Evaluation of the q Factor of Irregular RC Buildings Designed According to EC8

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SUMMARY:
This paper addresses the problem of seismic design of irregular RC buildings according to EC8 (EN 1998-1) and define: i) the complexity of the analysis model, ii) the design analysis methods to use and iii) the design penalty, through the specification of a lower behavior factor q. The reliability of this approach is evaluated through the investigation of the seismic performance of typical forms of vertically irregular buildings having different levels of irregularity. For this purpose, an automated and therefore objective design procedure is used, which has been developed for this purpose on the OpenSees inelastic analysis platform. Given the module versatility, different levels of vertical (in height) and/or torsional (in plan) irregularity can be quantified by suitably defining and/or removing selected structural members or entire bays in plan and elevation, by locally adjusting the bay length or floor height (stiffness irregularity) or by locally increasing the acting vertical loads in plan (mass irregularity).

Keywords: Stiffness, irregularity, Behavior factor, RC Buildings, Setbacks, EC8

1. INTRODUCTION

Architectural and functional requirements in the formation of building systems often impose on the structural engineer specific requirements in form that result in structural systems having vertical irregularities in geometry, stiffness or mass. When such buildings are located in a region of high seismicity, the engineer needs to have a thorough understanding of the seismic response of such irregular structures and reliable guidelines for the establishment of a safe, economic and reliable structural design that accounts for such irregularity, during inelastic response.

Several studies have been carried out in order to evaluate the seismic response of irregular buildings. Concentrating our emphasis on vertically irregular buildings, Aranda (1984) compared the ductility demands of regular and setback structures by analyzing such systems using bas input motions recorded on soft soil. He concluded that setback structures demand higher ductility than regular structures, the effect being more pronounced in the tower portion of the building. Ruiz and Diederich (1989) examined the seismic performance of buildings with a weak first story. A parametric study was carried out for 5- and 12- story buildings with a weak first story and, in some cases, with brittle infill walls in the superstructure. They concluded that the behavior of such irregular buildings greatly depends on the ratio of the dominant periods of excitation and response, the resistances of the upper to the first stories and on the base shear coefficient used for design.

Shahrooz and Moehle (1990) observed that damage is concentrated in the tower portion of a setback structure due to high rotational ductilities. Furthermore, they noted that the fundamental mode dominates the response in the direction parallel to the setback. Nasaar and Krawinkler (1991) evaluated the seismic demand parameters for bilinear and stiffness degrading Single-Degree-of-Freedom (SDOF) systems and for three types of Multi-Degree-of-Freedom (MDOF) structures of 3-, 5-, 10-, 20-, 30-, and 40-story height (and corresponding fundamental periods of 0.217, 0.431, 0.725, 1.220, 1.653 and 2.051 sec, respectively). The three types of MDOF structures considered were: a) the beam hinge model, in which plastic hinges form in beams only, b) the column hinge model, in which plastic hinges form in columns only, and c) the weak first story model, in which plastic hinges form in columns of the first story only. As far as the SDOF model are concerned, the inelastic strength and cumulative damage demands were evaluated statistically for specified target ductility ratios, thereby developing expressions relating the strength reduction coefficient R to period and target ductility ratio. In the study on MDOF models, it was found that the strength required for a specified target ductility ratio depended on the type of failure mechanisms that developed during severe earthquakes. Finally, they observed that the weak first story led to large variations in ductility and overturning moment demands.

Zeris, Tassios, Lu and Zhang (1992) examined the strength reduction factor (q) of vertically irregular plane reinforced concrete frames. Frames were designed according to the Eurocode EC8 and they had different heights in the first story. A computer design-nonlinear analysis algorithm was used in order to estimate q, which adopts collapse criteria of global or interstory drift and local curvature ductility comparisons at all critical regions. Using the program DRAIN2D for inelastic analysis, the critical base excitation intensity was evaluated, for a given earthquake record, at which nominal collapse was obtained due to the exceedence of drift limits and local curvature ductility demands at critical regions of the members. The previous procedure was demonstrated in the evaluation of the q factor of three six story, three bay RC frames with different first story heights. It was concluded that the estimated response reduction factors are higher than those assumed for design, except from the frame with a relatively tall first story.

Valmudsson and Nau (1997) examined the earthquake response of 5-, 10-, and 20- story framed structures with uniform mass, stiffness and strength distributions. The time-history analysis was compared with equivalent lateral force procedures; following the evaluation of the response, they concluded that a 50% increase in the mass of one floor increased the ductility demand by not more than 20%. Reducing the stiffness of the first story by 30%, while keeping the strength constant, increased the first story drift by 20-40%, depending on the design ductility. Contrary to the above, when the strength of the first story was reduced by 20%, the ductility demand increased by 100-200%, depending on the design ductility level. Also, a reduction of the first story strength and stiffness proportionally by 30% increased the ductility demand by 80-200%, depending on the design ductility level. Consequently, they concluded that the response parameters were not dependent on mass and stiffness requirements, but only on strength.

Al-Ali and Krawinkler (1998) focused on the effects of vertical irregularity by considering height-wise variations of seismic demand. They examined a 10-story building which was designed according to the strong-beam-weak-column philosophy. For their parametric study, they used an ensemble of 15 strong ground motions from the Western U.S., recorded on rock or firm soil after 1983. The building was analyzed in the time domain under both elastic and inelastic response, for different combinations of the distributions of mass, stiffness and strength. They concluded that mass irregularity was the least significant, while strength irregularity was more significant than stiffness irregularity. Moreover, stiffness and strength irregularity, in combination, was shown to be the most unfavorable factor for the response.

Das and Nau (2003) investigated the definition of structural irregularity for different vertical irregularities, namely stiffness, strength, mass and the presence of non-structural masonry infill, as prescribed in building codes. The set of buildings examined comprised low- to medium – rise structures including five, ten and twenty stories. Linear and nonlinear dynamic time-history analyses were performed on an ensemble of 78 buildings with different story stiffness, strength and mass ratios. All buildings had three bays in the direction of the ground motion. The buildings were designed according to Chapter 21 of ACI 318-99 and UBC 1997. They observed that most structures considered in their study performed well when subjected to the design earthquake ground motion. As a result, they concluded that the restrictions on the applicability of the equivalent lateral force procedure given in building codes, are unnecessarily conservative for certain types of vertical irregularities considered in their study. In more detail, they observed that the presence of irregularity alters the inelastic response of the building causing marked increases in the inelastic story drift, in the vicinity of the irregularity; however, in no case did the drift exceed the code-specified limit of 2%. Furthermore, the damage indices monitored were insensitive to both the mass ratio and the location of the heavier mass. For all categories of the buildings studied, the demands did not exceed the computed curvature ductility capacities for which the members were designed, despite the large increase in curvature ductility demand that was obtained in the plastic regions, in the vicinity of the irregularities.

Several building codes address the seismic design of RC buildings having vertical irregularities. For example, in the recent version of IS 1893 (Part 1)-2002 (BIS, 2002), an irregular configuration for buildings is being defined explicitly. In fact, five types of vertical irregularity are defined, namely: a) stiffness irregularity, b) mass irregularity, c) setback irregularity (in geometry), d) discontinuity in capacity, e) in-plane discontinuity in lateral-force-resisting elements in the vertical direction. Moreover, in the NEHRP code (BSSC, 2003) vertical irregularities are classified similar to IS 1893 (Part 1)-2002 (BIS, 2002) while a structure is defined as irregular if the ratio of one of the system parameters (such as mass, stiffness or strength) between adjacent stories exceeds a minimum prescribed value. These values have been defined using judgmental criteria. In addition to the issue of irregularity classification, building codes require the use of dynamic analysis methods for irregular structures to establish the design lateral force distribution, rather than using equivalent lateral force procedures. In what follows, the irregularity quantification and the analysis methods dealing with vertical irregularity, in accordance with the requirements of EC8 (2004), are considered in more detail.

2. CRITERIA FOR STRUCTURAL REGULARITY AND NON-REGULARITY IN EC8

In accordance with EC8 (2004), building structures are categorized into being regular or non-egular for the purpose of seismic design. Vertical irregularity significantly affects the behavior factor q, which should be decreased in the case of the buildings being non-regular in elevation. In particular, for irregular buildings in elevation, the decreased values of the behavior factor are given by multiplying the reference values by 0.8. In EC8 the behavior factor q is defined as follows (Eqn. 2.1):

(2.1)

where
q_o is the basic value of the behavior factor, dependent on the type of the structural system and its regularity in elevation,
k_w is the factor reflecting the prevailing failure mode in structural systems with walls (see EC8, Section 5.2.2.2).

For buildings that are regular in elevation in accordance with EC8, the basic values of q_o are given in Table 2.1.

Table 2.1. Basic values of the behavior factor q_o for systems which are regular in elevation

STRUCTURAL TYPE	Ductility Class Medium	Ductility Class High
Frame system, dual system, coupled wall system	$3.0\ \alpha_u/\alpha_1$	$4.5\ \alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\ \alpha_u/\alpha_1$
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

α_u and α_1 as defined in EC8 (2004).

For buildings which are not regular in elevation, the value of q_o should be reduced by 20%. In EC8 (2004), a building can be categorized as regular in elevation, if all the following conditions are satisfied:

1) All lateral load resisting systems, for example structural walls, shall extend without interruption from their foundations to the top of the building or to the top of the relevant zone of the building, if setbacks exist at different heights.

2) Both the lateral stiffness and the mass of the individual stories shall remain constant or decrease gradually without abrupt changes, from the base to the top of a particular building.

3) In regard to framed buildings, the ratio of the actual story resistance to the resistance required by the analysis should not vary disproportionally between adjacent stories.

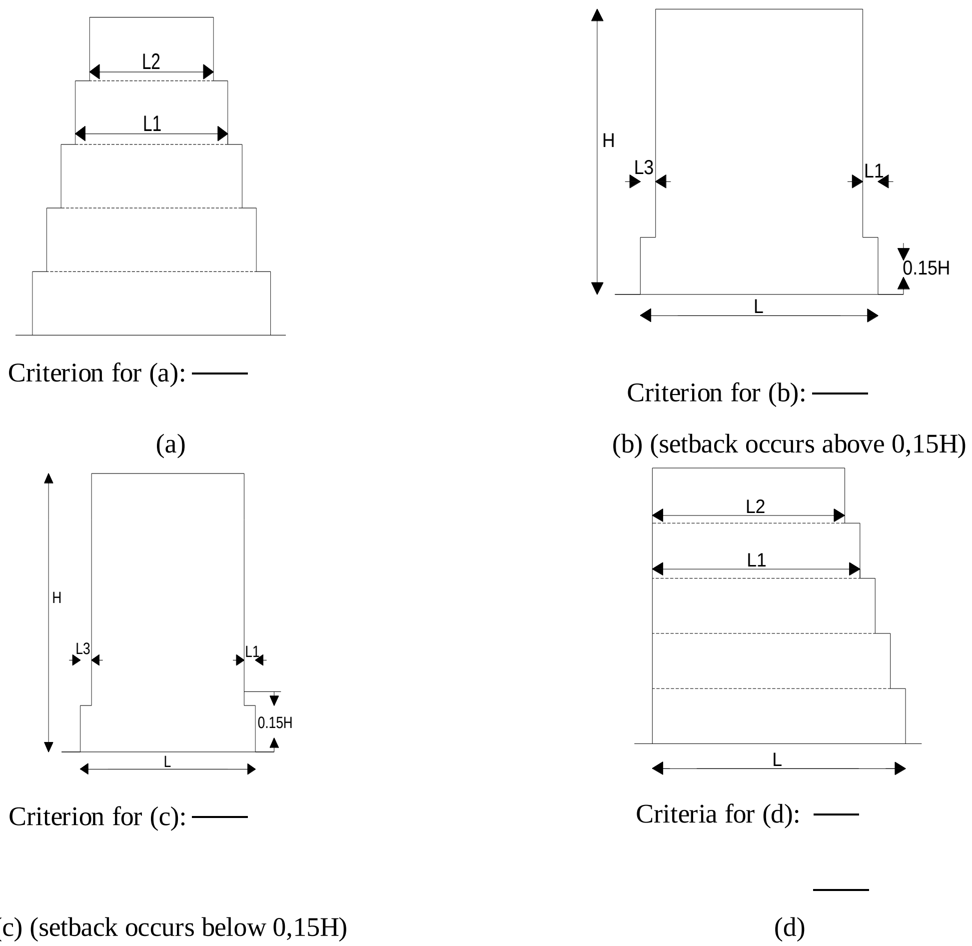

Figure 1. Criteria for (ir)regularity of buildings with setbacks following EC8 (2004).

4) In case of existing setbacks, the following additional conditions apply:

a) For gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20% of the previous plan dimension in the direction of the setback (Fig. 1.a, b).

b) For a single setback within the lower 15% of the total height of the main structural system, the setback shall be not greater than 50% of the previous plan dimension (Fig. 1.c). In this case the structure of the base zone within the vertically projected perimeter of the upper stories, should be designed to resist at least 75% of the horizontal shear force that would develop in that zone in a similar building without the base enlargement.

c) In case of setbacks without symmetry, for each face, the sum of the setbacks in all the stories shall be not greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10% of the previous plan dimension (Fig. 1.d).

If any one of the above is not satisfied, the building will be assumed as non-regular and the behavior factor *q* shall be decreased. It should be mentioned that a behavior factor *q* of up to 1.5 shall be used in deriving the seismic actions, regardless of the structural system and the regularity in elevation.

3. DESCRIPTION OF THE ALGORITHMIC ANALYSIS PROCEDURE

In order to evaluate the adequacy of the seismic design provisions of irregular frame buildings designed according to EC2 and EC8 (2004), a general frame design algorithm has been developed, as described herein, for defining inelastic RC frame designs of such irregular structures in an objective manner. The algorithm is developed on the Open System of Earthquake Engineering Simulation (OpenSees) platform (Mc Kenna et al., 2007). Given the versatility of this code and the fact that the entire building definition and simulation is entirely programmable in the Tcl language syntax and the OpenSees object structure, it is possible to objectively develop a fully automated and therefore objective design procedure for EC8 compliant design generations. The use of this procedure allows the user to define groups of irregular systems and to perform sensitivity analyses on these, by changing their relevant design parameters; in each case, a large number of inelastic simulations can subsequently be evaluated (resources permitting), for entire building types, thereby quantifying their inelastic performance under both static and/or dynamic excitation. In what follows, the algorithm (outlined in Fig.2 below), is described in some detail.

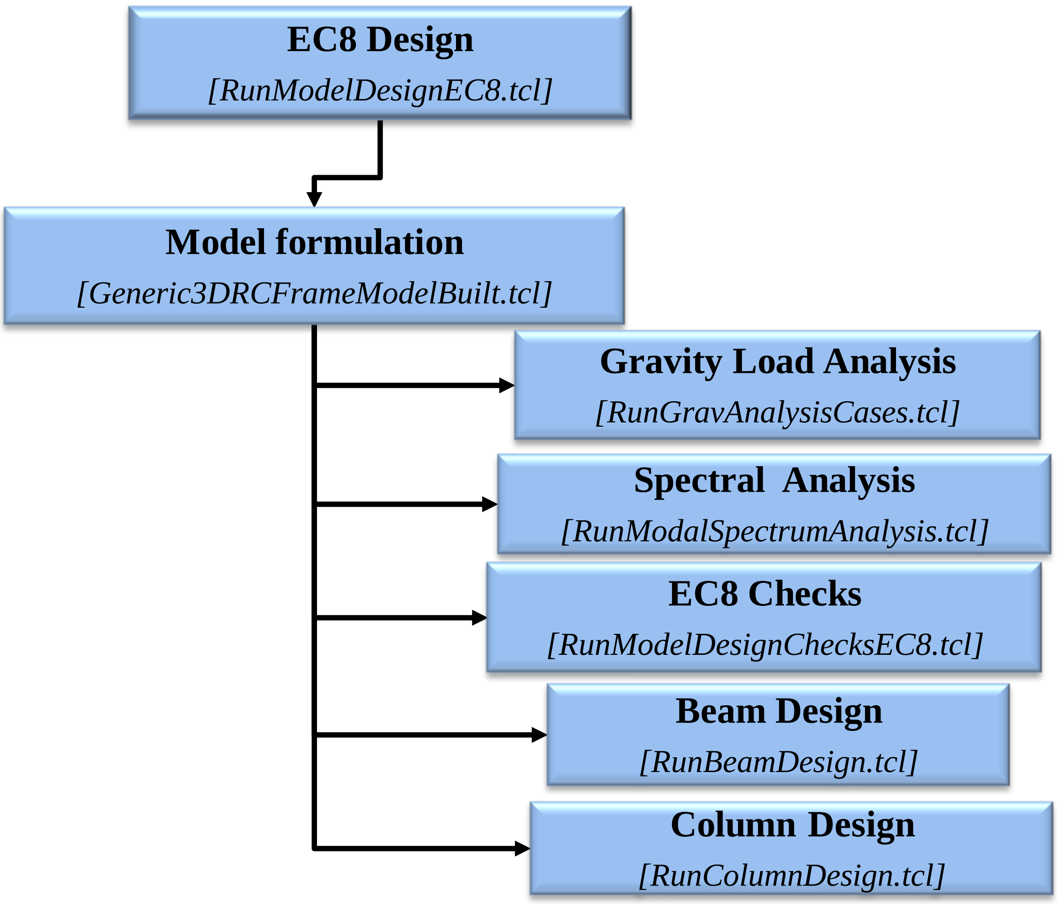

Figure 2. Principal modules of the EC8 RC frame design algorithm.

3.1 Model Formulation. All necessary system and EC8 design parameters are defined, including: i) the three-dimensional geometry of the frame, ii) the material capacities, iii) the initial design and subsequent analysis properties (partial safety factors for load and material strengths, Ductility Class, q_o factor), iv) the proposed cross section dimensions of the elements (namely vertical element with floor and entire beam lines per floor and per bay), v) the uniformly imposed area loads in plan (excluding self weights), vi) the topological connectivity between elements (slabs and their support boundary conditions each side, beams, columns) and vii) the base constraints.

The frame is assumed to be defined on a regular orthogonal grid of floor levels along the vertical (y) axis and transversely, along the (x,z) direction of bays. Following this convention, the definition of frame geometry includes the definition of the number of stories, the (variable) heights of these stories the number of bays in each direction and the length of these bays, respectively. Unequal lengths of entire bays or story heights are possible to define; members with a zero cross section are considered by the algorithm as missing and not contributing to mass and stiffness (e.g. a setback, a "planted" column or a discontinuous beam forming an arcade). Following this point, the algorithm entirely generates in OpenSees / Tcl syntax the entire frame input file for subsequent inelastic static and time history analysis, using the base module OpenSees. The structural model encompasses all the necessary commands, in order to subsequently implement the gravity and modal analysis.

Ultimate Limit State Analyses. All applicable load cases are considered and their corresponding design force envelope resultants are obtained for the structural elements. These are: i) load combinations of Dead plus Live Loads only, using the proper maximum and minimum partial load factors (favorable and unfavorable) and a checkerboard distribution of Live Loads at each floor; ii) the Dead plus reduced Live (by Ψ_2) plus Seismic Load combinations, using either the equivalent static load distributions or the modal distributions (depending on the system parameters, as per EC8) and corresponding accidental eccentricities simultaneously applied over the entire height on each side of the floor centre of mass. At this stage, serviceability limit state combinations are not considered.

RC Beam and Column Design. Following the envelope definitions of internal forces, design procedures are assumed for beams and columns. Regarding the design of RC beams, the following apply:
Initially the user specifies the Ductility Class per EC8. Then, in accordance with the design algorithm for the beams, the following are established for every critical section of a beam (three sections are considered, namely end i, end j and midspan section m):

1. Calculation of moments M_{sd}.
2. Calculation of μ_{lim} from general Tables CEB.
3. Calculation of A_{s1}, A_{s2} and comparison with maximum and minimum limits from Eurocode 2.
4. Finally, calculation of beam capacity and final steel ratios.

The capacity of the beams is evaluated prior to the design of the columns, so that joint overstrength coefficients can be determined for these elements. After the design of beams, every beam-column joint is checked and an additional envelope combination is created for column design, using the seismic combination loads.

Thereafter, the columns are designed. In more detail, each column section is initially checked for maximum compressive loads under the gravity plus seismic load combinations and following this, the section is designed for uniaxial bending. The procedure is performed four times, once for every axis and critical section of column element (y-axis, z-axis, base section, top section), observing in each case the Code required minima and maxima in each direction (the constructability of the reinforcement layout is required). Following the definition of the flexural strength in each direction, the section is verified in biaxial bending. As the case is for the beams, if the maximum steel ratios established are insufficient for the biaxial bending and axial load demands, a message appears for section inadequacy and the procedure stops, requesting an increase in the dimensions of the specific element; no automatic redesign is considered at this point. Eventually, the final steel ratios are extracted for all the beams and columns of the frame, satisfying strength, material characteristics and EC2 - EC8 (2004) limitations.

Inelastic Analyses. Once the design of the frame is completed, he RC structure is assembled as a set of fiber line inelastic elements (the *nonlinearBeamColumn* element is used), evaluating inelastic action at selected section segments at the Gauss Lobatto integration locations within the element (Mc Kenna et al., 2007); due to element limitations, all the beams are automatically split into five sub elements per span (three Gauss segments each), since the subject element does not handle internal distributed loads; columns are defined using five section segments. Subsequently, section objects using the "*Section*" command are automatically established, for all the beams and columns, with each fiber section object being composed of fibers, associated with a uniaxial material (*uniaxialMaterial Concrete01* and *Steel01* are used, respectively), an area and a location (y,z); all these parameters are established from the design section characteristics (section sizes, material properties and steel reinforcement) previously defined.

Following the definition of the frame elements, the algorithm proceeds with Gravity Load and Static Pushover analyses. Based on the OpenSees conventions, for the cases of Gravity and Lateral analyses two different Load patterns are defined, using the "*Pattern*" command. The corresponding Dead plus factored Live Loads in the span are automatically evaluated as vertical point loads at the interior sub-beam element nodes, in order to define the gravity load distribution. The lateral loads are applied at the frame nodes, connected to each other using diaphragmatic elastic truss elements of very large stiffness. The analysis response parameters are obtained through a set of "*recorder*" commands, providing information on nodal displacements, global force components and plastic rotation histories of the beams and columns.

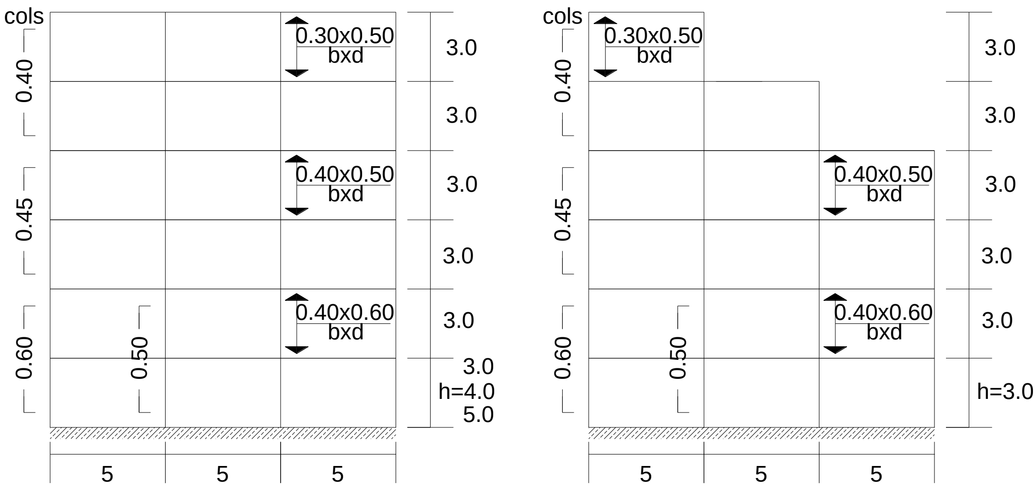

Figure 3. Frame geometry

4. INELASTIC CHARACTERISTICS OF TYPICAL LOW RISE IRREGULAR FRAMES DESIGNED PER EC8

The algorithmic procedure described above is demonstrated in the evaluation of the *q* factor of four six story, three bay in x-axis, three bay in z-axis RC frames having different vertical irregularities (a regular frame is also considered for reference). All frames have three spans 5m long in the x-axis and three spans 5m wide in the z-axis. The three frames differ only at their ground story, being 3m, 4m and 5m tall. The fourth frame has a regular story height but encompasses two setbacks, at the fifth and sixth story (Fig. 3).

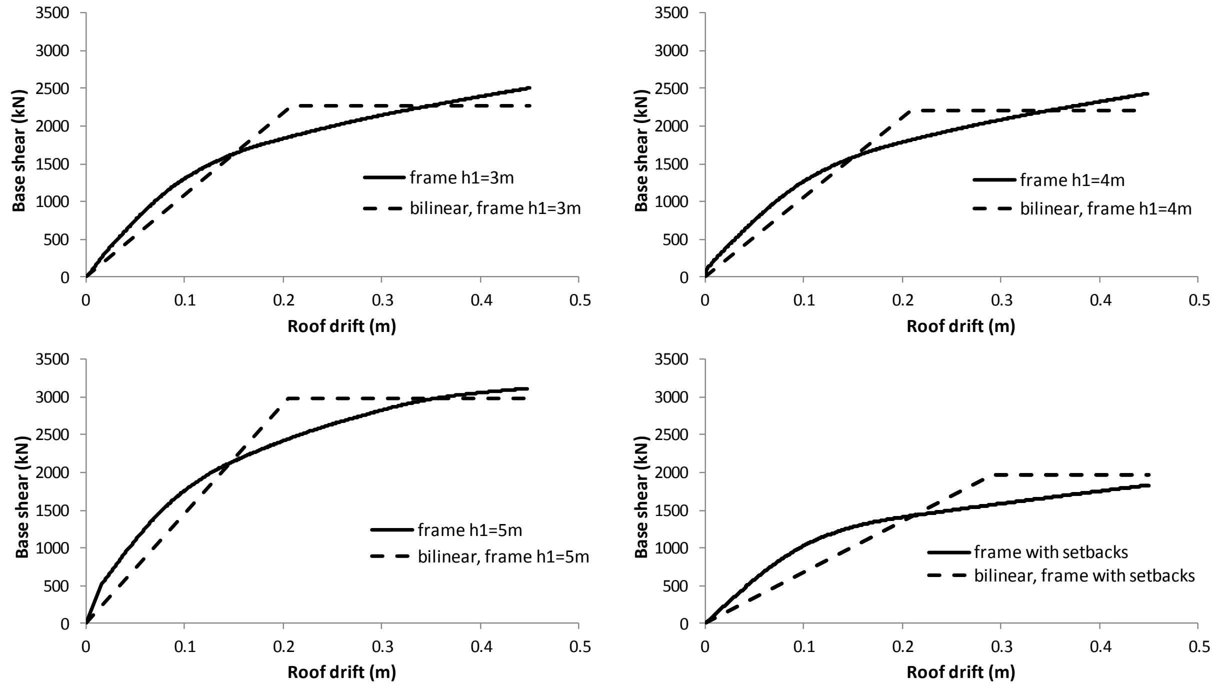

Figure 4. Inelastic base shear-roof displacement characteristics of the four frames

All frames were designed as medium ductility class (DCM) structures, for a peak ground acceleration of 30% of the acceleration due to gravity. Live vertical loads for residential construction and dead loads due to self weight plus an additional surcharge load of 2.5 kN/m^2 for light partitions and finishes were assumed. The frame cross section dimensions are shown in Fig. 3. The frames were designed using a design *q* of 3.5 and they were loaded under a monotonically increasing triangular load pattern, leading to the base shear-roof displacement characteristics compared individually and to each other in Fig. 4 and Fig. 5. In all cases, the equivalent bilinear base shear-deformation diagrams are also evaluated, following the usual convention of equal areas under the inelastic and bilinear curves and an initial elastic stiffness passing through the point of 60% of the peak base resistance.

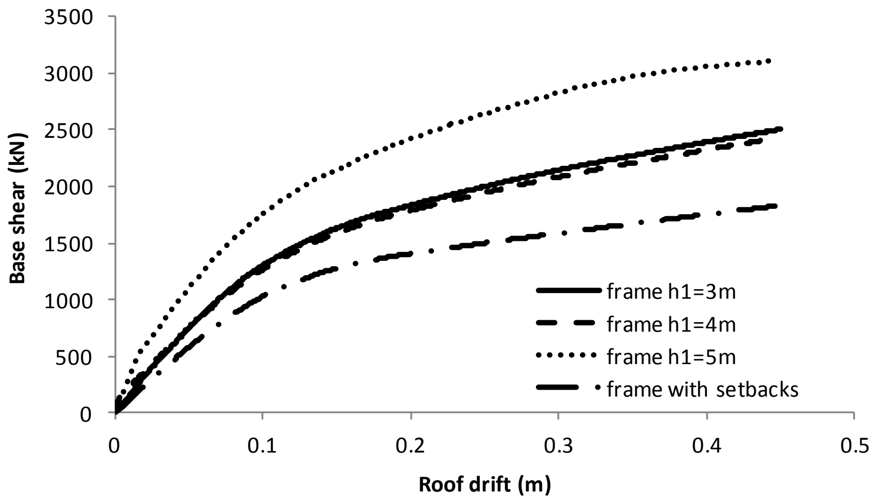

Figure 5. Comparative chart of inelastic base shear-roof displacement characteristics of the four frames

As expected, the influence of second order effects is more pronounced for the frame with setbacks, which is the most flexible and exhibits the lowest design base shear. Moreover, the frame with a 5m ground story develops the lowest plastic rotation demands, as shown in Fig. 6. This frame presents very low inelastic rotational demands, in contrast with the frame having a ground story of h1=4m which develops similar plastic rotation demands with the regular frame building (h1=3m). The frame with setbacks has lower demands than regular building, but higher than the frame with a ground story h1=5m.

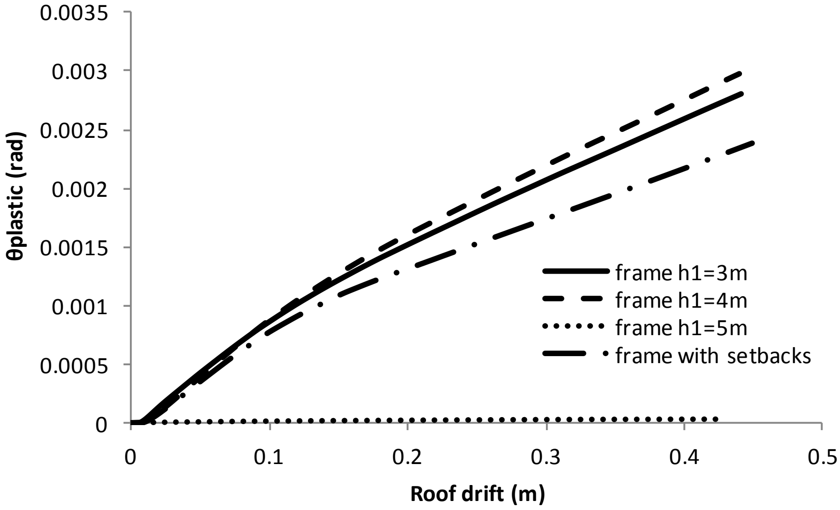

Figure 6. Comparative chart of $\theta_{plastic}$-roof displacement characteristics of the four frames

The variation of interstory drifts between all floors and all frames is shown in Fig. 7. Frames h1=3m and h1=4m attain their peak interstory drift at the fourth floor. On the contrary, frame h1=5m presents high interstory drift in fifth floor and as expected, frame with setbacks develops the highest interstory drift in sixth floor and generally it develops the highest interstory drifts in all floors, comparing to other frames.

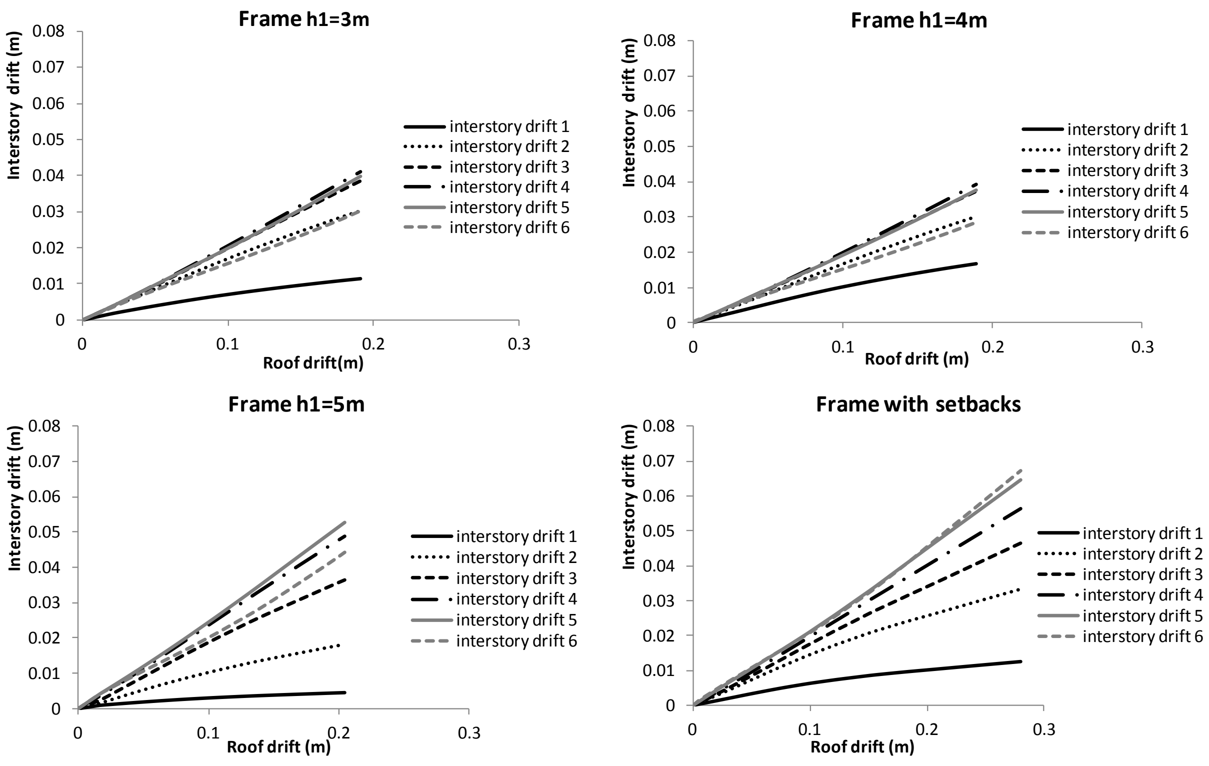

Figure 7. Comparative charts of interstory drift versus roof displacement characteristics for the four frames (all stories are depicted).

In accordance with EC8 (2004), q factors for the frames considered were calculated based on the estimated load – deformation characteristics established using code *RunModelDesignEC8*. The q factor depends on the type of structural system and on its regularity in elevation. From the above results and taking into account the target point demand for each frame for the Collapse Prevention Performance level, the corresponding q factor for the regular frame h1=3m and the frame h1=4m was calculated to be 6.60. For the frame with setbacks, a q factor of 4.50 seems to be sufficient. On the contrary, the frame with h1=5m demands a q factor equal to 8.50.

5. CONCLUSIONS

A computer algorithm which has been developed within the OpenSees analysis framework was presented. The algorithm can be used for the definition, formation and development of an EC2 and EC8 (2004) compliant design of a three-dimensional framed RC building on a regular grid pattern and, subsequently, the analytical evaluation of its strength reduction factor, q. For the objective evaluation of this factor, a nonlinear analysis following an automated linear analysis and design procedure are employed, in order to establish the inelastic demands.

The method is demonstrated for four typical low rise RC frames, initially designed per EC8. Three of these frames have different first ground story heights while the fourth is regular in height but has recessed setbacks in the fifth and sixth floors, with the remaining geometric characteristics being the same for all frames. From the analysis results, it is concluded that the estimated response reduction factors q are higher than those assumed for design, for all cases considered.

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