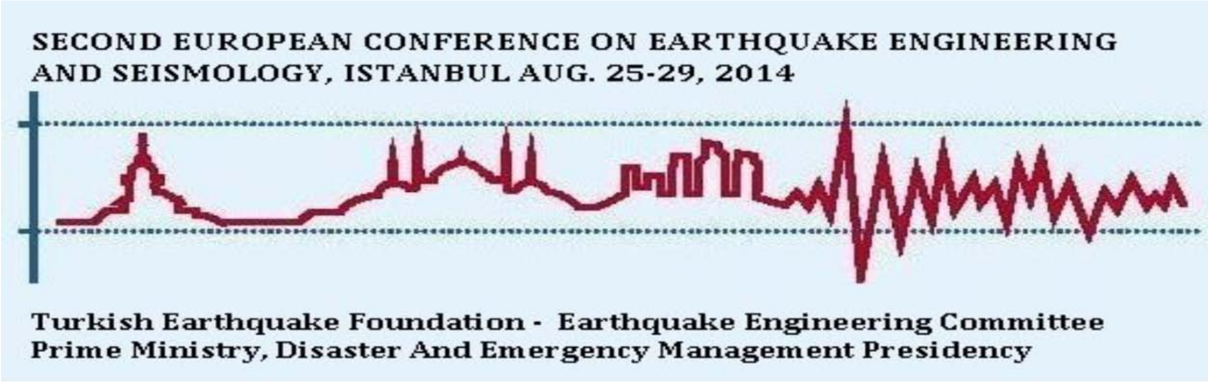


STRUCTURAL ASSESSMENT OF A 3-STORY R/C RESIDENTIAL BUILDING FOR SEISMIC LOADS

Athanasios PAPADOPOULOS [1], George D. MANOLIS [2], Asimina ATHANATOPOULOU-KYRIAKOU [3]

ABSTRACT

In this paper, the seismic assessment of an existing 3-story irregular R/C building, designed for low seismicity according to early Greek national seismic codes, is carried out. To this end, linear and static non-linear analyses were performed using the finite element method program SAP 2000 to investigate the building's capacity to withstand seismic loads and to predict structural inadequacies based on displacements corresponding to estimated performance points. A macro-model formulated on lumped plasticity was adopted with plastic hinge rotation capacities conforming to the provisions of EC8, Part 3 (CEN 2004). The extended N2 method was used to account for torsional effects. Also, models taking into account the effect of masonry infill walls were developed and the results were compared against those produced in the absence of masonry infills. In sum, the aim here is to investigate the seismic capacity of a representative building constructed between 1960 and 1985 that no longer conforms to modern seismic codes. Various strengthening schemes and design options based on the present analysis can then be decided upon, all of which focus on identifying weak structural elements in the original building.

INTRODUCTION

Around the year 1954 the first Greek national code (RD54) with provisions for earthquake loading was introduced. This code, based on the allowable stress design method, did not provide for issues such as detailing for earthquake resistance, local ductility of members, capacity design, etc., and was revised thirty years later in 1985 (MOD84) and finally in 2000 (EAK). As a result, the vast majority of the building stock in Greece is not in compliance with the latest developments in earthquake engineering and are likely to present severe structural inadequacies. Having been designed for low, if any, lateral force resistance and lacking the necessary detailing to ensure ductility, they often prove to be incapable to carry the seismic loads suggested by current codes such as EC8.

As expected, the need for structural strengthening and repair of existing buildings has quickly emerged over the last few years and several new codes have been published to this end. The evaluation of seismic performance is the first step before any potential retrofitting strategy is mapped, in order to assess the building's capacity and check whether or not it satisfies all necessary criteria to achieve the desired performance level. Part 3 of EC8, in agreement with most modern national codes, has adopted a "performance-based" approach, which allows for the definition of different acceptance criteria on seismic assessment. More specifically, it introduces three performance levels, referred to as Limit States:

(1) Limit State of Damage Limitation (LS of DL)

[1] Graduate Student, National Technical University, Athens, Greece, th.papadopoulos@outlook.com
[2] Professor, Aristotle University, Thessaloniki, Greece, gdm@civil.auth.gr
[3] Professor, Aristotle University, Thessaloniki, Greece, minak@civil.auth.gr

(2) Limit State of Significant Damage (LS of SD)
(3) Limit State of Near Collapse (LS of NC).

In the present work, the capacity assessment of an existing R/C building has been carried out, in an attempt to point out the structural inadequacies of a building not designed according to modern code provisions and study the effects of what is now an inadequate structural design. This is a first step in an effort to map economically viable retrofit strategies.

BUILDING DESCRIPTION

The 3-story building was constructed in 1982 using the first ever Greek seismic design code, the Royal Decree of 1959 (RD59), and has since been used for residential purpose within the framework of a social housing program. The floor plan is shown in Figure 1, where three beams run throughout the longitudinal direction, forming longitudinal frames. In the transverse direction, only two external and two internal (one of which is discontinuous) cross-beams are present and no other links are provided between the longitudinal frames. A deep beam supports the stairwell by forming two short columns. Column sections decrease with height, while brick infill walls are present in all floors except for the ground floor (i.e., a 'pilotis' type building with a soft first story).

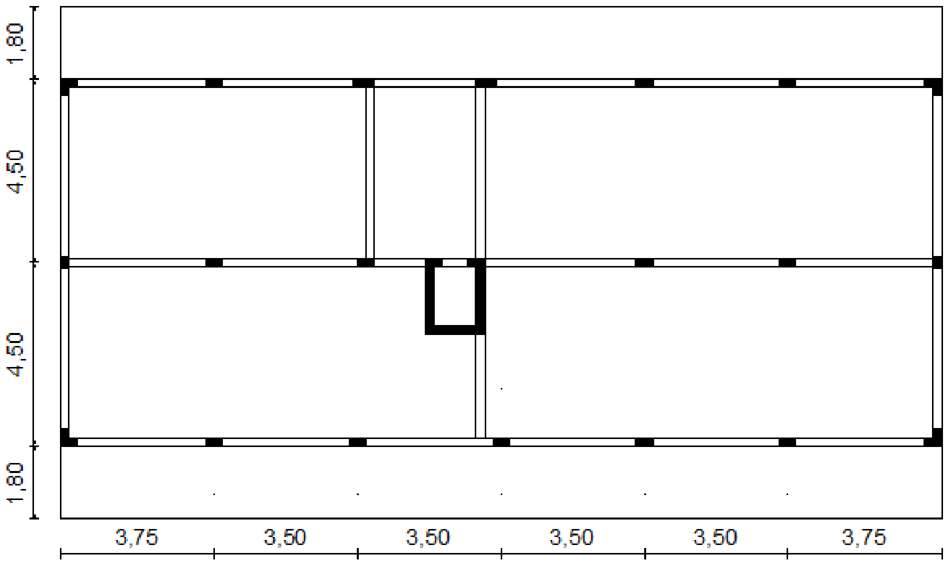


Figure 1. Typical floor plan for the 1982 three-story R/C building

MODELING

Materials, section geometry and reinforcement were all defined according to the original design drawings. Concrete grade B225 (equivalent to C16/20) and steel grade StIII ($f_{yk}=420\ MPa$) were specified with elasticity modules taken as *27.5* and *200 GPa*, respectively. The mean value properties of all building materials were assumed and then used in the analysis. A confidence factor value *CF=1.2* was also assumed referring to a normal knowledge level.

Structural members were modelled using typical beam-column frame elements with rigid ends to account for the beam-to-column joint zone. A diaphragm constraint has been assigned at each floor assuming that slabs are sufficiently stiff. The core wall was modelled with equivalent frame elements and rigid links were used to connect its constituent parts. The stairwell was neglected for simplicity. Masonry infill panels were modelled with equivalent diagonal struts, whose area was determined by multiplying the panel thickness by an equivalent width. Their strength was subsequently reduced to account for out-of-plane failure, while infills with openings greater than 50% of the overall panel dimensions were neglected (OASP, 2012). Infills with openings lesser than 50% of the overall panel dimensions were modelled by applying reduction factors to strut width (see Tsikas and Dritsos, 2009) instead of using the proposed multi - strut configuration. For linear analysis purposes, the elastic stiffness of all members was taken as *0.5I* (*I* being the moment of inertia of the cross section) assuming cracked sections. For the static non-linear analysis a more accurate (and more conservative from a deformation control viewpoint) value was needed, and therefore the secant stiffness to the yield-point given by $M_yL_v/3\theta_y$ was used (Fardis, 2009).

Moment hinges were assigned at the beam ends, PMM hinges with axial force-moment interaction were assigned at the column and wall ends, and finally axial hinges were used for the diagonal struts. The moment-rotation (M-θp) relations and the interaction curves for biaxial bending and axial force were determined for each member section. For this purpose, two cross-section analysis programs were used, namely RCCOLA-90 (Kappos, 2006) and XTRACT (Chadwell and Imbsen, 2004).

Next, the deformation capacity of beams, columns and walls were defined in terms of the chord rotation θ (i.e., the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span) was calculated according to EC8 - Part 3 that has adopted the Panagiotakos and Fardis (2001) proposals. Hence, ultimate and yielding chord rotations were derived from Eqs (1) and (2) below as

$$\theta_{um} = \frac{1}{\gamma_{el}} \cdot 0.016 \cdot (0.3^{v}) \cdot \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c \right]^{0.225} \cdot \left(\frac{L_v}{h} \right)^{0.35} \cdot 25^{\left(\alpha\rho_{sx} \frac{f_{yw}}{f_c} \right)} \cdot \left(1.25^{100 \cdot \rho_d} \right) \qquad (1)$$

$$\theta_y = \phi_y \cdot \frac{L_v + \alpha_v \cdot z}{3} + 0.0013 \cdot \left(1 + 1.5 \cdot \frac{h}{L_v} \right) + 0.13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \qquad (2)$$

Equation (1) was multiplied by 0.825 for members without detailing for earthquake resistance and divided by 1.60 in the case of wall sections. Chord rotations were evaluated as previously described, in reference to the EC8 - Part 3 Limit States:
(1) LS of DL: $\theta_{DL} = \theta_y$
(2) LS of SD: $\theta_{SD} = 3/4\ \theta_u$
(3) LS of NC: $\theta_{NC} = \theta_u$

As shown in Figure 2, five points labeled A, B, C, D, and E were used to define the force-displacement behavior of the hinge and three points labelled DL, SD, and NC are used to define the acceptance criteria for the hinge. The residual moment capacity was taken as 20% of the ultimate value.

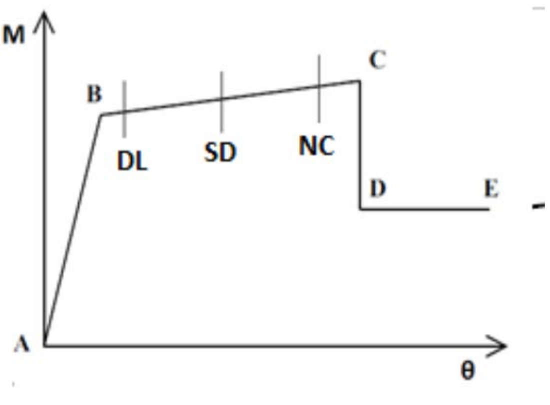


Figure 2. Moment - chord rotation curve for R/C sections

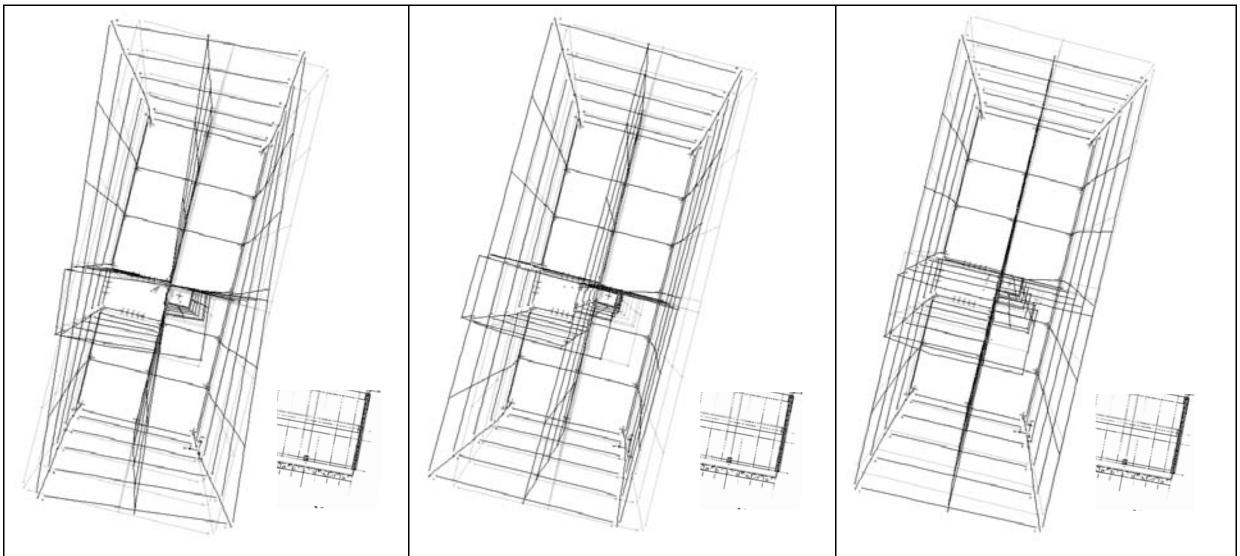
A total vertical load calculated from the G + 0.3·Q seismic load combination was imposed and distributed to the beams according to their tributary floor area. The equivalent total mass of each story was automatically calculated and distributed to the nodes for dynamic analysis.

DYNAMIC PROPERTIES

A modal analysis was first performed on the building under study. The values of the natural period of vibration *T* and the relevant percentage of the effective modal mass *M** in the *X, Y* and *Θ* (i.e., rotation around the vertical Z axis) principal directions of seismic motion are presented in Table 1 below. The fundamental mode of vibration is shown Figure 3, along with the next two modes. We observe that rotation dominates the fundamental mode shape and the corresponding natural period *T=1.107 sec* is determined using the secant to the yield-point stiffness. According to the requirements of EC8, the building is torsionally sensitive, since the torsional radius in the *y* direction, as defined according to the Greek National code Annex, is larger than the radius of gyration.

Table 1. R/C building natural periods and effective modal mass percentages for three modes

Mode	Period (Sec)	U_X (%)	U_Y (%)	R_Z (%)
1	1.107	0.003	0.086	0.426
2	0.908	0.002	0.631	0.221
3	0.720	0.715	0.001	0.067



1st Mode: Torsional	2nd Mode: Translational y	3rd Mode: Translational x

Figure 3. First three modes of vibration for the three-story R/C building

LINEAR ANALYSIS

Nowadays, seismic assessment and retrofit is mostly displacement-based, since most existing buildings fail to meet all the rules and criteria, which entitle a force reduction factor *q* higher than the value attributed to over-strength alone. This would result in extreme force demands and in all likelihood most existing buildings would be deemed structurally inadequate. Instead, each member should be checked individually in terms of deformation demands derived by employing the elastic response spectrum and assuming the equal displacement rule. Nevertheless, for this simplification to be made, uniformity of the chord rotation ductility ratio demands should be ensured according to EC8-Part 3. This is evaluated by verifying that the maximum demand-to-capacity ratio ($\rho_i = D_i/C_i$) in flexure does not exceed its minimum value over all primary ductile members that have $\rho_i > 1$ by more than a factor with recommended value of *2.5*.

Next, the $\rho_i = D_i/C_i$ ratio of the demand D_i obtained from the analysis for a seismic design combination with the EC8 elastic spectrum for soil type B and low seismicity (0.16g), to the corresponding capacity C_i for each element, was calculated. Almost all elements were found to exceed their yield moment. The ratio ρ_{max}/ρ_{min} exceeded by far the value of *2.5* set by EC8 and therefore elastic analysis was deemed unsuitable. The distribution of ρ_i ratios is presented in Figure 4 below.

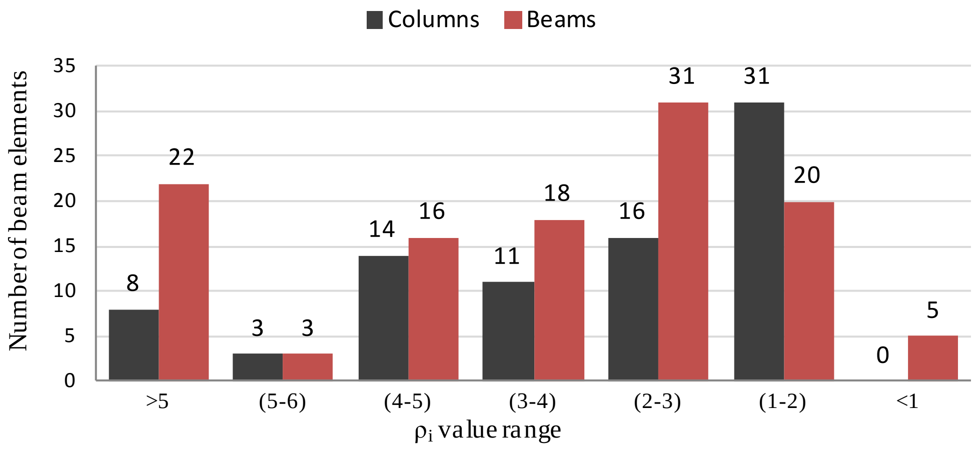


Figure 4. Assessment of beam and column member capacity ratios based on linear analysis

The ρ_i value for most members was found to fall in the 1- 4 range, while beams in the transverse direction reach even higher values, verifying the inadequacy of the original structural system to support lateral loads in the transverse direction.

STATIC NON-LINEAR ANALYSIS

The seismic behaviour of the R/C structure was then studied through a series of static non-linear pushover analyses. Capacity curves, representing the relation between the base shear force *(V)* and the roof displacement *(d)*, were determined under monotonically increasing lateral loads. A uniform and a modal load pattern were assumed for each orthogonal direction. The capacity curves obtained from models with and without masonry infills, and subjected to uniform and modal load patterns, are all presented in Figures 5 and 6.

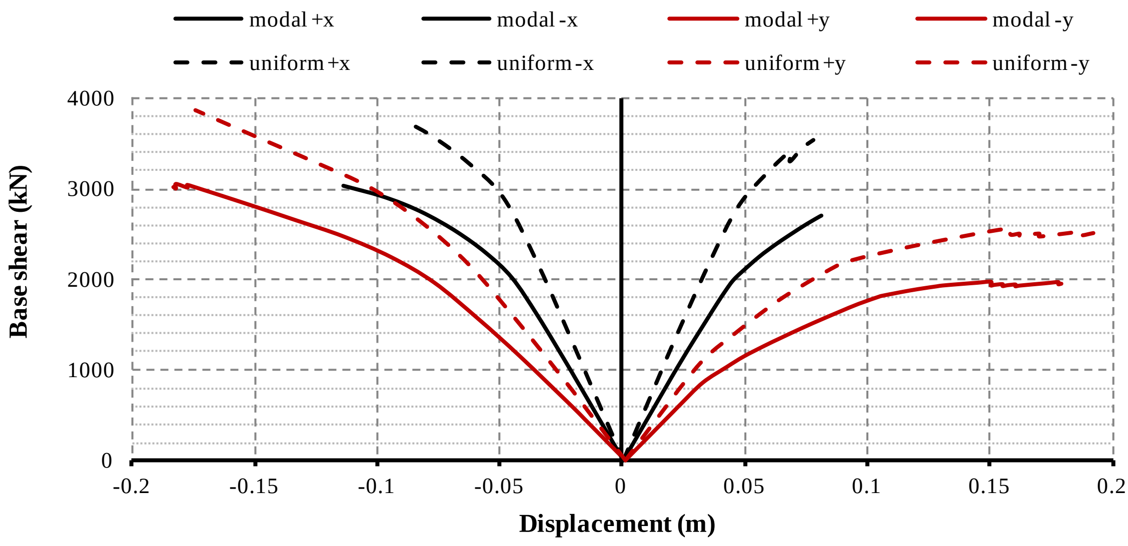


Figure 5. Capacity curves for modal and uniform load patterns for a numerical model without masonry infills

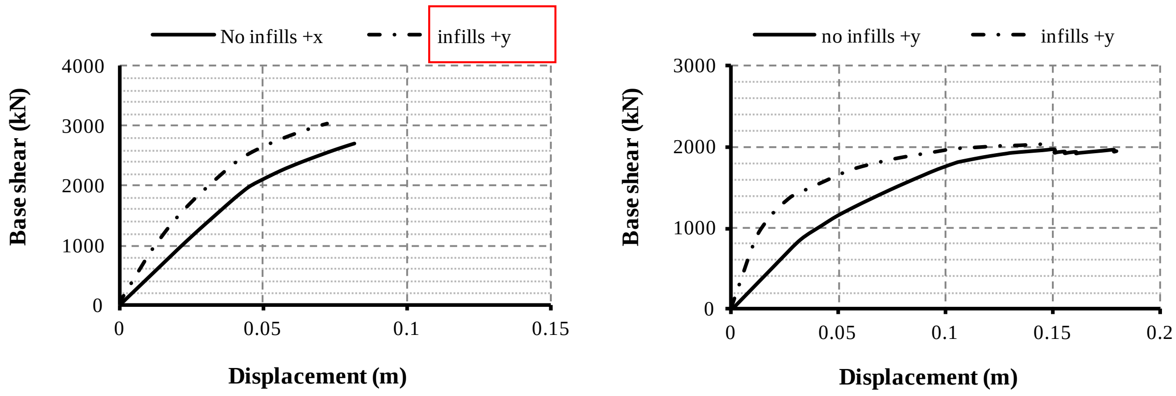


Figure 6. Capacity curves for numerical models with and without masonry infills in the *X*, *Y* directions

Performance points were calculated according to EC8 and the previously defined limit states of 'Damage Limitation', 'Significant Damage' and 'Near Collapse' were introduced to assess the state of structural elements, see Figures 7 and 8.

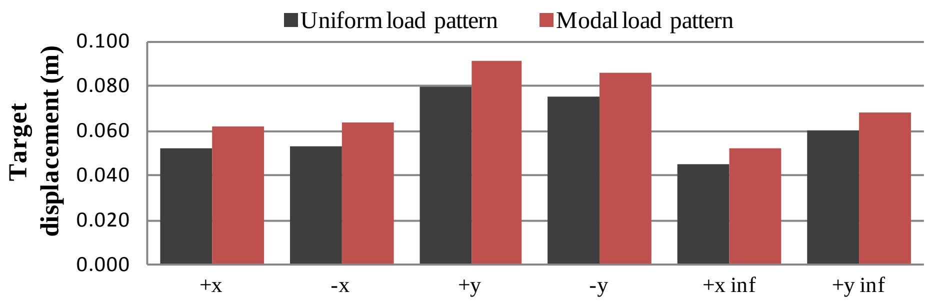


Figure 7. Target displacements for all directions of loading and for building models that account for infill walls

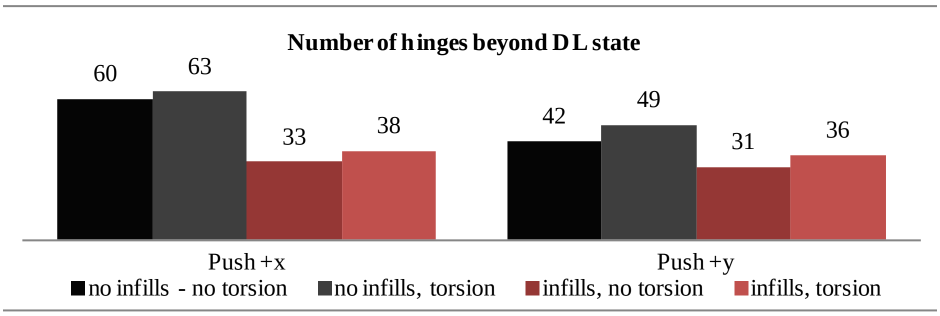


Figure 8. Number of plastic hinges in teh R/C building that exceed the 'Damage limitation' limit state

In terms of flexural strength, performance points were found for all directions of loading, despite the high number of structural elements that yielded. However, many elements, mostly beams, presented important deficiencies in terms of shear strength and thus a premature brittle failure of the structure should be considered. Most columns seem to behave better and are more likely to present ductile failure. However, special consideration has to be given to the short columns-deep beam configuration mentioned before. The *1.60 m* deep beam supports the stairwell at around mid-height and is connected with two columns of increased size and reinforcement, forming a frame of sizeable stiffness. Normally, shear demands are higher, while the stiff beam also leads to a faster change in the columns' axial load values due to lateral loads. In Figure 9, moment *M* and axial force *N* values for the column that is experiencing a reduction in the axial load value are plotted for the various steps of the pushover analysis. Also plotted are the shear strength (calculated according to EC2) multiplied by each step's shear span and taking into account the axial load drop. It is obvious that shear failure precedes flexural failure and this state is reached prior to the estimated performance point.

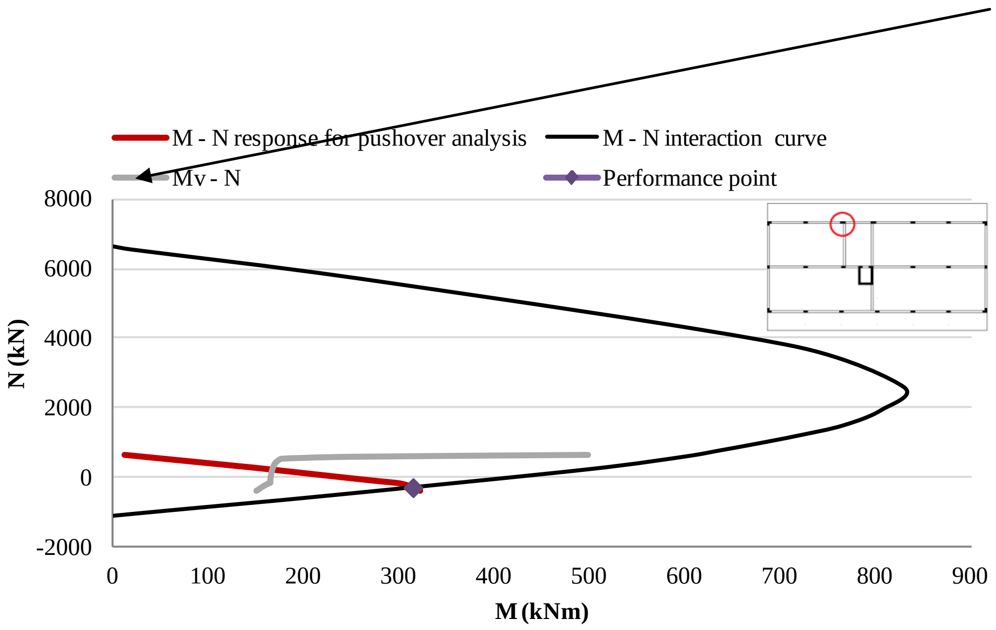


Figure 9. Bending and shear resistance of a typical column in the three-stoory R/C building

TORSIONAL EFFECTS

Torsional effects were taken into account by applying correction factors, defined as the ratio between the normalised roof displacements obtained by elastic modal analysis and that obtained by pushover analysis according to the extended N2 method that was proposed by Fajfar et al. (2005). Follow-up studies conducted by several researchers (Pinho et al, 2008; Bhatt and Bento, 2011) have helped validate this quasi-static, nonlinear approximate analysis. Next, normalised roof displacements from elastic modal analysis that were smaller than *1.0* were set equal to *1.0*. All relevant quantities obtained by pushover analysis were then multiplied with the appropriate correction factors.

As expected, pushover analysis is not able to reproduce the torsional response of the building. This is true especially in the *y*-direction, where the predominantly translational mode is being coupled with torsional motion and produces a torsional amplification effect at the stiff side because of the additive contribution of the two motions on that side. Lateral displacements and inter-story drifts are also presented in Figures 10 and 11 for a transverse and a longitudinal frame, respectively.

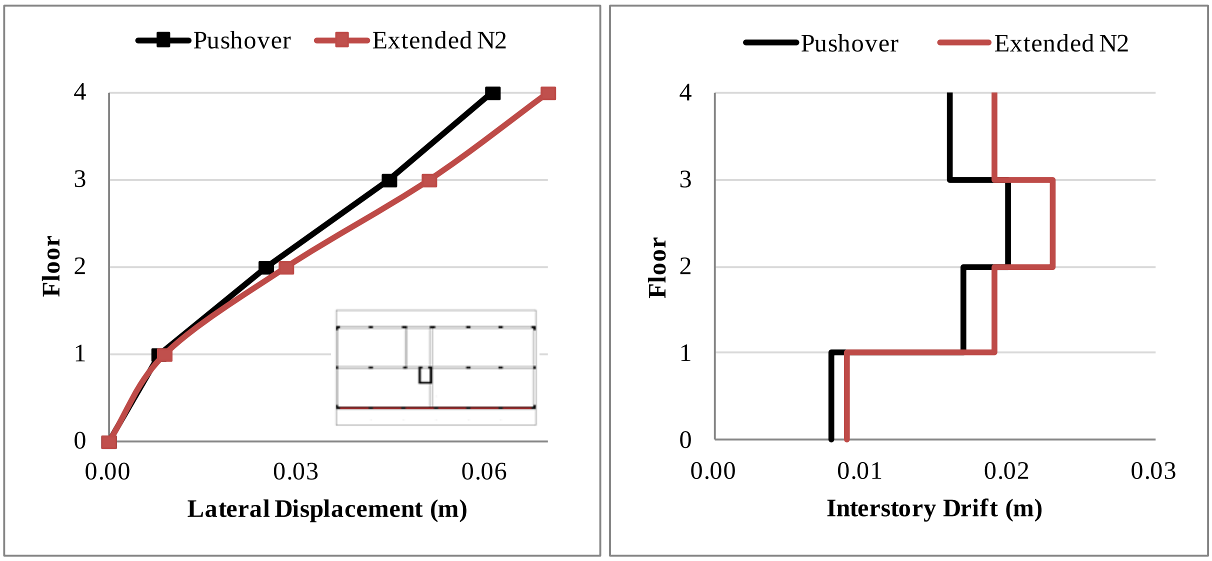


Figure 10. Lateral displacements and interstory drifts for an *x*-direction frame of the R/C building

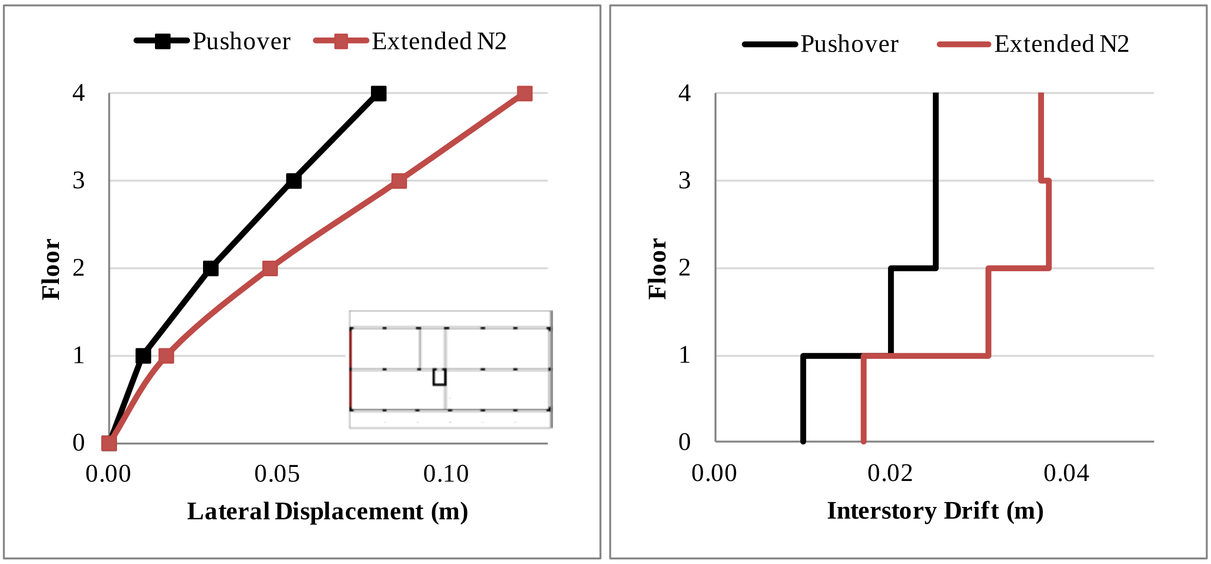


Figure 11. Lateral displacements and interstory drifts for a *y*-direction frame of the R/C building

STRUCTURAL ASSESSMENT

The results of the above structural analysis effort can now be summarized in Figure 12 below, where all members of the first floor are shown and the ones presenting with the most important strength deficiencies are identified in color.

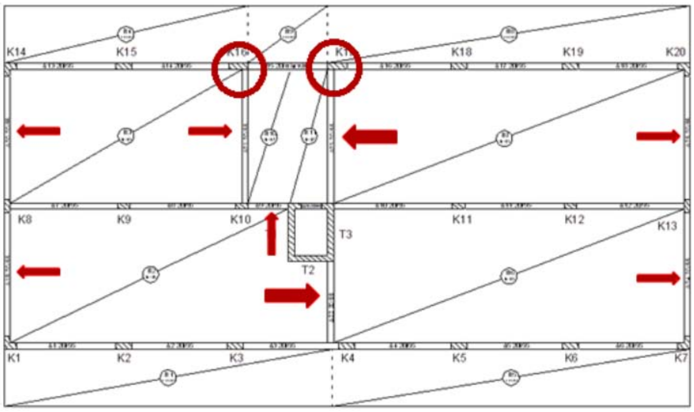


Figure 12. Structural members in teh first floor lay-out of the R/C building most likely to fail under seismic loads

CONCLUSIONS

To summarize the above structural evaluation effort, linear dynamic analysis proved inadequate for the three-story R/C building under investigation because of non-uniformity in the elements' chord rotation ductility. For this end, a quasi-static pushover analysis was employed as a simple way to study the anticipated non-linear response of the R/C building under seismically-induced lateral loads. The results obtained suggest that when solely flexural failure is considered, a large number of elements yield, but none exceeded the 'Significant Damage' limit state. Next, torsional amplification effects due to the buildings asymmetry in the stiffness distribution causes more structural elements to yield and needs to be considered. The presence of masonry infill panels decreased target displacements and was generally beneficial to the structure. The soft story (or 'pilotis') phenomenon was not observed in the ground floor, mainly because of the column sections there had a larger cross-section and additional steel reinforcement. As previously stated, the structural system was found inadequate to successfully support lateral loads in the transverse direction, while the short columns-deep beam configuration in the longitudinal direction is deemed problematic. Finally, the presence of a single, centrally located core wall leads to considerable torsional effects that tend to aggravate the behavior of the structure, causing early yielding in the perimeter beams, as well as in the beams connected to the walls.

Nevertheless, the engineering approximations and uncertainties involved in setting up the numerical FEM models for the R/C building under study should be kept in mind. All information was taken directly from design drawings that might not depict the actual constructed building. Sufficient anchorage was assumed for reinforcements and possible flexibility of joints was ignored. In addition, the approximate nature of pushover analysis, especially for structures that present significant torsional effects, casts some doubts on the validity of the numerical results, despite the use of all appropriate correction factors. The modeling of the core wall with equivalent frame elements and plastic hinges is another approximation that might introduce some deviation from what is the actual response. Finally, the modeling of the masonry infill walls also involved uncertainties, while soil-structure-interaction was neglected because of the good quality soil deposits and the presence of a raft foundation.

Based on all the above structural analysis results, produced by exercising careful engineering judgment, it is concluded that the building in its present state is structurally deficient and incapable to safely withstand seismic load demands, as currently prescribed by EC8. The originally poorly conceived design results in a structural system outlay that is further aggravated by the consequences of outdated seismic design requirements by today's standards, pointing out to the need for strengthening measures.

REFERENCES

Bhatt C, Bento R (2011), "Assessing the seismic response of existing RC buildings using the extended N2 method", *Bulletin of Earthquake Engineering*, Vol. 9, Issue 4, pp 1183 - 1201

CEN (2004) Eurocode 2: Design of concrete structures – Part 1-1: General – Common rules for building and civil engineering structures, European Committee for Standardization, Brussels, Belgium

CEN (2005) Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules seismic actions and rules for buildings, European Committee for Standardization, Brussels, Belgium

CEN (2006) Eurocode 8: Design of structures for earthquake resistance – Part 3: Strengthening and repair of buildings, European Committee for Standardization, Brussels, Belgium

Chadwell C and Imbsen R (2004) "XTRACT: A Tool for Axial Force - Ultimate Curvature Interactions", Structures 2004: pp. 1-9.doi: 10.1061/40700(2004)178

Fajfar P, Marusic D and Perus I (2005) "Torsional Effects in the Pushover-Based Seismic Analysis of Buildings", *Journal of Earthquake Engineering,* Vol. 9, No. 6, 831–854

Fardis MN (2009) Seismic Design, Assessment and Retrofitting of Concrete Buildings, Springer, New York

Kappos, AJ (1996) "RCCOLA-90 Program For The Inelastic Analysis Of Reinforced Concrete Cross-Sections User’s Manual", Department of Civil Engineering, Imperial College, London, UK

OASP (2012) Greek Organization for Seismic Planning and Protection: "National Greek Retrofitting Code", Ministry for Environmental Planning and Public Works, Athens, Greece (in Greek)

Panagiotakos TB, Fardis MN (2001), "Deformations of RC Members at Yielding and Ultimate", *ACI Structural Journal*, Vol. 98, No. 2, pp. 135-148

Pinho R, Bento R, Bhatt C (2008), “Assessing the 3D Irregular Spear Building with Nonlinear Static Procedures”, Proceedings of the 14th Word Conference on Earthquake Engineering, Beijing, China, paper 05-01-0158

SAP (2000) Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures, Computers and Structures Inc., Berkeley, California, USA

Tsikas P, Dritsos S (2008), “Investigation of the Influence of Wall Partitioned Openings in Reinforced Concrete Frame Structures”, 3rd Greek Conference on Earthquake Engineering and Engineering Seismology, paper No. 2004 from the Conference CD Proceedings, Athens, Greece (in Greek)