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INELASTIC MODELING SENSITIVITY OF THE PREDICTED SEISMIC PERFORMANCE OF AN EXISTING RC BUILDING

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SUMMARY

Inelastic modeling of entire reinforced concrete (RC) buildings under seismic excitation is a complex problem that influences directly the predicted seismic performance. Modeling assumptions and conventions adopted become more important in existing RC frame response predictions, due to these structures' structural characteristics and non conforming detailing. The problem is investigated for a typical existing five-story RC frame which has been designed for moderate seismicity according to the past generation of Greek seismic codes. Different plane frame finite element models are formulated adopting state of the art as well as state of the practice analysis codes and finite element formulations. The seismic performance of each model is estimated, following both a conventional static pushover as well as nonlinear time-history analyses under different levels of seismic intensity. The models range from the simple yet widely adopted in practice concentrated plasticity elements with axial-flexural strength interaction only, to the more complex distributed damage stiffness or flexibility-based fiber elements accounting or not for joint deformations. The results of the analyses are compared at the global and primarily the local damage prediction levels, to reveal substantial discrepancies and scatter in key performance Response Indices introduced in a Performance Based (re)Design approach by the model limitations, which are often ignored. It is concluded that, in addition to standardization of the criteria and procedures of evaluation, the analytical model for evaluating these Response Indices should also be well defined to avoid error and conflict.

1. INTRODUCTION AND STATEMENT OF THE PROBLEM

Currently enforced Performance Based Design (PBD) Methods, based on an inelastic analysis prediction of the structural response under static load profiles resembling the primary modal response profile of the building are gaining wide acceptance for seismic evaluation and retrofit / rehabilitation design of existing reinforced concrete (RC) buildings. Such methods are typically based on establishing the common intersection of a ductility dependent inelastic response spectrum and the capacity curve [ATC-40, 1996, Fajfar, 1999 and FEMA 1997, 2005]. The former is a single degree of freedom estimate of the maximum global response of the system, often denoted 'the inelastic demand'; several sources of uncertainty are introduced in the estimation of this demand, such as the variability of the ground motion characteristics (wave content, duration, rate of energy release and amplitude) and the manner in which the spectral inelastic response is related to the elastic one as a function of limited resistance and system ductility [Miranda and Ruiz-Garcia, 2002]. The latter represents the structural 'supply' and is merely a representation in equal terms of the resistance – deformation characteristics of the system, typically being the estimated base shear under the imposed load pattern and the corresponding roof (or

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primary mode center of mass) lateral deformation. Depending on the methodology adopted, other Response Indices – such as the interstorey drift – have also been adopted, down to the local demand level, whereby the inelastic rotation at the critical regions is examined with imposed roof deformations.

The problem of reliable evaluation of local indices parameters for existing RC buildings is more critical than for new designs and renders itself to a wider dispersion in the predicted results, due to the way these structures are designed and detailed. For the present study, a typical existing RC building in Greece, designed and constructed during the 60s, is considered. Buildings belonging in this category are numerous in large urban centers within Greece and are located in all the different seismicity zones currently established in modern seismic design codes. They have been designed according to the past generations of codes [RD59, 1959], following allowable stress procedures and simplified structural analysis models. They usually have regular column spacing with relatively short spans, while their perimeter frames are infilled with unreinforced masonry walls, having vertically continuous openings. Although vertically irregular forms of such buildings are often encountered in practice – and have therefore been studied extensively [Repapis, Vintzeleou and Zeris, 2006] – a regular structure is considered herein for simplicity. Typically, early 60s structures do not have shear walls, relying only on frame action; the structural members are narrow, structural materials exhibit wide scatter in their mechanical properties while structural elements and the building itself possess little or no critical region reinforcement or capacity design provisions. For this reason, they are generally considered as frames non-conforming to current standards.

The problem investigated herein is to evaluate the influence of using different finite element FE model formulations for the establishment of critical seismic performance indices, used in PBD of this type of existing RC buildings. Response indices of interest, in a broad sense, include the type of collapse mechanism prediction, evaluation of the capacity curve and the distribution and magnitude of interstorey drift profiles and – most importantly – prediction of the local plastic rotation demand history, with imposed roof deformation. These are evaluated herein, following alternative FE model formulations, using both static pushover (SPO) as well as incremental dynamic scaled cloud [Jalayer, 2003] and IDA analyses [Vamvatsikos and Cornell, 2002, 2004].

2. DESCRIPTION OF THE EXAMPLE BUILDING

The building analyzed as a case study is a typical existing RC frame structure located in seismicity zone II within the three zone system adopted by the 1959 Greek Code [RD59, 1959], according to which the building is designed. The structure is a regular in plan and elevation RC frame, three (transverse) by four (longitudinal) bays in plan and five stories high. The typical storey height is uniform throughout and equal to 3.00m, while bay sizes are 3.50m in each direction and is characteristic of this generation of building forms that are characterized by dense column spacing. Columns of the structure are square in section, varying between 35x35cm at the first and second floors, reduced to 30x30cm at the third floor. The corner columns remain 30x30cm in the stories above, while the interior frame columns are further reduced to 25x25cm from this point up. The beams are kept to dimensions 20/50cm in all cases. Concrete slabs are 12cm thick in all the floors acting in two way action.

The frame is designed following the allowable stress method, using concrete Grade B160 conforming to current Grade C12 (namely 12MPa characteristic cylinder compressive strength) and smooth StI reinforcement, conforming to current Grade S220 (namely 220 MPa nominal yield tensile strength). Since, however, the as built condition of the structure is evaluated herein, average material characteristics are used in the analysis model formulation. It is therefore assumed that the average compressive strength of the aged concrete is 18 MPa, while the tensile yield strength of the steel is 310 MPa, as determined from tensile tests of such type of smooth reinforcement. The structure is located in zone II seismicity and is assumed to be built on rock. Hence, the service level seismic design base shear coefficient is equal to 6% of its inertia weight, which is equal to the full unfactored acting dead plus live load of the structure acting in a uniform distribution with height. Following an allowance by RD59 [1959], member capacity checks under the seismic load combination are estimated allowing for a 20% increase in the reinforcement allowable tensile stress of 120 MPa.

The design loads adopted are: i) the structure self-weight, which includes the perimeter double leaf clay masonry infill walls of 25cm thickness, ii) a uniform surcharge equal to 2.50 kN/m^2, which (following an allowance of the loadings code) includes 1.00 kN/m^2 to account for the interior 10cm thick moveable clay masonry partitions and iii) a live load of 2.00kN/m^2 (residential use). According to the currently enforced seismic design code in Greece [EAK, 2000], masonry infills are only introduced (directly for the perimeter walls and indirectly for the interior ones) as a mass only, contributing to the inertia load and natural frequency of the building but not its lateral stiffness. Detailing practices adopted at the time of construction involve the use of bent up bars in beams; furthermore, shear reinforcement is only used to resist service level forces, therefore, no capacity design procedures are adopted either for shear or for joint flexural capacity design. Similarly, no critical region confinement is specified.

3. STRUCTURAL MODEL FORMULATION

Following the limitation of one of the programs used herein for comparison purposes and making use of the plan regularity, the example structure is modeled and analyzed as a two-dimensional plane frame model along the longitudinal direction of the example structure (Fig.1). For this purpose, diaphragms are assumed to be rigid in each floor and therefore the lateral degrees of freedom are in most cases (unless otherwise specified) slaved to a single degree of freedom per floor, at the centre of plan. Following EAK [2000] requirements, the tributary vertical loads and corresponding inertia mass under seismic excitation load cases are equal to the dead loads plus 30% of the live load. These are distributed at the longitudinal beams and as joint reactions from the out-of-plane frames, according to the connecting diaphragm tributary areas. All masses are assumed to be lumped at the nodes. For the time history integration analyses reported, the damping coefficient is chosen as mass proportional, such that damping in the first fundamental mode of the cracked structure is equal to 5% critical. In all analyses reported herein, second order effects are included following the corresponding modeling conventions of each analysis program.

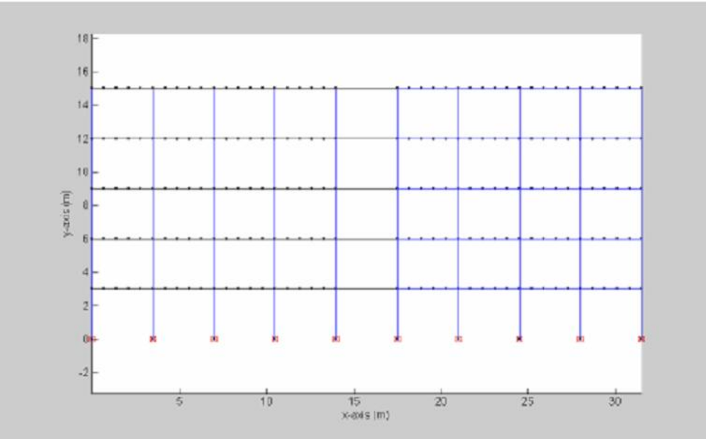


Fig.1 Model representation of the example building.

3.1 Modeling Conventions

For purpose of model comparisons, the inelastic analyses presented in this study are performed using two widely adopted computer programs, namely Drain-2DX [Allahabadi and Powell, 1988] and OpenSEES [McKenna, Fenves, Jeremic and Scott, 2000]. In each case, the different FE modeling capabilities available in each program are used. Both of the above FE codes are widely established and used for routine nonlinear static and dynamic analyses of buildings in PBD as well as for research purposes, for the estimation of system response under different model formulations.

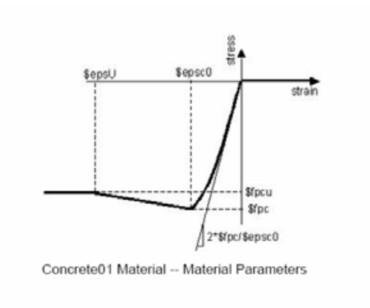
Due to model limitations in D2DX, and in order to approximate accurately changes in reinforcement along each beam and the internal forces under distributed loads, beams are modeled using five elements in each case, equally spaced at 0.7m apart. With the exception of model Disp_10 below, columns are modeled using one element per member only. Overall, the following FE models have been considered herein:

- Lump1: Lumped plasticity elements with axial load – bending moment (N-M) interaction for columns and Moment dependence for beams, using equivalent bilinear characteristics established from section analysis; hardening again set to 5% initial stiffness. Concrete modulus is set to 28 GPa [Drain2-DX].
- Lump2: As above, but set hardening stiffness to 0.5% and reduce the material elastic modulus to a secant value 75% of the above (21 GPa) [Drain-2DX].

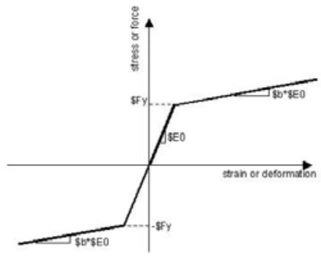
- Disp_1: Distributed plasticity elements monitored along different integration points, using a stiffness approximation formulation for the beams and columns, one element for each column; section model follows a distributed fiber based section formulation [OpenSees].
- Disp_10: Distributed plasticity elements monitored in different integration sections, using a stiffness approximation for the beams and columns, but allow for ten elements per each column; again, section model follows a distributed fiber based section formulation [OpenSees].
- Force: Distributed plasticity elements monitored in different sections, using a flexibility based formulation for beams and columns, one element for each column; section model follows a distributed fiber based section formulation [OpenSees].
- Force_with_rigid_joints: As previously, but allowing for the joint rigidity to be modeled at the beam-column intersections using stiff elastic members and additional joint nodes, as commonly assumed in analysis models of framed structures.
- Constant_hinge: Column elements follow a discrete hinge beam-column element formulation with a constant plastic hinge length following a distributed fiber based section formulation [OpenSees] and a constant plastic hinge length equal to the member depth.

3.2 Section Models

For the Llumped plasticity models using a bilinear lumped spring representation, concrete material modulei of elasticity of 28 and 21 GPa are used; in both cases, following also recommendations of current seismic design codes [EAK, 2000], a reduced section moment of inertia is used for beams, equal to half the uncracked section stiffness. Two extreme cases of post yield hardening stiffness are specified, namely 5.0% and 0.5% of the initial (Lump1 and Lump2 respectively). Section characteristics are evaluated from fiber section analysis, using a trilinear steel and a nonlinear softening concrete stress-strain characteristic accounting for any confinement of the core section, according to a material model proposed by Mander, Prestley and Park [1988] (Fig. 2).



(a)



(b)

Fig.2. Stress – Strain diagram for (a) unconfined and confined concrete and (b) for reinforcing steel.

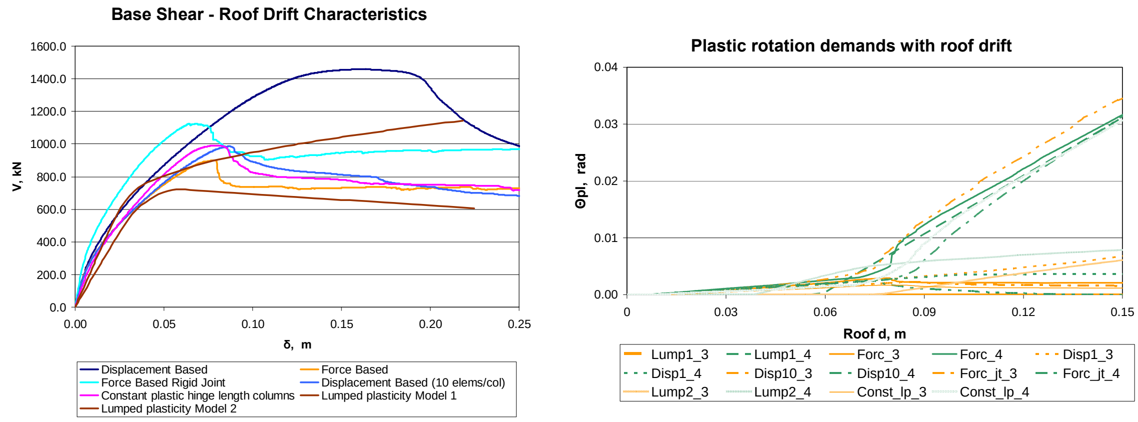
The distributed damage fiber section models Disp, Force and Constant_hinge, belong to the category of elements of distributed damage, monitored in critical sections which are discretized into an assemblage of steel and concrete fibers under the assumption that plane sections remaining plane [Spacone, Filippou and Taucer, 1996, Neuenhofer and Filippou, 1997]. The fiber constitutive relations are based on average material properties and the same confinement model representation as for the lumped section evaluation above. Typically, these elements evaluate internally the inelastic moment-curvature characteristics of all defined control sections at the ends and at the interior Gauss points (Force, Disp) or at the end only critical regions (Constant_Hinge) of the members.

Five sections are specified in the columns and three for each beam sub element (or fifteen per beam). In all cases, the analysis takes into account through equilibrium the effect of axial load variation in the columns as well as flexure related phenomena of concrete cracking and yielding of the steel. Depending on the formulation adopted, the internal stiffness or flexibility is interpolated and piecewise internal compatibility or equilibrium is preserved along the monitored sections, with the rest of the element satisfying completeness in an average sense only. The problems of equilibrium violation in Disp as opposed to the Force formulations have been investigated and demonstrated by Zeris and Mahin [1988, 1991], yet their influence to the prediction of plastic rotations in routine PBD analysis is investigated herein.

4. STATIC PUSHOVER ANALYSES OF THE EXAMPLE FRAME

4.1 Static Pushover Analysis

All different models of the example frame are analyzed under a nonlinear Static Pushover (SPO) analysis, using a triangular profile of lateral deformation. Depending on the software used, either direct displacement control (Opensees) or a mixture of initial load control followed by displacement control (Drain-2DX) is used, for numerical convergence and accuracy. In all cases, the iteration convergence tolerance is higher than 10^{-4} and at least 50 iterations per step are used. Characteristic SPO inelastic predictions are expressed in Fig. 3, in terms of : i) the capacity curve, namely the base shear – imposed roof deformation profile of the models and ii) the variation of local plastic rotation in the columns (denoted θ_p) with lateral roof deformation, up to an imposed maximum roof deformation of 15cm (namely 1% of the building height). Due to limitations of space and in order to present critical results, the inelastic rotations of the third and fourth floor columns at the base critical region are displayed only; the columns of interest are located at the leftmost column line of the exterior frame, which, due to the overturning effect of the lateral load and the small tributary slab areas, suffers a monotonic reduction in axial load during the SPO. From the different analyses performed, plastic hinge rotations within the example frame were concentrated primarily in these elements, although deviations exist depending on the formulation used, as described herein. Due to the way this building was designed following past seismic provisions, a soft storey mechanism is the predicted form of a response, with the location of the soft storey, varying with the model, between the third and the fourth floor.



`(a) (b)

Figure 3. Comparison of key performance predictions following SPO analyses for different FE models: (a) Capacity curves and (b) Plastic rotation of the 3rd and 4th story exterior columns at the floor level critical region.

In Fig. 3(a), the variation in the capacity curves is examined between the different formulations. Since most models employ cracked section stiffness and the same initial or 25% reduced concrete modulus, all capacity curves exhibit similar initial response up to about 500 kN of lateral resistance and 2cm roof drift: following modal analysis at this point, all the frames exhibit an initial cracked first mode of vibration equal to between 0.78 - 0.85 sec. The only initial response deviation from the capacity curves of the entire set is expectedly by model Forc_with_rigid_joints, which is based on a flexibility formulation (Forc_1) with all beam-column joints assumed to be non-deformable. Compared to Forc_1, this model exhibits an increased stiffness as well as lateral resistance which are primarily attributed to a modification of the lateral failure mechanism, as also indicated in Fig. 3(b) below. The Lump models exhibit the highest sensitivity to second order effects (particularly the model with 0.5% hardening) but exhibit similar lateral response as the rest of the set. However, a change in their post yield hardening stiffness (Lump_2) also results in a modification of the lateral resisting mechanism, with the soft storey formation changing to the third instead of the fourth floor. Finally, comparing the distributed damage elements Force, Disp_1 and Disp10, a completely different behavior is obtained; the Disp_1 model exhibits a 50% increase in base shear resistance as well as a considerably higher deformability compared to the Flexibility based model (Force), something that is bound to affect also the target point prediction level of this model during PBD. The Force model yields similar lateral strength as the Lump and Constant_Hinge models, with a direct influence of the critical columns' softening behavior to the overall global response (Fig. 3(a)) at around 7cm roof deformation, something not picked up by the Disp_1 frame which deforms up to 16cm with increasing strength.

After the discretization of the columns (model Disp_1) is increased to ten elements (model Disp_10) per column, the SPO curves of the Force and Disp_10 models converge and, therefore, so will do the target point predictions. Considering the local demands, however, of Fig. 3(b), there is at least a 35 % underprediction - still - of the plastic rotational demands of the fourth story column of these two models: as a consequence, while models Lump2, Force and Constant_hinge models exhibit similar roof – θ_{pl} dependence and a violation of the acceptance criterion of 1.0% rad at 8.3 cm drift, the Disp_10 model predicts the same plastic rotation at 9.8 cm, whereby the other models already yield 1.5% rad demands. Due to severe internal equilibrium violation, Model Disp_1 is completely unable to establish even failure of the fourth story column, with the maximum θ_{pl} predicted during the present analysis reaching a value of 0.36% rad. Finally, as also noted previously, it is interesting to note that the use of a rigid joint in the Forc_rigid_joint model results in plastic rotations identical to the fourth story column of the Force model, albeit at the third floor (Fig. 3(b)).

5. DYNAMIC ANALYSES OF THE EXAMPLE FRAME

5.1 Time History Analyses of Unscaled Ground Motions

The sensitivity of modeling predictions is pursued further in the time domain, for the two main candidate models yielding the most detailed modeling capabilities and yet the highest dispersion of SPO results, namely the flexibility (Force) and the stiffness (Disp_1) models (Opensees). For this purpose, the example structure is further analyzed in the time domain under recorded earthquake excitation. Ten accelerogram record components recently recorded in Greece are used herein; with the exception of the Lefkada 2003 record, which, however, is enveloped by the Code design response, all records are obtained in seismicity regions which correspond to the design seismicity region II (following RD59 [1959])of the example structure. All the records are used as recorded, without any scaling. Apart from the source location, a criterion for the record selection has been the predominant record period (ranging from 0.20 sec for Athens 1999 to 0.50 sec for Aigio 1996 and Lefkada records) and the duration of the strong motion (ranging from short impulsive type – Parnitha 1999 – to longer durations - Kalamata 1986 and Lefkada). The relative influence of the frequency content of each strong motion is shown in Fig. 4, where the elastic spectral amplifications of the records are compared to the elastic and inelastic design response spectra in effect today for the example structure in the period range of interest [EAK, 2000].

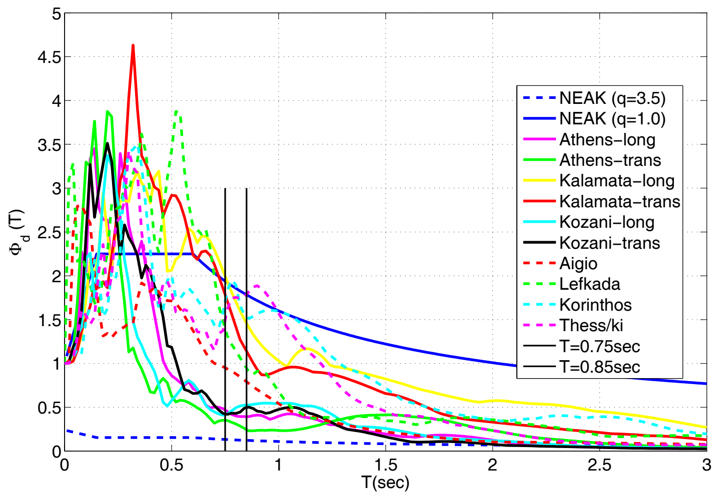


Fig.4 Elastic Design Response Spectra for the earthquake selection.

Typical plots of the peak maximum plastic rotations with roof displacement obtained in time history analysis of two representative types of base inputs are shown in Fig.5, where predicted plastic rotations are again compared to the ATC-40 [1996] and FEMA [1997] limits (of non conforming frames) for both FE models of interest herein. Not all the graphical results are given due to space limitation, since similarities exist between analyses., however, the entire set of peak deformation and θ_{pl} demands in the columns are summarized in Table 1. In this Table, the corresponding mean value and the 16%-84% percentile bracket dispersions of the maxima along the two axes (roof deformation and θ_{pl}), for either of the two models are estimated for each record, as also shown in colored lines at the scatter plots of Fig. 5 for the two records in display.

Examination of the results indicates that also for the dynamic analysis of the building at hand, the Force model consistently predicts higher plastic rotational demands at the columns – well above the 1.0% rad limit of

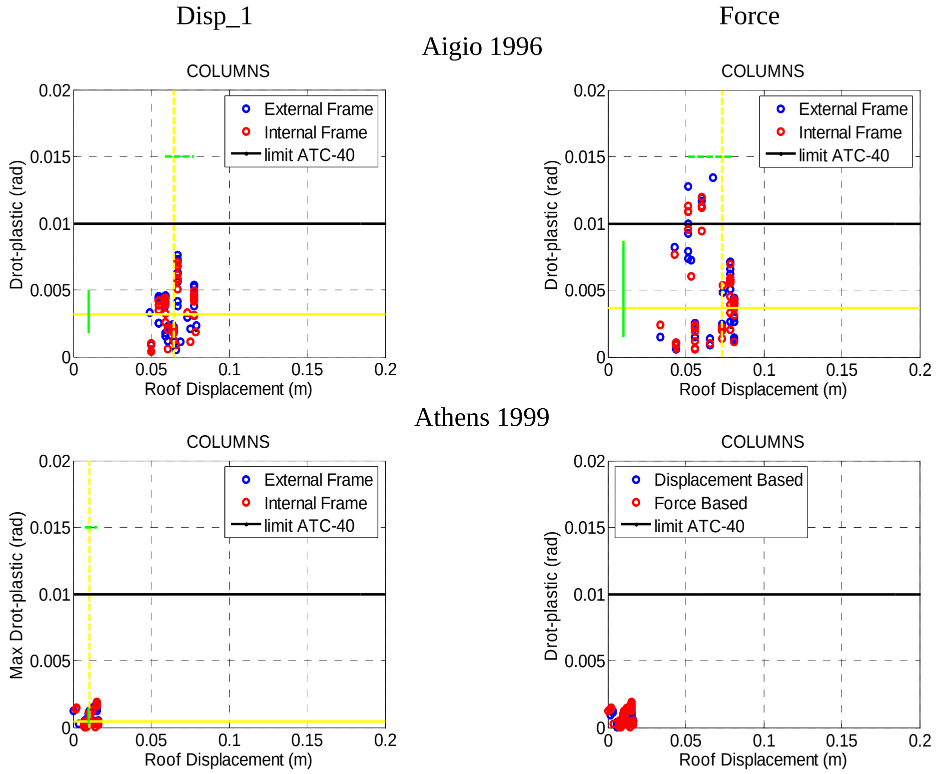


Fig.5 Comparison of the predicted maximum θ_{pl} demands and the corresponding roof drift for the Force and Disp models, following three critical base input excitations.

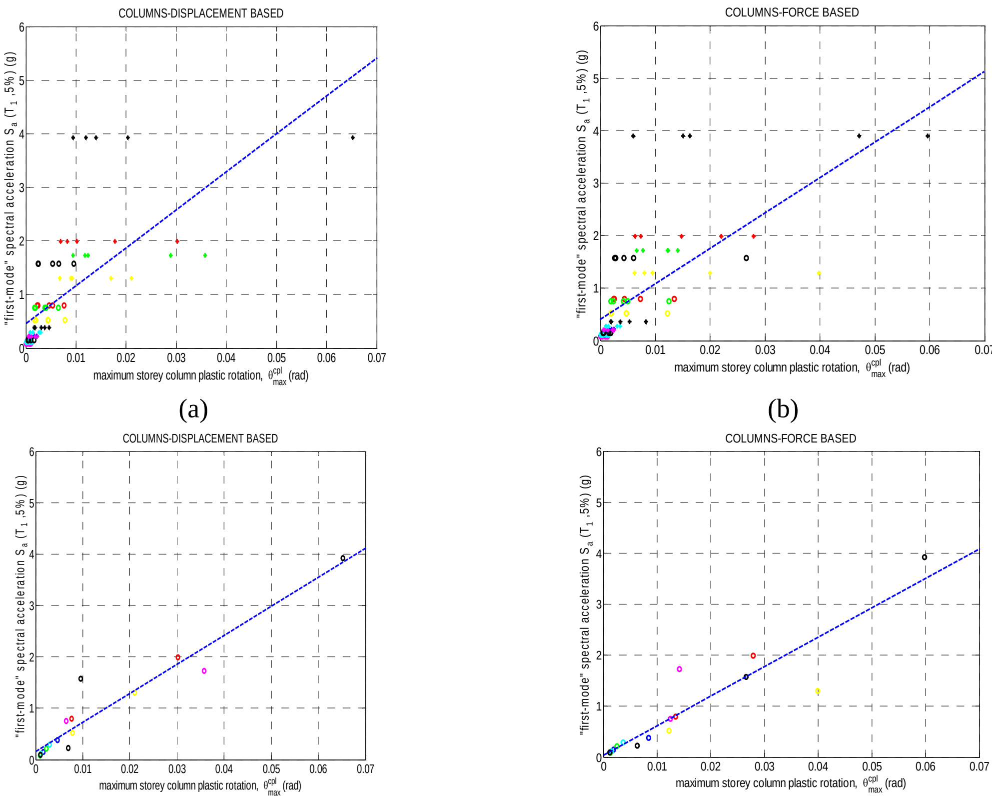
Table 1: Statistical estimates of the peak roof drift – plastic rotation demands in the example frame columns for all the ground motions considered (Force and Disp model).

		Roof δ , cm		θ_{pl} , % rad	
Excitation	Model	Median	Dispersion (84%-16%)	Median	Dispersion (84%-16%)
AIGIO	Disp_1	6.4	1.9	0.32	0.32
	Force	7.3	2.9	0.37	0.73
KALAMATA TRANS	Disp_1	6.2	1.1	0.25	0.27
	Force	6.2	5.1	0.30	0.77
KALAMATA LONG	Disp_1	7.9	2.1	0.25	0.33
	Force	6.2	2.0	0.28	0.70
ATHENS TRANS	Disp_1	0.8	0.4	0.02	0.07
	Force	0.8	0.3	0.03	0.10
ATHENS LONG	Disp_1	0.8	0.3	0.01	0.06
	Force	0.8	0.2	0.02	0.10
KOZANI LONG	Disp_1	1.1	0.9	0.04	0.10
	Force	1.1	0.8	0.04	0.13
KOZANI TRANS	Disp_1	1.1	1.0	0.04	0.10
	Force	1.1	0.8	0.04	0.13
LEFKADA	Disp_1	8.6	4.8	0.38	0.50
	Force	6.3	5.5	0.32	2.15
KORINTH	Disp_1	8.9	0.9	0.34	0.39
	Force	9.9	5.7	0.46	1.36
THESSALONIKI	Disp_1	2.7	1.9	0.08	0.10
	Force	3.7	1.1	0.15	0.18

acceptance, even though the dispersions along the ordinate (roof drift) are comparable between the two model frames considered. This is more critical in the case of the Aigio earthquake rather than the Athens record, whose primary excitation period was away from the first mode of the structure, thereby imposing minor inelastic deformations in the columns. It is of interest to compare also the dispersion pattern for the two models, as depicted clearly in Figures 5(a) and (b): while the Disp_1 model tends to concentrate the inelastic action predictions fairly uniformly among the participating members (all the floors), the Force model picks up extreme behavior of selected columns, whose predicted demands are much higher than the upper percentile of statistical evaluation and fail the acceptance criterion.

5.2 Statistical Estimation of Inelastic Rotations Using Scaled Cloud and IDA Analyses

The statistical evaluation of the expected performance under a set of extreme earthquake events of similar intensity can be established analyzing in the time domain a narrow or a wide data set of input records with a range of peak ground accelerations, yielding a statistical variation of the corresponding 5% damped spectral acceleration (the Intensity Measure) on one hand and the corresponding Response Index (typically the interstorey drift), or, in our case, i) the column plastic rotation per story and ii) the maximum column rotation over the entire building for each record. Of these proposed methods, the narrow data set analysis is the Scaled Cloud Analysis [Jalayer, 2003], in which these two peak response indices above are evaluated at 250% of the records' peak ground acceleration over the entire set, yielding a regression fit between the spectral acceleration at the first mode and the corresponding plastic rotations (Fig. 6).



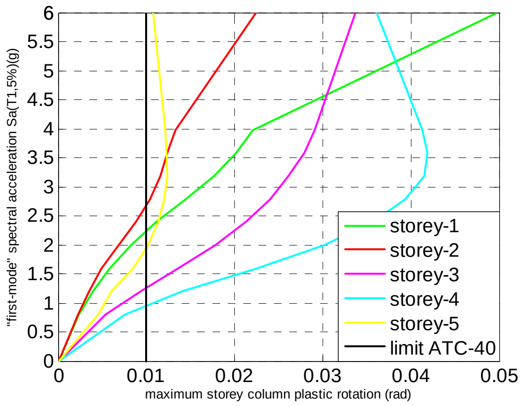
(a) (b)

(c) (d)

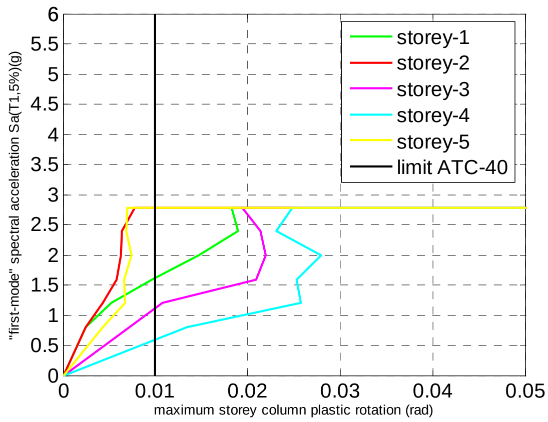
Fig. 6. Scaled cloud analysis and regression approximation of θ_{pl} for the 250% scaled events for the stiffness or flexibility FE models: (a)–(b) Max.θ_{pl} of any column per story, (c)–(d) Max.θ_{pl} of all columns.

Comparison of the Scaled Cloud Analysis results shows that the two models' responses exhibit similar regression fits, despite these models' differences in the response prediction following either an SPO or the single record inelastic analysis reported previously. It is therefore evident that such a model influence, obtained in SPO analysis or individual point estimates of the response may be phased out in an averaging sense over a set of excitations, unless proper scaling of the record set is enforced up to collapse.

In order to establish the contribution of FE modeling to the dynamic analysis performance prediction, a wider data set analysis is therefore used, involving an Incremental Dynamic Analysis (IDA) of the subject record set [Vamvatsikos and Cornell, 2002, 2004]; following this methodology, the set is analyzed over the entire range of record intensities, starting from a low service level and up to dynamic instability excitation level. The Intensity Measure covers in this case a wider range of time history analyses and is able to capture possible dependencies of the collapse mechanism and/or distribution and magnitude of the selected Response Index to this Intensity Measure. The results of such an IDA procedure for the Aigio 1996 record set are compared in Fig. 7 for the two models under investigation, considering the maximum column plastic rotation per story as the design Response Index which is subsequently compared to the limiting values of ATC-40 [1996] and FEMA [1997] for non-conforming frames.



(a)



(b)

Fig. 7 Predicted IDA curves for the maximum story column plastic rotation under the 1996 Aigio record, (a) Disp_1 and (b) Force models.

It is seen that the analysis over the entire range of intensities is able to capture the variation due to the model predictions, while at the same time demonstrating the significance of modeling sensitivity to the critical inelastic rotational demands: while the Disp_1 model test frame responds dynamically with higher demands at the fourth floor column line, a ground soft story mechanism seems to eventually prevail the response, with a corresponding increase in this story's column rotations but no instability. On the other hand, the use of the Force model exhibits a consistent behavior with increasing Intensity, whereby plastic demands concentrate and subsequently grow increasingly at the 4^{th} floor of the test structure. Finally, in terms of the actual point of exceeding the critical 1.0% rad limit, the critical spectral intensity for Aigio is twice as high for Disp_1 than Forc.

6. CONCLUSIONS

A typical five story non-ductile RC frame building which has been designed following past seismic regulations in Greece is analyzed in the nonlinear range both in static (pushover) as well as dynamic (incremental) analysis, as these are adopted in current seismic regulations under development in Greece, similar to other regions of high seismicity. The purpose of our study is to investigate the influence of element modeling to the predicted local damage indices of the subject building, using state of practice as well as widely accepted research tools which are widely available and are being used by the design profession. Such inelastic performance predictions are essential in order to qualify the structure following PBD evaluation as well as to evaluate possible redesign and strengthening interventions. Following the comparison of the global performance characteristics predicted, the form of collapse mechanism exhibited by the different models and the corresponding relationships between the global performance index (namely roof deformation) and the local damage intensity (namely, the local plastic rotational demands in the critical regions of the columns), the following are observed:

- The predicted collapse mechanism under SPO inelastic analysis and the corresponding global roof deformation – local plastic rotation dependencies among the different models may be considerably different. Due to this discrepancy, the predicted plastic rotational demands at a roof deformation of 1.0% of the building height vary to within 100% between each other, among the different formulations considered.
- Similar deviations were observed also under dynamic analyses, assuming that the structure is excited in the time domain by ten natural base excitations that have been recorded in similar seismicity zones within Greece

as the example structure under investigation. In this case, variations with modeling of the column inelastic rotations were more pronounced than variations of resulting roof deformation, while also lacking the capability to identify failure (assuming a failure rotation of 1.0% rad as the limit [ATC-40, 1996 and FEMA, 1997].

- Flexibility models exhibit higher demands and lower capacities compared to stiffness based elements, similar to the lumped plasticity formulation used in SPO predictions. However, the opposite holds true for dynamic analysis: in Scaled Cloud predictions, the differences between the two models were not as significant.
- The predictions of the stiffness formulations vary considerably with the number of elements used to model a single column element. Reducing the size of the elements to 10% the column length yields a better approximation of the global SPO Capacity curve (compared to that of the flexibility element predictions); yet, the local inelastic predictions of plastic rotation still lag those of the latter formulation by as much as 35%.
- In view of the discrepancies obtained from using different FE models or modeling conventions (e.g. the use of a rigid joint for the structural joints), the predicted θ_{pl} for acceptance or rejection of the structure should be interpreted with great caution and engineering judgment rather than as a hard performance index that is actually representative of the physical response, particularly for existing buildings. Possible dispersion of the predictions of this index from inelastic static and dynamic analyses may be significant.
- For a reliable application of PBD, the modeling conventions for estimating acceptance or rejection of indices, particularly since an inelastic analysis is employed, should be part of the legal Standard.

7. REFERENCES

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