INFLUENCE OF ELEMENT MODELING ON THE PREDICTED SEISMIC PERFORMANCE OF AN EXISTING RC BUILDING

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ABSTRACT

The effect of different element modeling formulations for reinforced concrete (RC) elements on the predicted performance of an RC building under seismic excitation is examined. The building selected is a typical existing five-story RC frame designed for moderate seismicity in the late 1960s, according to the older generation of Greek seismic codes with no special provisions for ductile behavior. Fiber elements are used to model the beams and columns using both stiffness and flexibility formulations, which lead to distinctly different behaviors. To evaluate the seismic performance of each alternate model, both static pushover and incremental dynamic analysis are used. The results are compared across all models, both at the local and the global level, to reveal the differences in the predicted seismic performance resulting from such a subtle modeling choice.

Introduction

The emergence of powerful computers has vastly improved our simulation abilities. Even for structures under seismic loading it is now feasible to model entire frames using distributed plasticity beam-column elements, e.g., fiber elements (Spacone et al 1996a,b,c), versus the simpler concentrated plasticity, point-hinge, elements. This is an enormous step forward, especially for reinforced concrete structures, allowing an accurate representation of the concrete section and its properties. Still, few advances come without problems before they are adequately understood: The two categories of fiber elements, the force-based and the displacement-based (Neuenhofer and Filippou 1997) have characteristics that can seriously influence the response, as shown, e.g., by Barbato and Conte (2005) for single elements and simple structures. What happens when a typical engineer uses such elements to evaluate the performance of a realistic structure? How far into the nonlinear range can we trust them for static or dynamic analysis?

We will investigate this issue by using a typical existing five-story RC frame designed according to the older generation of Greek seismic codes. The seismic performance of the structure will be evaluated using the standard static pushover analysis and the more recent incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002). Thanks to the “load-

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incrementing" nature of these procedures we can track the structural performance across all response levels and limit-states and extract useful information about the accuracy and stability of fiber elements under both static and dynamic loads.

Building Design Details

The building presented is a regular reinforced concrete (RC) existing structure three by four bays in plan, representative of RC construction in the 60s. It is five stories high with a storey height of 3.00m and regular 3.50m bay sizes in each direction. It has been designed for allowable stresses according to the 1959 seismic design code (RD59 1959), using C16 concrete (16MPa characteristic cylinder compressive strength) and smooth S220 reinforcement (220MPa nominal yield strength). The design loads adopted are the structure self-weight (including the perimeter 25cm thick infill walls), a uniform surcharge equal to 2.50kN/m^2 (including interior 10cm thick moveable partitions), and a live load of 2.00kN/m^2. According to the current design code (EAK 2000), the interior masonry infills are included as an additional uniform surcharge load over the entire floor plan, equal to 1.00kN/m2. The weight of the exterior infill walls is applied directly on the perimeter frame beams, expressed as a uniform load of 3.60kN per meter height of the wall per meter length. The detailing practices adopted at the time of construction involve the use of bent up bars in beams and inadequate capacity design shear reinforcement and critical region concrete confinement. The columns are square in section, 350x350mm at the first and second floor, reduced to 300x300mm at the third floor. For higher stories only the corner columns remain 300x300mm, while the inner ones are further reduced to 250x250mm. The beams are kept to dimensions 200/500mm in all cases. The building is assumed to be located in zone II seismicity of RD59 (out of three zones) and is therefore designed for an allowable stress level seismic base shear coefficient equal to 6% of the full dead plus live weight of the structure, applied following a uniform lateral distribution with height. For the seismic load combination, a 20% increase in the reinforcement allowable stress is allowed per RD59.

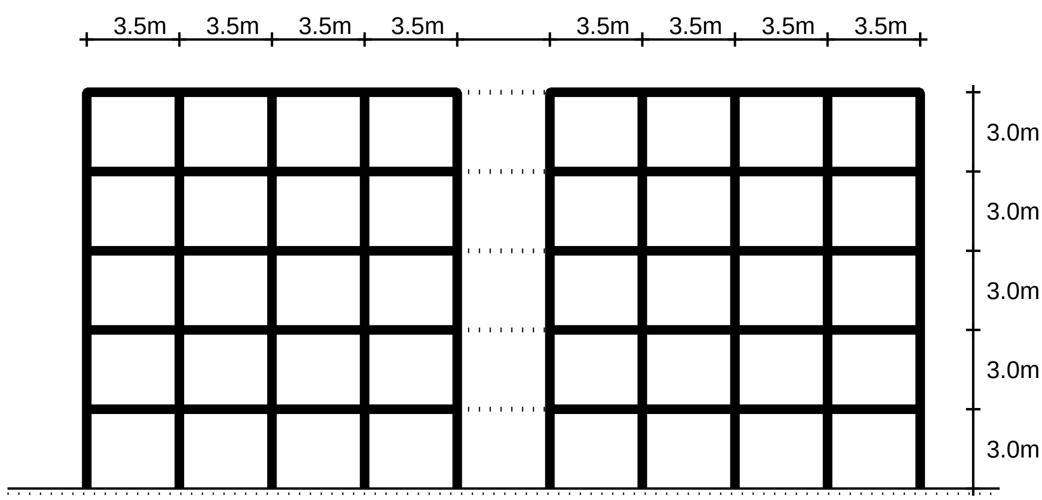


Figure 1. The structural model of the 5-story building. The slabs of the internal and external frames are rigidly connected at each story.

Structural Model

We chose to model the building as a 2D plane frame along its longitudinal direction (four bays). Due to the symmetry in the resulting two external and two internal four-bay frames, it suffices to model just one half of the structure, using one internal and one external frame side-by-side. These are rigidly connected to each other at each story level (Fig. 1) to simulate the rigid slab constraint. Following currently enforced analysis conventions for earthquake design (EAK 2000), vertical loads and corresponding masses are estimated from the dead loads plus only 30% of the live load for seismic load analyses. Using OpenSEES (McKenna et al 2001) as the analysis platform allows the modeling of the beams and columns using displacement (stiffness formulation) or force-based (flexibility formulation) fiber beam-column elements. The concrete sections are discretized in full detail into steel fibers and concrete fibers. The slab reinforcement within the effective width is included in the calculation of the flexural inelastic characteristics of the beams in negative bending, assuming different effective slab widths between interior and exterior frame members (namely 1.0m and 0.5m, respectively). The fiber constitutive relations are modeled using average material properties. The mean unconfined concrete strength is assumed to be equal to 18MPa, for concrete grade C16 (B120), while the effect of confinement (which is practically very low given the wide spacing of the stirrups) is taken into account following the confinement model proposed by Mander et al (1988). Based on actual reinforcement tests from 14mm diameter smooth reinforcing bars, the average yield stress of the reinforcement is equal to 310MPa. To successfully capture the varying reinforcement along each beam and the initial moment distribution due to gravity loading, each beam is uniformly subdivided into five beam-column fiber elements. On the other hand, the columns are modeled either as a single element or they are uniformly subdivided into three elements in series, especially when testing the behavior of the displacement-based element. In all, three distinct models are generated by the choice of the fiber formulation and the column discretization:

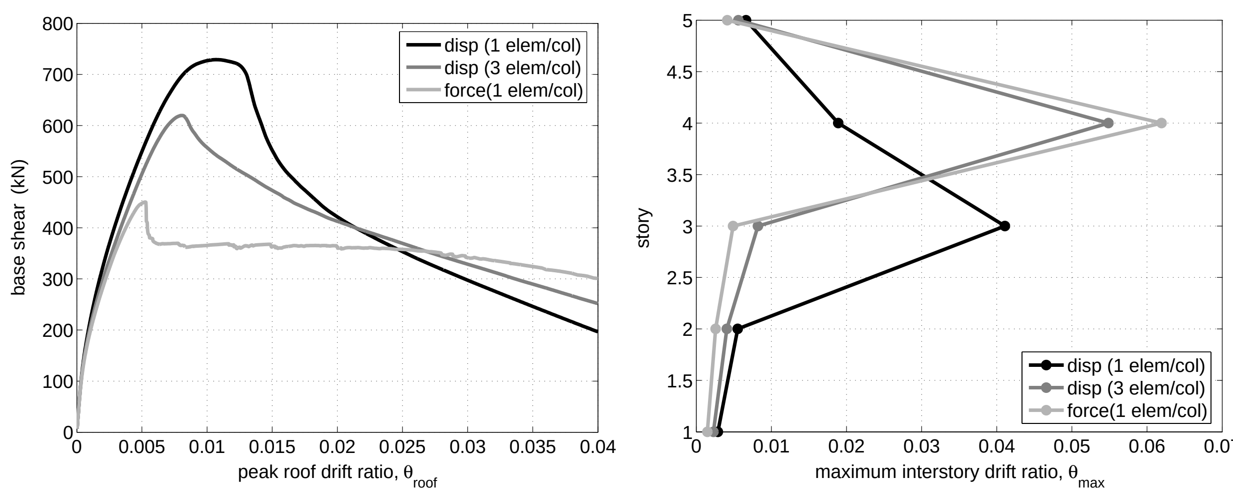
- Disp1: Displacement-based beams and columns, one element for each column
- Disp3: Displacement-based beams and columns, three elements for each column
- Force1: Force-based beams and columns, one element for each column

In all cases, a first order treatment of the effect of geometric nonlinearities (P-Δ effects) is included following the program modeling conventions. The final result is a relatively high-frequency structure with a first-mode period T_1 = 0.57sec.

Static Pushover Analysis

The static pushover analysis (e.g. Fajfar 1999), despite the criticism regarding its accuracy, remains a useful tool for evaluating the seismic response of structures. Using a triangular load-pattern we have obtained the pushover curves in terms of base shear versus roof drift θ_{roof} for the three models, as shown in Fig. 2a. Therein we observe that the force-based model (force1) is the most severe of the three up to about 2.5% roof drift. If we look at the interstory drift profiles (Fig 2b) the force1 model clearly predicts a story mechanism occurring at the fourth story. On the other hand, both displacement-based models predict much higher loads in this range, although a better refinement of the mesh (disp3 model) seems to help convergence to the force-based results. This becomes more apparent in the interstory drift profiles, where we can see how the mesh refinement has helped the disp3 model: While the disp1 predicts a mechanism involving the third and maybe the fourth story, the disp3 model (at the expense of considerably more degrees of freedom) has accurately located the mechanism at the fourth story.

Curiously enough, the situation is reversed when proceeding beyond 2.5% roof drift. There, the displacement-based models seem much more severe than the force-based, although only by a relatively small percentage. Still, even in this range, increasing the number of elements per column seems to help the displacement based-element converge to the force-based solution. Due to this reversal of roles between the force1 and the disp3 model we cannot claim that the displacement-based model is underpredicting demands in this setting. The two models simply find different mechanisms for structural collapse, and the results may be quite unpredictable at high response levels. Still, the force-based element clearly emerges as the more accurate one, as further mesh refinements do not change dramatically its response (e.g., Barbato and Conte 2005), while the mesh plays a major role for the displacement-based.



(a) (b)

Figure 2. The pushover curves (a) and the interstory drift profiles for $\theta_{roof} = 1.5\%$ (b) for a triangular load pattern for three different element-modeling options. Increasing the number of column elements in the displacement-based formulation clearly helps convergence to the force-based solution.

Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is an emerging analysis method that offers thorough seismic demand and capacity prediction capability (Vamvatsikos and Cornell 2002). It involves performing a series of nonlinear dynamic analyses under a multiply scaled suite of ground motion records, selecting proper engineering demand parameters (EDPs) to characterize the structural response and an intensity measure (IM), e.g. the 5% damped first-mode spectral acceleration, $S_a(T_1, 5\%)$, to represent the seismic intensity, and then generating curves of EDP versus IM for each record. On such IDA curves we may define limit-states and estimate the probabilistic distribution of their capacities, but we can also use them to compare different structures or models.

To perform IDA we used a suite of twenty records representing a scenario earthquake, as introduced in Table 1. These belong to a bin of relatively large magnitudes of 6.5 – 6.7 and moderate distances, all recorded on firm soil and bearing no marks of directivity. Each of these records was appropriately scaled to cover the entire range of structural response, from elasticity, to yielding, and finally global dynamic instability. At each scaling level a nonlinear dynamic analysis was performed and the EDPs were extracted. In our case we chose to retain the maximum, over all stories, interstory drift ratio, $\theta_{\max}$, and the maximum of each story's beam and column plastic rotations, $\theta_{\max}^{bpl}$ and $\theta_{\max}^{cpl}$. By interpolating such pairs of IM and EDP values for each individual record we get twenty continuous IDA curves, as shown in Figure 3 for $S_a(T_1, 5\%)$ and $\theta_{\max}$.

The twenty IDA curves may be easily summarized into their 16, 50 and 84% fractile IDAs by taking cross-sectional quantiles of the EDP at given levels (cross-sections) of the IM (Vamvatsikos and Cornell 2004). The summarized curves, plotted in Figure 4 for each of the three models, allow an accurate comparison of the dynamic behavior of the different element formulations for the 5-story building (Fragiadakis et al 2005).

Table 1. The suite of twenty ground motion records used.

Event Station	R [a] (km)	Soil [b]	φ [c] (deg)	PGA (g)
Superstition Hills 1987 (M=6.7) [d]				
1. Plaster City	21.0	C,D	135	0.12
2. Brawley	18.2	C,D	225	0.16
3. Wildlife Liquef. Array	24.4	–,D	090	0.18
4. Westmoreland Fire Station	13.3	C,D	090	0.17
5. El Centro Imp. Co Cent	13.9	C,D	000	0.36
San Fernando, 1971 (M=6.6)				
6. LA Hollywood Sto Lot	21.2	C,D	090	0.21
Imperial Valley 1979 (M=6.5)				
7. Chihuahua	28.7	C,D	012	0.27
8. Plaster City	31.7	C,D	045	0.04
9. Compuertas	32.6	C,D	015	0.19
10. El Centro Array #12	18.2	C,D	140	0.14
11. El Centro Array #13	21.9	C,D	140	0.12
12. Westmoreland Fire Station	15.1	C,D	090	0.07
13. El Centro Array #1	15.5	C,D	140	0.14
Northridge 1994 (M=6.7)				
14. Leona Valley #2	37.7	C,–	000	0.09
15. Lake Hughes #1	36.3	C,C	000	0.09
16. LA Hollywood Sto FF	25.5	C,D	090	0.23
17. LA Baldwin Hills	31.3	B,B	090	0.24
18. Canoga Park - Topanga Can	15.8	C,D	106	0.36
19. LA N Faring Rd	23.9	C,B	000	0.27
20. LA Fletcher Dr	29.5	C,D	144	0.16

[a] Closest distance to fault rupture
[b] USGS, Geomatrix soil class
[c] Component orientation
[d] Moment magnitude

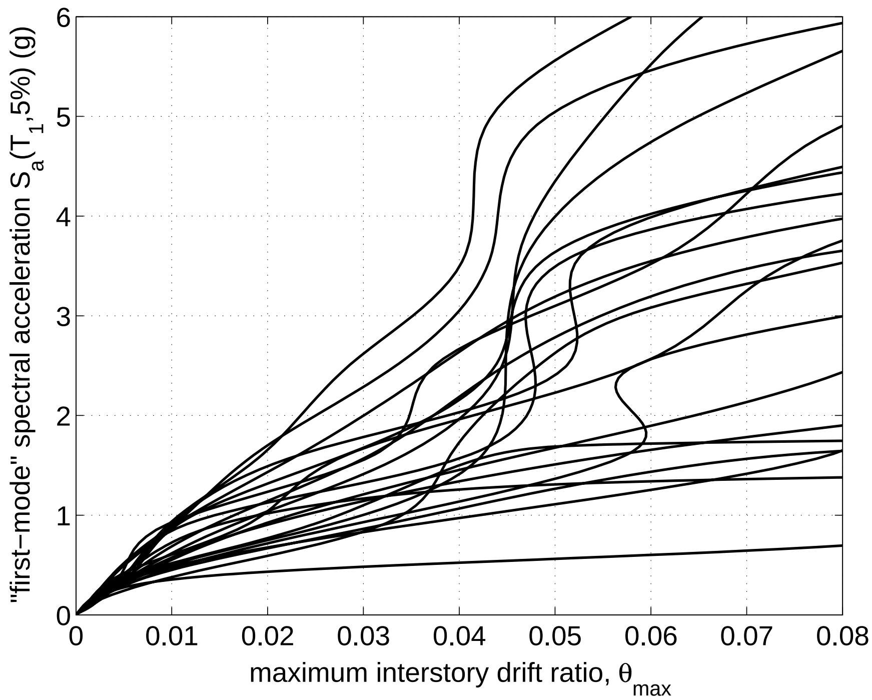


Figure 3. The 20 IDA curves versus θ_{max} for the force-based element model. The wide dispersion should be expected for this short-period building.

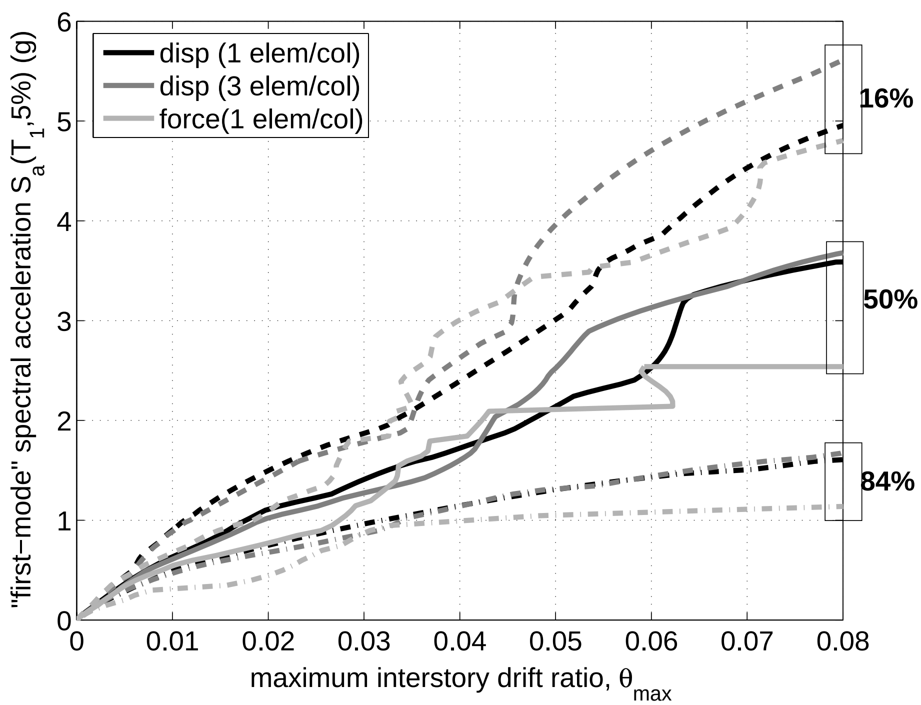
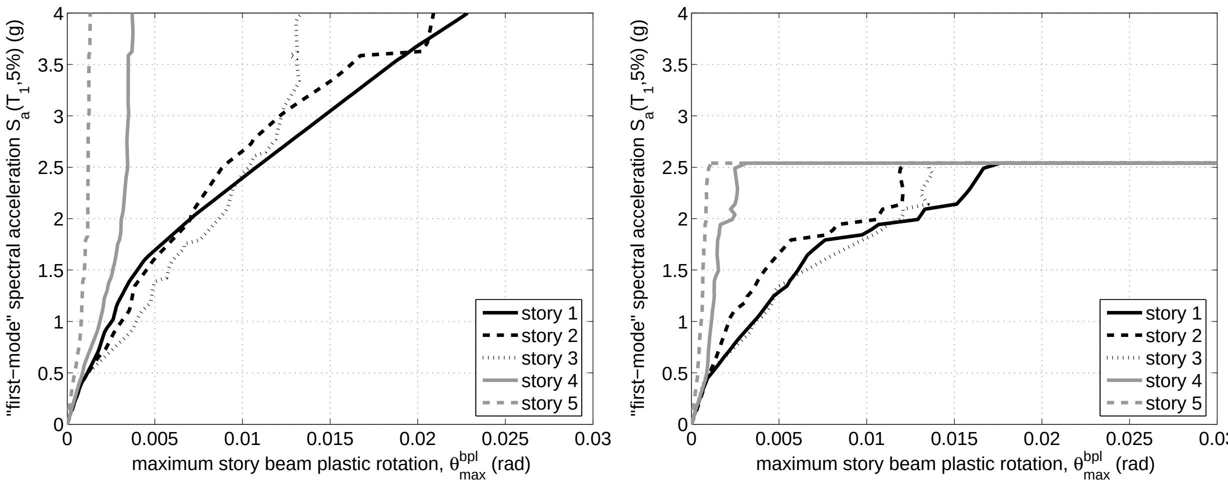


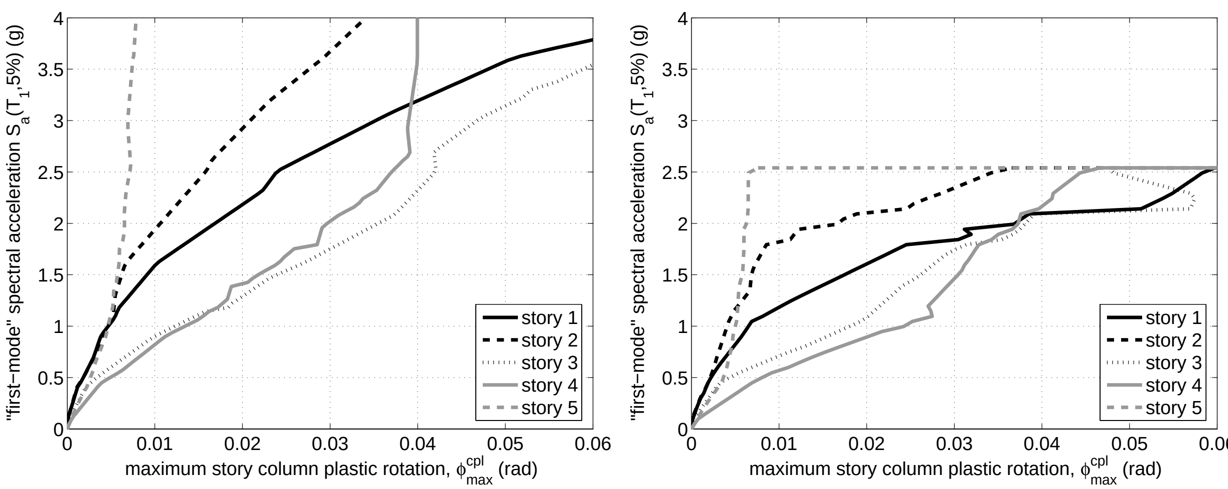
Figure 4. The fractile 16, 50 and 84% IDA curves for θ_{max} and each of the three models.

Looking at Fig. 4, it becomes obvious that for all fractiles the force-based model shows higher demands and lower capacities. While in general the dispersion due to the record-to-record variability is roughly the same among the three models, there is large difference in the fractile curves between displacement and force-based, practically for any level of response beyond the elasticity. On the other hand, the two displacement-based models show little difference to distinguish them from the noise induced by the record-to-record variability. Perhaps the most important difference between the models appears at the rather unrealistic response levels beyond $\theta_{\max} = 8\%$. There we can see that the force-based model easily collapses, showing a distinct flatline, while the displacement-based ones happily continue to higher intensities without any instability.



(a) (b)

Figure 5. The median IDA curves for the maximum beam plastic rotation of each story: (a) for the disp3 and (b) for the force1 models.



(a) (b)

Figure 6. The median IDA curves for the maximum column plastic rotation of each story. (a) for the disp3 and (b) for the force1 models.

These trends become more apparent if we study the IDA curves of a more localized EDP, such as the maximum story beam or column plastic rotations, $\theta_{\max}^{bpl}$ and $\theta_{\max}^{cpl}$ respectively. These appear in Figs. 5a and 6a for the disp3 model and in Figs. 5b and 6b for the force1 model. In all cases, the displacement-based model fails to capture the high demands that appear with the force-based element, practically from the onset of nonlinear behavior. At least, having discretized the beams with 5 elements, we get similar trends in the predicted beam plastic rotations, i.e. the first and third story are consistently the critical ones (Fig. 5). On the other hand, using only three elements for the columns is clearly not enough; the third story seems to prevail in the column plastic rotations for the displacement-based (Fig. 6a), but clearly it is the fourth story that is most critical according to the more accurate force-based model (Fig 6b).

The most importance difference though is the numerical instability apparent in the force-based results that clearly dominates the comparison. Can we believe the flatlines produced by the force-based element? Are they a fact, i.e. caused by legitimate global instability due to the softening of the members, P-Δ effects etc., or are they a fiction, i.e., a by-product of some element non-convergence? It is well known that force-based elements are inherently less stable (e.g. Neuenhofer and Filippou 1997) than displacement-based ones, and this has been encountered in the analysis where we had to use many local and global iterations coupled with adaptive solution techniques and fine time-stepping to achieve the flatlines shown in the figures. Still, practically speaking, the interstory drifts values in the order of 7% that are reached before instability appears are already quite high for this older RC structure. Furthermore, we have no compelling reason to question the legitimacy of these flatlines numerically since they are our most reliable estimates after having exhausted all numerical tricks in our bag. Nevertheless, the danger remains for the practicing engineer (or the typical commercial program) who will typically disregard the numerics and discover some quite early collapses that will lead to very conservative predictions.

Conclusions

The influence of element modeling with fiber elements was investigated using a 5-story non-ductile reinforced-concrete frame. Comparing through both static pushover and incremental dynamic analysis, the force-based elements have been found to display higher demands and lower capacities relative to displacement-based ones practically for the whole nonlinear range. Still, this is not an absolute point as in the later portions of the static pushover curve the exact opposite seems to occur. Displacement-based elements have also been shown to be quite dependent on the mesh size, needing several more elements versus the force-based ones to display reasonable accuracy. While this may seem as an expensive strategy, the computational cost is somewhat mitigated by the need to have many iterations and finer time-stepping when dealing with force-based elements. Especially, in the nonlinear range the inherent numerical instabilities associated with the flexibility formulation can prove to be a formidable obstacle for the practicing engineer. Then it becomes quite possible that early numerical instabilities at the element level will be mistaken for collapse, and lead to extremely conservative estimates of the building performance. It may seem easier to recommend the force-based elements for general use, but this will only be feasible with robust solution algorithms to accompany them.

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