# Seismic Behavior of Shape Memory Composite Bars in RC Moment Frames

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#### SUMMARY:

Fiber reinforced polymer (FRP) reinforcing bars have been used in concrete structures as alternate to conventional steel reinforcement, in order to overcome corrosion problems. However, due to the linear behavior of the commonly used reinforcing fibers, they are not considered in structures which require ductility and damping characteristics. The use of superelastic shape memory alloy (SMA) fibers with their nonlinear-elastic behavior as reinforcement in the composite could potentially provide solution for this problem. Small diameter SMA wires are coupled with polymer matrix to produce SMA-FRP composite which is sought in this research as reinforcing bars. SMA-FRP bars are sought in this study to enhance the seismic performance of reinforced concrete (RC) moment resisting frames (MRFs) in terms of reducing their residual inter-storey drifts while still maintaining the elastic characteristics associated with conventional FRP. Response of this suggested composite reinforcement has been investigated experimentally by first manufacturing and then performing uni-axial tests in the laboratory. Analytical material models are developed for the composite materials based on experimental results. Three storey one bay and six storey two bay RC MRF prototype structures are designed with steel, SMA-FRP and glass-FRP reinforcement. Incremental dynamic analysis technique is used to investigate the behaviors of the two frames with the three different reinforcement types under a suite of ground motion records. It is found that the frames with SMA-FRP composite reinforcement exhibit higher performance levels including lower residual inter-storey drifts, high energy dissipation and thus lower damage, which is of essence for structures in high seismic zones.

Keywords: Fiber reinforced polymer, Shape memory alloys, Reinforced concrete, Incremental dynamic analysis.

# **1. INTRODUCTION**

Although FRP offers many benefits over steel reinforcement, it induces unsatisfactory structural ductility and serviceability due to its elastic brittle behavior, limited rupture strains, and lower modulus of elasticity. The near linear elastic nature of FRP leads to very limited energy dissipation capability in FRP reinforced concrete (RC) structures. Lack of ductility and energy dissipation has restricted FRPs' use in seismic applications (Harris et al. 1998). To address the issue of ductility and energy dissipation in FRP RC structures, this study investigated the use of a new type of composite, known as shape memory alloy-FRP (SMA-FRP) as reinforcement for RC structures. SMA-FRP reinforcing bar comprises polymeric resin reinforced with small diameter superelastic SMA fibers. The typical flag shape hysteresis associated with superelastic SMAs is a direct result of a reversible stress-induced phase transformation between austenite and martensite phases (See fig. 1(a)). The nonlinear, yet pseudo-elastic behavior typical of SMA fibers (Naito et al. 2001) will allow SMA-FRP composite reinforcement to exhibit hysteretic and ductile behavior with minimal damage to the RC structure. This is not likely in steel reinforcement, which exhibit ductility through permanent damage to the steel and concrete which often yields significant residual (permanent) deformations in the structure. Thus, SMA composite reinforcement can both capture the non-corrosive properties typical of FRP reinforcement and at the same time exhibit the ductility similar to steel with negligible residual drifts as achieved in FRP.

SMAs have been known for decades, but they have not been used much in the building industry until recently (Janke et al. 2005). The superelasticity and shape memory phenomena typical of SMAs

encouraged many researchers to study the application of SMAs in civil structures (e.g. DesRoches. et al. 2004, Nehdi et al. 2009, Shin and Andrawes 2010, among others). Among the researches that are relevant to this present study are the studies by Saiidi et al. (2007) and Nehdi et al. (2009), who investigated the use of SMAs in the plastic hinge zone of a RC beam. Unlike this study, these researchers used large diameter rods made of NiTi SMAs as reinforcement. The use of large diameter SMA bars reduces the hysteretic area of the SMA (Saiidi et al. 2007) and thus impacts its ability to dissipate energy compared to small diameter wires. Moreover, manufacturing large diameter SMA rods is extremely expensive compared to small diameter wires, which makes use of large SMA bars in real applications cost-prohibitive. To overcome these problems, the SMA-FRP hybrid reinforcing bars studied in this paper comprises small diameter superelastic NiTi SMA wires with or without FRP bonded together using a polymeric resin matrix, as shown in fig.1(b). The primary focus of the study is to explore the use of SMA-FRP hybrid bars as reinforcement in RC moment resisting frame (MRF) structures subjected to earthquakes. These composite reinforcements were first manufactured and tested in the laboratory. The performance of MRF, which were first designed and then subjected to incremental seismic analyses was analytically compared between use of steel, conventional GFRP and SMA-FRP reinforcement types.



Figure 1. Schematic drawing of (a) Stress-strain hysteresis of superelastic SMA. (b) SMA-FRP hybrid composite rebar cross-section.

# 2. EXPERIMENTAL MANUFACTURING AND TESTING

#### 2.1. Material Selection

The host material selected for use in manufacturing of epoxy resin with high elongation properties. The resin system selected for manufacturing composite are suitable for use in high performance applications such as coatings, adhesives, civil engineering, electronic and electrical insulation. For reinforcing material, S-glass FRP was selected for use in hybrid composite because of their improved stiffness and elongation properties over carbon and aramid FRP. Finally Nickel Titanium (NiTi) SMA wires with diameter of 500µ m were used as primary reinforcement in addition to S-glass fibers in the composite coupon specimens. Several studies (Gong et al., 2002) have shown decrease in the forward transformation stress (loading plateau) and decrease in the residual strain (increase in residual elongation / displacement) with continued cycling. Once the SMA wires were trained to give stable properties, they were embedded in resin matrix to produce two composite specimens with varying reinforcement ratio. Fig. 2 shows methodology utilized for preparation of SMA-FRP composite

specimen. The HSMA-3 and HSMA-7 specimens had 3 & 7 SMA wires with reinforcement ratio of 8.46 and 20.3%, respectively. Another composite specimen (SFHC-3) with S-glass fiber in addition to SMA wires was manufactured and tested.





# 2.2. Uniaxial Cyclic Testing

For the purpose of testing all the dog bone specimens and for SMA wire training, a 97 kN uniaxial servo-controlled hydraulic frame was used. The frame was attached with hydraulic pump, actuator, load cell and mechanical grips. A uniaxial tensile loading and unloading protocol was developed in order to find the tensile properties of the material and composite. In the loading protocol, both loading and unloading was governed by strain control, based off of values of elongation being read by extensometer with constant strain rate of 0.254 mm/min (0.0083 Hz). The testing protocol was programmed to have each tensile cycle to be 1% strain increment. Results from quasi-static tensile test of four specimens, namely M10 (pure resin), HSMA-3, HSMA-7 (Only SMA wires were used as reinforcement) and SFHC (both SMA and GFRP was used as reinforcing material) is shown in fig. 3.



Figure 3. Cycle wises comparison of stress strain curves of HSMA-3, HSMA-7 and M10 resin and SFHC-3 specimens (a) 1% strain cycle (b) 2% strain (c) 3% strain (d) 4% strain

# **3. ANALYTICAL MODELING**

#### 3.1. Prototype MRF

A 2-D three storey one bay (3S1B) and six storey two bays (6S2B), reinforced concrete MRF were designed to investigate the behavior of steel, GFRP and SMA-FRP composite reinforcement. Finite element program, OpenSees (Mazzoni et al. 2009) which has been specifically designed for seismic analysis and earthquake simulations, was utilized to perform the inelastic analyses on the MRF structures considered in this study. Fiber section approach and nonlinear beam-column elements were utilized to define the cross-section and the elements, respectively. Cross-sections are modeled by defining geometric and material properties, while fibers form the basis of distributed plasticity models. The cross section is discretized into fibers, which allows the section to be further divided into small areas with different constitutive models. In order to restrict the cost of material associated with use of NiTi in SMA-FRP composite, the reinforcing composite is only provided in the plastic hinge zones of MRF where high inelasticity is expected to accumulate. Generally, the length of plastic hinge varies between 1.2h to 2h, where h is the depth of the section (Paulay & Priestley, 1992). Mechanical couplers can be used to tie the two different types of reinforcement after their cutoff points outside the plastic hinge zone, which for this study was kept constant to be 1.5h. Also in this study it is assumed that no slippage occurs at the mechanical coupler and a perfect bond exists between all reinforcement types and concrete material. Fig. 4 shows analytical modeling technique used in this study.



Figure 4. Analytical modeling technique utilized in OpenSees.

## 3.2. Material Properties and Modeling

Uniaxial material models have been defined in OpenSees to capture the constitutive behavior of concrete and reinforcement materials. Concrete02 model, which considers concrete's tensile strength, was selected to represent the concrete behavior. The constitutive behavior of confined concrete was considered by using the model developed by Mander et al. (1988). For unconfined concrete, a compressive strength of 30 MPa and a strain of 0.002 mm/mm corresponding to peak compressive strength were assumed. Base model for confinement was developed for steel reinforcement and was then kept the same for the sake of consistency and comparison with SMA-FRP and GFRP sections. Three different reinforcing materials were used in this study namely, steel, SMA-FRP, and GFRP. The steel behavior was described using Steel02 model predefined in OpenSees, which is based on the Giuffre-Menegotto-Pinto model with isotropic strain hardening (Menegotto and Pinto 1973). GFRP and epoxy resin were modeled using elastic and elastic perfectly plastic material models, respectively, while SMA material was modeled using flag shape 'SMA' uniaxial material, predefined in OpenSees. For analytical modeling, it was assumed that both SMA-FRP and GFRP composite reinforcement constitutes of 65% fiber and 35% resin in terms of volumetric ratio. Parallel material command was used to join the epoxy and SMA / FRP material models, in which the strains are equal while stresses and stiffness's are additive. Grade 60 steel with modulus of elasticity of 200 GPa was used to design the structure. For GFRP, rupture strain of 0.034 mm/mm and modulus of elasticity of 65 GPa was used. Transformation stresses values were used for NiTi, austenite to martensite start stress,  $\sigma_{AMS}$  = 550 MPa, austenite to martensite finish stress,  $\sigma_{AMF} = 560$  MPa, martensite to austenite start stress,  $\sigma_{MAS}$  = 145 MPa, martensite to austenite finish stress,  $\sigma_{MAF}$  = 138 MPa. The modulus of elasticity of the NiTi SMA fiber was assumed to be 69 GPa. Experimental results were used to calibrate the analytical models used in this study which were capable of predicting closely, the initial stiffness, rupture stress and strain, and the post-rupture behavior, achieved from experimental results.

#### 4. DESIGN OF PROTOTYPE MOMENT RESISTING FRAMES

Reinforced concrete MRFs (3S1B & 6S2B) were designed to investigate the behavior of steel, GFRP and SMA-FRP composite reinforcement. Initially, steel reinforcement was used to design the frame, as all the guidelines for design are available for concrete members reinforced with steel rebars. Later, steel reinforcement was replaced by SMA-FRP and GFRP composite rebars. In order to restrict the cost of material associated with use of NiTi in SMA composite, the reinforcing composite is only provided in the plastic hinge zones of MRF where high inelasticity is expected. Loads were considered

on the structure assuming it to be an office building with values of loads based on American Society of Civil Engineers (ASCE-7) standard guidelines (ASCE 2005). International Building Code (IBC) (IBC) 2006) was used to compute the base shear for the structure. Equivalent static force (ESF) procedure, as described in IBC-2006 and ASCE 7 was used to distribute the design base shear along the height of the structure. Structural analysis was performed to obtain the design moment and shear with the special seismic load combination allowed by IBC-2006. An iterative process was carried out next using the design response spectrum to determine the area of the SMA-FRP and GFRP reinforcement using moment curvature analyses. Based on M- $\phi$  curves, capacity of the sections was matched to the desired capacity (demand) and reinforcement ratios were determined. M- $\varphi$  analysis revealed that the steel cross section has been designed for higher moment as compared to SMA-FRP and GFRP cross sections due to higher stiffness. Due to this inherent variation in stiffness of the three reinforcements, the frames were not designed for same seismic force levels. ACI code was used to ensure that the designed section is not critical in shear. Same cross section was used for the entire length of beam and column in all the floors. Beam cross section was 300mm x 500mm, while the square column had a dimension of 400 mm. The procedure for determining the reinforcement ratio of SMA-FRP and GFRP is explained in the flow chart shown in fig. 5.



Figure 5. Design procedure for frame with SMA-FRP and GFRP composites.

# 5. INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis (IDA) method was used in this study which involves subjecting a structural model to one or more ground motion records. Each record is then scaled to multiple levels of intensity, thus producing one or more load displacement curves (Vamvatsikos and Cornell, 2001). IDA curves for steel, SMA-FRP and GFRP frames were developed to find the corresponding PGA which would cause the crushing of core concrete in the column or rupture of reinforcement whichever is earlier. Four ground motion records were used to conduct nonlinear time history analysis on the MRFs. These selected natural earthquake records and their characteristics are shown in table 1. The four ground motion records were selected to have variation in predominant period as well as the frequency content of the record, in order to assess variation in response of MRFs. The ground motion records represent a sample of earthquake with low, medium and high frequency content. This variation was considered critical in evaluating response of the 3S1B MRF under various seismic hazard parameters.

Earthquake	Station	PGA(g)	Effective Duration	<b>Predominant Period</b>
2005 Kashmir	Abbotabad	0.236	60.01 sec	0.85 sec
1994 Northridge	24278 Castaic	0.516	29.97 sec	0.55 sec
1978 Tabas	9101 Tabas	0.835	32.84 sec	0.24 sec
1999 Kocaeli	Kocaeli	0.219	29.995 sec	0.31 sec

Table 1. Characteristics of selected ground motions.

#### 5.1 Maximum Inter-Storey Drift Ratio

From the analyses results, IDA curves were generated for 3S1B, which are a plot between maximum inter-story drift (ID-%) and the PGA (g) for each record as shown in fig. 6. Fig. 6(a) which is the

response of 3S1B subjected to Kashmir ground motion shows that for the lower value of PGA, steelreinforced frame shows less ID values as compared to both SMA-FRP and GFRP reinforced frames. This could be attributed to lower stiffness of both SMA-FRP and GFRP material as compared to steel. However, after a PGA of about 0.6g, SMA-FRP-reinforced frame exhibited less ID values as compared to frame with steel reinforcement. At a PGA value of 1.2g, SMA-FRP frame exhibited less ID by 7%, despite having 60% less initial modulus of elasticity as compared to steel. On the other hand GFRP-reinforced frame experienced much higher ID as much as 21% as compared to steel and SMA-GFRP frame despite the fact that it was able to sustain lesser PGA intensity before collapse. Fig. 6(b) which is the response of frame subjected to Northridge ground motion also exhibits high ID values for GFRP-reinforced frame. However, for this record the steel and SMA-FRP-reinforced frames showed comparable ID values at same PGA. In this case GFRP-reinforced frame exhibited much higher ID as compared to both steel and SMA-FRP. Fig. 6(c) is the response of frames subjected to Tabas ground record. The results show that the steel-reinforced frame experienced an ID of 4.16% at a PGA of 2.42g when core concrete in columns started to experience crushing. However SMA-FRPreinforced frame experienced maximum ID of 4.2% before reaching collapse at a PGA of 2.08g. GFRP-reinforced frame, which experienced ID of 4.8%, suffered collapse at a lower PGA of 1.54g due to rupture of GFRP within plastic hinge zone. Similar results were observed for the response of 3S1B frame subjected to Kocaeli ground motion record. Maximum drifts experienced by steel, SMA-FRP and GFRP reinforced frames were 3.24%, 4.16% and 4.8%, respectively, before collapse. Higher IDs in GFRP frames are attributed to lower material stiffness and sudden rupture of GFRP in beams. By running IDA technique, the collapse mechanism (yielding of reinforcement and crushing of core concrete at the same location) was corresponded to maximum PGA which the frames can sustain just before their collapse (or causing collapse). From the results it can be stated that steel reinforced frames were able to sustain higher maximum PGA for all four earthquake records because of higher ductility and strength. The average increase in maximum PGA for 3S1B frame reinforced with steel and SMA-FRP was 42% and 20% respectively, as compared to GFRP reinforced frame. Similar results were observed for 6S2B frames.



#### 5.2 Residual Inter-Storey Drifts

During any seismic event structures which get severely damaged, exhibit permanent deformations. For MRFs permanent deformations are generally quantified in terms of residual drifts. Residual inter-story drifts (IDs) are therefore a good signal of the level of permanent damage sustained by the structure and are sourced by nonlinear behavior and yielding of reinforcement, development of cracks in concrete

and crushing of concrete within plastic hinge zone. Fig. 7 shows the residual ID plots of 3S1B frame, when subjected to the four earthquake records. By comparing the residual ID values, it was observed that, SMA-FRP-reinforced frame on average exhibit about 84% less residual ID compared to 3S1B steel reinforced frame. Frame with steel reinforcement initially experienced less residual ID as compared to SMA-FRP and GFRP reinforced frames till the PGA range of 0.4g-0.5g due to higher stiffness. However once the steel reinforcement started to yield, it developed higher residual ID, for the same value of PGA, as compared to SMA-FRP reinforced frame. Because SMA exhibit almost negligible amount of residual strains after training process, the behavior of frame with SMA-FRP composite reinforcement in plastic hinge zone was found to be much superior. On the other hand, GFRP reinforced frame, which reached collapse limit state at a lower PGA value, exhibited on average 49% higher residual inter story drift values as compared to SMA-FRP reinforced frame but 62% lesser residual inter story drift values as compared to steel reinforced frame. Irrespective of frequency content of earthquake excitation, SMA-FRP-reinforcement showed much better performance in ways to curb damage by recovering the applied strains. This tendency and property to re-center itself after experiencing inelasticity is the hallmark of SMA material and is that which distinguishes it from GFRP and steel as reinforcing material.



#### 5.3 Dissipated Hysteretic Energy

The ability of a structure to dissipate energy during a seismic event is also an important desirable seismic parameter. This dissipated energy is the direct result of damping and hysteretic nature of the material. But energy alone will not be enough indication to show the desirable characteristic of SMA-FRP reinforcement. In order to illustrate the concept, hysteretic behavior of 3S1B frame reinforced with steel, SMA-FRP and GFRP when subjected to Kashmir earthquake record is shown in fig. 8. This figure is a plot between roof drift (ratio between displacement and height of frame) and base shear reaction. The response shown in the figure is governed by the hysteretic nature of material response and is shown for the PGA level causing failure in GFRP frame. Fig. 8 (a) clearly shows superior performance of steel reinforcement followed by SMA-FRP composite, in terms of energy dissipation, which is calculated, based off of cumulative area enclosed within the displacement vs. base shear plot. This improved potential comes from the fact that the stress-strain curve of steel and SMA-FRP contains more hysteretic area than GFRP. Steel and SMA-FRP reinforced frame when compared with GFRP frame showed 108% and 65% more energy dissipation, respectively. As mentioned before, the response of the three frames should be seen in light of energy dissipated and the permanent residual drifts, as both parameters govern the desirable response of a structure under seismic loading. Fig. 8

shows that although steel frame was able to dissipate much more energy as compared to other two frames, it suffered residual drifts of 4.85% measured at the roof level. On the other hand, GFRP frame suffered 1.8% permanent drifts and failed at much lower PGA. Fig. 8 (b) shows the main benefits of using SMA-FRP composites which is their ability to dissipate energy with minimal permanent damage / residual drifts (0.82% residual drift).



Figure 8. Base Shear vs. roof drift for MRF subjected to Kashmir earthquake record.

## 6. CONCLUSIONS

This study focused on the use of SMA-FRP composite as a new reinforcement for RC MRFs to improve their seismic behavior and reduce their residual drifts after an earthquake. SMA Composite specimens were prepared / tested in the laboratory and results were used to calibrate analytical material models. 3S1B and 6S2B prototype MRFs were designed and analyzed with steel, SMA-FRP, and GFRP rebars at the plastic hinge regions. IDA technique was used to study the seismic behavior of the frame under four different earthquake records. It was found that initially the frame with steel reinforcement experience less IDs as compared to the frames reinforced with SMA-FRP and GFRP, because of its higher initial stiffness. However, when steel starts yielding, the response tends to flip and steel frame experiences more ID values as compared to frame with SMA-FRP composite reinforcement. GFRP frames experienced more ID levels as compared to SMA-FRP frame due to rupture of GFRP reinforcement in the beams. Steel reinforced frames were able to experience more PGA value before reaching ultimate limit state of crushing of concrete in the column. GFRP reinforced frames were able to sustain least value of ground motion intensity and in all the cases failed at earlier stage of IDA as compared to frames reinforced with other two reinforcement types. It was also found that frame with steel reinforcement was able to dissipate more energy and showed higher initial stiffness as compared to frames with SMA-FRP and GFRP reinforcement, but at the cost of residual ID. Frame with steel reinforcement experienced 84% and 62% more residual IDs as compared to SMA-FRP and GFRP reinforcement, respectively. The SMA-FRP reinforced frame was not only able to dissipate more energy compared to GFRP frame for the same PGA values, but also showed much lesser residual ID values as compared to frames with steel and GFRP reinforcements. This study showed that the use of SMA-FRP rebars in the plastic hinge zones of MRFs improves significantly the ductility, energy dissipation, and residual drifts compared to GFRP reinforced frames, thus improving the overall performance of frame under seismic hazard.

### REFERENCES

- DesRoches R, McCormick J and Delemont M 2004 Cyclic properties of superelastic shape memory alloy wires and bars J. Struct. Eng. ASCE 130 38–46
- Gong, J. M., Tobushi, H., Takata, K., Okumura, K., and Endo, M. (2002). "Cyclic superelastic deformation of TiNi shape-memory alloy." *Materials Science Forum*, **394-395**, 245–248.
- Harris H G, Somboonsong W and Ko F K 1998 New ductile hybrid FRP reinforcing bar for concrete structures J. Compos. Constr. 2 28–37
- Janke L, Czaderski C, Motavalli M and Ruth J 2005 Applications of shape memory alloys in civil engineering structures-overview, limits and new ideas *Mater. Struct.* **38** 578–92
- Mander J B, Priestley J N and Park R 1988 Theoretical stress strain model for confined concrete J. Struct. Eng. 114 No. 8
- Mazzoni S et al 2009 Open System for Earthquake Engineering Simulation (OpesSees). *OpenSees Command Language Manual* (Berkeley, CA: Pacific Earthquake Engineering Research Center, University of California)
- Menegotto M and Pinto 1973 Method of analysis for cyclically loaded RC plane frame including changes in geometry. *Preliminary Report IABSE* vol **13**, pp 15–22
- Naito H, Sato J, Funami K, Matsuzaki Y and Ikeda T 2001 Analytical study on training effects of pseudoelastic transformation of shape memory alloys in cyclic loading *J. Intell. Mater. Syst. Struct.* **12** 295–300
- Nehdi M, M S Alam, M A Youssef 2009 Development of corrosion-free concrete beam column joint with adequate seismic energy dissipation. *Engineering Structures* **32** (2010) 2518\_2528
- Paulay T and Priestley M J N 1992 Seismic Design of Reinforced Concrete and Masonry Buildings (New York: Wiley)
- Said A M and Nehdi M L 2004 Use of FRP for RC frames in seismic zones: part II. Performance of steel free GFRP reinforced beam-column joint *Appl. Compos. Mater.* **11** 227–45
- Saiidi M, Zadeh M, Ayoub C and Itani A 2007 A pilot study of behavior of concrete beams reinforced with shape memory alloys *J. Smart Mater. Civ. Eng.* ASCE **19** 454–61
- Shin M and Andrawes B 2010 Experimental investigation of actively confined concrete using shape memory alloys *J. Eng. Struct.* **32** 656–64
- Vamvatsikos D and Cornell A C 2001 Incremental dynamic analysis Earthq. Spectra 20 523-53