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OBJECTIVE EVALUATION OF THE q FACTOR OF IRREGULAR RC BUILDINGS DESIGNED ACCORDING TO EC8 - DESIGN AND ANALYSES PROCEDURES

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Abstract. *The seismic design of reinforced concrete (RC) mass or stiffness irregular buildings, in accordance with the classifications of Eurocode 8, is adjusted through the quantification of irregularity, through the use of irregularity indices and corresponding irregularity criteria. As a consequence, the design behavior factor in this case is no longer constant over a group of structures of similar form and building material, but varies on a case by case basis, since it involves modification of both behavior factors defining the design q, namely q due to structural overstrength (q_{Ω}) and q due to the local ductility supply of the members (q_{μ}). The reliability of this design approach is evaluated herein through the investigation of the inelastic performance of a form of irregular building with different degrees of irregularity per EC8. Focusing primarily herein - for the sake of simplicity - on vertical irregularity, an automated and therefore objective design analysis procedure is presented, which has been developed on the Open System of Earthquake Engineering Simulation platform (OpenSees). Following the definition of the basic framing configuration, characteristic loads and material grades, the procedure includes the following automatic steps: i) successive analysis of the frame for all EC8 required combinations, ii) evaluation of the internal design forces and envelopes thereof, iii) evaluation of flexural and shear reinforcement in all member critical regions, iv) evaluation of the local ductility supply of the members, v) formulation of the inelastic three-dimensional model of the structure (base is assumed fixed), using spread damage fiber section finite elements, vi) three dimensional analysis of the model both statically (modal load profiles) and in the time domain (incremental base excitation); and vii) establishment of response details and damage indices, such as seismic shear and deformation profiles, the collapse pattern, local and global ductility and supplied q. Given the module versatility, different levels of vertical and/or torsional irregularities can be established by removing selected structural members in plan, by adjusting selectively the plan or height geometry or by locally increasing the acting vertical loads. The process is applied to a typical tower; the code prescribed design procedure and irregularity indices are imposed and their reliability is established through local damage comparisons with the assumed design values.*

1 INTRODUCTION

Various functional and architectural requirements render important the presence of vertical irregularities in geometry, stiffness or mass between adjacent stories or continuous with height. Such vertical discontinuities increase the sensitivity to damages from earthquake ground motions and there are many examples of building failures in past earthquakes due to vertical irregularities. So, 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 [1] and Shahrooz and Moehle [2] compared the ductility demands of regular and setback structures by analyzing such systems. They concluded that setback structures demand higher ductility than regular structures and the effect is more pronounced in the tower portion of the building. Das and Nau [3] observed the same as aforementioned authors. 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 and results in large increases in curvature ductility demands in plastic regions.

Ruiz and Diederich [4] examined the seismic performance of buildings with a weak first story. 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 the base shear coefficient used for design. Moreover, Nasaar and Krawinkler [5] evaluated their seismic demand parameters and found that the strength required for a specified target ductility ratio depends 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.

Valmudsson and Nau [6] examined the earthquake response of multi-story framed structures with uniform mass, stiffness and strength distributions. They concluded that the response parameters were not dependent on mass and stiffness requirements, but only on strength. At the same time, Al-Ali and Krawinkler [7] focused on the effects of vertical irregularity by considering height-wise variations of the seismic demand and observed 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 under inelastic response.

Zeris et al. [8] examined the behavior factor q of vertically irregular plane reinforced concrete frames, designed according to Eurocode 8, with different heights in the first story. A computer design-nonlinear analysis algorithm was used in order to estimate q, which adopted collapse criteria of global or interstory drift and local curvature ductility comparisons in all the 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 exceedance of the above collapse criteria. The procedure was applied in three six story, three bay RC frames with different first story heights; it was concluded that the estimated q were higher than those assumed for design, except from the frame with a relatively taller first story.

Repapis et al. [9] compared the earthquake response of typical regular and irregular in height existing RC buildings, designed with past non-conforming seismic provisions in Greece. The buildings differed in the form of discontinuity of their beams and columns or according to the distribution of masonry infills in height. They concluded that past building generations with relatively short column spacing behaved relatively better, and that buildings with a taller first story behaved better than buildings with a penthouse recess or with short ground story columns due to the infill. Also, the influence of the distribution in height of the infill walls was significant, causing an increase in inelastic demands.

Chuny et al. [10] observed that irregular buildings in plan and elevation led to severe damage under strong earthquakes. Based on their results they proposed changes in their designs that contributed favorably to the seismic capacity of the irregular structure. Simultaneously, Sarkar et al. [11] investigated the vertical geometric irregularity in stepped building frames. They proposed a measure for vertical irregularity, involving the fundamental period of the stepped building superstructure, as a function of regularity index, which was shown to be more reliable than existing measures.

Contrary to all the above, Athanassiadou [12] examined multistory RC frame buildings, irregular in elevation, which have been designed to the provisions of the EC8 for the High (DCH) and Medium (DCM) ductility classes; the study concluded that the effect of the ductility class on the cost of buildings was negligible, while the seismic performance of all irregular frames appeared to be equally satisfactory, not inferior to (and in some cases superior than) that of their regular counterparts, even for base inputs twice as strong as the design earthquake. Moreover, Elassaly [13] examined the seismic response of irregular in height RC buildings, which had been designed according to the most common type of irregular buildings in Egyptian building environment. This type of buildings was distinguished from other studies in that it exhibited a varying first floor height. The study considered the seismic performance of such vertically irregular RC buildings and concluded that, in some cases, their response was superior to that of regular configuration, in terms of drift ratio and damage index limitations.

The above literature review reveals that the seismic behavior of setback or tower frames is a rather controversial issue, since some works indicate adequate seismic performance, while some others show the opposite for those frames. Therefore, more research work is needed in order to better understand the elastic and inelastic seismic response of such frames.

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,[14]), 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,[15]) vertical irregularities are classified similar to IS 1893 (Part 1) - 2002 (BIS) 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, as a common denominator, 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 in accordance with the requirements of EC8 [16] are considered in more detail, since this is the design code whose special irregularity provisions are under investigation herein. In order to address more efficiently research with engineering practice, an automated design procedure is developed on the Open System of Earthquake Engineering Simulation (OpenSees) platform [18]. The main features of the procedure are presented in the following sections.

2 CRITERIA FOR REGULARITY OF BUILDINGS WITH SETBACKS IN EC8

In accordance with EC8 [16] building structures are categorized into being regular or non-regular 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 reduced values of the behavior factor are obtained by multiplying the reference values by 0.8. In EC8 [16] the behavior factor q is defined as follows (Eqn.1):

$$q=q_o \times k_w \geq 1.5 \tag{1}$$

where

q_o is the basic value of the behavior factor, dependent on the type of the structural system and its regularity in elevation. For buildings that are regular in elevation, in accordance with EC8, the basic values of q_o are given in Table 1, below.

k_w is the factor reflecting the prevailing failure mode in structural systems with walls (see EC8, Section 5.2.2.2, (11)P).

STRUCTURAL TYPE	Ductility Class Medium	Ductility Class High
Frame system, dual system, coupled wall system	3.0 α_u/α_1	4.5 α_u/α_1
Uncoupled wall system	3.0	4.0 α_u/α_1
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

Table 1: Basic values of the behavior factor q_o for systems which are regular in elevation.

α_u and α_1 as defined in EC8 [16].

For buildings which are regular in plan and for frames or frame - equivalent dual systems the following approximate values of α_u/α_1 may be used:

− One-story buildings: α_u/α_1=1.1.

− Multi-story, one-bay frames: α_u/α_1=1.2.

− Multi-story, multi-bay frames or frame-equivalent dual structures: α_u/α_1=1.3.

Also, for buildings which are not regular in elevation, the value of q_o should be reduced by 20%. A building can be categorized as regular in elevation, if all the following conditions are satisfied:

1) All lateral load resisting systems, for example the 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.

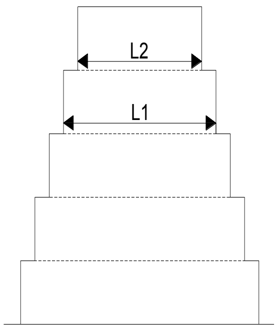
3) In regard to framed buildings, the ratio of the actual story resistance to the resistance required by the analysis should not vary disproportionally between adjacent stories.

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.1a, b).

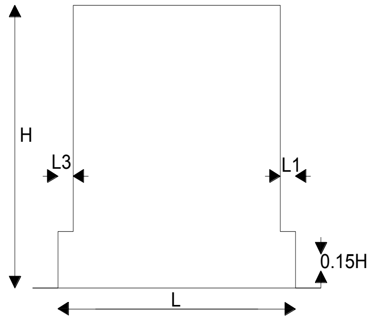
b) For a single setback within the lower 15% of the total height of the main structural system, the setback shall be not greater than 50% of the previous plan dimension (Fig.1c). In this case the structure of the base zone within the vertically projected perimeter of the upper stories, should be designed 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.1d).



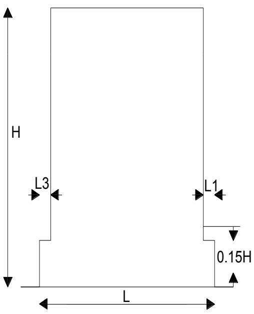
Criterion for (a): $\frac{L1-L2}{L1} \leq 0{,}20$

(a)



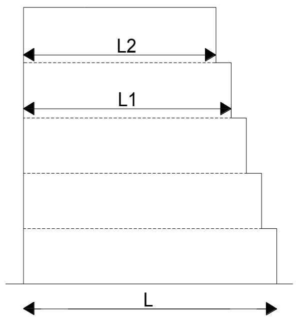
Criterion for (b): $\frac{L3+L1}{L} \leq 0{,}20$

(b) (setback occurs above 0,15H)



Criterion for (c): $\frac{L3+L1}{L} \leq 0{,}50$

(c) (setback occurs below 0,15H)



Criteria for (d): $\frac{L-L2}{L} \leq 0{,}30$

$\frac{L1-L2}{L1} \leq 0{,}10$

(d)

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

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 noted, however, that a behavior factor q of as low as 1.5 should be used in deriving the seismic actions, regardless of the structural system and the regularity in elevation.

3 DESCRIPTION OF MODELING AND ANALYSIS PROCEDURE

A general frame design and analysis algorithm has been developed for the automated definition of RC frame designs of irregular structures (and their corresponding inelastic analysis models) in an objective manner, in order to evaluate the adequacy of the seismic design provisions of irregular RC buildings according to EC2 [17] and EC8 [16]. The algorithm was developed on the OpenSees platform [18] and is been coded entirely in Tcl, an open source language with which the OpenSees language has also been developed. Due to 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 create a fully parametrically automated and therefore objective design procedure for EC8 [16] compliant design generations. This procedure gives the opportunity to the user to define groups of irregular systems compliant to the codes and to perform sensitivity analyses on these, by varying their design parameters. The algorithm includes both static analysis and/or dynamic excitation for the evaluation of the seismic performance of the design. In what follows, the algorithm (outlined in Fig.2 below), is described in some detail.

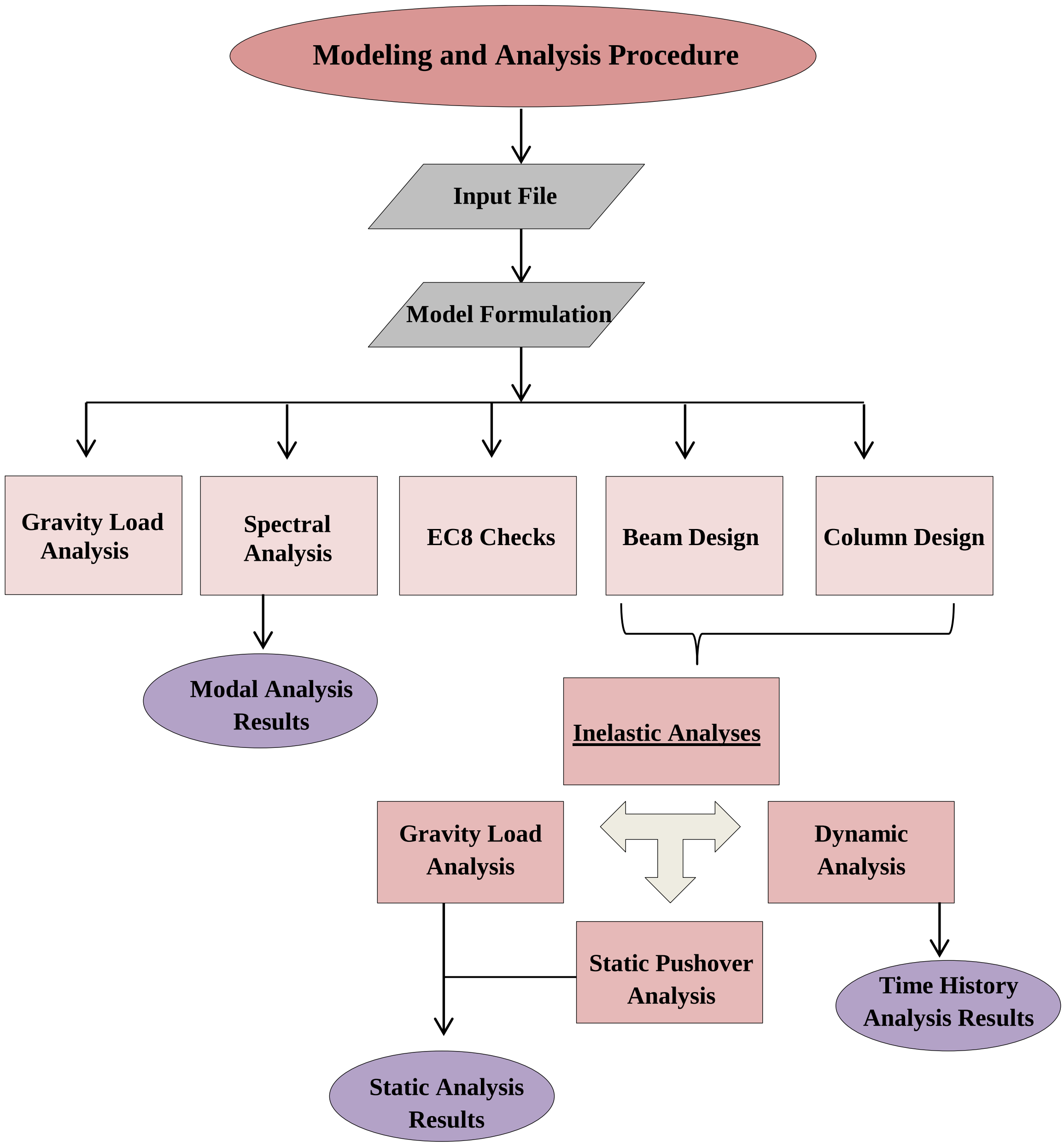


Figure 2: Principal Modules of the EC8 RC frame design and analysis algorithm.

3.1 Input File and Model Formulation

The Input File includes all necessary information for the definition of the geometry of the examined frame and the EC8 design parameters. In more detail, it contains input for: i) the material capacities, ii) the initial design and subsequent analysis properties (partial factors for load and material strengths, Ductility Class, basic behavior factor q_o, the design spectrum parameters), iii) the proposed cross section dimensions of the elements (namely vertical elements per floor and entire beam lines per floor and per bay), iv) the uniformly imposed area loads in plan (excluding self-weights, which are automatically established) and v) the three-dimensional geometry of the frame. 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” discontinuous column or a discontinuous beam forming an arcade in any location). Moreover, the definition of frame geometry includes the definition of the number of stories, the (variable) heights of these stories and 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.

In the Model Formulation file then, the three-dimensional frame model is initially defined on a regular orthogonal grid of floor levels along the vertical (y) axis and transversely, along the (x,z) direction of bays, observing the frame geometry and boundary conditions (commands *model*, *node* and *fix*, see [18]). Following this point, the algorithm entirely generates the topological connectivity between elements (slabs and their support boundary conditions each side, beams, columns) and the base constraints. In order to perform the initial elastic analysis model, the connectivity is achieved using “*element elasticBeamColumn*” [18], defining linear finite elements by specifying their Young Modulus, the Shear Modulus, the torsional moment of inertia of cross section and lastly the second moment of area about the local axes, compatible with the input geometry. Eventually, loads are imposed in each element (in slabs, in beams and finally in columns) and the algorithm calculates the nodal coordinates and masses of the frame for all the following analyses (command *mass*, see [18]).

3.2 Gravity Load – Spectral Analyses and EC8 Checks

Linear elastic structural models are successively analyzed and the corresponding design force envelope resultants are obtained for the structural elements for all applicable load cases. 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, [18]) and corresponding accidental eccentricities simultaneously applied over the entire height on each side of the floor center of mass. At this stage, serviceability limit state combinations are not considered.

After these analyses, the algorithm evaluates, checks and prints the adequacy of the design according to the restrictions of EC8, including the limit for interstory drift and the limit for second order (P-δ) effects, as well as all relevant irregularity indices and checks. Moreover, this file contains all necessary information about the spectral characteristics of the subject frame, given a first point of view for the frame analysis.

3.3 Member Design

Following the corresponding design force envelope resultants, design procedures are assumed for beams and columns.

Concerning the design of RC beams, the following apply:

Every beam is divided into three sections, namely end i, end j and midspan section m and the beams are designed according to the user defined the Ductility Class per EC8. Consequently, the following are established for every critical section of a beam:

1. Calculation of moments M_{sd} and μ_{sd}.

2. Calculation of A_{s1}, A_{s2} and comparison with maximum and minimum limits from Eurocode 2 and 8 using the reinforcing Tables based on EC2 flexural analysis.
3. Finally, the beam capacity and final steel ratios are evaluated.
4. Inadequate choice of initial dimensions or code requirement violation issue corresponding messages and the procedure stops.

According to the design of RC columns, the following apply:

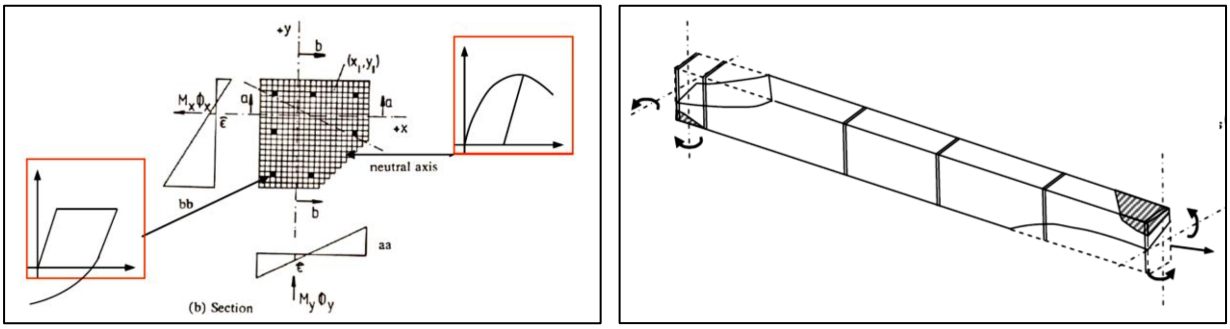
1. Each column section is checked for maximum permitted normalized compressive load (v_d) under the gravity plus seismic load combinations.
2. The section is designed for uniaxial bending with compressive load for all load combinations. This procedure is repeated four times, once for every axis and critical 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 section is also verified, in terms of amount and layout of reinforcement provided).
3. The flexural strength in each direction is defined.
4. The section is verified in biaxial bending. If the maximum steel ratios established are insufficient for the biaxial bending and axial load demands or the layout is improper, a message appears for section inadequacy and the procedure stops. Eventually, the final steel ratios are extracted for all the beams and columns of the frame, satisfying strength and material characteristics limitations.
5. After the design of beams and columns, every beam-column joint is checked, according to EC8 ($\sum M_{\mathrm{Rc}} \geq 1{,}3 \sum M_{\mathrm{Rb}}$, see [18]), so as to take into account the joint overstrength. In case that no restrictions are met, the algorithm corrects the moment of column and redesigns it, ending in final steel ratios.
6. Eventually, columns are checked for confinement in critical and non-critical regions, resulting in the transverse reinforcement (namely $\alpha\omega_d$, see [18]).
7. The column is checked and designed for shear, if confinement steel is insufficient, and new confinement properties ($\alpha\omega_d$) are evaluated.

3.4 Inelastic Analyses

Once the design of the frame is completed, the algorithm creates automatically the entire *RunBuilding.tcl* input file describing the entire inelastic finite element model of the designed structure for inelastic analyses. The file includes all the information for the geometry, the element connectivity, the fiber sections of the members and the corresponding uniaxial materials for concrete and steel, loads and analysis specifications and output information.

As before, for the definition of the design model, the three-dimensional frame geometry and masses with its boundary conditions are defined using commands: *model*, *node*, *fix*, and *mass*, [18]. Section objects are automatically established through the "*Section Fiber*, *patch* and *layer*" (see [18]) commands for all the beams and columns, with each fiber section object being composed of fibers, associated with an assembly of uniaxial materials (commands *uniaxialMaterial Concrete01* and *Steel01* are used, respectively, see [18]), in the corresponding local coordinate system (command *geomTransf*, see [18], also including the rigid offsets of half the contributing member size in each end); all these parameters are established from the design section characteristics (section sizes, material properties and steel reinforcement) previously defined. Figure 3 depicts the using uniaxial materials for concrete and steel and the distributed - plasticity element, from the library of OpenSees [18].

In terms of the finite element model representation per se, beams are created using displacement interpolation spread damage elements (*element dispBeamColumn*, see [18]). All the beams are automatically split into five sub elements per span with three Gauss integration monitored sections each, in order to handle internal transversely distributed loads in the beam (case Dead plus Ψ_2 Live). Columns, on the other hand, are defined using equilibrium interpolation spread damage elements (*element forceBeamColumn*" (see [18]), with five integration points per column.



a) b)

Figure 3: a) Uniaxial Materials and b) Distributed Plasticity Element.

The algorithm proceeds with the analysis of the design, including, in sequence, Gravity Load followed by Static Pushover analyses or/and Dynamic analysis. In the first case, two different Load patterns are defined, one for Gravity Loads and the other for Lateral Loads, (command *pattern Plain*, see [18]). The corresponding Dead plus factored Live Loads are automatically evaluated as vertical loads from the design of beams and columns. The Lateral Loads are applied at the frame nodes, connected to each other using diaphragmatic elastic truss elements (*element truss*, see [18]) of very large stiffness. In case of a time history integration dynamic analysis, the time resets to zero so that the dynamic analysis starts from time zero (command *loadConst* see [18]). In this case, the user defines the acceleration record for examination and applies the ground motion to the structure at the base (command *uniform excitation pattern*, see [18]). Additional user defined information for the dynamic analysis includes the acceleration record time series, the digitization time step of the ground motion and the analysis time step, the scale factor applied to the ground motion and the direction in which the motion is to be applied. In all cases, prior to the analysis, test, convergence, system numbering and the model constraints are also specified by the code.

In the end of the analysis, the response parameters are obtained through a set of output (*recorder*, see [18]) specifications, providing information on nodal displacements, global force components and plastic rotation histories and critical section information of the beams and columns. Nodal profiles, the frame geometry, mode shapes, internal force diagrams and selected time history outputs are subsequently graphically post processed using MATLAB [19].

4 OVERVIEW OF THE CASE STUDY

The algorithmic procedure described above is demonstrated in a six story irregular in elevation RC frame, with four bays along the x-axis and four bays along the z-axis (Fig.4), which has been designed according to the provisions of EC8 for the DCH and DCM ductility classes. The spans are 5m long both in x-axis and z-axis. The irregularity exists from the third floor and above by reducing the bays to two, so as to create a central tower. Live vertical loads for residential construction of 2.0 kN/m^2 and dead loads due to self-weight plus an additional surcharge load, including light partitions, of 2.1 kN/m^2 and a perimeter infill load of 3.6 kN/m^2 of facade area, were assumed. The frame cross section dimensions are shown in Fig.4.

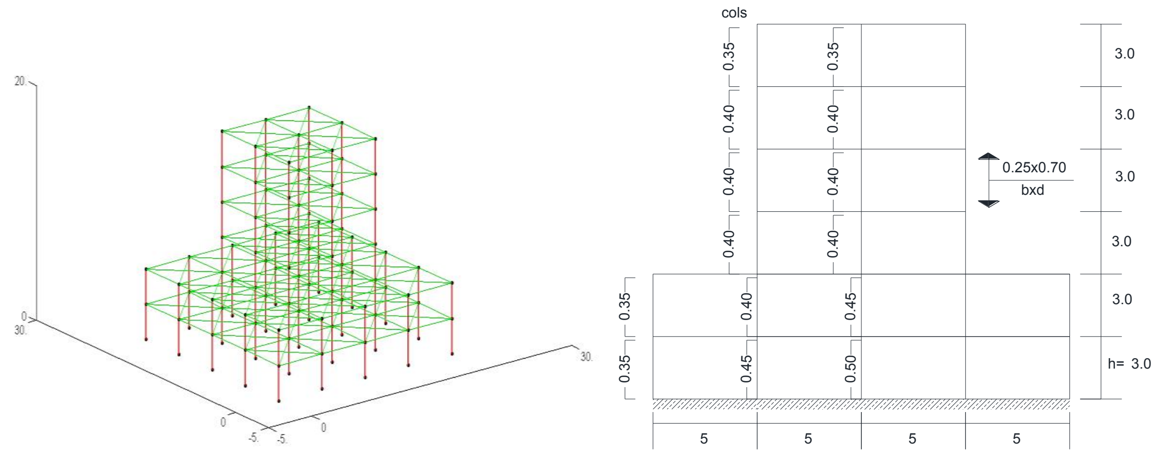
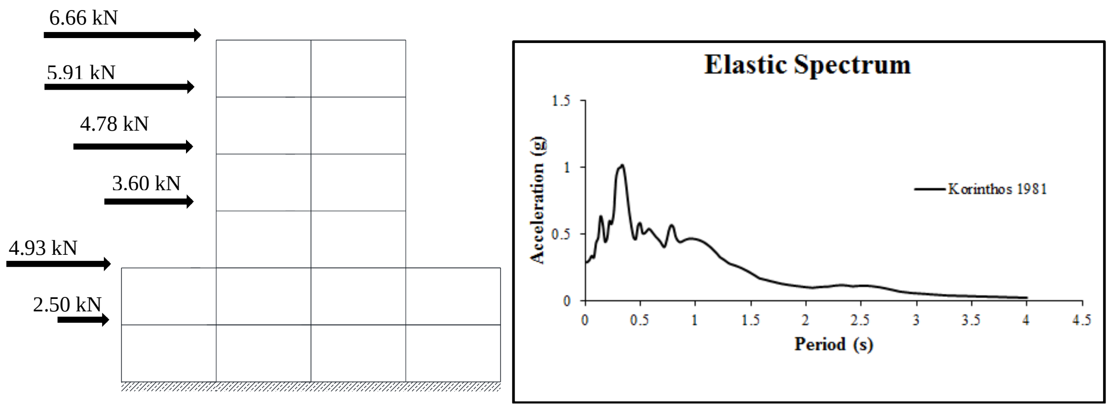


Figure 4: Frame Geometry (dimensions in m).

The structures were designed using a design behavior factor q of 3.12 and 4.68 for DCM and DCH respectively, with the first fundamental mode period of each frame equal to T_1= 0.514sec and T_1=0.535sec and for medium seismicity (design PGA of 0.24g). They were subsequently analyzed under: i) a Static Pushover (SPO) using a monotonically increasing modal load pattern, following the first lateral mode (Fig.5a) and ii) a time history dynamic analysis, using the OTE accelerogram of the 1981 Korinth earthquake (Fig.5b), with a peak ground acceleration (PGA) of 0.28g; in the latter case, two levels of excitation were considered, at 100% of the PGA (denoted PGA1) and at 200% of the PGA (denoted PGA2), respectively.



a) b)

Figure 5: a) Pushover Load Profile and b) Base Input Accelerogram Spectrum.

5 SEISMIC RESPONSE AND ESTIMATION OF THE DESIGN *q*

As mentioned above, the output from the OpenSees model enables the user to obtain information of the model response for nodes and elements, so as to check not only the local damages of elements, but also the global damage of the entire frame. The plastic hinges which are developed in frames during the analyses, are shown in Fig.6 for the External and Internal Frames and they are comparable for both ductility classes. The External Frame developed low demands of plastic hinge rotations ($\theta_{plastic}$ < 0.005 rad), in contrast with Internal Frame, which exhibited large demands of plastic rotations in the bottom and the top of the third floor columns and beams at the recess, at the base of the tower.

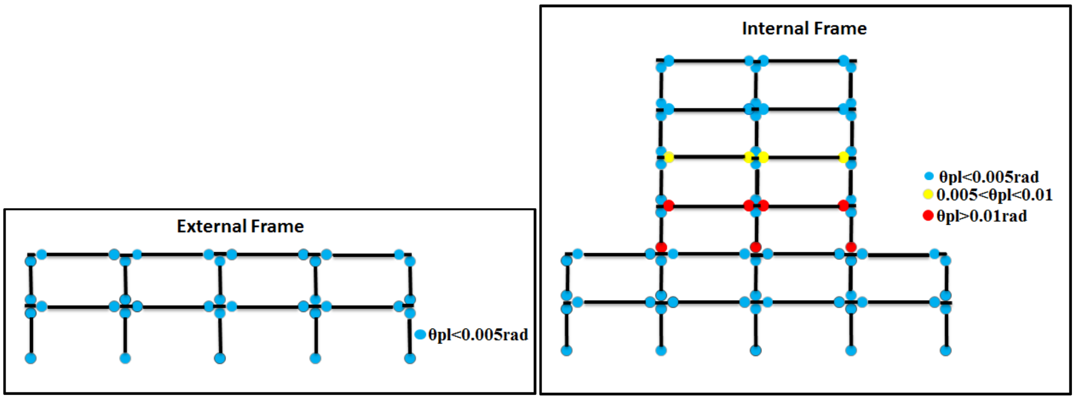


Figure 6: Distribution of Plastic Hinges in External and Internal Frames.

The inelastic demands in terms of plastic rotations for the interior frame column at the third floor are examined further in Fig.7, for SPO and the two dynamic analyses PGA1 and PGA2 and for both ductility classes DCM and DCH; it is seen that the plastic rotation demands of the base (end i, bottom) of the subject column do not exceed the limits of EC8 for the Collapse Prevention Performance level in all load cases, static or dynamic, whereby the maxima refer to the SPO demand at the target point, estimated according to EC8, and to the maximum absolute values, for the time history analyses.

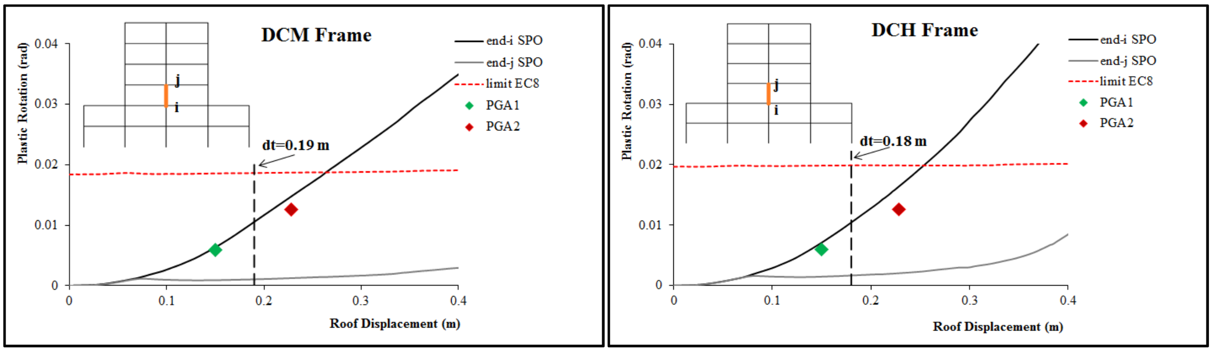


Figure 7: Plastic Rotation – Roof Displacement demands of the third floor column, for DCM and DCH Frames.

The interstory drift profiles of the examined frames for all executed analyses are compared in Fig.8. In all cases, relatively high demands in interstory drift are consistently developed in the third and fourth floor; generally, unlike the DCH frame, for the DCM frame larger interstory drifts are demanded by the SPO analysis, compared to the dynamic analysis under the 100% PGA earthquake.

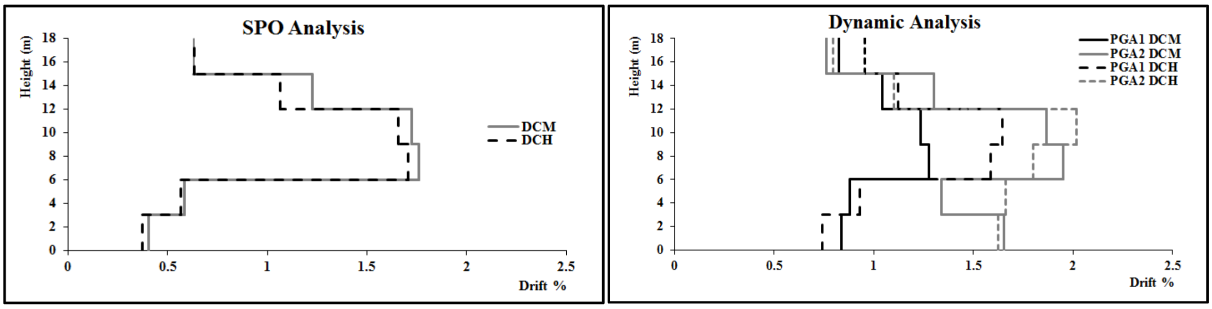


Figure 8: Comparative Profile Drifts DCM – DCH Frames for SPO and Dynamic Analysis.

On the other hand, the response of both frames under dynamic analysis depends strongly on the intensity of imposed excitation, with the interstory drifts reaching values as high as 2% at the tower base and comparable amplitudes at the base of the building, under the extreme event PGA2 (Fig.8).

The corresponding target point demands under SPO analysis are depicted below in Fig.9 for the two frames in normalized base shear - roof displacement characteristics. In both 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.

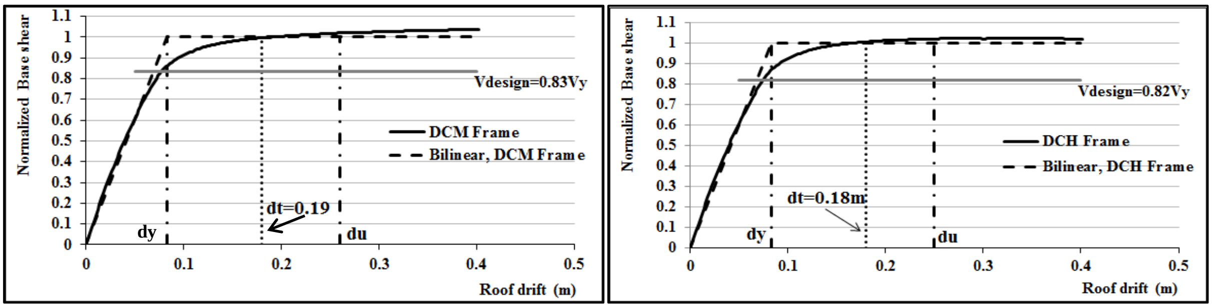


Figure 9: Normalized Base Shear-Roof displacement characteristics of the two frames.

The calculation of the q factor supplied by the structure depends on the type of structural system and on its regularity in elevation, which define both contributing factors, namely the ductility supplied (q_u) as well as the structural overstrength (q_Ω). From the results of the SPO analysis, taking into account the ultimate deformation supply for each frame for the Collapse Prevention Performance level (at column failure), the supplied q_u of the DCM frame is calculated to be 3.0 and of the DCH frame is calculated to be 3.1. Based on the overstrength factors q_Ω from the SPO analysis (Fig.9), it can be concluded that the supplied q exceeds the design value in both frames.

6 CONCLUSIONS

- A computerized procedure is developed on the OpenSees platform, aiming to produce a compliant design and to establish the inelastic seismic performance of a given vertically irregular three-dimensional RC building, according to the limitations of EC2 and EC8.
- The efficacy of the method is demonstrated using two typical RC frames having a central tower configuration (vertical irregularity) at the bottom third of the building, which are designed for Class Medium and High respectively in a zone of medium seismicity.
- For this class of buildings considered, it is shown that in both cases, the base of the tower develops high demands in plastic rotation and drift, concentrating a flexibility in this floor, thereby making it sensitive to the localization of damage.
- Regarding the supplied behavior factor of the structures, q, it is concluded that it is greater than the design q, accounting for irregularity, for both cases considered.
- It is observed that the two frames do not differ in regards to their inelastic response prediction, either using static pushover or dynamic analysis, since they developed similar inelastic characteristics, both in local and global level.

REFERENCES

[1] G.R. Aranda, Ductility Demands for R/C Frames Irregular in Elevation, *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, U.S.A., Vol. 4: 559-566, 1984.

[2] B.M. Shahrooz, J.P. Moehle, Seismic Response and Design of Setback Buildings, *Journal of Structural Engineering*, ASCE, **116**(5), 1423-1439, 1990.

[3] S. Das, J.M. Nau, Seismic Design Aspects of Vertically Irregular Reinforced Concrete Buildings, *Earthquake Spectra*, **19**(3), pp. 455-477, 2003.

[4] S.E. Ruiz, R. Diederich, The Mexico Earthquake of September 19, 1985 – The Seismic Performance of Buildings with Weak First Storey, *Earthquake Spectra*, **5**(1), pp. 89-102, 1989.

[5] A.A. Nassar, H. Krawinkler, Seismic Demands for SDOF and MDOF Systems, Report No. 95, *The John A. Blume Earthquake Engineering Center*, Department of Civil and Environmental Engineering, Stanford University, Stanford, U.S.A., 1991.

[6] E.V. Valmundsson, J.M. Nau, Seismic Response of Building Frames with Vertical Structural Irregularities, *Journal of Structural Engineering*, ASCE, **123**(1), pp. 30-41, 1997.

[7] A.A.K. Al-Ali, H. Krawinkler, Effects of Vertical Irregularities on Seismic Behavior of Building Structures, Report No. 130, *The John A. Blume Earthquake Engineering Center*, Department of Civil and Environmental Engineering, Stanford University, Stanford, U.S.A.,1998.

[8] C.A. Zeris, T.P. Tassios, Y. Lu, G.F. Zhang, Influence of Irregularity on the q Factor of RC Frames, Proceedings, *Tenth World Conference of Earthquake Engineering*, 1992.

[9] C. Repapis, C. Zeris, E.Vintzileou, Evaluation of the Seismic Performance of Existing RC Buildings II: A Case Study for Regular and Irregular Buildings, *Journal of Earthquake Engineering*, **10**(3), 429-452, 2006.

[10] T. Chunyu, L. Junjin, Z. Hong, C. Jinzhe, Experimental study on seismic behavior of an irregular high-rise building, *15th World Conference on Earthquake Engineering*, Lisbon, Portugal, September 24-28, 2012.

[11] P. Sarkar, A. Meher Prasad, D. Menonb, Vertical geometric irregularity in stepped building frames, *Engineering Structures*, **32**, 2175-2182, 2010.

[12] C.J. Athanassiadou, Seismic performance of R/C plane frames irregular in elevation, *Engineering Structures*, **30**, 1250–1261, 2008.

[13] M.M. Elassaly, Seismic Hazard Evaluation of Regular and Vertical Irregular R.C. Buildings in EGYPT, for Different Ground Motion Characteristics, *15th World Conference on Earthquake Engineering*, Lisbon, Portugal, September 24-28, 2012.

[14] BIS, IS 1893(Part 1) – 2002: *Indian Standard Criteria for Earthquake Resistant Design of Structures. Part 1 – General Provisions and Buildings (Fifth Revision)*, Bureau of Indian Standards, New Delhi, 2002.

[15] BSSC, *The 2003 NEHRP Recommended Provisions for New Buildings and Other Structures, Part 2: Commentary (FEMA 450)*, Building Seismic Safety Council, National Institute of Building Sciences, Washington, DC, U.S.A., 2003.

[16] EC8, European Committee for Standardization, Eurocode 8: *Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings,* 2004.

[17] EC2, European Committee for Standardization, Eurocode 2: *Design of concrete structures - Part 1: General rules and rules for buildings*, 2004.

[18] F. McKenna, G. L. Fenves, B. Jeremic, M. H. Scott, *OpenSees Command Language Manual*, 2007. *http://opensees.berkeley.edu*.

[19] MATLAB Primer, R2015a, the Mathworks Inc, Natick, MA, 2015.