PLASTIC LIMIT ANALYSIS OF SELF-CENTERING STEEL MOMENT RESISTING FRAMES WITH POST-TENSIONED FRICTION DAMPED CONNECTIONS

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ABSTRACT:

This paper introduces the plastic limit analysis of self-centering (SC) steel moment resisting frames (MRFs) with post-tensioned friction damped connections (PFDC) subjected to lateral forces. A PFDC includes post-tensioned (PT) high strength strands running parallel to the beam with friction devices located on the beam flanges. The connection minimizes inelastic deformation to the components of the connection as well as the beams, columns and panel zones, and requires no field welding. Experimental and analytical results show that SC-MRFs with PFDCs have good energy dissipation, self-centering capability, and strength. Past research on SC-MRFs with PFDCs has identified the need to develop a simplified analysis approach to evaluate the actual capacity of this type of system. The paper presents a plastic limit analysis methodology that uses basic concepts of simple plastic theory to estimate the actual lateral capacity of SC-MRFs with PFDCs.

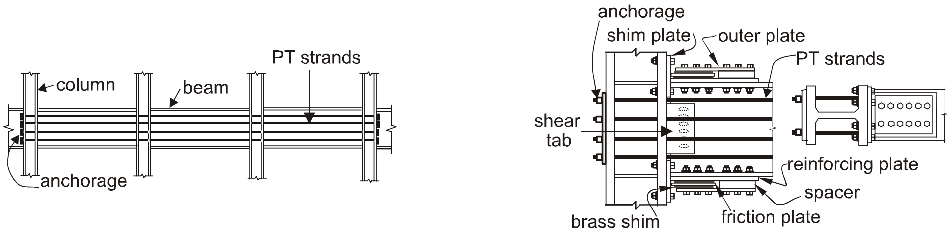
KEYWORDS:

Connections: post-tensioned friction damped connection

1. INTRODUCTION

During the last ten years, extensive research has been carried out on self-centering (SC) steel moment resisting frames (MRFs) in several countries. As an alternative to welded beam-to-column connection construction, the authors developed a post-tensioned friction damped connection (PFDC) for use in seismic resistant steel MRFs. The connection utilizes high strength steel strands [Fig. 1(a)] that are post-tensioned after the friction devices are installed. The post-tensioning strands run through the column, and are anchored outside the connection region [Fig. 1(b)]. A properly designed PFDC has several advantages: (1) field welding is not required; (2) the connection is made with conventional materials and skills; (3) the connection has an initial stiffness similar to that of a typical welded connection; (4) the connection is self-centering without residual deformation, thus the MRF will not have residual drift after an earthquake if significant residual deformation does not occur at the base of the ground floor columns; and (5) the beams and columns remain essentially elastic while the friction devices provide energy dissipation.

Researchers at Lehigh University have conducted experimental studies of SC-MRF subassemblies (Ricles et al. 2002, Garlock et. al 2002), and developed SC-MRF analytical models (Ricles et al. 2001, Rojas et al. 2005). The main objectives of these studies were: (1) to understand the behavior of SC-MRF under cyclic loading, (2) to develop seismic design guidelines of SC-MRFs, and (3) to evaluate the seismic performance and capacity of SC-MRFs under strong ground motion shaking. For the analytical studies, the DRAIN-2DX computer program (Prakash et al. 1993, Herrera et al. 2001) was used to develop analytical models of SC-MRFs in order to determine the actual capacity of this type of systems. However, the development of the analytical model and the corresponding nonlinear analyses are tedious and time consuming. Therefore, there is a need to develop a methodology to carry out a plastic limit analysis in order to more efficiently determine the actual lateral capacity of SC-MRFs with PFDCs.



(a) (b)

Figure 1. Schematic elevation of (a) one floor of a MRF with PFDCs and (b) connection details.

This paper presents a simple plastic limit analysis methodology suitable for hand computations to determine the structural capacity of SC-MRFs with PFDCs. The results of nonlinear static analysis are compared to the results of the plastic limit analysis in order to validate the proposed methodology.

2. RELATED RESEARCH

The development of PFDCs is related to PT steel MRF connections and passive frictional dampers. Experimental and analytical studies were conducted at Lehigh University to investigate the behavior of an innovative PT wide flange beam-to-column moment connection with top-and-seat angles for steel MRFs (Garlock et al. 2002, Ricles et al., 2002; and Ricles et al., 2001). The results show that the PT steel connections can provide adequate strength and stiffness for a MRF subjected to cyclic loading. Petty (1999) carried out preliminary experimental studies of PT connections with passive frictional dampers located at the beam web. Petty's results show that friction is a viable way to dissipate energy in a PT steel connection. Rojas (2003) and Rojas et al. (2005) developed a PFDC connection for new construction, where the details are those shown in Fig. 1. Nonlinear time history analyses of a SC- MRF designed with PFDCs showed that the frame can perform better than a MRF with conventional welded connections, where the former has minimal damage to the members and essentially no residual drift following a severe earthquake.

Rojas et al. (2004) y Caballero et al. (2005) addressed the use of PFDCs for the retrofit of existing steel MRF buildings. Wolski et al. (2006) developed a bottom flange friction device (BFFD) located beneath the beam in order to avoid interference with the floor slab. An experimental program was developed to evaluate the performance of the BFFD in a PT connection. The beam specimen (and reinforcing plates) and post tensioning strands were designed so that no damage to these elements would occur during the test. The behavior of these elements is well understood from prior experimental research by Garlock (2002). Figure 2 shows a photo of one of the specimens tested by Wolski.

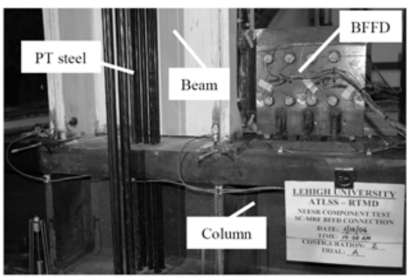


Figure 2. Photo of test specimen: PT connection with a BFFD (Wolski et al. 2006).

3. POST-TENSIONED FRICTION DAMPED CONNECTION

Before introducing the proposed plastic limit analysis methodology to evaluate the structural lateral capacity of a SC-MRF with PFDCs, it is necessary to present the structural concept of this type of system.

3.1 Connection Details

The post-tensioned high strength strands run parallel to the beam and are anchored outside of the connection. Due to the initial post-tensioning force applied to the strands, the beam flanges are compressed against the column flanges [see Fig. 1(a)]. As shown in Fig. 1(b), reinforcing plates are placed on the beam flanges in order to limit beam compression yielding and thus, to minimize structural damage. Shim plates are placed between the column flange and the beam flanges so that only the beam flanges and reinforcing plates are in contact with the column. This enables good contact to be maintained between the beam flanges and column face, while protecting the beam web from yielding under bearing.

Friction devices are located at the beam flanges, consisting of a friction plate sandwiched by two brass shim plates that are inserted between the beam flange reinforcing plate and an outer plate. All plates are bolted to the beam flanges. Long slotted holes are drilled on the friction plate. The shim plate serves as a tee flange that the friction plate is attached to. Friction is generated when the beam flanges and outer plate slide against the stationary friction plate when the beam rotates about the center of rotation situated at the mid-depth of the reinforcing plates (see Fig. 3). The brass shim plates are used to produce a stable friction force and to control the energy capability of the PFDC (Petty 1999). A shear tab with slotted holes is bolted to the beam web and welded to the column flange to transmit the shear forces.

3.2 Flexural Behavior

The moment-relative rotation (M-θ_r) curve for a PFDC when subjected to cyclic loading is shown schematically in Fig. 3. The behavior is characterized by a gap opening and closing at the beam-column interface. The total moment resistance of the connection is provided by the moment due to the friction force and axial force in the beam. The beam axial force in a SC-MRF is from the initial post-tensioning force in the strands, the additional force developed due to elongation of the strands, and from the floor diaphragm of the SC-MRF. For simplicity, the post-tensioned forces are assumed to be acting at the centroidal axis of the beam while the friction forces are assumed to be acting at the mid-depth of top and bottom friction plates.

Under applied moment, the connection initially behaves as a fully restrained connection, where the initial stiffness is similar to that of a fully restrained welded moment connection when θ_r is equal to zero (events 0 to 2 in Fig. 3). Once the magnitude of the applied moment M reaches the moment resistance due to the initial post-tensioning force in the strands, decompression of the beam from the column face occurs. The moment at which this occurs (event 1) is called the decompression moment. The applied moment continues to increase between events 1 and 2 as the rotation of the beam is restrained by the resistance of the friction component. At event 1 the friction force is minimal and increases gradually up to its maximum value at point 2, which is the point of incipient rotation. The maximum value of the friction force is computed using classical friction Coulomb's theory.

The stiffness of the connection after gap opening is associated with the elastic axial stiffness of the post-tensioned strands. With continued loading, the strands elongate producing an additional force, which contribute to resist the total applied moment. Yielding of the strands eventually may occur at event 4. Upon unloading (event 3), θ_r remains constant. At event 5, the friction force is zero. Between events 5 and 6 the friction force changes direction and starts increasing until reaching its maximum value at event 6. Between events 6 and 7, the beam rotates until the beam top flange is back in contact with the shim plate, but not compressed. Between events 7 and 8 the value of the friction force decreases with the beam being compressed against the shim plates and M equal to zero at event 8. A complete reversal in the applied moment will result in a similar connection behavior occurring in the opposite direction of loading, as shown in Fig. 3. Because a residual friction force exists at event 8, the forces in the system are indeterminate until event 2 is again reached. Thus, there is no clear point of decompression on the curve following the first half-cycle.
As long as the strands remain elastic, and there is no significant beam yielding, the post-tensioning force is preserved and the connection will self-center upon unloading (i.e., θ_r returns to zero rotation upon removal of the connection moment and the structure returns to its pre-earthquake position). The energy dissipation capacity of the connection is related to the force developed between the friction surfaces.

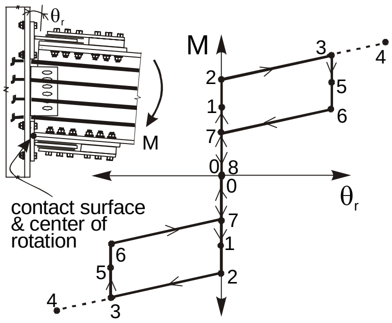


Figure 3. Moment-relative rotation behavior

4. PROPOSED SEISMIC DESIGN APPROACH FOR SC-MRFS WITH POST-TENSIONED FRICTION DAMPED CONNECTIONS

The proposed design approach is a performance based design (PBD) approach where seismic building performance levels are related to expected earthquake ground motion levels. The design approach uses two seismic performance levels defined in FEMA 369 (FEMA 2000b): (1) the "immediate occupancy" performance level, which describes a post-earthquake damage state in which only limited structural and nonstructural damage has occurred; and (2) the "collapse prevention" performance level, which describes a post-earthquake damage state in which the building is on the verge of partial or total collapse. The design approach considers two earthquake ground motion levels defined in FEMA 368 (FEMA 2000a): (1) the design basis earthquake (DBE); and (2) the maximum considered earthquake (MCE). The MCE has a 2% probability of being exceeded in 50 years while the DBE is defined as 2/3 the intensity of the MCE, with an approximate 10% probability of being exceeded in 50 years.

The proposed design approach developed for SC–MRFs with PFDCs (Rojas 2003) has two objectives: (1) to achieve the immediate occupancy level under DBE ground motions; and (2) to achieve the collapse prevention level under MCE ground motions. Under DBE ground motions, the structural components of the PFDC system should not develop inelastic behavior, except for minimal yielding in the beam flanges at the end of the reinforcing plates. As a result, the building is ready to be reoccupied after the DBE. The PT strands and friction devices are designed to remain elastic under MCE ground motions. Some inelastic behavior is expected in the panel zones, beams, and columns, with the onset of local buckling occurring in a few of the beams. With an appreciable amount of local beam buckling, a loss of post-tensioning would occur leading to a degradation in frame capacity. Thus, under MCE ground motions, the frame is expected to loose some of its self-centering capacity, but not collapse.

5. PLASTIC LIMIT ANALYSIS FOR SC-MRFs

5.1 Plastic Limit Theory

The main assumptions in conventional simple plastic limit theory of steel frames are presented in Beedle (1958). They are summarized as follows:

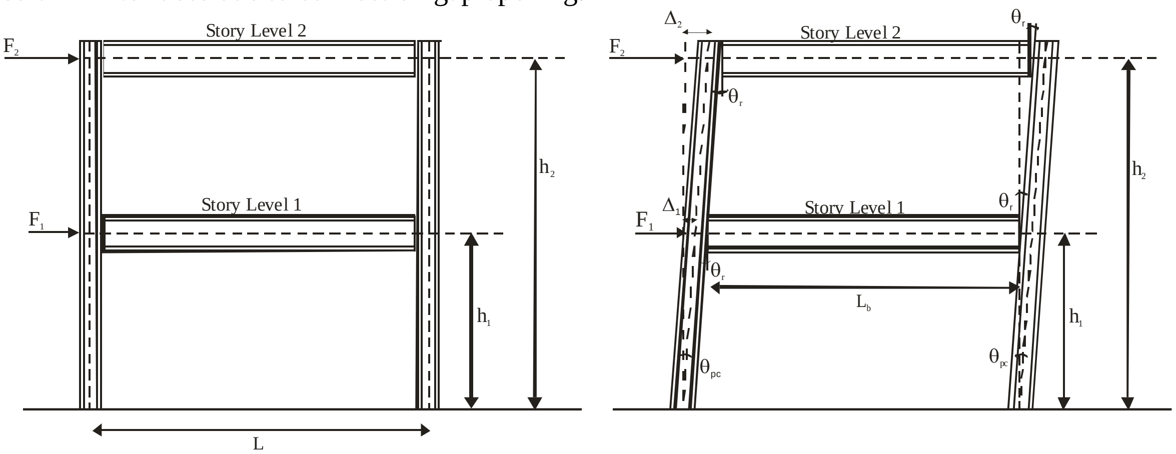
(1) First order deformations are only considered. The deformations are assumed to be sufficiently small so that equilibrium conditions can be formulated for the undeformed structure.
(2) No instability will occur before reaching the plastic limit load.
(3) The connections provide full continuity so that the plastic moment M_p can be transmitted.
(4) The influence of normal forces on the plastic moment M_p are neglected; and
(5) The loading is proportional.

These assumptions combined with a moment-curvature relationship that asymptotically approaches the plastic moment are the fundamentals of the "simple plastic theory".

5.2 Plastic Limit Theory for SC-MRFs

The plastic limit analysis methodology proposed in this paper for SC-MRFs with PFDCs takes into account the same assumptions of simple plastic theory with the following adjustments. First, instead of beam plastic rotations, SC-MRFs with PFDCs develop relative rotations, θ_r, due to the gap opening at the beam-column interface. Second, the connections are designed to reach moments of βM_{pb} to take into account that the beams have axial force due to the post-tensioning in addition to the bending moment. β is a resistance reduction factor that is generally less than to 1.0 and M_{pb} is the beam plastic moment. Third, the influence of the axial force on the column plastic moment M_{pc} is considered introducing a reduction factor α.

Suppose that the beam sidesway mechanism determines the lateral capacity of a SC-MRF with PFDCs. Figure 4(a) shows a two story one bay frame with PFDCs subjected to increasing lateral forces F_1 and F_2. Initially the system behaves as a typical steel frame with fully restrained connections as mentioned in Section 3.2. As the lateral forces increase, the connections develop decompression and with further loading friction is overcome and the connections start developing gap opening. For larger values of F_1 and F_2, the frame deforms as seen in Figure 4(b) where plastic rotations, θ_{pc}, develop at the column bases a relative rotations, θ_r, develop at the beam-column interfaces due to connection gap opening.



(a) undeformed (b) deformed

Figure 4. Two story SC-MRF with PFDCs

Applying the principle of virtual displacements for the deformed structure, it can be stated that:

$$EW = F_1\Delta_1 + F_2\Delta_2 \qquad (1a)$$

$$\Delta_1 = \theta_{pc} h_1 ; \qquad \Delta_2 = \theta_{pc} h_2 \qquad (1b)$$

where EW is the external work done by the lateral forces F_1 and F_2 as they move through the small lateral displacements Δ_1 and Δ_2, θ_{pc} are the column plastic rotations that cause the frame to displace the amounts Δ_1 and Δ_2, and h_1 and h_2 are the heights of the stories 1 and 2 measured from the ground level, respectively. Combining equations (1a) and (1b) the external work can be written as follows:

$$EW = \sum F_i h_i \theta_{pc} \qquad (2)$$

The internal work, IW, developed at the beam-column connections and at the column plastic hinges is:

$$IW = \beta \sum M_{pb}\theta_r + \alpha \sum M_{pc}\theta_{pc} \qquad (3)$$

where all the variables are already defined. θ_r and θ_{pc} are related by the following expression:

$$\theta_r = \frac{L}{L_b}\theta_{pc} \qquad (4)$$

where L and L_b are the centerline and clear beam lengths, respectively. Combining equation (3) with (4), the internal work can be written as follows:

$$IW = \left(\beta \frac{L}{L_b} \sum M_{pb} + \alpha \sum M_{pc} \right) \theta_{pc} \qquad (5)$$

Since the external work must be equal to the internal work according to the principle of virtual displacements, the following equation is obtained:

$$\sum F_i h_i = \beta \frac{L}{L_b} \sum M_{pb} + \alpha \sum M_{pc} \qquad (6)$$

In Equation (6), M_{pb} is taken as Z_bF_y and M_{pc} is taken as Z_cF_y, respectively, where F_y is the specified minimum yield stress of the material, and Z_b and Z_c are the plastic section modulus of the beam and the column, respectively.

5.3 Application of Plastic Limit Theory for a Six Story SC-MRF with PFDCs

To illustrate the proposed methodology, a six story-four bay SC-MRF with PFDCs will be used. The seismic performance of this frame was extensively studied by Rojas (2003). The design of the frame was based on the following assumptions: (1) IBC 2000 equivalent lateral load procedure for special MRFs was applicable; (2) the structure is an office building located on stiff soil in the Los Angeles area; (3) the design accelerations were determined using the deterministic limit of the IBC 2000 site-specific procedure; and (4) A992 steel sections were used. Figure 5 shows the elevation details of the frame including the columns and beam sections, and the design lateral forces expressed as a percentage of the frame design base shear. The design base shear and the dead seismic weight of the frame were 3185 kN and 47550 kN, respectively.

The PFDCs were designed for values ranging between 0.88 to 0.95 M_{pb} corresponding to a relative rotation, θ_r, of 0.018 radians. The reinforcing plates were 1829 mm long. Figure 5 also indicates the initial post-tensioning (T_o) and the friction forces (F_f) provided in the SC-MRF. More details can be found in Rojas (2003).

In order to apply Equation (6) to the frame used in this article, the values for L/L_b, β, and α need to be defined. From the geometry indicated in Figure 5, the ratio between L and L_b can be taken approximately as 1.05. The value of β is determined using an average value among the values used in the design of the PFDCs. The value of β determined is 0.90. Finally, the value of α is determined taking into account that under strong earthquakes, the column plastic moment M_{pc}, is reduced by about 25% due to the influence of the axial force. Thus, the value of α proposed in this methodology was equal to 0.75. Using these values in Equation (6) gives a lateral capacity of 0.185W.

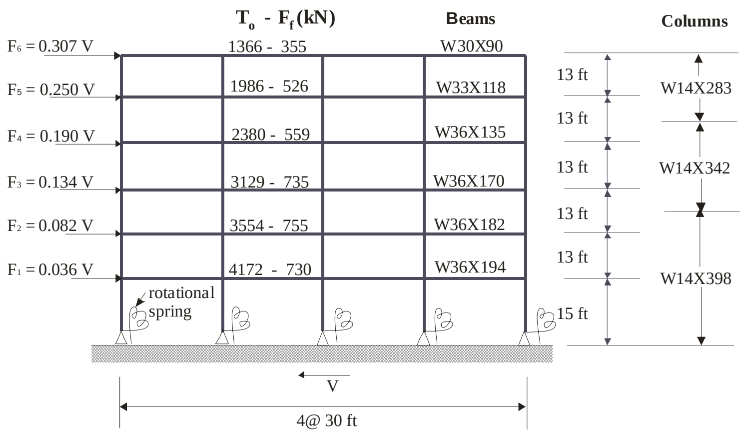


Figure 5. Six Story-four bay SC-MRF with PFDCs

5.4 Verification of Frame Lateral Capacity

In order to validate the proposed plastic limit analysis methodology, the results of the nonlinear static pushover analysis performed by Rojas (2003) will be used. Rojas developed a sophisticated nonlinear mathematical model of the SC-MRF with PFDCs described in Section 5.3 and presented in Figure 5. The model was developed with the aid of the computer program DRAIN-2DX. The lateral loads applied to the frame were distributed over the height of the frame in accordance with the IBC 2000 provisions. The details of the model can be found in Rojas (2003).

Figure 6 shows the relationship between the normalized base shear (V/W) and the roof drift (θ_{roof}) obtained from the nonlinear static pushover analysis performed by Rojas. The lateral capacity obtained by the proposed methodology agrees very well with the value of 0.183W found by Rojas (2003). θ_{roof} is defined as the roof displacement divided by the height of the frame. The figure also summarizes the limit states that occurred in the frame. The first connection gap opening occurs below the design base shear. The beginning of a significant reduction in the frame lateral stiffness occurs at a base shear of 0.106W (θ_{roof} = 0.53%) due to gap opening of several connections. First yielding occurs at the base of the ground floor columns when the base shear is 0.13W and θ_{roof} is 1.11%. Beam compression yielding at the end of the reinforcing plates begins when θ_{roof} is 1.90% at about DBE ground shaking level. With increasing loading, more yielding develops in the panel zones and floor diaphragm collector beams. Once enough yielding has developed (beyond the MCE level) the curve starts becoming flat indicating that the lateral capacity of the frame is being reached. This lateral capacity is 0.183W when θ_{roof} is 5%. No inelastic behavior occurred in the strands.

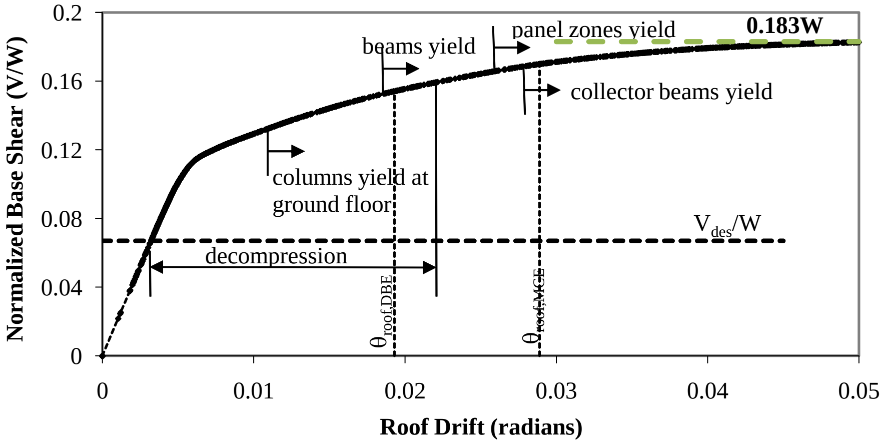


Figure 6. Normalized base shear vs. roof drift for the SC-MRF with PFDCs.

6. CONCLUSIONS

A plastic limit analysis methodology for self-centering (SC) steel moment resisting frames (MRFs) with post-tensioned friction damped connections (PFDC) has been presented. The methodology uses basic concepts of simple plastic theory with some minor adjustments to estimate the actual lateral capacity of SC-MRFs with PFDCs. The lateral capacity obtained by the proposed methodology was compared to the results of nonlinear static pushover analysis in order to validate the plastic limit analysis of this type of system. This lateral capacity of 0.185W agrees very well with the value of 0.183W found by Rojas (2003) when θ_{roof} is 5%. It can be concluded that the proposed plastic limit analysis methodology can be used to predict accurately the actual lateral capacity of SC-MRFs with PFDCs.

7. ACKNOWLEGMENTS

The research reported herein was supported by the Escuela Superior Politecnica del Litoral of Guayaquil-Ecuador. The authors are grateful to Dr. Maria Garlock for her valuable comments to this paper. The findings, opinions, and conclusions expressed in the paper are the authors' and do not necessarily reflect the opinions of those acknowledged here.

8. REFERENCES

Beedle, Lynn S. (1958). "Plastic Design of Steel Frames", John Wiley & Sons, New York, USA,

Caballero, M. (2005). "Rehabilitación Sísmica de Pórticos de Acero Resistentes a Momento con Conexiones Postensadas y Elementos de Fricción." Tesis de Grado, Facultad de Ingeniería en Ciencias de la Tierra, Escuela Superior Politécnica del Litoral, Guayaquil – Ecuador.

FEMA (2000a). "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 - Provisions," *Report No. FEMA 368*, Federal Emergency Management Agency, Washington, D.C.

FEMA (2000b). "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 - Commentary," *Report No. FEMA 369*, Federal Emergency Management Agency,Washington, D.C.

Garlock, M. (2002). "Full-Scale Testing, Seismic Analysis, and Design of Post-Tensioned Seismic Resistant Connections for Steel Frames." P.h.D. Dissertation, Civil and Environmental Eng. Dept., Lehigh University.

Garlock, M., Ricles, J., and Sause, R. (2002). "Experimental Studies on Full-Scale Post-Tensioned Seismic-Resistant Steel Moment Connections." Proceedings of the 7th U.S. National Conference on Earthquake Engineering (NCEE), Boston, MA.

Herrera, R., Sause, R., and Ricles, J. (2001). "Refined Connection Element (Type 05) for DRAIN-2DX, Element Description and User Guide," ATLSS Report No. 01-07.

Prakash, V., Powell, G., and Campbell, S. (1993). "DRAIN-2DX Base Program Description and User Guide, Version 1.0." Report No. UCB/SEMM-93/17 & 18, Structural Engineering Mechanics and Materials, Dept. of Civil Engineering, University of California, Berkeley, CA, December.

Petty, G.D. (1999). "Evaluation of a Friction Component for a Post-Tensioned Steel Connection." M.S. Thesis, Civil and Environmental Eng. Dept., Lehigh University, 328pp.

Ricles, J., Sause, R., Peng, S.W., and Lu, L.W. (2002). "Experimental Evaluation of Earthquake Resistant Posttensioned Steel Connections." Journal of Structural Engineering, ASCE, 128(7), 850-859.

Ricles, J., Sause, R., Garlock, M, and Zhao, C. (2001). "Post-Tensioned Seismic Resistant Connections for Steel Frames." Journal of Structural Engineering, ASCE, 127(2), 113-121.

Rojas, P., Ricles, J.M., and R. Sause, (2005). "Seismic Performance of Post-Tensioned Steel MRFs with Friction Devices." Journal of Structural Engineering, ASCE, 131(4), 529-540.

Rojas, P., Garlock, M, Ricles, J., Sause, R., (2004). "Seismic Rehabilitation of Steel Moment Resistant Frames with Post-Tensioned Friction Damped Connections.", VIII Simposio Nacional de Ingeniería Sísmica, Tlaxcala, México.

Rojas, P. (2003). "Seismic Analysis, Design, and Evaluation of Post-Tensioned Friction Damped Connections for Steel Moment Resisting Frames." Ph.D. Dissertation, Civil and Environmental Eng. Dept., Lehigh University, Bethlehem, PA.

Wolski, M., Ricles, J.M., and R. Sause, (2006). "Seismic Resistant Self-Centering Steel Moment Resisting Frames With Bottom Flange Friction Devices," *Proceedings, 5th International Conference on Behavior of Steel Structures in Seismic Areas – STESSA 2006*, Yokohama, Japan.