second-level pre-earthq. Insp.–Reinf. data	Second-level pre-earthq. Insp.–No reinf. data	Non-linear static analysis
No Infill Walls	770.39	938.26	620.75
Inf.Wall.1	1358.44	1523.31	930.22
Inf.Wall.2	**1064.42**	**1232.29**	**660.62**
Inf.Wall.3	987.02	1154.89	746.85
Inf.Wall.4	**878.70**	**1046.57**	**654.40**
BUILDING Γ			
	Second-level pre-earthq. Insp.–Reinf. data	Second-level pre-earthq. Insp.–No reinf. data	Non-linear static analysis
No Infill Walls	697.44	807.53	499.91
Inf.Wall.1	1296.75	1382.87	884.71
Inf.Wall.2	**997.10**	**1095.20**	**614.47**
Inf.Wall.3	920.04	1021.23	697.11
Inf.Wall.4	**808.74**	**914.38**	**528.94**

In all cases examined, the failure mechanism determined by applying the Second-level pre-earthquake inspection with known reinforcement data, was found to be flexural, in full agreement with the analysis results for both buildings. Seismic resistance values obtained when the Second-level pre-earthquake inspection was applied without reinforcement data, were higher than those obtained when using reinforcement data. This is reasonable considering that, for this case, only the shear strength of the members is taken into account, as flexural failure, which was the critical one, is not being checked.

Fig.3 presents the results of the seismic resistance of the buildings for good and poor construction detailing and wedging. It is important mentioning that, according to the provisions of Second-level pre-earthquake inspection, the contribution of infill walls to the total seismic resistance must not exceed 40% of the seismic resistance provided by the vertical structural members (i.e. RC columns, RC walls). In Fig.3 this is demonstrated as “Limit 40%”.



(a)



(b)

Fig. 3. Earthquake resistance obtained by Second-level pre-earthquake inspection (with and without considering reinforcement data) and Non-linear static analysis for good (a) and poor (b) construction detailing and weding of infill walls.

It is observed that in all buildings, the maximum seismic resistance (V) provided by the vertical structural elements as well as the contribution of the infill walls, as determined by the Second-level pre-earthquake inspection, always result to slightly higher values than the corresponding ones of the non-linear static analysis. For the case of good construction detailing and wedging of infill walls without openings, the contribution of infill walls was high, to the extent that it exceeded the maximum contribution limit set by the Second-level pre-earthquake inspection (40% of vertical elements resistance). However, this case is not realistic and was considered only for the purpose of investigating the limits of the acceptable contribution of infill walls.

5.2 Failure indices obtained by Second-level pre-earthquake inspection and Non-linear static analysis

Fig.4 and Fig.5 demonstrate the values of the failure indices obtained by the Second-level pre-earthquake inspection [1] for both the cases that reinforcement data are available ($\boldsymbol{\lambda_\Delta}$) and not available ($\boldsymbol{\lambda_{\Delta,v}}$), together with the corresponding values obtained by the non-linear static analysis. For this case, indices are determined in terms of base acceleration ($\boldsymbol{\lambda_{ag}}$) and in terms of maximum failure index ($\boldsymbol{\lambda_{max}}$) for good and poor construction detailing and wedging of masonry infill walls.

As can be observed in Fig.4 and Fig.5, failure indices λ_Δ are in very good convergence with λ_{ag} compared to the failure indices λ_{max} of the columns. In fact, for the common cases of buildings with infill walls with openings, failure indices λ_Δ were in great convergence with the corresponding results of the non-linear static analysis for both buildings and for all the cases that were examined in this study. However, for $\lambda_{\Delta,v}$ lower values were obtained as expected, due to the higher value of the seismic resistance that was calculated for this case (Table 6).

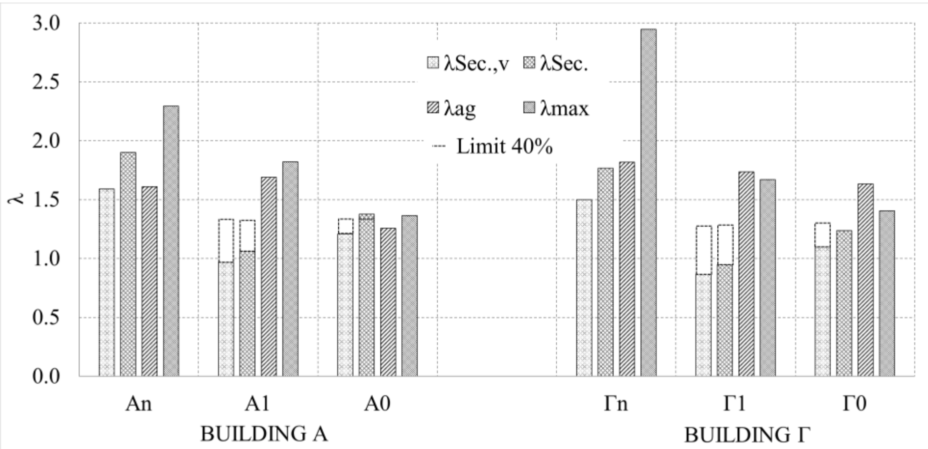


Fig. 4. Failure indices obtained by Second-level pre-earthquake inspection and Non-linear static analysis for good construction detailing and wedging of infill walls.

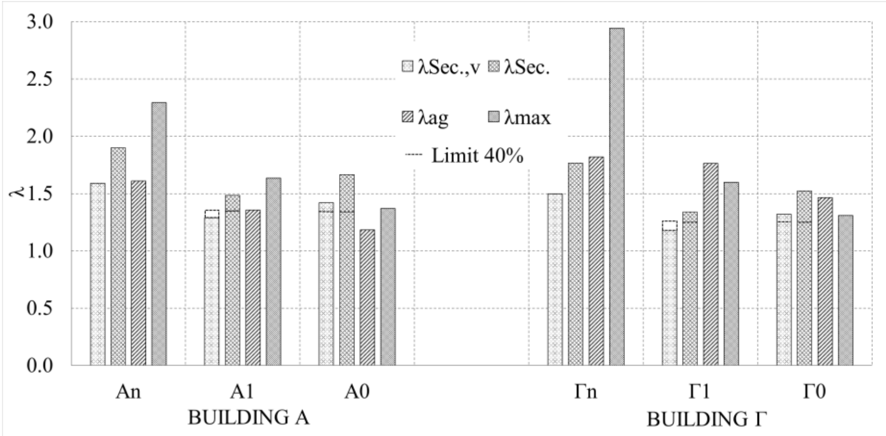


Fig. 5. Failure indices obtained by Second-level pre-earthquake inspection and Non-linear static analysis for poor construction detailing and wedging of infill walls.

In order to evaluate the effect on the results, of the changes made in the recently revised version of the Second-level pre-earthquake inspection, the values of the failure indices obtained by applying both versions of the provisions [5-6] are presented in Table 7 together with the corresponding results from the non-linear static analysis.

As can be observed, the values of the failure indices obtained by the methodology of the pre-revised version [6] are always quite higher than the corresponding ones obtained by the methodology of the revised version [5], are independent of the presence of masonry infill walls and deviate more from the results of the non-linear static analysis. The main reason for these differences is that in the pre-revised version [6]: (a) lower values are used for the behavior factor q (which for the cases examined is considered equal to $\mathbf{q = 1.7}$ instead of $\mathbf{q = 2.0}$ that is dictated by the revised version) and (b) the contribution of masonry infill walls to the seismic resistance of the structure is ignored.

Table 7. Failure indices λ

Second-level pre-earthquake inspection					
		v.2022 - Reinf. data	v.2022 – No reinf. data	v.2018 – Reinf. data	v.2018 – No reinf. data
BUILDING A	No Infill Walls	1.90	1.56	2.24	1.84
	Inf.W.1	1.08	0.96	2.24	1.84
	Inf.W.2	**1.38**	**1.19**	**2.24**	**1.84**
	Inf.W.3	1.48	1.27	2.24	1.84
	Inf.W.4	**1.67**	**1.40**	**2.24**	**1.84**
BUILDING Γ	No Infill Walls	1.76	1.46	2.07	1.72
	Inf.W.1	0.95	0.85	2.07	1.72
	Inf.W.2	**1.23**	**1.08**	**2.07**	**1.72**
	Inf.W.3	1.33	1.15	2.07	1.72
	Inf.W.4	1.52	1.29	2.07	1.72

Non-linear Static Analysis			
		λ_{max}	λ_{ag}
BUILDING A	No Infill Walls	2.30	1.61
	Inf.W.1	1.82	1.69
	Inf.W.2	**1.37**	**1.26**
	Inf.W.3	1.64	1.36
	Inf.W.4	**1.37**	**1.18**
BUILDING Γ	No Infill Walls	2.95	1.82
	Inf.W.1	1.67	1.73
	Inf.W.2	**1.41**	**1.63**
	Inf.W.3	1.60	1.76
	Inf.W.4	**1.31**	**1.47**

6 Conclusions

In this present study, a comparison was made between the results of the approximate methodology described in the Second-level pre-earthquake inspection of RC buildings [5-6] and the corresponding ones obtained from a non-linear static analysis in order to validate the reliability of the approximate method. This method, because of its approximate nature, simplifies the procedure of estimating the seismic capacity of an existing RC structure. This methodology was applied for both the cases where reinforcement data for the vertical structural elements are and are not available. Based on the results obtained from this study for both buildings and for all different cases the following conclusions can be drawn. It is out of the question that further research is needed to be conducted by testing different types of buildings so that solid and safe conclusions can be drawn for a wider range of structures:

- The failure indices λ of the Second-level pre-earthquake inspection, when reinforcement data of columns are available, were in great convergence with the corresponding results in terms of base acceleration obtained by a non-linear static analysis for the cases where the infill walls had a reasonable size and location of openings. When the maximum column failure index was used as the main failure index of the non-linear analysis, case-by-case differences were observed.
- When applying the methodology of the Second-level pre-earthquake inspection, a greater seismic demand is estimated compared to the one resulting from the non-linear static analysis, due to the lower value of the period obtained by the empirical equation (Table 4), but also a relatively greater seismic resistance (Table 6). Thus, despite these discrepancies, the values of the failure indices λ obtained from both methods have eventually a good convergence.
- The contribution of masonry infill walls to the seismic resistance of the buildings was in all cases higher than the one that was determined by the non-linear static analysis. The deviation was significant when infill walls were of high resistance and without openings.
- In the buildings examined, the failure mechanism determined by the Second-level pre-earthquake inspection with available reinforcement data, was found to be flexural for all RC columns, in full agreement with the results of the non-linear static analysis. When the Second-level pre-earthquake inspection was applied for the case that no reinforcement data were available, the failure indices were found to have lower values, but even in this case analysis results were approximated quite satisfactorily.
- The results obtained by the Second-level pre-earthquake inspection according to its recent first revision (v.2022) [5] were in better convergence with the ones obtained by the non-linear static analysis, compared to the results obtained when the pre-revised version was applied (v.2018) [6].

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