

RAPID PRELIMINARY SEISMIC ASSESSMENT METHODOLOGY FOR NON-CONFORMING REINFORCED CONCRETE BUILDINGS

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Abstract. *In this paper, application of a rapid, yet efficient methodology for the evaluation of failure modes of lightly reinforced substandard buildings is presented. The method determines the limiting shear resistance of the structure as the least value supported by the columns' pure flexural, degraded shear, anchorage or lap-splice and joint shear resistance mechanisms. For application of the methodology only knowledge of the basic geometric and material properties of the building is required. The proposed methodology is applied to a number of Reinforced Concrete buildings that failed during past strong earthquakes for verification reasons. It is shown that the proposed methodology is ideal for rapid preliminary seismic assessment and it can be used both as a diagnostic tool for identification of the building's fragility and the prevailing mechanism of building failure.*

1 INTRODUCTION

Recent earthquakes have repeatedly illustrated the deficiencies of brittle reinforced concrete (RC) buildings built according to earlier design codes. This class of buildings, which today represents the majority of the built environment in the greatest part of the world, is typified by a number of features such as small section columns, relatively stiff beams, inadequately confined joints and insufficient anchorage of longitudinal and transverse reinforcement. Although the application of detailed assessment procedures for seismic evaluation of every single existing structure is of vital importance, the immense volume of required work makes it seem an unrealistic scenario. This difficulty could be removed by applying a rapid evaluation procedure where information readily available for most buildings will be used; in such a rapid assessment, critical features of the structure that render it more vulnerable, could be immediately identified thereby simplifying the process of singling out those structures that represent a major threat to human life in the event of an earthquake.

The objective of this paper is to present a methodology where the building's geometrical characteristics, material properties and reinforcement detailing of the structural elements are considered sufficient information so as to single out the most vulnerable buildings that are likely to collapse in a potential strong earthquake, but can also be used to determine the objectives for their rehabilitation. Existing RC buildings, "non-conforming" according to modern standards, are assessed based on prioritizing of the various alternative modes of failure. Mechanisms considered refer to column flexure, degraded shear, anchorage and lap-splice development, connection punching and exceedence of joint shear capacity; the rate of degradation of these mechanisms with increasing displacement amplitude and number of cycles is idealized through simplified mechanistic constructs which allows the prioritizing of failure modes as degradation proceeds. Furthermore, the seismic vulnerability of this category of buildings is assessed as a function of interstorey drift demand imposed by the design earthquake [1]. For confirmation, the methodology is applied to a number of RC buildings that collapsed during the 1999 Athens earthquake. Results indicated that in all cases, buildings failed in a brittle manner due to anchorage failure of column longitudinal reinforcement in the joints' regions.

2 METHODOLOGY FOR RAPID EVALUATION OF FAILURE MODES

2.1 Background information

In the first 2/3rds of the 20th century, when massive rebuilding and urbanization of cities took place throughout Southern Europe, construction details were not strictly addressed by the design codes and as such, they were seldom specified in design drawings of that era. Implementation in practice was, to a large extent, determined by the experience of the laborers and the foreman responsible for the site. Credentials were never questioned and supervision was relatively lenient; the reinforcing cages were usually assembled on site, empirically in most cases and often using makeshift tools for hook bending and bar cutting. Today, even in the rare occasion where a plan drawing from the 1960's specifies details for stirrup placement, it is questionable whether these stirrups had been actually placed as specified. Reconnaissance studies from collapsed buildings in major earthquake events suggest that at least in those, collapsed cases, there was a marked lack of, or primitive use of stirrups. But underuse of stirrups was the prevalent practice of the era. Usually stirrups were rectangular, with 90° hooks in the ends, made of smooth reinforcement, using bar sizes of \varnothing_t equal to 6 mm or at best 8 mm, spaced at 250 mm to 300 mm; however, occasional examples of even wider spacing have also been cited. Steel quality was StIII ($f_{yk}=420$ MPa) for longitudinal reinforcement and StI

($f_{yk}=220$ MPa) for stirrups, concrete quality ranged between Bn150 to Bn200 [2] (i.e. a concrete characteristic strength of 12 to 16 MPa according to today's standards). Lap splices were usually unconfined, whereas starter bars had random lengths. Joints, being considered points of reinforcement congestion since they accommodated both beam and column primary reinforcement, were usually left without stirrups, for convenience of construction. Single column footings were mostly used and, in well-attended structures, they were joined at the column base with lightly reinforced, small section connecting beams.

Despite these systematic inadequacies, the percentage of buildings that have collapsed in major earthquakes in the Mediterranean basin is relatively low when compared to the total number of available buildings that belong to this substandard construction category; this is not true in other parts of the world such as in Haiti (Port-au-Prince 2010, [3]) One issue that can be concluded, however, from post-earthquake reconnaissance evidence and forensic evaluations of collapsed buildings which led to loss of human life, is that collapse usually occurs by the formation of a mechanism before the development of any form of ductility. In this work it is demonstrated by paradigm that in most cases of such "killer collapses", displacement at the onset of failure is lower than the nominal yield displacement of the structure. This is contrary to the focus of most modern assessment methodologies that are based on comparisons between ductility demand and supply. Note that the usual point of reference in these procedures is columns where transverse reinforcement is not adequate to support shear strength under displacement reversals beyond flexural yielding of the individual members of a structure. For this reason, values of rotation capacity in what is termed "non-conforming" members are usually deemed in the range of 1% to 1.5%. This focus could be justifiable for structures with columns where the member size (in the range of 500 mm or higher) is twice that of typical stirrup spacing used in the 1960's, so that a nominal 45° shear plane in a column would intersect at least two stirrups. But in case of structures with smaller column sizes, in the range of 350 mm, as is often encountered in older residential buildings, the practical spacing of stirrups from the 1960's (250 mm) could mean that a nominal potential sliding plane could be formed, intersecting no stirrup at all and therefore being unable to mobilize any form of shear resistance.

Other unfavourable characteristics of RC concrete buildings constructed prior to the introduction of capacity design principles and modern detailing practices are the relatively stiff beams intended to control serviceability requirements. For this class of buildings, a rapid evaluation for the potential mode of failure could be focused on the calculation of the limiting strength of building columns, since post-earthquake reconnaissance reports illustrate column failure as the primary cause of building collapse, being associated with the loss of gravity load-carrying capacity.

2.2 Available diagnostic tools

In light of the limited knowledge of actual construction details in older structures, preliminary assessment targeted toward identification of the most vulnerable buildings must necessarily rely on a marginal collection of data that is readily available, such as the overall geometric details of the structure (number of floors, floor height, floor area, location and gross geometry of load carrying members in plan), on the implicit assumption that all reinforcing details are represented by the historical construction information for the period and region of construction of the building studied. With these data, building seismic vulnerability is based on the following two criteria:

(a) A *stiffness index assessment*, which is used to quantify interstorey drift demand.

(b) A *strength assessment*, which is used to determine the weakest mechanism of resistance, likely to control the sequence of failure of the vertical elements of the structure.

(a) Stiffness Index Assessment: From the earliest earthquake studies the area ratio of the load-bearing elements in a structure has been used to quantify the magnitude of lateral stiffness. The importance of this parameter was reflected in the very first Seismic Codes worldwide, whereby the required floor area ratio of walls was set proportional to the number of floors in the structure, to secure adequate seismic performance. Recent studies have systematically explored the relationship of generalized stiffness, K^* , to the floor area ratios of columns, ρ_c , walls, ρ_{wc} , and infill walls, ρ_{wm} , in a structure [1]. It was shown by systematically exploring the relationship between stiffness and fundamental period, T ($T \approx 2\pi\sqrt{0.8W/(gK^*)}$), that the displacement demand under the design earthquake, S_d , could be expressed in terms of ρ_c , ρ_{wc} , and ρ_{wm} (where the seismic hazard is given in relative displacement vs. period spectrum coordinates). Using an estimate of the morphology of the shape of vibration of the structure (Fig. 1) it is possible to develop generic charts of the type shown in Fig. 2 that relate drift demand in the critical floor of the structure to a combined stiffness geometric index, defined as:

$$\text{Dual systems:} \quad k = \rho_c + \eta_{wm} \rho_{wm}^e \quad (1a)$$

$$\eta_{wm} = \frac{25E_{wm}}{E_c \left(\frac{h_i^2}{l_{m,ave}^2} + 2.5 \right)} ; \rho_{wm,i}^e = \rho_{wm,i} + \frac{\left(4 \frac{h_i^2}{l_{m,ave}^2} + 2.5 \right) E_c}{\left(4 \frac{h_i^2}{l_{w,ave}^2} + 2.5 \right) E_{wm}} \rho_{wc,i} \quad (1b)$$

$$\text{Frames:} \quad k' = \rho_c + \eta'_{wm} \rho_{wm} \quad (2a)$$

$$\eta'_{wm} = \frac{10 \cdot f_{wk} \sqrt{\left(1 + \frac{h_i^2}{l_{w,ave}^2} \right)}}{E_c \theta_1} \quad (2b)$$

where E_c , E_{wm} are the elastic moduli of concrete and masonry, respectively, ρ_c is the columns' area ratio in the floor plan, ρ_{wm}^e is an equivalent compound dimensionless area index that represents both masonry walls and RC walls expressed in terms of masonry wall properties, h_i is the storey height, $l_{m,ave}$, $l_{w,ave}$ are the average length of masonry and RC wall, respectively, f_{wk} is the compressive strength of masonry and θ_1 is the estimated chord rotation demand of the first storey.

Because of inadequate shear resistance, collapse of substandard buildings in a critical earthquake could be prevented if the interstorey drift demand, θ_u , in the critical floor (if the critical floor is the first floor, $\theta_u = \theta_1$), is below the estimated drift at failure, which is given by:

$$\theta_{fail} = \frac{V_{u,lim}}{V_{y,flex}} \cdot \theta_{y,crit} \leq \theta_{y,crit} \quad (3)$$

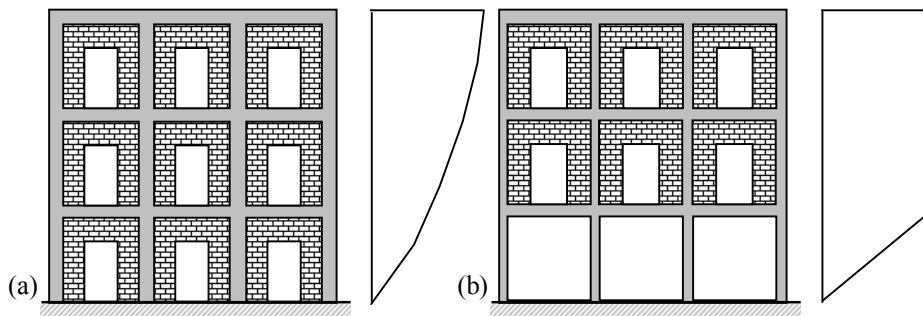


Figure 1: Lateral displacement profiles; (a) shear-type; (b) soft-storey.

In the above, $\theta_{y,crit}$ is the estimated interstorey drift at longitudinal reinforcement yielding of the critical floor – for typical frame structures (with floor heights around 3m) this is usually in the range of 0.5%. Values $V_{y,flex}$ and $V_{u,lim}$ are the estimated base shear strengths at the onset of yielding of column reinforcement and of transverse reinforcement, respectively. For typical older construction, this ratio is usually in the range of 30% - 60%, depending on the size of the columns for the typical details used in the period of reference (the ratio $\approx 30\%$ for column section sizes in the range of 300 mm, to 60% for larger column sections in the range of 500 mm – detailed expressions for calculating these terms for each individual column are given in the following section). The plot in Fig. 2 has been drawn for the EC8 design spectrum (Type 1) using a peak ground acceleration of 0.36g; thus, demand (ordinate in the graph) should be scaled down by the ratio of $a_g/(0.36g)$ for lower levels of peak ground acceleration coefficient, a_g .

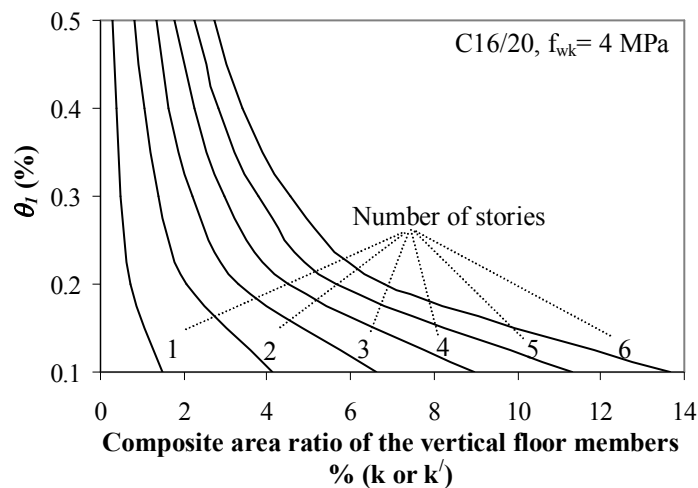


Figure 2: Vulnerability curves that relate the required composite floor area ratio of vertical members to levels of interstorey drift, $\theta_1 \leq 0.5\%$, which are of interest for buildings susceptible to brittle failures. Note that the baseline value of 0.1% is the estimated drift limit associated with diagonal cracking of unreinforced concrete flexural members.

(b) Strength Assessment: Collapse occurs when the vertical elements of the critical floor lose their load carrying capacity. This process may be initiated either by loss of lateral load resistance of the columns, or by punching and loss of support of the floor diaphragms. To be able to establish a hierarchy of possible failure modes which threatens the integrity of a structure under lateral sway, all mechanisms of response must be compared on a common basis of

reference. In this study, the typical form of a moment diagram along the column lines of a swaying structure is used to establish this common frame of reference. As depicted in Fig. 3, the typical column experiences a moment reversal at the ends, owing to the transfer of moment from the horizontal to the vertical elements, occurring at the beam-column joints (in frame structures) or at the slab-column connections in flat-slab structures. The approximate location of points of inflection (zero moment) along the column length enables the establishment of a static relationship between the critical strengths of various mechanisms that could be responsible for column failure along the line of a single column, and the column shear sustained when any of these phenomena is occurring, as follows:

(i) Plastic hinging at the column end (i.e. $M_{col} = M_y$), $V_{y,flex}^{col} = 2 \cdot M_{col} / h_{st}$, where h_{st} is the deformable length of the column (i.e. the clear storey height, or free column length in captive columns).

(ii) Exhaustion of column shear strength, $V_{u,shear}^{col}$

(iii) Exhaustion of column lap-length development capacity of tension reinforcement F_b :

$$V_{u,lap}^{col} = 2 \cdot [F_b \cdot d \cdot (1 - 0.4 \cdot \xi) + N_{g+0.3q} \cdot (0.5 \cdot h_{st} - 0.4 \cdot \xi \cdot d_{col})] / h_{st}$$

where ξ is the depth of compression zone normalized with respect the column section depth, d_{col} , and $N_{g+0.3q}$ is the axial compression load in the column owing to the combination of unfactored gravity loads plus a reduced contribution of live load (this is the average column axial load; the column axial load fluctuates about this value due to the overturning effects of the earthquake). The value of ξ ranges between 0.22 and 0.30 for columns with a reinforcing ratio of less than 1.5% and a variety of normalized axial load ratios, and for this reason the above result is not very sensitive to this parameter (Fig. 4).

(iv) Exhaustion of joint shear strength (in frame structures), $V_{j,u}$. Column shear is estimated from equilibrium of moment transfer: $V_{u,joint}^{col} = [V_{j,u} \cdot d_{beam}] / h_{st}$.

(v) Exhaustion of slab punching strength (in flat slab construction), $M_{u,slab}$. The associated column shear is again estimated from equilibrium of moment transfer (assuming this is an intermediate floor of the structure, i.e., not a roof connection): $V_{u,slab}^{col} = M_{u,slab} / h_{st}$ where $M_{u,slab}$ the strength for moment transfer of the critical punching perimeter at the slab – column connection.

Evaluation of the prevalent failure mode is obtained from the inequality:

$$V_{u,lim}^{col} = \min \left\{ V_{y,flex}^{col}, V_{u,shear}^{col}, V_{u,lap}^{col}, V_{u,joint}^{col} \text{ or } V_{u,slab}^{col} \right\} \quad (4)$$

Expressions for calculating the individual strength terms are given in the appendix. These may be subject to change as the knowledge base in reinforced concrete leads to improved models for the individual mechanisms of resistance and may differ between established code frameworks.

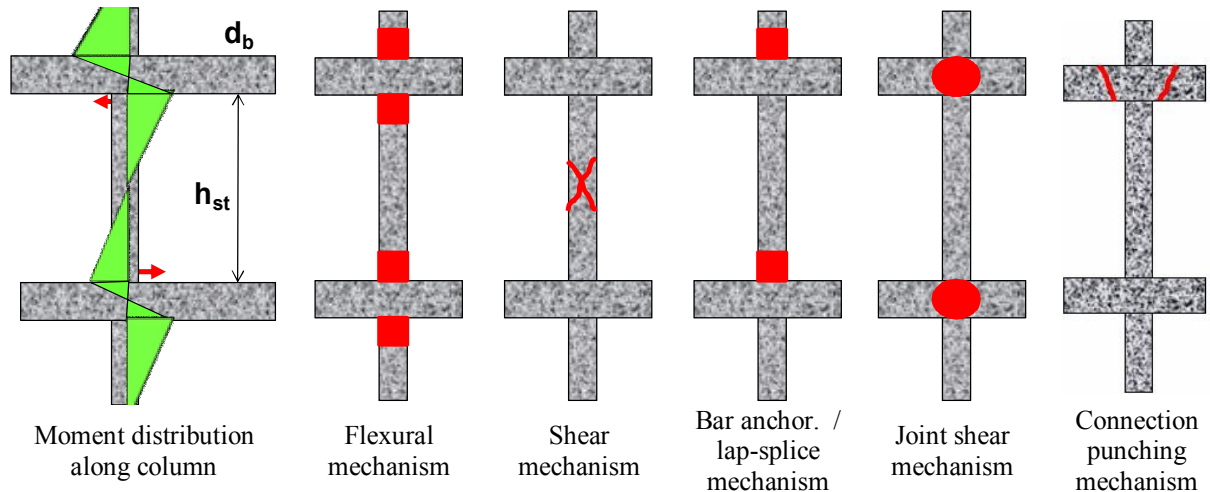


Figure 3: Moment distribution and possible failure modes of a reinforced concrete column.

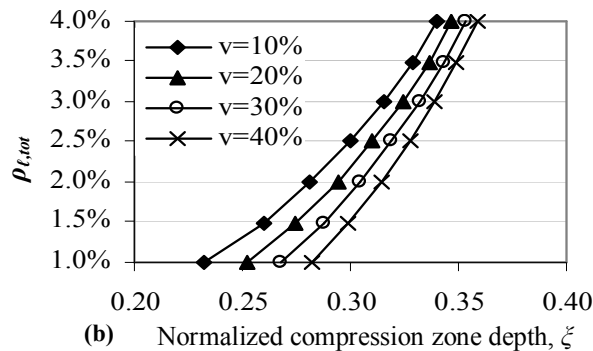


Figure 4: Relationship between ζ , $\rho_{t, tot}$ and normalized axial load, ν , for columns.

In the following, a number of buildings collapsed during the 1999 Athens earthquake are used to illustrate performance of the proposed rapid screening approach.

3 APPLICATION OF THE METHODOLOGY

The proposed methodology for rapid evaluation of the limiting resistance of “non-conforming” RC buildings through systematic prioritizing of the individual column shear strengths is applied to two RC buildings that collapsed during the 1999 strong ground motion of Athens. Both buildings were located in the northern region of Athens, where the ground motion possessed “near-field” characteristics. The methodology is applied so as to estimate the buildings’ drift at the state of their failure as well as to calculate the limiting base shear that the buildings could sustain upon failure, but also to test the ability of the methods to identify the high seismic vulnerability of such structures.

Building A was a two-storey fully symmetric in plan, industrial building, with external plan dimensions of 38.00 m by 26.00 m (Fig. 5). The first and the second storey heights were 5.40 m and 5.30 m, respectively. Building A was separated by seismic gaps from two adjacent wing buildings along the two smaller sides. The structural system was formed as an orthogonal grid of columns, beams and slabs, according to typical construction practice of RC frame structures in Southern Europe. Details of the column geometry and longitudinal reinforcement

are presented in Table 1. Column stirrups were smooth, rectangular ties, approximately categorized based on site reconnaissance as $\text{Ø}8/300$ mm. Slab thickness was 0.15 m. All perimeter beam (PB) cross sections were 0.70 m (height) by 0.30 m (width), 0.70×0.45 m for beams spanning between columns (MB) and 0.70×0.25 m for the secondary beams (SB). During the earthquake the building collapsed without any horizontal dislocations of its structural elements, whereas the two adjacent buildings were intact. From tests of core samples, the mean value of concrete compressive strength was determined as 18.7 MPa, whereas steel yielding and ultimate stresses were found to be for the longitudinal reinforcement 431.5 MPa and 512.0 MPa, respectively, and for the stirrups 402.0 and 553.0 MPa, respectively.

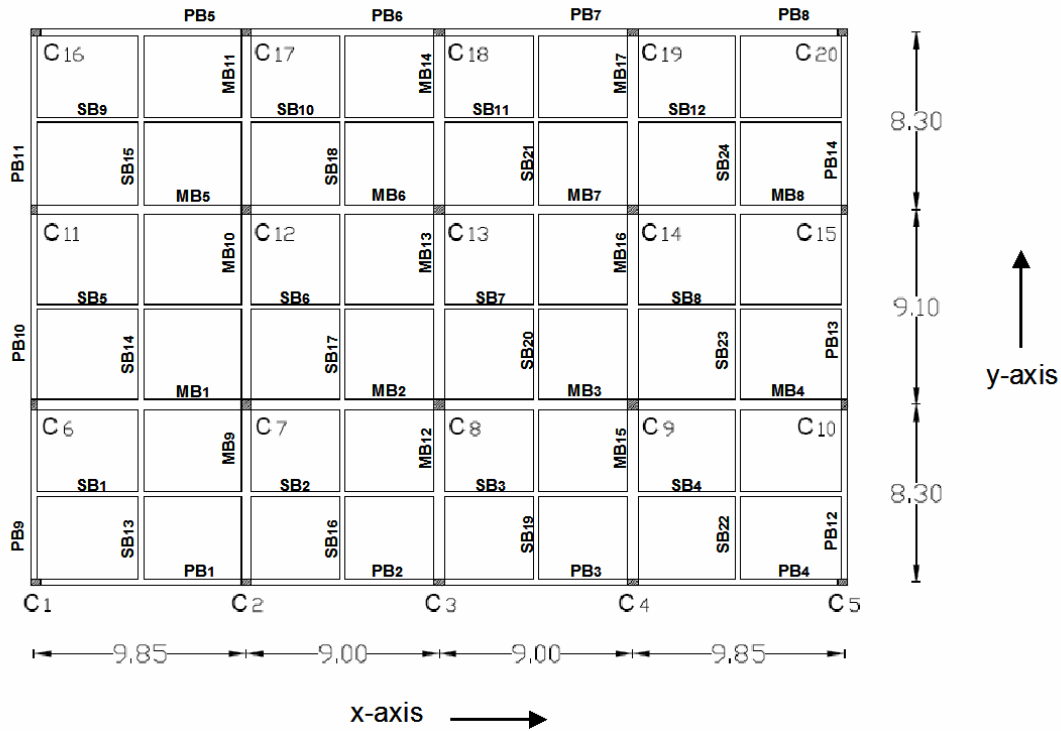


Figure 5: Plan configuration of Building A.

Column	Dimensions(mm)	Long. Reinforcement (mm)
C ₁ , C ₅ , C ₁₆ , C ₂₀	450 / 300	4 $\text{Ø}20$
C ₂ , C ₃ , C ₄ , C ₁₇ , C ₁₈ , C ₁₉	450 / 300	8 $\text{Ø}20$
C ₆ , C ₁₀ , C ₁₁ , C ₁₅	300 / 450	8 $\text{Ø}20$
C ₇ , C ₈ , C ₉ , C ₁₂ , C ₁₃ , C ₁₄	450 / 450	12 $\text{Ø}20$

Table 1: Column dimensions and longitudinal reinforcement of Building A.

The results of the *strength assessment* of the first storey columns in x and y direction for Building A (Fig. 6) are presented herein. The prevailing failure mechanism for all the columns of the first storey was failure due to exhaustion of column lap-length development capacity of tension reinforcement. The total base shear that the building could sustain at the onset of this mode of failure was 1022 kN and 891 kN in x and y direction, respectively, whereas the flexural strength was estimated equal to 2094 kN and 1845 kN in x and y direction, respectively. Hence, failure was expected to occur when the imposed base shear reached 49% and 48% of the flexural resistance in x and y direction, respectively. Given the

axial load of the first storey columns for the $g+0.3q$ load combination (the normalized axial load ratio was estimated as $v_{ave}=0.22$, $W=13000$ kN), the maximum peak ground acceleration that Building A could sustain was only 7.9%(=1022/13000) and 5.9%(=891/13000) of g , in the x and y directions, respectively (note that the 1999 Athens earthquake had a peak ground acceleration, $pga=0.38g$ (Sepolia station)).

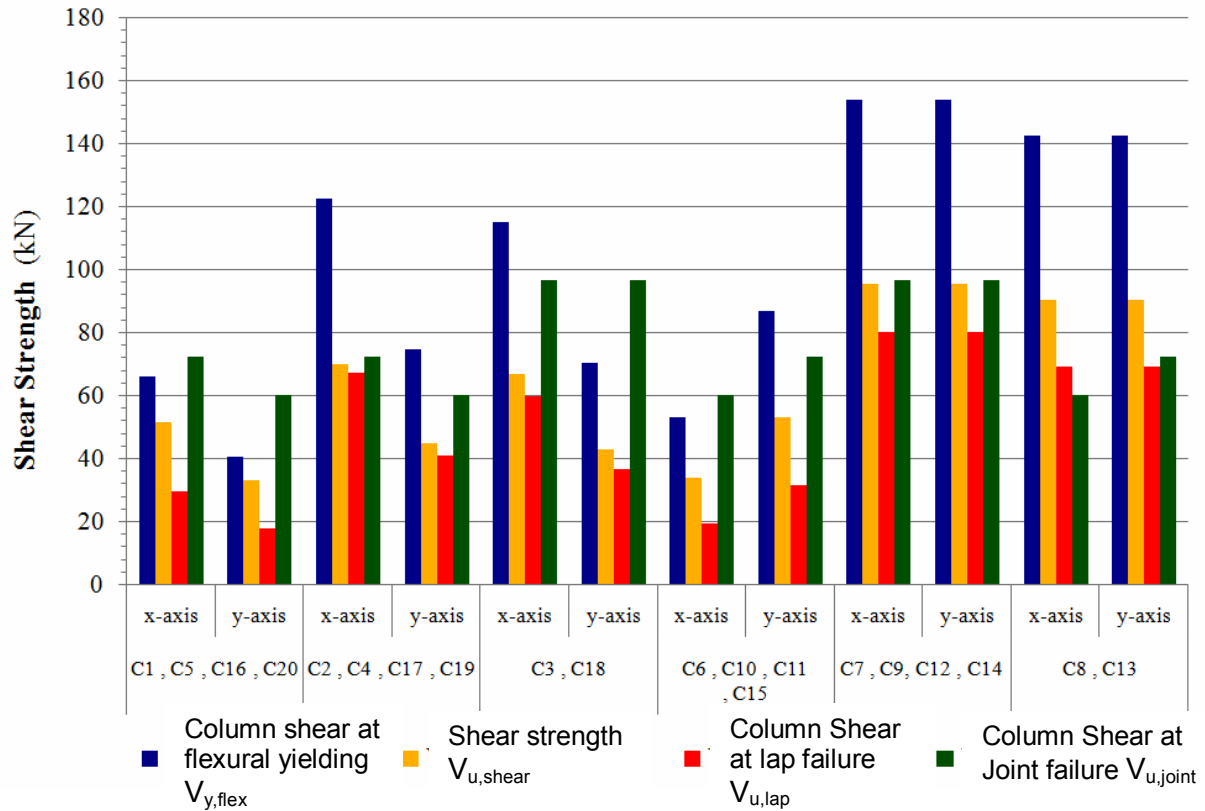


Figure 6: Strength of the columns of building A against the various mechanisms of failure; each group of bars in the chart represents the estimated column strengths against alternative failure modes, for a given column type and direction of seismic action (x or y). Labels under the bar charts identify the columns that fall under the specific category.

As calculated from the building's properties, the area ratio of the columns in both x and y directions was $\rho_{c,x}=\rho_{c,y}=0.3\%$. The average nominal drift at yield, $\theta_{y,nom}$, was estimated equal to 1.26% and 1.10% in x and y direction, respectively (Table 2). Due to the premature failure of columns, failure is expected to occur at earlier drift levels. According to Eq. (3) and the results of the *strength assessment* presented in the previous paragraph, those drift levels are determined as $\theta_{fail}^x=1.26 \cdot 1022/2094=0.62\%$ and $\theta_{fail}^y=1.10 \cdot 891/1845=0.53\%$ in x and y direction, respectively.

Column	$\theta_{v,nom}^{x*}$	$\theta_{v,nom}^{y*}$
C ₁ , C ₅ , C ₁₆ , C ₂₀	1.54%	0.99%
C ₂ , C ₃ , C ₄ , C ₁₇ , C ₁₈ , C ₁₉	1.54%	0.99%
C ₆ , C ₁₀ , C ₁₁ , C ₁₅	0.99%	1.54%
C ₇ , C ₈ , C ₉ , C ₁₂ , C ₁₃ , C ₁₄	0.99%	0.99%
Average	1.26%	1.10%

$\theta_{y,nom} = (2.14\varepsilon_y/d) \cdot (L_s/3)$ [4], where ε_y : steel strain at yield, d : depth of the cross section, L_s : clear shear span length

Table 2: Nominal drift at yield for the columns of Building A.

According to the methodology developed by Thermou and Pantazopoulou [1], the vulnerability curve for Building A was estimated for Type I elastic spectrum of EC8 [5] using as peak ground acceleration the magnitude recorded in the Athens 1999 earthquake (Sepolia station) ($p_{ga}=0.38g$) and $\rho_{vm}=0$ in Fig. 7. For a drift value equal $\theta_{fail}^x=0.62\%$ and $\theta_{fail}^y=0.53\%$ (as defined after the reduction of nominal drift at yield due to premature failure of the columns, see Fig. 6) demand in terms of ρ_c is defined by the horizontal dashed lines in Fig. 7. The percentage of columns required in order to maintain the drift levels in the x and y direction below the failure limit would have to be equal to 0.64% and 0.75%, respectively (i.e. in order to survive the earthquake the building should have 2.1 and 2.5 times the available area ratios of columns in Building A). This leads to the conclusion that the area ratio of nonductile columns in the existing building ($\rho_c=0.3\%$) was insufficient for the seismicity level of the Athens 1999 earthquake as evidenced by its dramatic collapse.

The vulnerability curve of Fig. 7 may be used alternatively to assess the performance of Building A by estimating the demand in terms of drift at the first storey given the percentage of columns. The vertical dashed line depicts the demand in terms of drift of the first storey, which is equal to 1.33% for $\rho_c=0.3\%$. This means that the earthquake would require from the structure, if the structure possessed ductility, a displacement ductility value of $\mu=1.33\%/1.26\%=1.05$ and $\mu=1.33\%/1.10\%=1.2$ in x and y direction, respectively. If this assessment tool would be used in practical preliminary retrofit design, it would suggest the required ductility level that should be targeted for in retrofitting the columns. This ductility limit is within the range that could be attained by the use of local interventions for seismic upgrading such as FRP jacketing, a finding that points to the criteria for establishing a retrofit scenario for structures assessed prior to the occurrence of a threatening earthquake event.

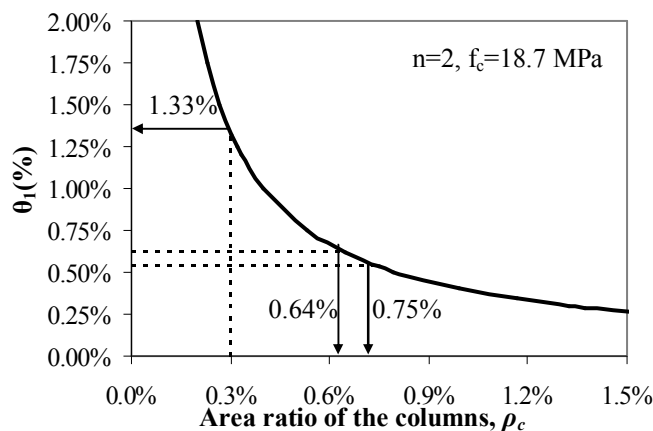


Figure 7: Vulnerability curve for Building A subjected to Athens 1999 earthquake estimated according to the methodology of Thermou and Pantazopoulou [1].

Building B was also an industrial building, having a 37.60 m by 22.80 m orthogonal plan (Fig. 8). The building had two basements and four storeys, each 2.85 m high. The structural system comprised a grid of columns which were connected with beams only along the buildings' perimeter, the beams having a section height of 0.60 m and 0.20 m web width. In the centre of the typical floor plan, columns supported a flat-plate Zoellner system, having a thickness of 0.22 m. During the earthquake the building collapsed, except for the stairwell in the corner of the plan. After tests conducted on material samples, the concrete was found to have a mean compressive strength of 20 MPa, while longitudinal reinforcement and stirrups were found to have smooth surface and were classified as S400 ($f_y = 400$ MPa) and S220 ($f_y = 220$ MPa) respectively. Column dimensions and longitudinal reinforcement details are presented in Table 3, whereas transverse reinforcement comprised $\text{Ø}6/300$ mm rectangular, smooth stirrups.

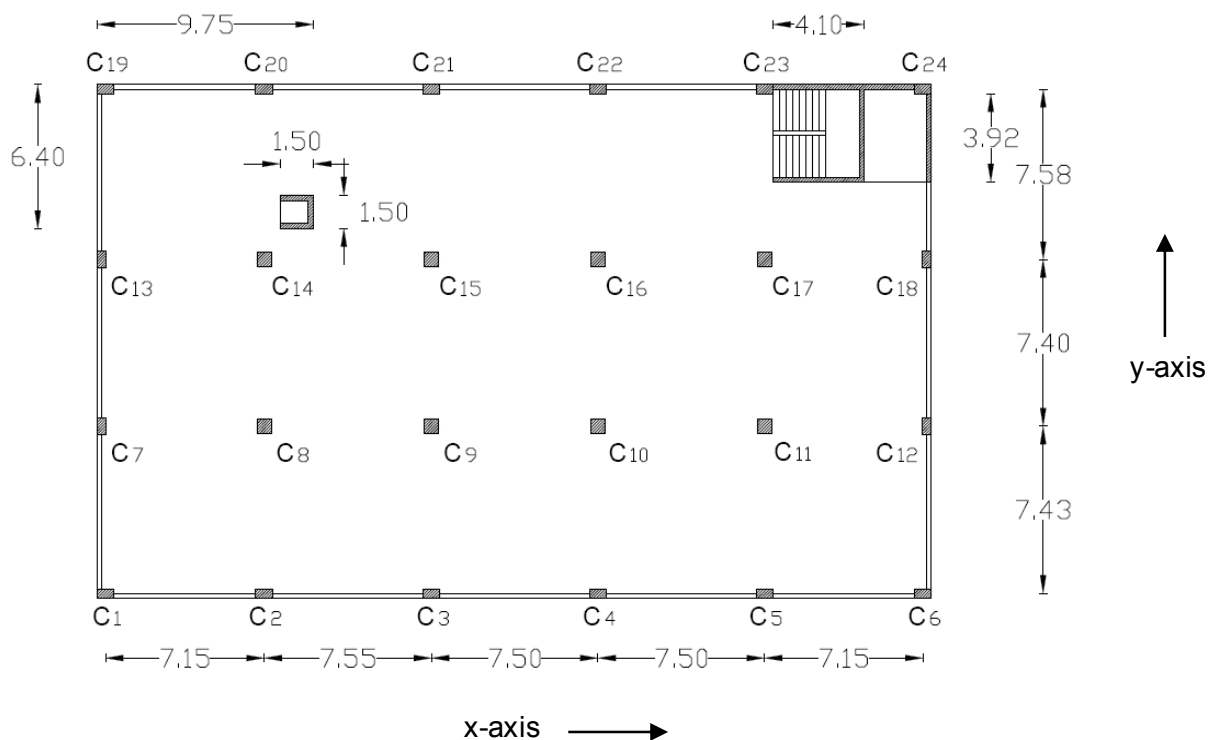


Figure 7: Plan configuration of Building A.

Column	Dimensions (mm)	Long. Reinforcement (mm)
C ₁ , C ₂ , C ₃ , C ₄ , C ₅ , C ₆ , C ₁₉ , C ₂₀ , C ₂₁ , C ₂₂	750 / 400	8 Ø16
C ₇ , C ₁₂ , C ₁₃ , C ₁₈	400 / 750	8 Ø16
C ₈ , C ₉ , C ₁₀ , C ₁₁ , C ₁₄ , C ₁₅ , C ₁₆ , C ₁₇	650 / 650	8 Ø20

Table 3: Column dimensions and reinforcement of Building B.

Strength assessment has revealed that the prevailing failure mechanism of the first storey columns was failure due to exhaustion of slab punching strength. Note that this type of failure (lowest green bars in Fig. 8) concerns all interior columns in directions where they were not

connected with beams but also exterior columns in the direction of action that bent them about their weak axis (e.g. C1 for seismic action in the y-direction). The total base shear that Building B could sustain at the onset of slab punching was 1477 kN and 861 kN in x and y directions, respectively, whereas the flexural strength was estimated equal to 4487 kN and 4136 kN in x and y direction, respectively. Thus, when the imposed base shear reached only 33% and 21% of the flexural resistance in x and y direction, respectively, failure would be anticipated. The maximum peak ground acceleration that Building B could sustain (direction y) for the $g+0.3q$ load combination (average normalized axial load ratio $v_{ave}=0.21$, $W=32500\text{kN}$) was only 4.5 % ($=1477/32500$) and 2.6% ($=861/32500$) of g , in the x and y directions, respectively (1999 Athens earthquake, $pga=0.38g$, Sepolia station).

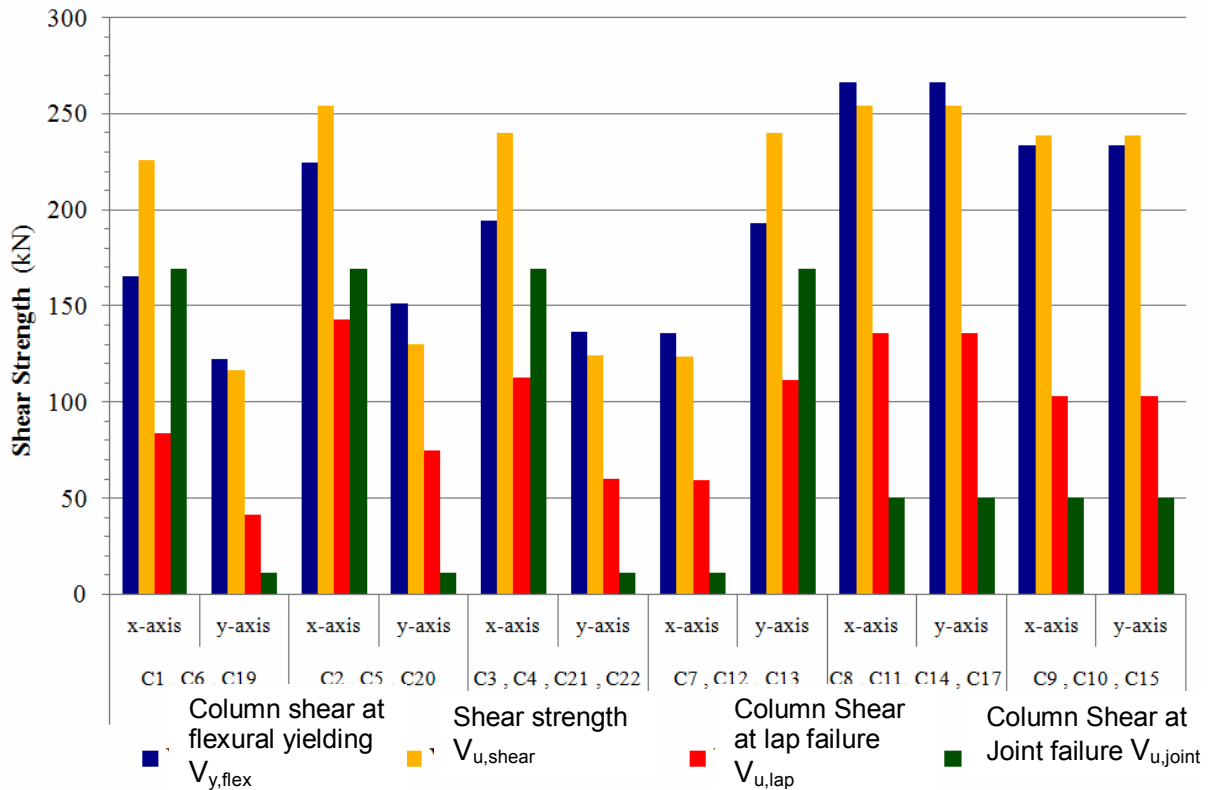


Figure 8: Strength of the columns of building B against the various mechanisms of failure; each group of bars in the chart represents the estimated column strengths against alternative failure modes, for a given column type and direction of seismic action (x or y). Labels under the bar charts identify the columns that fall under the specific category.

Stiffness assessment of Building B follows, where, according to the building's characteristics, the columns' area ratio in both x and y directions was $\rho_{c,x}=\rho_{c,y}=0.88\%$, whereas the wall area ratio in x and y direction was $\rho_{w,x}=0.35\%$ and $\rho_{w,y}=0.24\%$, respectively. In the following calculations, and in order to keep the procedure simple, only the area of columns was taken into account (the influence of the wall was considered low and further burdening the structure with torsional action due to its eccentric placement). The average nominal drift at yield, $\theta_{y,nom}$, was estimated equal to 0.42% and 0.35% in the x and y direction, respectively (Table 4). The drift levels at which failure due to premature failure of columns would be expected to occur were $\theta_{fail}^x = 0.42 \cdot 1477 / 4487 = 0.14\%$ and $\theta_{fail}^y = 0.35 \cdot 861 / 4136 = 0.07\%$ in the x and y directions, respectively.

Column	Drift at x-direction, θ_x^*	Drift at y-direction, θ_y^*
C ₁ , C ₂ , C ₃ , C ₄ , C ₅ , C ₆ , C ₁₉ , C ₂₀ , C ₂₁ , C ₂₂	0.55%	0.28%
C ₇ , C ₁₂ , C ₁₃ , C ₁₈	0.28%	0.55%
C ₈ , C ₉ , C ₁₀ , C ₁₁ , C ₁₄ , C ₁₅ , C ₁₆ , C ₁₇	0.33%	0.33%
Average	0.42%	0.35%

$\theta = (2.14\varepsilon_y/d) \cdot (L_s/3)$ [4], where ε_y : steel strain at yield, d : depth of the cross section, L_s : clear shear span length

Table 4: Column dimensions and reinforcement of Building B.

Limiting demand to avoid failure, is defined in terms of ρ_c by the horizontal dashed lines in Fig. 9 for $\theta_{fail}^x=0.14\%$ and $\theta_{fail}^y=0.07\%$ (these values point to significantly reduced drifts compared to the nominal drift at yield, owing to the premature failure of the columns, see Fig. 8). Thus, utilizing the vulnerability curve extracted for Type I elastic spectrum of EC8 [5] for the recorded peak ground acceleration in the 1999 Athens earthquake (pga=0.38g, Sepolia Station), it is concluded that: According to the characteristics of Building B, the percentage of required columns with the available brittle details in order to keep the drift levels in the x and y directions below the failure values would have to be 8.67% and 16.57%, respectively (9.8 and 18.7 times higher than the available area ratios of columns in the actual building). From the above, it is evident that the percentage of columns, given the mode of construction (brittle details) in the existing building ($\rho_c=0.88\%$) was entirely insufficient for the seismicity level of the Athens 1999 earthquake.

If reversely, Fig. 9 is used to define demand in terms of drift at the first storey for the existing area ratio of columns ($\rho_c=0.88\%$), in the hypothetical case where these possessed displacement ductility, then the vertical dashed line indicates the demand in terms drift equal to 1.36% (Fig. 9). This means that the earthquake would require from the structure, if the structure possessed ductility, a displacement ductility value of $\mu=1.36\%/0.42\%=3.2$ and $\mu=1.36\%/0.35\%=3.9$ in x and y direction, respectively, a value that is somewhat excessive to be attainable by local interventions such as FRP-jacketing should such have been implemented prior to the earthquake; rather, these ductility values point to a deficiency in stiffness that could have been supplemented by infill walls in some critical bays of the lower floors, thereby mitigating the ductility demands to lower, more attainable through proper retrofit, values.

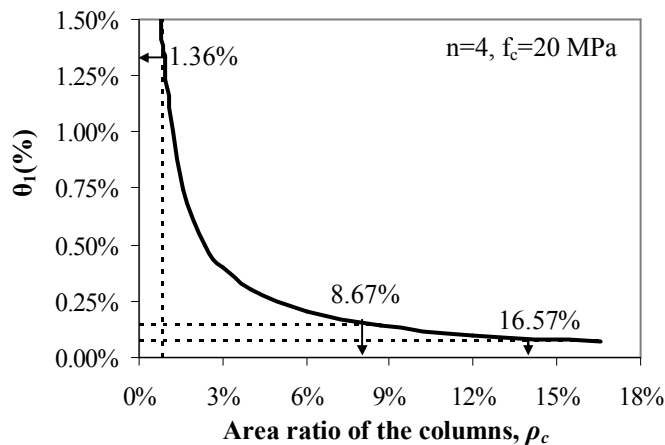


Figure 9: Vulnerability curve for Building B subjected to Athens 1999 earthquake estimated according to the methodology by Thermou and Pantazopoulou [1].

4 CONCLUSIONS

A methodology for determining the ultimate storey drift at the state of maximum seismic response and prioritizing the potential failure mechanisms in the load carrying system of concrete buildings classified as “non-conforming” according to modern standards was presented in this paper. Mechanisms considered refer to column flexure, shear, anchorage lap/splice development capacity and joint shear or slab punching strength, as failure of vertical structural elements is directly related to building severe damage or collapse. The methodology can be applied to every individual column of the building, regardless of its location; if flexural strength may be supported by the shear, anchorage, joint resistance and slab punching mechanisms, the calculated limiting strength can be compared to the developing shear force derived from a seismic analysis. However, in most cases of existing structures, brittle failure modes are prioritized to occur prior to flexural yielding, so that no ductility may be realized, whereas the building collapses at displacements lower than the yield point. In this case, the methodology may also be applied for the determination of the maximum ground acceleration that the building could sustain prior to retrofit, as illustrated in the former examples, thereby serving as a tool of rapid evaluation of the buildings’ seismic vulnerability. It is also possible to obtain, through this method, the magnitude of displacement ductility that would be required through column retrofit, in order to eliminate the risk of collapse due to excessive displacement demand.

REFERENCES

- [1] G.E. Thermou, and S.J. Pantazopoulou, *Assessment indices for the seismic vulnerability of existing RC buildings*, Journal of Earthquake Engineering and Structural Dynamics, 40(3), 293-313, 2011 (early view, first published 21/7/10).
- [2] Deutsches Institut für Normung (DIN). Beton un Stahlbetonbau: Bemessung und Ausführung. *DIN 1045*, Berlin, 1972.
- [3] M.O. Eberhard, S. Baldrige, J. Marshall, W. Mooney, G.J. Rix, *The M_w 7.0 Haiti Earthquake of January 12, 2010*, USGS/EERI Advance Reconnaissance Team: Team Report V1-1, February 23, 2010.
- [4] T. Paulay, and M. N. J. Priestley. *Seismic Design of Reinforced Concrete and Masonry Buildings*, 1992 (John Wiley & Sons, Inc., New York).
- [5] Eurocode 8. Design of structures for earthquake resistance—Part 1: general rules, seismic actions and rules for buildings. EN1998-1-2004:E, European Committee for Standardization (CEN), Brussels, 2004.
- [6] Pantazopoulou V, Syntzirma D. Code expressions for deformation capacity of lightly reinforced concrete members - a comparative study. ACES Workshop: Advances in Performance-Based Earthquake Engineering 2009, Corfu, Greece.
- [7] ACI-ASCE Committee 352. *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures*, American Concrete Institute, Farmington Hills, MI, 2008.
- [8] Eurocode 2. Design of concrete structures—Part 1-1: general rules and rules for buildings. EN1992-1-2005:E, European Committee for Standardization (CEN), Brussels, 2004.

APPENDIX

The following expressions can be used for calculating the individual strength terms of a column. These expressions represent the present state of the art at the field. Nevertheless, they may be subject to change as the knowledge base in reinforced concrete leads to improved models for the individual mechanisms of resistance.

(i) Calculation of column flexural yield:

Uniaxial bending [6]:

$$V_{y,flex}^{col} \approx \left[\rho_{l,tot} \cdot \frac{f_y}{f_c} \cdot (1 - 0.4 \cdot \xi) + v \cdot \left(\frac{h_{st}}{d_c} - 0.8 \cdot \xi \right) \right] \cdot b_c \cdot d_c^2 \cdot f_c / h_{st} \quad (A1)$$

Biaxial bending: $V_{y,flex}^{col}$ can be considered equal to the 70% of the respective value of uniaxial bending.

where $\rho_{l,tot} = A_{s,tot}/A_g$; $A_{s,tot}$: total area of longitudinal reinforcement; f_y : steel yield stress, f_c : concrete compressive strength; ξ from Fig. 4; $v = N/(A_g f_c)$; N : compression force acting in the section from $g+0.3q$ load combination (compression as a positive value); h_{st} : clear storey height; d_c : column depth; b_c : column width.

(ii) Calculation of column shear strength [6]:

Monotonic loading:

$$V_{u,shear}^{col} = V_w + V_c \quad (A2)$$

$$V_w = \begin{cases} 0 & \text{for } d_c < S \\ A_{st} \cdot f_{st} \cdot \frac{d_c}{S} & \text{for } d_c \geq S \end{cases} \quad (A3)$$

Note: For 90° stirrup anchorage V_w must be reduced by 50%

$$V_c = \begin{cases} 0.5 \cdot \sqrt{f_c} \cdot \left(\frac{d}{L_s} \cdot \sqrt{1 + \frac{N}{0.5 \cdot \sqrt{f_c} \cdot A_g}} \right) \cdot A_g & \text{if } v \geq (\rho_{s1} - \rho_{s2}) \cdot f_y / f_c \\ 0 & \text{if } v < (\rho_{s1} - \rho_{s2}) \cdot f_y / f_c \end{cases} \quad (A4)$$

Postyielding response:

$$V_{u,shear} = 0.6 \cdot (V_w + V_c) \quad (A5)$$

where, A_{st} : area of stirrup leg; f_{st} : yield stress of stirrups; S : spacing of stirrups; L_s ($\approx h_{st}/2$): shear span length; $\rho_{s1} = A_{s1}/A_g$; $\rho_{s2} = A_{s2}/A_g$; A_{s1} , A_{s2} : area of tension and compression reinforcement, respectively.

(iii) Calculation of the longitudinal bar development capacity [6]:

At lap:

$$F_s = 1.4 \cdot (A_{tr} \cdot f_{st} \cdot n_{st} / n_b + p \cdot f_t \cdot L_b) \leq f_y \cdot (\pi \cdot D_{b,l}^2 / 4) \quad (A6)$$

where, A_{tr} : total area of stirrup legs along one direction of restraint; n_b : total number of bars restrained by a total of n_{st} stirrups, $p = 2\sqrt{2}(c+D_{b,l})$; c : concrete cover; $D_{b,l}$: longitudinal bar diameter; f_t : concrete tension strength.

At anchorage in beam-column joint:

Calculate F_s as if at lap, by substituting p by the concrete cover, c , which is the actual length of the crack path.

At ultimate conditions:

$$F_s = 1.4 \cdot (A_{tr} \cdot f_{st} \cdot n_{st} / n_b) \leq f_y \cdot (\pi \cdot D_{b,l}^2 / 4) \quad (A7)$$

(iv) Calculation of joint shear strength [7]:

$$V_{j,u} = \gamma_j \cdot \sqrt{f_c} \cdot b_c \cdot d_c \quad ; \quad \gamma_j = \begin{cases} 1.6 & \text{for interior joints} \\ 1.0 & \text{for exterior joints} \\ 0.6 & \text{for corner joints} \end{cases} \quad (A8)$$

(v) Calculation of slab punching strength [8]:

$$V_{u,slab}^{col} = 0.12 \cdot \min \left\{ 1 + \sqrt{\frac{200}{d}}, 2 \right\} \cdot (100 \cdot \rho_l \cdot f_c^{1/3}) \cdot d_{slab} \cdot 0.25 \cdot u_{crit} \cdot (d_c + 4 \cdot d_{slab}) / h_{st} \quad (A9)$$

where, d_{slab} : slab depth; u_{crit} : the critical punching perimeter around the typical column in flat plate construction.