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AN APPROXIMATE METHOD TO ASSESS THE SEISMIC CAPACITY OF EXISTING RC BUILDINGS

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Abstract

A new approximate method for the assessment of reinforced concrete buildings is demonstrated and validated through a realistic example via non-linear static analysis. This method aims to handle the problem when a building's reinforcement information is unknown. In order to deal with such a problem, two reinforcement assumptions are made. The first one considers zero reinforcement, while the second one considers the minimum reinforcement amounts of Eurocodes 2 and 8. For both assumptions, the safety indices of an existing building are calculated, and the results are compared with the corresponding ones of a non-linear static analysis. It is shown that this approximate technique is able to predict with acceptable accuracy the safety indices and thus, can be successfully used for the preliminary analysis and assessment of a building with unknown reinforcement amounts. Moreover, both assumptions, i.e., ignoring or accounting for minimum reinforcement, produced quite similar results.

Keywords: Approximate method, Reinforced concrete, Preliminary seismic analysis, Building assessment, Non-linear static analysis

1 INTRODUCTION

During the last 20 years or so, the need for a strengthening assessment of reinforced concrete (RC) buildings has been increased rapidly, mostly because the majority of existing buildings were constructed in the years 1950-1970, which means that these buildings have exceeded their design working life with regard to present regulations. In addition, these buildings were designed according to old regulations and with simple calculation tools, which do not reflect today's higher standards. Since their number is quite large, it is difficult to simultaneously assess all of them with advanced analysis methods and thus, they should be prioritized on the basis of their seismic vulnerability. Moreover, difficulties are increased when their design and construction details are not available to engineers, which is the case for many existing buildings.

To solve these problems, recently, a working group set up by EPPO (Earthquake Planning and Protection Organization of Greece) has proposed a method for the preliminary assessment of RC buildings [1]. It is an approximate method for assessing the seismic capacity of existing RC buildings in accordance with the seismic requirements of current regulations. Since it is based on simple calculations, there is no need of a detailed model and time-consuming advanced analysis methods.

A major characteristic of this approximate method is its ability to estimate the capacity of structures, for which their reinforcement details are unknown, making it particularly important and original. The proposed method is applied for two assumptions for the amount of reinforcement. The first one considers zero longitudinal and transverse reinforcement, while the second one considers the minimum reinforcement amounts, for all the horizontal and vertical members.

The above method is demonstrated herein in detail through an application to an existing RC building with unknown reinforcement details. Its degree of accuracy is validated through comparisons with the more advanced and accurate non-linear static analysis method. The values of interest are the safety indices of the examined building as found by both methods, i.e. approximate and advanced, and according to these results, conclusions are drawn for the accuracy of the proposed method to predict the seismic capacity.

The proposed approximate method consists of two main parts: Informational and computational. The first part is based on 13 criteria from which a reduction factor β is deduced, to be used in the computational part. These criteria defined by the proposed method are the following: Existing structural damage, reinforcement oxidation, normalized axial load, regularity in plan, stiffness distribution in plan – torsion, regularity in elevation, stiffness distribution in elevation, mass distribution in elevation, short columns, vertical discontinuities, forces route and transfer, neighboring buildings, and faulty workmanship or non-structural damage that has occurred either during or after construction [1]. The second part of the proposed method includes the following steps: i) Determination of the seismic demand, ii) determination of the seismic resistance and iii) determination of the safety indices [1].

The present work mainly focuses on the second part of the method, which consists of the computational process. In section 2, the building to be examined is presented and in section 3 it is evaluated by the proposed method, while in section 4 it is re-evaluated by the more accurate non-linear static analysis. In section 5 the results are compared and finally in section 6 all the important conclusions are drawn.

2 DESCRIPTION OF THE CASE STUDY RC BUILDING

The examined RC building was constructed in 1988. Its floor plan is square-shaped at all stories, with a total length and width equal to 15 m, as shown in Figure 1. The building consists of 5 floors, with the ground floor height being 5.50 m and the remaining floor heights being 3.50 m (Figure 1). The ground, 3rd and 4th floors are used as offices, while the 1st and 2nd floors contain machinery. The columns are 0.60x0.60 m in the ground floor, 0.50x0.50 m in the 1st and 2nd floors and 0.40x0.40 m in the 3rd and 4th floors. The Π-shaped shear wall of the elevator is 3x3x0.25 m, and the beams are 0.25x1.00 m. The material properties are C16/20 for concrete and S500 for the reinforcing steel.

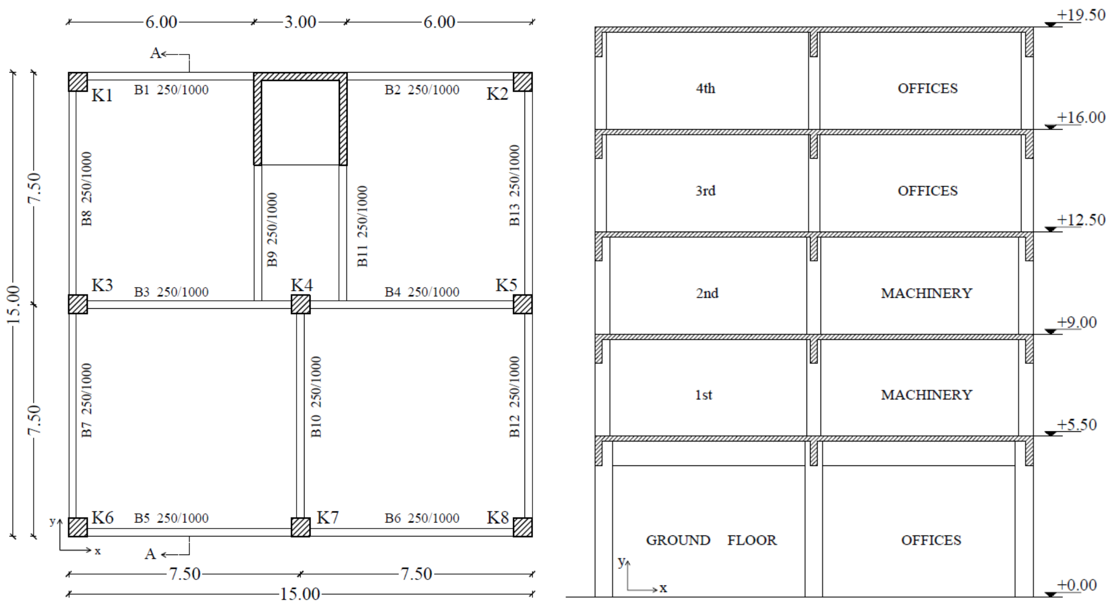

Figure 1: Story plan and section A-A.

2.1 Loads

The dead loads (G) include the self-weight of the structure from material properties (25 kN/m^3 for the RC), the toppings (1 kN/m^2), the outer masonry walls (3.6 kN/m^2), and the roof insulations (2 kN/m^2). The live loads (Q) include the ones in the office floors (2 kN/m^2 with ψ_2 = 0.3, where ψ_2 is a combination coefficient for the quasi-permanent variable action), in the machinery floors (5 kN/m^2 with ψ_2 = 0.9), in the stairs and balconies (2 kN/m^2 with ψ_2 = 0.3), and in the roof (1 kN/m^2 with ψ_2 = 0.3). Seismic loads (E) were calculated in accordance with the EC8 [2] response spectrum with a ground acceleration equal to $a_g = 0.24g$ (where g denotes the acceleration due to gravity, 9.81 m/sec^2), soil type B (medium dense sand or stiff clay), and seismic zone II.

The influence of the concrete slabs was taken into account by modeling all beams as T-beams with an effective width, b_{eff}, by defining a diaphragm on each floor and by considering a dead load distributed appropriately on beams. The total mass of the building was found to equal $M_{tot} = 1589$ tonnes.

2.2 Dynamic characteristics

The period, T, of the building was calculated in two different ways:

1. According to the approximate equation of EC8 [2], which is called empirical period and was obtained according to the following equation:

$$T = C_t\, H^{\frac{3}{4}} \tag{1}$$

   where C_t is equal to 0.05 and H is the height of the building starting from the foundation.
2. By applying an elastic analysis, which is called the analysis period. The analysis period resulted from modal analysis using the secant-to-yield stiffness for all the members, which was determined by section analysis. The percentage of mass participation for the 1st mode resulted in 85.6% and 0.28% for the x and y directions, respectively, and for the 2nd mode in 0.95% and 78.7% for the x and y directions, respectively.

In Table 1 the empirical and analysis periods are presented for each direction.

Seismic Direction	Empirical T (sec)	Analysis T (sec)
x	0.46	1.82
y		1.27

Table 1: Empirical and analysis periods for each direction.

2.3 Reinforcement details

Since the reinforcement of the examined structure is unknown, two assumptions for the members' reinforcement are made: i) zero longitudinal and transverse reinforcement values and ii) minimum reinforcement according to the EC8 and EC2 requirements [2,3].

The minimum longitudinal reinforcement, ρ_{min}, for the beams was taken as equal to $\rho_{min} = 0.5\, f_{ctm}/f_{yk}$ (where f_{ctm} is the mean tensile strength of concrete and f_{yk} is the characteristic yield strength of the reinforcement). For the columns, the minimum longitudinal reinforcement was taken as equal to 10Φ20, 10Φ16 and 12Φ12 for the 0.60x0.60, 0.50x0.50, and 0.40x0.40 columns, respectively, which corresponds to 8‰ of their cross-sectional area. For the walls, it was taken as equal to 4Φ12 at the corners and 7Φ8 in between per 30 cm, as shown in Figure 2. The transverse reinforcement of all the members was considered to be equal to Φ8/250.

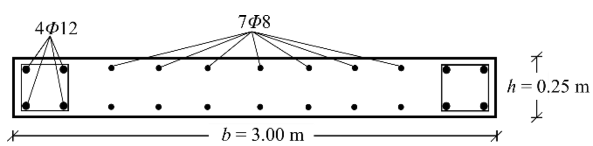

Figure 2: Wall reinforcement.

3 APPLICATION OF THE PROPOSED APPROXIMATE METHOD

The determination of the existing reinforcement amounts of the structural elements in RC buildings is often a time-consuming procedure. In order to minimize the time of collecting all the reinforcement information, the proposed method can be used easily for alternative assumptions of the reinforcement amounts, as it avoids this time-consuming part.

This section describes in detail the steps of the computational process of the proposed approximate method, by applying them to the examined RC building. In particular, the approximate process for determining the safety indices, including seismic demand and resistance, is described. Both assumptions of members' reinforcement amounts, i.e., zero and minimum, are used in the calculations.

3.1 Determination of the seismic demand

The first step is to calculate the seismic demand, V_{req}, in terms of base shear force, which is determined according to the design spectrum for each direction and is defined by the following equation [2]:

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

where, M is the structure mass and S_d is the design spectrum acceleration at period T. This study is performed for the Significant Damage performance level (or Level B). The behavior factor, q, is obtained by KANEPE [4], depending on the performance level and the direction for which the check is being conducted. It was taken to equal $q_{B,x} = 1.70$ and $q_{B,y} = 2.30$ due to unfavorable and favorable presence of the infills in the structure for the x and y directions, respectively. Table 2 summarizes the values of the seismic demand, V_{req}, for the empirical and analysis period of the structure.

Seismic Direction	V_{req} (kN)	
	Empirical T	Analysis T
x	6602	1813
y	4878	1923

Table 2: Seismic demand, V_{req}, for each period.

3.2 Approximate determination of the seismic resistance

The next step is to calculate the basic seismic resistance, V_{R0}, of the members of the ground floor by the following equation [1]:

$$V_{R0} = a_1 \sum V_{Ri}^{columns} + a_2 \sum V_{Ri}^{walls} + a_3 \sum V_{Ri}^{short\ columns} \tag{3}$$

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 and α_1, α_2 and α_3 are values that can be taken according to the proposed method as [1]:

$\alpha_1 = 0.5$, $\alpha_2 = 0.7$, $\alpha_3 = 0.9$ in structures with columns, walls and short columns
$\alpha_1 = 0.7$, $\alpha_2 = 0.9$ in structures with columns and walls but without short columns
$\alpha_1 = 0.7$, $\alpha_3 = 0.9$ in frame structures without walls, and with short columns
$\alpha_1 = 0.8$ in frame structures without walls and short columns

The examined RC structure has columns and walls, but not short columns. Therefore, $\alpha_1 = 0.7$ and $\alpha_2 = 0.9$.

In the case that the amount of reinforcement of the examined building is equal to minimum values, the strength of the vertical members, V_{Ri}, is obtained by:

$$V_{Ri} = \min[(V_{Rd,s}, \mathrm{V}_{R,max}), V_M] \tag{4}$$

where the shear strengths $V_{Rd,s}$ and $\mathrm{V}_{R,max}$ are obtained by EC2 or KANEPE (or similar to EC8-3) [3,4,5] and V_M is the flexural strength and is equal to $V_M = M_R/L_s$, where L_s is obtained by KANEPE [4].

In the case that the amount of longitudinal and transverse reinforcement is being ignored, i.e. equal to zero, the strength of the vertical members, V_{Ri}, is obtained by:

$$V_{Ri} = \min(V_{Rd,s}, \mathrm{V}_{R,max}) \tag{5}$$

where calculations are made considering zero total longitudinal reinforcement ratio $\rho_{tot} = 0$, zero contribution of transverse reinforcement to shear resistance $V_w = 0$ and the plastic part of chord rotation ductility factor $\mu_\theta^{pl} = 1$. In this case, the bending contribution is not taken into account.

The final seismic resistance, V_R, is defined by the following equation [1]:

$$V_R = \beta\, V_{R0} \tag{6}$$

Among the 13 criteria mentioned in the introduction of this paper, the ones that most affected the reduction factor value were: i) the normalized axial load with the maximum value on the center column (K4) of the structure and with the criterion grade equal to $\beta_3 = 4$, ii) the stiffness distribution in plan – torsion with the criterion grade equal to $\beta_5 = 1$, and iii) the stiffness distribution in elevation with the criterion grade equal to $\beta_7 = 3$. The reduction factor resulted in $\beta_x = 0.81$ and $\beta_y = 0.83$ for the x and y directions, respectively. In Table 3, the basic (V_{R0}) and the final (V_R) seismic resistance of the vertical members of the ground floor are presented, for each assumption of the reinforcement amounts and seismic direction.

Seismic Direction	Total strength	Ignoring Reinforcement	Minimum Reinforcement
x	Basic V_{R0} (kN)	1628	1513
	Final V_R (kN)	1319	1226
y	Basic V_{R0} (kN)	1928	2006
	Final V_R(kN)	1600	1665

Table 3: Seismic resistance V_{R0} and V_R of the vertical members according to the proposed method.

3.3 Determination of the safety index

Finally, the last step is to calculate the safety index, λ, for each direction, without taking into account the effect of the transverse direction, using the following equation [1]:

$$\lambda = \frac{V_{req}}{V_R} = \frac{V_{req}}{\beta\, V_{R0}} = \frac{\lambda_0}{\beta} \tag{7}$$

In Table 4, the final safety indices are presented, which were calculated for performance level B, using the empirical and analysis period, for the assumptions of ignoring and accounting for minimum reinforcement amounts.

Seismic Direction	Ignoring Reinforcement		Minimum Reinforcement	
	Empirical T	Analysis T	Empirical T	Analysis T
x	5.01	1.38	5.38	1.48
y	3.05	1.20	2.93	1.15

Table 4: Safety indices of the proposed approximate method for ignoring and accounting for minimum reinforcement amounts.

From Table 4, the high influence of the period assumption on the calculated safety indices can be observed. The use of the empirical period in the calculations results in 3-4 times higher safety indices than the ones resulted from the use of the more accurate period (i.e. analysis period). Moreover, the examined assumptions for the existing reinforcement (ignoring or not) resulted in quite similar safety indices values. Differences between them did not exceed 10%.

In the next section, in order to check the above results use is made of the more accurate non-linear static analysis method, for the assessment of the same building.

4 APPLICATION OF THE NON-LINEAR STATIC ANALYSIS METHOD

In order to validate the proposed method, the non-linear static analysis method is employed. This method, also known as pushover analysis, is a widespread method used to evaluate and redesign old buildings, as well as to design new ones. A significant advantage of the method versus approximate methods is the ability to simulate with high accuracy the inelastic behavior of members through a stress-strain (or force-displacement) diagram.

The modeling of the building, i.e., beams, columns and walls, was made according to KANEPE [4] with frame elements with plastic hinges at their ends, using the finite element software SAP2000 [6]. All cross-sectional data, i.e. yield moment, axial force and moment interaction, curvature and chord rotation angle, were calculated using an appropriate cross-section analysis tool [7].

The mechanical behavior of the structural elements was described by a force-displacement diagram. This behavior is defined at the two ends of each member (plastic joints). In this work, all members were modeled to account for flexural failure based on the bending moment-chord rotation, $M-\theta$, relationship (Figure 3). Moreover, for columns and walls, the bending moment-axial load, $M-N$, interaction was taken into account, while for beams this axial force was assumed to equal zero. The Π-shaped shear wall (elevator) was modeled by three frame-elements with dimensions of 3 x 0.25 m that were appropriately connected to each other with rigid elements.

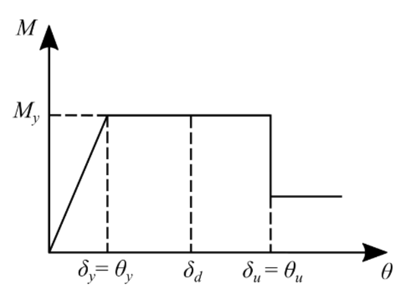

Figure 3: Capacity curve in terms of $M-\theta$.

The three performance levels identified by KANEPE and EC8-3 [4,5] are the following: Damage Limitation or Level A, Significant Damage or Level B and Near Collapse or Level C. The limits of each performance level are defined as a function of the deformation value, $\delta_d = \theta$, of the members, where θ is the chord rotation. These limits are obtained according to the following equations of KANEPE [4]:

$$\begin{gathered}\delta_d = \delta_y, \text{ for the performance level A}\\ \delta_d = \left[0.5\left(\delta_y + \delta_u\right)\right]/\gamma_{Rd}, \text{ for the performance level B}\\ \delta_d = \delta_u/\gamma_{Rd}, \text{ for the performance level C}\\ \gamma_{Rd} = 1.5\end{gathered} \tag{8}$$

where δ_y is the yield deformation, δ_u is the ultimate deformation and γ_{Rd} is a partial safety factor.

In this paper, the results will be demonstrated only for the Significant Damage performance level (Level B) due to space limitations.

A representative model of the building is shown in Figure 4.

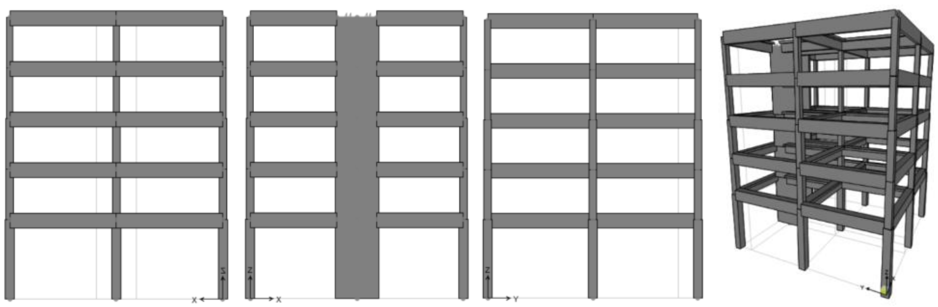

Figure 4: Sections along the x and y directions and 3D model view.

4.1 Determination of seismic resistance and seismic demand

In this paper, two alternative ways are used to determine the seismic resistance of the whole structure. In the first one (Local definitions), the maximum resistance of the structure is defined when one vertical element reached the maximum acceptable deformation for the examined performance level. In the second one (Global definitions), the maximum resistance of the structure is defined through Eqs. (8), where δ_y and δ_u are defined by linear approximation of the capacity curve of the building as shown in Figure 5.

Figure 5 shows the shear force-displacement curve resulting from the non-linear static analysis, for the x and y directions. This curve is appropriately linearized according to KANEPE [4], i.e. converted into two straight lines. The yield displacement, δ_y, is considered as the point in which the first failure in one vertical member occurs, i.e. the first exceedance of the performance level A for local and global values. The local ultimate displacement, δ_u, is considered as the point in which the maximum limit of the performance level C is exceeded, and the global ultimate displacement δ_u is considered as the point in which the maximum base shear force is achieved. In Figure 5, only the local and global limits of the performance level B are shown.

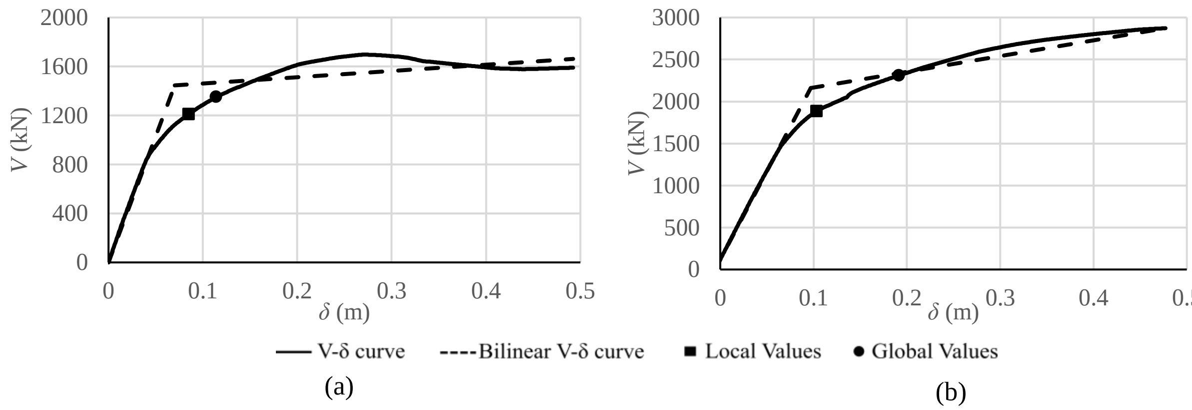

Figure 5: Shear force-displacement curve (a) x and (b) y directions.

In Table 5, the displacement and base shear force acceptable limits, $\delta_{lim,B}$, and, $V_{lim,B}$, respectively, of performance level B are presented.

Damage	Seismic Direction	$\delta_{lim,B}$ (m)	$V_{lim,B}$ (kN)
Local	x	0.085	1210
	y	0.103	1884
Global	x	0.114	1352
	y	0.195	2310

Table 5: Seismic resistance for performance level B in terms of deformation and base shear force.

The seismic demand, V_{req}, is determined according to the design spectrum for each direction and is defined by Eq. (2). In this case, the behavior factor q is obtained by the shear force-displacement curve, which resulted from the non-linear analysis, as the ratio of the ultimate shear force to the yield shear force and is equal to $q_x = 1.51$ and $q_y = 1.56$ for the x and y directions, respectively. Finally, the seismic demand for the non-linear analysis resulted in $V_{req,x} = 2042$ kN and $V_{req,y} = 2837$ kN for the x and y directions, respectively.

4.2 Safety indices of the non-linear static analysis

The safety indices of the non-linear static analysis are defined in two different ways: a) based on the base shear force and b) based on the displacement. Both ways refer to local and global definitions.

For the 1st case, the safety index, λ, for each direction is defined as follows:

$$\lambda = \frac{V_{req}}{V_s} \tag{9}$$

where V_{req} is obtained using the behavior factor which was calculated as described in section 4.1 and $V_s = V_{lim,B}$ is the base shear force and is obtained from the shear force-displacement curve of the non-linear analysis depending on the performance level being examined. For performance level

B, the values of V_s are presented in Table 5. The safety indices are found using the local and global definitions and are indicated in this paper as Force Local Values (FLV) and Force Global Values (FGV), respectively.

For the 2nd case, the safety index, λ, for each direction is defined as follows:

$$\lambda = \frac{\delta_t}{\delta_d} \tag{10}$$

where δ_t is the target displacement calculated according to KANEPE [4] using only the analysis period of the building and $\delta_d = \delta_{lim,B}$ is the acceptance limit of the examined performance level in accordance with KANEPE [4]. The safety indices are found using the local and global definitions and are indicated in this paper as Displacement Local Values (DLV) and Displacement Global Values (DGV), respectively.

In Table 6, the final local and global safety indices of the non-linear static analysis are presented, which were calculated for performance level B and based on the base shear force and displacement, using the empirical and analysis period.

Damage	Seismic Direction	Base shear force		Displacement	
		Empirical *T*	Analysis *T*	Empirical *T*	Analysis *T*
Local	x	6.18	1.69	0.74	3.12
	y	3.81	1.51	0.60	1.70
Global	x	5.53	1.51	0.58	2.32
	y	3.10	1.23	0.31	0.91

Table 6: Local and global safety indices of the non-linear static analysis based on the base shear force and displacement.

From Table 6, large differences between the results from the empirical and the analysis period can be observed, as in Table 4. Here, the empirical period is used only for comparison purposes, because the use of non-linear analysis methods implies that the more accurate period (i.e. the analysis period) can be easily calculated and hence used for the analyses.

5 RESULTS COMPARISON

This section presents comparisons between the approximate and the more accurate non-linear analysis method, in the form of diagrams. Figures 6 to 8, present the ratios $\rho = V_{approximate}/V_{analysis}$ and $\varepsilon = \lambda_{approximate}/\lambda_{analysis}$, indicating the ratio of the approximate to the accurate method for the seismic resistance and the values of the safety indices, respectively. The cases near to unity indicate that the results of the approximate and the accurate methods are close. The seismic resistance of the members as resulting from the approximate method is calculated on the basis of the following assumptions: a) ignoring the existence of the reinforcement, i.e. zero amounts of longitudinal and transverse reinforcement and b) taking into account the minimum reinforcement amounts. On the other hand, the seismic resistance of the members for the accurate method is calculated by taking into account only the minimum reinforcement amounts because it is impossible to do the analyses with zero reinforcement.

The indicators of Figures 6 to 8, i.e. FLV, FGL, DLV and DGV represent the cases of the non-linear analysis as described in section 4.2 of the present study, which were compared to those of the proposed method. All the results refer to the use of the empirical and analysis period of the structure.

5.1 Seismic resistance

Figure 6 presents the ratio ρ of the seismic resistance resulting from the approximate and the accurate methods, for the x and y directions. It can be observed that the ratio ρ ranges from almost 0.7 to 1.1. Generally, the seismic resistance of the structure resulting from the proposed method appears to be in quite good agreement with the respective ones obtained from the non-linear analysis, as all cases are quite close to unity. It appears that higher accuracy is achieved when the local definition of the safety index is used (FLV case).

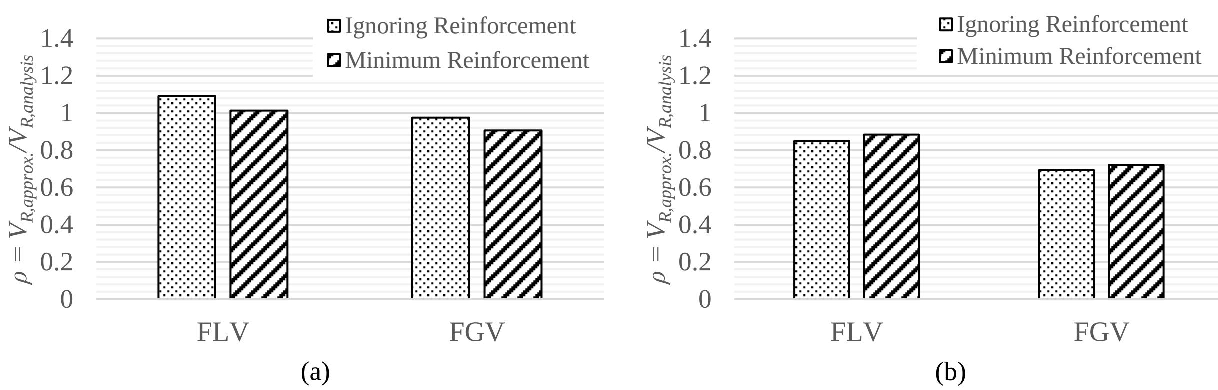

Figure 6: Ratio ρ for the seismic resistance for a) x direction and b) y direction.

Figures 7 and 8 demonstrate the ratio ε of the safety indices resulting from the approximate and the accurate methods, for the x and y directions and for the empirical and analysis period, respectively.

From Figure 7, it can be observed that the ratio ε ranges from almost 0.8 to 1.0. Moreover, it can be seen that higher accuracy is achieved when comparing the proposed method with the global values of the non-linear analysis (FGV case). Using the empirical period of the structure, it is quite clear that the results of the proposed method, for both assumptions of the reinforcement amounts, are very close to the ones obtained by the base shear force of the non-linear analysis. This is because, in contrast to displacements, forces are not so sensitive to the stiffness (or period) assumption.

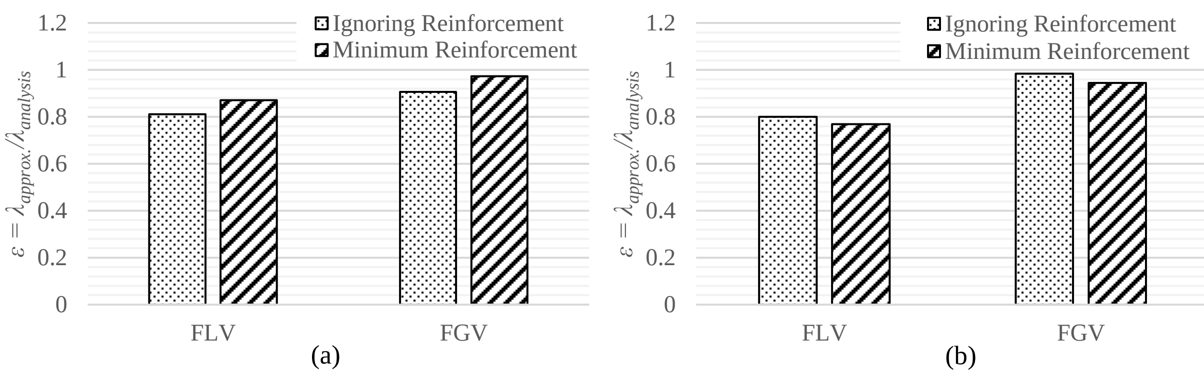

Figure 7: Ratio ε for the safety indices – Empirical period- a) x direction and b) y direction.

From Figure 8, it can be seen that the ratio ε ranges from almost 0.4 to 1.0. Moreover, it can be observed that higher accuracy is achieved when comparing the proposed method with the global values of the non-linear analysis based on the base shear force (FGV case). On the other hand, when comparing the proposed method with the values of the non-linear analysis based on the displacement (DLV and DGV cases), the two methods have great differences. Using the analysis period of the structure, it is quite clear that the results of the proposed method, for both assumptions of the reinforcement amounts, are very close to the ones obtained by the base shear force of the non-linear analysis.

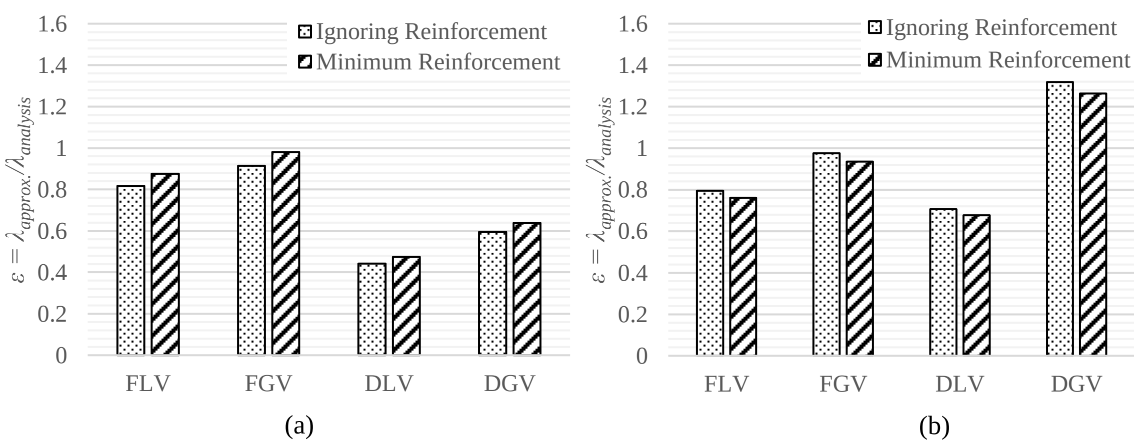

Figure 8: Ratio ε for the safety indices – Analysis period- a) x direction and b) y direction.

6 CONCLUSIONS

In this study, a new approximate method for the assessment of RC buildings proposed by EPPO (Earthquake Planning and Protection Organization of Greece) has been demonstrated through a realistic application on a RC building and validated via non-linear static analysis. This method simplifies the assessment procedure, while it can also deal with the problem when a building's reinforcement information is unknown. When comparing the results of both methods, the following conclusions can be drawn:

- The seismic resistance of the structure which resulted from the proposed method appears to be in quite good agreement with the respective ones obtained from the non-linear analysis, with the best results obtained when the local definition of the safety index was used.
- Examined assumptions for the existing reinforcement (ignoring or not) result in quite similar safety indices values. Differences between them did not exceed 10%.
  Furthermore, the results were found very close to those obtained from the non-linear analysis when considering the seismic capacity in terms of base shear, and a higher accuracy can be observed when the global definition of the safety index was used.
  The same degree of accuracy was not found when indices were defined through the deformation capacity of the structure. However, it is worth noticing that also in this case, the global definition of the safety index results in better accuracy than when the local definition was used.

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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, 2018.

[2] EC8, Eurocode 8 Part 1, *Design of structures for earthquake resistance, general rules, seismic actions and rules for buildings*. European Standard EN 1998-1, European Committee for Standardization (CEN), Brussels, 2004.

[3] EC2, Eurocode 2 Part 1.1, *Design of concrete structures: General rules and rules for buildings*. European standard EN 1992-1-1, European Committee for Standardization (CEN), Brussels, 2004.

[4] 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.

[5] EC8, *Eurocode 8 Part 3, Design of structures for earthquake resistance: Assessment and retrofitting of buildings*. European Standard EN 1998-3, European Committee for Standardization (CEN), Brussels, 2005.

[6] SAP2000, *Static and Dynamic Finite Element Analysis of Structures,* Computers and Structures Inc, Berkeley, California, 2010.

[7] M. G. Sfakianakis, *Software Manual*, Patra, 2015.