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Ductile corrosion-free self-centering concrete elements

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Abstract

Corrosion is a major factor in the deterioration of reinforced concrete (RC) structures. To mitigate this problem, steel bars can be replaced with glass-fiber-reinforced-polymer (GFRP) bars. However, the lack of ductility of GFRP-RC elements has prevented their use in many structural applications, especially in seismic areas. Superelastic shape memory alloy (SMA) bars have been proposed to be used in seismic areas because of their self-centering characteristics. Also, they have the added advantage of being corrosion resistant. This paper examines the combined use of SMA and GFRP bars to achieve ductile self-centering and corrosion-free elements. The first challenge for such a proposal relates to designing concrete frames, reinforced with SMA and GFRP bars, that have adequate lateral performance in terms of initial stiffness, ductility, and strength. A comprehensive parametric study is conducted to better understand the structural behavior of concrete elements reinforced with SMA and/or GFRP bars. Results from the study are utilized to develop design equations that allow designing an SMA/GFRP RC section to replace a steel RC section, while maintaining lateral strength, stiffness, and ductility. To examine the adequacy of the developed equations, a six-storey concrete frame is designed, and its lateral performance is examined using pushover analysis.

Keywords: GFRP, SMA, Residual Deformation, Ductility, Stiffness, Strength, Corrosion.

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1. Introduction

Replacing steel bars with fiber-reinforced polymer (FRP) bars has emerged as an innovative solution to overcome corrosion problems. Moreover, FRP bars have the advantages of: (1) high resistance to electrical and magnetic fields, (2) high strength, (3) lightweight, and (4) availability [1, 2, 3]. Due to their inability to dissipate seismic energy, the use of FRP bars in concrete structures is limited [4]. FRP reinforced concrete (RC) structures have significantly less ductility and energy dissipation capacity than steel RC structures.

The behaviour of FRP RC elements was examined by many researchers including: Benmokrane et al. [5]; Alsayed et al. [6]; Ferreira et al. [7]; Tavares et al. [8]; Rasheed et al. [9]; Wegian and Abdalla [10]; Li et al. [11]; Rougier and Luccioni [12]; Barris et al. [13]; Lau and Pam [14]; Al-Sunna et al. [15]; Kara and Ashour [16]; Billah and Alam [17]; Mias et al. [18]; and Prachasaree et al. [19]. An experimental program for concrete columns reinforced with longitudinal and transverse GFRP bars was conducted by Tobbi et al. [20, 21, 22]. Their study revealed that GFRP RC columns withstood loads similar to or higher than steel RC columns. The experimental results were then used to develop a strength model for square columns reinforced with longitudinal and transverse GFRP bars. Experimental programs on circular concrete columns reinforced with GFRP hoops and spirals were performed by Pantelides et al. [23], Mohamed et al. [24], and Afifi et al. [25]. The behaviour of GFRP-confined concrete cores were found to be similar to that of steel-confined concrete cores. Based on the experimental results, Afifi et al. [26] developed a mechanical model for circular concrete columns reinforced with GFRP spirals or hoops. The model accounted for the transverse reinforcement volumetric ratio, strength, spacing, and configuration. Another experimental study on circular and square concrete columns with longitudinal and transverse GFRP hoops/spirals was carried out by Prachasaree et al. [19]. They

concluded that spiral transverse reinforcement is the most effective in terms of confining pressure and ductility.

Another material, that received significant attention from researchers, is superelastic shape memory alloy (SMA). Its ability to undergo large deformations and, then, retrieve the original shape upon unloading, combined with its high corrosion resistance, makes it a perfect replacement for steel bars [27]. Ni-Ti SMA, composed of nickel and titanium, is highly efficient in recovering large strains and is found to be one of the most appropriate alloys for structural applications. Auricchio and Sacco [36] described the behavior of Ni-Ti SMA through a simple 1-D phenomenological model. DesRoches et al. [42] evaluated the properties of Ni-Ti SMA under cyclic loading and assessed its potential for applications in seismic resistant design. Otsuka et al [43] and Duerig et al. [44] described the characteristics, fabrication techniques and thermomechanical treatment of Ni-Ti SMA. Tanaka et al. [45] introduced ferrous SMA, which has higher maximum superelastic strain than Ni-Ti SMA.

However, the high cost of SMA bars prevents their use as the main reinforcing bars. Instead, researchers have used SMA bars only at the critical locations and used steel bars elsewhere [28, 29, 30, 31]. The seismic performance of an eight-storey concrete frame reinforced with SMA bars at its beam-column connections and steel bars at the other regions was examined by Alam et al. [27] considering ten ground motions. The analytical results showed that this system can recover most of its post-yield deformations. Youssef and Elfeki [30] also studied the seismic performance of concrete frames reinforced with steel and SMA bars. They reported the locations of SMA bars that are expected to increase the seismic capacity and reduce the residual deformations of typical RC frames. Although these studies solve the implication of the high cost of SMA bars, the increase

in the instantaneous deformations, because of the low modulus of elasticity of SMA as compared to steel, presents a concern for many researchers.

Nehdi et al. [32] experimentally assessed the usage of concrete joints reinforced with SMA bars at the plastic hinge region and GFRP at other regions. They concluded that the SMA-GFRP hybrid system can adequately dissipate the seismic energy while reducing seismic residual deformations. Similar conclusion was reached by Billah and Alam [17] after analyzing a set of SMA-GFRP concrete columns under cyclic loading.

In the present study, the concept of a hybrid reinforcement configuration (GFRP and SMA) was adopted to achieve ductile and corrosion free concrete elements. The SMA bars are used to reinforce the plastic hinge regions. The FRP and SMA bars are assumed to be connected using suitable couplers as suggested by Alam et al. [33]. The objectives of this study are: (1) to assess the performance of concrete elements reinforced with SMA bars at the plastic hinge regions and GFRP bars at other regions, and (2) to develop design criteria to ensure that the SMA/GFRP frame lateral performance is acceptable in terms of stiffness, strength, and ductility. The following sections provide details about the modeling assumptions, lateral performance of FRP-SMA RC frames, parametric study to develop the design criteria for FRP-SMA RC frames, and a case study to examine the developed criteria.

2. Modeling Assumptions

2.1 Material Constitutive Models

The concrete is modeled using a uniaxial nonlinear constant confinement concrete model that follows the constitutive relationship proposed by Mander et al. [34] and the cyclic rules proposed by Martinez-Rueda and Elnashai [35]. Two different concrete compressive strengths (f'_c) are

considered in this paper (30 MPa and 40 MPa). The yield strength, modulus of elasticity, and strain hardening parameters for the reinforcing steel bars are assumed to be 400 MPa, 200,000 MPa, and 0.02, respectively.

The properties of the transverse GFRP reinforcement, which were reported by Tobbi et al. [20] and summarized in Table 1, are adopted (E_f is modulus of elasticity, f_{fu} is the ultimate tensile strength, and ε_{fu} is the ultimate tensile strain). The properties of the longitudinal GFRP bars depend on the bar size and the modulus of elasticity. The used values are summarized in Table 2.

Table 1 Transverse GFRP reinforcement properties (Tobbi et al. [20])

	E_f (MPa)	f_{fu} (MPa)	ε_{fu} (%)
Straight portion	44,000	640	1.45
Bent portion		400	

Table 2 Longitudinal GFRP reinforcement properties

Bar Diameter (mm)	$E_f = 46,000$ MPa		$E_f = 65,000$ MPa	
	f_{fu} (MPa)	ε_{fu} (%)	f_{fu} (MPa)	ε_{fu} (%)
10	827	1.80	1372	2.11
13	758	1.65	1312	2.02
16	724	1.57	1184	1.82
19	690	1.50	1105	1.70
22	655	1.42	1059	1.63
25	620	1.35	1000	1.54

A typical stress-strain curve of superelastic SMA is presented in Fig. 1. The figure shows the 1D-superelastic model by Auricchio et al. [36]. The parameters defining the model and their assumed values, as reported by Youssef and Elfeki [30], are: $f_y = 400$ MPa (austenite to martensite starting

stress), f_{P1} = 510 MPa (austenite to martensite finishing stress), f_{T1} = 370 MPa (martensite to austenite starting stress), f_{T2} =130 MPa (martensite to austenite finishing stress), ϵ_l = 6% (superelastic plateau strain length), E_a = 62,500 MPa (modulus of elasticity), and E_s = 1,780 MPa (modulus of elasticity between f_y and f_{P1}).

Fig. 1 Superelastic model of SMA by Auricchio et al. [36].

2.2 Modeling

The commercial finite element software Seismostruct [37] is used in the current study. The software accounts for both geometric and material nonlinearities. Fiber modelling approach is utilized to account for the spread of inelasticity along the member length and across the section area. The sectional stress-strain state is obtained through the integration of the nonlinear stress-strain response of the cross-sectional fibers. The element formulation uses two integration Gauss points per element. Beams and columns are individually modeled as cantilevers with length $(0.5L)$ or $(0.5H)$, where L is the beam span and H is the column height. A lateral load, P_L , is assumed to be acting at the cantilever's tip. The value of the axial load is assumed zero for the beams and P_A

for the columns. Ten displacement-based frame elements [37] are used to model each of the beams and columns. The spread of inelasticity along the member length is captured using 200 fibers. The section stress-strain state is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers forming the cross section. The proposed modeling technique has been extensively validated at Western University [27, 30, 31, 32, and others].

When modeling a complete RC frame, ten elements are utilized to model each beam and column. The cross sections of the beams and columns is divided into 300 fibers, as recommended by Youssef and Elfeki [30]. Fixed supports are assigned for all of the first storey columns at their connections with the foundations. Additionally, the beams are modeled as T-sections to account for the floor slab. The beam-column joints are assumed to be rigid. Dead loads are applied before applying the lateral loads.

2.3 Failure Criteria

The ultimate strain of the unconfined concrete in the beams, $\varepsilon_{u(unconfined)}^s$, is assumed equal to 0.004. For confined steel RC columns, the core concrete ultimate strain, $\varepsilon_{u(confined)}^s$, is calculated using Eq. (1) by Paulay and Priestley [38].

$$\varepsilon_{u(confined)}^s = \varepsilon_{u(unconfined)}^s + \frac{1.4\rho_s f_y \varepsilon_{sm}}{k_h^s f'_c} \quad (1)$$

where ρ_s is the ratio of the volume of transverse reinforcement of concrete core measured to the outside of the transverse reinforcement, ε_{sm} is the steel strain at maximum tensile stress, k_h^s is the confinement factor, which can be obtained from the charts provided by Priestley and Wood [39].

The ultimate strain for columns confined with GFRP lateral stirrups (ε_u^f) is calculated using Eq. (2) by Afifi et al. [26].

$$\varepsilon_u^f = [0.000937(f'_{co})^{0.25}] \cdot \left[0.63 + (70.6 - 1.76f'_{co}) \sqrt{\frac{f_{le}}{f'_{co}}} \right] \quad (2)$$

where f'_{co} is the strength of unconfined concrete and f_{le} is the effective lateral pressure. k_h^f is the confinement factor for GFRP stirrups, which can be obtained using Eq. (3) by Tobbi et al. [22].

$$k_h^f = 1 + 1.23 \left(\frac{f_{le}}{f'_{co}} \right)^{0.71} \quad (3)$$

Both the beam and column elements are assumed to fail when the concrete reaches the ultimate strain. Frames are assumed to fail when the ultimate strain of concrete is reached in all columns of the same storey.

3. Lateral Performance of SMA-GFRP RC Frames

A six-storey steel RC frame (Frame 1) designed by Youssef and Elfeki [30], and shown in Fig. 2, is considered in this section. The yield strength of the steel bars is 400 MPa and the concrete compressive strength is 28 MPa. Cross sections of the beams and columns are presented in Fig. 3. The frame was redesigned utilizing SMA and GFRP instead of the steel bars (Frame 2). The same cross-section areas and details are maintained. Steel bars are replaced with equal areas of GFRP bars (E_f of 46,000 MPa) except at the plastic hinge areas, where SMA bars are used. The length of the SMA bars is calculated using Eq. (4) that was proposed by Paulay and Priestley [38] and recommended for SMA RC elements by Alam et al. [40] and Wang [41]. Mechanical couplers are assumed to link the SMA and GFRP bars based on the recommendations of Billah and Alam [17]. Fig. 4 shows a beam-column joint tested by Nehdi et al. [32]. SMA bars were used at the plastic hinge regions of the joint and GFRP bars were utilized elsewhere. Screw lock-adhesive type couplers were utilized to connect the GFRP bars to the SMA bars. The coupler has two parts: a

stainless-steel pipe to be filled with epoxy resin to connect to the FRP bar and a screw-lock coupler to connect to the SMA bar.

GFRP and SMA bars are assumed to have adequate development lengths. Additionally, their mechanical connections cause portion of the bar forces to be transferred to the concrete through bearing [17]. Thus, perfect bond is assumed between the concrete and the bars. Deformations are expected to be concentrated near the middle of the SMA bars and reduce at their ends to reach values compatible with the GFRP bars. This behaviour is expected since the middle of the SMA bar is aligned with the center of the plastic hinge region. Inelastic deformations are significantly reduced away from this center.

$$L_p = 0.08 \cdot L + 0.022 \cdot d_{SMA} \cdot f_y \quad (4)$$

where L is half the clear span of the considered beam, d_{SMA} is the SMA bar diameter, and f_y is the yield strength of the SMA bars.

The pushover curves for Frames 1 and 2 are shown in Fig. 5. They show the variation of the roof displacement with the applied load. It can be observed that Frame 2, which is expected to be superior in corrosion protection and residual deformations, has lower failure load (-11%), displacement at failure (-10%), initial stiffness (-151%), and ductility (-180%). Such performance is directly related to the modulus of elasticities of the SMA and GFRP bars that are much lower than that of steel bars. The results emphasize the need for new design criteria for frames reinforced with GFRP and SMA bars.

Fig. 2 Elevation and plan views of Frame 1 [30]

Fig. 3 Cross sections of beams and columns of Frame 1 [30]

Fig. 4 Mechanical coupler in a beam-column connection [32].

Fig. 5 Pushover curves for Frame 1 and Frame 2.

4. Design of FRP-SMA RC Frames

A comprehensive parametric study is first conducted to understand the lateral performance of RC elements reinforced with combination of SMA and GFRP bars. Results from this study are then utilized to form a design method for such elements.

4.1 Parametric Study

The parameters for the analyzed beams are: width ($b_b=250$ or 400 mm), height ($h_b=400, 600,$ or 800 mm), beam length ($L= 5.0$ or 7.0 m), area of tension bars ($A_s =$ minimum reinforcing area A_{smin} , half of the maximum allowable area $\frac{1}{2}A_{sb}$, or maximum allowable area A_{sb}), area of compression steel bars ($A_s'=0$ or 20% of A_s), and length of the SMA bars ($L_{SMA}=L_p$ or $\frac{1}{2}L_p$). The reinforcement ratio at the tension side (ρ) is defined as $\rho = \frac{A_s}{b_b h_b}$.

The parameters for the analyzed columns are: width ($b_b=250$ or 400 mm), section height ($h_b= 400, 600,$ or 800 mm), and column height (3.0 m or 4.0 m). Only GFRP bars are used in the columns as capacity design requires plastic hinges to form in the beams. Three levels of compressive axial load ($10\%, 40\%,$ and 70% of the column axial load capacity) are considered. The reinforcement ratio is varied from 1% to 4% with an increment of 1% . The number of longitudinal bars and the arrangement of the stirrups are selected similar to those of the steel reinforced columns.

The analysis for each of the considered cases of the SMA-GFRP-RC elements (576 beams and 288 columns) is conducted by applying the axial load, P_A , followed by an incremental lateral load, P_L . The lateral performance is then compared to that of an element reinforced with steel bars (steel RC element) in terms of the overall performance, ductility, initial stiffness, and capacity. The lateral performance for each steel RC element and corresponding GFRP-SMA element are expected to take the shape shown in Fig. 6. The areas formed between the two curves, A_1^c and A_2^c ,

are calculated for each case. If the difference between the two areas is less than 5%, the overall performance is judged as acceptable. The ductility, initial stiffness and capacity are also compared and a difference less than 10% is considered acceptable.

If the lateral performance of the GFRP-SMA RC element is judged unacceptable, its design is revised by changing the section height by a factor F_h and/or the area of the reinforcing bars by a factor F_r . Values of 0.5 to 2.0 with an increment of 0.05 are examined for each factor until a section with almost the same lateral performance as the steel RC section is identified. SeismoStruct batch facility [37] is used to conduct the required iterations.

Fig. 6 Expected pushover curves of steel RC element and SMA/GFRP element.

The modifying factors for the beams are found to be affected by the compressive (ρ') and tension (ρ) reinforcement ratios, properties of the GFRP bars, as well as the length of SMA bars (L_{SMA}). On the other hand, the modifying factors for the columns are affected by the level of the axial compressive load (P), reinforcement ratio (ρ) as well as the modulus of elasticity of GFRP (E_f).

The beam span, column height, cross sectional width, and concrete strength are found to have insignificant effect on the modifying factors.

Variation of the beam modifying factors is shown in Figs. 7 and 8 for E_f of 46,000 MPa and 65,000 MPa, respectively. The factors in Fig. 8 are lower than those in Fig. 7 because of the higher E_f . For each E_f , it can be noticed that the height factor (F_h) has a constant value that is greater than 1, which is responsible for adjusting the element stiffness. The reinforcement factor (F_r) reduces the area of the bars to maintain the load capacity at approximately the same level. The ductility is provided by the SMA bars. F_r varied from 0.65 to 0.83 for values of ρ varying between 0.27% and 4.22%.

The modifying factors for the columns are shown in Figs. 9 and 10 for $E_f = 46,000$ MPa and 65,000 MPa, respectively. For $E_f = 46,000$ MPa and ρ values of 1% to 4%, F_r varies from 0.84 to 0.90 and F_h varied from 1.10 to 1.32. There is a slight decrease in the height factor with the increase in the column's compressive axial force. For $E_f = 65,000$ MPa and ρ values of 1% to 4%, F_r varies from 0.84 to 1.00 and F_h varies from 1.06 to 1.10.

Figs. 7 to 10 show that F_h is always greater than or equal one and F_r is always less than or equal one. It can also be noticed that F_h values for beams are significantly greater than those for columns. However, F_r values for beams are always less than those for columns.

(a) $\rho' = 0, L_{SMA} = L_p$ (b) $\rho' = 0, L_{SMA} = \frac{1}{2} L_p$

(c) $\rho' = 0.2\rho, L_{SMA} = L_p$ (d) $\rho' = 0.2\rho, L_{SMA} = \frac{1}{2} L_p$

Fig. 7 Modifying factors for GFRP reinforced beams with $E_f = 46,000$ MPa

(a) $\rho' = 0$, $L_{SMA} = L_p$ (b) $\rho' = 0$, $L_{SMA} = \frac{1}{2}L_p$

(c) $\rho' = 0.2\rho$, $L_{SMA} = L_p$ (d) $\rho' = 0.2\rho$, $L_{SMA} = \frac{1}{2}L_p$

Fig. 8 Modifying factors for GFRP reinforced beams with $E_f = 65,000$ MPa.

(a) $P = 0.1P_{\max}$ (b) $P = 0.4P_{\max}$

(c) $P = 0.7P_{\max}$

Fig. 9 Modifying factors for GFRP reinforced columns with $E_f = 46,000$ MPa.

(a) $P = 0.1P_{\max}$

(b) $P = 0.4P_{\max}$

(c) $P = 0.7P_{\max}$

Fig. 10 Modifying factors for GFRP reinforced columns with $E_f = 65,000$ MPa.

4.2 Proposed Design Method

It is recommended to design the section using steel bars and then modify the section height and the area of bars using the factors F_h and F_r . Values for these factors are given by Eqs. (5) and (6) for beams reinforced with GFRP and SMA bars. Eqs. (7) and (8) are for columns reinforced with GFRP. These equations were derived using statistical analysis of the parametric study results. Applying in the equations requires knowledge of the steel reinforcement ratio, the plastic hinge length, the column axial load, and the GFRP modulus of elasticity. Units of E_f is MPa in Eqs (5) to (8).

$$F_h = 1.55 \left(\frac{46,000}{E_f} \right)^{0.18} \quad (5)$$

$$F_r = \begin{cases} 0.65 & \rho \leq 0.36 \\ 0.11\rho + 0.62 & 0.36 \leq \rho \leq 1.21 \\ 0.74 & 1.21 \leq \rho \leq 2.54 \\ 0.04\rho + 0.64 & 2.54 \leq \rho \leq 4.04 \\ 0.8\lambda & 4.04 \leq \rho \end{cases} \quad (6a)$$

$$\lambda = \begin{cases} 1.04 & \rho' = 0, L_{SMA} = L_P \\ 1.00 & \rho' = 0, L_{SMA} = \frac{1}{2}L_P \\ 1.00 & \rho' = 0.2\rho, L_{SMA} = L_P \\ 0.98 & \rho' = 0.2\rho, L_{SMA} = \frac{1}{2}L_P \end{cases} \quad (6b)$$

$$F_h = 0.037\zeta_1\rho + 1.1\eta_1 \quad (7a)$$

$$\zeta_1 = \begin{cases} +0.27 & E_f = 65,000 \text{ and } P = 0.1 \text{ to } 0.4P_{max} \\ -0.08 & E_f = 65,000 \text{ and } P = 0.7P_{max} \\ +1.0 & \text{Otherwise} \end{cases} \quad (7b)$$

$$\eta_1 = \begin{cases} +0.95 & E_f = 65,000 \text{ and } P = 0.1P_{max} \text{ or } 0.4P_{max} \\ +1.0 & \text{Otherwise} \end{cases} \quad (7c)$$

$$F_r = -0.02\zeta_2\rho + 0.93\eta_2 \quad (8a)$$

$$\zeta_2 = \begin{cases} +2.0 & E_f = 65,000 \text{ and } 0.4P_{max} \\ +1.65 & E_f = 65,000 \text{ and } 0.7P_{max} \\ +1.0 & \text{Otherwise} \end{cases} \quad (8b)$$

$$\eta_2 = \begin{cases} +1.10 & E_f = 65,000 \text{ and } 0.4P_{max} \\ +1.10 & E_f = 65,000 \text{ and } 0.7P_{max} \\ +1.0 & \text{Otherwise} \end{cases} \quad (8c)$$

5. Case Study

The six-storey RC frame (Frame 1) is considered to further examine the developed design method. Frames 2 and 3 are assumed to have the same section dimensions as Frame 1. The reinforcing bars are replaced with similar areas of GFRP or SMA bars in Frames 2 and 3. E_f is assumed 46,000 MPa for Frame 2 and 65,000 MPa for Frame 3.

The design of Frames 4 and 5 follows the developed method. The value of ρ at the plastic hinge region for Beams 1 and 2, shown in Fig. 2, is 0.77%. The ρ values at other regions of Beams 1 and 2 are 0.44% and 0.52%, respectively. For the columns, the values of ρ are 2.3%, 2.5%, 2.33%, 2.7%, and 1.6% for Col 1, Col 2, Col 3, Col 4, and Col 5, respectively. The exterior column axial forces are 34%, 27%, 29%, 21%, 16%, and 12% of the axial load capacities of the columns in the 1st, 2nd, 3rd, 4th, 5th, and 6th storey, respectively. The axial load ratios for the interior columns are 30%, 24%, 26%, 20%, 14%, and 9%. The section height and the area of the reinforcing bars are adjusted using the developed method. The modifying factors are presented in Table 3. The pushover curves for the five frames are shown in Fig. 11 and the key parameters are summarized in table 4. The pushover curves show the variation of the roof displacement with the applied load. The differences between the initial stiffness, failure load, strength, and ductility of Frame 1, Frame

2, and Frame 3 are quite significant. On the other hand, the lateral performance of Frames 4 and 5 is very similar to that of Frame 1.

Table 3 Modifying factors for Frames 4 and 5

	Frame 4		Frame 5	
Section	F_h	F_r	F_h	F_r
Beam 1 ^P	1.55	0.70	1.46	0.70
Beam 2 ^P	1.55	0.70	1.46	0.70
Beam 1 ^O	1.55	0.67	1.46	0.67
Beam 2 ^O	1.55	0.67	1.46	0.67
Col 1	1.19	0.89	1.12	0.93
Col 2	1.19	0.88	1.12	0.92
Col 3	1.19	0.88	1.12	0.93
Col 4	1.20	0.87	1.12	0.93
Col 5	1.20	0.87	1.12	0.90

^P At the plastic hinge region

^O Outside the plastic hinge region

Fig. 11 Lateral performance of Frames 1 to 5.

Table 4 Key parameters from pushover analysis

	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
Initial stiffness, K_i (kN/mm)	14.08	7.24	7.76	11.4	12.7
% Difference in K_i as compared to Frame 1	0	-49	-45	-19	-10
Failure load, F_L (kN)	1620	1448	1513	1753	1785
% Difference in F_{Li} as compared to Frame 1	0	-11	-7	8	10
Failure displacement, F_D (mm)	661	603	602	655	657
% Difference in F_{Di} as compared to Frame 1	0	-8.8	-8.9	-0.9	-0.6

6. Conclusions

In this paper, the concept of hybrid GFRP and SMA bars is adopted to have a corrosion free RC frame that possesses adequate ductility, strength, and stiffness. The SMA bars are used as reinforcement at the plastic hinge regions to allow recovering the inelastic lateral deformations. Mechanical couplers are assumed to link the SMA and GFRP bars.

A six-storey steel RC frame is considered. Frames 2 and 3 are revised designs of Frame 1, which utilize SMA and GFRP instead of steel bars. The pushover curves for the three frames show that Frames 2 and 3, which are expected to be superior in corrosion protection and residual deformations, have lower failure load, displacement at failure, initial stiffness, and ductility when compared to Frame 1.

A comprehensive parametric study is conducted to form a design method for such elements. 576 beams and 288 columns are considered in this study. Based on this study, modification factors for the section height and area of steel are proposed. Values for these factors can be estimated using

equations (5), (6), (7), and (8). The developed factors are used to modify the design of frames 2 and 3, which have led to a lateral performance that is comparable to the steel RC frame.

Conclusions of this study are limited to the examined configurations. Future analytical and experimental studies examining the validity of the proposed design equations for other configurations are needed. Future research should also include accurate modeling of the mechanical couplers and bond behaviour of the SMA and GFRP bars.

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