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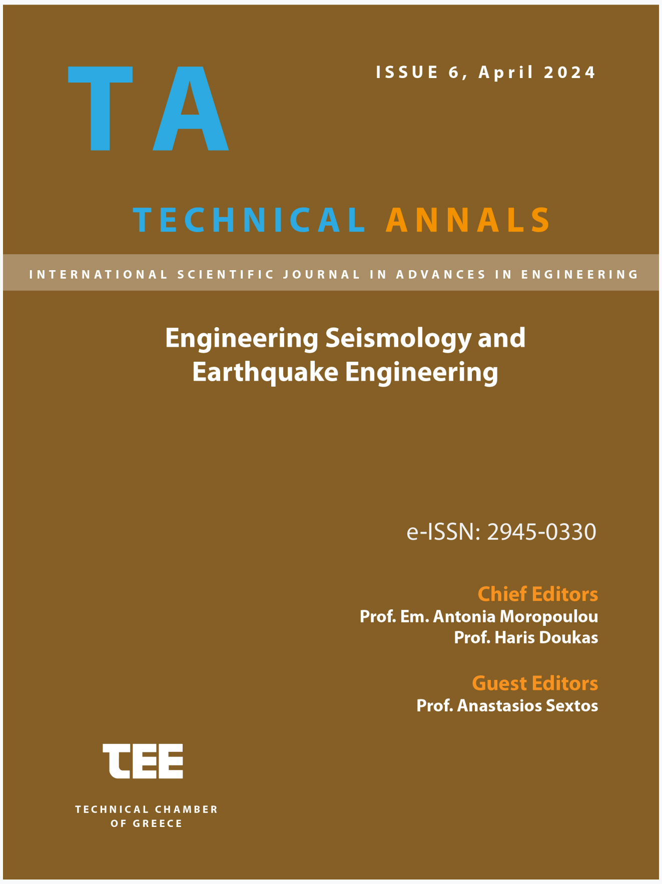
Technical Annals

Vol 1, No 6 (2024)

Technical Annals



Assessment of the seismic capacity of existing RC buildings short columns or soft storey, in accordance with the Second-Degree Pre-earthquake Inspection

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doi: [10.12681/ta.36868](https://doi.org/10.12681/ta.36868)

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To cite this article:

Panagopoulou, M., & Lamprini Zochiou. (2024). Assessment of the seismic capacity of existing RC buildings short columns or soft storey, in accordance with the Second-Degree Pre-earthquake Inspection. *Technical Annals*, *1*(6). https://doi.org/10.12681/ta.36868

Assessment of the seismic capacity of existing RC buildings short columns or soft storey, in accordance with the Second-Degree Pre-earthquake Inspection

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Abstract. The seismic capacity assessment of two three-storey reinforced concrete buildings constructed prior to 1984 is being conducted in this paper. Specifically, the case of short columns or soft storey on the ground floor is investigated and a comparison is made regarding the seismic response obtained for each case. The approximate method of the Second-degree pre-earthquake inspection is applied for the case of known and unknown reinforcement amounts. The obtained results are then compared to results obtained from non-linear static analysis. The primary criterion for the comparison is the failure index of the buildings, as derived from each method. The buildings are categorized into seismic categories according to the Second-degree pre-earthquake inspection. These are compared with the seismic classifications determined by *KANEPE*. The results of the failure indices and seismic categories according to the Second-degree pre-earthquake inspection were in good agreement with the corresponding results of failure indices in terms of acceleration and the seismic classifications according to *KANEPE* obtained when the non-linear static analysis is applied. This agreement is particularly pronounced when known amounts of reinforcement are considered for the vertical elements. Furthermore, the seismic vulnerability of buildings with soft storey or short columns was confirmed in both methods in a similar manner.

Keywords: Reinforced concrete, Non-linear static analysis, Pushover analysis, Second-degree pre-earthquake inspection, Building assessment, soft storey, short columns, failure index.

1 Introduction

The Second-degree pre-earthquake inspection [1-2] for reinforced concrete buildings (RC) employs a failure index λ as the main criterion for building classification. This failure index is derived from an approximate method based on seismic demand, as defined in current assessment provisions. The methodology involves several approximate estimates without the need of using a detailed analysis model. The method focuses on the strength of the vertical elements of the structure and can be applied, albeit with reduced reliability, even when there is insufficient information about the amount of reinforcement and detailing. In this paper the revised version (2022) of the methodology is applied, whereas results from the application of the method based on the pre-existing version (2018) of [2] are presented in [6].

The capability of rapidly estimating the structural capacity of a building under seismic loads using approximate assessment methods, according to the current design and assessment provisions (Eurocodes, *KANEPE* [3], *KADET* [4]), poses a challenge. Therefore, the critical question, concerns the reliability level of the results obtained from this method. In the study of Zochiou et al. [5], a preliminary answer is attempted within the framework of investigating a group of RC buildings. In [5] the main objective of the study was to assess the influence of masonry infill walls and the absence of data regarding the reinforcement of the columns.

In this study, the investigation focuses on the influence of the presence of short columns or a soft storey (pilotis), and for comparison purposes, the building types remain the same as in the first study [5]. In both studies, the reliability check of the results of the approximate Second-degree pre-earthquake inspection method is conducted based on comparison with the results of a non-linear static analysis (pushover analysis). The comparison is made according to the recent revision of the Second-degree pre-earthquake inspection (2022) [1]. Specifically, in this study, the following aspects are investigated:

- The effect of the presence of short columns in the structure.
- The impact of the presence of a soft story (pilotis) on the ground floor of the building.
- The effect of applying the methodology without sufficient information on the reinforcement amounts and details of vertical elements.

It should be noted that in the aforementioned revision of the Second-degree pre-earthquake inspection [1], there were fundamental changes in the way the failure index λ is determined compared to the original text [2]. The original text did not consider the contribution of masonry infill walls to the seismic resistance of the building, and the adopted values for the behavior factor q were lower. Thus, in order to assess the impact of the aforementioned changes, the failure indices λ are determined based on the original text [2], and the respective results are compared. Similar to [5], two typical RC buildings with different plan view layout are assessed. The assessment is conducted for the “performance level B”.

2 Description of the case study RC building

Two RC frame buildings have been examined which have been constructed prior to 1984. The first structure (building A) has a rectangular floor-plan. The second structure (building B) has an L-shape floor plan. The corresponding structural drawings are provided in Fig. 1. Both buildings have three storeys, with a floor height of 3.00m and accessible roof. Each building is examined under conditions of short columns or a soft storey on the ground floor, assuming the presence of infill walls. To examine the effect of masonry infill walls on both buildings, 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. The cases to be investigated are presented in Tables 1 and 2.

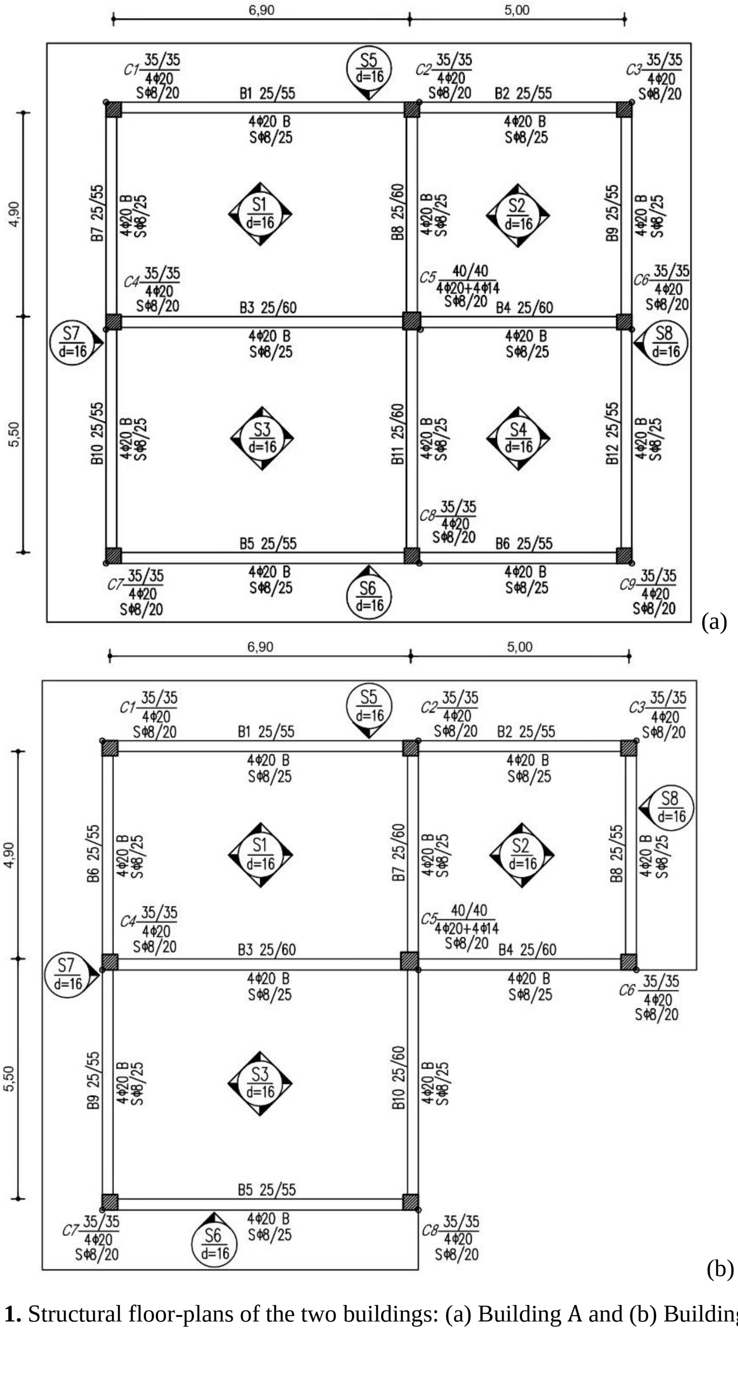


Fig. 1. Structural floor-plans of the two buildings: (a) Building A and (b) Building B.

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

Building A	Building B	Description of the case study RC Buildings
A_n	B_n	Reference buildings without considering the contribution of masonry infill walls
A	B	Reference buildings considering presence and contribution of masonry infill walls with openings in all levels
A_s	B_s	Buildings with short columns at the perimeter of the ground floor at a height of 1.10m considering the contribution of masonry infill walls with openings in the two upper floors
A_p	B_p	Buildings with soft storey (pilotis) on the ground floor considering the contribution of masonry infill walls with openings in the two upper floors

Table 2. Different categories of masonry infill walls.

Inf. Wall.g	Good construction detailing and wedging of masonry infill walls considering the presence of openings
Inf. Wall.p	Poor construction detailing and wedging of masonry infill walls considering the presence of openings

2.1 Reinforcement amount and details, Loads and Design Spectrum

The perimeter columns have a cross-section of 35x35cm. The central one has a cross-section of 40x40cm. The perimeter beams have a cross-section of 25/55 with 4Φ16 bottom reinforcement at mid-span, from which 2Φ16 are bent at the supports. The internal beams have a cross-section of 25/60 with 4Φ20 bottom reinforcement at mid-span, from which 2Φ20 are bent at the supports. All beams have 2Φ8 top reinforcement which do not participate in shear resistance at the supports due to their insufficient anchorage length. The perimeter columns are reinforced with 4Φ20 bars at the corners, and the central one with 4Φ20 and 4Φ14 bars (1Φ14 in the middle of each side). The transverse reinforcement is rectangular Φ8/20 for all columns and Φ8/25 for the beams with poor anchorage. The thickness of the slabs is assumed to be 16cm.

The concrete compressive strength was considered with an average value of 18 MPa, and a "quasi" characteristic value was 14 MPa, assuming a "Sufficient" Data Reliability Level (DRL) [3]. The corresponding values for the tensile strength of longitudinal steel reinforcement and ties were considered 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 non-structural screed ($1.3\ \mathrm{kN/m^2}$) 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 surface loads of all floors and the 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 $G + 0.3Q$ loading combination.

The seismic loads (E) were calculated in accordance with the EC8 [7] design spectrum, with a ground acceleration equal to $a_g = 0.24g$, (where g denotes the acceleration due to gravity, $9.81\ \mathrm{m/sec^2}$), soil type B (medium dense sand or stiff clay) and seismic zone II. For the Second-degree pre-earthquake inspection, the soil index is taken equal to $S = 1.00$ and the design spectrum is used with behavior factor equal to $q = 2.00$. For the non-linear static analysis (pushover analysis), the elastic spectrum is used and the soil index is considered equal to $S = 1.20$.

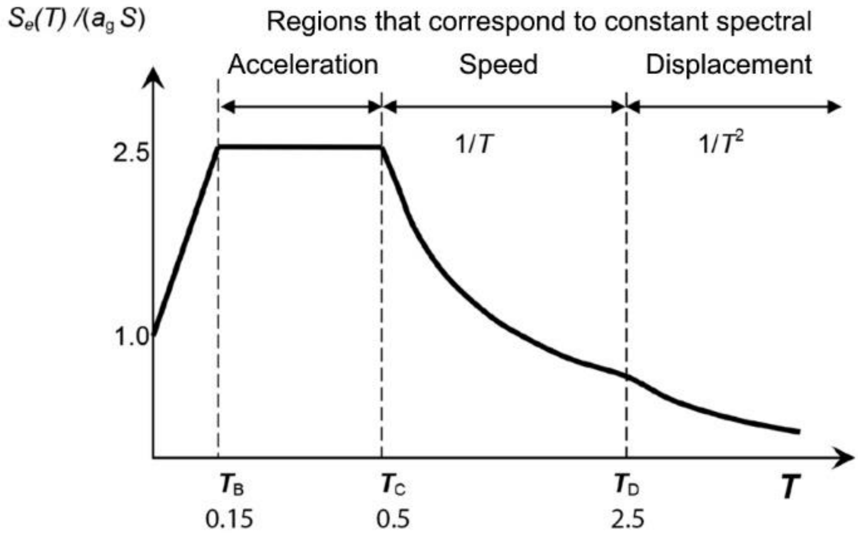


Fig. 2. Elastic Spectrum.

2.2 Dynamic Properties

The period, T (Empirical period) of the building, when applying the methodology of the Second-degree pre-earthquake inspection was determined according to the approximate equation of the Greek Code of Structural Interventions - *KANEPE* [3] from Eq.1:

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

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 (analysis period), the period T was determined according to the direction and distribution of the seismic loading and the structural properties of the buildings. The results obtained from the two methods are shown in Table 3. The indication "No Infill Walls" stands for case of the buildings $\mathrm{A_n}$ and B_n.

Table 3. Empirical and Analysis Periods T.

	EMPIRICAL	ANALYSIS					
		A	B	A_s	B_s	A_p	B_p
No Inf. Wall	0.41	1.36	1.32	-	-	-	-
Inf. Wall.g	0.41	0.71	0.75	0.63	0.82	0.99	0.96
Inf. Wall.p	0.41	1.06	0.78	0.71	0.71	1.17	1.02

Similar to [5], the period T obtained by the approximate equation from *KANEPE* [3], is equal for both structures with values much smaller than those resulting by the non-linear static analysis (pushover analysis). In the analysis, the periods T in all examined 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-degree pre-earthquake inspection predicts, where the period of the structure is between the T_B and T_c periods of the spectrum and therefore, corresponds to the plateau of the spectrum ($T < T_c$) (Fig. 2).

High values of the periods T of pushover analysis of the six building categories that were examined and are given in Table 3 are due to the fact that the effective stiffness of the members is obtained at the guilt point of the section using Eq.2 of *KANEPE* §7.2.3 [3]. As a result, the effective stiffness values of pushover analysis are significantly low for all examined buildings compared to the ones of the uncracked section. This is further demonstrated by Table 4, where the effective stiffness of the ground floor column C5 of building A is presented as a percentage of the respective uncracked section, for primary loading direction 90°. The same values of the effective stiffness arise for the rest of the primary loading directions (0°, 180° and 270°). As observed, the effective stiffness (EI_{eff}) has a low percentage ranging from 8.7% to 19.1% of the uncracked section stiffness (EI_{gross}). These values cannot exceed 25% of the uncracked section stiffness as dictated by *KANEPE* §7.1.2.2 [3]. This has been concluded in an earlier study of Bardakis and Dritsos [8] where the procedures of *FEMA 356* [9] and *KANEPE* [3] are compared. In the procedure of *FEMA 356* [9], high values of effective stiffness are used, as is also in the Second-degree pre-earthquake inspection examined in the present study.

Table 4. Correlation between effective stiffness and uncracked section stiffness for C5.

Angle (deg)	EI_{eff}/EI_{gross}
90	8.7%
	14.3%
	14.6%
	17.8%
	19.1%

3 Application of the approximate method

Analysis is performed for the "Significant Damage Performance" level (Level B) [3]. The behavior factor q, for all buildings is taken $q = 2.00$ according to Table 4 of the methodology of the Second-degree pre-earthquake inspection [1]. For the case of buildings with a soft storey, an additional investigation is performed considering a $q = 1.5$, as they could reasonably be characterized as buildings with unfavorable presence of masonry infill walls.

The basic seismic resistance (V_{R0}), of the building of the critical floor (generally the ground floor) is determined (see also [5]) by Eq.2:

$$V_{R0} = \alpha_1 \sum V_{Ri}^{columns} + \alpha_2 \sum V_{Ri}^{walls} + \alpha_3 \sum V_{Ri}^{short\ columns} + \sum V_{Ri}^{infill\ walls} \qquad (2)$$

where $V_{Ri}^{columns}$ is the seismic resistance of each column, V_{Ri}^{walls} is the seismic resistance of each wall, $V_{Ri}^{short\ columns}$ is the seismic resistance of each short column, $V_{Ri}^{infill\ walls}$ is the seismic resistance of each infill wall and α_1, α_2 and α_3 are values that can be taken according to the proposed method [1].

For the examined cases (frame structural system with RC columns) the values for reference buildings (A, B) and buildings with soft storey $(\mathrm{A_p}, \mathrm{B_p})$ are taken according to the proposed method as $\alpha_1 = 0.85$ and $\alpha_2 = \alpha_3 = 0$. For the case of buildings with short columns $(\mathrm{A_s}, \mathrm{B_s})$, the corresponding values are $\alpha_1 = 0.70$, $\alpha_2 = 0$, $\alpha_3 = 0.85$.

The shear resistance of the masonry infill walls $V_{Ri}^{infill\ walls}$ is calculated by the following Eq.3:

$$V_{Ri}^{infill\ walls} = 0.3\, f_{wc,s}\ t_w\ b_w\ \left(\frac{l}{L}\right) \qquad (3)$$

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 [1], whereas t_w, b_w stands for the thickness and the effective width of the infill wall respectively. The contribution of infill walls in the resistance of the structure is governed by *KANEPE* §7.4.1 [3]. 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 [3].

According to the methodology of the approximate method of the Second-degree pre-earthquake inspection, the maximum shear force that the vertical elements can withstand is calculated based on the existing reinforcements, by checking the expected failure mechanism (flexural or shear) as per $V_{R,i} = min\ (V_{Rd}, V_M)$. The flexural strength (V_M) is equal to $V_M = M_R/L_s$, where L_s is obtained by *KANEPE* [3].

In cases where reinforcement data is not available, there is the option for an approximate estimation of lower accuracy to be performed, considering only the shear strength of the vertical elements (V_{Rd}), without checking their flexural capacity. The method is applied by assuming that $V_{R,i} = V_{Rd}$. In this last case, the results are obtained by assuming that, in the examined cases, the height of the compression zone x is equal to $0.35d$, and the ductility index μ_{θ}^{pl} is equal to 2.5. The justification for these assumptions can be found in [5]. The value of V_{Rd} is determined based on the expression of *KANEPE* Appendix 7C [3]. The shear resistance of the structural elements has been determined using the Eq. C1 *KANEPE* of the aforementioned Appendix [3].

However, for short columns, the shear strength cannot exceed the limit value $V_{R,max}$, corresponding to web failure due to inclined compression, as given by Eq. C5 *KANEPE* of the same Appendix [3]. In particular, in the case of buildings (A_s, B_s), when reinforcement data is available the shear strength of vertical members *($V_{R,i}$)* is obtained by $V_{R,i} = min\ [(V_{Rd}, V_{R,max}), V_M]$, otherwise when reinforcement data in not available, by $V_{R,i} = min\ (V_{Rd}, V_{R,max})$.

In Table 5, the results of the shear strength of the columns of buildings (A & A_p) and (B & B_p) are presented, applying the Eq. C1 *KANEPE* of Appendix 7C [3]. The values of shear strength for building A and building with soft storey A_p are the same, similarly for buildings B and B_p, as the height of the cross section does not change according to the Eq. C1 *KANEPE* of Appendix 7C [3]. As observed, the critical failure mechanism for all cases of building's columns was the flexural failure *(V_M)*, except for the buildings with short columns. In Table 6, the results of the shear strength of the columns for buildings with short columns are presented, applying the Equations C1 and C5 *KANEPE* of Appendix 7C [3]. As observed, the critical failure mechanism in shear for all cases of short columns was the failure in inclined compression *($V_{R,max}$)*.

Table 5. Shear resistance of RC columns – Approximate and Precise values for Buildings A & A_p , B & B_p.

Second degree pre-earthquake Insp.	BUILDINGS A & A_p			BUILDINGS B & B_p		
	Reinf. Data		No Reinf. Data	Reinf. Data		No Reinf. Data
COLUMNS	V_{Rd} (KN) Eq.C1	V_M (KN)	V_{Rd} (KN) $x = 0.35d$ $\mu_{\theta}^{pl} = 2.5$ Eq.C1	V_{Rd} (KN) Eq.C1	V_M (KN)	V_{Rd} (KN) $x = 0.35d$ $\mu_{\theta}^{pl} = 2.5$ Eq.C1
1	107.31	92.44	108.35	107.31	92.44	108.35
2	126.11	101.80	133.52	126.50	101.88	134.32
3	100.40	85.58	101.94	104.75	89.87	105.94
4	126.22	101.82	133.76	126.70	101.92	134.73
5	185.46	158.41	173.30	177.57	157.52	181.45
6	122.10	101.19	126.27	98.06	83.30	99.84
7	109.16	94.31	110.12	115.09	100.41	115.93
8	127.11	102.00	135.58	108.05	93.19	109.06
9	102.13	87.27	103.52	–	–	–

Table 6. Shear resistance of RC columns – Approximate and Precise values for building A_s-B_s.

Second-degree pre-earthquake Insp.	BUILDING A_s				
	Reinf. Data			No Reinf. Data	
COLUMNS	V_{Rd} (KN) Eq.C1	$V_{R,max}$ (KN) Eq.C5	V_M (KN)	V_{Rd} (KN) $x = 0.35d$ $\mu_\theta^{pl} = 2.5$ Eq.C1	$V_{R,max}$ (KN $\mu_\theta^{pl} = 2.5$ Eq.C5
1	275.74	111.17	459.10	267.26	108.34
2	367.65	131.57	508.73	393.10	127.65
3	245.73	105.83	424.74	235.22	103.42
4	368.28	131.76	508.84	394.28	127.83
5	185.47	-	158.42	195.64	-
6	346.56	125.77	505.77	356.84	122.09
7	283.81	112.64	468.49	276.11	109.70
8	372.96	133.23	509.72	403.43	129.23
9	253.22	107.14	433.24	243.10	104.63
Second-degree pre-earthquake Insp.	BUILDING B_s				
	Reinf. Data			No Reinf. Data	
COLUMNS	V_{Rd} (KN) Eq.C1	$V_{R,max}$ (KN) Eq.C5	V_M (KN)	V_{Rd} (KN) $x = 0.35d$ $\mu_\theta^{pl} = 2.5$ Eq.C1	$V_{R,max}$ (KN) $\mu_\theta^{pl} = 2.5$ Eq.C5
1	275.74	111.17	459.10	267.26	108.34
2	369.75	132.22	509.11	397.10	128.26
3	264.59	109.15	446.23	255.19	106.49
4	370.81	132.55	509.31	399.17	128.58
5	536.15	140.75	850.37	553.54	135.69
6	235.61	104.08	413.35	224.72	101.81
7	309.70	117.49	499.03	305.17	114.16
8	278.98	111.76	462.86	270.80	108.88

The final seismic resistance V_R, is determined for each direction by Eq.4:

$$V_R = \beta \times V_{R0} \tag{4}$$

where β is the reduction factor determined according to 13 criteria which are used to assess the vulnerability of a structure, as described in the methodology [1], and evaluated with a value of 1 to 5, where 1 corresponds to the highest reduction.

In the case of building with short columns, all criteria were graded with 5 for both buildings, except for criterion 3 (Normalized axial load), which resulted in $\beta_3 = 2$ for building A and $\beta_3 = 3$ for building B. These values were obtained taking into consideration the average value of the normalized axial load of the vertical elements which were $0.35 \leq v_d = 0.39 < 0.45$ for building A and $0.25 \leq v_d = 0.34 < 0.35$ for building B. In criterion 7 (Stiffness distribution in elevation) both buildings were graded equal to $\beta_7 = 1$, given that the ductility of the ground floor exceeds the ductility of the first floor with a value of $462\% > 150\%$ in the x direction and $548\% > 150\%$ in the y direction for building A and $506\% > 150\%$ in the x direction and $491\% > 150\%$ in the y direction for building B. In criterion 9 (Short columns), according to the calibration of the criterion using the value of the quantity l/h of the supports and the gravity factor corresponding to it, the grade is equal to β_9 =1 for building A and $\beta_9 = 1.67$ for building B. Only in building B, criterion 5 (Stiffness distribution in plan – torsion) is differentiated in the x direction, where the normalized eccentricity is $\varepsilon_x = 0.076 > 0.05$, corresponding to a grade equal to β_5 =4. Consequently, the values of the reduction factor β for building A_s and B_s in the two main directions were $\beta_x = \beta_y = 0.76$ and $\beta_x = 0.74$, $\beta_y = 0.77$ respectively.

In the case of buildings with a soft storey (pilotis), Criterion 3 (Normalized axial load), was graded the same as in the reference buildings (see [5]), equal to $\beta_3 = 3$ for both buildings A and B. Criterion 7 (Stiffness distribution in elevation), was graded equal to $\beta_7 = 1$, for both buildings, as the stiffness of the first floor exceeds the one of the ground floor with values of $245\% > 150\%$ in the x direction and $269\% > 150\%$ in the y direction for building A, and $277\% > 150\%$ in the x direction and $303\% > 150\%$ in the y direction for building B. Criterion 5 (stiffness distribution in plan – torsion), was graded equal to β_5 =4, as the eccentricity in the loading direction x was found to be $\varepsilon_x = 0.067 > 0.05$. As a result, the values of the reduction factor β for buildings A_p and B_p in the two main directions were $\beta_x = \beta_y = 0.86$ and $\beta_x = 0.84$, $\beta_y = 0.86$ respectively.

It is noted that values of the reduction factor β for the reference buildings A and B have been derived in [5] and are $\beta_x = \beta_y = 0.98$ 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 determined considering the available seismic resistance and the seismic demand according to Eq.5 as follows:

$$\lambda_x = \frac{V_{req,x} + 0.30V_{req,y}}{V_{R,x} + 0.30V_{R,y}} \;,\; \lambda_y = \frac{V_{req,y} + 0.30V_{req,x}}{V_{R,y} + 0.30V_{R,x}} \tag{5}$$

The classification of the building into a seismic category of Second-degree pre-earthquake inspection is done based on the capacity factor δ, as follows according to Eq.6:

$$\delta = min\left\{\frac{1}{\lambda_x}, \frac{1}{\lambda_y}\right\} \tag{6}$$

Further information on this matter can be found in [10].

4 Application of the Non-linear static analysis method

A non-linear static analysis (pushover analysis) method is also employed, in accordance with the provisions of *KANEPE* [3] for a "performance level B". As described in [5], the failure indices λ of the buildings are defined in two ways: a) based on the minimum horizontal ground acceleration a_g for which the first failure of a vertical member of the building occurs for an acceptable "performance level B", as defined in Eq.7, and b) through the maximum failure index λ_{max} of the vertical elements of the structure for a "performance level B1" and for all possible loading combinations.

$$\lambda_{ag} = \frac{a_{g,ref}}{\alpha_g} \tag{7}$$

where $a_{g,ref}$ is the reference horizontal ground acceleration, 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 this case of the study, the reference horizontal ground acceleration equals to $0.24g$.

The seismic category of the building is determined according to *KANEPE* [3] based on the ratio $a_g\ /a_{g,ref}$, i.e., the quantity $1\ /\ \lambda_{ag}$ and is defined as the maximum target for assessment or redesign that a building can achieve for a selected "performance level B". More details on this matter are presented in [10].

The pushover analysis was conducted utilizing the commercial software *FESPA* of LH Logismiki [11]. The structural system is a three-dimensional concrete frame. The beam elements used in the analysis are considered as 3D beam-column elements with concentrated plasticity in both edges. It is a displacement-based beam element, where plastic penetration is taken into account. The flexural plastic hinges have been defined using the cord rotation equations provided by *KANEPE* [3]. The model has been checked for both flexure and shear plastic hinges. For shear, the provisions of Chapter 7 *KANEPE* §7.2.4.2 and §7.2.5 [3] are followed. The displacement-based control, that has been used was the multi-control point. The minimum number of steps (static analysis) to be performed in each pushover analysis with incrementally increasing load until the end of the analysis, which is defined as the exceedance of the given maximum displacement for the "performance level B", is equal to 120. The displacement control arises from the minimum number of steps, and the maximum displacement is 3% of the building's height.

The mechanic behaviour of a structural element, or of a critical region of a structural element is described through a diagram of force "F" versus deformation "δ". In the present study the choice of F and δ are moment "M" and chord rotation "θ" at the ends of the element, where θ incorporates the sum of flexural and shear deformations, as well as the rotation of member ends due to reinforcement slip [3].

The non-linear laws of the reinforced concrete sections that were used to the non-linear analysis are defined in terms of $M-\theta$, where a) the deformation in yielding, is determined by the chord rotation θ_y, using the Eq. S.2a *KANEPE* §7.2.2 [3], b) the deformation in failure, is determined by the mean value of chord rotation at failure θ_{um}, using the Eq. S.11a *KANEPE* §7.2.4.1 [3] and c) the yielding moment, $M_y = M_u$ is determined by the Eq. A.6a *KANEPE* of Appendix 7.A [3]. An indicative representation of the relation of moment "M" as a function of chord rotation "θ" is presented in Fig.3 for "performance level B" of the ground floor column C5 of building A.

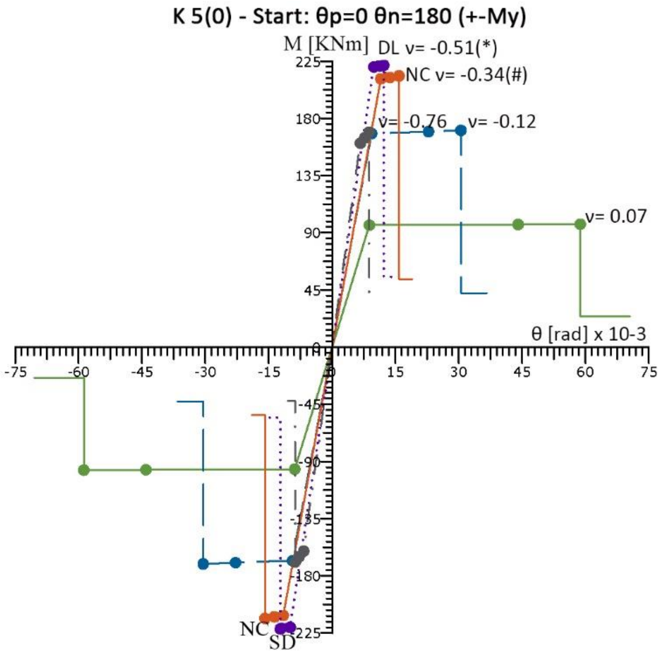
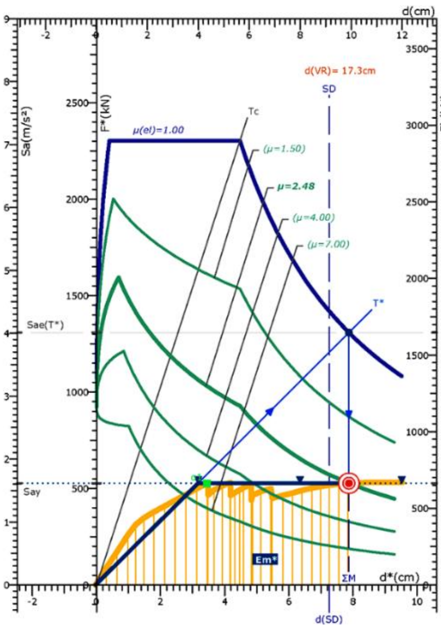
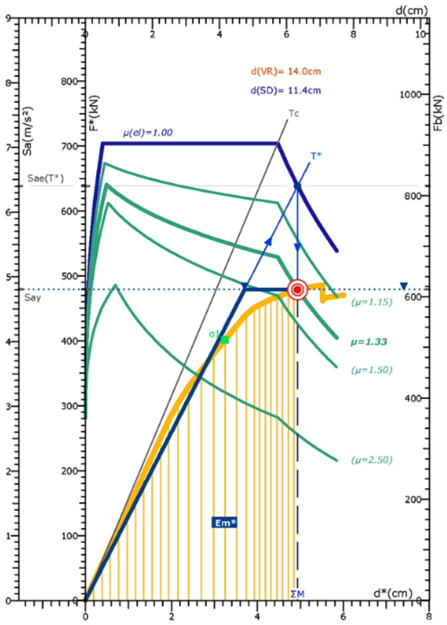


Fig. 3. Moment "M" as a function of chord rotation "θ"

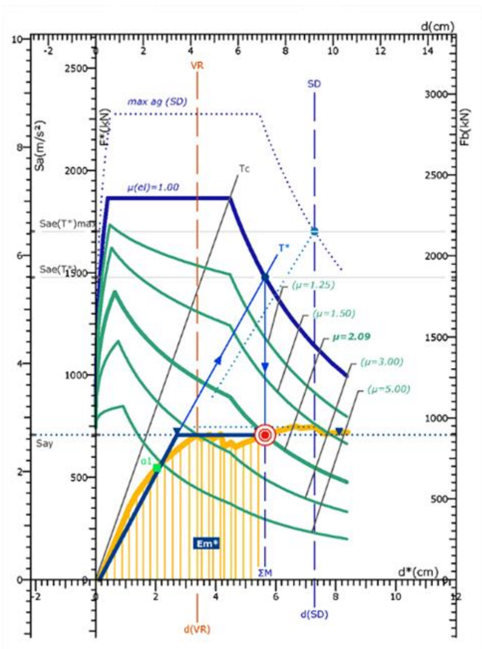
Pushover analysis significantly evaluates the expected performance level of the structural system by the building's capacity curve. Based on this capacity curve, the target displacement expected to occur during the earthquake is estimated. Fig. 4 presents the capacity curves for a "performance level B" for the direction $(90° + 30\% \cdot 0° + eX)$ for the column C5 of the ground floor of the buildings A, A_s and A_p respectively for good construction detailing and wedging of masonry infill walls considering the presence of openings. Moreover, the acceleration displacement response spectrum is also evident. In Fig. 4(a) and Fig. 4(b) the dashed blue line (SD) is located to the left of the target displacement, indicating that column C5 fails in flexure. In Fig. 4(c) the same column fails in shear, as the dashed blue line (SD) is located to the right of the target displacement, and the dashed orange line (VR) precedes the target displacement. The yellow line represents the capacity curves. The blue line represents the elastic spectrum $\mu(el) = 1.00$ and the green line represents the inelastic spectrum arising from the transformations of Eq. 6a and Eq. 6b *KANEPE* §7.2.6.2 [3]. The T^* is the period of an elastic single degree of freedom system.



(a)



(b)



(c)

Fig. 4. Capacity curves for column C5 for building A (a) reference (b) soft storey and (c) short columns.

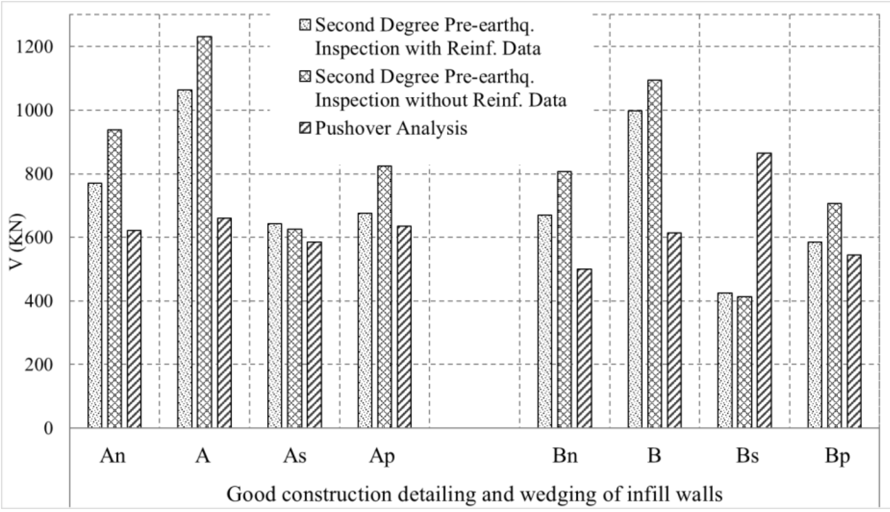
5 Results and Discussion

5.1 Determination of seismic resistance of Second-degree pre-earthquake inspection and Non-linear static analysis

In Table 7, the results of seismic resistance, in terms of base shear force for the reference buildings, those with short columns, and those with soft storey (pilotis) are presented. Fig. 5 presents, the results of shear forces considering good and poor construction detailing and wedging of masonry infill walls on the two upper floors. The results from the application of the methodology of the Second-degree pre-earthquake inspection are presented for both cases previously mentioned above, i.e. when data about reinforcement amounts of the vertical elements are available and when they are not available. Regarding the non-linear static analysis (pushover analysis), the value of the shear force presented in Table 7 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". The failure mechanism determined by applying the Second-degree pre-earthquake inspection with known reinforcement data, was found to be flexural for the case of reference buildings and pilotis, while for the case of buildings with short columns, it was found to be shear, in full agreement with the analysis results for both buildings. The values of the seismic resistance obtained when the Second-degree pre-earthquake inspection was applied without reinforcement data were higher than those obtained using reinforcement data. This is reasonable considering that, for this case, only the shear strength of the members is taken into account.

Table 7. Maximum Seismic Resistance.

	BUILDING A			BUILDING B		
	Second degree pre-earthquake Insp.		Non-linear Static Analysis	Second degree pre-earthquake Insp.		Non-linear Static Analysis
	Reinf. data	No Reinf. data		Reinf. data	No Reinf. data.	
REFERENCE BUILDINGS						
No Infill Walls	770.39	938.26	620.75	669.54	807.53	499.91
Inf. W.g	1064.42	1232.29	660.62	997.10	1095.20	64.47
Inf. W.p	878.70	1046.57	654.40	808.74	914.38	528.94
BUILDINGS WITH SHORT COLUMNS						
Inf. W.g	642.57	625.47	854.00	423.94	412.60	865.00
Inf. W.p	642.57	625.47	848.00	423.94	412.60	849.00
BUILDINGS WITH SOFT STOREY						
Inf. W.g	676.06	823.37	634.95	585.85	706.59	544.07
Inf. W.p	676.06	823.37	629.33	585.85	706.59	529.74



(a)

(b)

Fig. 5. Earthquake resistance obtained by Second degree pre-earthquake inspection (with and without considering reinforcement data) and Non-linear static analysis for (a) Good and (b) Poor construction detailing and wedging of infill walls.

5.2 Determination of the failure indices by Second-degree pre-earthquake inspection and Non-linear static analysis

Fig. 6 and Fig. 7 demonstrate the values of failure indices ($\boldsymbol{\lambda_{Sec.}}$) related to the Second- degree pre-earthquake inspection [1], for both the cases of available, and unavailable ($\boldsymbol{\lambda_{Sec.,v}}$) reinforcement data. The corresponding values obtained by the non-linear static analysis (pushover analysis) for the failure index in terms of base acceleration ($\boldsymbol{\lambda_{ag}}$) and in terms of maximum failure index ($\boldsymbol{\lambda_{max}}$) are also presented for good and poor construction detailing and wedging of masonry infill walls. The dashed line indicates the value corresponding to the adoption of the behavior factor $\boldsymbol{q = 2.0}$ in the case of buildings with a soft storey (pilotis).

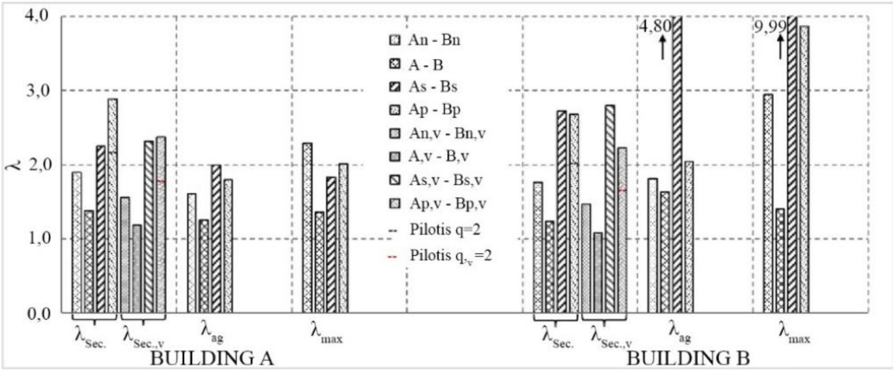


Fig. 6. Failure indices obtained by Second-degree pre-earthquake inspection and Non-linear static analysis for good construction detailing and wedging of infill walls.

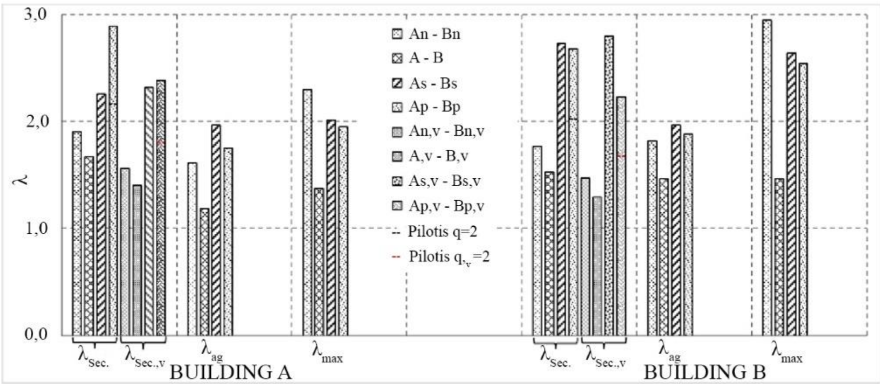


Fig. 7. Failure indices obtained by Second-degree pre-earthquake inspection and Non-linear static analysis for poor construction detailing and wedging of infill walls.

As can be observed from Fig. 6 and Fig. 7, failure indices $\lambda_{Sec.}$ are in very good agreement with λ_{ag} in comparison to the failure indices λ_{max} of the columns. This can be explained considering that the failure indices λ_{max} represent local member deficiencies that can lead to incorrect conclusions about the overall behavior of the structure.

Table 8. Failure indices λ.

Buildings	Categories of Infill Walls	Second-degree pre-earthquake inspection								Non-linear Static Analysis	
		v.2022 - Reinf. data		v.2022 - No Reinf. data		v.2018 - Reinf. data		v.2018 - No Reinf. data		λ_{max}	λ_{ag}
REFERENCE BUILDINGS											
A	No Infill Walls	1.90		1.56		2.24		1.84		2.30	1.61
	Inf. Wall.g	1.38		1.19		2.24		1.84		1.37	1.26
	Inf. Wall.p	1.67		1.40		2.24		1.84		1.37	1.18
B	No Infill Walls	1.76		1.46		2.07		1.72		2.95	1.82
	Inf. Wall.g	1.23		1.08		2.07		1.72		1.41	1.63
	Inf. Wall.p	1.52		1.29		2.07		1.72		1.31	1.47
BUILDINGS WITH SHORT COLUMNS											
A	Inf. Wall.g	2.25		2.32		2.51		2.58		1.83	2.00
	Inf. Wall.p	2.25		2.32		2.51		2.58		2.01	1.97
B	Inf. Wall.g	2.73		2.80		3.21		3.30		9.99	4.80
	Inf. Wall.p	2.73		2.80		3.21		3.30		2.64	1.97
BUILDINGS WITH SOFT STOREY											
	q	2.0	1.5	2.0	1.5	1.7	1.3	1.7	1.3	-	-
A	Inf. Wall.g	2.17	2.89	1.78	2.37	2.55	3.33	2.09	2.74	2.01	1.80
	Inf. Wall.p	2.17	2.89	1.78	2.37	2.55	3.33	2.09	2.74	1.95	1.75
B	Inf. Wall.g	2.01	2.68	1.67	2.22	2.36	3.09	1.96	2.56	3.87	2.05
	Inf. Wall.p	2.01	2.68	1.67	2.22	2.36	3.09	1.96	2.56	2.54	1.88

Similar results are presented in Table 8 to evaluate the influence of the recent changes in the Secon-degree pre-earthquake inspection guidelines [1], showing the values of failure indices according to both versions of the provisions [1-2] along with the corresponding results from the non-linear static analysis (pushover analysis). As observed, the values of failure indices according to the methodology of the pre-revised version [2] are consistently higher than those from the revised version [1], are independent of the presence of masonry infill walls, and deviate more from the corresponding results of the non-linear static analysis (pushover analysis). The main reason for these differences is that in the pre-revised version [2]: a) lower values are adopted for the available behavior factor q (which for the cases examined is considered equal to $\boldsymbol{q = 1.7}$ instead of $\boldsymbol{q = 2.0}$ that is dictated by the revised version [1]) and b) the contribution of masonry infill walls to the seismic resistance of the structure is disregarded.

It is observed that the failure indices are significantly higher for buildings with short columns and soft storey compared to the reference buildings in both the Second-degree pre-earthquake inspection and the non-linear static analysis (pushover analysis), for both types of masonry infill walls (Inf. Wall.g, Inf. Wall.p). For buildings with pilotis, the results from the Second-degree pre-earthquake inspection and the analysis converge more for a value of the behavior factor $q = 2.0$ compared to the ones when the value $q = 1.5$ is used. Therefore, it is considered reasonable to adopt the value $q = 2.0$ for the Second-degree pre-earthquake inspection for buildings with soft storey (pilotis). This could be interpreted considering that the deficiency of the soft storey, due to the non-uniform distribution of masonry infill walls along the height of the building, is already being considered through the vulnerability factor *β*.

5.3 Categorization of buildings into Seismic Categories via the method of the Second-degree pre-earthquake inspection and Seismic Classifications as defined by *KANEPE*

In Table 9, the results of the classification of buildings into seismic categories according to the Second-degree pre-earthquake inspection and seismic classes according to the approximate equation of the Greek Code of Structural Interventions - *KANEPE* [3] are presented. In all cases, the classification is being done according to the capacity factor $\boldsymbol{\delta = 1/\lambda}$, where for seismic classes $\boldsymbol{\delta = 1/\lambda = a_g/a_{g,ref}}$. The results of the Second-degree pre-earthquake inspection are presented for both the cases of known and unknown reinforcement amounts of the vertical elements (Second degree pre-earthquake inspection with / without reinforcement data respectively).

Table 9. Seismic Categories of Structures.

Buildings	Infill Walls	Second-degree pre-earthquake inspection				Non-linear Static Analysis	
		Reinf. Data $\delta = 1/\lambda$	Seismic Category-Reinf. Data	No Reinf. Data $\delta = 1/\lambda$	Seismic cate-gory-No Reinf. Data	$\frac{a_g}{a_{g,ref}}$	Seismic Classes by *KANEPE*
REFERENCE BUILDINGS							
A	Inf. W.g	1/1.38=0.72	K2	1/1.19=0.84	$K2^+$	1/1.2= 0.79	$B2^+$
	Inf. W.p	1/1.67=0.60	K2	1/1.40=0.71	K2	1/1.1= 0.85	$B2^+$
B	Inf. W.g	1/1.23=0.81	$K2^+$	1/1.08=0.93	$K2^+$	1/1.6= 0.61	B2
	Inf. W.p	1/1.52=0.66	K2	1/1.29=0.78	$K2^+$	1/1.4= 0.68	B2
BUILDINGS WITH SHORT COLUMNS							
A	Inf. W.g	1/2.25=0.44	K3	1/2.32=0.43	K3	1/2.00= 0.50	$B3^+$
	Inf. W.p	1/2.25=0.44	K3	1/2.32=0.43	K3	1/1.97= 0.51	$B3^+$
B	Inf. W.g	1/2.73=0.37	K3	1/2.80=0.36	K3	1/4.80= 0.21	B4
	Inf. W.p	1/2.73=0.37	K3	1/2.80=0.36	K3	1/1.97= 0.51	$B3^+$
BUILDINGS WITH SOFT STOREY							
A	Inf. W.g	1/2.17=0.46	$K3^+$	1/1.78=0.56	$K3^+$	1/1.80= 0.56	$B3^+$
	Inf. W.p	1/2.17=0.46	$K3^+$	1/1.79=0.56	$K3^+$	1/1.75= 0.57	$B3^+$
B	Inf. W.g	1/2.01=0.50	$K3^+$	1/1.67=0.60	K2	1/2.05= 0.49	$B3^+$
	Inf. W.p	1/2.01=0.50	$K3^+$	1/1.67=0.60	K2	1/1.88= 0.53	$B3^+$

It is observed that a great convergence exists between the seismic categories derived from the Second-degree pre-earthquake inspection and the corresponding seismic classifications outlined in *KANEPE* [3]. This convergence is particularly conspicuous in cases where precise information regarding reinforcement amounts of the vertical elements was available.

6 Conclusions

In this present study, the reference buildings described in [5] were examined but considering the present of short columns and soft storey (pilotis) on the ground floor. The assessment of their seismic capacity was done by applying the approximate methodology of Second-degree pre-earthquake inspection and results were validated by comparison with the corresponding ones of a non-linear static analysis (pushover analysis). Subsequently, failure indices were determined, leading to the structural categorization of the buildings into seismic categories as defined by the Second-degree pre-earthquake inspection and seismic classifications as derived by *KANEPE* [3]. The outcomes derived from the examined buildings in this study lead to the following conclusions. It is evident that further research is imperative, involving a more extensive examination of diverse building cases, to establish comprehensive and reliable conclusions applicable to a broader spectrum of structures:

- The failure mechanism determined by the Second-degree pre-earthquake inspection with available reinforcement data for vertical elements, was found to be flexural in the reference buildings and in buildings with a soft storey on the ground floor. In contrast, structures with short columns exhibited shear failures, notably attributed to the exceedance of the web's resistance in inclined compression. These findings were confirmed by the results derived from the non-linear static analysis (pushover analysis).
- The buildings which were examined, with or without the existence of a soft storey or short columns, the failure indices (λ) and corresponding seismic categories obtained when applying the Second-degree pre-earthquake inspection, were, in most case, in great convergence with the corresponding results of the non-linear static analysis (pushover analysis). In the analysis, failure indices are given in terms of acceleration (λ_{ag}) and seismic classifications are considered as defined in *KANEPE* [3]. The convergence between the values of the failure indices was lower for the case where reinforcement data were unavailable but as good regarding seismic classifications. However, it is imperative to underscore that, under no circumstances, does this observation permit a direct correspondence between the seismic classifications according to *KANEPE* [3] and the corresponding seismic categories determined by the Second-degree pre-earthquake inspection.
- The seismic vulnerability of buildings with soft storey or short columns, compared to the reference buildings, was confirmed in both methods in an equivalent manner.
- For the assessment of buildings with soft storey, using the methodology of the Second-degree pre-earthquake inspection, it is reasonable to use a behavior factor equal to $q = 2.0$, rather than using the value of $q = 1.5$.

References

1. EPPO, Second level pre-earthquake assessment of RC buildings, Earthquake Planning and Protection Organization of Greece (EPPO), Greek Ministry for Environmental Planning and Public Works, Athens, Greece (1st Revision 2022).
2. EPPO, Second level pre-earthquake assessment of RC buildings, Earthquake Planning and Protection Organization of Greece (EPPO), Greek Ministry for Environmental Planning and Public Works, Athens, Greece (2018).
3. KANEPE, Greek code for structural interventions. Earthquake Planning and Protection Organization of Greece (EPPO), Greek Ministry for Environmental Planning and Public Works, Athens, Greece (2017).
4. KADET, Greek code for the assessment and structural interventions of masonry structures. Earthquake Planning and Protection Organization of Greece (EPPO), Greek Ministry for Environmental Planning and Public Works, Plan 2021, Athens, Greece (2021).
5. Zochiou L., Panagopoulou M., Dritsos S.: An approximate method to assess the seismic capacity of existing RC buildings. In: 5th ΠΣΑΜΤΣ, Athens, 20-22 October (2022).
6. Vasileiadi M., Dritsos S.: An approximate method to assess the seismic capacity of existing RC buildings. 7th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering, Crete, Greece, 24–26 June (2019).
7. EC8, Eurocode 8 Part 3, Design of structures for earthquake resistance: Assessment and retrofitting of buildings. European Standard EN 1988-3, European Committee for Standardization (CEN), Brussels (2005).
8. Bardakis V.G., Dritsos S.E.: Evaluating assumptions for seismic assessment of existing buildings. Soil Dynamics and Earthquake Engineering 27 (2007) 223-233. Elsevier.
9. FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, American Society of Civil Engineers (ASCE), Washington, D.C. (2000).
10. Apostolidi E., Dritsos S.: Seismic categories and seismic classifications – Comparisons. In: 5th ΠΣΑΜΤΣ, Athens, 20-22 October (2022).
11. FESPA, LH LOGISMIKI, High Performance Software, Version 1.2.82, Athens, Greece (2016).