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CYCLIC BEHAVIOR OF POST-TENSIONED STEEL CONNECTIONS WITH SHAPE MEMORY ALLOY ANGLES

Rahmzadeh, Ahmad[1,3], Alam, M Shahria[2]

[1] Ph.D. Student, School of Engineering, The University of British Columbia, Canada

[2] Associate Professor, School of Engineering, The University of British Columbia, Canada

[3] a.rahmzadeh@ubc.ca

Abstract: Following the 1994 Northridge earthquake, a number of code-designed steel moment frame buildings experienced unanticipated brittle fracture in beam-column connection areas. Reinforcing beam flange and using reduced beam sections were some solutions to localize damage at a pre-defined region. Although such configurations keep the damage away from critical areas, the residual drift after experiencing large deformations is inevitable. Post-tensioned steel connections are meant to overcome this problem. In such innovative connections, high strength steel strands are utilized and post-tensioned to provide a connection with a self-centering feature. Bolted top and seat angles can be added to these systems to serve as ductile components and dissipate energy. Shape memory alloys (SMAs), with excellent superelastic property and flag-shaped hysteresis, can be used to reduce localized damages in the angles. In this paper, three-dimensional finite element analyses are conducted to study the seismic performance of the post-tensioned steel connections with angles made of SMA material.

1 Introduction

In current practice, structures are designed so as to resist earthquakes by dissipating energy through localizing damage in ductile structural components. However, although the “Life Safety” performance level may be satisfied by the structure during an earthquake, it may suffer severe damages and exhibit substantial residual drift which harms the functionality of the structure after the earthquake. Repairs or in severe cases demolition may be required for the structure to become functional. Miranda (2010) noted that in ductile structures the amount of permanent residual drift, instead of collapse, is the most determinative factor in economic losses. In other words, when exposed to strong ground motions, ductile structures are highly resistant to collapse, but, there is a significant probability of being demolished after an earthquake due to residual drifts. It is often more economic to demolish, rather than to repair a building with large residual drifts, even if the structural damages can be repaired. If the earthquake is strong enough, many structures in a region experience damages which can significantly impact societies through economic losses.

Enormous economic losses in the 1994 Northridge and 1995 Kobe earthquakes was a wakeup call for a whole new look into structural design philosophies. As a response to this, extensive research has been conducted in the recent years to develop higher-performance structural systems, known as self-centering structures, with the ability to limit the damage. Housner (1963) showed that the survival of a number of tall structures in the Chilean earthquakes of May 1960, was because of their rocking motion, and since then, rocking systems have been studied for buildings and bridges. The behavior of these rocking structures is characterized by gap opening and closing at the base of the structure or at pre-specified joints which provide substantial deformation capacity.

Ricles et al. (2001; 2002) extended the concept of self-centering, which had been previously developed for precast concrete connections, to moment resisting steel connections. Instead of welding, they assembled beams and column by means of post-tensioned strands and used top and seat bolted angles as a source of energy dissipation. Therefore, the beam deforms elastically when moment is applied. As the bending moment increases, the force in the flange under tension increases as well. As soon as the flange tension exceeds the initial pre-compression, a gap will form and continue to grow with moment being increased. This gap opening behavior provides a large ductility capacity before the occurrence of yielding in the beam or the panel zone. The forces in the post-tensioned strands restore the beam-column connection to its original configuration when the applied moment is removed, and the formed gap will disappear. Consequently, nine large-scale post-tensioned steel moment connection subassemblies with different configurations were built and tested by Ricles et al. (2002) under cyclic lateral loading. The results of their study show that the post-tensioned connection had adequate initial stiffness, strength, re-centering capability and energy dissipation pertinent to the angle sizes (Ricles et al., 2002). Garlock et al. (2005) tested six full-scale specimens to investigate the effect of the initial post-tensioning force, the number of post-tensioned strands, and the reinforcing plate length. Their experimental work demonstrated the stable hysteretic behavior of the specimens before the occurrence of local buckling in the beams and yielding in the strands.

Other forms of energy dissipators have been used by researchers. Christopoulos et al. (2002) utilized buckling restrained bars that are able to yield in axial tension and compression to provide energy dissipation for the post-tensioned steel beam-column connections. Good energy dissipation and large deformation capacity, while keeping the beam and column undamaged, were reported (Christopoulos et al., 2002). Rojas et al. (2005) developed a post-tensioned friction connection in which a slotted shear tab was utilized to connect the beam web to the column, and two friction plates were placed on the top and bottom of the beam flanges. The results of their nonlinear dynamic time history analyses under earthquake ground motion input demonstrated the significant re-centering capability of the proposed connection (Rojas et al., 2005). To avoid interference with a floor slab, Wolski et al. (2009) developed a post-tensioned connection with a beam bottom flange friction device. Their experimental results were representative of the reliable energy dissipation of the connection, but with an asymmetric response (Wolski et al., 2009).

In addition to experimental studies, various researchers have investigated the behavior of post-tensioned connections through analytical studies. Kim and Christopoulos (2009) used the ANSYS finite element package to model a tested post-tensioned steel connection with frictional devices on the beams. Deng et al. (2013) first tested four full-scale post-tensioned steel connections with bolted angles, and then modeled their specimens using the ABAQUS finite element package. Vasdravellis et al. (2013) first developed an hourglass pin as the energy dissipator, and then investigated the cyclic behavior of self-centering steel connections with such dissipators through experimental studies and finite element modeling. Moradi and Alam (2015) simulated the specimens tested by Ricles et al. (2002) in ANSYS and examined the effect of reinforcing plate sizes, the material property of the angles and the initial post-tensioning forces.

Among the different energy dissipators proposed by researchers for the post-tensioned steel beam-column subassemblies, the bolted steel angles are superior regarding fabrication and installation. However, they are severely damaged after cyclic loading and should be removed. Due to the capability of eliminating permanent deformations, shape memory alloy (SMA) has been gaining popularity in civil engineering (Alam et al., 2007). To further investigate the possibility of minimalizing damage in self-centering steel connections, in this contribution, SMA is used as the constitutive material of the angles.

2 Validation study

To validate or verify the finite element procedure, the specimen PC4 tested by Ricles et al. (2002) was modeled in ANSYS (2016). Figure 1 illustrates the connection details for PC4 which is an interior beam-column connection. The subassembly is comprised of a W14×311 wide flange column and two W24×62 wide flange beams.

Shim plates with the dimension of 275×254×9.5 mm were placed between the angles and beam flanges and the column. To increase the bending strength of the beams, 254×57×12.7 mm reinforcing plates were welded to the beam flanges. A size of *L*203×203×15.9 mm was used for the angles with a gauge length-to-thickness ratio of 4. The angles were bolted to the beams and column with A325 (25.4 mm) bolts. Four high strength strands each with an area of 140 mm^2 on each side of the beam were utilized to post-tension the beams. Yield stress σ_y and tensile strength σ_u for the steel materials are given in Table 1.

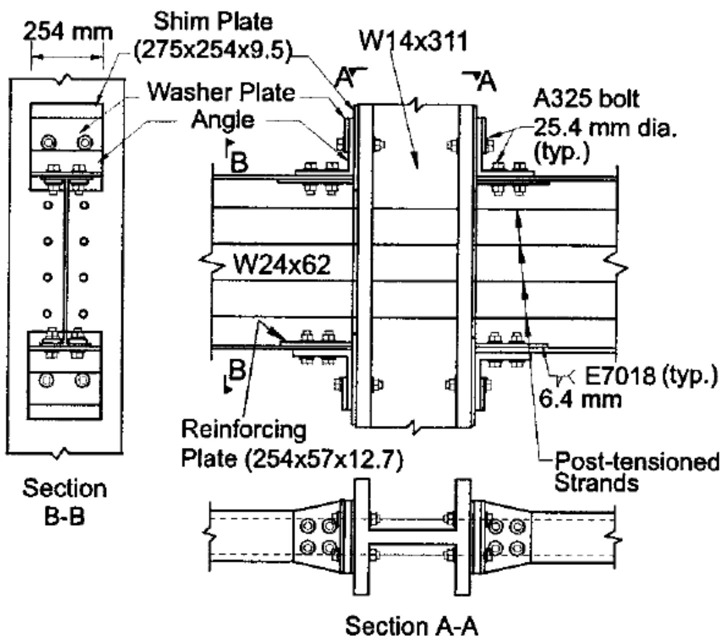


Figure 1: Specimen PC4 (Ricles et al., 2002).

Table 1: Material Properties (Ricles et al., 2002).

Stress*	Beam flange	Beam web	Angle	Reinforcing plate	Shim plate	Post-tensioned strand
σ_y	230	266	263	843	843	1305
σ_u	421	450	465	895	895	1864

* Units: MPa

A three-dimensional model of PC4 was developed in ANSYS. Since symmetry conditions were applicable, half of the specimen was modeled. The solid model was meshed using the SOLID185 element which has eight nodes with six degrees of freedom at each node. Frictional contacts with a static friction coefficient of 0.4 were defined between the bolt nut and washer plate, the washer plate and the angle, the angle and the shim plate, the shim plate and column, the bolt head and the column, the bolt head and the angle, the angle and beam flange, the reinforcing plate and the bolt nut, the strand and the column, the beam and strand nut. Also bonded contacts were used between the reinforcing plate and the beam flange. For the boundary conditions, roller supports were defined at the beam ends and also pin support conditions were applied at the base of the column. Pre-tensioning forces in the bolts and post-tensioning forces in the strands were applied using the PSMESH and SLOAD commands. A more detailed explanation of the modeling procedure can be found in Moradi and Alam (2015). The following modifications were made to Moradi and Alam's (2015) procedure:

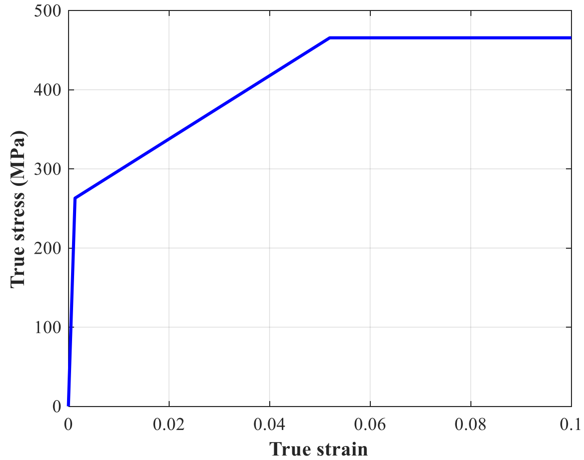
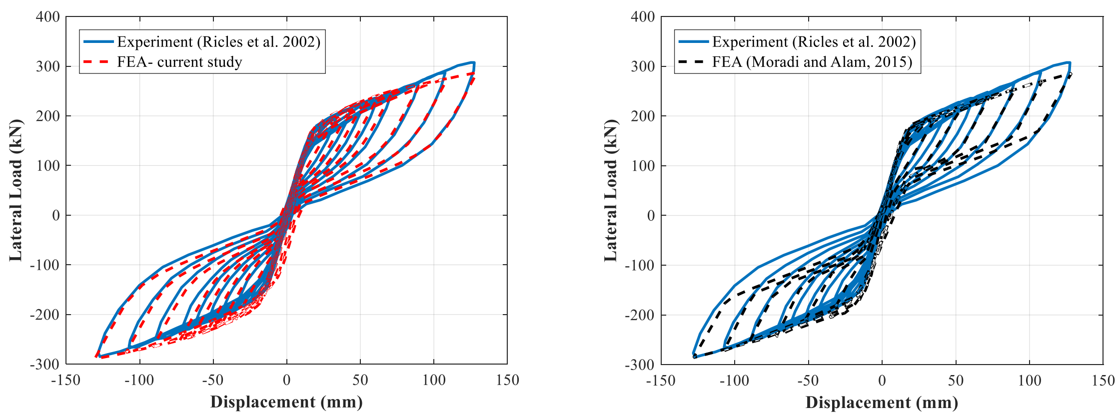


Figure 2: Considered material model for the angles.

- A trilinear kinematic material model, as shown in Figure 2, was used for the angles. The reason for that is that the bilinear kinematic model has no physical meaning in large strains, which is the case here, and gives incorrect results.
- Since there was no failure in the bolts in the actual test by Ricles et al. (2002), the bolt diameter was not reduced.

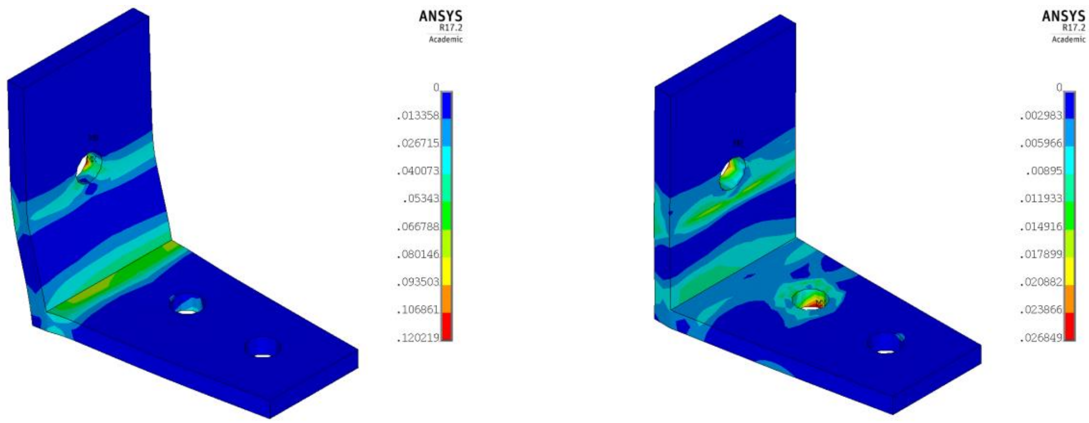
Adapting the modifications above, the analytical results were enhanced as can be seen in Figure 3.



(a) (b)

Figure 3: Analytical results in comparison with the test results: (a) current study (b) Moradi and Alam (2007)

Figure 4 shows the Von Mises plastic strain at the largest drift and at the end of analysis. The residual plastic strains indicate that the angles are damaged and should be removed.



(a) (b)

Figure 4: Plastic strain at the: (a) largest drift (b) end of analysis

3 Post-tensioned Steel Connections with SMA angles

To eliminate the residual strain, an SMA with the superelasticity effect was used as the material for the angles. Figure 5 illustrates the stress–strain curve for a superelastic SMA. The properties of the considered SMA materials are given in Table 2, in which ε_s is the superelastic plateau strain length, E the elasticity modulus, f_y the austenite to martensite starting stress, f_{p1} the austenite to martensite finishing stress, f_{T1} the martensite to austenite starting stress and f_{T2} the martensite to austenite finishing stress.

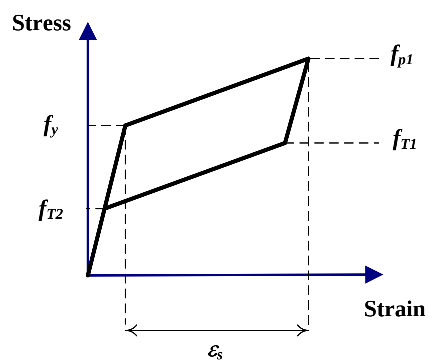


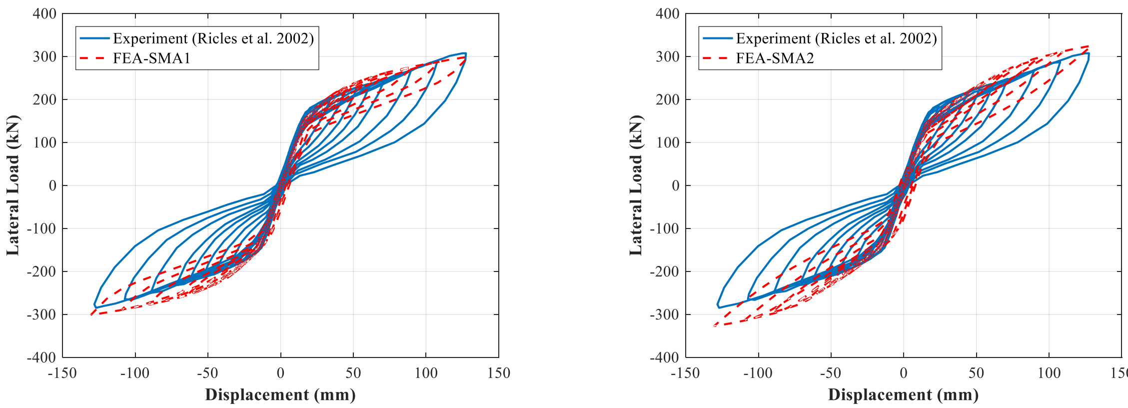
Figure 5: Superelastic SMA stress–strain response.

Table 2: SMA Properties*

SMA type	Alloy	ε_s	E	f_y	f_{p1}	f_{t1}	f_{t2}	Reference
SMA1	$NiTi_{45}$	0.08	68000	435	535	335	170	Ghassemieh et al. (2012)
SMA2	FeNiCuAlTaB	0.135	469000	750	1200	300	200	Tanaka et al. (2010)

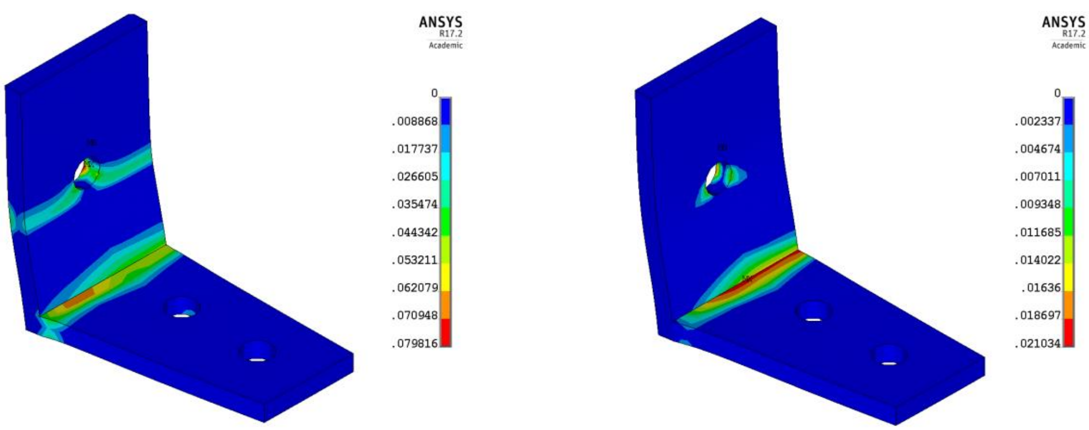
* Units: MPa

Figure 6 demonstrates the lateral load-displacement behavior of PC4 with the SMA angles. In addition, the Von Mises plastic strain in the angles corresponding to the largest drift is shown in Figure 7. The results of analyses showed no residual strain in the angles after the cyclic loading, since the strain did not exceed the plateau strain length. However, the energy dissipation capacity of the specimens with the SMA angles was shown to be less than the ones with steel angles. The total amount of dissipated energy for PC4 is 99.2 kN.m while for SMA1 and SMA2 are 35 kN.m and 33.8 kN.m, respectively. Although it was expected that SMA2 would dissipate more energy than SMA1, the results show that it dissipated somewhat less energy. The reason behind this is its high yield strength which resulted in experiencing low plastic deformations as is shown in Figure 7. It should be noted that the cumulative energy dissipation was calculated based on a single cycle at each drift level.



(a) (b)

Figure 6: Force-displacement response of (a) PC4 with SMA1 angles, (b) PC4 with SMA2 angles



(a) (b)

Figure 7: Plastic strain at the largest drift for (a) PC4 with SMA1 angles, (b) PC4 with SMA2 angles

4 Conclusion

In this paper, a 3D finite element model of a previously tested post-tensioned steel connection by Ricles et al. (2002) was generated and analyzed under cyclic loading. The analytical results showed a good agreement with those of the actual test. The steel angles, as the source of energy dissipation, experienced damages during the cyclic loading. To eliminate the residual damage, the superelastic SMA material was used for the angles. It was found that although no plastic strains remained in the SMA angles after the cyclic loading, the dissipated energy was not as good as the steel angles. So, in regions of high seismicity where energy dissipation is a matter of concern, the use of this type of dissipator is not recommended.

5 Acknowledgements

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