SEISMIC ANALYSIS AND DESIGN OF COMPOSITE STRUCTURES WITH CONCRETE-FILLED STEEL TUBES

K.A. Skalomenos[1], G.D. Hatzigeorgiou[2]
D.E. Beskos[3]

1. Kyoto University, Japan
2. Hellenic Open University, Greece
3. Tongji University, China

@ 16ECEE, June 18-21, Thessaloniki, Greece

INTRODUCTION

- Composite building structures in steel and concrete under seismic loading offer significant advantages over structures either in steel or concrete separately.

- Structural steel has high strength and ductility, results in lighter structures with lower foundation demands and reduces erection time. Reinforced concrete provides high rigidity and compressive strength and is fire resistant, durable and economical.

- Composite members combining steel and concrete enjoy the advantages of both materials. One can mention here composite slabs, composite columns and innovative composite structural systems (e.g. BRB systems).

For general reviews on composite structures one can mention those of Deierlein (2000), Shanmugem and Lakshmi (2001), Leon et al. (2008) and Zhao et al. (2010).

INTRODUCTION

- Composite columns are either concrete-encased steel (CES) or concrete-filled steel tubular (CFT) ones as shown in Figure 1 below

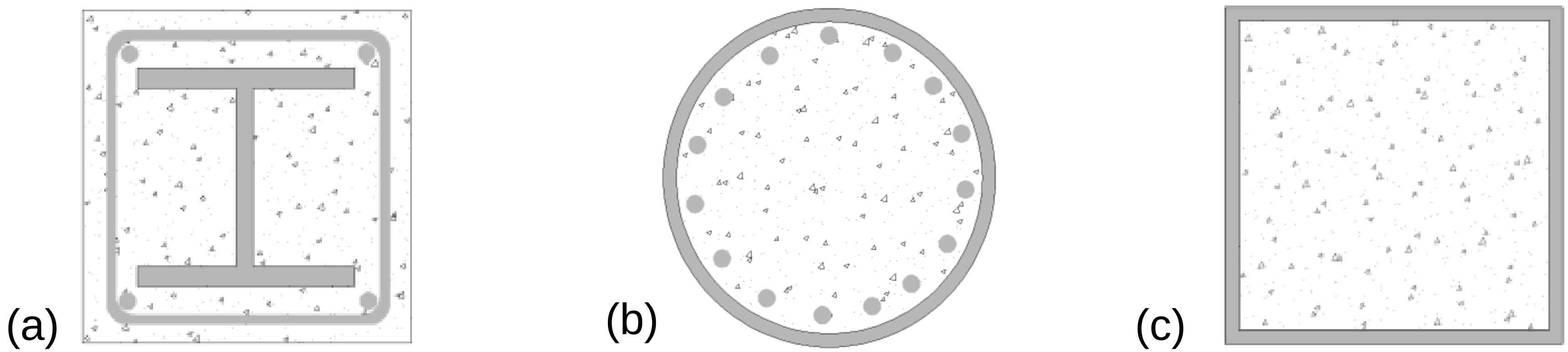

Figure 1 Configuration of composite columns: (a) Structural steel shape fully encased in reinforced concrete; (b) Concrete-filled tube with reinforcement bars; and (c) Concrete-filled tube.

INTRODUCTION

- In CES columns, one or more steel members of standard profiles are encased in concrete. Concrete restrains local and overall buckling of steel members and fire-proofs them. However, formwork and additional transverse and longitudinal reinforcement are required.

- In CFT columns, the steel tube replaces the formwork, improves stiffness, strength and ductility, and provides confinement for concrete with less or no reinforcement, while concrete restrains local buckling of tube and fireproofs it acting as a heat sink.

- The present work, provides an overview on seismic analysis and design of CFT columns and frames composed of those columns with emphasis on recent work of the authors.

CONCRETE-FILLED STEEL TUBE COLUMNS
EXPERIMENTAL STUDIES

- One can mention the early tests on CFT columns under constant axial force and varying flexural load by Furlong (1967) and Tomii et al. (1977)

- Tests on CFT columns under constant axial force and cyclic flexural load by Gourley and Hajjar (1993), Aho and Leon (1997), Zhang et al. (2009) and Perea et al. (2014)

- Current design codes (AIJ, 1997; EC4, 2004; AISC, 2010; ACI, 2011) specify width-to-thickness ratio limits and procedures for estimating elastic stiffness and axial load and moment capacities of CFT columns

CONCRETE-FILLED STEEL TUBE COLUMNS EXPERIMENTAL STUDIES

- Tests on CFT columns made of high-strength (HS) steel under cyclic load have been done by Varma et al. (2002) and Inai et al. (2004)

- Skalomenos et al (2016) at Kyoto University considered ultra-high strength (UHS) steel of 780-1000 MPa and found out that the steel tube remains elastic up to 1.5 times the larger storey drift of 2.0% than the conventional steel. Furthermore, specimens did not suffer strength deterioration until a 6.0% storey drift due to local buckling delay.

- Figure 2 shows normalized moment-storey drift response for conventional and UHS CFT columns.

CONCRETE-FILLED STEEL TUBE COLUMNS EXPERIMENTAL STUDIES

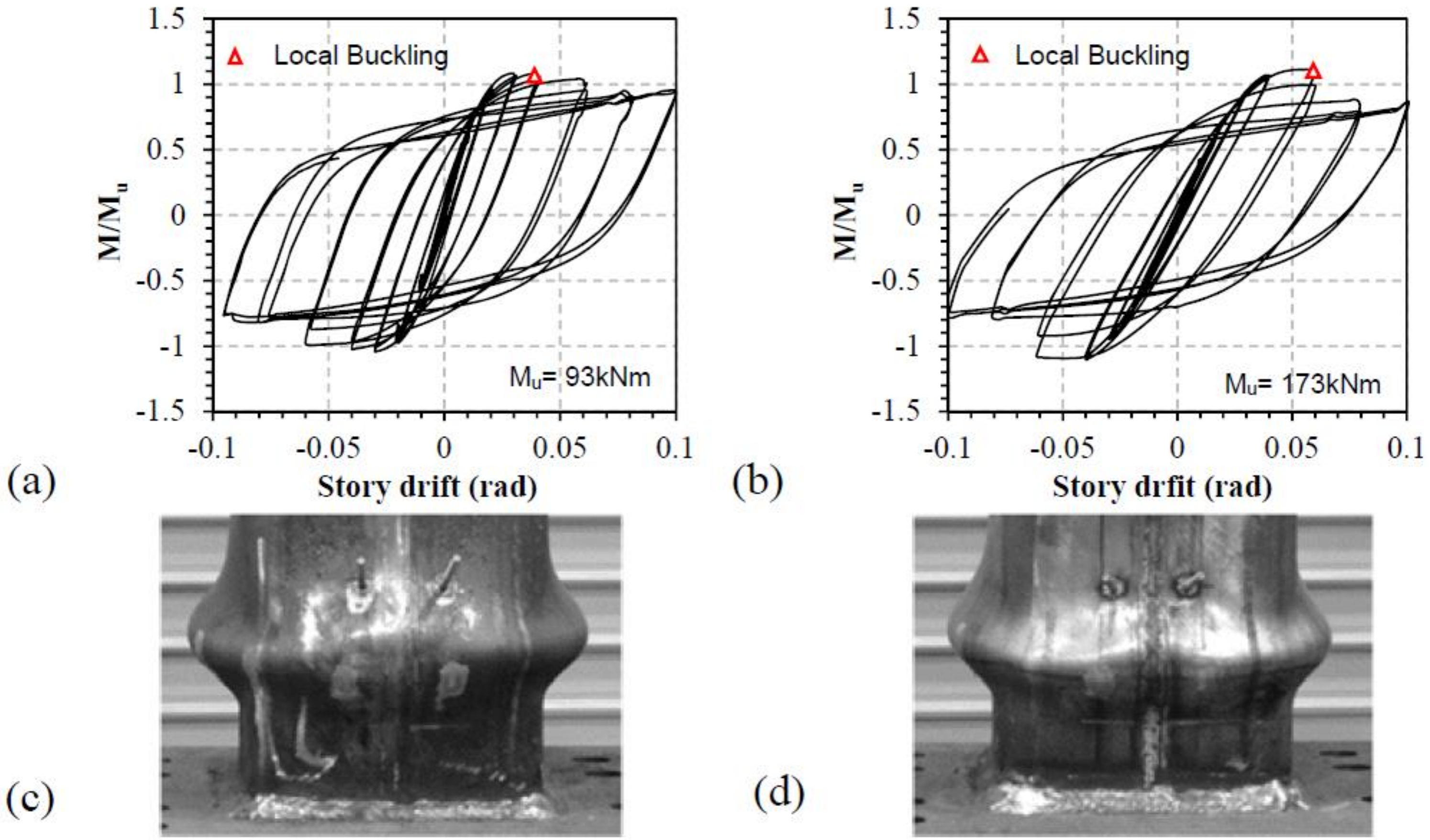

Figure 2. Normalized moment-story drift response of (a) conventional CFT; (b) HS steel CFT; and buckling failure mode of (c) conventional CFT and (d) HS CFT (Skalomenos et al., 2016)

1) **Distributed plasticity models**

They are based on beam theory in conjunction with distributed plasticity theory (fiber or layered models). Here we mention those of Hajjar and Gourley (1997), Hajjar et al. (1998) and Varma et al. (2002). The models are cyclic and account for material and geometric nonlinearities including steel-concrete interface slip, concrete strength and stiffness degradation, concrete confinement and steel local buckling. These models are highly accurate but of high computational cost. Thus, they can be practically used only for columns or simple frames but not for complex frames or extensive parametric studies.

2) **Simplified models**

- Han et al (2003) and Inai et al (2004) have developed simplified models on the basis of their tests. However, these models cannot simulate deterioration.

- Skalomenos et al (2013, 2014) and Serras et al (2016) for square and circular CFTs, respectively, have developed simplified models of concentrated plasticity type, which can take into account all the aspects of the distributed plasticity models at a lower computational cost. Thus, they can be easily used for estimating the seismic behavior of composite frames accurately and efficiently. They are described in some detail in the slides that follow.

Simplified CFT models of Skalomenos et al (2013, 2014)

These models have been constructed as follows:

i. A very detailed and accurate 3-D finite element model based on the advanced software ATENA was developed to simulate the behavior of CFT columns under constant axial load and cyclic flexural load. The finite element modeling takes into account hysteretic concrete and steel behavior with degradation, concrete-steel interface slip and local buckling of steel tube as well as concrete confinement. This model is calibrated by experimental results.

Simplified CFT models of Skalomenos et al (2013, 2014)

ii. The above 3-D finite element model is used to conduct extensive parametric studies for the creation of a response databank in terms of various geometrical, load and material parameters. From this databank explicit empirical expressions are derived through regression analysis that provide all the required parameters of 3 well known hysteretic models (Bouc-Wen, Ramberg-Osgood, Al-Bermani). Using these calibrated hysteretic models comparisons with additional numerical and experimental results are made for further adjustments.

iii. The above 3 enhanced hysteretic models are used to simulate the inelastic behavior of concentrated plasticity models using simple plastic hinges.

CONCRETE-FILLED STEEL TUBE COLUMNS NUMERICAL SIMULATION

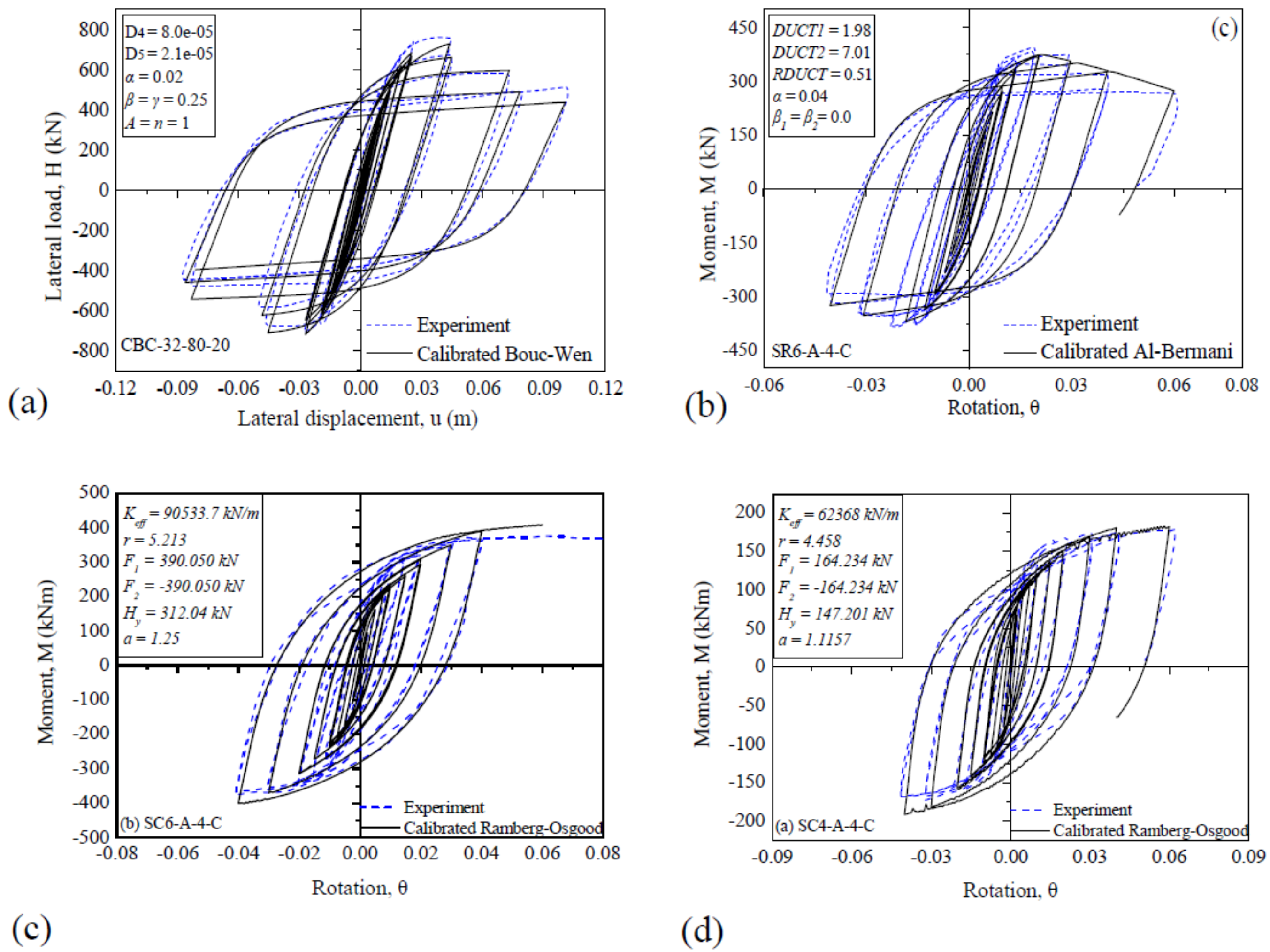

Figure 3 Proposed hysteretic models against experimental data: (a) square; (b) square; (c) circular; and (d) circular

CONCRETE-FILLED STEEL TUBE COLUMNS
IN MOMENT RESISTING FRAMES

EXPERIMENTAL STUDIES

- Kawaquchi et al (2000): 10 reduced-scale 1-storey, 1-bay CFT-MRFs (strong beams – weak columns) under vertical load and alternately repeated horizontal load.

- Chen et al (2004): pseudodynamic test of full-scale 3-storey, 3-bay CFT-MRF (with braces in middle bay)

- Herrera et al (2008): pseudodynamic test of ½ scale,4-storey with basemement, 2-bay CFT-MRF (weak beams – strong columns) under gravity and seismic loading

- Tsai et al (2008): pseudodynamic test of full scale, 3-story, 3-bay CFT-MRF under gravity and seismic loading

NUMERICAL STUDIES

- Herrera et al (2008) developed a CFT column model to determine the seismic response of their CFT-MRF test in the framework of the DRAIN-2DX software. For CFT columns they adopted the hinge-fiber element model with local buckling of Varma et al (2002). Panel zones/connections and beams were simulated with spring and zero-length fiber-based elements, respectively.

- Tort and Hajjar (2010) used their mixed finite-element formulation for 3-D nonlinear dynamic analysis of CFT columns to determine the cyclic response of a 1-story, 1-bay portal frame with rectangular CFT columns and steel girder tested by Kawaguchi (2000). The typical characteristics of the CFT members were captured by the model.

NUMERICAL STUDIES

- Skalomenos et al (2014) used their simplified CFT column model in the framework of the RUAUMOKO software to determine the seismic response of the Herrera et al (2008) CFT-MRF. The Ramberg-Osgood hinge model was used for steel beams in conjunction with Ruaumoko's degradation model and the proposed by Lignos and Krawinkler (2011) relations. Connections were modeled by two elastic-perfectly plastic springs with elastic constants K_v and K_θ from EC3 (2005). The scissors model with a rotational spring was used for the panel zone.

  Figure 5 of next page presents comparisons between numerical and experimental results. The agreement is shown to be satisfactory.

NUMERICAL STUDIES

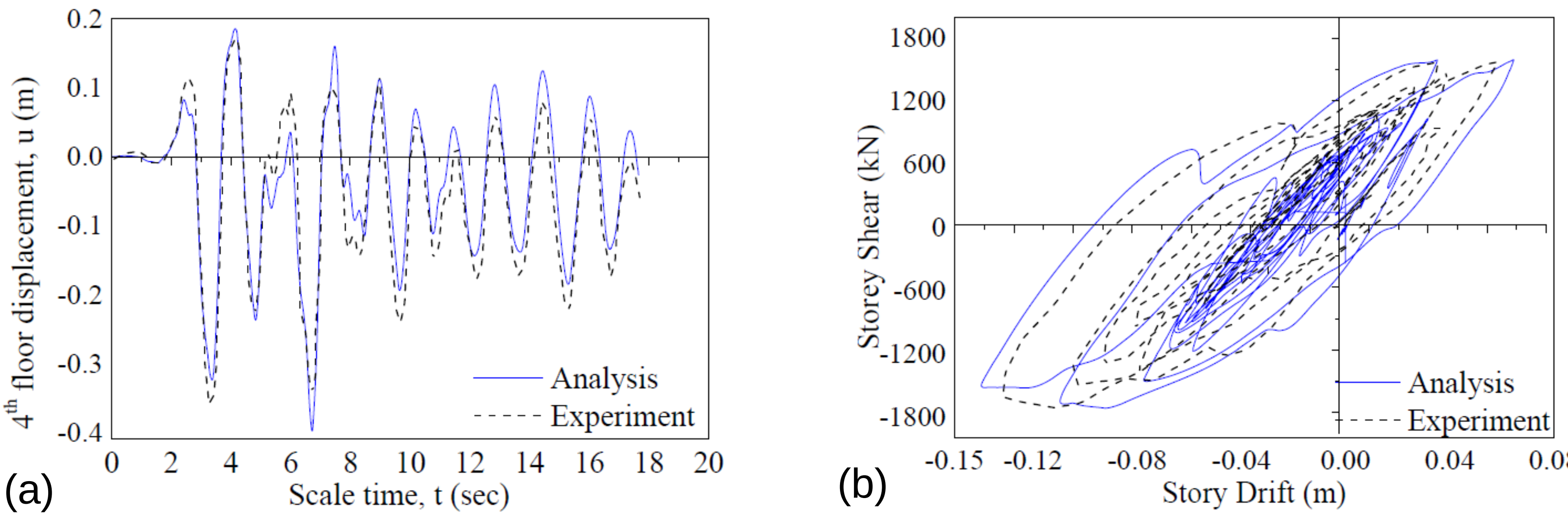

Figure 5 Comparisons of experimental and computational results: a) 4th floor displacement history; b) 1st story shear-story drift response for MCE test (Skalomenos et al, 2015).

PERFORMANCE-BASED SEISMIC DESIGN

Performance levels are defined on the basis of damage (structural and non-structural) which are associated to seismic actions defined on the basis of intensity. Thus, for the case of three performance levels, one has according to FEMA-356 the following performance-seismic action and corresponding damage limit-intensity limit values:

IO	LS	CP
FOE	DBE	MCE
0.30 (PGA_{LS})	PGA_{LS}	1.50 (PGA_{LS})
IDR = 0.7% $\mu_\theta = 2$	IDR = 2.5% $\mu_\theta = 7$	IDR = 5.0% $\mu_\theta = 9$

SEISMIC DESIGN OF CFT-MRFs

HYBRID FORCE/DISPLACEMENT (HFD) SEISMIC DESIGN METHOD

1) The HFD, originally developed for steel structures (Karavasilis et al & Tzimas et al, 2008-2013), combines the advantages of the Force-Based Design (FBD) and the Displacement-Based Design (DBD) methods and reduces or eliminates their disadvantages

2) The HFD designs in one step (strength checking) and not in two as the FBD does since deformation checking is automatically satisfied (*q* is a function of the deformation target values)

3) The HFD utilizes the familiar to engineers acceleration design spectrum and works on the original structure unlike the DBD, which employs the displacement design spectrum and works on the SDOF substitute structure

SEISMIC DESIGN OF CFT-MRFs

HYBRID FORCE/DISPLACEMENT (HFD) SEISMIC DESIGN METHOD

4) The HFD uses both structural and non-structural deformation metrics (IDR, μ_θ)

5) The HFD works in a performance-based seismic design framework with 3-5 performance levels

6) The HFD requires the availability of the 7 empirical expressions shown in the next slide

7) They are obtained by nonlinear regression analysis of a response databank created by seismically analyzing 96 plane regular CFT-MRFs under 100 ordinary (far-fault) motions

HYBRID FORCE/DISPLACEMENT (HFD) SEISMIC DESIGN METHOD

EMPIRICAL EXPRESSIONS FOR THE HFD DESIGN METHOD

$$q = 1 + 1.90 \cdot (\mu_r^{0.76} - 1) \quad (1)$$

$$\mu_d = min(\mu_{r,IDR}, \mu_{r,\theta}) \quad (2)$$

$$\mu_{r,IDR} = u_{r,IDR} / u_{r,y} \quad (3)$$

$$u_{r,y} = H^{b_1} n_s^{b2} e_s^{b3} e_c^{b4} \;, \quad e_s = 235/f_y \;, \quad e_c = 20/f_c \quad (4)$$

$$u_{r,IDR} = \beta\, H\, (IDR) \quad (5)$$

$$\beta = 1 - 0.547 \cdot \left(n_s^{0.259} - 1\right) \cdot \rho^{-0.154} \cdot \alpha^{-0.334} \cdot e_s^{-0.130} \quad (6)$$

$$\mu_{r,\theta} = 1 + 1.078 \cdot \left(\mu_\theta^{0.936} - 1\right) \cdot n_s^{-0.044} \cdot \rho^{0.144} \cdot \alpha^{0.375} \cdot e_s^{0.211} \quad (7)$$

$$\rho = \frac{\sum (I/l)_b}{\sum (I/l)_c} \;, \quad \alpha = \frac{M_{RC,1,av}}{M_{RB,av}} \quad (8)$$

HYBRID FORCE/DISPLACEMENT (HFD) SEISMIC DESIGN METHOD

BASIC STEPS OF THE HFD DESIGN METHOD

1) Definition of the basic building attributes
2) Definition of the performance level
3) Definition of input parameters (performance metrics IDR, μ_θ)
4) Estimation of input variables ($u_{r,y}$, T, α, ρ)
5) Determination of behavior factor *q* using Eq. (1) – (7)
6) Determination of seismic forces
7) Strength checking
8) Iterative process for final section selection

DESIGN EXAMPLE OF HFD SEISMIC DESIGN METHOD

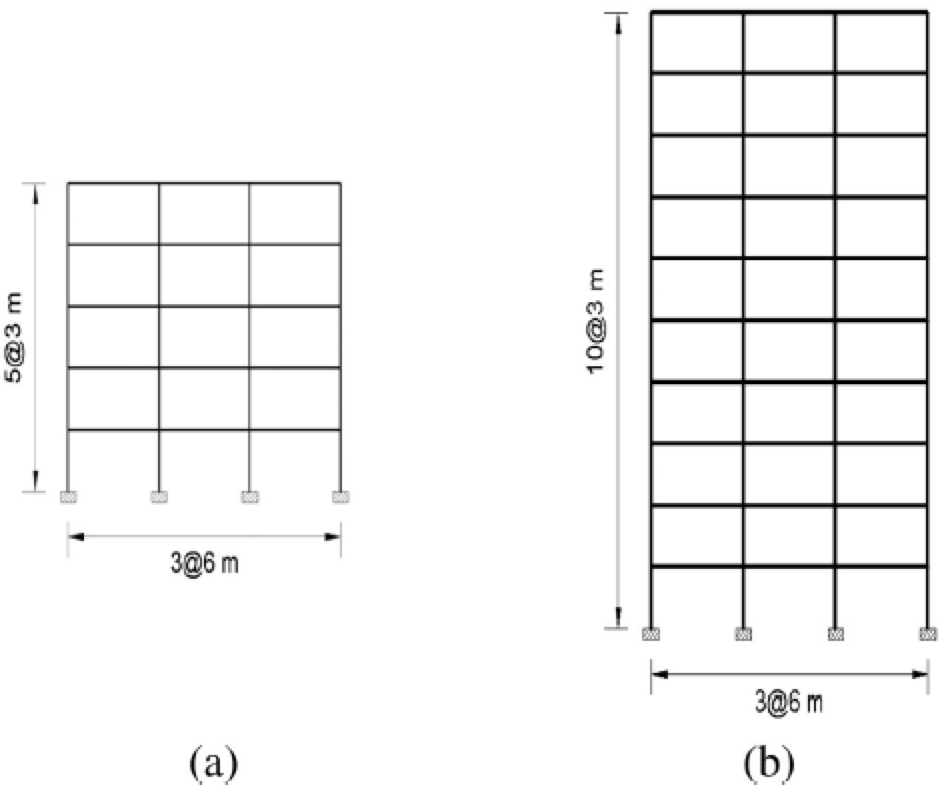

Figure 6 Composite frames (a) with five stories and (b) with ten stories.

	HFD	EC8
n_s	**Columns-Beams**	**Columns-Beams**
5	450x20-IPE400	420x20-IPE400
	420x20-IPE450	420x20-IPE450
	400x16-IPE400	400x16-IPE450
	350x16-IPE360	350x16-IPE400
	300x16-IPE360	320x16-IPE360
10	500x20-IPE450	450x20-IPE450
	450x20-IPE500	420x20-IPE500
	400x20-IPE500	400x20-IPE500
	400x20-IPE450	400x20-IPE500
	350x20-IPE450	350x20-IPE450
	350x20-IPE400	350x20-IPE450
	320x16- IPE400	320x16- IPE400
	320x16- IPE360	320x16- IPE400
	300x16- IPE360	300x16- IPE360
	300x16- IPE330	300x16- IPE330

COMPARISON WITH NONLINEAR DYNAMIC ANALYSIS AND LIMIT VALUES FOR 3 PERFORMANCE LEVELS

Table 2. 10-story CFT-MRF

	IO		LS		CP	
	TH	**EST**	**TH**	**EST**	**TH**	**EST**
IDR_{max} (%)	0.67	0.51	2.05	1.80	3.10	2.70
CHECK	< 0.7	OK	< 2.50	OK	< 5.0	OK
μ_θ	1.0	1.0	2.92	2.95	3.98	4.20
CHECK	< 2	OK	< 7.0	OK	< 9.0	OK

TH: time history (nonlinear) analysis; **EST:** estimation of HFD

DAMAGE INDEX EMPIRICAL EXPRESSION

- Consider as example the Park-Ang (1985) damage index of the form

$$D_{PA} = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE$$

δ_m = maximum deformation

δ_u = ultimate deformation under monotonic loading.

$\int dE$ = dissipated energy

Q_y = yield strength

β = constant (0.12 – 0.3)

- Utilizing the previously described response databank for CFT-MRFs one can obtain the following empirical expression for D (Kamaris et al 2016)

SEISMIC DAMAGE ASSESSMENT OF CFT-MRFs

DAMAGE INDEX EMPIRICAL EXPRESSION

$$D_{PA}^{c} = 0.24 \cdot n_s^{0.16} \cdot \alpha^{-0.30} \cdot \left(\frac{S_{\mathrm{a}}}{g}\right)^{0.39} \left(\frac{235}{f_s}\right)^{0.05}$$

for CFT column bases

$$D_{PA}^{b} = 0.31 \cdot n_s^{0.33} \cdot \alpha^{-0.12} \cdot \left(\frac{S_{\mathrm{a}}}{g}\right)^{0.43} \left(\frac{235}{f_s}\right)^{-0.08}$$

for steel beams

- Similar expressions have been also derived on the basis of three other damage indices and can be found in Kamaris et al (2016)

DAMAGE INDEX EVALUATION EXAMPLE

3 storey/3 bay CFT-MRF under DBE (PGA=0.35g, soil B)

Damage indices for column bases.

Damage Index	“Exact” Value	Approx. Value	Error (%)
D_{PA}	0.157	0.187	16.0

Damage indices for beams.

Damage Index	“Exact” Value	Approx. Value	Error (%)
D_{PA}	0.355	0.335	5.6

“Exact” refers to NLTH analysis with 8 motions spectrum compatible to the DBE.

CONCLUSIONS AND FUTURE DEVELOPMENTS

1) Ultra-high strength steel (UHS) improves the seismic performance of CFT columns by increasing their elastic deformation capacity and delaying the failure of local buckling.

2) Simple, yet accurate, plastic-hinge models were developed for square and circular CFT columns including strength and stiffness deterioration, which can be successfully used for analysis and design.

3) Development of simplified hysteretic models for other types of composite members, such as beam/columns with fully or partially concrete encased steel sections is a subject of future studies.

4) The seismic performance of a wide range of CFT-MRF structures under several levels of seismic hazard was investigated through time-history dynamic analyses.

CONCLUSIONS AND FUTURE DEVELOPMENTS

5) On the basis of the DBD and FBD methods, a new performance-based seismic design method, the Hybrid Force/Displacement (HFD) method, was developed for CFT-MRFs.

6) The HFD method utilizes larger behavior factors compared with those proposed for steel structures by current design codes thereby leading to more economical designs when using composite columns.

7) An empirical methodology for a rapid seismic damage assessment of CFT-MRFs was also developed. Next step is the combination of this methodology with probabilistic seismic hazard analysis models to assess the collapse risk of composite frames.

THANK YOU
FOR YOUR ATTENTION