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The effect of finite element modeling assumptions on collapse capacity of an RC frame building

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Abstract. The main objective of seismic codes is to prevent structural collapse and ensure life safety. Collapse probability of a structure is usually assessed by making a series of analytical model assumptions. This paper investigates the effect of finite element modeling (FEM) assumptions on the estimated collapse capacity of a reinforced concrete (RC) frame building and points out the modeling limitations. Widely used element formulations and hysteresis models are considered in the analysis. A full-scale, three-story RC frame building was utilized as the experimental model. Alternative finite element models are established by adopting a range of different modeling strategies. Using each model, the collapse capacity of the structure is evaluated via Incremental Dynamic Analysis (IDA). Results indicate that the analytically estimated collapse capacities are significantly sensitive to the utilized modeling approaches. Furthermore, results also show that models that represent stiffness degradation lead to a better correlation between the actual and analytical responses. Results of this study are expected to be useful for in developing proper models for assessing the collapse probability of RC frame structures

Keywords: time history analyses; collapse capacity; incremental dynamic analysis (IDA); RC buildings

1. Introduction

Structural collapse capacity is usually defined as the maximum ground motion intensity level that the structure is able to retain its stability. Accurate estimation of the collapse capacity is essential since it is the primary source of life and economic losses during a severe earthquake. Seismic design codes address this need and present the fundamental provisions to ensure that the building keeps its performance during inelastic deformations. Structural collapse is a complex phenomenon that is controlled by multiple factors. It should be pointed out that full-scale experimental testing is one of the best methods to identify the collapse mechanism of structures subjected to seismic loadings. Nonetheless, full-scale collapse tests are costly to perform and not in widespread use at all laboratories. Numerical simulations can be utilized as an alternative method to assess the inelastic responses of the structures subjected to cyclic loadings.

Details and results of the studies based on evaluating the correlation between analytical model responses and experiment test results are summarized below. Furthermore, evaluation of the accuracy of finite element analysis in seismic performance assessment of structures is taken into account in the scope of these investigations.

Negro and Colombo (1998) considered the response of a full-scale four-story RC frame building that was tested pseudo-dynamically in European Laboratory for Structural Assessment (ELSA). Research indicated that the lumped-plasticity model provides reliable results for predicting the nonlinear dynamic behavior of RC concrete frames.

Lee and Kang (1999) investigated the correlation between analytical response and experimental test results of a 1/12 scale 10-story RC frame building which was designed according to the Korean seismic code, 1988. Results confirm that models based on plastic hinge beam-column elements provide accurate estimates of ultimate strength and story drift ratios. Moreover, it was observed that the sequence of occurrence and distribution of plastic hinges were similar to the test results.

Building a mathematical model for a comprehensive seismic performance assessment involves making a series of modeling assumptions. Several finite element modeling approaches have been formulated to establish reliable models that can predict the nonlinear dynamic behavior of RC type structures. Performances of the widely-considered nonlinear finite element modeling (FEM) techniques in simulating the failure response of the RC members subjected to dynamic loads have been investigated by several researchers.

Zendaoui *et al.* (2016) investigated the applicability of four representative numerical strategies through the comparisons with experimental results of a four-story bare-frame type structure which was tested at ELSA laboratory (Pinto *et al.* 2002). They considered four modeling

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approaches:

1. Force-based with distributed plasticity, 2. Force-based with lumped plasticity, 3. Displacement-based with distributed plasticity, 4. Displacement-based with lumped plasticity.

Results obtained from the analytical model based on these assumptions were compared with the experimental results to evaluate the accuracy level of each modeling assumption. Critical demand parameters such as top displacement, inter-story drift, maximum story shear, damage pattern, and energy dissipation capacity of the frame structure are considered in the analyses. Results indicate that a minimum of four integration points are needed to obtain reliable results. It should be pointed out that the lumped-plasticity model based on the plastic hinge formulations proposed by Paulay and Priestley (1992) and Berry *et al.* (2008) showed better agreement in predicting the seismic performance of RC frame. Moreover, the displacement-based formulation with a lower bound of four-elements provided better performance regarding the convergence response. Additionally, it was concluded that the limit states based on the plastic rotations captured in the force-based analytical model were found to be effective in estimating the structural damages.

Ebrahami *et al.* (2013) performed a study based on the collapse simulation of a 10-story RC building utilizing the fiber-based nonlinear formulation. An incremental dynamic analysis was conducted using a set of 20 strong ground motion records. Results show that the structural collapse mechanism simulation assessment employing the fiber-based formulation is more accurate in comparison with results obtained from the lumped-plasticity model.

Performance of nonlinear modeling techniques based on structural member tests was investigated by Rodrigues *et al.* (2012). Three different nonlinear models based on force or displacement-based plasticity and lumped-plasticity elements (plastic hinge length was evaluated by considering the provisions in Paulay and Priestley (1992)) are considered to simulate the biaxial response of the 24 full-scale cantilever RC columns subjected to cyclic uniaxial and biaxial lateral loadings. Seven integration points were considered for the force-based formulation. On the other hand, in the displacement-based formulation, discretization of six elements with two Gauss-Legendre integration points per element was considered. Based on the comparison of analytical and experimental results, the global envelope response was found to be simulated with sufficient accuracy. Nonetheless, significant differences were obtained in the strength deterioration in the case of higher drift demands and energy dissipation.

Several modeling strategies can be utilized in the assessment of seismic performance by means of analytical modeling. However, building a mathematical model involves critical assumptions related to element formulation, hysteresis rules, and material stress-strain relationship. In this paper, the influence of major finite element modeling assumptions in predicting structural collapse capacity of a reinforced concrete frame building was investigated. For this purpose, commonly-used finite element formulations (i.e., lumped and distributed plasticity) and hysteresis rules (Bilinear, Takeda (Takeda *et al.* 1970), and Fiber) are taken into account.

Investigation of the performances of commonly utilized nonlinear finite element modeling (FEM) techniques in simulating the responses of RC members subjected to dynamic loads, has been a popular research field. A critical limitation related to commonly utilized displacement-based element formulation is the utilization of cubic displacement shape functions which leads to linear curvature variation along each element. In order to circumvent this limitation, a new quadratic shape function-based element formulation with constant axial load criterion was developed by (Izzuddin *et al.* 1994) for the efficient nonlinear analysis of RC members. This formulation was implemented in the nonlinear analysis program ADAPTIC (Izzuddin *et al.* 1989). Results from the application of this element formulation for the analysis or RC frames under seismic action was presented by (Karayannis *et al.* 1994). Additionally, they investigated the capabilities introduced by performing automatic mesh refinement within an adaptive analysis framework. Results of applications show that adaptive analysis is a useful and time-saving technique for nonlinear analysis of frame type structures and higher-order quartic shape functions can be considered to increase the level of accuracy on the prediction of displacements of structural members.

Pushover analyses are conducted to evaluate the ultimate deformation capacities of the models. Seismic collapse capacity of each model is obtained via Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002). Accordingly, the structure is subjected to a set of excitations with increasing intensity. The considered modeling schemes are evaluated through the comparisons with experimental results of a three-story RC bare frame building detailed in the study by Fardis (2002). Results of this study are expected to be valuable in selecting the appropriate finite element modeling scheme in evaluating the structural collapse capacity.

2. Collapse capacity prediction

The incremental dynamic analysis, so-called “IDA” (Vamvatsikos and Cornell 2002) was recently adopted by the U.S. Federal Emergency Management Agency (FEMA) guidelines as a method to determine the collapse capacity point of the structures under seismic loads. In this approach, each ground motion is scaled to a specified spectral acceleration (S_a) level at the first mode period T_1 of the structure. Time history analyses are performed repetitively. In each step, scaling factor of the ground motion is increased, and the inter-story drift ratio, (IDR) is calculated at the diaphragm point (Center of mass, CM) of each story. The maximum value of the IDR is considered as the IDR_{max} for that scaling factor. This process is repeated until the collapse limit conditions are reached. For the identification of the collapse limit, Vamvatsikos and Cornell (Vamvatsikos and Cornell 2002) presented two conditions (Damage Measure and Intensity Measure).

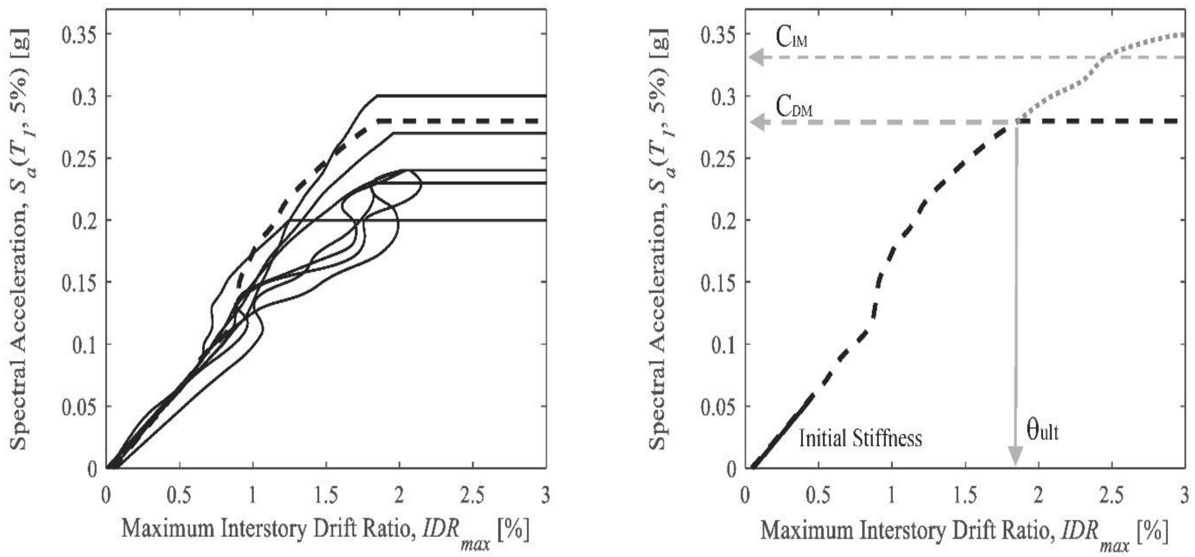


Fig. 1 CDM and CIM based rules for identifying the capacity point

1. (*DM*): maximum inter-story drift ratio reaches or exceeds the ultimate deformation capacity.
2. (*IM*): Slope of the IDA curve is reduced to 20% of the initial (elastic) slope.

The collapse capacity depends on the assumed threshold values on *IM* and *DM* based rules. The point at which the IDA curve reaches a flat plateau (large structural response corresponding to a small increase in the ground motion intensity) indicates the dynamic instability of the structural system. In some cases, IDA curves show a waving behavior. Originally, incremental dynamic analyses were mainly implemented in two dimensional (2D) structures. The test building considered in this study was subjected to bi-directional seismic actions during the experiment. Therefore, a three-dimensional (3D) model of the building subjected to bi- directional actions were considered in this study. Accordingly, the ultimate inter-story drift ratio capacity for each direction of the building was evaluated separately to identify the C_{DM} s in two orthogonal directions.

3. Description of modeling schemes

The aim of this study is to investigate the effects of alternative finite element modeling assumptions on the predicted collapse capacity of an RC frame structure. To achieve this, numerical analyses are performed by utilizing different finite element modeling schemes which are available in the OpenSees framework (McKenna *et al.* 2009). OpenSees framework provides a comprehensive set of material models, hysteresis rules, and element formulations that can be utilized in nonlinear dynamic response history analyses of structures. Additionally, parallel processing extension enables users to speed up the analyses process by making use of computer clusters. Moreover, it should be noted that other well-established programs (e.g., DRAIN2D (Kanaan and Powell 1975), Perform 3D (CSI 2006), ADAPTIC (Izzuddin *et al.* 1989)) that have the capability of performing nonlinear dynamic response history analyses can be also taken into account to perform further investigations on the effects of modeling assumptions on the predicted collapse capacity. Except for some unique features specific to each particular code, there is some level of commonality in the element formulations and hysteresis models implemented in those codes and the ones available in OpenSees.

The numerical modeling assumptions utilized in this paper are based on finite element modeling strategies. In this study, two major properties are considered in the development of alternative modeling approaches: (1) element formulation, and (2) hysteresis rule. Two different widely-used finite element models (i.e., distributed and lumped plasticity) are considered for representing the spread of plastic deformations within an element. Four alternative element formulations are considered here: 1. Displacement-based (D), 2. Force-based (F), 3. Beam-with-hinges (B), 4. Zero-length (Z).

These finite-element formulations are based on different assumptions related to the relationship between global member-end deformations and the local deformations within the member which are paired with different hysteresis rules: 1. Bilinear (B) 2. Takeda (T) 3. Fiber-section (F). As a result, a total of 11 alternative finite-element models of the test structure are established (Fig. 2 (a)).

3.1 Element formulations

In *distributed plasticity* approach, the plastic deformations are not concentrated to specific locations along the element but instead they are spread over a distance within the plastic hinging region. Distributed plasticity models can be implemented either by utilizing the displacement-based formulation (stiffness-based approach) or force-based formulation (flexibility-based approach).

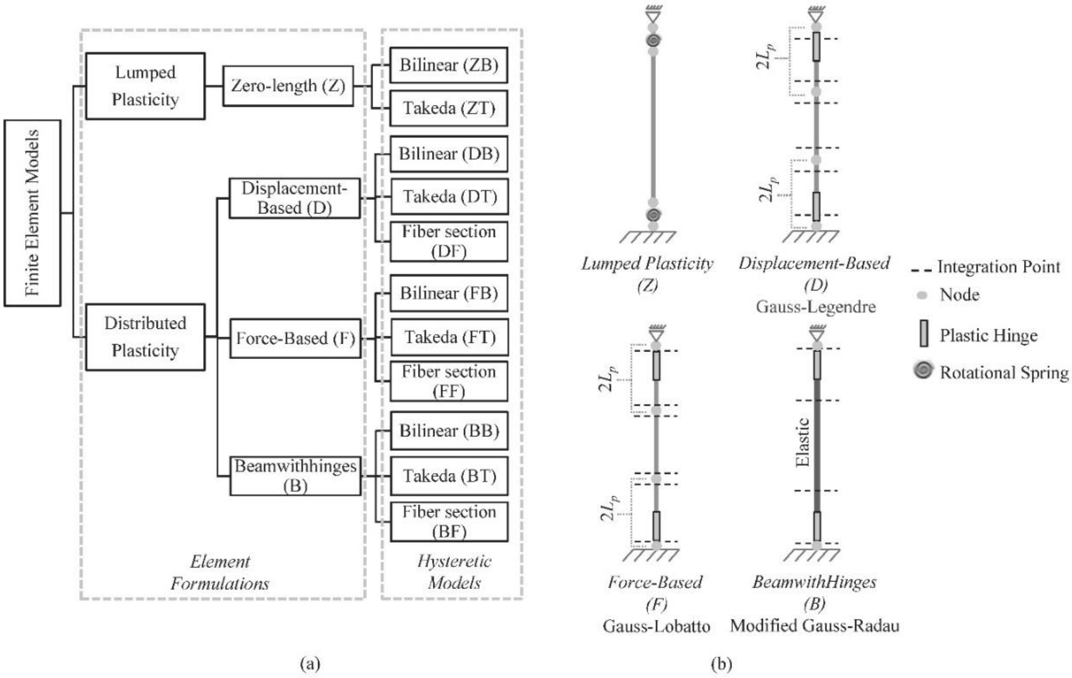


Fig. 2 (a) Finite element modeling approaches considered in analyses and (b) discretization schemes

In *displacement-based* element formulation (Bathe 1996) curvatures and axial deformations are evaluated based on the assumed displacement shape functions. Typically, a third order displacement shape function is utilized for the transverse displacements. This leads to the enforcement of a linear curvature variation within each element. Thus, a refined discretization scheme of structural components is required to capture the non-linear post-elastic curvature distribution along the RC components.

The *force-based* element formulation (flexibility-based approach) is based on the interpolation of member end-forces for capturing the internal forces at each section (Taucer *et al.* 1991). Coleman and Spacone (2001) comprehensively addressed that due to the inelastic deformation localization issues, the simulated response of the force-based element is sensitive to the adopted discretization scheme. Hence, an appropriate discretization scheme over RC components and suitable integration point distribution over each finite element is needed in order to obtain reliable results.

More recent *BeamwithHinges* element formulation was proposed by Scott and Fenves (2006). The formulation is based on the force-based formulation with two-points Gauss-Radau integration along the plastic hinging regions. This model enables considering the plasticity non-uniformly spread over the plastic hinge length. Additionally, the force-deformation characteristics of the middle segment are assumed as elastic. In principle, it is a macro-element that consists of two force-based beam-column elements that are connected with an elastic Bernoulli beam segment in between.

Adopting an appropriate discretization scheme in analytical models plays a crucial role in stimulating the plastic curvature accumulation along the plastic hinge regions. In this study, the discretization scheme proposed by Yazgan (2009) is implemented. Yazgan (2009) concluded that a good approximation can be obtained by discretizing structural components into multiple finite elements by taking into account the expected lengths of plastic hinging regions. This approach is represented in Fig. 2(b). It should be pointed out that two Gauss-Lobatto and Gauss-Legendre integration points (IP) are employed for the force and displacement-based formulations, respectively. In the present study, lengths of elements located at the plastic hinging regions are set equal to twice the plastic hinge length, Lp. The plastic hinge lengths of the structural components are evaluated by using the equation proposed by Priestley *et al.* (2007).

Typically under seismic loads, inelastic deformations in the slender RC components (e.g., beams and columns) are concentrated at two ends of the member where maximum bending moments occur. To this end, the lumped plasticity model is developed to localize the inelastic deformations to two zero-length hinges located at the element ends. Accordingly, the hysteresis behavior of RC (stiffness degradation in flexure and shear) can be captured by the adoption of an appropriate moment-rotation springs at two ends of the component.

3.2 Hysteresis rules

In the present research, widely-considered cyclic stress-strain models are adopted to simulate the response of materials under seismic actions. Considered models aim at simulating the stiffness degradation, strength deterioration, and energy dissipation.

The *bilinear* hysteresis model is a non-deteriorating hysteresis rule in which the system behavior remains elastic until the yield strength is reached. Following the yield point, the initial stiffness, K_{ini} turns to post-yield flexural stiffness until the unloading. Given that stiffness degradation and energy dissipation are not captured by this model, it has limited accuracy in simulating the nonlinear behavior of RC structures under seismic loads.

In this research the simplified version of the *Takeda* hysteresis model (Takeda *et al.* 1970) with a bilinear backbone curve that was proposed by Saiidi and Sozen (1979) is employed. Unlike the Bi-linear model, stiffness degradation is considered in the Takeda model based on the maximum plastic deformation. *Hysteretic* model in OpenSees is utilized for representing the modified Takeda hysteresis model. Based on the recommendations by Saiidi and Sozen (1979), the unloading stiffness parameter, γ is considered as 0.5.

The *fiber* section model is the most advanced modeling approach in capturing the post-elastic flexural cyclic response of RC sections. In the fiber section model, cross-section of a structural member is divided into tiny pieces of cells that are referred to as fibers which follow a uniaxial hysteretic stress-strain behavior. The main objective of the fiber model and force-deformation rules is to simulate the material nonlinearity which accounts for the inelastic behavior in reinforced concrete. Despite the force-deformation rules which reflect the member behavior as a whole, the fiber model uses a set of stress-strain relations to characterize the sectional response. Therefore, no initial calibration of the moment-curvature hysteretic rule is required. Moreover, the interaction between axial force and bending moment is directly taken into account. On the other hand, it is challenging to reproduce flexural-shear interaction.

In the analysis, material model *Steel01* implemented on OpenSees platform is employed to capture the cyclic response of reinforcement steel. *Steel01* is a bi-linear constitutive model with post-yield strengthening. *Steel01* material is characterized by three parameters including yield stress f_y, initial elasticity modulus K_{ini}, and post-yield modulus defined by the strain hardening ratio b. The stress-strain behavior of the confined concrete fiber was modeled using the *Concrete04* stress-strain model available in OpenSees which based on the model proposed by Mander *et al.* (1988). Additionally, the *Concrete01* uniaxial-material was utilized to simulate the stress-strain behavior of the unconfined concrete. It may be noted that the code is based on the constitutive model proposed by Kent and Park (1971).

4. Experimental test unit properties

An experimental building model is considered to illustrate the influence of finite modeling assumptions on the predicted collapse capacity. For this purpose, the SPEAR building (Fardis and Negro 2006) was selected for the analyses. The SPEAR building is a full-scale, three-story, asymmetric RC bare frame that was tested pseudo-dynamically at the European Laboratory for Structural Assessment (ELSA), Ispra-Italy. Details of the design procedures and structural members are available in Fardis and Negro (2006). Moreover, typical structural plan, elevation and cross-sectional properties of structural members are provided in Fig. 3 and material characteristics are given in Table 1.

Table 1 Steel properties based on material test result, adapted from (Dolsek and Fajfar 2005)

Bar Φ [mm]	Yield strength f_y [MPa]	Yield strain ε_y [%]	Ultimate strength f_u [MPa]	Ultimate strain ε_u [%]	Young's modulus E [MPa]
8	467	0.227	584	13.1	206000
12	459	0.223	570	17.4	206000

The structure represents characteristics of an existing building without any provisions for earthquake resistance. Plan irregularity, utilization of smooth bars, insufficient transverse reinforcement in the structural members are the weak points of the selected bare frame. Confinement effect of the core concrete is neglected in the numerical model due to inadequate transverse reinforcement and large spacing of stirrups. Mean compressive strength of concrete is considered as 25 [MPa] in analyses as reported by Dolsek and Fajfar (2005). The average yield strength of steel was rather high and was measured as 459 [MPa] and 377 [MPa] for the 12mm and 20mm bars, respectively (Table 1). The plastic hinge lengths of the structural components are evaluated by using the equation proposed by Priestley *et al.* (2007). Beams B4-B9 have plastic hinge lengths equal to 24 [cm] while all the other columns and beams have plastic hinges that are 41 [cm] long.

Self-weight of structural members, finishing loads of 0.5 [kN/m^2], and imposed live loads of 2 [kN/m^2] are considered as the vertical loads. Story masses are assigned to the center of mass of each story (rigid diaphragms are viewed at the floor levels). Total translational masses, calculated as 67.3 [t] each of the first and second floors and 62.8 [t] for the third floor.

Two perpendicular components of Herceg-Novi (Montenegro earthquake) ground motion record was modified to match the EC8 elastic design spectrum (Type 1, soil C). They were utilized as the input excitations of the bidirectional pseudo-dynamic test. In the first test, the intensity of peak ground acceleration, PGA was set equal to 0.15 [g] so that only minor damages occurred. Subsequently, an additional test was run at an increased intensity of 0.2 [g].

Critical demand parameters (i.e., displacement at the center of mass, inter-story drifts and top displacements in two perpendicular directions) are calculated and compared with experimental results at each ground motion intensity level for the considered set of FE modeling approaches.

5. Prediction of collapse capacity

Ultimate inter-story drift capacity of the frame was identified by performing pushover analyses. In these analyses, rotations at the critical locations were compared with the ultimate rotation capacities. These rotations were identified by performing moment-curvature analyses of the sections based on the recommendations by Priestley et al. (1996).

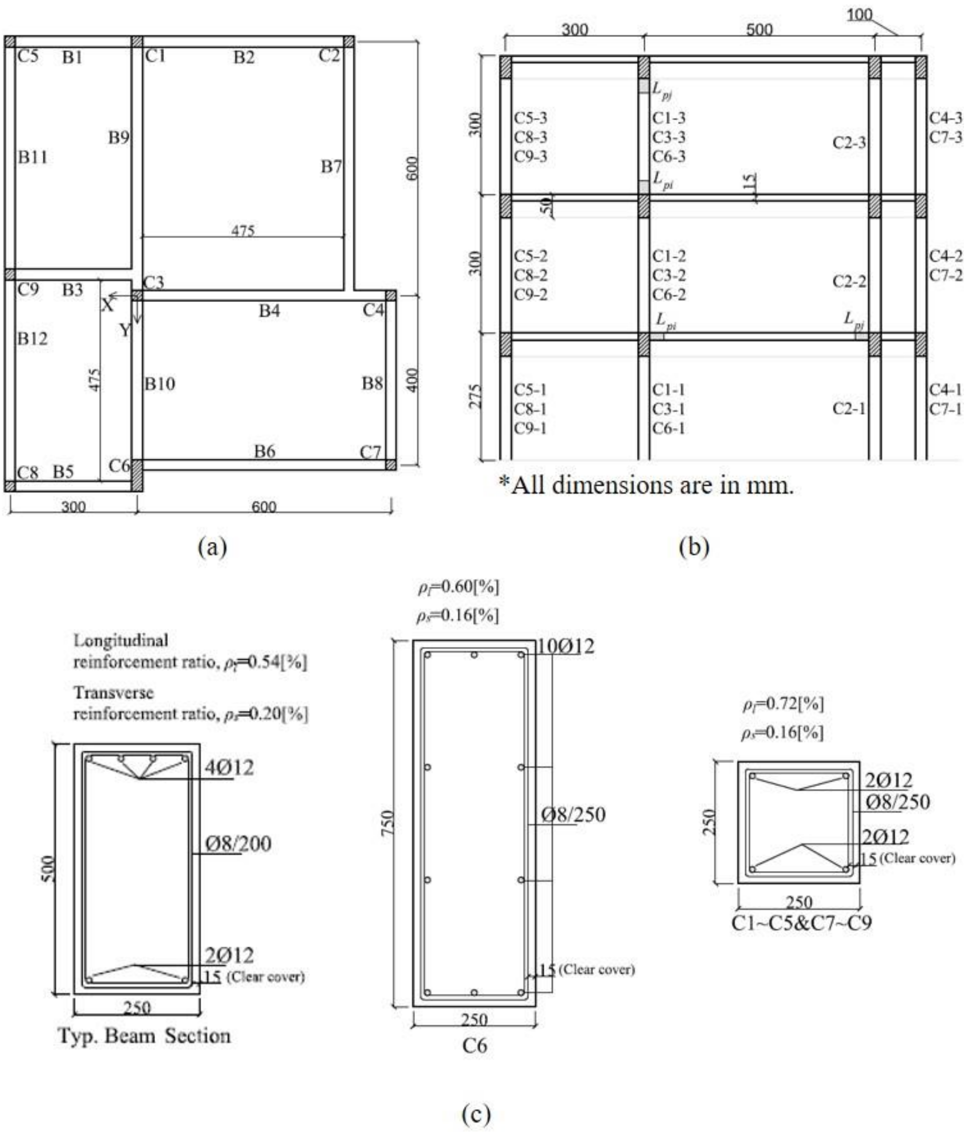
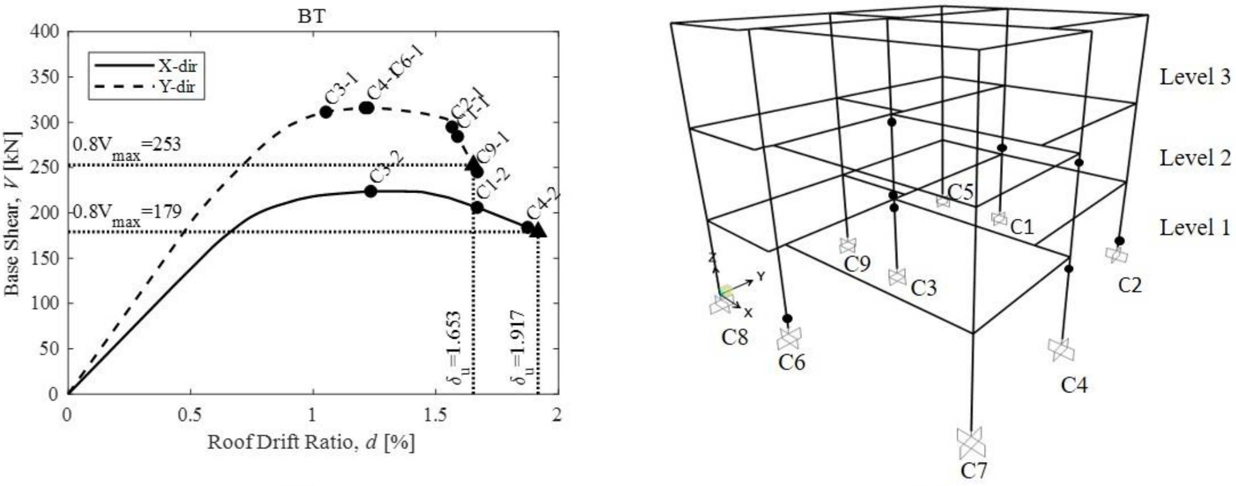


Fig. 3 (a) Plan view, (b) elevation and (c) typical reinforcement details of the SPEAR building

Usually, evaluating the modal periods of vibration and mode shape vectors is the first step in dynamic analysis of structures. It is worth noting that the cracked cross-sections were used here in evaluating the modal periods of vibration. Accordingly, the first three periods of vibration of the SPEAR building are calculated for alternative FE modeling assumptions (Table 2). Additionally, it should be noted that the SPEAR frame was tested as a bare frame without any infill walls.

The most valuable and useful information that is acquired in pushover analyses is detecting the ultimate deformation capacity for each alternative FE approach. Here, the ultimate deformation capacity is defined as the drift level at which the lateral load has dropped to 80% of its peak load. This capacity is also considered as the damage measure (DM) indicator in IDA. Pushover analyses are conducted in both, X and Y directions. An inverted triangular load pattern is applied along the height of the SPEAR building (at the center of mass, CM point). The displacement control strategy is adopted to capture the horizontal displacement in each step of analysis until the ultimate inter-story drift capacity is reached. In each step, the section that has reached the ultimate curvature is indicated with a dark circle in Fig. 4(b). Cross-sectional properties of structural members and percentages of reinforcements are provided in Fig. 3.

The base-shear versus roof displacement relationship (capacity curve) obtained using the BeamWithHinges element formulation and Takeda hysteresis (designated as 'BT' model), is presented in Fig. 4. Moreover, the plastic hinge occurrences in progressive stages are illustrated in this figure. In this notation, the first character "C" is indicative of column, the first number represents the column number, and the second number indicates the story level. Results presented in Fig. 4 show that the base shear resistance capacity of the building in Y-direction is approximately 40% stronger than that in X-direction. Moreover, it can be observed that the first hinge is expected to occur in column C3 at the ground story. Additionally, a difference can be observed between the descending



(a) (b)

Fig. 4 (a) Nonlinear static pushover curve for BT model along X and Y directions, and (b) a 3D illustration and plastic hinge formations for the model, BT

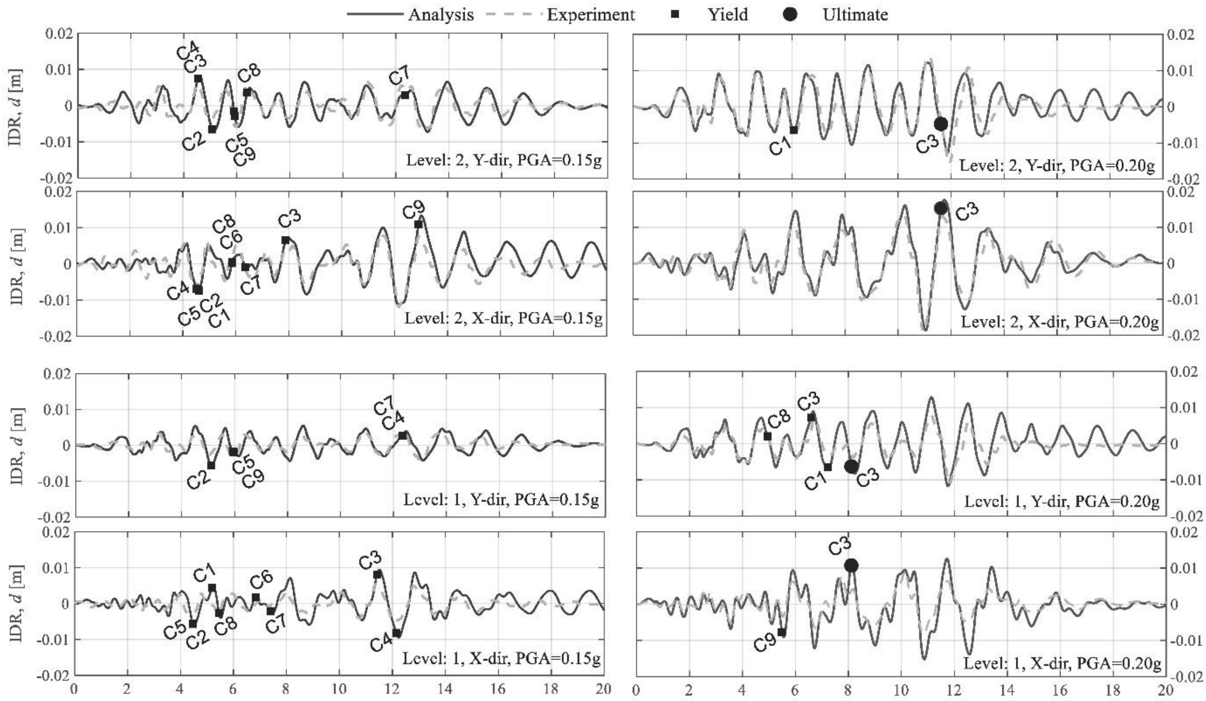


Fig. 5 Drift traces of the experimental responses and analysis results of BT model for 1st and second levels: (a) *PGA* = 0.15g, and (b) *PGA* = 0.20g tests

segments of the pushover curves related to the X and Y directions. Specifically, the pushover curve evaluated along Y direction represents a steeper descending segment compared to that part of the pushover curve along the X-direction. This steep decrease of base shear can be attributed to the fact that the drift ratios corresponding to the onset of ultimate deformation capacities for two of the columns are very similar along Y direction compared to those evaluated along the X-direction. This almost simultaneous failure of two columns results in a sharp decrease in the total base shear after 1.5 % roof drift ratio in the Y-direction (Fig. 4(a)).

The stiffest column in the building model is the column C6 and its stronger axis is located along Y direction. As a result, the building is 1.4% stiffer in Y direction compared to X direction, as seen in Fig. 4. On the other hand, the ultimate drift capacity along X direction is 11.5% higher than that along Y direction.

Column C3 is subjected to the axial load ratios (N/f_cA_g) of 28.8%, 19%, and 9.3% at the first, second, and third story levels, respectively. For column C3, severe damages are initiated in the drift ratio of 1.05% at the first story when

Table 2 Results obtained using different FE approaches

Model	K_{ini} [kN/m]		T [s]			δ_u^* [%]		V_{max}^{**} [kN]		$d^{\dagger\dagger}_{t,p}$ [%]			
										0.15g		0.2g	
	X	Y	1st	2nd	3rd	X	Y	X	Y	X	Y	X	Y
Exp			0.85^+	0.78^+	0.66^+					0.78	0.57	1.21	1.18
BB	277	373	1.32	1.08	0.90	1.93	2.18	225	318	0.83	0.70	1.12	0.87
BT	276	373	1.32	1.08	0.90	1.92	1.65	224	316	0.94	0.66	1.30	1.09
BF	295	402	1.05^+	0.97^+	0.81^+	1.46	1.30	233	323	0.88	0.56	NC	NC
DB	280	414	1.25	1.07	0.86	2.25	2.13	245	351	0.76	0.85	0.94	1.09
DT	280	414	1.25	1.07	0.86	1.84	1.51	243	351	0.82	0.68	1.28	0.89
DF	368	559	0.81^+	0.73^+	0.50^+	1.56	1.29	289	389	0.76	0.63	0.89	0.80
FB	274	407	1.26	1.08	0.87	2.18	2.00	228	329	0.75	0.84	0.99	1.07
FT	274	407	1.26	1.08	0.87	1.89	1.56	227	329	0.93	0.57	1.26	1.00
FF	349	533	0.83^+	0.74^+	0.51^+	1.40	1.18	242	339	0.61	0.77	NC	NC
ZB	257	349	1.32	1.17	0.98	2.03	2.37	233	324	0.73	0.73	0.88	0.97
ZT	257	350	1.32	1.17	0.98	2.07	1.75	232	321	1.02	0.73	1.35	1.11

+: Modal periods for uncracked stiffness

$^*\delta_u$: Ultimate drift

$^{**}V_{max}$: Peak base shear

$^{\dagger\dagger}d_{t,p}$: Roof peak drift

Table 3 Columns plastic hinges that reach the ultimate capacity during the IDA

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<tr><th rowspan="2">Column Level</th><th colspan="2">BB</th><th colspan="2">BT</th><th colspan="2">BF</th><th colspan="2">DB</th><th colspan="2">DT</th><th colspan="2">DF</th><th colspan="2">FB</th><th colspan="2">FT</th><th colspan="2">FF</th><th colspan="2">ZB</th><th colspan="2">ZT</th></tr>
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<tr><td>C7</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>x</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>C8</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>x</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>C9</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

the building is pushed in the Y-direction. Following the damage accumulations, a drift ratio of 1.23% at the second story is evaluated when the building is pushed along the X-direction (Fig. 4(a)).

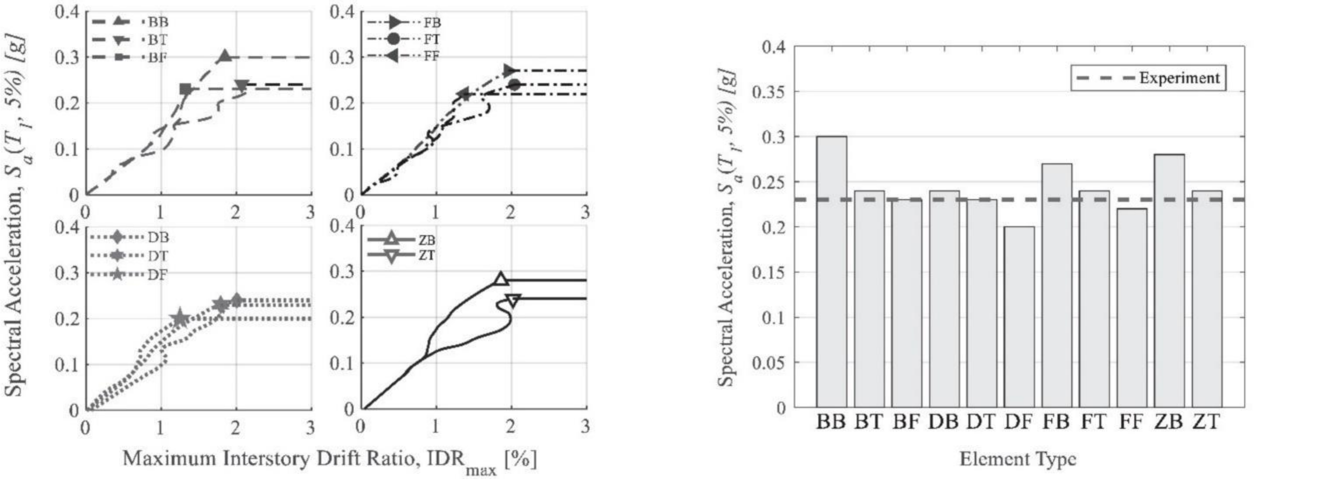
Because of the strong-beam, weak-column design of the structure, beam hinging is not detected in the FE analyses results. The pushover analyses were repeated using each of the considered 11 modeling approaches. The highest ultimate displacement capacity (2.25%) is obtained using the DB model while the smallest capacity (1.18%) is obtained using the FF model. This difference clearly indicates the sensitivity of estimated ultimate displacement capacity to the adopted element formulation and section hysteresis model. Further details about the resulting ultimate displacement capacities are presented by Ghaemian (2017).

Since the test building was subjected to two levels (i.e., 0.15 [g] and 0.2 [g]) of excitation, responses of the FE models were simulated under these two excitation levels, as well. Rayleigh’s damping model with an assumed viscous damping of 2% and the Newmark time integration scheme was utilized in the analyses.

Displacement traces obtained for the first and second tests using the BT model are presented in Fig. 5 In overall, the simulated displacement traces are in agreement with those that were measured during the tests for modeling approaches. It should be pointed out that the divergence between experimental observation and the analytical prediction within the last seconds of the tests is due to the lack of accuracy of the considered models in capturing the actual response of the building after significant yielding.

It is seen that the errors associated with the simulated maximum drift ratios range between 0.8% (FT) and 49.1% (DB). Essentially, detailed calibration of the specific properties of each FE model can lead to reductions in these errors. However, the main objective of this study is not calibration of the models for an individual test. Instead, the main objective here is to identify the range of errors that are associated with the commonly utilized modeling approaches without performing any calibration. In the past, there has were other studies (e.g., Jeong and Elnashai 2005) which focused on calibration of a specific modeling approach for predicting the response of the ELSA frame.

Evaluation of collapse safety is the key challenge in seismic risk assessment. Several alternative studies have been done for the evaluation of capacity of structures



(a) (b)

Fig. 6 (a) Incremental dynamic analyses curves and (b) collapse capacities obtained using the considered modeling approaches

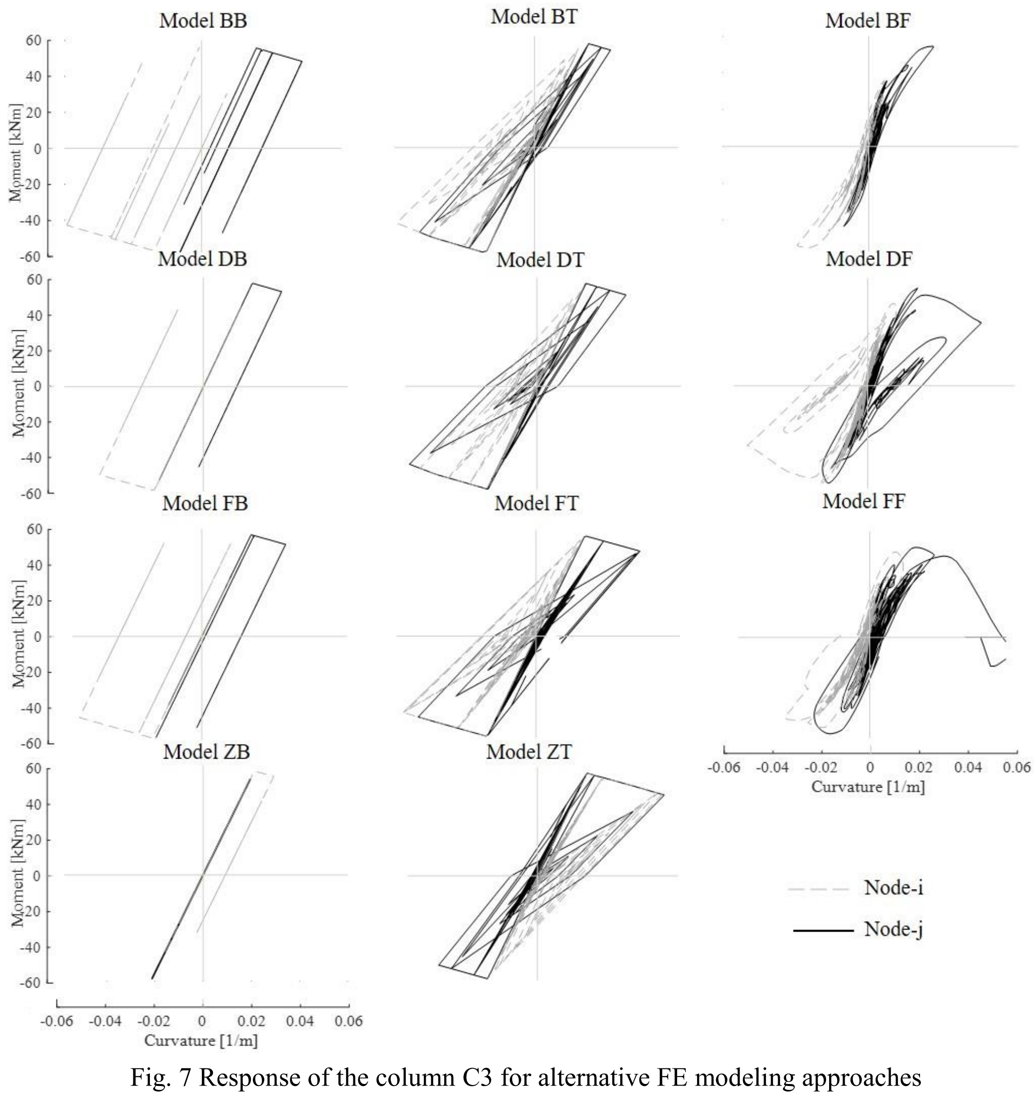


Fig. 7 Response of the column C3 for alternative FE modeling approaches

against collapse (Manie *et al.* 2015, Xia *et al.* 2016). Incremental dynamic analysis (Vamvatsikos and Cornell 2002) method is utilized in this study for predicting the collapse capacity of the test building.

For all modeling approaches, the DM based criterion is found to be the controlling criterion for the assessment of the onset of the collapse. It is seen in Fig. 6(a) that the collapse capacities estimated for the same building can vary depending on the specific modeling approach utilized in the analysis. In order to understand the reasons behind these differences, the set of plastic hinges that are simulated to deform beyond their ultimate limits were identified for each modeling approach. Since the building has a strong beam-weak column system (i.e., typical of seismically deficient buildings), in all simulations column hinges were found to reach their ultimate capacities before the beam hinges.

For each modeling approach, the specific sets of column plastic hinges that reach the ultimate capacity during the collapse, are presented in Table 3. The results indicate that both the utilized element formulation and the hysteresis model have considerable impact on the simulated collapse mechanism.

In Fig. 6(a), IDA curves for a set of alternative modeling approaches are presented. Here, X-axis represents the maximum inter-story drift ratio, and Y-axis is the spectral acceleration level which considered at each step of IDA. The collapse points obtained using the considered models of the test building are indicated with markers on the curves.

In the IDA method, collapse capacity of the structure is measured in terms of peak spectral acceleration at the fundamental period of vibration of the structure. Fig. 6(b) illustrates the resulting collapse capacities obtained using the considered FE modeling approaches. Spectral acceleration, Sa (T1) demand for the model building was equal to 0.23 [g] during the second test. This level is indicated in the Fig. 6(b) as a reference threshold. Since it is known that the test building had not collapsed during the test, actual collapse capacity of the building should be higher than this limit for the considered excitation.

For all modeling strategies, the collapse point is above this limit (i.e., Sa (T1) = 0.23[g]) except for the DF and FF models. The bilinear hysteresis model results in the highest collapse point among all the models. This can be associated with the fact that stiffness degradation is not considered in the bilinear model, unlike the other models.

Simulated responses of column C3 which was observed to sustain heavy damage during the actual test, are represented in Fig. 7. Each figure corresponds to a different modeling approach. It can be seen that –unlike the fiber-section model- bilinear and Takeda hysteresis models result in larger hysteresis cycles that lead to significant energy dissipation during large displacement cycles. On the other hand, the bilinear model unlike the fiber-section and the Takeda results in no energy dissipation during small-displacement cycles. These differences in energy dissipation characteristics and stiffness degradation results in differences being exhibited in the simulated response history, as well as the collapse capacity.

6. Conclusions

The effect of major finite modeling assumptions on collapse capacity evaluation of an RC frame building is presented and discussed in this paper. Discussion is based on comparisons between the experimental test result and the analytically predicted collapse capacity. Based on the investigations, the following results are found:

- The predicted global responses of the model RC building obtained using a set of 11 alternative FE modeling approaches, are in good correlation with the experimental results. Based on this premise, the constructed analytical models are assumed to be suitable for the evaluation of collapse capacity.
- Results obtained by assuming different finite-element formulations (i.e. force-based, displacement- based, etc.) lead to differences in the simulated nonlinear response.
- The adopted hysteretic rules have a more explicit influence on the predicted collapse capacities than the element formulations.
- When the stiffness degradation is considered in the analytical model, the resulting peak drift ratios are observed to be more accurate compared to those obtained using non-degrading behavior.
- When the element discretization is properly established and a stiffness-degrading model is utilized, the predicted collapse capacity is observed to be insensitive to the utilization of lumped or distributed plasticity formulations.
- Collapse capacities predicted using bilinear hysteresis models were found to be up to 16% higher than those predicted using the fiber section model.
- Due to the moment-axial force interaction in fiber sections, the predicted collapse capacities obtained using fiber-section models are up to 4% to 50% lower than that obtained using other section models.
- It was observed that the simulated failure mechanism (i.e. failed plastic hinges) at the onset of collapse, is sensitive to both the adopted element formulation and the hysteresis model.
- Column responses simulated using alternative modeling approaches indicated that during large displacement cycles bilinear and Takeda hysteresis models lead to greater hysteresis loops being simulated and more energy being dissipated. On the other hand, during smaller deformation cycles bilinear model does not result in any energy dissipation whereas noteworthy hysteresis loops are simulated using Takeda and fiber-sections during such cycles.

Results of this study, clearly indicates that collapse capacities obtained by means of FE analyses can be sensitive to the adopted modeling strategy. This sensitivity should be taken into account while establishing FE models of RC building for the purpose of assessing their capacity against collapse.

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