*2015 2nd International Conference on Geological and Civil Engineering*
*IPCBEE vol. 80 (2015) © (2015) IACSIT Press, Singapore*
*DOI: 10.7763/IPCBEE. 2015. V80. 5*

# Nonlinear Seismic Response of 3D Steel Buildings with Welded and Post-Tensioned Connections

Alfredo Reyes-Salazar [1+], Manuel Soto-Lopez [2], Israel Gaxiola-Avendaño [2], Eden-Bojorquez [2] and Arturo Lopez-Barraza [1]

[1] Professor, Universidad Autónoma de Sinaloa, Culiacán, Sinaloa, México.
[2] Graduate student, Universidad Autónoma de Sinaloa, Culiacán, Sinaloa, México

**Abstract.** The seismic responses of steel buildings with semi-rigid post-tensioned connections (PC) are estimated and compared with those of steel buildings with typical welded connections (WC). The results indicate that the seismic response is smaller for the buildings with PC connection. The reason for this is that the buildings with PC dissipate more hysteretic energy. In addition, unlike the case of buildings with WC, the hysteretic energy is mostly dissipated at the PC which implies much smaller structural damage in beams and columns. Consequently, steel buildings with PC are a viable option in high seismicity areas due to the fact that brittle failure is avoided, and also because of their smaller response and self-centering connection capacity.

**Keywords:** 3D Steel Buildings, Welded and Post-Tensioned Connections, Seismic Loading, Global and Local Response Parameters, Nonlinear Seismic Response.

## 1. Introduction

Extensive damage was observed in beam-column welded connections in steel moment-resisting frames during the Northridge Earthquake of 1994 and the Kobe Earthquake of 1995. One of the typically damaged steel beam-column connections was the bolted-web, welded-flange connection. Brittle fractures initiated within the connections at very low levels of plastic demand, and in many cases, while the structures remained essentially elastic. Fractures initiated at the complete joint penetration weld between the beam bottom flange and column flange. This forced the profession to reexamine seismic design practices existed before these events (including structural systems and materials) as well as to propose alternative connections.

Semi-rigid post-tensioned connections (PC) have been recently proposed as a viable alternative to welded connections (WC) of moment resisting steel frames in high seismicity areas. They are structural elements which include energy dissipating elements and high strength strands, in addition to beam and columns. Field welding is not required and the initial connection stiffness is similar to that of a welded one Moreover, they provide capacity of self-centering and energy dissipation. The beams are post-tensioned to the columns by the strands, which are oriented in the direction of the axes of the beam. The connections are designed to prevent brittle fractures in the area of the nodes of the frames, which can cause severe reduction in their ductility, as occurred in many cases during the Northridge and Kobe Earthquakes. Under the action of a strong earthquake, beams and columns remain essentially elastic concentrating the damage on the energy dissipating elements, which can be easily replaced at low cost. A typical PC is illustrated in Fig. 1.

The performance of steel frames with this type of connection has been studied by several researchers [1]-[9]. The general conclusions of these studies are that the responses of the frames with PC are smaller than those of the frames with WC, that the frames were able to undergo large inelastic deformations (drifts larger

+ Corresponding author.
*E-mail address*: reyes@uas.edu.mx.

than 5%) with minimum damage in beams or columns and consequently minimum residual drift and strength degradation. In spite of the important contributions of these studies, most of them were limited to structural sub-assemblages or to plane models. Moreover, results in terms of local response parameters like axial load or bending moment at particular elements have not been considered.

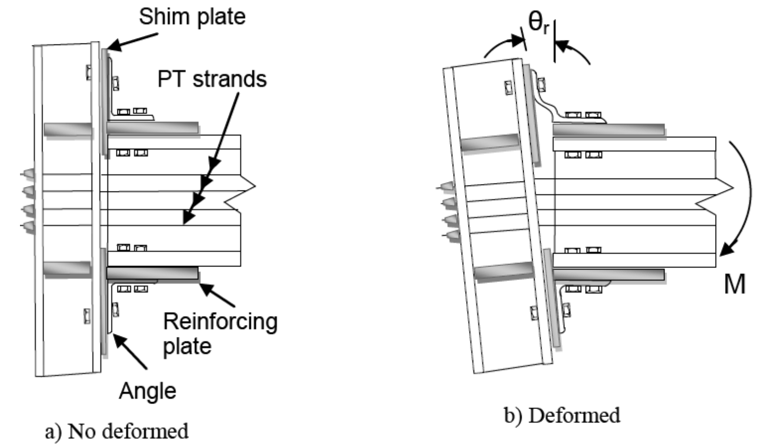


Fig. 1. Post-tensioned semi-rigid connection

## 2. Objectives of the study

In this paper, the nonlinear seismic responses of steel buildings with PC are estimated and compared with those of steel buildings with typical WC. The comparison is made in terms of several response parameters, first for 3D representations of the buildings and then for 2D representations. Finally a comparison is made between the results of 3D and 2D models.

Table 1: Beam and columns section for Models 1 and 2.

| Model | Moment resisting frames | | | | Gravity frames | | |
|---|---|---|---|---|---|---|---|
| | Story | Columns | | Girders | Columns | | Girders |
| | | Exterior | Interior | Below | Penthouse | Others | |
| 1 | 1\2 | W14×257 | W14×311 | W33×118 | W14×82 | W14×68 | W18×35 |
| | 2\3 | W14×257 | W14×311 | W30×116 | W14×82 | W14×68 | W18×35 |
| | 3\Roof | W14×257 | W14×311 | W24×68 | W14×82 | W14×68 | W16×26 |
| 2 | -1\1 | W14×370 | W14×500 | W36×160 | W14×211 | W14×193 | W21×44 |
| | 1\2 | W14×370 | W14×500 | W36×160 | W14×211 | W14×193 | W18×35 |
| | 2\3 | W14×370 | W14×500,W14×455 | W36×160 | W14×211,W14×159 | W14×193,W14×145 | W18×35 |
| | 3\4 | W14×370 | W14×455 | W36×135 | W14×159 | W14×145 | W18×35 |
| | 4\5 | W14×370,W14×283 | W14×455,W14×370 | W36×135 | W14×159,W14×120 | W14×145,W14×109 | W18×35 |
| | 5\6 | W14×283 | W14×370 | W36×135 | W14×120 | W14×109 | W18×35 |
| | 6\7 | W14×283,W14×257 | W14×370,W14×283 | W36×135 | W14×120,W14×90 | W14×109,W14×82 | W18×35 |
| | 7\8 | W14×257 | W14×283 | W30×99 | W14×90 | W14×82 | W18×35 |
| | 8\9 | W14×257,W14×233 | W14×283,W14×257 | W27×84 | W14×90,W14×61 | W14×82,W14×48 | W18×35 |
| | 9/Roof | W14×233 | W14×257 | W24×68 | W14x61 | W14×48 | W16×26 |

## 3. Methodology and structural models

For numerical evaluation of the issues discussed earlier, the nonlinear seismic responses of two steel buildings with perimeter moment resisting frames (PMRF), which were used in the SAC steel project [10] are considered in this study. Specifically the 3-, and 10-level buildings located in the Los Angeles area are used. The RUAUMOKO Computer Program [11] is used for the time history nonlinear dynamic analysis. These buildings are supposed to satisfy all code requirements existed at the time of the project development. Sizes of beams and columns, as reported [10], are given in Table 1 for the two models. Additional information for the models can be obtained from the project report [10]. The buildings are modeled as complex MDOF systems. Each column is represented by one element and each girder of the perimeter moment resisting frames (PMRF) is represented by two elements, having a node at the mid-span. The slab is modeled by near-rigid struts, as considered in the project [10] report. Each node is considered to have six degrees of freedom when the buildings are modeled in three dimensions.

Only the PMRF are considered while post-tensioning the connections. The PC frames were designed according to the recommendations proposed by Garlock et. al. [5], which basically starts with the design of the steel frames as usually is done (considering WC) and then, the semi-rigid post-tensioned connections are designed to satisfy the requirements of serviceability and resistance conditions.

To study the responses of the models comprehensively and to make meaningful conclusions, they are excited by twenty earthquake motions in time domain with different frequency contents, recorded at different locations (hard and intermediate soils) at the following stations: Fun Valley, Reservoir 361; Convict Creek; Cerro Prieto; Parkfield, Joaquin Canyon; Olympia Hwy Test Lab; Utilities Bldg, Long Beach; El centro, California; Centerville Beach, Naval Facility; Gilroy Array Sta No 4; Olympia Hwy Test Lab; Castaic-Old Ridge Route; Long Valley Dam; El Centro-Imp Vall Dist; Palo Alto; UCSB Goleta FF; Parkfield Fault Zone 14; Chihuahua; Canoga Park, Santa Susana; Ferndale, California; Indio, Jackson Road. The predominant periods of the earthquakes vary from 0.11 to 0.66 sec. The earthquake time histories were obtained from the Data Sets of the National Strong Motion Program (NSMP) of the United States Geological Surveys (USGS). Additional information on these earthquakes can be obtained from these data base. The earthquakes are scaled up in such a way that the models develop a similar target interstory displacements for any of the earthquakes. Drifts of 1%, 2% and 3% are considered.

Yielding angles are considered in the connections in this study. L8x8x3/4 G50 angles were used in the 3-level building. For the 10-level building, L6x6x1, L8x8x3/4 and L6x6x5/8 angles, were used for the second through eight, ninth, and tenth stories, respectively. In both models, 7/8" A325 bolts and 5/8" G50 strands were used. The number of strands is about 18 for all the stories except for the top level where it is 10. The pre-tension is 33% of yielding stress. Each PC is represented by a structural element which are modeled by using the flag-shaped bi-linear hyteresis model of the Ramuoko Computer Program [11]. It exhibits good accuracy in comparison with experiment results [5], [9].

## 4. Results

The seismic responses for the three-dimensional representation of the steel buildings with WC are estimated and compared with those of the corresponding buildings with PC. Results in terms of interstory shears, for both, *N-S* and *E-W* directions, are discussed first. The ratio given by the expression

$$V_1 = \frac{V_{WC}}{V_{PC}} \tag{1}$$

is used for that purpose. In Eq. 1, $V_{WC}$ and $V_{PC}$ represent the interstory shear for the steel buildings with welded and post-tensioned connections, respectively. Results for $V_1$ are presented in Fig. 2 for the 3-level model, the *N-S* and E-W directions and drifts of 1%, 2% and 3%. In this figures, the word “ST” stands for the story level. It can be observed that the $V_1$ values significantly vary from one earthquake to another without showing any trend, even thought the models were deformed to a similar level of deformation. It reflects the effect of the earthquake frequency contents and the contribution of several modes on the structural responses. The most important observation that can be made is that the values of $V_1$ are larger than unity indicating that the interstory shears are larger for the models with WC, values of up to 1.5 are observed in some cases. The reason for this is that more hysteretic energy is dissipated in the buildings with PC. Moreover, the energy dissipated in beam and columns is negligible, implying minimum structural damage. The values of $V_1$ in general tend to increase with the story number.

The roof displacements for the models with WC and PC are now estimated. The displacement ratio given by the equation

$$D_1 = \frac{D_{WC}}{D_{PC}} \tag{2}$$

which is used to make the comparison, where $D_{WC}$ and $D_{PC}$ represent the same as before, except that now roof displacements are used instead.

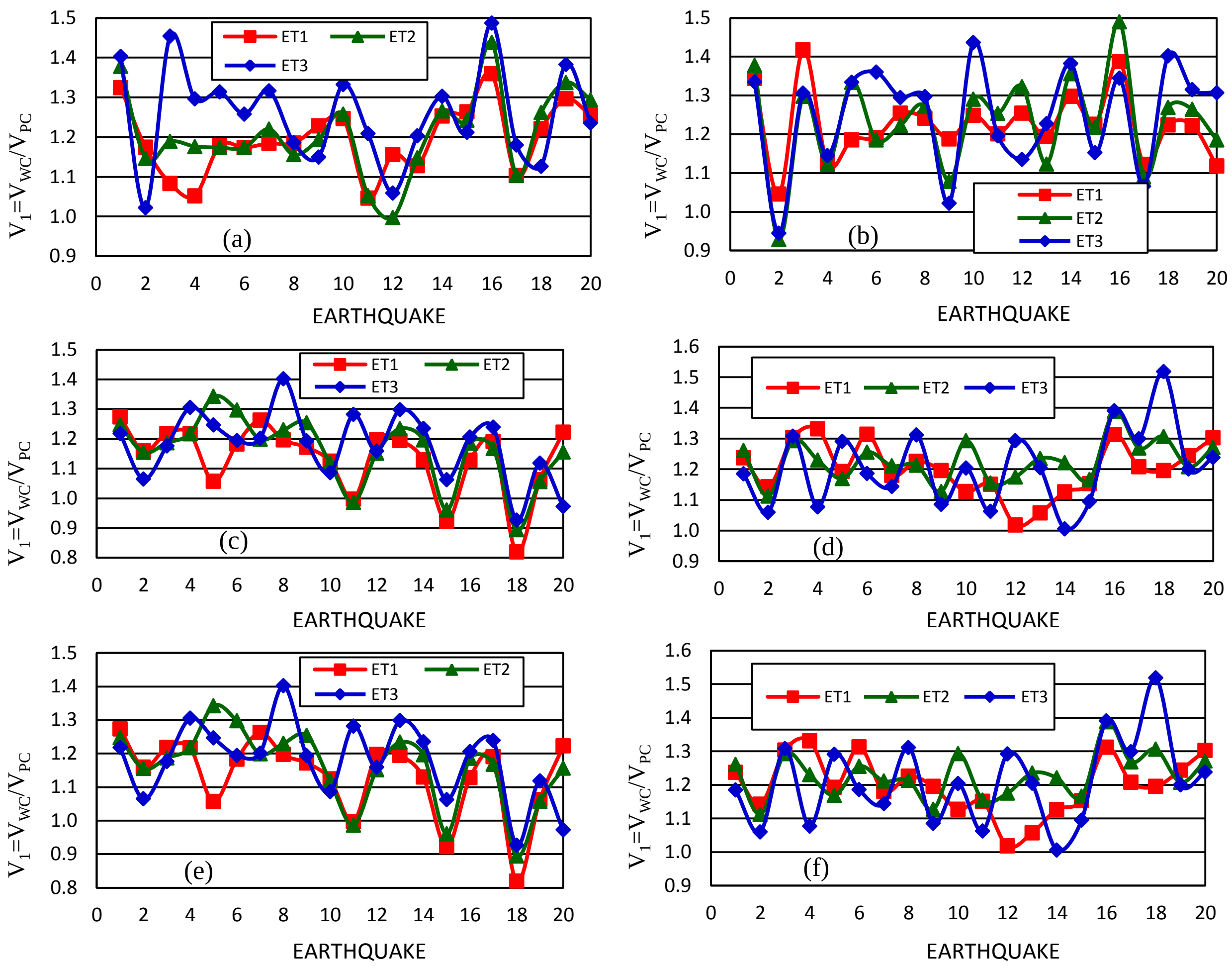


Fig. 2. Values of the $V_1$ parameter, (a) N-S and 1%, (b) E-W and 1%, (c) N-S and 2%, (d) E-W and 2%, (e) N-S and 3%, (f) E-W and 3%.

The results are given in Fig. 3 for the 3-level model, the *N-S* and E-W directions and drifts of 1%, 2% and 3%. As for the case of shear, it is observed that the $D_1$ values significantly vary from one earthquake to another, that they are similar for the N-S and E-W directions and that the values are larger than unity indicating larger roof displacements for the frames with WC. However, the magnitude of $D_1$ is larger than that of $V_1$. The only additional observation that can be made is that $D_1$, in general, tend to increase as the target drift displacement increases.

Similar ratios to those of interstory shear and roof displacements are also calculated for axial loads and bending moments ($A_1$ and $M_1$) at some columns of the base, but the results are not shown. The observations made before are essentially the same for these parameters. The $V_1$, $D_1$, $A_1$ and $M_1$ parameters are also estimated for the 10-level building The results indicate that, as for the case of the 3-level model, the seismic responses are larger for the case of buildings with WC. The only additional observation that can be made is that the response ratios may be significantly larger for the 10-level model. The interstory shear, roof displacement, axial load and bending moment ratios were also estimated for the 2D structural representation of the buildings. Results indicate that the values of these rations can be larger for the 3D models, particularly for local response parameters (axial load and bending moments)

## 5. Conclusions

The seismic responses of steel buildings with semi-rigid post-tensioned connections (PC) are estimated and compared with those of steel buildings with typical welded connections (WC). Two steel buildings with perimeter moment resisting frames (PMRF), which were used in the SAC steel project, and twenty strong motions are considered in the study.

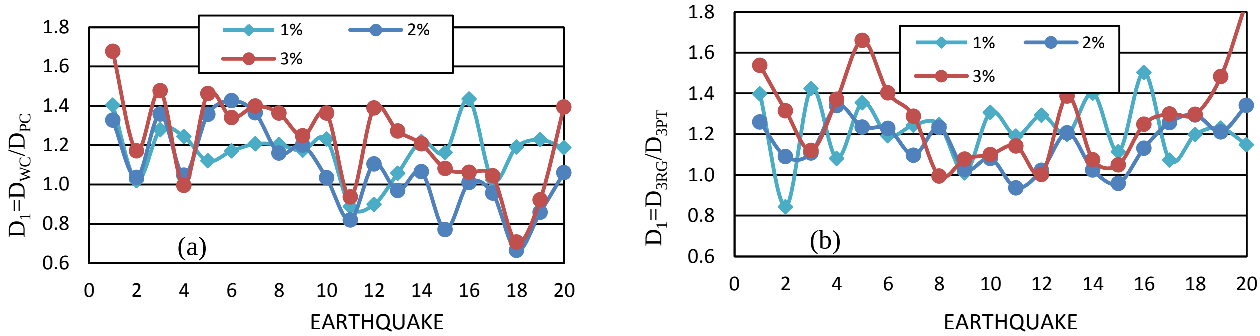

Fig. 3. Values of the $D_1$ parameter, (a) N-S, (b) E-W.

The results indicate that the seismic response in terms of interstory shears, roof displacements, axial load and bending moments are smaller for the buildings with PC connection. The reason for this is that the buildings with PC dissipate more hysteretic energy than those with WC. In addition, unlike the case of buildings with WC, the hysteretic energy is mostly dissipated at the PC which implies that the structural damage in beams and columns is not significant. Consequently, steel buildings with PC are a viable option in high seismicity areas due to the fact that brittle failure is avoided, and also because of their smaller response and self-centering connection capacity.

## 6. Acknowledgements

This paper is based on work supported by La Universidad Autónoma de Sinaloa (UAS), under grant PROFAPI-2013/157.

## 7. References

[1] J.M. Ricles, R. Sause, M. Garlock, C. Zhao. Post-tensioned seismic-resistant connections for steel frames. *ASCE Journal of Structural Engineering* 2001, **127**(2): 113-121.

[2] C. Christopoulos, A. Filiatrault. Seismic response of post-tensioned energy dissipating moment resisting steel frames. *Proceedings of the 12th European Conference on Earthquake Engineering* 2002, London, UK, paper No. 61.

[3] M. Garlock, J. Ricles, R. Sause. Cyclic load tests and analysis of bolted top-and-seat angle connections. *J. Struct. Eng.*, **129**(12): 1615–1625.

[4] C. C. Chou, J.H. Chen, Y.C. Chen, K.C. Tsai. Evaluating performance of post-tensioned steel connections with strands and reduced angle plates. *Earthquake Engineering and Structural Dynamics*, 2006, **35**:1167–1185.

[5] M. Garlock, R. Sause, J. Ricles. Behavior and design of post-tensioned steel frames system. *ASCE Journal of Structural Engineering*, 2007, **133** (3): 389-399.

[6] C. C. Chou, K. C. Tsai, W. C. Yang. Self-centering steel connections with steel bars and a discontinuous composite slab. *Earthquake Engineering and Structural Dynamics*, 2008, **38**:403-422.

[7] M. Wolski, J.M. Ricles, R. Sauce. Experimental study of a self-centering beam-column connection with bottom flange friction device. *ASCE Journal of Structural Engineering*, **135**(5): 479-488.

[8] C. C. Chou, J. H. Chen. Seismic design and shake table test of a steel post-tensioned self-centering moment frame with a slab accommodating frame expansion. *Earthquake Engineering and Structural Dynamics*, 2011, **40**: 1241-1261.

[9] A. López-Barraza, E. Bojórquez, S. E. Ruiz, A. Reyes-Salazar. Reduction of maximum and residual drifts on post-tensioned steel frames with semi-rigid connections. *Advances in Materials Science and Engineering*, 2013, ID 192484, doi:10.1155/2013/192484.

[10] A. Carr A. RUAUMOKO *Inelastic Dynamic Analysis Program*, University of Cantenbury, Department of Civil Engineering.

[11] Federal Emergency Management Agency, State of the Art Report on Systems Performance of Steel Moment Frames Subjected to Earthquake Ground Shaking, 2000, SAC Steel Project, Report FEMA 355C.