Alexandria Engineering Journal (2012) **51**, 53–60

Alexandria University

Alexandria Engineering Journal

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A new beam-column model for seismic analysis of RC frames – Part II: Model Verification

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Received 3 April 2010; accepted 4 December 2010
Available online 29 August 2012

KEYWORDS

Beam-column models;
Reinforced concrete;
Seismic analysis;
Inelastic analysis;
Frame structures

Abstract In this investigation, the performance of the simplified Flexibility-Based Fiber Model (FBFM), proposed in Part I of this study, is evaluated. The proposed model relies on calculating the inelastic lengths at the ends of the Reinforced Concrete (RC) beam-column member in every load increment and using preset flexibility distribution functions along the inelastic lengths to integrate the overall element response. The model eliminates the need for monitoring the responses of many segments distributed along the member length which results in a significant reduction in computations.

The model performance is evaluated in this study on a one element level of a beam-column element and on a structure level of a 3-story frame. The selected structures are subjected to static push-over, static cyclic and earthquake loading conditions. The results of the proposed model are compared with the outcomes of the conventional FBFMs. The comparison is achieved using global performance parameters such as the maximum drift ratios and local performance parameters such as the maximum strains in steel and concrete at the plastic hinge regions. The analysis conducted indicates that the proposed model is capable of describing with satisfactory accuracy and computational efficiency the response of RC frame structures.

1. Introduction

A new RC beam-column model is developed by the authors in part I of this study (this issue) for seismic analysis of RC frame structures. The model is a simplified version of the FBFMs, which rely on dividing the element length into small segments and dividing the cross section of each segment into concrete and steel fibers.

The proposed model is simpler than the FBFMs as it does not require monitoring the responses of many segments along the element length, which results in a significant reduction in

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Peer review under responsibility of Faculty of Engineering, Alexandria University.

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http://dx.doi.org/10.1016/j.aej.2010.12.003

computations. In the proposed model, only the two end sections are subdivided into fibers and uniaxial material models that consider the various behavioral characteristics of steel and concrete under cyclic loading conditions are assigned for the cross section fibers. The end section flexibility coefficients are obtained by integrating the fiber responses.

The inelastic lengths at the ends of the proposed model are divided into two inelastic zones; cracking and yielding. The inelastic lengths vary according to the loading history and are calculated at each load increment. The overall response of the member is estimated using a preset flexibility distribution functions along the element length. A flexibility factor η is utilized to facilitate selecting the proper flexibility distribution shape. The proposed beam-column model is implemented into the general purpose computer program DRAIN-2DX [1] which was developed for inelastic cyclic analysis of structures.

The proposed model has the same advantages of the conventional FBFMs which include; (a) accounting rationally for the axial–flexural interaction, (b) providing the strains of the fibers as an output during seismic response which can be used for seismic damage evaluation of RC members, and (c) accounting for the spread of plasticity both over the cross sections and along the member length.

In this paper, the performance of the proposed model is evaluated on the one-element and on the structure levels. The analyzed structures included a cantilever beam and a 3-story frame. The structures are subjected to static pushover, static cyclic and earthquake loading conditions. The global and the local performance parameters of the analyzed structures, obtained using the proposed model, are compared with the corresponding performance parameters obtained using the well known FBFM that has been presented by Taucer et al. [2]. Global performance parameters considered in this study include the maximum drift ratios, while local performance parameters include the maximum strains in steel and concrete at the plastic hinge regions of the various beam-column members.

2. The selected RC structures and model parameters

Two structures are considered in this study for evaluating the proposed model [3]. The first is the cantilever beam shown in Fig. 1a, which is considered for evaluating the model on the one-element level. The cantilever beam is assumed to be loaded with lateral and axial compressive loads at the free end and to have a rectangular cross section that is 25 cm wide and 60 cm deep. The cantilever beam is reinforced with four-longitudinal bars of 16 mm diameter at each of the top and the bottom sides and laterally with 8 mm stirrups at 125 mm spacing.

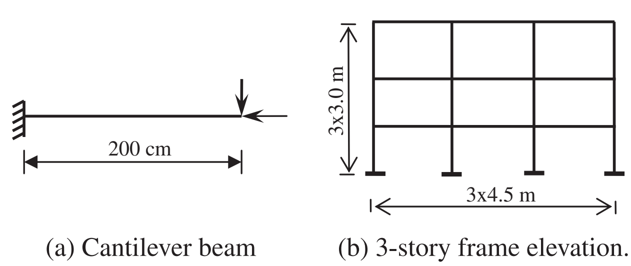


Figure 1 Two selected structures.

The second structure selected in this study is the 3-story frame shown in Fig. 1b. The frame has a 3.0 m story height and has three equal bays each 4.5 m wide. The frame members have 30 cm wide rectangular sections. The beam depth is 50 cm, while the exterior and the interior column depths are 30 and 40 cm, respectively. Fig. 2 shows the frame reinforcement-details with all beams and columns having the same reinforcement details given in the figure.

Gravity loads acting on the frame are considered in the analysis and are equal to 294.3 kN/floor. The masses of the tributary floor areas are assumed to be lumped at the beam column joints and are equal to 5 ton at each exterior joint and 10 ton at each interior joint.

In the two structures considered in this study, the longitudinal steel is assumed to have initial modulus of elasticity of 200,000 MPa, a strain hardening ratio of 3.0%, and a yield strength of 360.0 MPa. The lateral steel is assumed to have a yield strength of 240.0 MPa. The concrete strength is assumed equal to 30.0 MPa and the concrete cover is considered equal to 2.5 cm.

The FBFM presented by Taucer et al. [2] is taken as a reference in evaluating the results of the proposed model. The comparison between the proposed and the reference models includes the accuracy in predicting the local and the global performance parameters as well as the computer analysis time and storage requirements. The prediction error of the proposed model is measured as the absolute difference between the predictions of the proposed and the reference models and is expressed as a percentage from the prediction of the reference model.

It should be noted that the original model of Taucer et al. [2] is based on neglecting the Concrete Tensile Strength (CTS). However, for the purpose of facilitating the comparison process between the proposed and the reference models, the material model presented in part I of this study which considers the CTS is also assigned for the Taucer model.

A sensitivity study is conducted to determine the appropriate number of longitudinal segments and cross section fibers to be used in the analysis of the Taucer model. The results obtained indicated that a number of 10 segments at each end of the beam-column member and 10 concrete fibers at each section are expected to yield reliable results.

The flexibility distribution factor η presented in Part I of this study [this issue] is utilized to facilitate selecting the proper flexibility distribution shapes in the inelastic zones of the proposed model. A value of 0.8 is assigned to η for analyzing the examples of this paper. The selection of this value is based on trail and error to obtain good results of the proposed model.

3. Lateral response of the cantilever beam

The cantilever beam is subjected to a monotonically increasing lateral load at the free end. A displacement controlled analysis is conducted until the cantilever free end reaches a 2% lateral drift ratio. Fig. 3a shows the load–displacement relationships obtained using the proposed and the Taucer model in case of considering the CTS.

The results presented in Fig. 3a indicate a good agreement between the proposed and the Taucer models. The error of the

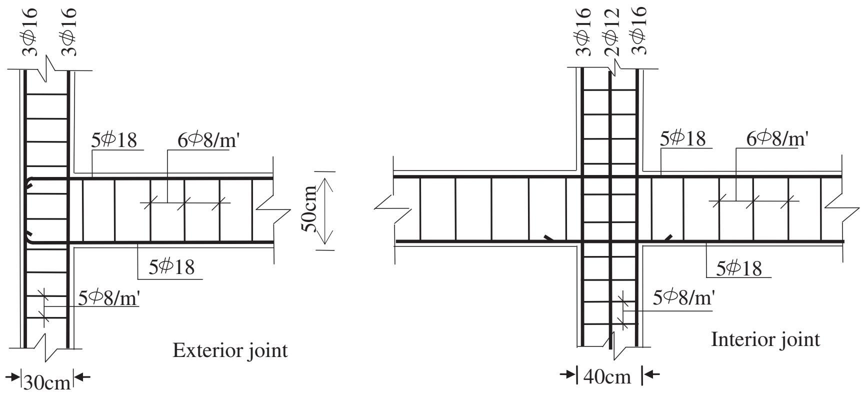


Figure 2 Steel reinforcement details of the 3-story frame.

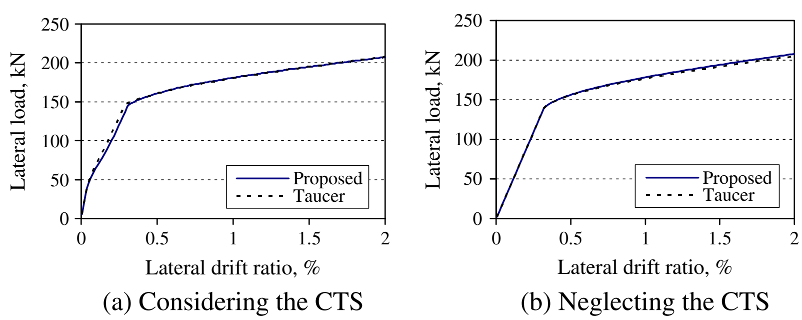


Figure 3 Lateral load versus lateral drift ratio of the cantilever beam.

proposed model in predicting the lateral load at 2% lateral drift ratio is equal to 0.11%. The results of the proposed model indicate that the lateral loads at the cracking and yielding points are equal to 27.18 kN and 142.67 kN, respectively.

On the local level, the maximum strains and stresses of the tensile steel and of the compressed concrete are also obtained. Table 1 summarizes the strain and stress values of the proposed and the Taucer models. The results presented in the table indicate that the maximum error of the proposed model in predicting the local results is about 5.3%.

The cracking and yielding lengths at the inelastic zones can also be obtained at any analysis step from the output data of the proposed model. At the end of the analysis, the cracking and yielding lengths were found equal to 111.4 and 62.4 cm, respectively.

Several models including Taucer et al. [2] and Chung et al. [4] neglected the CTS for simplicity and because it has a small effect on the frame seismic response. The solution of the cantilever beam is repeated with ignoring the CTS. The global analysis results are shown in Fig. 3b, while the local strain predictions for the tensile steel and the extreme fiber of the compressed concrete are presented in Table 1. These results indicate good agreement between the responses of the proposed and the Taucer models in case of neglecting the CTS. The error of the proposed model in predicting the lateral load at 2% drift ratio is equal to 1.6%.

The results shown in Fig. 3 indicate that the CTS affects the initial stiffness of the cantilever beam. With the increase in the lateral loading, cracks initiate in the cross sections and the cantilever lateral stiffness becomes more close to the cracked stiffness as shown in the figure. Considering the CTS will have no effect on either the yield strength or the post-yield stiffness of the cantilever beam. Accordingly, it is a common practice to neglect the CTS in the earthquake analysis of frame structures since it only influences the response of RC sections during cycles prior to yielding as stated by Taucer et al. [2].

The solution time of the proposed model on a personal computer is found approximately equal to 23% of the corresponding time required for the Taucer model. Computer storage requirements for the proposed model are found less than

Table 1 The cantilever local response.

Local performance parameters	Considering the CTS			Neglecting the CTS		
	Proposed model	Taucer model	Error (%)	Proposed model	Taucer model	Error (%)
Steel tensile strain	0.296E−01	0.281E−01	5.34	0.278E−01	0.275E−01	1.10
Concrete compressive strain	0.273E−02	0.261E−02	4.60	0.265E−02	0.262E−02	1.15
Steel tensile stress	0.527E02	0.518E02	1.70	0.516E+02	0.514E+02	0.39
Concrete compressive stress	0.244E01	0.246E01	0.81	0.245E+01	0.246E+01	0.41

that required for the Taucer model. The number of integer and real variables required for one element of the proposed model is approximately 16% of the corresponding number required for the Taucer model.

4. Effect of the axial loads and the strain hardening ratio

The behavior of RC columns is significantly influenced by the existence of axial loads. The current model takes into account the effects of axial force on the strength and deformation responses of the RC member. The cantilever beam is subjected to both axial and lateral loads. The axial load level is assumed equal to 500 kN which represents 33% of the compressive axial capacity of the cantilever cross section. Fig. 4a shows the relationship between lateral load and lateral drift of the cantilever beam. The error of the proposed model in predicting the lateral load at the end of the analysis is equal to 3.25%. Table 2 summarizes the strain measurements at the fixed end of the cantilever. The results presented in Fig. 4a and Table 2 indicate that there is a good agreement between the proposed and the Taucer models.

The lateral response of the cantilever is obtained under the effect of various axial load levels as shown in Fig. 4b. The applied axial load levels are presented as percentages of the compressive axial capacity (P_d) which is calculated according to code requirements. The results presented in Fig. 4b shows that the axial load has a significant effect on the stiffness and strength characteristics of the RC member. The results indicate that an increase in the axial load level results in an increase in the flexural strength and stiffness of the RC member. These results agree well with the conclusions presented by Kaba and Mahin [5].

The effects of the strain hardening ratio of reinforcement steel are investigated using the proposed model. The cantilever beam is subjected to lateral loading and is analyzed several times using different levels of steel strain hardening ratios. Fig. 5a shows the analysis results which indicate that the proposed model is sensitive to the variation in the strain hardening ratio. The results shown in the figure indicate that the higher is the strain hardening ratio, the higher is the post-yield stiffness of the beam-column member.

To study the effect of steel strain hardening ratio on the length of the yielding zone of the cantilever beam, the relationship between the strain hardening ratio and the yielding length (as a ratio of the member length) is presented in Fig. 5b based on the analysis that carried out using the proposed model. The results presented in Fig. 5b indicate that the higher is the steel strain hardening ratio, the longer is the yield length of the cantilever beam. This is because the increase in the strain hardening ratio of the steel reinforcement increases the post-yield stiffness of the beam-column member when subjected to lateral loading. The increase in the post-yield stiffness increases the yield length of the beam-column member.

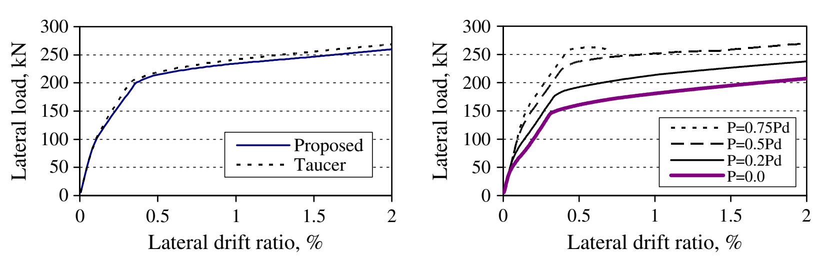
5. Cantilever cyclic response

The free end of the cantilever beam is subjected to a displacement controlled cyclic loading with displacement amplitudes of 1.0, 2.0, 3 and 4.0 cm which are equivalent to 0.5%, 1.0%, 1.5% and 2.0% lateral drift ratios. The CTS is ignored in the cyclic analysis and the axial force is assumed equal to zero. Fig. 6a represents the relationships between the lateral load and the lateral drift ratio of the free end. The displacement prediction of the proposed model is very close to the prediction of the Taucer model. For each displacement cycle, the maximum strain in the tensile steel at the fixed end of the cantilever is calculated and presented in Fig. 6b. The differences in the strain results between the proposed and the Taucer models are small. This indicates that the local behavior of the proposed model is in good agreement with that of the Taucer model in case of the static cyclic lateral loading. The solution times of the proposed and the Taucer models were computed. It was found that the analysis time of the proposed model is about 24% of the analysis time of the Taucer model.

6. Cantilever seismic response

The cantilever beam described before is subjected to the S00E component of El Centro record which has been recorded during the Imperial Valley, California earthquake of May 18, 1940. A lumped mass is assumed at the free end of the cantilever beam to produce a 1.0 s period in the lateral direction of the cantilever. The dynamic analysis of the cantilever is performed using a 3.0% viscous damping and a 0.005 s time step increment.

The earthquake records are scaled to different Peak Ground Acceleration (PGA) levels to excite the structure into the inelastic range. The maximum selected PGA level is 0.16 g and is selected to produce a maximum drift ratio of the cantilever free end close to the 2.0% level. The relationship between the maximum lateral drift ratio of the cantilever free end and the PGA of the earthquake is presented in Fig. 7a using the proposed and the Taucer models. The results presented in the figure indicate a good agreement between the two models in predicting the global seismic behavior of the cantilever beam.

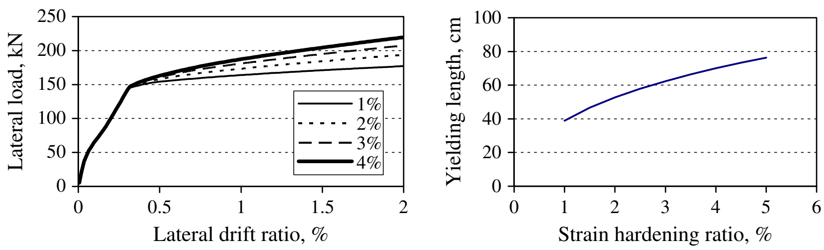


(a) 500 kN axial load level. (b) Various levels of axial load.

Figure 4 Axial load effect on the cantilever lateral response.

Table 2 Effect of axial load on the cantilever local response with considering the CTS.

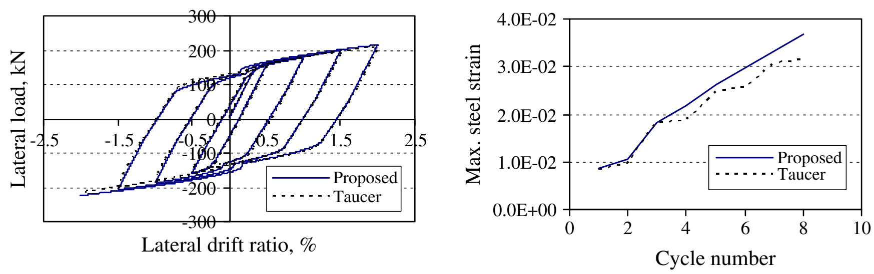
Local performance parameters	Proposed model	Taucer model	Error (%)
Steel tensile strain	0.287E−01	0.300E−01	4.33
Concrete compressive strain	0.566E−02	0.580E−02	2.41
Steel tensile stress	0.522E+02	0.529E+02	1.32
Concrete compressive stress	0.191E+01	0.189E+01	1.06



(a) Lateral load-lateral drift relationships

(b) Variation of the plastic hinge length

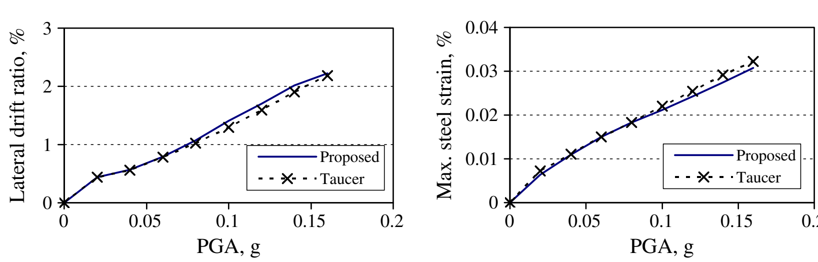
Figure 5 Effects of the steel strain-hardening-ratio.



(a) Global response

(b) Local response

Figure 6 Cantilever cyclic response.



(a) Lateral drift ratio versus the PGA

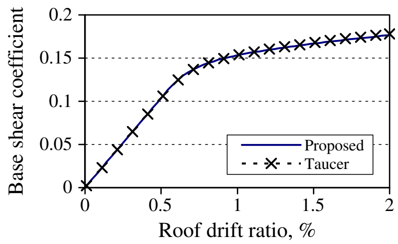
(b) Steel tensile-strain versus the PGA

Figure 7 Seismic response of the cantilever beam.

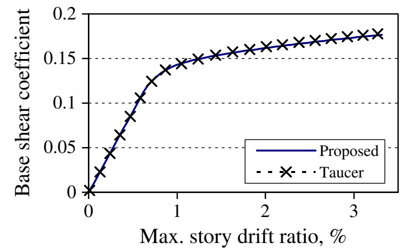
The local responses of the cantilever beam are presented in Fig. 7b. The figure shows the maximum strains of the tensile steel at the fixed-end of the cantilever beam at different PGA levels obtained using the proposed and the Taucer models. The results presented in Fig. 7b indicate that the local predictions of the proposed model are in good agreement with those of the Taucer model.

7. Frame pushover response

Static pushover analysis is a practical procedure to assess the deformability and the damage vulnerability of existing and newly designed frames without the need to perform a complex dynamic analysis. The pushover analysis provides estimates of global and local deformation demands which the structure is

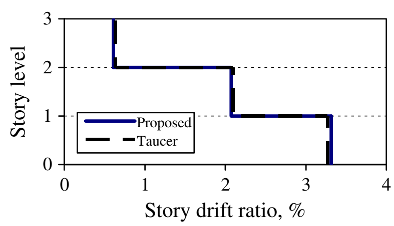


(a) Roof drift ratio versus the BSC

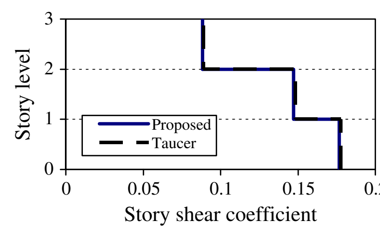


(b) Max. story drift ratio versus the BSC

Figure 8 Static pushover response of the 3-story frame.



(a) Distribution of story drift ratios



(b) Distribution of story shear coefficients

Figure 9 Distribution of story drifts and story shears along frame height.

likely to experience when subjected to an lateral loading. These estimates of deformations can be used in evaluating the integrity and the ductility characteristics of the structural system.

In this study, the static pushover analysis is carried out by applying a static lateral load having an inverted-triangular distribution along the frame height. A displacement-controlled analysis is conducted until the structure reaches a 2% roof drift ratio. The analysis is conducted with considering the gravity loads and with ignoring the CTS. The assumption of ignoring the CTS in the lateral load analysis of R.C frames is considered as these frames may be subjected to high levels of dead loads, live loads, wind loads, and possibly to minor earthquakes. The initial flexural stiffness of a cross section is expected to be closer to the cracked stiffness based on neglecting the CTS.

Fig. 8a shows the relationships between the roof drift ratio and the Base Shear Coefficient (BSC), while Fig. 8b shows the relationships between the maximum story drift ratio and the BSC. Fig. 9a shows the distribution of the story drift ratios along the frame height at the end of the analysis, while Fig. 9b shows the variation of the story shear coefficients along the height of the frame at the end of the analysis.

The analysis results shown in Figs. 8 and 9 indicate good agreement between the predictions of the proposed model and those of the Taucer model. The error in results between the two models is very small. The computer time needed for carrying out the pushover analysis when using the proposed model is equal to 16% of the corresponding time required when using the Taucer model.

8. Frame cyclic response

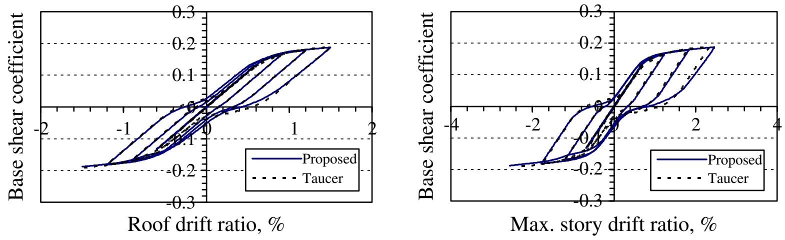
The static cyclic response of the 3-story frame is obtained by applying a static lateral load having an inverted-triangular distribution along the frame height. A displacement-controlled cyclic analysis is conducted with roof displacement amplitudes of 2.7, 5.4, 8.1, 10.8 and 13.5 cm which are equivalent to 0.3%, 0.6%, 0.9%, 1.2% and 1.5% roof drift ratios. The analysis is carried out with considering the gravity loads and with ignoring the CTS.

Fig. 10a represents the relationships between the BSC and the roof drift ratios obtained using the proposed and the Taucer models. The results presented in the figure indicate that the roof drift prediction of the proposed model is in good agreement with the prediction of the Taucer model.

The relationships between the BSC and the maximum story drift ratio of the 3-story frame are presented in Fig. 10b. The results shown in Fig. 10 indicate that the maximum story drift ratios obtained using the proposed model are in good match with those predicted by the Taucer model.

9. Frame seismic response

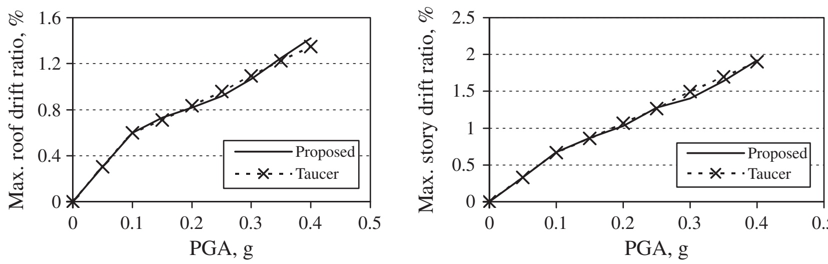
The 3-story frame is subjected to the S00E component of El Centro record which has been recorded during the Imperial Valley, California earthquake of May 18, 1940. The dynamic analysis of the 3-story frame is performed using a 3.0% viscous damping and a time step increment of 0.005 s. The earthquake record is scaled to different PGA levels to excite the structure



(a) Roof drift ratio versus the BSC

(b) Max. story drift ratio versus the BSC

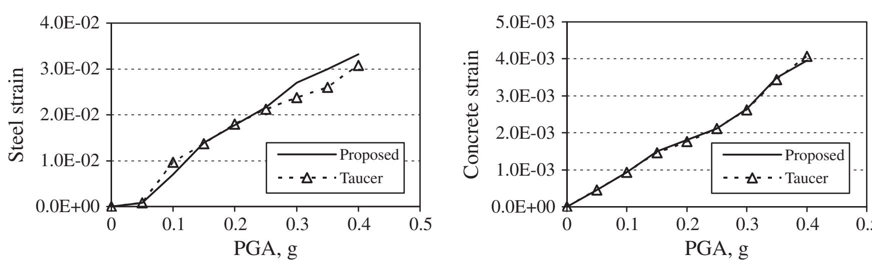
Figure 10 The static cyclic response of the 3-story frame.



(a) Roof drift ratio versus the PGA

(b) Maximum story drift ratio versus the PGA

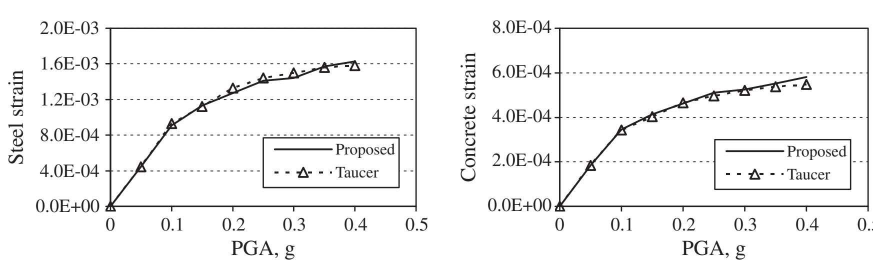
Figure 11 Global seismic response of the 3-story frame.



(a) Peak steel tensile strains

(b) Peak concrete compressive strains

Figure 12 Local seismic response of the frame columns.



(a) Peak steel tensile strains

(b) Peak concrete compressive strains

Figure 13 Local seismic response of the frame beams.

well into the inelastic range. The maximum selected PGA level is 0.4 g which produces a maximum story drift ratio higher than the 2% level.

The relationships between the envelope values of roof drift ratio and the PGA of the earthquake which are obtained using the proposed and the Taucer models are presented in Fig. 11a. The relationships between the PGA and the envelope values of the maximum story drift ratios are presented in Fig. 11b. The results presented in Fig. 11a and b indicate good agreement between the proposed and the Taucer models in predicting the global seismic behavior of the 3-story frame.

The local seismic response of the 3-story frame members is calculated using the proposed and the Taucer models. The maximum steel tensile-strain-levels and the maximum concrete compressive-strain-levels developed at the plastic hinge regions are calculated at different PGA levels. Fig. 12a and b show the maximum strains in the tensile steel and in the concrete fibers that developed in the frame columns.

Fig. 13a and b show the maximum steel tensile-strains and concrete compressive-strains that developed in the frame beams. The differences in the strain results between the two models are relatively small and indicate that the local behavior of the proposed model is in good agreement with that of the Taucer model in case of seismic loading.

10. Conclusions

The predictions of the proposed model are compared with the results of a conventional FBFM. The comparison is achieved on the one element level of a beam-column element and on the structure level of a 3-story frame under static pushover, static cyclic and earthquake loading conditions. Local and global performance parameters are used in the comparison. The analysis conducted indicates that the proposed model is capable of describing, with satisfactory accuracy and computational efficiency, the response of RC frame structures subjected to various loading conditions. Moreover, the model is capable of accounting for the axial–flexural interaction and is sensitive to the variation in the steel strain-hardening-ratio.

In the case of the 3-story frame, the solution time of the proposed model is approximately equal to 16% of the solution time of the conventional FBFM. Also the computer storage requirement for the proposed model is significantly less than that of the conventional FBFM. The number of integer and real variables required for one element of the proposed model is approximately 16% of the corresponding number of the conventional FBFM.

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