ADVANCED COMPOSITE MATERIALS IN FLEXURAL MEMBERS FOR AUTO-ADAPTIVE STRUCTURAL RESPONSE MODIFCATION

Gregor Fischer
Department of Civil and Environmental Engineering
University of Hawaii, U.S.A.

Victor C. Li
Department of Civil and Environmental Engineering
University of Michigan, U.S.A.

Keywords: ECC, moment resisting frame, composite, collapse mechanism, passive response control

1 INTRODUCTION

The response of moment resisting frame structures to seismic excitation is strongly dependent on the ability of particular structural members to sustain relatively large inelastic deformations without significant degradation of lateral and axial load-carrying capacity. Conventional reinforced concrete frame structures are typically designed according to the strong column/weak beam concept, which prescribes inelastic deformations to occur exclusively in the beam members to dissipate energy while the columns remain elastic in order to maintain stability and prevent possible collapse (Fig.1a). This ideal frame deformation mechanism, enforced by a strength differential between beams and columns intersecting at joint locations, however, usually requires the formation of plastic hinges at the base of the first story columns in order to initiate frame sway and utilize the energy dissipation capacity of the beam members. The formation of plastic hinges at the column base is anticipated and not necessarily critical for the stability of the moment resisting frame, assuming that further inelastic deformations occur exclusively in the beam members. Due to axial and shear forces at the column base, the plastic hinge regions of these members must be provided with relatively large amounts of transverse reinforcement to ensure ductility under reversed cyclic loading conditions by proper confinement of the concrete core, resistance to shear and buckling of longitudinal reinforcement. Furthermore, residual deformations in structural members and in the frame system may require extensive rehabilitation efforts. Most importantly, however, the possibility of formation of additional plastic hinges in the columns above or within the first story in conjunction with plastic hinges at the column base may lead to a kinematic mechanism and collapse of the structure (Fig.1b).

The frame configuration investigated in this paper does not require the formation of plastic hinges at the column base in order to initiate frame sway and subsequent utilization of inelastic rotations in the beam plastic hinges (Fig.1c). In the suggested configuration, the formation of plastic hinges at the column base is prevented by employing advanced composite materials, in particular Fiber Reinforced Polymer (FRP) reinforcement combined with a ductile engineered cementitious composite (ECC) to substitute brittle concrete. These FRP reinforced ECC column elements have a relatively large elastic deformation capacity and sufficient flexural strength to enforce inelastic deformations in the beam members in accordance to the strong column/ weak beam concept.

Engineered cementitious composites (ECC) are a fiber-reinforced cement-based composite material micromechanically designed to achieve a tensile stress-strain behavior analogous to that of metals. Unlike the dislocation micromechanics in the plastic deformation regime of metals, the inelastic deformation behavior of ECC is based on the formation of multiple cracking while undergoing pseudo-strain hardening. This composite material utilizes randomly oriented fiber reinforcement at a moderate volume fraction ($V_f<2\%$), which are added to the cementitious matrix during the mixing process.

Utilizing the particular load-deformation characteristics of steel and FRP reinforced structural members in the suggested moment resisting frame system, a bi-linear load-deformation behavior can be obtained with considerable energy dissipation capacity and reduced residual displacements at unloading. The auto-adaptive stiffness modification is expected to reduce base shear forces during a seismic event by increasing the period of the structural system at exceeding a particular horizontal displacement.

The model response (Fig.8) of the conventional frame configuration (S-1) is outlined by the elastic stiffness and ultimate capacity as determined from plastic frame analysis. Beyond formation of a kinematic mechanism at ultimate, a perfectly plastic behavior of the conventional configuration is assumed.

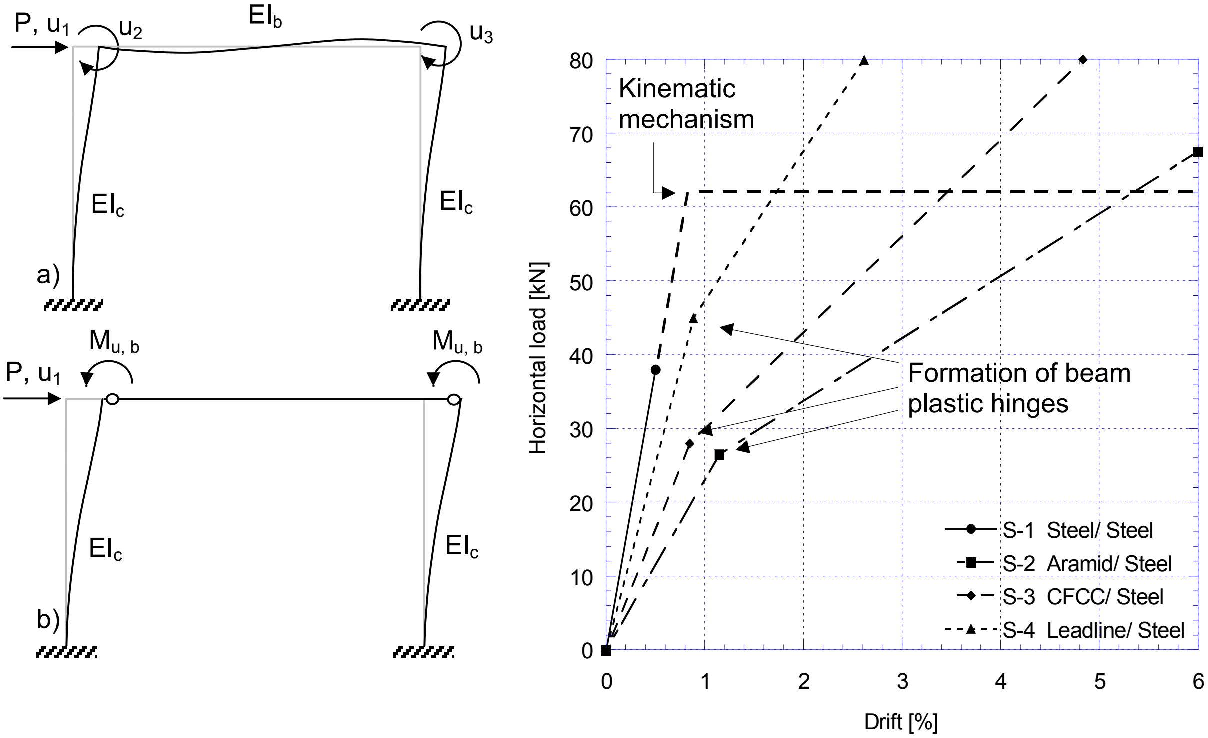

Figure 8 **Model for a) initial response and b) secondary response stage; theoretical response of tested specimens**

The theoretical response of the suggested configuration (S-2, S-3, S-4) obtained from analysis (Fig.8) shows the intended two-stage response mechanism triggered by the formation of plastic hinges in the beam member as compared to the typical elastic/plastic load-deformation behavior of specimen S-1 exclusively utilizing steel reinforcement. The spectrum of bi-linear responses obtained from the particular specimens discussed in this paper indicates the potential of this concept to design a frame structure for a specific bi-linear system behavior by employing appropriate combinations of beam and column members with respective reinforcement type and ratio.

3.3 Experimental observations

The control specimen (S-1) indicates an elastic/plastic load-deformation behavior due to the combination of steel reinforced beam and columns (Fig.9a). The onset of inelastic frame deformation becomes apparent at approximately 1% drift (40kN) as a result of the initiation of plastic hinges forming in the beam and column members. A significant reduction of frame stiffness and formation of a complete kinematic mechanism occur at approximately 2% drift (60kN), which is in reasonable agreement with predictions of the ultimate capacity of the specimen (62kN) based on the tensile stress-strain behavior of the steel reinforcement assuming yielding and strain hardening.

Specimen S-2 (Fig.9b) shows a relatively low initial stiffness compared to specimen S-1 primarily because of the lower flexural stiffness of the FRP reinforced columns, however, also because of the lower yield strength of the beam in S-2 as compared to S-1. This reduction in beam yield strength is necessary due to the lower flexural stiffness of the FRP reinforced columns in order to initiate yielding at similar frame displacements as in S-1. Prior to reaching 1% drift, a predominant formation of flexural cracking at the top and base of the columns suggests a double curvature deflection mode of the column members while the beam remains elastic. Beyond 1% drift, flexural crack formation at the ends of the beam together with a noticeable change in frame stiffness indicate yielding of the beam member and formation of plastic hinges at the beam/column intersection at continuing elastic deformations of the column members. Due to the elastic deformation behavior of the columns, further increasing lateral frame displacements require increasing lateral load, i.e the system has non-zero stiffness and a kinematic mechanism is not formed.

Specimen S-3 (Fig.9c) and S-4 (Fig.9d), both with FRP reinforced columns, similarly show a bi-linear load-deformation response, however, with different transition load and displacement at switching from initial to secondary response stage. In specimen S-3, the transition between initial and secondary response stage is more pronounced due to a larger stiffness of the FRP reinforced columns as compared to specimen S-2. The modification of frame stiffness occurs at approximately 1% drift, which coincides closely with the predicted response. Beyond this point, the specimen continues to deform in this mode at a secondary frame stiffness with inelastic rotations in the beam plastic hinges and further elastic response of the column members.

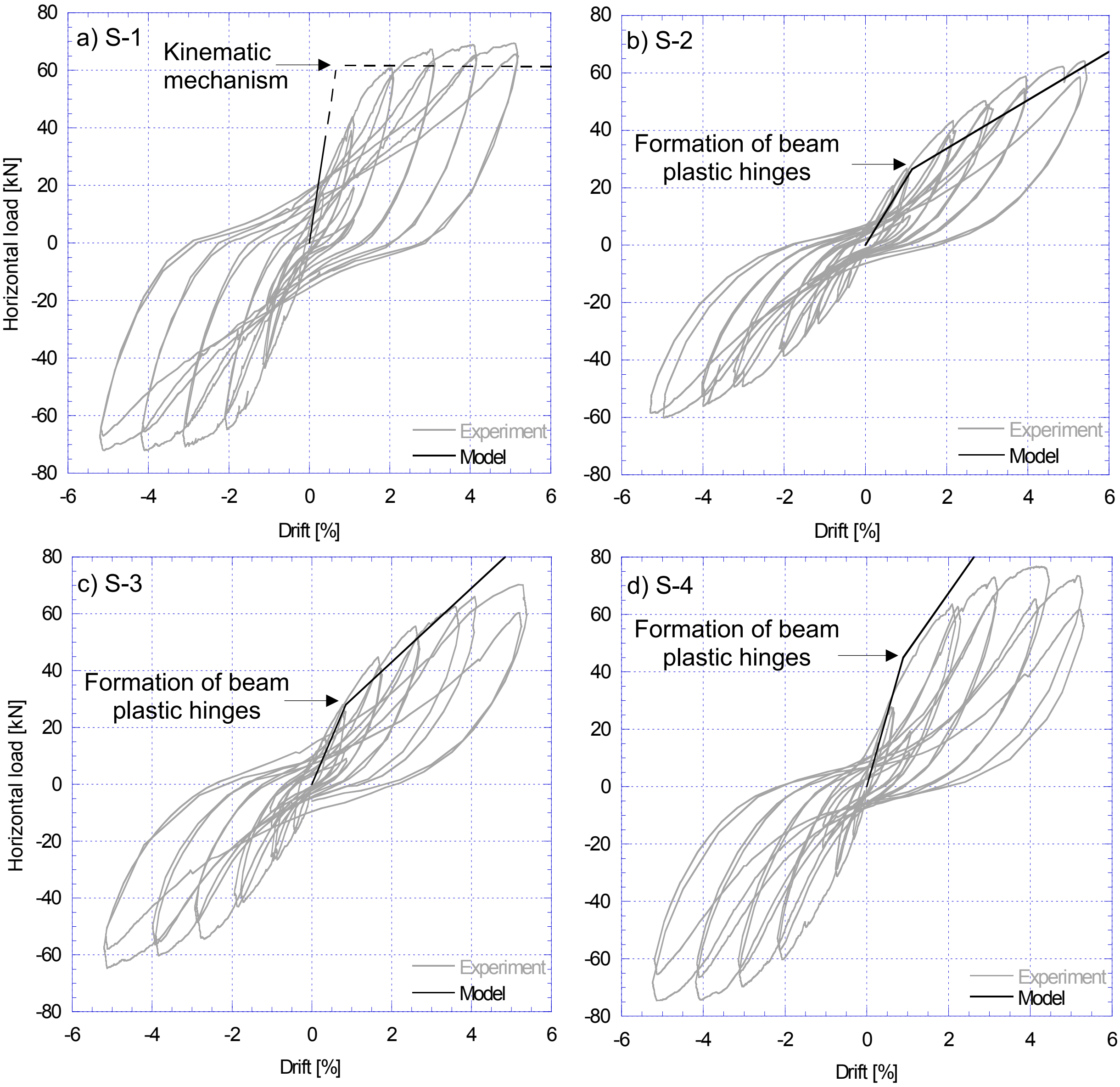

Figure 9 Load-deformation response of tested specimens

The frame stiffness in specimen S-4 is similar to that observed in specimen S-1 and considerably larger than that of specimens S-2 and S-3 due to a relatively large flexural stiffness of the FRP reinforced columns. The transition in frame stiffness occurs prior to 1% drift due to beam yielding, followed by a plateau in the load-deformation graph beyond 4% drift, which is a result of predominant shear deformations in the column members. This apparently ductile frame deformation behavior is caused by ductile shear deformation of the ECC matrix.

3.4 Residual displacements and energy dissipation

Besides the bi-linear load-deformation response and avoiding the formation of a kinematic mechanism, a reduction of residual displacements at unloading from the target drift level and ability to dissipate energy by inelastic rotations of the beam plastic hinges are intended characteristics of the suggested frame configuration.

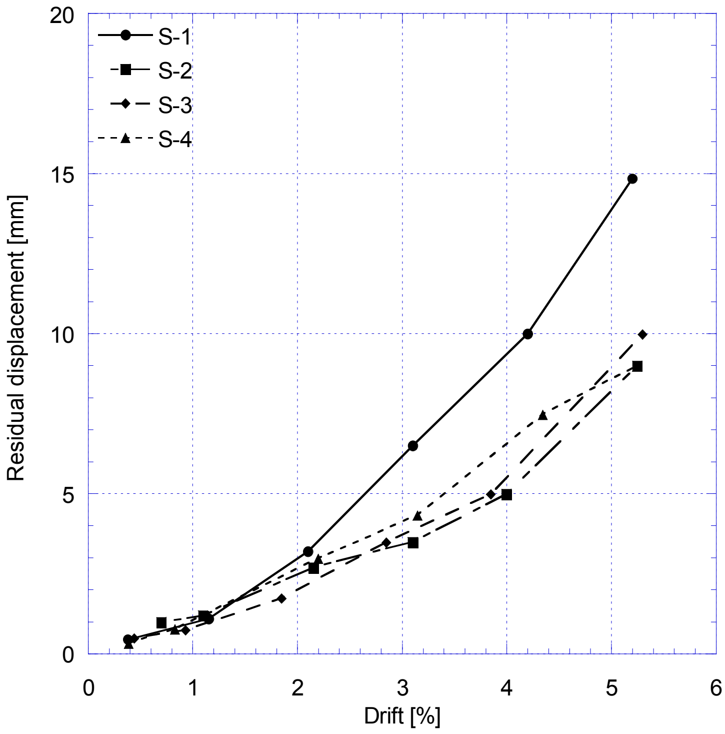

Figure 10 **Comparison of residual displacements**

The comparison of the residual displacements of the tested specimens (Fig.10) shows similar, relatively small residual displacements at lower drift levels in the elastic deformation stage. Differences between the conventional (S-1) and the suggested configurations (S-2, S-3, S-4) become apparent beyond 1% drift when inelastic deformations occur in the beam member as well as in the column members of specimen S-1. Due to the elastic deformation behavior of the columns and resulting self-centering capability of the suggested configuration, the residual displacements in S-2, S-3, and S-4 are significantly reduced as compared to S-1, where the formation of plastic hinges at the column base causes relatively large residual displacements. Residual deformations observed in specimens S-2, S-3, and S-4 are due to the formation of plastic hinges in the beam, which induces a permanent rotation at the top of the column members and consequently prevents complete elastic retraction of the frame structure at unloading.

The self-centering capability in the suggested configuration, however, also leads to a reduction in energy dissipation in the secondary response stage as compared to the conventional configuration (Fig.11). Despite the absent contribution of inelastic rotations at the column base in specimens S-2, S-3, and S-4, they show considerable energy dissipation from inelastic rotations at the beam plastic hinges as intended in the response concept. Furthermore, for practical applications of the suggested concept in multistory frames, the contribution of plastic hinge formation at the column base of the conventional frame configuration to total energy dissipation in the structure is expected less significant.

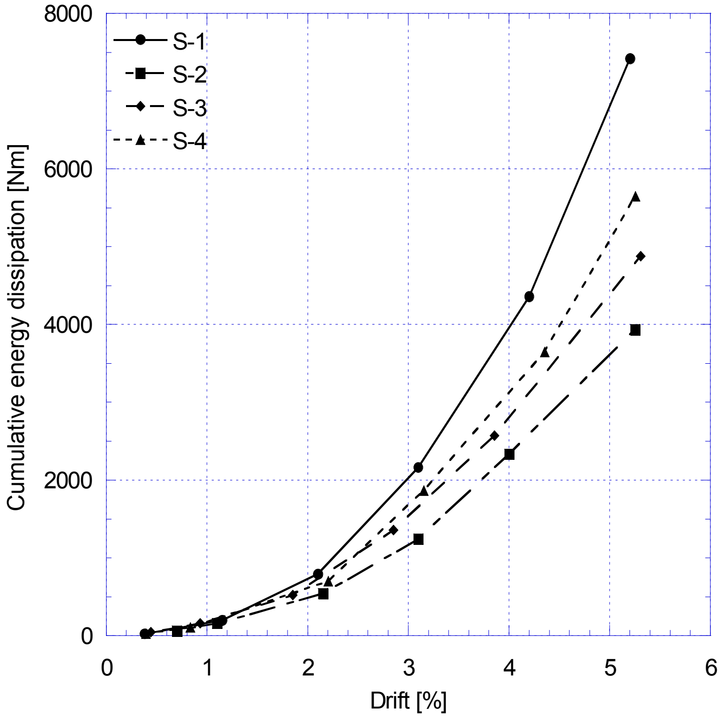

Figure 11 **Comparison of cumulative energy dissipation**

4. CONCLUSIONS

In this paper, an alternative frame deformation mechanism is conceptually outlined and experimentally verified. The interaction of composite beam and column members with particular load-deformation characteristics results in a moment resisting frame system with bi-linear load-deformation behavior and reduced residual displacements. In contrast to conventional moment resisting frames assembled exclusively from steel reinforced members, the configuration introduced in this paper does not require the formation of plastic hinges at the column bases in order to form plastic hinges in the beam member and utilize its inelastic rotation capacity for energy dissipation. Consequently, a potential collapse mechanism is not formed and increasing frame displacements require further increasing lateral load.

The bi-linear load-deformation response of the suggested frame system is defined by its initial and secondary frame stiffness. The transition between these response stages is triggered by the formation of plastic hinges in the beam member. Tests of different configurations of the suggested system presented in this paper have shown the potential of this concept to design a moment resisting frame for a specified response in terms of initial and secondary stiffness as well as transition load and displacement upon which the frame auto-adaptively modifies its response characteristics. The transition between initial and secondary frame stiffness is intended to increase the period of this structure in order to decrease the shear forces acting on the system in case of seismic excitation.

In the suggested response mechanism, inelastic deformations and energy dissipation are exclusively assigned to the beam member of the frame while the columns remain elastic particularly at the column base and do not form plastic hinges at this location. The relatively large elastic deformation capacity of the columns is achieved in this particular case by combining elastic FRP reinforcement with a ductile, engineered cementitious composite (ECC). The interaction of ductile ECC matrix and elastic FRP reinforcement results in a relatively large deflection capacity of the FRP reinforced ECC column members.

Besides the reduction of base shear forces due to an elongation of the period of the suggested frame system, the reduction of residual displacements and self-centering capabilities of the structure are expected to reduce permanent damage and the need for rehabilitation of the structure after experiencing a seismic event.

REFERENCES

ACI Committee 440 (1996), “State-of-the-Art Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures”, ACI 440R-96, American Concrete Institute, Farmington Hills, MI

Harris, H.G., Samboonsong, W, Ko, F.K. (1998), “New ductile FRP reinforcing bar for concrete structures”, ASCE Journal for Composites for Construction, Vol.2, No.1, February 1998, pp.28-37

Fischer G., Li, V.C. (2002a), “Effect of matrix ductility on deformation behavior of steel reinforced ECC flexural members under reversed cyclic loading conditions”, to be published in ACI Structural Journal

Fischer G., Li, V.C. (2002b), “Deformation behavior of FRP reinforced flexural members under reversed cyclic loading conditions”, to be published in ACI Structural Journal

Fischer, G. (2002), “Deformation behavior of reinforced ECC flexural members under reversed cyclic loading conditions”, PhD thesis, Department of Civil and Environmental Engineering, University of Michigan

Lees, J.M., Burgoyne, C.J. (1999), “Experimental study of Influence of bond on flexural behavior of concrete beams pretensioned with Aramid Fiber Reinforced Plastics”, ACI Structural Journal, V.96, No.3, May-June 1999, pp.377-385

Li, V.C., "Engineered Cementitious Composites - Tailored Composites Through Micromechanical Modeling," in Fiber Reinforced Concrete: Present and the Future edited by N. Banthia, A. Bentur, A. and A. Mufti, Canadian Society for Civil Engineering, Montreal, pp. 64-97, 1998

Naaman, A.E., Jeong, S.M. (1995) “Structural ductility of concrete beams prestressed with FRP tendons”, Non-metallic (FRP) Reinforcement for Concrete Structures, Proceedings of the Second International RILEM Symposium (FRPRCS-2), E&FN Spon, London

Nanni, A. (1993), “Flexural behavior and design of RC members using FRP reinforcement”, ASCE Journal of Structural Engineering, Vol.119, No.11, November 1993, pp.3344-3359

TABLES

Reinforcement material	Diameter [mm]	Modulus [GPa]	Yield strength [MPa]	Yield strain [%]	Ultimate strength [MPa]	Ultimate strain [%]
Steel (S-1)	10	210	410	0.2	600	15
Aramid (S-2)	8	54	-	-	1800	3.8
C-FRP (S-3)	6.2	137	-	-	1800	1.8
C-FRP (S-4)	8	147	-	-	1800	1.3

Table 1 Material properties of longitudinal reinforcement

	Columns				Beam (steel reinforced)		
	Reinforcement	EI_{cr} $*10^{10}$ [Nmm^2]	M_y [kNm]	M_u [kNm]	EI_{cr} $*10^{10}$ [Nmm^2]	M_y [kNm]	M_u [kNm]
S-1	Steel	19.1	6.0	8.0	21.3	5.0	7.5
S-2	Aramid	4.2	-	12.5	13.3	2.9	3.5
S-3	C-FRP	6.5	-	14.5	13.3	2.9	3.5
S-4	C-FRP	9.9	-	17.4	21.3	5.0	7.5

Table 2 Summary of specimen configurations