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# FLOWCHART OF ASSESSMENT STUDIES OF SEISMIC CAPACITY ABOUT HELLENIC R/C BUILDING USING EUROCODE EN 1998-3

Triantafyllos K. MAKARIOS[1], Athanasios P. BAKALIS[2]


## ABSTRACT

The present paper aims, on the one hand, to formulate a suitable flowchart to assess the seismic capacity of asymmetric reinforced concrete (r/c) buildings using Eurocode EN 1998-3 and, on the other hand, to exploit the results of the assessment about the seismic capacity of the Hellenic r/c buildings. In order to investigate the abovementioned targets, Eurocode EN1998-3 is applying on a set from real, multi-storey, asymmetric, r/c buildings, which had been designed by old Hellenic Codes in various eras over the last fifty years. All the buildings examined were found to be structurally inadequate, according to the seismic performance matrix of EN 1998-3. More forward, the abrogation of inhered inabilities of the buildings, with added new structural members or strengthening of the initial ones with reinforced concrete jackets or combination of the previous two methods, was imperative. All demand checks by EN 1998-3 were included in a flowchart which is missing from the code. This flowchart is applied with absolute success in 17 r/c buildings in the frame of a non-financial Investigative Program. From the results of nonlinear analyses, a mean elastoplastic-perfect diagram between the effective Peak Ground Acceleration and the average Drift of the profile in elevation of the "as strengthened" r/c buildings is emerged. Based on this diagram, we can pre-estimate the final results of a (flowchart) retrofit-study on building's seismic-capacity using only linear analyses. This information is very important for the building's owner, before the detailed retrofit-study, to understand the effectiveness of the building strengthening scenario.


*Keywords: Non-linear Static Analysis (Pushover); Assessment process of EN1998-3; Assessment process of KAN.EPE.; Capacity curve; Performance Levels; Seismic Capacity*

## 1. INTRODUCTION

The first revision of the Hellenic Code of Structural Interventions (KAN.EPE.) was made in 2013 and in the same year, the optional application of Eurocodes, including Eurocode EN 1998-3 corresponding to KAN.EPE., was also introduced in our country. In the year 2017, the second revision of KAN.EPE. came into force. Both regulations provide the Civil Engineer with the necessary elements needed to evaluate the seismic performance of existing individual buildings and to prepare a design study of retrofitting measures. However, what is missing from the two above-mentioned regulations is an appropriate flow diagram of the study, which, if it existed, would certainly increase the perception of the average Civil Engineer who is called upon to apply them in a perfect way. This is what the present work attempts to cover, using the experience gained from the application of the provisions of the above regulations to a series of real, high-rise, asymmetric reinforced concrete buildings in Greece that had been studied under the Hellenic Regulations in various periods over the last few decades. In all buildings, their static deficiency was demonstrated in accordance with the requirements of the Seismic Performance Matrix set out in EN 1998-3 (paragraph 2.1.(1)P and note in 2.1.(3)P) and then (at study level) the inherent weaknesses of the buildings were removed, either by adding new structural stiffness and resistance elements (usually walls), or by strengthening the original sections with reinforced concrete jackets, or by combining the two previous solutions. All the required checks were integrated

[1]Professor (Asst), Institute of Structural Analysis and Dynamics of Structure, School of Civil Engineering, Aristotle University of Thessaloniki, GR-54124, Greece, makariostr@civil.auth.gr
[2]Dipl. Eng. MSc, Institute of Structural Analysis and Dynamics of Structure, School of Civil Engineering, Aristotle University of Thessaloniki, GR-54124, Greece, abakalis@civil.auth.gr

into a single flowchart (Figure 1) that is presented. The present paper also provides aggregate results from the assessment of the seismic capacity of buildings. These results demonstrate the effectiveness level of EN 1998-3 provisions and thus, useful conclusions can be drawn for the retrofitting of these buildings.

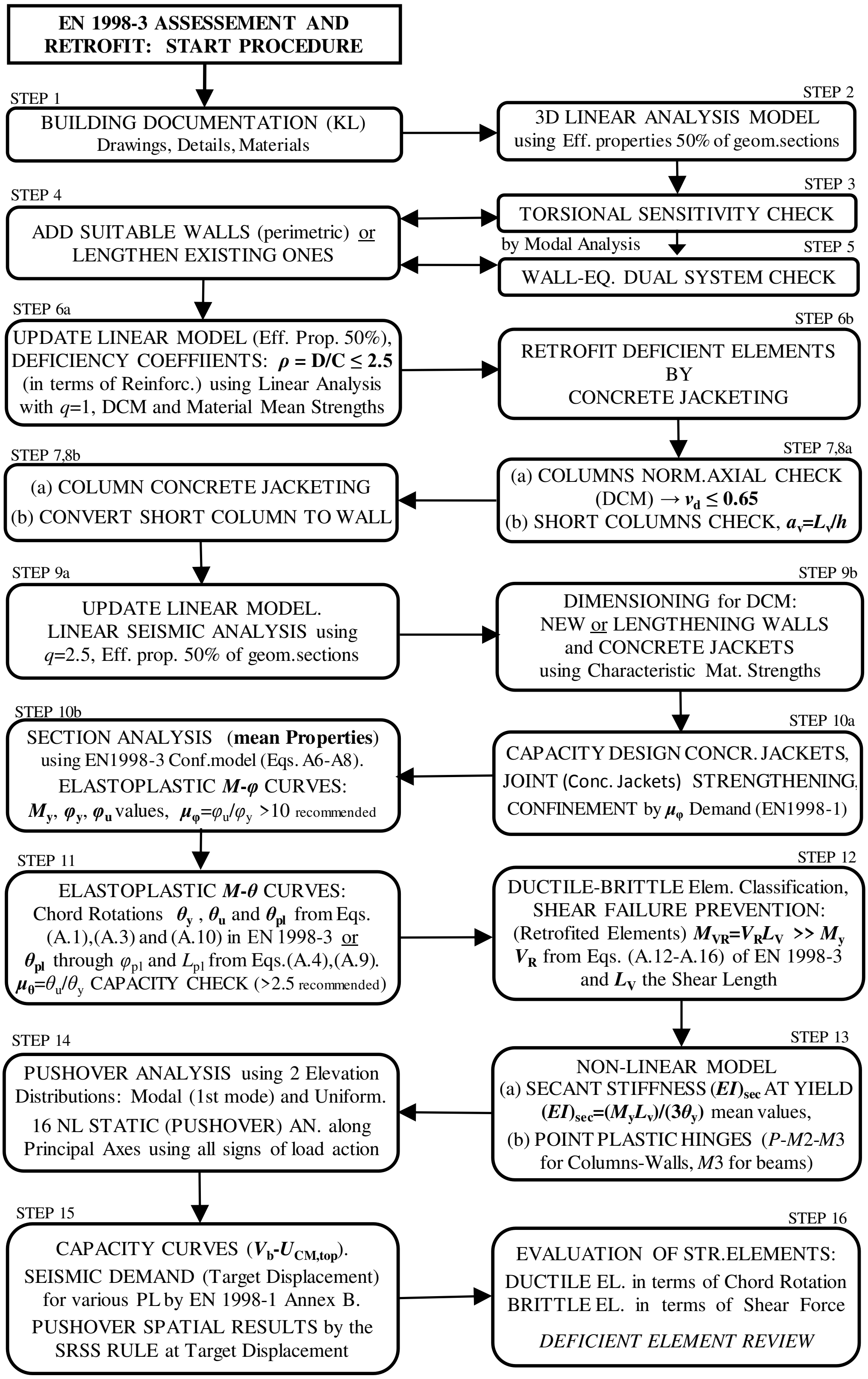



Figure 1. Flowchart of assessment and retrofit procedure of Eurocode EN 1998-1.

## 2. METHODOLOGY

In the frame of an internal, non-financial, Investigative Program that took place at the Institute of Structural Analysis & Dynamics of Structures, Division of Structural Engineering, School of Civil Engineering of Aristotle University of Thessaloniki, a series of Diploma Theses have been drafted over the last three years, all of which had the same goal, while only the study object was changing (Athanasiadis 2016, Valanos 2016, Varsamis 2016, Gkiokas 2016, Loutroukis 2015, Lolos 2016, Migkos 2016, Pilavaki 2016, Sarantou 2016). Each Diploma Thesis had as its main objective the assessment / retrofitting of an existing multi-storey asymmetrical r/c building under Eurocode EN 1998-3, but differed from each other in respect to each building under consideration. Given that no flowchart is proposed both from EN 1998-3 and KAN.EPE. (2nd Revision), in the Diploma Theses we came up, after further consideration, on a new flowchart (Figure 1) that is as much as possible compatible with the provisions of the abovementioned regulations. This flowchart had the following key points:

1. A theoretical estimate of the actual quality of concrete and its corresponding (mean) modulus of elasticity as well as of the class of the reinforcing steel was made for each investigated building, since our investigation was primarily directed to determine the computational path and not a real retrofit study with immediate application. Consequently, neither destructive nor other *in-situ* methods of detecting material quality (e.g. core sampling, ultrasonic and rebound hammer methods, bolt pull out, degree of carbonation control, magnetic detection of reinforcement layout and details, etc.) were used, and no visual identification of the reinforcing steel was made. Both the actual quality of the concrete and its modulus of elasticity were theoretically estimated on the basis of the proposed relations of paragraphs 3.1.2 (6) and 3.1.3(2)&(3) of the Eurocode EN 1992-1-1. Additionally, the reinforcing steel class was determined from the original design drawings and it was considered that what is shown in the design drawings and seismic study papers is correctly constructed and therefore there is Full Knowledge of the data of each building, i.e. we are in the KL3 according to EN 1998-3. Obviously, in the case of a real study, the data documentation process proposed by the regulations should be followed. Also, it is worthily note that in the second revision of KAN.EPE. (Annex 3.1) the designer can use "default" characteristic strength material values for the concrete and steel reinforcement under certain conditions but in this case the Knowledge Level is Low.

2. The development of a 3D analysis model for each asymmetric multi-storey building is implemented based on the Eurocode EN 1998-1 modeling instructions, retaining the same cross sections of the original static design and the gravitational loads of the seismic combination $G + 0.3 \cdot Q$, where $G$ is the permanent gravity load and $Q$ is the corresponding live. It is noted that in the case of beam supporting discontinued (cut-off) vertical elements, the mass of the column is imported in the beam span as concentrated mass (with the vertical degree of freedom enabled) and with increased value to give the continuous distributed mass of the beam from its direct bearing gravity load. It is obvious that since we have in the model masses with enabled vertical degrees of freedom, the vertical seismic component must be considered in calculations, regardless if it is mandatory or not.

3. Using reduced bending and shear stiffness at 50% of the geometric cross-sections (see paragraph 4.3.1(7) of EN 1998-1), a modal analysis was performed for each asymmetric multi-storey building to determine whether it is "torsional flexible" or not. The previous check is accomplished using the most exact criterion of the Dynamics of Structures according to which if under torsional loading about the vertical axis, the largest proportion of the effective mass moment of inertia (about the vertical axis) occurs in one of the first two modes of the building, then the latter has a "torsional sensitivity".

4. In case the examined building has a "torsional sensitivity," we decide to place walls in sufficient layout and in appropriate positions or to lengthen the original walls (if they exist), in order to remove the "torsional sensitivity" and this is controlled by a repeated process (with tests). It is noted that if this requirement is met, i.e. the building is assumed to have a small rotation about vertical axis, then we can approximately and well documented apply the Annex B of EN 1998-1

to assess the seismic target displacement since the scientific documentation of this Annex refers only to multi-story planar frames. Indeed, according to paragraph 4.3.3.4.2.1(3) of EN 1998-1, it is stipulated that in torsional stiff buildings, two planar models can be used, one for each principal horizontal direction, thus paving the way for the application of Annex B EN 1998-1.

5. It is checked whether the building can be classified as a "wall-equivalent dual system" based on paragraph 5.1.2 of EN 1998-1. According to EN 1998-1, a building is classified as "wall-equivalent dual system" if the shear resistance of the walls at the building base is between 50% and 65% of the total seismic resistance of the whole structural system. If additional walls are required (in relation to the fourth step), then we insert new walls or increase the length of those of the previous step (again with tests). Once this is implemented, then we are exempted from the mandatory insertion of the masonry infills in the model according to paragraph 5.9.1 of KAN.EPE. and this is particularly important for the calculation cost of the study. On the contrary, if the building cannot be described as "wall-equivalent dual system", then, on the one hand, the modeling of the masonry infills is mandatory (although, ultimately, only a few or reduced strength masonry infills will be inserted into the model due to their openings or to their slenderness (Bakalis 2013)) and, on the other hand, the non-linear analysis provides large horizontal displacements. This is due to the fact that the non-linear building model is very flexible since its elements have been supplied with their (very low) secant stiffness $EI_{\text{sec}}$ at yield. This sometimes triggers second-order phenomena (P-D) and need to address the associated problems.

6. After the aforementioned wall adequacy test carried out in the previous two steps, and after the above necessary walls are inserted into the building model, we proceed to the calculation of coefficients of deficiency $\rho$ (paragraph 4.4.2(1)P of EN 1998-3) or $\lambda$ (paragraph 5.5.1.1 of the KAN.EPE.) which are equal to the ratio $D_i/C_i$ between the demand obtained from a linear analysis under the load seismic combination and the corresponding capacity for the $i$-th ductile primary element of the structure. For this purpose, it is necessary, according to the above regulations, to proceed with the analysis of the building and its «theoretical» dimensioning according to current code requirements (EN 1998-1), for both vertical gravitational loads and the design earthquake using a behavior factor $q$ = 1.00, as is defined explicitly in paragraph 4.4.2.(2)P of EN 1998-3 or 5.5.1.1 of KAN.EPE. Additionally, mean value properties of the existing materials must be used (EN 1998-3 par. 2.2.1(4)P and KAN.EPE. par. 5.5.1.1) suitably modified according to the data reliability levels (KL). Then the above deficiency coefficient $\rho$ or $\lambda$ will be calculated in terms of reinforcement quantities. It should be noted that this analysis and dimensioning must be done in the model that already includes the necessary walls, to consider the redistribution of the stress on the various structural elements due to the insertion of the above walls. The requirements for thorough dimensioning are recommended to be taken for the medium ductility class (DCM) defined in EN 1998-1, as the building is existing and therefore there are inherent weaknesses, and consequently it is not possible to meet the stringent requirements of the high ductility class (DCH) that are defined by EN 1998-1 for new constructions. From this check, the above coefficient of deficiency $\rho$ or $\lambda$ is calculated for each critical cross-section. For each beam critical cross-section, the coefficient $\rho$ is calculated twice, for tension of both the upper and lower beam fibers.

   Since the maximum value of $\rho$ or $\lambda$, indicating when the strengthening of a structural member is mandatory, is not defined by the regulations, the question arises as to what that value may be. However, in the comments of paragraph 5.5.1.1 of KAN.EPE., it is explicitly stated that a value of $\lambda > 4$ indicates a "clear deficiency" while in paragraph 5.5.2.a is stated that if $\lambda > 2.5$ it is forbidden to use linear static or dynamic analysis. The latter is also stated in paragraph 4.4.2(1)P of EN 1998-3 for the ratio $\rho_{max}/\rho_{min}$, if we assume $\rho_{min} = 1$. It should be noted that "the global behavior $q$-factor approach" of EN 1998-3 and of KAN.EPE. are the only linear (static or dynamic) methods that allow a global behavior factor greater than one. In the other two linear analysis methods of EN 1998-3 (lateral force analysis and modal response spectrum analysis) and of KAN.EPE. (elastic static and dynamic analysis with local ductility factors $m$) the global behavior factor that is used is one (Bakalis 2013). Also, in non-linear analysis methods, there is no sense of the behavior factor. Therefore, considering also the paragraphs 4.2(3)P and 2.2.2(3) for the Near Collapse (NC) limit state of EN 1998-3 and having in mind that in the strengthened building all requirements for both the ductile and brittle members will be satisfied, the maximum global behavior ($q$) factor that can be used in a linear method of analysis, in order to design the retrofitting of a building, cannot exceed 2.50, otherwise it would violate the $\rho$ or $\lambda$ <2.50 which should normally be applicable to each primary element of the structure. Therefore, indirectly we conclude that, in this case, the maximum acceptable value of $\rho$ or $\lambda$ is 2.50 which we will use in order to obtain the minimum possible sectional stress for the dimensioning of the reinforced concrete jackets and of the new added or original lengthening elements (walls).

7. The check of the normalized axial force ($v_d$), in the columns that were not found inadequate in the previous step, is assumed to be valid if $v_d$ <0.65 as defined in paragraph 5.4.3.2.1(3)P of the EN 1998-1 for the DCM. If a column does not satisfy this check, a reinforced concrete jacket must be used to prevent cross-column lateral dilation due to the increased compressive axial force.
8. The short columns (by location or nature) are checked. Frequently, for each planar frame considered, the conversion of at least one short column into a wall can degrade the problems encountered as it relieves the rest due to the active limitation of the horizontal diaphragm displacements. However, it should be noted that when the building is designed as a "wall-equivalent dual system" according to paragraph 5.1.2 of EN 1998-1, the phenomenon of short columns development is degraded.
9. The reinforced concrete jackets for the strengthening of the (deficient) structural members and the new added (or lengthened) walls are dimensioned. To achieve this, the intended concrete jackets must be inserted into the building model (which already has the additional walls), provided of course that they are "mantles" of reinforced concrete which entails changing the cross-sections of the structural elements (as opposed to the case where fibre-reinforced polymers (FRP) are used and therefore the cross-sections are not changed). Then, a linear seismic analysis of the building with a maximum global behavior factor $q$=2.50 (see step 6) is performed using the characteristic strength values of the materials. The results of this analysis are the minimum stress levels for dimensioning the reinforced concrete jackets and the new added (or lengthened) walls.
10. Both the new strengthened cross-sections and the initial cross-sections that did not show any deficiency were modeled with fiber elements in order to calculate the Moment-Curvature ($M$-$\varphi$) diagram of the cross-section (considering any possible bending plane) and the spatial diagram $P$-$M_3$-$M_2$ in biaxial bending with axial force. It is noted that the new hoops to be placed in the reinforced concrete jackets of the columns should satisfy the EN 1998-1 Equation (5.15) of the minimum mechanical volumetric ratio of the confinement steel in the critical cross-sections and since only perimetric hoops can be placed in the concrete jackets, sometimes Ø12/80 (mm) confinement hoops of class B500c or more must be placed. Accordingly, for the extreme columns at the edges of the walls, the Equation (5.20) of EN 1998-1 must be satisfied. It should be noted that the confinement hoops of reinforced concrete jackets must be known because they strongly affect the confinement concrete model of columns to be used immediately below. Thus, the examined cross-section must be separated into an unconfined and a confined concrete region, while there is also a question on whether (or not) to consider the original column cross-section to be confined (or unconfined). The latter usually has sparse hoops (not closed in most of old buildings) but now is encapsulated by the concrete jacket and therefore cannot be deformed transversely. In this phase, the mean material properties (suitably modified according to the data reliability levels - KL) are used, as well as the proposed model of confined concrete from Equations (A.6-A.8) of EN 1998-3 which are quite similar with the Equations (6.16-6.18) of the second revision of KAN.EPE.

   The Moment-Curvature ($M$-$\varphi$) diagram must be converted to an equivalent elastoplastic-perfect by applying a reliable procedure (e.g. by Caltrans, to maintain the stability of the calculations in the non-linear analysis) from which the mean moment at yield $M_y$ and the mean curvature at yield $\varphi_y$ can graphically be found (which reflect the yield of the entire cross-section and not the beginning of yield of a steel reinforcement bar) as well as the available curvature ductility $\mu_{\varphi} = \varphi_{u}/\varphi_{y}$. Here, care should be taken to ensure that there is sufficient curvature ductility (that is, as a recommendation, we can be satisfied if $\mu_{\varphi} > 10$ although this limit is not mandatory for every cross-section case, especially for walls).

It is also recommended to use the capacity design procedure (in flexure and shear) for the dimensioning of all the concrete jackets in the vicinity of joints so that the plastic hinges always appear on the beams and not the columns. To accomplish this, the reinforced concrete jackets of all the beams should first be dimensioned, and then, having known the strength moments of the beams, the dimensioning of the reinforced concrete jackets of the columns should follow. Also, the joints of the building from which column or beam concrete jackets begin (concrete jackets joints) should be suitably strengthened to be able to transfer the growing stress from the deformation of the beams to the columns. Finally, it is clarified that the beam concrete jackets can be restrained at the ends of the beams or, in the case of beams supporting columns, in the vicinity of a discontinued (cut-off) supporting column, provided that the proper anchorage length of the reinforced concrete jackets is available (at least as long as twice the critical length of the beams, i.e. $2h_{b}$, where $h_{b}$ is the height of the cross-section of the beam).

11. The elastoplastic-perfect capacity diagram of Moment-Curvature (*M-φ*) is converted into an elastoplastic-perfect capacity diagram of Moment-Chord Rotation (*M-θ*), where chord rotation $\theta$ is defined as the slope of the shear length $L_{v}$ of the examined structural member end-section, i.e. $\theta = \delta/L_{v}$, where $\delta$ is the displacement of the "free end" of the shear length $L_{v}$ assuming that as an ideal cantilever. The available chord rotation $\theta_{y}$ at yielding and the available chord rotation $\theta_{u}$ at ultimate are determined by using appropriate Equations in EN 1998-3 (A.10-A.11 & A.1-A.3) and KAN.EPE. while the difference gives directly the available plastic chord rotation $\theta_{pl} = \theta_{u} - \theta_{y}$. The later is theoretically equal to the available plastic rotation taking place within the plastic hinge length $L_{pl}$, i.e. theoretically equal to the integral of the plastic curvatures $\varphi_{pl}$ in the plastic hinge length as can be calculated indirectly by the Equation (A.4) of EN 1998-3. The appropriate plastic hinge length $L_{pl}$ is given by the Equation (A.9) of EN 1998-3.

12. A check is performed in order to prevent the possibility of a shear failure over the bending one. The shear strength $V_{R}$ is calculated from Equations (A.12-A.16) of EN 1998-3. Given that the shear length $L_{v}$ is known, which implies that a plastic hinge has already been developed at both ends of the structural element, the moment $M_{V_{R}}$ at the extreme cross-section, when the shear force is equal to the shear strength, is easily calculated, i.e. $M_{V_{R}} = V_{R} \cdot L_{v}$. It must always be true that $M_{V_{R}} \gg M_{y}$, where $M_{y}$ is the end cross-section plasticity moment from the elastoplastic-perfect diagram *M-φ*. It is noted here that the above check always tends towards the safety side in the case where plastic hinges do not develop at the ends of the column (due to the capacity design of concrete jackets, see step 10). Also, in this step, the classification of all structural elements (and behavior mechanisms) to ductile and brittle ones is performed. According to EN 1998-3, the former should be verified in terms of deformations while the latter in terms of strengths. An element is characterized as ductile when all the following apply: $\mu_{\theta} > 2$ or $\mu_{\varphi} > 3$ and $M_{V_{R}} \gg M_{y}$ and $a_{v} = M/(V \cdot h) = L_{v}/h > 2$ according to the notes in paragraph 7.1.2.6 of KAN.EPE.

13. The secant stiffness at yield of an examined element cross-section is calculated from the equation $EI_{sec} = (M_{y} \cdot L_{v})/(3 \cdot \theta_{y})$ in accordance with paragraph A.3.2.4(5) of EN 1998-3. Then, the numerical average of the $EI_{sec}$ values of the two element's end cross-sections for positive and negative bending is considered as the secant stiffness at yield of the structural element. The $EI_{sec}$ value, which is mandatory for all the main building structural elements from the beginning of the non-linear analysis, is considered to be constant over the entire shear length $L_{v}$, which can be approximated to half the clear length of the structural elements under consideration, be it columns or beams (for shear walls, KAN.EPE. in paragraph 7.2.3 states that $L_{v}$ can be taken different in each floor, equal to half of the distance of its base section at each floor until the topmost section of the wall. Also, for beams with one direct supporting end, $L_{v}$

may be taken equal to the total clear span of the beam). Here, of course, there is the contradiction "... why should we use in the model the secant stiffness at yield (and from the beginning of the analysis) in the case where plastic hinges cannot be developed at the ends of the examined column due to the capacity design?". Additionally, as is sometimes shown after the end of non-linear analysis, even in the state of imminent collapse, there are still many structural elements of the building that have not developed plastic hinges at both ends (and in several cases not even one). Thus, by introducing the secant stiffness at yield into all the main structural elements of the building, we obtain a particularly flexible model where the resulting displacements are always larger than the corresponding real ones even in the imminent collapse, while for the "Damage Limitation" performance level the deviation in displacements often appears particularly large. Therefore, for the higher seismic performance levels such as that of "Damage Limitation" and at performance levels that are even earlier (i.e. in the linear region), the non-linear analysis proposed by EN 1998-3 and KAN.EPE. gives very large displacements, sometimes unrealistic.

14. In this step, the necessary nonlinear static analyses of the buildings are performed. For this purpose, the non-linear building model is formed, providing all its elements with their secant stiffness $EI_{\mathrm{sec}}$ at yield and inserting point plastic hinges at their end-sections. According to EN 1998-3, sixteen (16) non-linear static analyses must be performed along the appropriate (main) building directions. This is since we consider the point of application of the lateral static floor load displaced (relative to CM) by the accidental eccentricity $e_{\mathrm{a,i}} = \pm(0.05 \text{ or } 0.10) \cdot L_{\mathrm{i}}$, where $L_{\mathrm{i}}$ is the (main) dimension of the floor plan normal to the direction of the seismic action.
Indeed, we have:

    - Two (2) examined main building directions (e.g. X & Y),
    - Two (2) floor locations of the lateral static loads along each main direction due to the accidental eccentricity,
    - Two (2) signs (+, -) of application of the lateral static floor loads according to paragraph 4.3.3.4.1(7)P of EN 1998-1,
    - Two (2) vertical distributions of the lateral static floor loads (a “modal” one due to the fundamental building mode along the direction under consideration and a “uniform” one with loads proportional to mass regardless of elevation).

    Therefore, $2 \times 2 \times 2 \times 2 = 16$ non-linear static analyses of the “bare” building arise, as we have an exemption (according to paragraph 5.9.1 of KAN.EPE.) from the mandatory reception of masonry infills when we have a "wall-equivalent dual system" based on paragraph 5.1.2 of the EN 1998-1. From each non-linear static analysis, a capacity curve (diagram between Base Shear force and the control node (Top) Displacement) of the building along the direction under consideration is produced. The control node is usually the Mass Centre of the top floor.
It should be noted that, in each solution, the performance of the capacity design of the concrete jackets must be checked because sometimes computational plastic hinges are developed in the columns despite the capacity design of the concrete jackets joints. If this happens then the concrete jackets of the columns must be further strengthened.

    It should also be noted that, according to EN 1998-1, the direction under consideration along which the lateral static floor forces should be applied must be the "main or appropriate" direction of the building (see 4.3.3.4.2.1(3), 4.3.1(5), 4.2.1.3(2), 4.3.3.2.3(2)P, 4.3.3.4.2.2(2)P, 4.3.3.1(7) &(8) of EN 1998-1), but no information is given in EN 1998-1 on how these directions are defined in the case of multi-storey buildings with a random orientation of the vertical structural elements. Also, it’s worth noting that the accidental eccentricity (that locates the lateral static floor loading) is not enough by itself to predict with safety the (real) coupling between the torsional vibrations (about vertical axis) with the translational ones. The previous issues are under investigation and remarkable conclusions have been drawn (Bakalis & Makarios 2017a,b,c).

15. Annex B of EN 1998-1 is implemented to evaluate the results of the 16 non-linear static analyses relative to the seismic demand. It is stressed here that it is necessary to define an ideal equivalent non-linear single-degree of freedom system to documented apply Annex B of EN

1998-1. The lateral stiffness of this ideal SDOF system must be equal to the slope of the first branch of the building's capacity curve, as derived from each nonlinear analysis, and not with less inclination as proposed in Annex B since the secant stiffness at yield ($EI_{\mathrm{sec}}$) was assigned to all structural elements. Using this Annex, typical earthquakes (in terms of peak ground acceleration) that correspond to the three seismic performance levels of the building can eventually arise.

16. Finally, we can take a picture of the stress and displacements that develop due to the spatial action of the two horizontal components of the seismic action. According to paragraph 4.3.3.5.1(6) of EN 1998-1, it is stipulated that, although we are in the non-linear region, we must carry out a SRSS-type combination on the results of the separate non-linear static analyses that are already known, considering all possible combinations between the two seismic components of non-linear analysis. These combinations are implemented in that step of each non-linear static analysis where the target displacement (step 15) is achieved, and from the sixteen combinations -for each vertical distribution of lateral floor forces- the envelope is taken. Of course, combinations are generally banned in the non-linear region, but in this case an exception can be made for the stress values ($M$, $Q$, $N$, $\sigma$) because these values are less than or equal to their yield ones since they have developed in cross-sections that have not entered the non-linear region. However, it is by no means possible to apply the same SRSS combinations to the inelastic displacements and deformations, which are usually deep in the non-linear region. Eurocode EN 1998-1 does not distinguish this severe differentiation. For the latter displacements and deformations, no combination can yield the right results, whereas the use of simple linear superposition (instead of SRSS) is much closer to the real result. On the other hand, the corresponding proposals of KAN.EPE., regarding the spatial action of the earthquake, do not stand up to rational criticism and that's why we are not dealing here further with them.

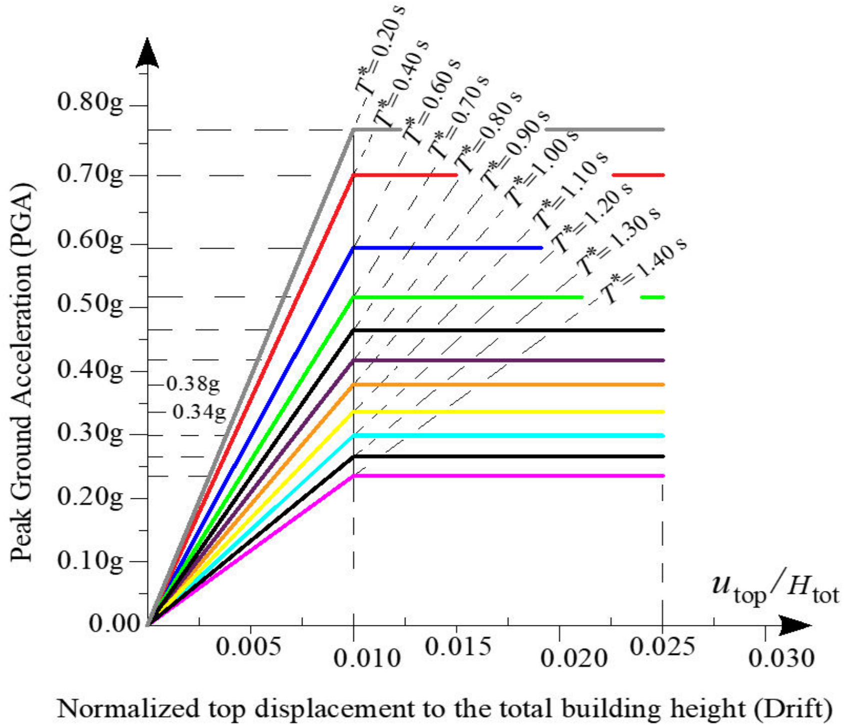
## 3. ANALYSIS

It is well known that in the non-linear static (pushover) analysis of a building, the pushover capacity curve, which is calculated in terms of "Base Shear" and "Top Displacement" of the building, plays the most important role. This diagram, according to Annex B of EN 1998-1, is converted into a "resistance diagram" of an elastoplastic-perfect ideal SDOF system which is supposed to be approximate "sufficiently equivalent" to the original building. This ideal SDOF system is mathematically necessary to be determined, to allow the use of the inelastic acceleration response spectrum in order to calculate the target displacement. The spectra, as is well-known, are defined exclusively in single-degree of freedom oscillators without having meaning in multi-degree of freedom systems if previously the latter do not convert mathematically into an equivalent SDOF system. At this point, it should be stressed that there are currently more precise procedures to define more successful ideal SDOF systems compared to this outdated proposed by Annex B of EN 1998-1 (Makarios 2005, Makarios 2009, Makarios and Asteris 2012, Makarios 2012a,b,c, Makarios 2013). Indeed, among others, an integrated and mathematically documented methodology, about the definition of an elastoplastic-perfect ideal SDOF system that is sufficiently equivalent to the highly asymmetric multi-storey buildings with torsional sensitivity (Makarios 2009, Makarios 2012a), stands out. From this more precise definition, it follows directly, as a partial case, the usual case of the planar multi-storey frames which are dealt with in Annex B of the EN 1998-1. Returning to this research work, we have explored the possibility of developing normalized resistance curves of the examined buildings to have, if possible, global application irrespective of the structure. The proposed normalization (Figure 2) is done by the following procedure:

1. The top displacement $u_{\mathrm{top}}$ of the multi-storey building is divided by the total height $H_{\mathrm{tot}}$ of the building to provide the average Drift ratio ($u_{\mathrm{top}}/H_{\mathrm{tot}}$) in elevation, where the height represents the distance of the last floor from the ground or from the ground floor in the case of an elevated basement and the top displacement corresponds to the displacement of the control node (i.e. the top floor node on which the lateral static force of the non-linear static procedure is applied, approximately CM).

2. Instead of the Base Shear, the peak ground acceleration (PGA) values are used. These values are calculated from the EN 1998-1 elastic acceleration spectrum and correspond to the performance levels of Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). The PGA values are obtained in conjunction with Annex B of EN 1998-1, as shown in the following example.

Provided that we follow the flowchart proposed in this paper, using concrete grade C30/37 and steel B500c for the design of the reinforced concrete jackets and for buildings with a "wall-equivalent dual system" based on paragraph 5.1.2 of the EN 1998-1, it was observed that the dispersion of these normalized resistance curves of the retrofitted buildings is rather small. The yield displacement $u_{\mathrm{top,y}}$ of the Mass Centre at the top of the building ranges from $0.0075 \cdot H_{\mathrm{tot}}$ to $0.0125 \cdot H_{\mathrm{tot}}$, with mean value $0.010 \cdot H_{\mathrm{tot}}$ and does not depend in particular on the age of the original building or on the number of floors, but mainly on the fundamental lateral period of the retrofitted structure (or, equivalent, from the lateral period of the ideal SDOF system -which is equivalent to the retrofitted real building- as defined in Annex B of the EN 1998-1). It is emphasized that the previous mean yield values concern non-linear building models with all their members supplied with their secant stiffness at yield $EI_{\mathrm{sec}}$. If a more accurate non-linear static analysis is performed, considering the step-by-step change of the building stiffness, the mean Drift value at yield of the retrofitted buildings becomes approximately equal to 0.80%. The new diagram is shown in Figure 2 and allows us to know approximately and in advance the final seismic performance of a building by linear calculations only, if we retrofit it according to the Eurocode EN 1998-3 provisions following the flowchart that proposed in this paper. This information is particularly important for the owner of the building who aims at its seismic upgrading.




*Note*: (a) The elements of NL models have been supplied with their secant stiffness at yield $(EI)_{\mathrm{sec}}$.
(b) More accurate NL models (e.g. with Fiber Hinges) provide a mean Drift at yield about 0.80%.

Figure 2. Mean elastoplastic-perfect diagram of the Peak Ground Acceleration (PGA) and the average Drift ratio ($u_{\mathrm{top}}/H_{\mathrm{tot}}$) of the “as strengthened” r/c buildings.

The key point of the diagram (Figure 2) is as follows:

*Data*: Let’s suppose that we have an asymmetric multi-storey r/c building of total height $H_{\mathrm{tot}} = 16$ m and of total lateral mass (of all floors) $m_{\mathrm{tot}} = 1200$ tn $(\mathrm{kN \cdot s^2/m})$. The two-uncoupled fundamental lateral periods along the actual or fictitious principal axes X and Y of the building, after its retrofitting with walls and reinforced concrete jackets, are calculated as $T_{\mathrm{X,unc}} = 0.55\ \mathrm{s}$ and $T_{\mathrm{Y,unc}} = 0.60\ \mathrm{s}$ respectively. The uncoupled fundamental lateral periods $T_{\mathrm{X,unc}}$ and $T_{\mathrm{Y,unc}}$ of the retrofitted building are obtained by linear calculations (i.e. modal analysis) of the uncoupled model, along the actual or fictitious principal directions of the building, assuming effective (reduced) stiffness to 50% of the geometric element sections according to paragraph 4.3.1 (7) of Eurocode EN 1998-1. It is noted that, following the flowchart of the present study, the examined building is considered "torsional stiff" (i.e. it is not "torsional sensitive", see step 4 of the flowchart) and will always be characterized as a "wall-equivalent dual system" according to paragraph 5.1.2 of EN 1998-1 and according to step 5 of the flowchart.

*Results*: From Figure 2 we can see the following:

1. the displacement at yield of the building, $u_{\mathrm{top,y}}$, of the roof Mass Centre is estimated:

   $u_{\mathrm{top,y}} = 0.010 \cdot H_{\mathrm{tot}} = 0.010 \cdot 16 = 0.16\ \mathrm{m}$

2. the ultimate displacement at failure of the building, $u_{\mathrm{top,u}}$, of the roof Mass Centre is estimated:

   $u_{\mathrm{top,u}} = 0.025 \cdot H_{\mathrm{tot}} = 2.5 \cdot u_{\mathrm{top,y}} = 2.5 \cdot 0.16 = 0.40\ \mathrm{m}$

3. the effective lateral period $T_X^*$ or $T_Y^*$ of the ideal SDOF system (along the principal building directions) that is considered equivalent to the retrofitted real building according to Annex B of EN 1998-1, can be estimated by the following equations:

   $$T_X^* = \frac{T_{\mathrm{X,unc}}}{\sqrt{0.30}} \quad , \quad T_Y^* = \frac{T_{\mathrm{Y,unc}}}{\sqrt{0.30}}$$

   The denominator $\sqrt{0.3}$ satisfactorily reflects the consideration of the secant stiffness at yield $EI_{\mathrm{sec}}$ of all structural elements which is necessarily taken into account in the non-linear analysis according to the paragraph A.3.2.4(5) of Eurocode EN 1998-3.
   Therefore, the effective lateral periods $T_X^*$ and $T_Y^*$ are estimated:

   $$T_X^* = \frac{T_{\mathrm{X,unc}}}{\sqrt{0.30}} = \frac{0.55}{\sqrt{0.30}} = 1.00\ \mathrm{s} \quad , \quad T_Y^* = \frac{T_{\mathrm{Y,unc}}}{\sqrt{0.30}} = \frac{0.60}{\sqrt{0.30}} = 1.10\ \mathrm{s}$$

4. From Figure 2, it follows that the peak ground accelerations that cause the yielding of the building will be $a_{\mathrm{g,X}} = 0.38\mathrm{g}$ for the X-seismic component and $a_{\mathrm{g,Y}} = 0.34\mathrm{g}$ for the Y-seismic component.

5. The spectral acceleration for the above two earthquakes according to the elastic spectrum of EN 1998-1 (for soil D, $S$=1.35, $\eta$=1.00, $T_c$=0.80 s) is:

   $$S_{e,X} = a_{\mathrm{g,X}} \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C}{T_X^*} = 0.38\mathrm{g} \cdot 1.35 \cdot 1 \cdot 2.5 \cdot \frac{0.80}{1.00} = 1.026\mathrm{g}$$

   $$S_{e,Y} = a_{\mathrm{g,Y}} \cdot S \cdot \eta \cdot 2.5 \cdot \frac{T_C}{T_Y^*} = 0.34\mathrm{g} \cdot 1.35 \cdot 1 \cdot 2.5 \cdot \frac{0.80}{1.00} = 0.918\mathrm{g}$$

6. Consequently, the two corresponding mean Base Shears that cause the yielding of the building can be estimated and then the pre-estimate of the mean elastoplastic-perfect resistance curve of the retrofitted building can be drawn (Figure 3):

   $V_{o,X} = m_{\mathrm{tot}} \cdot S_{e,X} = 1200 \cdot 1.026\mathrm{g} = 12078.07\ \mathrm{kN}$
   $V_{o,Y} = m_{\mathrm{tot}} \cdot S_{e,Y} = 1200 \cdot 0.918\mathrm{g} = 10806.70\ \mathrm{kN}$

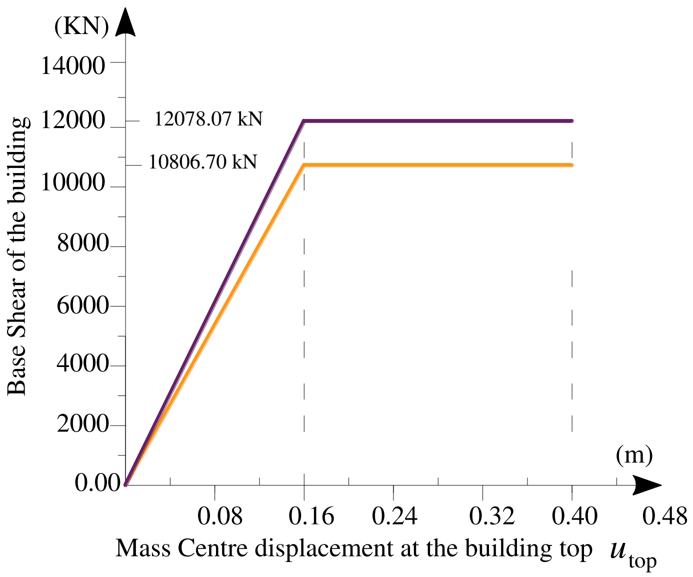



Figure 3. Pre-estimate of the mean elastoplastic-perfect capacity curve of the “as strengthened” r/c building

With this way, a safe pre-estimate of the mean elastoplastic-perfect resistance diagram of the building under consideration (after the necessary strengthening interventions) is determined. This pre-estimate was based solely on the use of linear analyses but exploiting the results of this relevant work investigation with non-linear analyses.

## 4. CONCLUSIONS

The current paper presented a new flowchart about the application of Eurocode EN 1998-3 and Hellenic Code of Structural Interventions (KAN.EPE. 2nd Revision). This new flowchart is trying to fill the gap of the above two regulations in order to increase the perception of the average Civil Engineer who is required to apply them perfectly. The proposed flowchart was implemented with absolute success in a series of design studies for the strengthening and assessment of the bearing seismic capacity of real, asymmetric, multi-storey r/c buildings in Greece, in the frame of a non-financial Investigative Program which took place in the Institute of Structural Analysis & Dynamics of Structures, School of Civil Engineering of AUTH. All the buildings examined had been designed by old seismic codes and present clear static deficiency according to the Seismic Performance Matrix set out in EN 1998-3. According to the flowchart, the most suitable retrofit measures for the examined buildings should be a combination of the following: (a) addition of new walls (or lengthening of existed ones) in order to characterize the buildings as “torsional stiff” and, at the same time, as “wall-equivalent dual systems”, (b) reinforced concrete jacketing of the deficient members. Afterwards, according to the flowchart, we can access the seismic performance of the “as strengthened” r/c buildings by non-linear static (pushover) analysis using a non-linear “bare” model without the reception of the masonry infills.

From the results of the non-linear static analyses that performed, a mean elastoplastic-perfect diagram between the Peak Ground Acceleration (PGA) and the average Drift ratio ($u_{\text{top}}/H_{\text{tot}}$) of the elevation profile of the “as strengthened” r/c buildings was parametrically determined. Also, the systematic procedure, using only linear analysis and the previous diagram, to approximately pre-estimate the result of a retrofit intervention in any multi-storey r/c building by adding new walls and reinforced concrete jackets, is presented in detail. This information is particularly useful to the owner of the building, prior to the detailed design study for the strengthening interventions, in order to complete his strategic planning, to understand the effectiveness of the building strengthening scenario and ultimately to decide to implement or not the building retrofit interventions.

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