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

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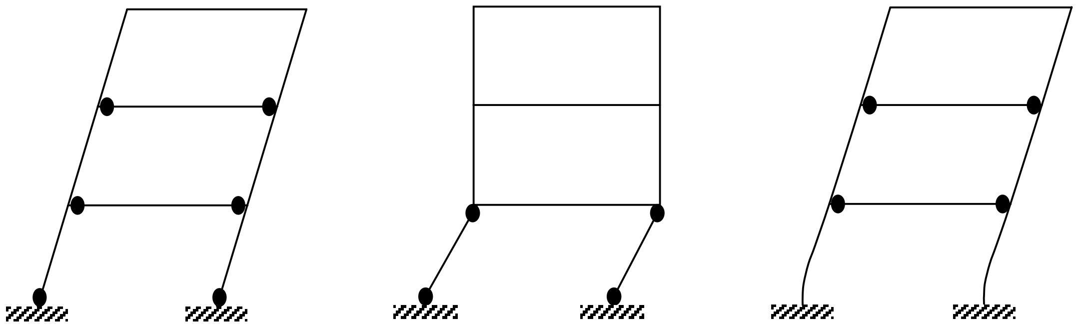
1 INTRODUCTION

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.



a) Strong column/ weak beam configuration

b) Soft-first story failure

c) Suggested configuration

Figure 1 Deformation behavior of moment resisting frames

2. RESPONSE CONCEPT

The structural response of moment resisting frame structures is largely determined by the interaction of beam and column members with respective cross-sectional properties and geometrical configuration. The flexural stiffness of the individual members is furthermore dependent on the degree of deflection as a result of successive formation of flexural cracking and inelastic compressive deformations in the cementitious matrix as well as inelastic deformation of the longitudinal reinforcement in tension.

Yielding of steel reinforcement in tension results in the formation of a plastic hinge in an under-reinforced flexural member, which is accompanied by a significant reduction of flexural stiffness at a yield moment M_y and yield deflection Δ_y (Fig.2). Beyond formation of a plastic hinge, a limited flexural strength increase may occur due to strain hardening of the steel reinforcement approaching the ultimate flexural capacity of the member.

The portal frame structure discussed in this paper serves as a simplified example of a moment resisting frame structure. The response of a conventional frame configuration with steel reinforcement in beam and column members at increasing lateral frame deformations can be sequentially described by elastic deformation behavior (Fig.3a), formation of plastic hinges in the beam member (Fig.3b), followed by formation of plastic hinges at the column bases (Fig.3c). At this ultimate state, the formation of a kinematic mechanism results in a statically unstable structure in which further displacement does not require increasing lateral load. At unloading, the frame remains in its deflected shape after some elastic unloading at a residual displacement Δ_r (Fig.3d). Besides the sectional properties of beam and column members, the geometry of the portal frame, in particular the ratio of beam to column length, may lead to the formation of plastic hinges in the columns prior to hinging in the beam, which, however, ultimately results in a kinematic mechanism as well.

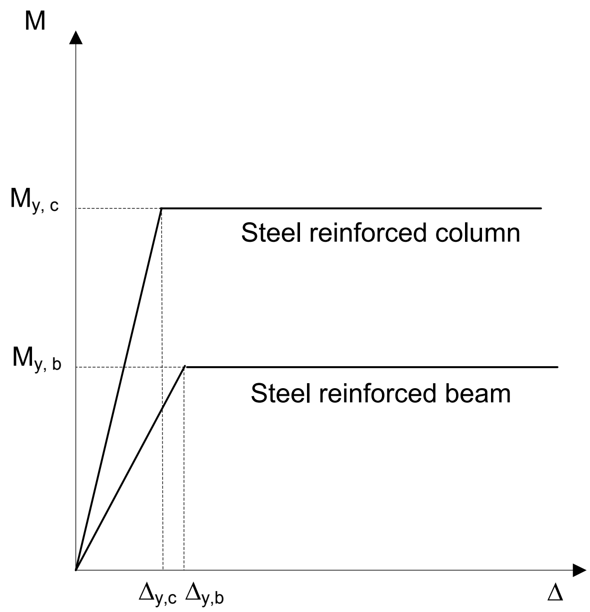


Figure 2 Schematic load-deformation behavior of steel reinforced beam and column members

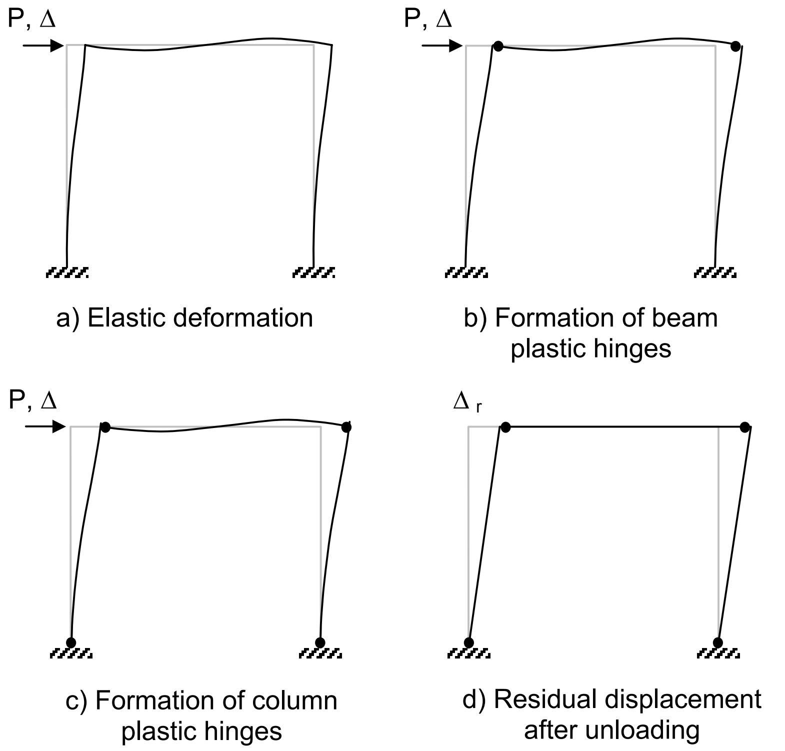


Figure 3 Deformation sequence of conventional steel reinforced frame configuration

In the suggested frame configuration, the longitudinal steel reinforcement in the column members is replaced by Fiber Reinforced Polymers (FRP) reinforcement. Generally, FRP reinforcement materials have a lower elastic modulus as well as higher tensile strength and elastic strain capacity compared to conventional mild steel reinforcement. Therefore, in FRP reinforced flexural members yielding does not occur and their load-deformation behavior remains nearly elastic up to failure. Non-linear deformations at approaching the ultimate state of these members are initiated by inelastic deformations of the cementitious matrix in compression. In order to achieve a similar flexural stiffness to steel reinforced members, the lower elastic modulus of FRP (50GPa to 150GPa) compared to steel (200GPa) can be compensated by an inversely proportional increase in longitudinal reinforcement ratio. By selecting an appropriate type and amount of longitudinal FRP reinforcement, flexural members with given geometrical configuration can be designed for flexural strength independent of flexural stiffness, in other words higher flexural strength does not necessarily imply higher flexural stiffness as in the case of exclusive use of longitudinal steel reinforcement.

It has been recognized, that FRP reinforced concrete members possess insufficient ductility as compared to the elastic/plastic deformation behavior of steel reinforced concrete members due to the elastic nature and relatively small ultimate tensile strain capacity of FRP reinforcement (ACI committee 440, 1996). Concepts to overcome this deficiency include ductile compression failure of concrete by providing extensive confinement or utilizing fiber reinforced concrete (Naaman and Jeong, 1995) as well as hybrid FRP reinforcement with inherent ductility (Harris et al., 1998). These concepts may provide a more ductile failure mode of FRP reinforced flexural members, however, do not increase their elastic deflection limit. The combination of FRP reinforcement with brittle concrete leads to a strain concentration in the FRP reinforcement in the vicinity of a crack location, which cannot be accommodated by the deformation capacity of FRP reinforcement. Studies on FRP reinforced concrete members therefore suggest partially debonded FRP reinforcement in order to increase the member deflection capacity (Lees and Burgoyne, 1999) and allow a reinforcement strain distribution over an extended length in the vicinity of a flexural crack (Nanni, 1993).

In this study, the combination of FRP reinforcement with an engineered cementitious composite (ECC) provides structural composite members with relatively large, elastic deflection capacity and flexural strength. Engineered cementitious composites are a particular class of fiber reinforced cement based composites with a moderate fiber volume fraction (V_f<2%), which are micromechanically designed to achieve a tensile stress-strain behavior analogous to that of metals (Li, 1998). The deformation mechanism of these FRP reinforced ECC members is fundamentally affected by the particular interaction of an elastic, fully bonded reinforcement material (FRP) combined with a ductile cementitious matrix (ECC). In the suggested frame configuration, the substitution of brittle concrete with this ductile ECC matrix in the FRP reinforced column members is necessary to achieve a relatively large elastic deflection capacity by distributing the flexural deformation over an extended portion of the member (Fischer and Li, 2002b). In addition, transverse reinforcement in form of stirrups may be significantly reduced in reinforced ECC members due to the intrinsic shear resistance and confinement effects of ECC.

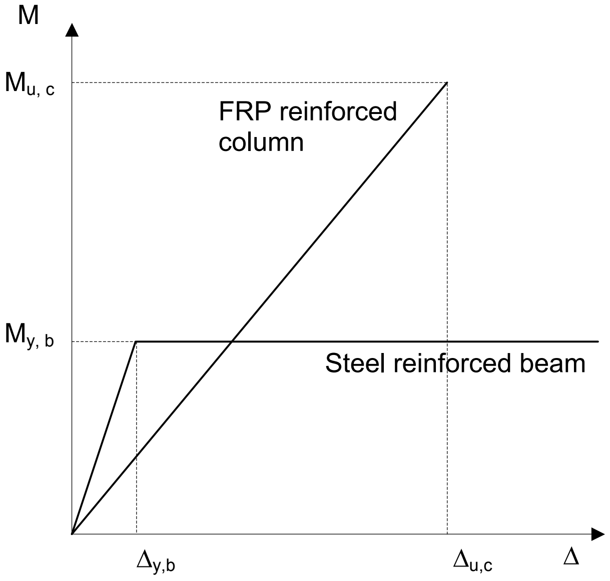


Figure 4 Schematic load-deformation behavior of steel reinforced beam and FRP reinforced column

In a portal frame structure assembled from a steel reinforced beam and FRP reinforced column members, the load-deformation response can be schematically described with an idealized graph of the individual beam and column response characteristics (Fig.4) at sequential frame deformation stages (Fig.5).

In the first stage at small, elastic frame displacements below Δ_{tr} and prior to the formation of plastic hinges, the steel reinforced beam has a larger flexural stiffness relative the FRP reinforced columns and experiences relatively small deflections, while the relatively soft columns largely accommodate the imposed lateral frame deformations. (Fig.5a). At this frame deformation stage, the frame responds at an initial, relatively large stiffness and resumes its undeformed shape at unloading.

At increasing frame deformations beyond Δ_{tr}, the system modifies its deformation mode and adapts to the increased level of lateral displacement by converting into a strong column/ weak beam mechanism (Fig.5b), effectively responding at a lower, secondary system stiffness. At this stage, plastic hinges in the beam are formed due to the flexural strength differential between beam and columns, thus dissipating energy by inelastic rotations in the beam plastic hinges. At further increasing lateral frame deformations, the columns remain elastic due to the elastic nature of the FRP reinforcement and prevent the formation of plastic hinges at their base, i.e. formation of a kinematic mechanism (Fig.5c). Because of the elastic deflection behavior of these particular column members, the permanent displacement Δ_r at unloading of the frame is reduced, however, not fully eliminated due to residual rotation at the beam-column joint induced by the presence of the plastic hinge in the beam member (Fig.5d).

Depending on structural performance requirements such as the expected level of excitation and acceptable temporary and residual displacements, the details of the frame response, i.e. initial and secondary stiffness as well as transition load P_{tr} and transition displacement Δ_{tr} can be defined in the design procedure. In contrast to conventional moment resisting frame systems, this suggested configuration shows an intrinsic bi-linear load-deformation response with auto-adaptive stiffness modification capabilities, which are expected to reduce the base shear forces in case of a seismic event.

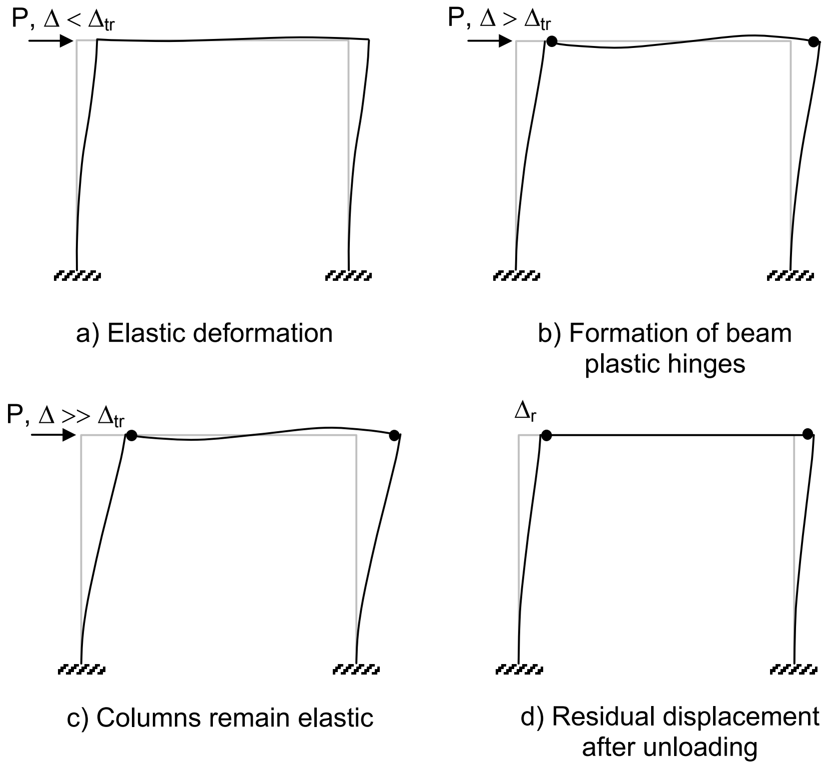


Figure 5 Deformation sequence of suggested frame configuration

3. VERIFICATION

3.1 Specimen configurations and test setup

The experimental verification of the above outlined concept is using portal frame specimens at approximately one-fifth scale. The response of three specimens of the suggested configuration with FRP reinforced columns and steel reinforced beam is compared to that of a conventional configuration exclusively using steel reinforcement in beam and column members. The main variables investigated are the flexural stiffness of the columns as well as the influence of beam yield strength on the response of the frame system. All specimens discussed in this paper used ECC matrix in order to focus the comparison on the effect of the longitudinal reinforcement materials. The influence of ECC on the structural behavior of reinforced ECC flexural members in comparison to reinforced concrete are discussed in separate work (Fischer and Li, 2002a, b) Transverse steel reinforcement is not used and confinement as well as shear resistance are entirely provided by the ECC matrix. Details of the specimen configurations and individual member flexural stiffness and nominal strength are summarized in Tables 1 and 2. The specimen geometry, reinforcement configuration (Fig.6) and test setup (Fig.7) are schematically shown. Lateral loading is applied through a loading frame equipped with a 100kN capacity actuator according to a displacement controlled loading sequence up to 5% drift. Axial loading is not applied to the column members.

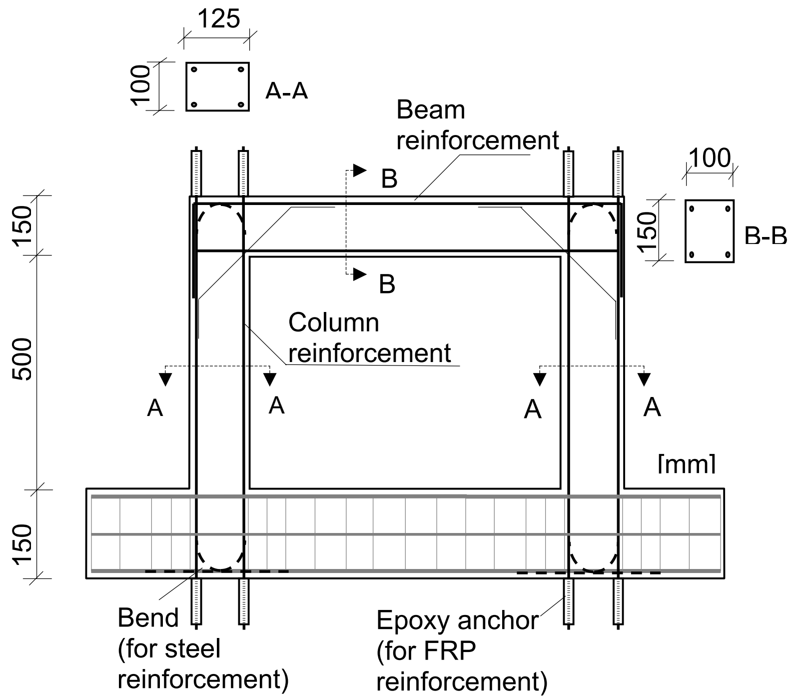


Figure 6 Specimen configuration and test setup

3.2 Model response

Considering the initial and secondary response stage separately, the expected load-deformation response of the suggested frame configuration in specimens S-2, S-3, and S-4 (Table 2) can be analytically derived. In the initial stage, beam and columns are deforming elastically and the frame behavior can be modeled using textbook methods considering lateral frame displacements u_1 as well as joint rotations u_2 and u_3 (Fig.8a). Beyond formation of plastic hinges in the beam member, the portal frame can be modeled as two elastic cantilever columns with a moment equal to the ultimate flexural strength of the beam applied at the beam/column joint. (Fig.8b). The derivations of the expressions for the load-deformation response in the initial and secondary response stage are described in detail elsewhere (Fischer, 2002).

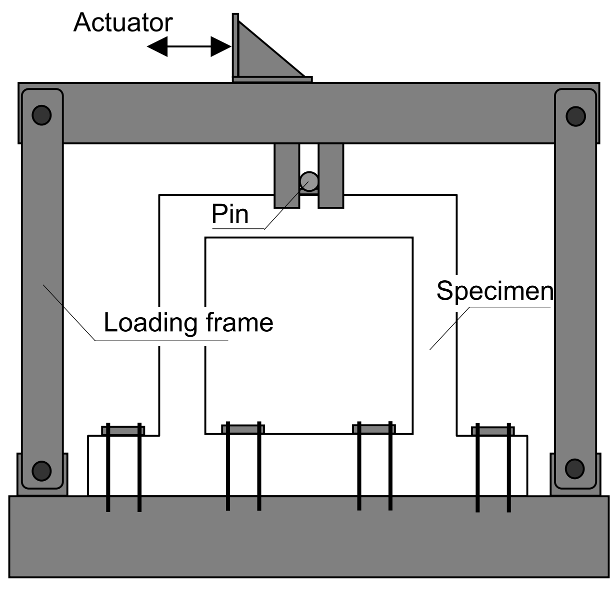


Figure 7 Schematic of test setup

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.

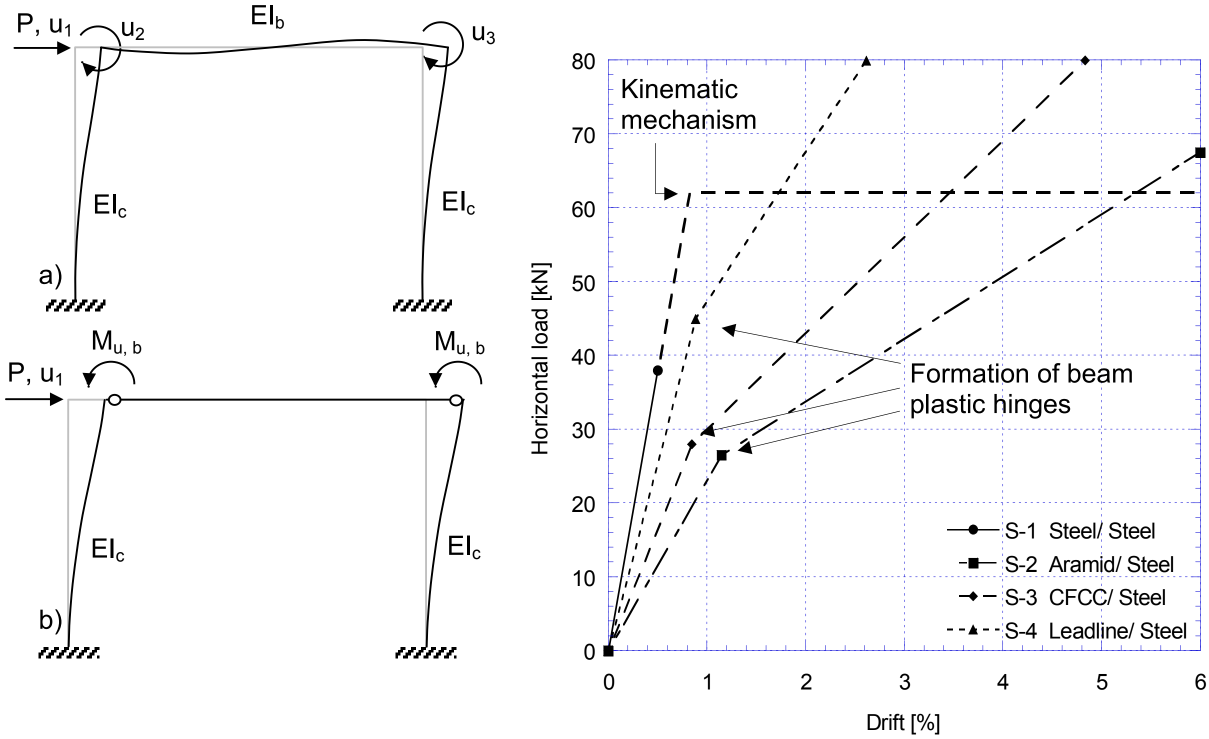


Figure 8 Model for a) initial response and b) secondary response stage; theoretical response of tested specimens

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

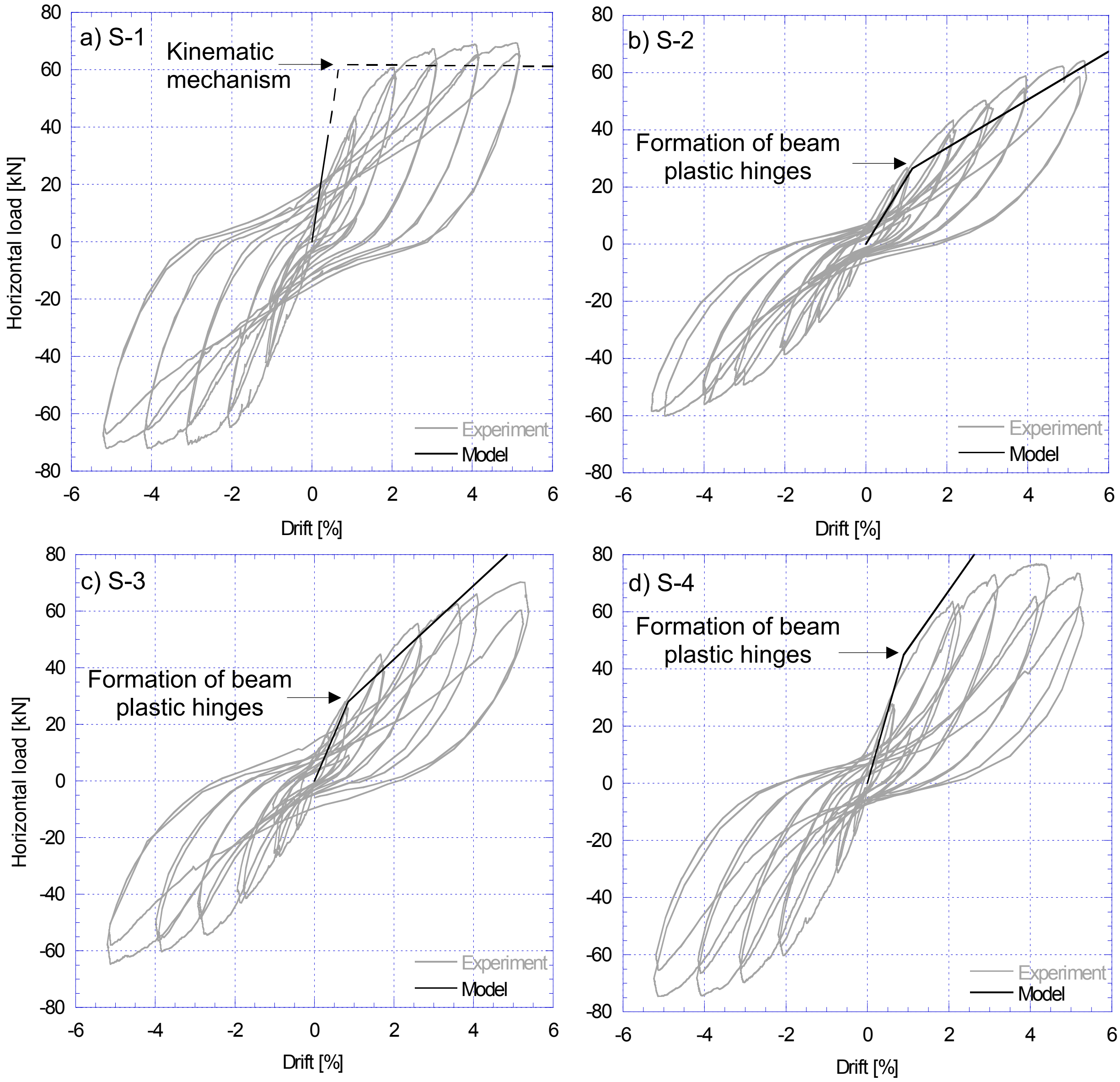


Figure 9 Load-deformation response of tested specimens

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.

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.

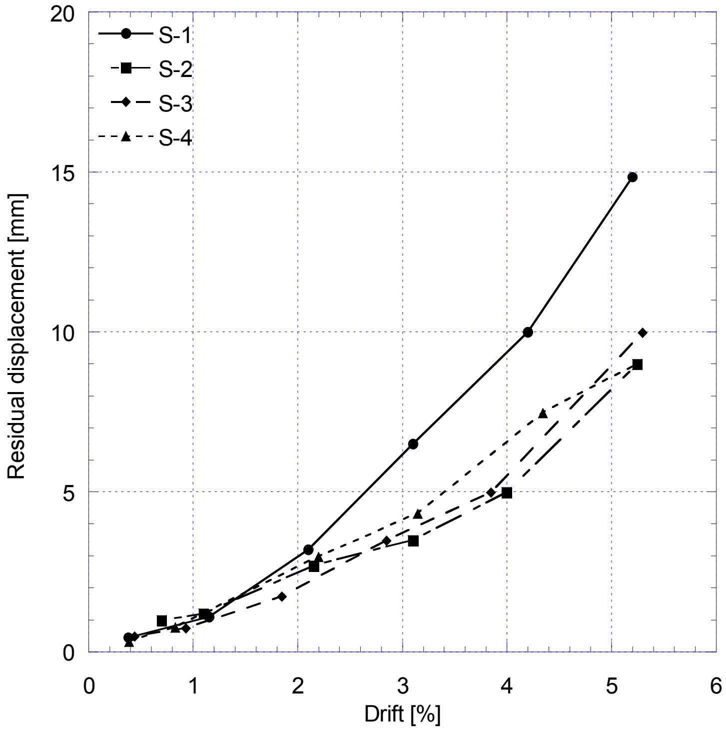


Figure 10 Comparison of residual displacements

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.

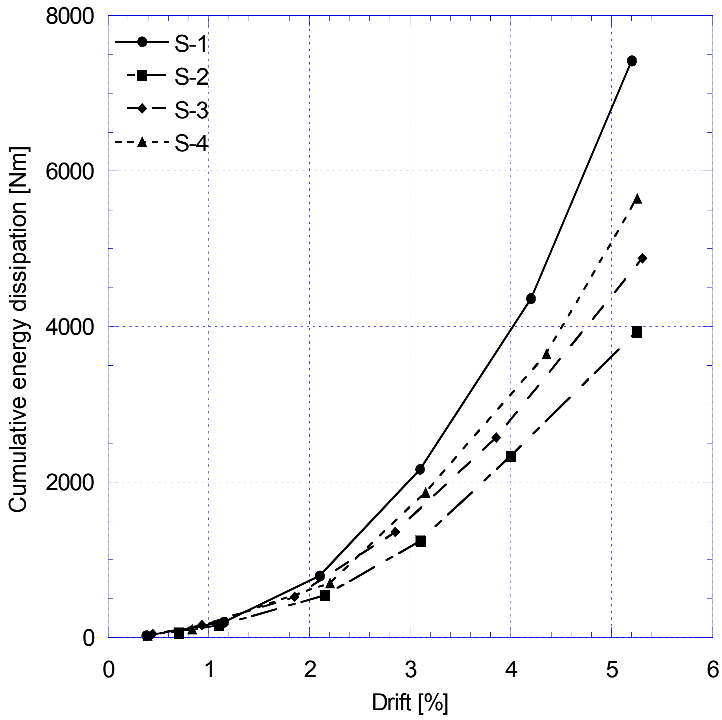


Figure 11 Comparison of cumulative energy dissipation

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.

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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