SEISMIC DESIGN AND ANALYSIS OF RECTANGULAR CONCRETE FILLED STEEL TUBE (RCFT) MEMBER AND FRAMES

C. Tort [1] and J.F. Hajjar[2]

[1] *Staff Engineer, Skidmore, Owings & Merrill LLP, Chicago, USA*
[2] *Professor, Dept. of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana, USA*
Email: cenk.tort@som.com, jfhajjar@illinois.edu

ABSTRACT :

This paper presents a computational study assessing the seismic response of rectangular concrete-filled steel tube (RCFT) members and frames to develop a reliability-based performance-based design methodology. A new analysis formulation was derived based on a mixed-finite element approach allowing geometrically and materially nonlinear analysis of RCFT frames under seismic and non-seismic loadings. The mechanism of load transfer and composite interactions between the steel tube and the concrete core were accounted for through calibrating the formulations based on a comprehensive range of experimental results in the literature. The formulation was then used to study the progression of damage within a series of RCFT frames to facilitate the development of performance-based design provisions for composite structures. Multiple earthquake records were selected representing ground motions with far-field characteristics, and the nonlinear dynamic seismic response characteristics of the frames were documented in terms of structural parameters describing the damaged state of the RCFT frames.

KEYWORDS: Composite, Concrete-Filled Steel Tube, Performance-Based Design, Mixed Formulation

1. INTRODUCTION

This paper introduces a computational study to document the seismic response of composite frames so as to provide a methodology for establishing performance-based design provisions for composite frame structures. A geometrically and materially nonlinear finite element formulation was derived to simulate the behavior of RCFT beam-columns under static and dynamic loadings as part of complete three-dimensional composite frames. A seismic demand assessment was then conducted using nonlinear time history analysis on several representative RCFT frame structures The trends of the response with respect to local and global properties of the RCFT frames were identified, and a reliability-based performance-based design methodology was established through combining the proposed computational model with empirical damage function equations developed by Tort and Hajjar (2004).

2. FINITE ELEMENT MODEL

A fiber-based finite element approach to modeling frame structures provides detailed information on their inelastic response in the context of conducting complete 2D or 3D frame analyses. The material nonlinear behavior is captured by monitoring the stress-strain response of each material fiber defined at every integration point along the element length. This allows defining and assessing performance levels ranging from localized concrete cracking to global instability of the structures. Fiber-based models can be derived following displacement-based, force-based, and mixed finite element principles. A mixed finite element method offers superior accuracy using a coarse mesh under nonlinear curvature fields and allows efficient quantification of geometric nonlinear effects and the slip response of the composite members (Alemdar and White, 2005; Tort and Hajjar, 2007).

In this research study, a 3D 18 DOF fiber-based mixed beam-column element was developed to simulate RCFT beam-columns. The additional 6 DOFs were introduced via separating translational DOFs of the steel tube and concrete core following the approach proposed by Hajjar et al. (1998). While independent translations were defined for the two media in three orthogonal directions to permit slip for an arbitrarily oriented element, differential movement was allowed only in the axial direction; the transverse (shear) translations of the steel and concrete were constrained to be the same via penalty functions. Considering the compatibility between the steel tube and the concrete core, the rotations were defined to be the same. The element stiffness matrices and internal forces were derived based on the Hellinger-Resissner variational principle, where displacement and force fields were chosen independently. Quadratic shape functions were employed to define the axial deformations, while the transverse deformations were defined via cubic-Hermitian shape functions. Linear shape functions were utilized for both axial forces and bending moments. The internal forces were obtained based on a non-iterative procedure by Alemdar and White (2005), where the unbalances between the internal and external forces were transferred and eliminated in the global Newton-Raphson iterations in an incremental-iterative nonlinear solution scheme. Details of the finite element formulation may be found in Tort and Hajjar (2007).

The stress-strain response of the material fibers were defined by comprehensive cyclic constitutive relations. Examining RCFT experiments available in the literature, the concrete material model developed by Chang and Mander (1994) was extended and calibrated with respect to the key characteristics of the RCFT members. The monotonic compression curve used in this research study follows the same behavioral trends as short RCFT column tests. A plain concrete response is assumed until peak compressive strength. A linear strength degradation response is then attained, where the rate of strength reduction increases with the slenderness of the steel tube cross-section and compressive strength of the concrete. Similarly, a constant residual stress level is also assumed after significant softening, with the value of the residual stress related to the slenderness of the steel tube, where higher stress levels are obtained for less slender cross-sections. The tensile response of the concrete core was assumed to be similar to that of plain concrete, and a robust formulation was implemented to permit repeated cycling into tension and compression. The cyclic formulations by Chang and Mander (1994) were also augmented with new rules to increase its comprehensiveness under random cyclic load excursions.

The stress-strain response of the steel tube was adopted from the uniaxial bounding surface model proposed by Mizuno et al. (1991). The model consisted of loading and bounding surfaces to trace the inelasticity. Experimental tests from the literature were also utilized to modify the model by Mizuno et al. (1991) to simulate the behavior of cold-formed steel tube of RCFT members. Studying the coupon tests, initial plastic strain values were introduced into the model to capture the effect of residual stresses in cold-formed tubes. Short RCFT column tests were also used to determine strain values at the initiation of local buckling. It was assumed that the steel tube experiences a linear strength degradation following local buckling and a constant residual stress level is preserved at high strain. In addition, the radius of the bounding surface is decreased based on the accumulated plastic work to account for the cyclic strength reduction due to local buckling.

The nonlinear cyclic slip response was also captured in the model by assigning a bilinear elasto-plastic slip vs. bond strength relation (Hajjar et al., 1998), calibrated to experimental tests of shear tab connections framing into RCFT columns. Details of the constitutive relations for the steel, concrete, and slip interface can also be found in Tort and Hajjar (2008).

The accuracy of the formulation was tested against a large number of experimental results available in the literature. For example, Figure 1 illustrates the comparison of the experimental and computational results obtained for a cyclically loaded RCFT beam-column specimen. The specimen was put into double curvature cyclically while constant axial load was being retained. The analysis was conducted with 2 finite elements and 3 integration points. The experimental and computational results were found to exhibit good correlation. Both the stiffness and strength reduction throughout the loading history were captured accurately. Figure 1 also shows the stress-strain history obtained for typical concrete and steel fibers, exhibitingthe characteristics of the RCFT members as described above.

3. SEISMIC DEMAND ASSESSMENT

Seismic demand is critical in determining the expected force and deformation levels experienced by structures so that design methods can be developed to achieve the selected performance objectives. Three-story frame structures were analyzed under a suite of earthquake records. The seismic demand was presented in terms of structural response parameters describing the behavior with a focus on the composite interaction observed in RCFT frames. The statistical measures of the analysis results are discussed. Through these examples, a general methodology for characterizing demand within RCFTs at the local and global levels is provided.

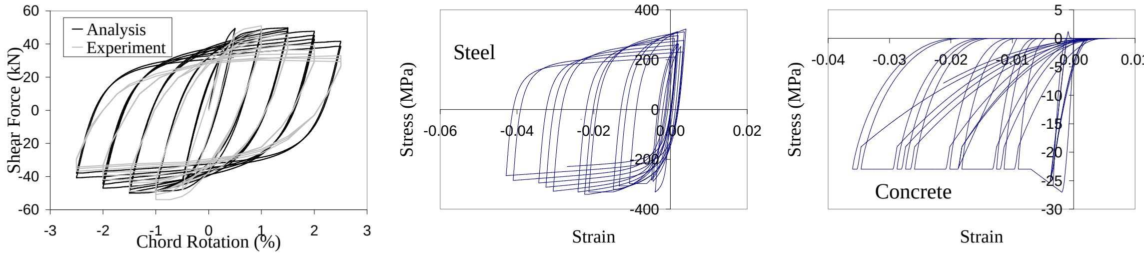

Figure 1 Comparison of computational and experimental results for a cyclically-loaded RCFT beam-column (Sakino and Tomii, 1981, Specimen CIIS3-3, Depth of steel tube (D)= Width of steel tube (B)= 100 mm, Thickness of steel tube (t)= 4.21 mm, Yield strength of steel tube (f_y)= 298 MPa, Concrete Compressive Strength (f'_c)= 27.0 MPa, Length of RCFT column (L) = 600 mm, Axial load level (P/P_o) =0.30)

3.1 Description of Theme Structures

Two three-story RCFT frame structures were utilized to investigate their seismic response. The frames were designed following the up-to-date design provisions (LaFore and Hajjar, 2005; AISC 2005). The RCFT columns of the frames were designed utilizing a wide range of material and geometric properties. The variation generated in the proportions of the steel tube and concrete core among the designed frames revealed the sensitivity of the seismic behavior against the degree and type of composite interaction. The buildings were all considered to be located in Los Angeles. The soil conditions were assumed to be on NEHRP Site Class D. Figures 2 and 3 describe the assumed properties of the designed frames.

3.2 Modeling of the Theme Structures

Both RCFT columns and steel girders were analyzed using the fiber-based distributed plasticity mixed-finite element method developed by Tort and Hajjar (2007). The RCFT columns were simulated with 1 finite element and 3 integration points along the element length. The steel girders were modeled with multiple elements per member as needed to account for the transverse point loads due to gravity from the out-of-plane beams framing into the span of the girders. The number of integration points for the beams were also kept as 3. The number material fibers along the depth of the RCFT and steel cross-sections were chosen as 8 and 10, respectively. The masses of the designed frames were assumed to be lumped at the joints. A mass and stiffness proportional damping was assumed, where the damping ratio was selected as 2% assuming that the response of RCFT frames exhibit similar damping characteristics with steel structures.

3.3 Selection of Ground Motion Records

Assessment of seismic demand requires sufficient number of acceleration time history records representing the seismic hazard at different return periods, describing intensity, frequency content, and duration in a comprehensive manner (Krawinkler et al., 2003). A suite of ordinary far-field earthquake records were selected. The dependency of structural response to near fault effect, magnitude, distance and soil interaction was not be covered so as to limit the scope of the work, since the main focus is to investigate the structural response at various hazard levels.

Medina (2002) collected a total of 40 earthquake records from the PEER strong motion database, which were all recorded on NEHRP Site Class D soil. The earthquake records used in this research study came from set provided by Medina (2002). The scaling of the earthquake records to match the design acceleration spectrum was performed based on spectral acceleration (S_a) to minimize the difference between design and median spectra for the first mode period of the structure. The records with a scale factor greater than 4 were eliminated since large scaling reduces the advantage of using real records and also weak records scaled to a higher level may produce lower demands than unscaled records (Shome et al., 1998; Bommer and Acevedo, 2004). The number of earthquake records was further reduced such that the S_a on the median spectrum attains a value close to the target S_a on the design response spectrum.

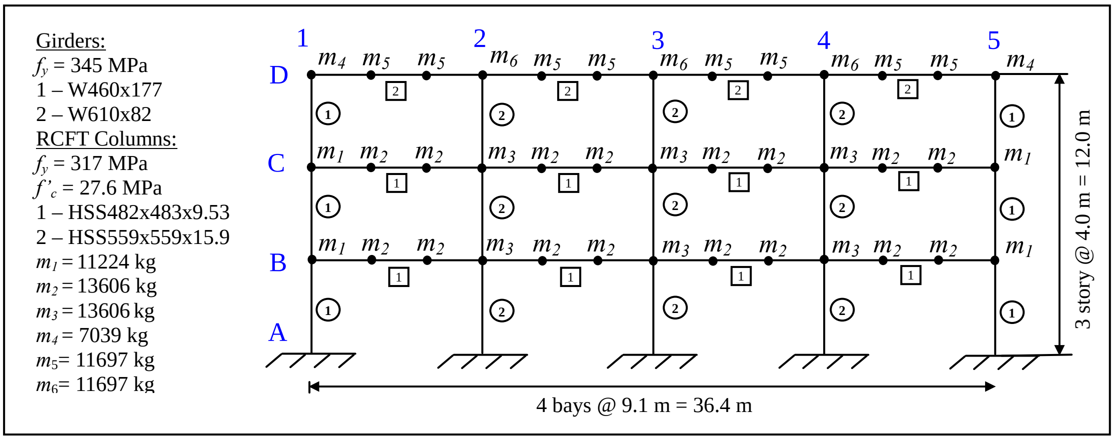

Figure 2 Description of Frame IIIa (LaFore and Hajjar, 2005)

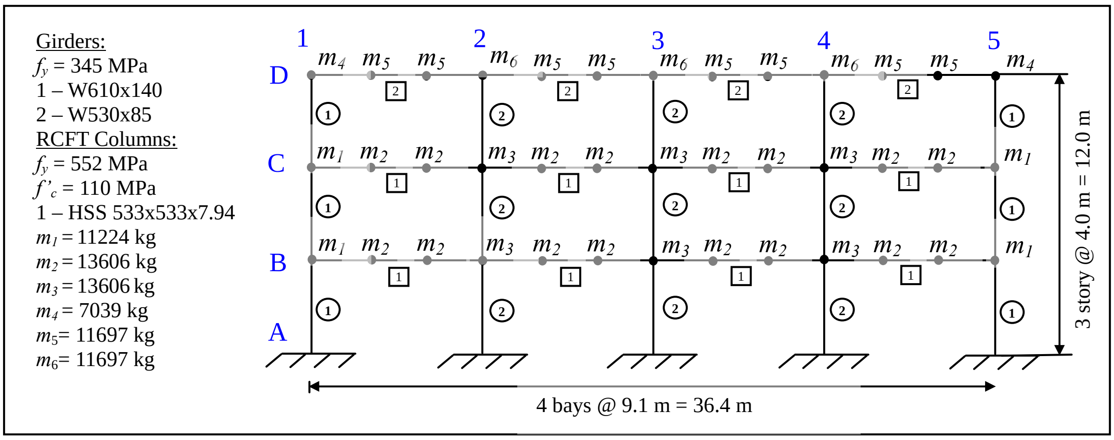

Figure 3 Description of Frame IIId (LaFore and Hajjar, 2005)

3.4 Local Performance

The local performance of the theme structures was investigated based on damage function values attained by the RCFT members and the stress-strain values of the individual steel and concrete fibers at the integration points. Tort and Hajjar (2004) proposed a deformation-based ($\hat{D}$) damage function equation to quantify the initiation of damage for specific limit states experienced by the RCFT members:

$$\hat{D} = \frac{d_{curr}}{d_o} \tag{3.1}$$

Deformation demand imposed on the RCFT member (d_{curr}) was obtained from the time history analysis results as the mean deformation experienced during the ground motions athe initiation of a specific limit state. The deformation at the peak lateral load level (d_o) was determined from the static push-over analysis of the structure, where the lateral deformation vs. moment response was produced for each RCFT column. d_o was assumed to be attained when the stiffness of the RCFT column becomes 10% of its initial value. Figures 4 and 5 show the distribution of the damage function values with their mean (μ) and standard deviation values (σ) from the earthquake records corresponding to 2%/50 hazard level for Frame IIIa and Frame IIId, respectively. It can be seen that the RCFT columns in the lower stories experience more severe damage with larger damage function demand. The extent of damage for Frame IIIa was usually lower compared to Frame IIId due to the more ductile behavior of the columns having lower strength concrete and lower *D/t* ratios. A similar trend was also observed for the 10%/50 hazard level (Tort and Hajjar, 2007). In Figures 4 and 5, the local damage distribution of the steel and concrete fibers over the RCFT cross-section located at the first story can be seen for a representative earthquake record, where the percentages of the fibers experiencing typical local damage levels are presented. It can be seen that for Frames IIIa and IIId, all the concrete and steel fibers underwent cracking and yielding damage states. Frame IIIa underwent significant concrete crushing and minor local buckling. On the other hand, Frame IIId experienced severe local buckling leading to its less ductile behavior compared to Frame IIIa.

3.5 Global Performance

Interstory drift ratio is often considered as a primary engineering demand parameter to assess the global performance of structures. Figures 6 and 7 show the interstory drift ratio vs. story shear force response of Frame IIIa and IIId, respectively. The data is presented for each story of the frames based on nonlinear time history analysis results. The analysis was conducted for multiple earthquake records corresponding 2%/50 hazard level and the results are shown for all the records, simultaneously (Tort and Hajjar, 2007). It can be seen that for both of the frame structures the 1st story RCFT columns are subjected to the largest deformation demands. The 1st story and 2nd story shear force demands were found to be similar. On the other hand, the ground motion records generated the smallest deformation and shear force demands for the 3rd story RCFT columns. The maximum interstory drift ratios were found to be 0.86% and 1.26% for Frame IIIa and IIId, respectively. From Figure 6 and 7, it can also be seen that Frame IIId experiences significant strength deterioration for some of the earthquake records due to the effect of the local buckling of the RCFT columns. On the other hand, Frame IIIa exhibits a stable response without any reduction in strength, which is attributed to the fact that the extent of local buckling and concrete crushing was not significant.

4. PERFORMANCE-BASED DESIGN METHODOLOGY

A reliability-based performance-based design methodology was developed utilizing the proposed finite element model and the capacity assessment study by Tort and Hajjar (2004). The design process initiates by selecting the targeted performance level and the corresponding hazard level. Based on the selected performance level (e.g., Immediate Occupancy, Collapse Prevention), the capacity of the RCFT members are calculated in terms of damage function values using the empirical equations given by Tort and Hajjar (2004). The load-deformation curves of the RCFT members are then constructed from static push-over analysis of the whole structure so that d_o values may be determined to normalize the demand quantities. Through conducting nonlinear time history analysis for the selected hazard level and earthquake records, the mean deformation levels at the initiation of limit states (d_{curr}) are quantified. The time history analysis results are superimposed on the static push-over response to evaluate the damage function values ($\hat{D}_{demand}$). The damage function values from nonlinear time history analysis are then compared to their capacity values from empirical equations by Tort and Hajjar (2004). This comparison allows checking the local damage states (concrete cracking, concrete crushing, steel yielding etc.) and performance levels attained by the RCFT members. The global demand of the structure may also be determined from the nonlinear time history analysis results. At the final stage, the quantified values of the demand and capacity of the RCFT structure are adjusted for uncertainty and randomness. The design is assumed to be successful if the factored values of capacity are greater than the factored values of demand and the targeted confidence level is satisfied. Tort and Hajjar (2007) derived the corresponding capacity and demand factors following the probabilistic framework of Cornell et al. (2002) developed for steel moment frame structures. Table 1 illustrates the capacity factors derived for a set of local damage states of RCFT members. It was found that local buckling has the smallest capacity factor due to the larger dispersion in the experimental data points to derive the empirical damage function equations. The global capacity factor was determined as 0.897 (Tort and Hajjar, 2007). In Table 2, the local and global demand factors are given for both Frame IIIa and IIId; Frame IIId has a larger demand factor since a larger dispersion in the nonlinear time history analysis results was obtained.

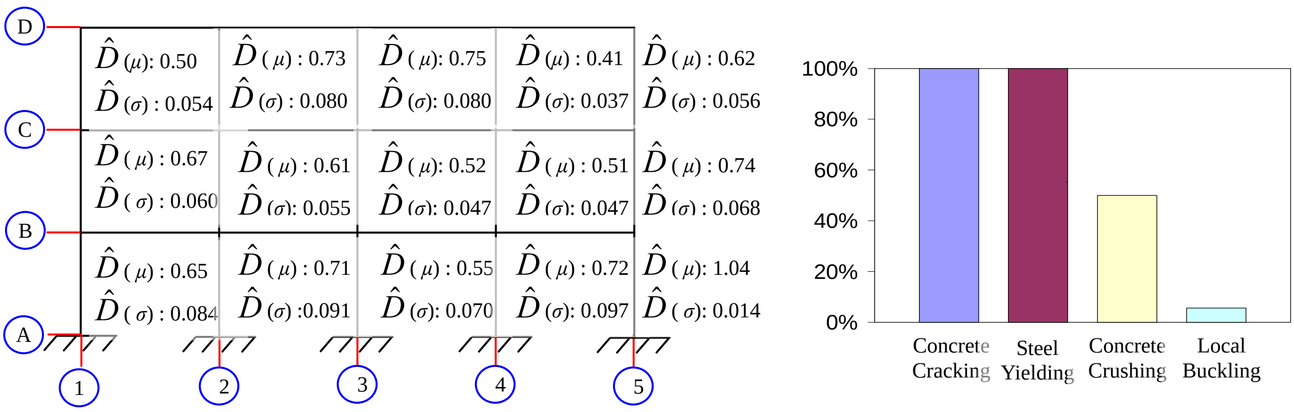

Figure 4 Damage function distribution for Frame IIIa

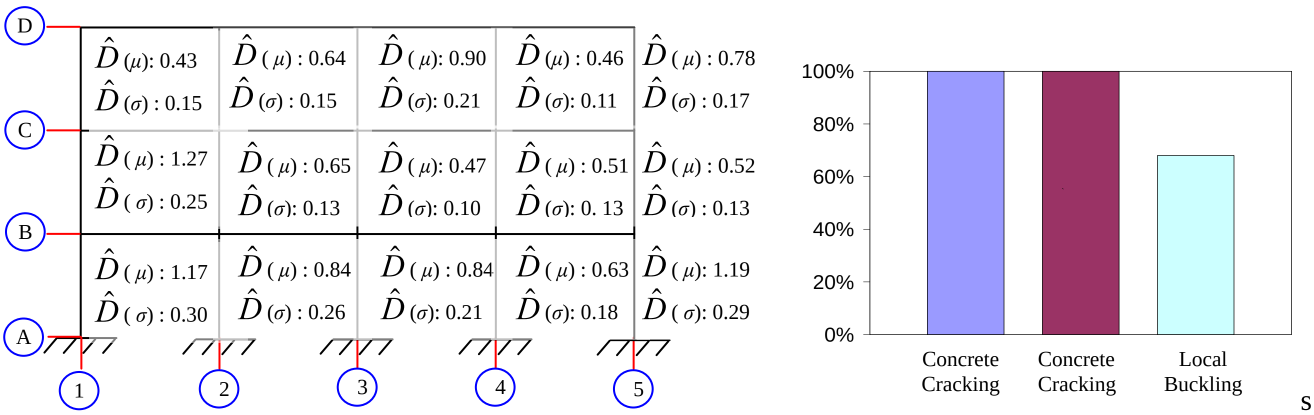

Figure 5 Damage function distribution for Frame IIId

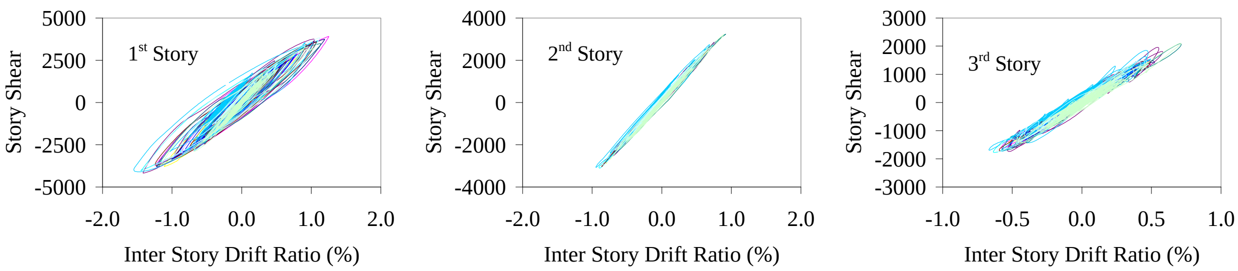

Figure 6 Interstory drift ratio and shear force demand at the 1st, 2nd and 3rd stories (Frame IIIa, 2%/50)

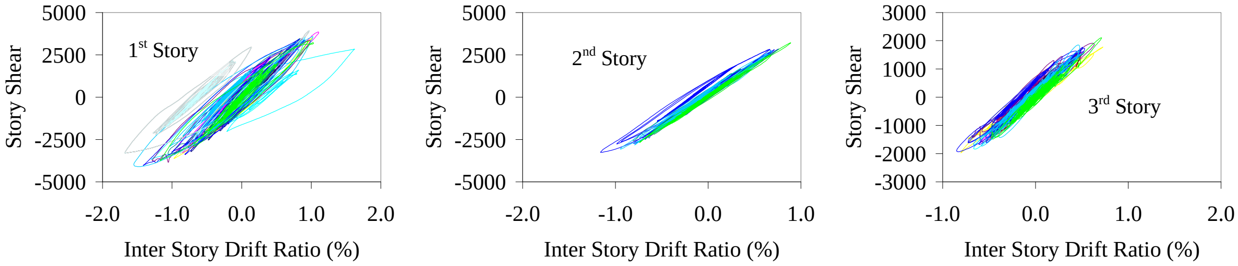

Figure 7 Interstory drift ratio and shear force demand at the 1st, 2nd and 3rd stories (Frame IIId, 2%/50)

Table 1 Summary of local capacity factors for RCFT members

Limit States	Yielding of Compression Flange	Yielding of Tension Flange	Local Buckling of Compression Flange	Local Buckling of Web
ϕ	0.61	0.82	0.45	0.79

Table 2 Summary of local and global demand factors for 3-story RCFT frame structures

Hazard Level	2%/50	10%/50
IIIa	1.03	1.01
IIId	1.08	1.15

5. CONCLUSIONS

In this research study, a new fiber-based mixed finite element model was introduced to analyze composite frames comprising RCFT beam-columns and steel girders. The proposed model and the material constitutive relations to be used with the model were found to be accurate and efficient under static and dynamic loads. A demand assessment study was performed for two representative frame structures. The uncertainty and randomness in the capacity of RCFT members from damage function equations and also those in the demand of RCFT members from nonlinear time history analysis were quantified to derive demand and capacity factors. The mixed finite element model and the damage function equations by Tort and Hajjar (2004) were then utilized to introduce a reliability-based performance design methodology.

6. ACKNOWLEDGEMENTS

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