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SEISMIC BEHAVIOUR OF EXISTING RC BUILDINGS

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SUMMARY

The problem of quantifying the actual capacity and deformability of existing building becomes necessary in case of pre- or post-earthquake interventions. This study is concerned with the evaluation of the overstrength and the ductility of existing reinforced concrete buildings in Southern Europe, and particularly in Greece. Typical classes of such buildings are selected and designed according to the earthquake resistant design codes in effect for each examined period. Afterwards, both local and global strength and inelastic deformation characteristics are evaluated. Finally, inelastic pushover analyses are performed for each building and their overstrength and global ductility are evaluated. The influence of various parameters is examined, such as the vertical irregularity and the contribution of masonry infill walls. The results from nonlinear pushover analyses indicate that, depending on the design code in effect, the overstrength of existing buildings can be significant, yet their ductility is limited. Furthermore, the presence of infill walls in perimeter frames increases considerably the stiffness of the structures and their global resistance to lateral loads. For the forms of irregularity considered in this study, buildings with columns discontinuities in the ground floor exhibit the worse performance.

INTRODUCTION

Reinforced concrete buildings built in Greece and in other Mediterranean European countries up to 1980s, represent a significant portion of the whole building estate of those countries, and have been designed either in the absence of seismic codes (before the 50s), or with past generation of codes. In Greece, a large number of existing buildings dates back to the 60's and 70's due to the fast growth of constructions after the World War II. Structural damages in these buildings were observed after the recent earthquakes, some of which were catastrophic. Therefore, the assessment of the structural behaviour of existing buildings under earthquake motion and the resulting seismic vulnerability is a key topic with significant economical and social implications. These buildings were constructed between the early sixties and through the eighties, they were designed following allowable stress procedures under relatively low seismic coefficients, on the basis of a simplified structural analysis. Little or no provisions for capacity design and no critical region detailing were incorporated. The quality of structural materials used in such structures is often questionable, while the detailing practices at the time of construction (e.g., use of bent up reinforcement, improper anchorage of top and bottom steel in the beams, use of smooth reinforcement) and past seismic loading history present significant causes of uncertainty in their expected seismic behaviour.

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These buildings are typically infilled with clay brick infill walls, typically not assumed to be part of the lateral resisting system. The design of the bearing system is performed neglecting the interaction between the frame structure and the infills. Apart from these facts, existing buildings further exhibit intentional irregularities with height. These can be typically classified as a taller first storey for shops, plan setbacks at higher storeys, discontinuity in the vertical members to create an open space at the ground floor and discontinuity in the horizontal members at the first floor, necessary for a shop mezzanine or a walkway at the ground storey. Further discontinuities also exist in the layout of the perimeter infill walls (potentially creating a soft storey).

The knowledge of expected structural behaviour of existing buildings, classified according to the time of design, and the form of irregularity, provides the Structural Engineer with useful planning information for the retrofit or strengthening of these structure, following the development of a National Code for this purpose. A comprehensive analytical research programme is ongoing at the Reinforced Concrete Laboratory, National Technical University of Athens, aimed at quantifying the inelastic response of existing structures. Of interest are the estimation of behaviour governing parameters such as the structural overstrength, the collapse mechanism, the expected local distribution of damage, the distribution of energy dissipation, the structure global ductility and a rational force reduction factor "q", for use along the lines of modern seismic provisions for new construction in Greece [1] and Eurocode 8 [2]. Results of this analytical investigation for certain groups of buildings considered are presented herein.

DESCRIPTION OF STRUCTURAL FORMS CONSIDERED

All buildings considered herein are typically cast-in-place reinforced concrete structures with beams cast monolithically with slabs and supported by columns. Following the evolution of Greek earthquake resistant design codes since 1959, existing regular or irregular reinforced concrete buildings in Greece can be broadly classified in the following four categories:

a) Buildings of the 60s (group 60). These structures have been designed according to the 1959 seismic design code [3] following allowable stress procedures and simplified structural analysis models. Such buildings usually have regular column spacing with relatively short bay sizes (3.0 to 4.0 m) and do not have shear walls. All perimeter frames are infilled with unreinforced masonry walls, typically 0.25 m thick, of good quality, with window openings usually in the same positions at each floor. Partition masonry walls 0.10 m thick are used in the interior of the structure. Partial perimeter infill irregularity may be encountered at the ground or any of the upper floors, either intentionally or when the use of the building changed from residential to commercial. The cross-section dimensions of columns are usually small, reflecting the tendency of early designs to be fairly economic in concrete usage, structural materials exhibit wide scattering in their mechanical properties while structural elements and the building itself possess no critical region reinforcement for confinement nor any capacity design provisions were used in their design.

b) Buildings of the 70s (group 70). These structures have also been designed according to the 1959 seismic design code [3] following allowable stress procedures but more elaborate structural analysis models. The column spacing is again regular but the bay sizes are increased (5.0 to 6.0 m). Narrow reinforced concrete shear wall cores were introduced in the 70s at the elevator shaft. Partial infill irregularity is more frequently encountered at the ground floors. Again, structural elements possess no critical region reinforcement for confinement nor any capacity design provisions were used in their design.

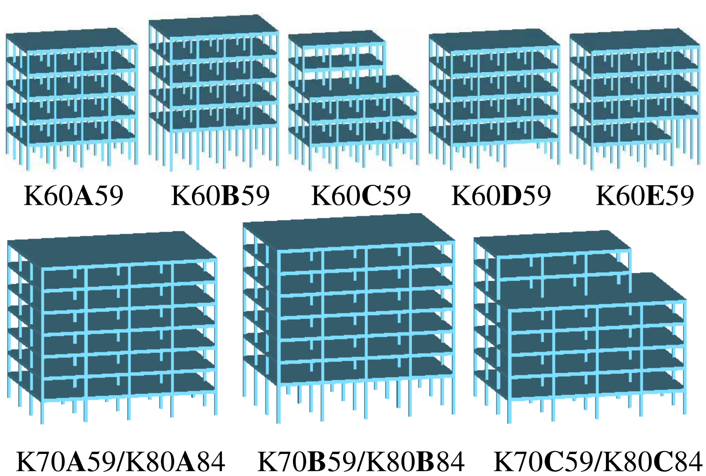
c) Buildings of the 80s (group 80). These structures are designed according to the 1984 Interim Seismic Provisions [4]. The seismic base shear coefficients are unmodified, while the design of buildings is still based on allowable stress verifications. Entire frame models using triangular seismic load distribution, substitute simplified analysis models. The building geometry remains the same with the 70s, although the buildings may now exhibit irregular column spacing and longer bay sizes, while often an open first storey (pilotis) is intentionally specified where the use of non-structural infill walls is avoided. Shear walls (primarily the elevator core and/or along the perimeter) are introduced and concrete member dimensions generally become wider. Some ductility provisions are introduced in the Interim Provisions [4] such as use of multiple closed stirrups with reduced tie spacing at critical regions, together with a requirement for a "pseudo" capacity design using allowable stress based resistance and strength.

d) Buildings of the 90s. These structures are designed primarily beyond 1995, following the application of the Greek Earthquake Resistant Design Code [1] and the Greek Code for Design of Concrete Works [5]. Both are ultimate limit state design codes encompassing most of the currently established ideas for local and overall structural ductility, introduced in contemporary seismic provisions in, among others, Eurocode 8 [2]. These modern seismic codes introduce concepts, among others, for local ductility, capacity design, weak beam strong column behaviour, confinement in critical regions, soil dependent response spectra, spectral methods of analysis and penalties for irregularity and excessive structural torsion. Structures in this period exhibit long spans, with or without an open first storey, adequate shear walls, sufficient concrete member dimensions and ductility provisions.

Out of a larger set of structural forms considered, five group of buildings are presented herein from the early period structures (group 60) and three of the other two categories (group 70 and 80), with various forms of vertical irregularity. Comparisons of infilled and bare frame structures are also presented. All the buildings are designed according to the Seismic Codes in effect and then their nonlinear deformation and strength characteristics are evaluated under pushover analysis. Methods proposed for evaluating existing buildings, such as the Capacity Spectrum Method in ATC-40 [6] and the N2 Method by Fajfar [7], are then used to evaluate the expected performance of these buildings. Some of the buildings are also designed according to the current seismic codes for comparison reasons. In the present study only results from pushover analyses are presented. Some comparisons of the results from pushover analyses with time history predictions using recorded or synthetic base excitations are presented in previous studies [8], [9].

SELECTION OF BUILDING MODELS

All buildings considered are four by three bays in plan (four bays in the direction of the earthquake). Wider buildings having more bays are presented in previous studies [8]. The early period buildings (group 60) are five storeys high, with a storey height of 3.00 m and regular 3.50 m bay sizes in both directions. The group 70 and 80 period buildings are seven storeys high, with a storey height of 3.00 m and 6.00 m bay sizes in both directions. The geometric representations of these buildings are shown in Figure 1.



K	Building
T1 – T6	Infilled Building
60/70/80	Period of construction (geometry)
A	Regular
B	Taller first floor of 5.0 m
C	One recessed bay at both directions at the upper two floors
D	Discontinuous column at the ground storey
E	Discontinuous beam at the ground storey
59/84/EAK	Earthquake Resistant Design Code
II	Seismic Zone II

Figure 1. Buildings Considered.

As it is mentioned above, masonry infills usually exhibit strong influence on seismic response of frame structures, as it appears from earthquakes and test results. The effects may be either positive or negative. Despite this fact, in conventional structural design of the buildings, the infills are usually neglected or taken into account indirectly in the current codes. In order to examine masonry's influence, for all structures considered, apart from the bare structures, fully and partially unreinforced masonry infilled perimeter frames are also studied and analyzed. Five different arrangements of unreinforced masonry infilled frames (with 0.25 m wide infills), are studied:

a) Building denoted T1: Fully infilled perimeter frames (Figure 2a).
b) Building denoted T2: Infilled perimeter frames leaving open ground floor (pilotis) (Figure 2b).
c) Building denoted T3: Partially infilled perimeter frames (Figure 2c).
d) Building denoted T4: Infilled perimeter frames leaving open intermediate (3rd) floor (Figure 2d).
e) Building denoted T5: Infilled perimeter frames leaving open two first floors (Figure 2e).

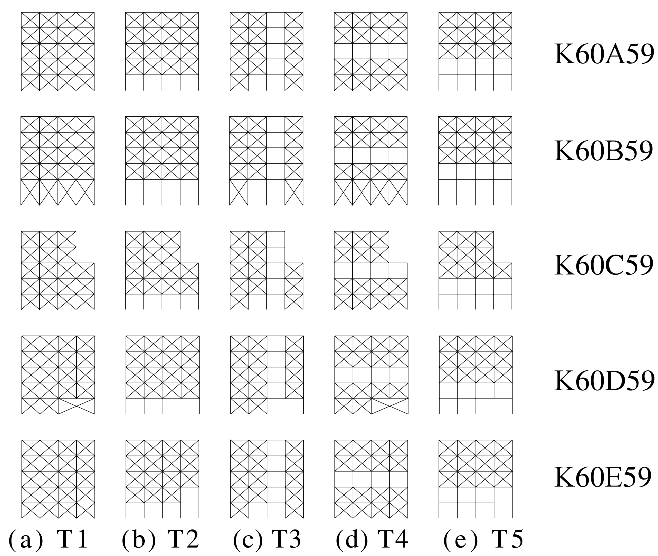


Figure 2. Types of masonry infilled perimeter frames

In order to study the influence of the shear wall cores, introduced in the 70s at the elevator shaft, the regular buildings K70A59 and K80A59 are designed and analysed without the presence of shear walls (Buildings denoted as K70A59nw and K80A59nw). Three buildings are designed for the medium seismic zone II, to take into account this parameter as well as to assess the effect of stronger earthquake motions. The sensitivity to the lateral force distribution is taken into account by performing different pushover analyses, for uniform and inverted triangular (design code) lateral force profiles, which represent the extreme cases. Finally, for comparison reasons, some buildings are designed and analysed according to the current Earthquake Resistant Design Code [1].

STRUCTURAL DESIGN OF BUILDINGS

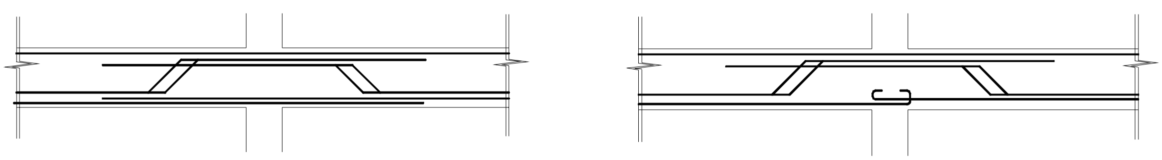
The design loads considered are 1.50 KN/m^2 surcharge and 2.00 KN/m^2 live load. According to the code in effect, the interior masonry infills are considered as an additional load uniform over the entire floor plan, equal to 1.00 KN/m^2, while exterior infills are 3.60 KN/m^2 of vertical surface area. Most buildings are designed for seismic zone I or II, following the 1959 Code [3] (hard soil), with allowable stress base shear coefficients of 4% and 6% the building weight, using an available commercial computer package [10]. According to the design codes in effect at the 60s through the late 80s, allowable service stresses for seismic load combinations are increased by 20%. All K60 buildings are designed with DIN B160 concrete (mean cube strength of 16 MPa) and DIN StI (S220 smooth) longitudinal and transverse reinforcement.

On the contrary, all K70 and K80 buildings are designed with DIN B225 concrete (mean cube strength of 22.5 MPa) and DIN StIII (S400 smooth) longitudinal and StI (S220 smooth) transverse reinforcement.

All K60x59 buildings for seismic zone I have 35x35 [cm] square columns at the first floor, reduced to 25x25 [cm] to the roof. An exception to this rule is buildings K60B59 and K60D59. In building K60B59 the columns of the ground floor are 40x40 [cm]. In building K60D59, column dimensions at the two sides of the vertical member discontinuity increase to 45x45 [cm] at the first floor, 40x40 [cm] at the second floor and 30x30 [cm] to the roof. For all buildings, column reinforcement ranges from 1.0% – 2.5% (gross section steel ratio). The beams are kept to dimensions 20/50 [cm], except for the beams under the discontinuous column which increase to 30/60 [cm]. In all cases, slabs are 12 [cm] thick. Generally, beams are lightly reinforced with steel ratios of the order of 0.4%, increasing to 1.7% for the beams supporting the discontinuous columns in building K60D59, which are normally more heavily reinforced. Column dimensions and the reinforcement ratio are increased for buildings designed for zone II.

K70x59 and K80x84 buildings, for seismic zone I, have 60x60 [cm] square columns at interior frames, and 90x25 [cm] columns at exterior frames of the ground floor. These dimensions decrease to 30x30 [cm] and 35x25 [cm] to the roof. These buildings have a shear wall core at the elevator shaft. The longitudinal reinforcement ratio (ρ) ranges from 1% to 3%. Compared to the 60s structures, the dimensions of the perimeter beams remain 25/50 [cm], while the interior beams increase to 20/60 [cm] and in some cases 30/60 [cm]. The longitudinal reinforcement ratios, however, in negative and positive bending (ρ- and ρ+) are increased relative to the K60x59 frames, due to the increased bay sizes of 6.0 m. It is noted that for these buildings, the slab thickness is increased to 16 [cm].

Improper detailing practices adopted at the time of construction are also considered in our analyses as follows. For building groups 60 and 70, insufficient anchorage in the joints of the bottom reinforcement bars of the beams is considered for the interior frames, while adequate anchorage is considered for the perimeter frame beams, since these beams were designed and constructed with additional checks based on plane frame analysis. Typical beam detailing considered in the analysis of the buildings of this period is shown in Figure 3. Thus, in the case of the perimeter frames, the requirement for increased development through the joint, results in the doubling of the effective bottom steel area, as shown below, something that was considered in the analysis. On the contrary, for building group 80, adequate anchorage is considered for both perimeter and interior beams, doubling the effective bottom steel area, as shown in Figure 3a.



(a) (b)

Figure 3. Beam Detailing for (a) Perimeter Frames and (b) Interior Frames

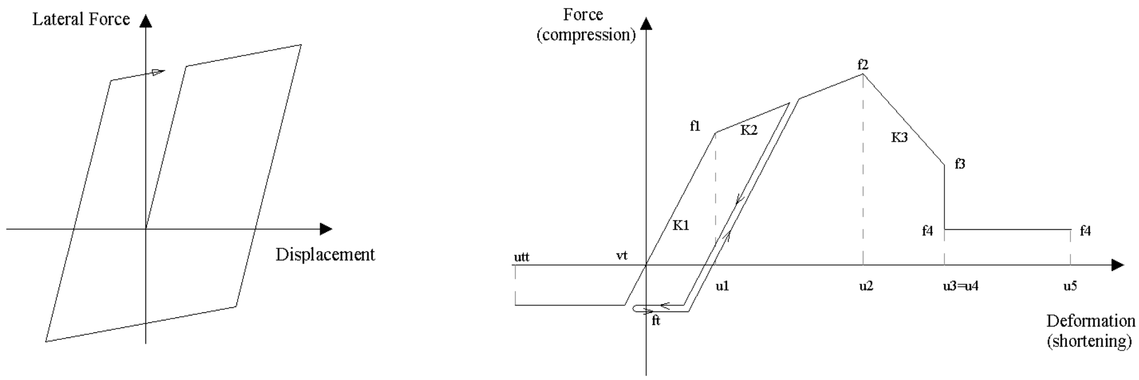
ANALYTICAL APPROACH

Modelling

All analyses are performed using an extended version of the computer program Drain-2DX by Allahabadi and Powell [11]. The selected buildings do not have plan irregularities. Therefore, the buildings can be subdivided into a series of vertical two dimensional frames, following the limitation of the program. Diaphragms are assumed to be rigid in each floor, thus the mass of all examined structures is taken lumped at the nodes. The active masses for inelastic analysis are assumed to be equal to dead loads plus only 30% of the design live load. Mass proportional damping is used. The damping coefficient is determined using the first mode periods of the undamaged structures.

Modelling of the frame

All the beams and columns of the structures are modelled using the two component lumped plasticity beam column element (type 02). The force-deformation model available for the beam-column element is shown in Figure 4a. Rigid end zones are used to represent the joints.

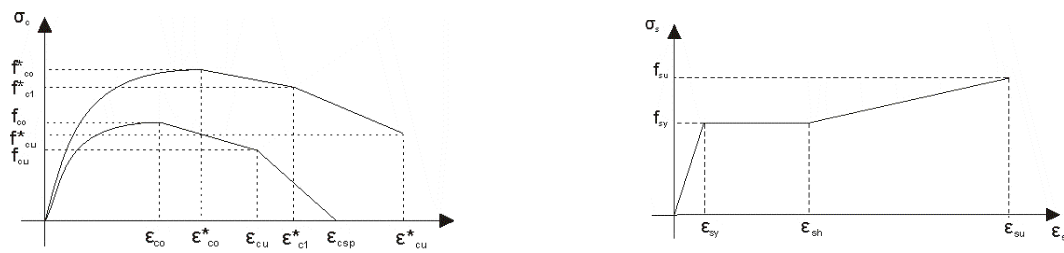


(a) (b)

Figure 4. Hysteretic behaviour of (a) beam-column elements and (b) infill walls

Reduced linear stiffness properties are adopted following the recommendations of the Greek Earthquake Resistant Design Code [1], in order to take into account the stiffness reduction due to cracking. 50% of the gross section properties are used for the beams, 100% for the columns and 67% for the shear walls. A post yield hardening stiffness of 0.5% of the elastic is specified for all members. Second order effects are included in the analysis following the initial gravity load distribution, acting together with the accidental earthquake load. For nonlinear analysis, effective slab widths of 1.0 m and 0.5 m are assumed for internal and external beams respectively, for the buildings with 3.5 m bay size. For the buildings with 6.0 m bay size, these values increase to 1.30 m and 0.65 m respectively. The slab reinforcement within the effective width is included in the calculation of the flexural inelastic characteristics of the beams.

For the estimation of maximum and yield curvatures, a section analysis module has been developed [12], for inelastic analyses of concrete sections. The analytical formulation is based on the plane section assumption under axial load and bending moment. The inelastic moment-curvature characteristics are developed for all the end critical regions of beams and columns of the subject buildings, using mean material properties and taking into account the effect of axial loads in the columns. The maximum strength is taken equal to 16 MPa and 22.5 MPa, for concrete DIN B160 and B225, respectively. For the reinforcing steel, the mean yield stress is taken as 310 MPa and 430 MPa for DIN StI and StIII, respectively, and the mean ultimate tensile strength is taken as 420 MPa and 630 MPa. In all cases, trilinear reinforcement behaviour and separate confined core and cover concrete constitutive models are considered (Figure 5). The confined concrete compressive strength is calculated for each member, according to its transverse reinforcement, following the model proposed in Model Code 90 [13].



(a) (b)

Figure 5. Stress – Strain diagram for (a) unconfined and confined concrete and (b) for reinforcing steel.

Modelling of the infill walls

The infill walls are modelled by equivalent diagonal struts, which carry loads only in compression. A simple element (Compression-tension link element – Type 09) provided by Drain-2DX was modified and used for modelling the infills [14]. The new element has trilinear behaviour with softening and remaining strength (Figure 4b). The trilinear envelope consists of an elastic part, a post-yield part with positive stiffness and a softening part with negative stiffness. The unloading stiffness is controlled by a parameter a between 0 and 1. For analyses presented herein, unloading stiffness is assumed equal to elastic (a = 0).

The overall mechanical properties of the masonry infill panels are determined from the elementary mechanical properties of the masonry, and are subjected to large uncertainties. Also, the properties of the masonry materials vary significantly, therefore a combination of material strengths is considered to represent weak and soft, and strong and stiff masonry. Nevertheless, in the present paper only results for rather good masonry are presented. Mainstone's approach is used to determine the initial stiffness and the effective width of the diagonal [15]. A simplified form, by Dolsek and Fafjar [16], of the expression proposed by Zarnic and Gostic [17], is used for the estimation of the maximum strength of infills, which takes place at an interstorey drift of 0.5%. The compression strength of the masonry infill is $f_m = 2.5$ MPa and the Young's modulus of elasticity $E_w = 750 \cdot f_m = 1.875$ GPa. The thickness of the masonry is 0.25 m.

Different properties for the equivalent struts are used, depending on the bay size or the height of the floor. The key model parameters of the infill resistance-deformation envelopes used are summarised in Table 1. In analyses it is assumed that there is no abrupt fall of infills' strength ($f_3 = f_4$), the strength decreases gradually until the residual value. Results with infills having abrupt reduction in their strength are presented elsewhere [9].

Table 1. Properties of the envelope of the equivalent diagonal strut in compression (For notation refer to Figure 4b, k is stiffness)

Panel Width [m]	Panel Height [m]	k_1 [KN/m]	$f_y = f_1$ [KN]	$f_{max} = f_2$ [KN]	u_2 [m]	k_3/k_1	u_3 [m]	$f_3 = f_4$ [KN]
3.5	3.0	27860.0	121.5	243.0	0.015	-0.10	0.089	36.5
3.5	5.0	15494.0	159.0	318.0	0.025	-0.10	0.199	47.7
6.0	3.0	39488.0	182.5	365.0	0.015	-0.10	0.093	54.8
6.0	5.0	32552.0	210.0	420.0	0.025	-0.10	0.134	63.0

Description of the limit state criteria

Local or global acceptance criteria are adopted, in order to estimate the lateral deformation capacity and therefore the limit resistance and global deformability and ductility of all the structures. Typically, all of the following response parameters are evaluated:

- Local inelastic rotation capacities at the end critical regions of beams and columns under different curvature or plastic hinge limits.
- Local shear force capacity of the individual members with or without axial load, depending on the member at hand.
- Global relative interstorey drifts to a value of 1.25% of the clear height.
- Global reduction of the base shear resistance of the structure to 85% its maximum due to second order effects, for the bare buildings only.
- Local failure of the masonry.

For the first criterion above, estimates of plastic rotation capacities are based on either section analyses or average values recommended by ATC-40 [6] for beams and columns of low compliance structures with non-conforming details (type C). Estimates of local plastic rotations $\Delta\theta_{pl}$ based on section analyses, are calculated following Eq. (1) below, using the predicted yield curvature φ_y and ultimate curvature supply φ_u of all the critical regions in all members:

$$\Delta\theta_{pl} = \Delta\varphi \cdot l_{pl}, \qquad \text{where} \quad \Delta\varphi = \varphi_u - \varphi_y \tag{1}$$

and the plastic hinge length l_{pl} is estimated with the following expressions ((2) to (4)):

- An expression provided by Paulay and Priestley [18], where d_b is the bars diameter and f_y the stress at yielding of the reinforcement:

  $$l_{pl} = 0.08 \cdot l_o + 0.022 \cdot f_y \cdot d_b, \text{ (units are in m and MPa)} \tag{2}$$

- An average plastic hinge length proportional to the depth of each section examined, where d is the section effective depth:

  $$l_{pl} = 0.50 \cdot d \tag{3}$$

Paulay and Priestley [18] suggested that this value may be often used with adequate accuracy for typical beam and column proportion.

- A more refined empirical expression provided recently by Panagiotakos and Fardis [19] for monotonic loading, where l_o is the shear span, a_{sl} is a zero-one variable for absence or presence of bar pullout from the anchorage zone beyond the section of maximum moment, d_b the bar diameter and f_y the yield stress of the reinforcement:

  $$l_{pl,monot} = 1.5 \cdot l_{pl,cyclic} = 0.18 \cdot l_o + 0.021\, a_{sl} \cdot d_b \cdot f_y \quad \text{(monotonic loading)} \tag{4}$$

Local shear force capacity is evaluated according to Eurocode 2 and 8, considering the concrete, the transverse reinforcement and the bent up reinforcement at the beams. Local failure of the infill walls is considered when they reach the strength degrading branch.

Evaluation of the Structural Overstrength and Ductility

The maximum deformability of each building at the roof level is established as the minimum roof deformation satisfying any of the above criteria, during the inelastic pushover analyses. The global equivalent yield displacement is estimated using an equal area bilinear approximation of the pushover curve, with an initial elastic secant stiffness intersecting the capacity curve at 60% of the maximum base shear, and a horizontal post-yield line that terminates at maximum displacement. As a consequence, global ductility is derived by dividing the maximum roof deformation with the equivalent yield deformation.

The structural overstrength ratio Ω is defined as the ratio of the maximum base shear resistance of the building, established from the inelastic static pushover analyses, to the ultimate limit state reference base shear, common to the design of all the buildings with the same geometry and seismic zone. The ultimate limit state reference base shear is defined as the product of the seismic zone coefficient ε, the weight W of the structure and the ratio of the allowable stress of the reinforcement, for flexural design, according to the prevailing Code, divided by the corresponding mean yield stress of the reinforcement.

RESULTS

A computer program (DrainExplorer) [12] was developed to post process the nonlinear analysis results and monitor in a step-by-step fashion the state of the structure during pushover and/or time history analyses. As a result, the drift limits, whereby limit state criteria considered above are exceeded, are provided. The program reads frame geometry, load profiles and inelastic analysis results from Drain-2DX [11], while the cross-section characteristics of all the members of the structure are analysed separately. The analysis generates the pushover curve, evaluates and checks the above limit state criteria step by step and plots the corresponding points on the pushover curve where these are exceeded. Therefore, for each limit criterion exceeded, the overstrength and ductility of the structure are estimated. In addition, the performance point is evaluated using two different performance point estimation methodologies: the "Capacity Spectrum Method" provided by ATC-40 [6] and the "N2 Method" by Fajfar [7]. Moreover, plastic hinge formation and local energy absorption per floor among different groups of elements, as well as the condition of the infill walls, where they exist, are shown step by step.

For every building described above, pushover analyses were performed with uniform and triangular distribution of lateral loads. In Figure 6 the resulting base shear – roof displacement characteristics (as well as their bilinear approximations), for all bare structures of group 60, are compared. At the same graph the points at which the limit state criteria considered are exactly exceeded, in any of the columns, are also shown, together with the corresponding performance point demands. In most analyses, it is observed that the most conservative value for the plastic hinge rotation capacity is evaluated when the plastic hinge length l_{pl} is estimated with the expression provided by Paulay and Priestley [18]. Therefore, at the following diagrams, only this limit is plotted for the plastic rotation capacity.

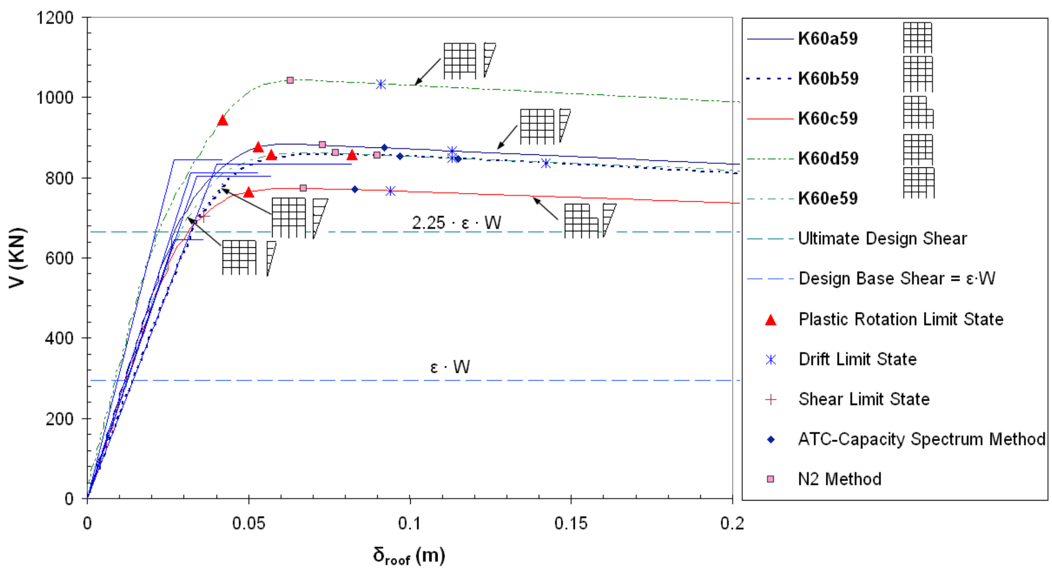


Figure 6. Inelastic pushover characteristics of buildings of the 60s for seismic zone I

The estimated base shear resistance V, overstrength Ω, global ductility μ and maximum roof displacement δ_u are given in Table 2 for most of the buildings considered. It can be seen that depending on irregularity, the minimum overstrength attained by bare frame structures of seismic zone I, for building group 60, ranges between 130% and 140%, which corresponds to maximum base shear resistance coefficient of 11.5% to 13% of their weight. Furthermore, it should be noted that, in general, performance values for inelastic response under a uniform distribution of lateral forces are relatively higher than under a triangular load distribution, and have a larger variation, e.g. overstrength ranges from 135% to 180%. Values presented herein, therefore, are based on analyses under an inverted triangular distribution of lateral forces, since they are more conservative. Ductility values in this case, for these buildings range from 1.3 to 2.0. More detailed results for bare frame buildings of the 60s are presented elsewhere [9].

Regarding the overstrength, a higher value is exhibited by the building with the discontinuous column, since both the dimensions and the reinforcement ratio of the columns are relatively larger. The larger columns due to the discontinuous column result even from vertical load design. A relatively lower value is obtained for the structure with the taller ground floor, while the setback or the discontinuous beams do not significantly influence the overstrength. Buildings with longer bay sizes, in general, exhibit lower values of overstrength. Finally, the overstrength of a structure is increased when it is designed with newer codes.

Table 2. Results from Pushover analyses.

Building	**T** [sec]	**V** [KN]	**Ω**	**μ**	$\boldsymbol{\delta_u}$ [m]	**Building**	**T** [sec]	**V** [KN]	**Ω**	**μ**	$\boldsymbol{\delta_u}$ [m]
K60A59	0.84	876.6	1.32	1.63	0.053	**K70C59**	1.01	1800.0	0.90	1.19	0.043
T160A59	0.44	2207.9	3.32	1.69	0.044	T170C59	0.59	4711.0	2.34	1.45	0.054
T260A59	0.51	1315.2	1.98	1.67	0.028	T270C59	0.61	2870.0	1.43	1.25	0.027
T360A59	0.51	1658.3	2.50	1.62	0.041	T370C59	0.64	3895.8	1.94	1.38	0.053
T460A59	0.59	980.5	1.47	1.62	0.028	T470C59	0.66	1679.3	0.84	1.25	0.015
T560A59	0.65	988.3	1.49	1.99	0.038	T570C59	0.68	2940.0	1.46	1.26	0.034
K60B59	0.97	858.7	1.29	2.03	0.082	**K70A59nw**	1.38	2570.3	1.11	1.52	0.101
T160B59	0.51	2103.6	3.17	2.01	0.064	T170A59nw	0.72	4109.3	1.79	1.35	0.045
T260B59	0.71	1017.1	1.53	2.05	0.043	T270A59nw	0.76	2870.4	1.25	1.23	0.029
T360B59	0.61	1517.8	2.29	1.93	0.059	T370A59nw	0.82	3348.3	1.45	1.28	0.045
T460B59	0.64	1032.3	1.56	1.59	0.033	T470A59nw	0.83	2710.3	1.18	1.21	0.032
T560B59	0.83	891.2	1.34	2.29	0.058	T570A59nw	0.90	2868.3	1.25	1.22	0.036
K60C59	0.72	705.0	1.29	1.33	0.036	**K80A84**	1.17	2300.5	1.00	1.12	0.051
T160C59	0.39	1845.2	3.36	1.45	0.036	T180A84	0.67	3871.3	1.68	1.27	0.034
T260C59	0.45	1049.5	1.91	1.29	0.018	T280A84	0.69	3777.7	1.63	1.30	0.035
T360C59	0.46	1519.6	2.77	1.55	0.042	T380A84	0.76	3415.5	1.48	1.24	0.037
T460C59	0.51	734.1	1.34	1.29	0.016	T480A84	0.74	1610.2	0.70	1.20	0.013
T560C59	0.57	930.0	1.69	1.37	0.025	T580A84	0.76	2520.4	1.09	1.22	0.024
K60D59	0.76	945.0	1.42	1.56	0.042	**K80B84**	1.33	2520.0	1.09	1.25	0.071
T160D59	0.43	2439.9	3.68	1.66	0.045	T180B84	0.75	4571.4	1.98	1.43	0.057
T260D59	0.47	1889.9	2.84	1.63	0.036	T280B84	0.83	4254.8	1.85	1.47	0.065
T360D59	0.49	1992.4	3.00	1.62	0.043	T380B84	0.87	4016.2	1.74	1.42	0.063
T460D59	0.55	1145.7	1.72	1.44	0.026	T480B84	0.83	1599.8	0.69	1.20	0.016
T560D59	0.57	1511.4	2.28	1.59	0.039	T580B84	0.94	2730.4	1.18	1.31	0.039
K60E59	0.87	856.9	1.33	1.69	0.057	**K80C84**	1.01	2295.0	1.15	1.13	0.05
T160E59	0.45	2135.8	3.31	1.69	0.044	T180C84	0.59	3829.2	1.91	1.23	0.035
T260E59	0.53	1234.9	1.92	1.70	0.028	T280C84	0.61	3760.4	1.87	1.31	0.036
T360E59	0.51	1607.7	2.49	1.58	0.041	T380C84	0.66	3417.6	1.70	1.23	0.038
T460E59	0.59	969.6	1.50	1.55	0.026	T480C84	0.66	1680.0	0.84	1.22	0.015
T560E59	0.68	951.3	1.47	2.08	0.042	T580C84	0.68	2520.0	1.26	1.24	0.025
K70A59	1.17	2156.8	0.93	1.19	0.055	**K80A84nw**	1.38	3304.7	1.44	1.83	0.164
T170A59	0.67	4402.8	1.91	1.39	0.046	T180A84nw	0.72	5775.8	2.50	1.41	0.076
T270A59	0.69	2870.4	1.24	1.24	0.026	T280A84nw	0.76	4620.6	2.01	1.39	0.059
T370A59	0.73	3703.6	1.61	1.32	0.046	T380A84nw	0.82	4883.8	2.12	1.37	0.078
T470A59	0.74	1610.2	0.70	1.23	0.014	T480A84nw	0.83	3699.9	1.61	1.29	0.048
T570A59	0.76	2590.4	1.12	1.25	0.027	T580A84nw	0.90	4130.2	1.79	1.47	0.068
K70B59	1.33	2430.0	1.05	1.42	0.092						
T170B59	0.74	3761.7	1.63	1.36	0.044	K60A59-II	0.76	973.5	1.00	1.72	0.053
T270B59	0.83	2800.4	1.21	1.24	0.037	K60D59-II	0.75	1120.0	1.12	1.59	0.049
T370B59	0.83	3209.9	1.39	1.28	0.048	K60E59-II	0.79	963.1	1.00	1.68	0.055
T470B59	0.82	1588.2	0.68	1.25	0.016	**K60AEAK**	0.63	1607.7	1.40	7.63	0.272
T570B59	0.94	2520.1	1.09	1.22	0.04	**K80AEAK**	1.28	3873.5	1.37	3.51	0.322

Ductility seems to depend on the bay size. Buildings with smaller bay sizes have higher ductility. The building K60C59 with the setback, has slightly smaller ductility because a column fails in shear below the setback. This structure appears weaker at the penthouse recess, since inelastic energy absorption mainly concentrates to the columns of the upper floors. The taller ground floor appears to increase the global ductility of the building, as long as this is considered into design. The building with the discontinuous columns has lower global ductility, since most inelastic action concentrates in these members, above the ground floor where the discontinuity occurs. The discontinuous beam does not influence the ductility of the structure. Finally, higher ductility is obtained when the structure is designed with newer codes.

In Figure 6, it can also be seen, that for all the structures with 3.5 m bay size, the performance point lies at greater drift than the one of the critical limit state, suggesting that the demand is higher than the capacity of the structure. This may improve if the frames are infilled. The lateral resistance of the buildings for seismic zone II is higher, yet their overstrength is reduced, because the increase is not in proportion to the increase of the seismic coefficient. On the other hand, the ductility is increased. The maximum drift capacity, compared to zone I frames, is slightly higher, but now the inelastic demand is almost doubled, making these buildings more critical from the point of view of higher seismic vulnerability.

In Figure 7 the inelastic characteristics, for the regular building K60A59, bare and infilled, are compared. It must be noted that only the perimeter frames are infilled. The maximum displacement at failure is decreased for the infilled structures, compared to the bare one. The critical limit state remains the plastic rotation capacity of columns. Shear failure of columns becomes more critical in the lower storeys of structures with partial height infills, as it is shown from other studies too [20]. The presence of the infills provides with a significant initial global stiffness increase. The shear capacity of columns is exceeded earlier than in bare frames, but this failure is not critical because plastic rotation capacity is exceeded first. In structures with an open storey, infills do not reach the strength degrading branch, since inelastic energy absorption concentrates to the columns of the open storey. However, for fully or partially infilled frames, infills in the lower part of the frame reach their maximum strength, but in a higher drift than the limiting one. The ductility is lower for the infilled structures, but the demand is also lower. After the failure of the infills, the lateral resistance approaches the bare frame level, but the performance demand is expected earlier. Partially infilled frames may behave satisfactory. Structures with weaker and softer infills exhibit smaller overstrength, but their global ductility is increased, as the interstorey stiffness ratio is reduced.

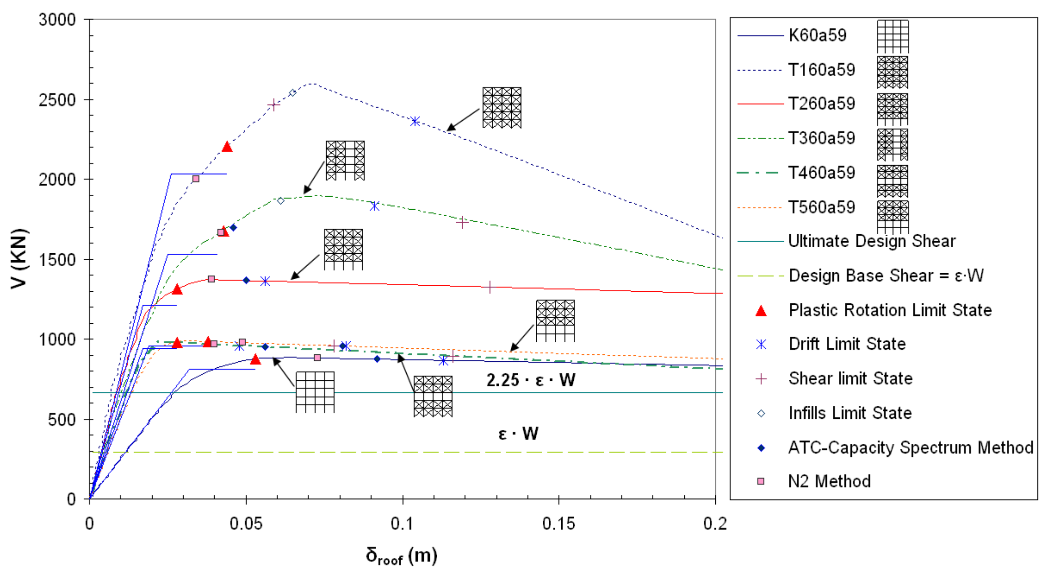


Figure 7. Inelastic pushover characteristics of building K60A59, bare and infilled structures

The structures of the 70s and the 80s have larger bay sizes (6.0 m) and a shear wall core at the elevator shaft. The shear failure is critical for these structures, because of the premature shear failure of RC walls. For this reason, the overstrength and the ductility of these structures are significantly smaller. On the contrary, in regular buildings K70A59nw and K80A84nw which don't have a shear wall core, the plastic hinge rotation capacity is the limiting failure criterion.

In Figure 8 the inelastic characteristics for the regular building of group 60, 70 and 80 are compared, in order to examine the influence of the design period and geometry of the frames. For this reason, structures without shear wall cores are compared (denoted as K70A59nw and K80A59nw). The overstrength and the ductility of building K60A59 are 130% and 1.65 respectively and are higher than for building K70A59nw (110% and 1.50). Building K80A59nw has higher overstrength and ductility from both K60A59 and K70A59nw (145% and 1.85), because this structure is designed according to the 1984 Interim Provisions [4] which introduced some ductility provisions and the requirement for a “pseudo” capacity design using allowable stress based resistance and strength. The demand is higher than the capacity for buildings of the 60s and 70s. On the contrary, for buildings of the 80s, without shear wall cores, the demand is lower than capacity. Finally, the demand is significantly lower for buildings designed according to the current codes.

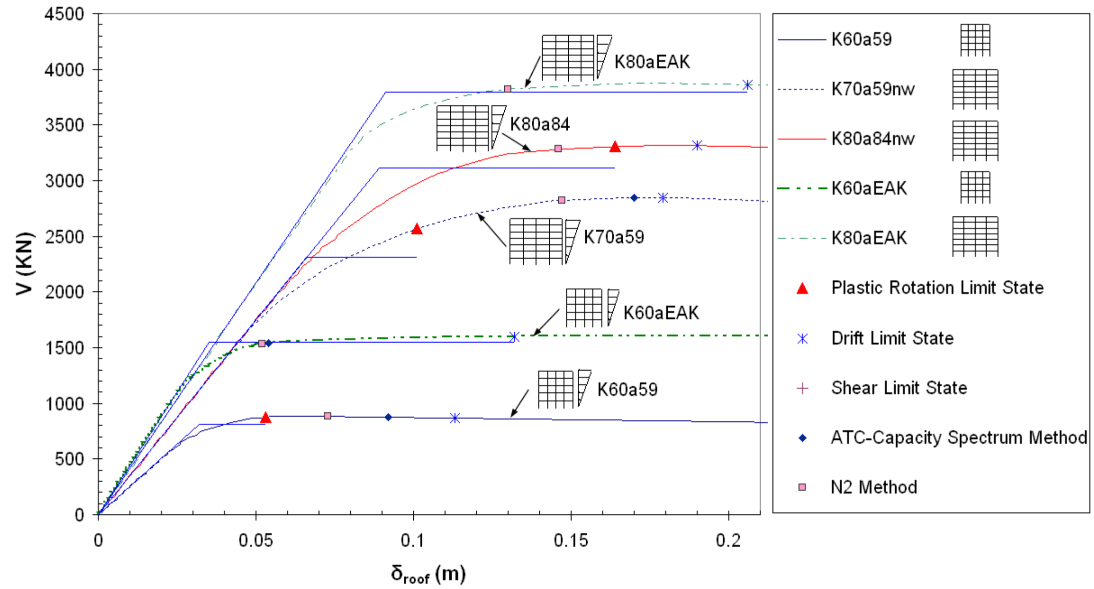


Figure 8. Inelastic pushover characteristics of the regular bare buildings

In Figure 9 the plastic hinge distribution of five structures is shown. Both the exterior and the interior frames are shown. The values at each joint indicate if the joint capacity design is satisfied, independently on whether or not this criterion was considered in design. Values less than one indicate that the capacity check is not satisfied at the joint, and the beams are stronger than the columns. In the same figure, the inelastic energy absorption at each floor, evaluated as the area under the bending moment – plastic rotation curves of hinged members, is also shown, as a percentage of the total energy absorbed at the building.

It can be seen that in the regular building K60A59, the capacity check is less than one, at most of the joints, while for the buildings of the 70s and the 80s is over one. This is in agreement with the plastic hinge distribution. The inelastic energy absorption in the case of triangular load profile results into a shift of the damage distribution towards the higher storeys. The exterior frames exhibit higher inelastic demands, since their columns bare lesser amounts of axial load, compared to the interior frames. Furthermore, for buildings of the 60s, relatively higher bottom reinforcement in the exterior beams results in high reserve flexural strength of these members, thereby concentrating all hinging primarily to the columns. On the contrary, buildings of the 70s, having bigger bay sizes, although they are not designed with joint capacity check, absorb the energy mainly in the beams. The same behaviour is exhibited by the buildings of the 80s, for which, however, capacity design is considered.

Buildings K60AEAK and K70AEAK, designed with the current codes with mandatory joint capacity design, concentrate the plastic hinges in the beams. A weak beam – strong column behaviour is presented, and the inelastic energy absorption is better distributed along the height of the structure. The seismic behaviour of these structures is superior to the buildings designed according to past generation of codes.

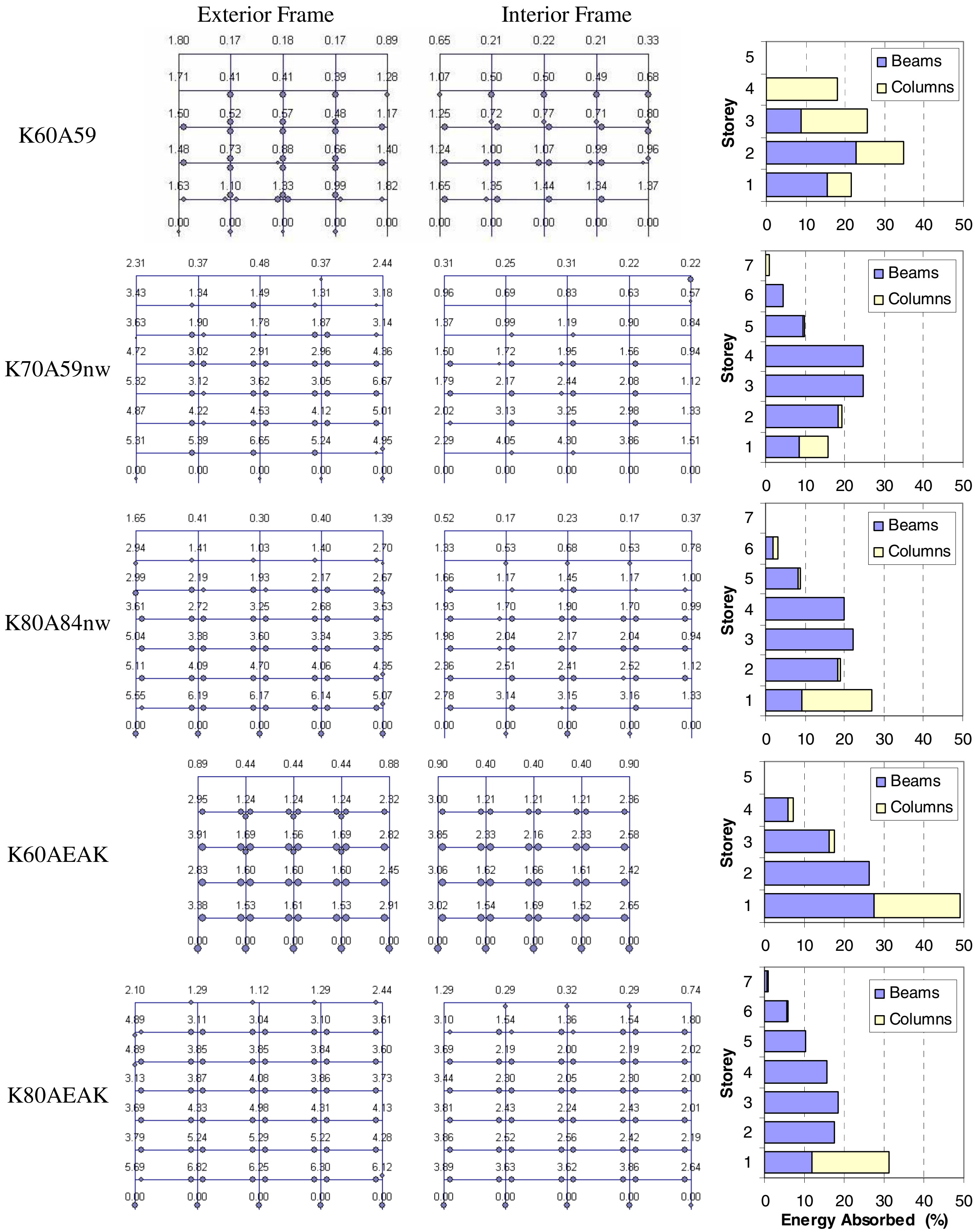


Figure 9. Plastic hinge pattern and local energy absorption over total energy with height (%). The values at each joint indicate the ratio of the sum of the capacity moments of the columns at the joint to the sum of the capacity moments of the beams coming to the joint ($\Sigma M_{RC} / \Sigma M_{Rb}$)

It must be noted, that it is also studied if shear capacity design is satisfied at each member of all the structures. It is shown that shear capacity design is satisfied for most of the beams and columns of the buildings with the small bay sizes, independently of the design code. On the contrary, for buildings with longer bay sizes, it is satisfied for the beams, but for columns it is satisfied only when the structure is designed according to the 1984 or the current design code. As expected, it is satisfied for buildings designed according to the current codes. More detailed results will be presented in the near future.

CONCLUSIONS

In summarizing, based on the analytical results described in this study, the following conclusions can be drawn for the inelastic response characteristics and the expected global ductility demand of typical existing regular and irregular reinforced concrete buildings designed under different generations of codes.

- Typical buildings of the 60s which have small bay sizes and are moderately high, behave better than the typical buildings of the 70s, designed by the same code, yet having wider bay spacing as well as being relatively taller. Buildings of the 70s and 80s with shear wall cores at the elevator shaft fail in shear at small lateral roof deformation. This is not the case for 70s designs without shear wall core. Such buildings, however, exhibit a lower overstrength and ductility than their counterparts of the 60s, mainly due to their larger bay sizes. On the other hand, buildings of the 80s with no shear wall core exhibit an improved behaviour than the latter two categories, with higher overstrength and ductility, since they are designed according to the 1984 Interim Provisions [4] which introduced some ductility provisions and the requirement for a “pseudo” capacity design using allowable stress based resistance and strength. The latter buildings exhibit failure at a larger drift, and the performance point demands are smaller than their provided deformation capacity.
- Infill walls influence the global and local behaviour of the structure considerably. In general, infills increase both stiffness and overstrength of the structure, but reduce its global ductility. An open floor may cause significant change in interstorey stiffness, creating a soft storey mechanism. On the other hand, partially infilled frames can exhibit an acceptable behaviour. It is noted, however, that due to the poor behaviour of buildings of the 70s, the influence of the infill walls is limited in these buildings, compared to the buildings of the 60s.
- The distribution of beam-column joints satisfying the joint capacity check enforced in modern code provisions, shows good correlation with the plastic hinge distribution obtained under inelastic pushover analysis. It has been observed that the behaviour of existing buildings that somehow satisfy this criterion, is generally acceptable, if shear failure is not prevailing.
- The assumed rigidity of the beam-column joint may not be totally correct for old structures with relatively small member dimensions and inadequate anchorage. This assumption needs therefore to be examined further with more reliable analytical models, because joints affect the structure stiffness and the inelastic behaviour, as well as they can fail themselves in shear.
- The behaviour of buildings designed according to current codes is superior, with the plastic hinges forming first in the beams instead of the columns, also exhibiting a satisfactory absorbed inelastic energy distribution along the height of the building. These buildings exhibit large values of overstrength (140% – 180%) as well as ductility (2.2 – 3.8). For these buildings the interstorey drift limit criterion is critical. Increasing the value of this limit, for which a relatively low value has been assumed for comparison with the older generation of buildings, even higher values for ductility supply are anticipated. It should also be noted that the performance point demand of these buildings is much lower than their deformation capacity.
- The results are sensitive to a number of analysis parameters, such as the lateral load profile and the performance point estimation method. Further analyses therefore need to be done, with other profiles too, (e.g. multimodal, adaptive pushover); the results need to be calibrated with time history analyses, using representative earthquake accelerograms.

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