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INELASTIC SEISMIC RESPONSE OF COMPOSITE PARTIALLY RESTRAINED MOMENT FRAMES IN SOFT SOILS

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Abstract

This paper examines in detail the seismic behavior of a type of semi-rigid composite connections best known in the international literature as Partially Restrained Composite Connections (PRCC). In addition, this paper described some efforts to promote the use of these connections for the design of low-rise buildings in Mexico City. Even when the C-PRMF system has been implemented in the codes and in real projects of the United States, the adaptation to Mexico City requires the evaluation of the local conditions such as the local seismicity, class sites, structural configurations, specifications and standards, among other variables. In order to meet this goal, analytical calibrations to experimental results of PRCC connections tested in the laboratory were performed. Thus, the influence of PRCC connections on the behavior of moment frames under local seismic loads through nonlinear static and dynamic analysis is studied. Nonlinear analysis results point out that PRCC connections are potentially applicable to low-rise special moment frames located at both hard and intermediate class sites in Mexico City.

Keywords: PR Connection; Composite Construction; C-PRMF; PRCC

1 Introduction

1.1 Partially Restrained Composite Connections (PRCC)

In the field of analysis and design of steel structures, it is common to simplify the behavior of connections assuming them as either totally rigid (fixed rotation) or perfectly pinned (free rotation). However, there are other type of connections known as semi-rigid (or partially restrained) moment connections, which do not fall within either the category of rigid nor simple connections, but between them. The Partially Restrained Composite Connection (PRCC) is one of the possible configurations of semirigid connections, where a seat angle, a double shear angle, and a reinforced concrete deck work as a composite moment connection as illustrated in Fig. 1.

The use of partially restrained composite connections (PRCC) for seismic applications has been endorsed by the AISC trough both its Seismic Provisions [1] and the Design Guide No. 8 [2] through a structural system known as Composite – Partially Restrained Moment Frame (C-PRMF). The main characteristic of the C-PRMF structural systems with PRCC connections is that the plastic hinges shall occur on the connection parts, as illustrated in Fig. 2, and not at the beam ends as intended in conventional moment frame systems with rigid connections (e.g., SMRF). In a good C-PRMF design with PRCC connections, both the beams and columns (except those at the base) shall remain elastic, as illustrated in Fig. 2.

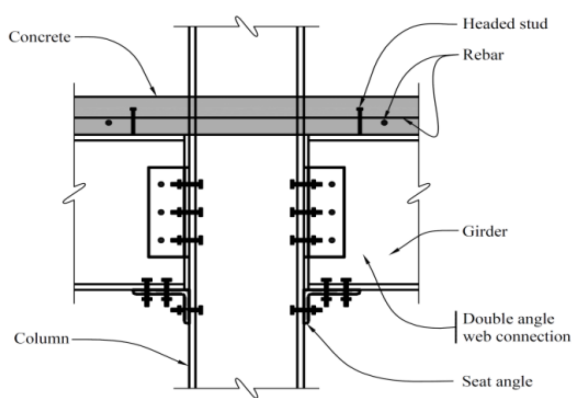


Fig. 1 – Configuration of a Partially Restrained Composite Connection (PRCC) [1].

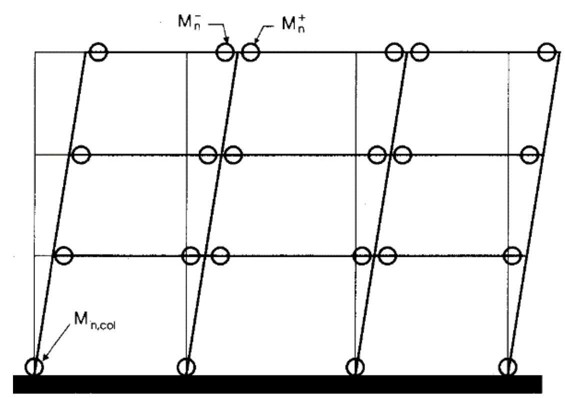


Fig. 2 – Collapse mechanism expected in a Composite – Partially Restrained Moment Frame (C-PRMF) [2].

Thus, the design of C-PRMF is different from design of conventional moment frames in three important characteristics:

1. Plastic hinges on the system are developed in the connection components (reinforcing bars and seat angles), and not within the beams as in conventional systems of rigid frames. Therefore, the PRCC are not designed to be stronger than the connecting beams.
2. Since the connections are neither rigid nor simple, the stiffness should be taken into account implicitly in the analysis in order to determine the required strength and drifts of the structural system. Major nonlinear effects are expected due to geometric nonlinearities and material inelasticity, especially in the PRCC components.
3. Since PRCC connections are generally weaker than rigid connections, the lateral resistance of the system will require a greater number of frames with semirigid connections, resulting in a highly redundant system.

According to the AISC Seismic Provisions [1], C-PRMF should ensure an expected minimum drift of 0.02 radians and, at this drift level, the PRCC connections shall be at least 50% of the nominal flexural strength of the steel beam (without composite action). The maximum strength of the connection is not stipulated, although it is recommended to be around close to the plastic moment of the steel beam.

1.2 Consideration of PRCC in the analysis of C-PRMF

One of the simplest methods for the structural analysis of C-PRMF systems is including rotational springs representing the observed performance on the moment-rotation (M-θ) curves of the PRCC connections. From experiments and parametric finite element studies, researchers like Ammerman and Leon [3], McCauley and Leon [4], Lin [5] and Kulkarni [6], reached the following expressions to characterize the behavior of PRCC connection for both positive and negative moment. These expressions are different due to asymmetric behavior regarding strength and rigidity.

In the case of negative moment (i.e., when the reinforced concrete slab is in tension) the curve is given by:

$$M_n^- = C_1\left(1 - e^{-C_2\theta}\right) + C_3\theta \tag{1}$$

where

$C_1 = 0.18(4A_sF_{yrb} + 0.857A_lF_y)(d + Y_3)$
$C_2 = 0.775$
$C_3 = 0.007(A_l + A_{wl})F_y(d + Y_3)$
$\theta =$ rotation milli-radians
$d =$ depth of the beam, (in.)
$Y_3 =$ distance from the top flange to the centroid of reinforcing steel, (in.)
$A_s =$ reinforcing steel area, (in.^{2})
$A_l =$ seat angle area, (in^2.)
$A_{wl} =$ gross area of double angle in the web, (in.^{2})
$F_{yrb} =$ yield strength of reinforcing steel, (ksi)
$F_y =$ yield strength of the seat angle and double angle in the web, (ksi)

In the case of positive moment (i.e., when the seat angle is subjected to tension), the curve is given by:

$$M_n^+ = C_1\left(1 - e^{-C_2\theta}\right) + C_3\theta + C_4 \tag{2}$$

where

$C_1 = 0.2400[0.48A_{wl} + A_l](d + Y_3)F_y$
$C_2 = 0.0210\left(d + \frac{Y_3}{2}\right)$
$C_3 = 0.0100(A_{wl} + A_l)(d + Y_3)F_y$
$C_4 = 0.0065A_{wl}(d + Y_3)F_y$

As an example, Fig. 3 shows a complete M-θ curve for a typical PRCC derived from equations (1) and (2). This curve corresponds to a connection with a W18x35 ASTM A36 steel beam, a concrete slab reinforced by 8#4 bars grade 60. The seat angle has an area (A_l) of 1535 mm^2, and the area of the angles in the web (A_{wl}) is 2742 mm^2. The depth (d) of the beam is 450 mm and the distance between the top flange and the centroid of the reinforcement (Y_3) is equal to 102 mm.

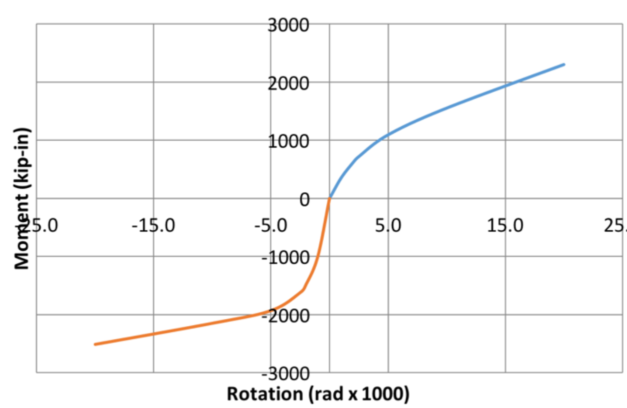
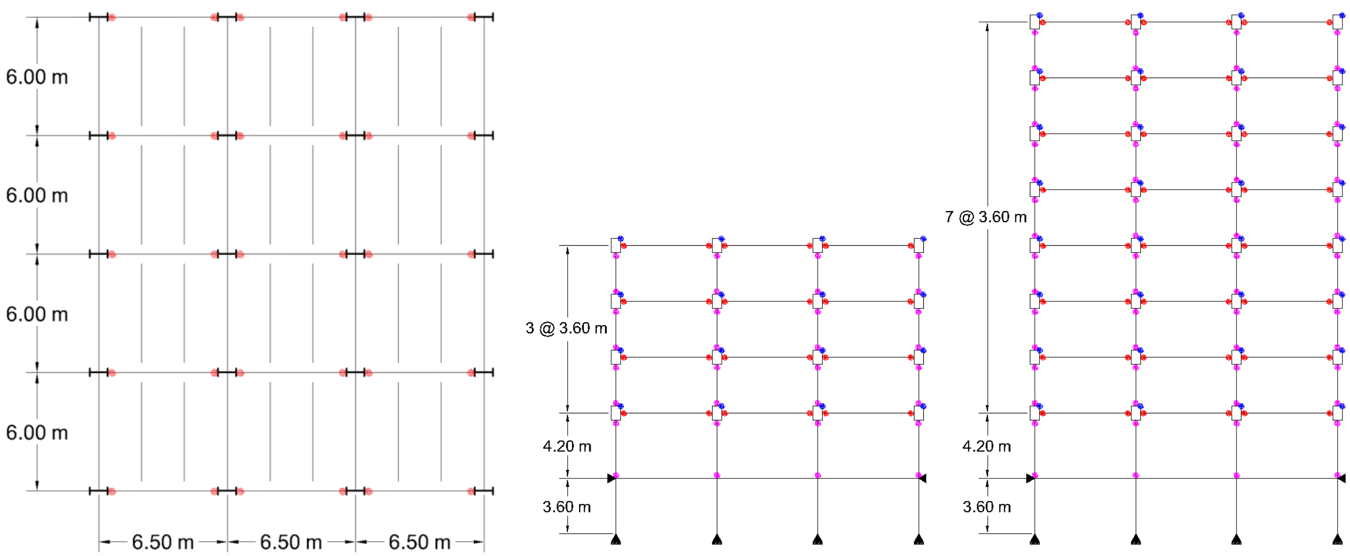


Fig. 3 – M-θ curve for a typical PRCC.

2 Prototypes

In order to evaluate the seismic behavior of C-PRMF and its potential use in Mexico City, three prototype buildings were designed. The typical layout for the three buildings is shown in Fig. 4(a), which consist of braced frames in the Y-axis, and C-PRMF in the X-axis (with PRCC connected to the column flanges). These buildings have four story and eight story levels, one basement, with elevations as shown in Fig. 4(b) and 4(c). The structure consists of WF columns and beams. The steel beams are composite with steel deck. For simplicity, the figure does not show the secondary beams, which brace laterally the main beams and provide support the floor system. In this figure, the points at the ends of the main beams and the column flanges represent PRCC, while those beams which reach the web of the columns are shear connections.



(a) Typical plan layout (b) Four-story frame (c) Eight-story frame

Fig. 4 – Prototype frames

The seismic lateral forces and allowed drifts for both service and ultimate limit states satisfied the Mexico City (NTC-DS) seismic code [7]. Seismic detailing met the requirements of the AISC No. 8 Design Guide [2] and the AISC Seismic Provisions [1]. The characteristics of the three proposed prototypes are summarized in Table 1.

The labels used for the prototype frames is as follows: The first digit corresponds to the number of the story levels, followed by the type of frame: S for a ductile or special frame (strength reduction factor, Q=4 by NTC-DS), or O for an ordinary frame (strength reduction factor, Q=2). The last digit (1 or 2) indicates the soil period in seconds on which the building stands. In order to evaluate the critical case when the system is in near resonance, i.e. when the fundamental period of the structure (T_1) approaches the fundamental period of vibration of the soil (T_s), frames with four story levels were assumed to be standing on ground with $T_s = 1\ s$., while the eight-story frame was assumed to stand on ground with $T_s = 2\ s$.

Table 1 – Summary of the studied archetypes

Frame	Levels	Ductility	Q	T_1 (sec)	T_s (sec)
4S1	4	Special (S)	4	1.26	1.0
4O1	4	Ordinary (O)	2	0.84	1.0
8S2	8	Special (S)	4	1.21	2.0

2.1 Analysis and design considerations

In the case of the buildings studied in this work, the design process was conducted through an iterative process, and with the help of the software ETABS. The PRCC connections were modeled using *MultiLinear Plastic Link* elements that help simulate moment-rotation curves like the one illustrated in Fig. 3. Another great benefit of this element is that also simulates hysteretic degradation of various types; in this study, a *Takeda* degrading model was used. The composite action in the beam was considered by increasing the moment of inertia. Caution was taken so the column was always stronger than the connection, thus preventing a weak floor mechanism. The following summarizes the features and design considerations for analysis and study of the buildings.

(a) The intensities of the loads are:
- Typical story dead load 3.5 kPa, maximum live load 2.5 kPa, and live load for earthquake 1.8 kPa.
- Roof dead load 3.1 kPa, maximum live load 1 kPa, and live load for earthquake 0.70 kPa.
- A facade load of 3.7 kN/m is applied on the perimeter of all levels.

(b) An office use is assumed for these buildings, so it belongs to the group B in the local code [7].

(c) The concrete properties for the slab (class 1 [7]) are:

Concrete volumetric weight:	$\gamma = 24\, kN/m^3$
Compressive strength:	$f'_c = 25\, MPa$
Modulus of elasticity of concrete:	$E_c = 4400\sqrt{f'_c}\,, MPa$
Yield strength of rebar:	$f_y = 422\, MPa$
Steel elasticity modulus	$E_s = 200\, GPa$

(d) It is assumed that the slab provides a rigid diaphragm constraint in its plane

(e) Steel for beams and columns is ASTM A992 with a yield strength of $F_y = 351.5\, MPa$.

(f) Steel angles in connection is ASTM A36 with a yield strength of $F_y = 253\, MPa$.

Table 2 summarizes the final design sections obtained by following the requirements of the AISC No. 8 Design Guide [2] and the seismic provisions AISC 341 [1].

Table 2 – Design summary

Prototype	Steel columns A992		A992 steel girders in composite action with floor	PRCC	
	Internal	External		Rebar	Thickness of seat angle L 6 x 4 x t
4S1	1-2: W14x82 3-4: W14x53	1-2: W14x68 3-4: W14x38	W21x62	1: 10#4, 2: 12#4 3: 8#4, 4: 6#4	1: $^1/_2$, 2: $^5/_8$ 3: $^7/_{16}$, 4: $^5/_{16}$
4O1	1-2: W14x145 3-4: W14x109	1-2: W14x120 3-4: W14x74	W33x130	1: 10#5, 2: 12#4 3: 12#4, 4: 6#4	1: $^7/_8$, 2: $^5/_8$ 3: $^5/_8$, 4: $^5/_{16}$
8S2	1-2: W14x233 3-4: W14x193 5-6: W14x159 7-8: W14x132	1-2: W14x193 3-4: W14x176 5-6: W14x132 7-8: W14x82	W36x150	1-3: 10#6, 4-6: 8#6 7: 12#4 8: 8#4	1-6: L6x6x1 7: $^5/_{16}$ 8: $^7/_{16}$

3 Nonlinear analysis model

Inelastic analysis was performed using the open source program OpenSees. Fig. 5 illustrates the PRCC assembled nonlinear model of the beam-column joint for the C-PRMF prototypes. The columns were modeled as elastic elements that connect to rotational springs to consider possible plasticity, while the main beams were modeled as nonlinear elements with fiber sections to take into account the composite action. A bilinear uniaxial steel material was used in the definition of the steel members. The yield strength for the structural steel was taken equal to $F_y = 345\, MPa$, while the yield strength for rebar was taken equal to $F_y = 414\, MPa$. The kinematic relationship of strain-hardening deformation of both materials is taken equal to 0.001. The concrete model for the slab included linear tension strength softening, while the compressive strength was taken as $25\, MPa$.

Fig. 5 shows two zero-length springs at the columns ends, two zero-length springs at the beams ends, and one zero-length spring at the panel zone. The springs at the column ends consider their potential material nonlinearity, which were modeled with bilinear materials based on studies by Ibarra *et al*. [9] and Lignos *et al.* [10]. The springs at the beam ends account for the PRCC nonlinear behavior, which were modeled using a material called *Pinching4* in the OpenSees program. The panel zone was modeled by a system of rigid elements articulated at the ends, and connected to a spring in the upper right corner represent potential hysteretic shear plasticity. Note that each of the components in the model try to represent the real behavior of the PRCC connection as discussed in detail in the following sections.

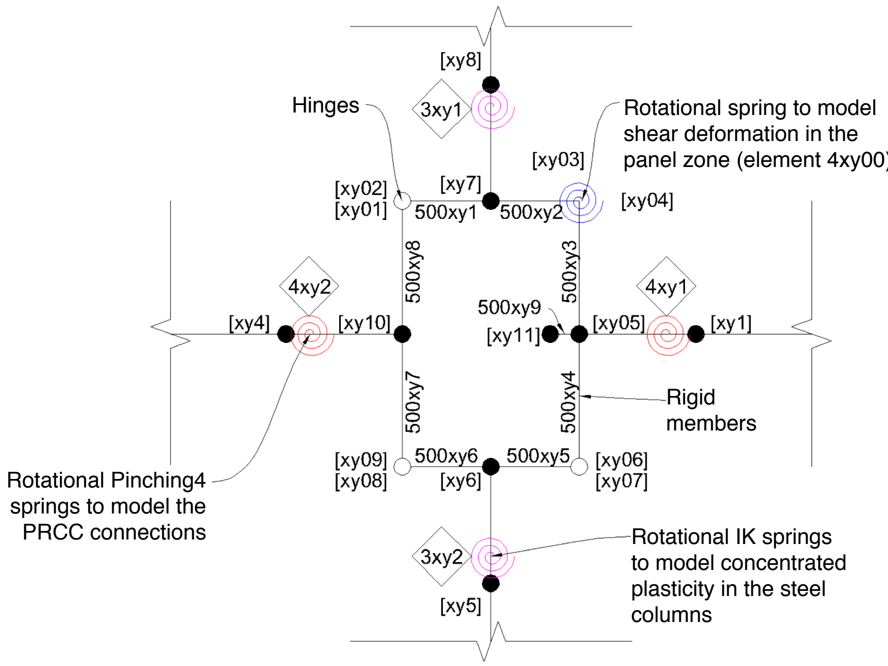


Fig. 5 – PRCC nonlinear model

3.1 Composite beams

The criteria adopted for determining the effective width in composite beams is described in the AISC Specification [11], where one quarter of the span from centers ($L/4$ = 1625 mm) controlled for the internal frames. The slab is a 20-gauge deck with 63.5 mm. of deep, and a concrete solid portion of 63.5 mm. Gravity loads from two secondary beams were assigned as point loads. Composite beams were defined by *forceBeamColumn* elements in the OpenSees software, which creates a non-linear force controlled element with distributed plasticity. The integration along the element is based on the rule of Gauss-Lobatto; thus, five integration points were used in these analysis.

3.2 Steel columns

Steel columns were modeled by an elastic member (*elasticBeamColumn*), which is coupled to the panel zone system through a nonlinear rotational spring (*IK*) that considers plastic hinges at the column ends. This constitutive model, proposed by Ibarra *et al.* [9], represents a bilinear hysteretic material with deterioration. The expected columns plastic resistance is given by $M_p = R_y Z_x F_y = 1.1 Z_x F_y$, where R_y = 1.1 considers the expected overstrength of ASTM A992 steel [1]. Similarly, the expected strength of the column is $M_c = 1.1 M_p$, assuming a post-yield ratio of 1.10. Values adopted in this study correspond to the default parameters proposed by the authors [9, 10]. Thus, a residual resistance factor κ equal to 0.40 and a final rotation θ_u equal to 0.15 were used. The deterioration parameters of the IK springs (θ_p, θ_{pc}, Δ) proposed by Lignos *et al.* [10] were obtained from calibration to experiments of steel sections.

3.3 Panel zones

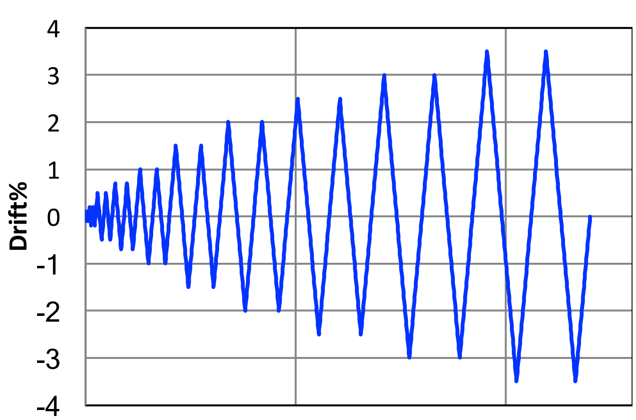
Several mathematical models have been proposed to describe the behavior of the panel zone; these models are obtained from either experiments or modifying some existing models. Generally, the models differ in the representation of the inelastic range, but all mostly agree in the elastic region. The model used in this work is the one proposed by Krawinkler [12], which consists of a trilinear shear-strain curve defined by the elastic, plastic, and hardening ranges.

3.4 Partially Restrained Composite Connections (PRCC)

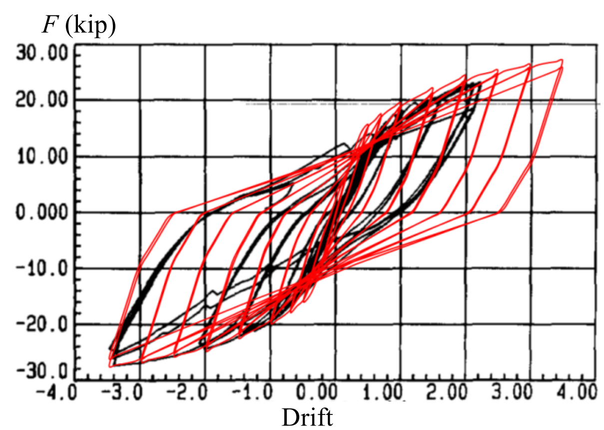
The inelastic response of the PRCC is modeled by rotational springs from a material that exhibits stiffness and strength degradation during cyclic loading. A rotational spring of zero-length with a *Pinching4* material was used to define the moment-rotation of the PRCC connection response, which connects to both the beam end and the panel zone system. In order to define the curve, four points on the positive side and four on the negative were used. Rotations at 2, 20, 30 and 80 milli-radians for both positive and negative sides were used. The flexural strength values at rotations of 2 and 20 milli-radians were obtained from equations (1) and (2) as recommended by [2]. The flexural strength at 30 and 80 milli-radians were obtained by multiplying the moment at 20 milli-radians by 1.1 and 0.2, respectively. It may seem exaggerated that the curve was extended up to very large rotations, but it has been demonstrated experimentally that the connection does not fail at 20 milli-radians, but extends beyond as documented in [3] and [4]. In this work the maximum rotation of 30 milli-radians was assumed, and from this point, the curve gradually descends thus avoiding the brittle failure sometimes observed in the experiments [3, 4] and involving numerical problems. It should be noted that the connection response to cyclic loading will not follow the envelope curve set defined by the *Pinching4* material, but this will remain separate due to the cyclic degradation. Thus, cyclic degradation will not be reflected in the static analysis (pushover), so the response in this analysis will be more optimistic.

4 PRCC calibration

Nonlinear behavior of the PRCC was calibrated based on the experimental test results reported by McCauley and Leon [4]. The test setup consisted of a cross configuration, with two W14x38 beams connected to a W14x120 column, both with a nominal yield strength of $F_y = 250\ MPa$. The column length was 3.94 m., while the beams length was 3 m. The load protocol used for these tests is shown on Fig. 6(a). The calibrated global response is compared with the experimental lateral load (F) – drift curve in Fig. 6(b). Similarly, the calibrated local response is compared with the experimental moment – rotation curves for the west (Fig. 7(a)) and the east (Fig. 7(b)) connection sides, respectively. These results show that the parameters in the nonlinear model reproduces adequately the behavior measured experimentally with the cyclic loading.



(a) Loading protocol



(b) Lateral load – interstory drift curve: experimental (black) vs. analytical (red)

Fig. 6 – Load protocol and global response of specimen SRCC3C [4]

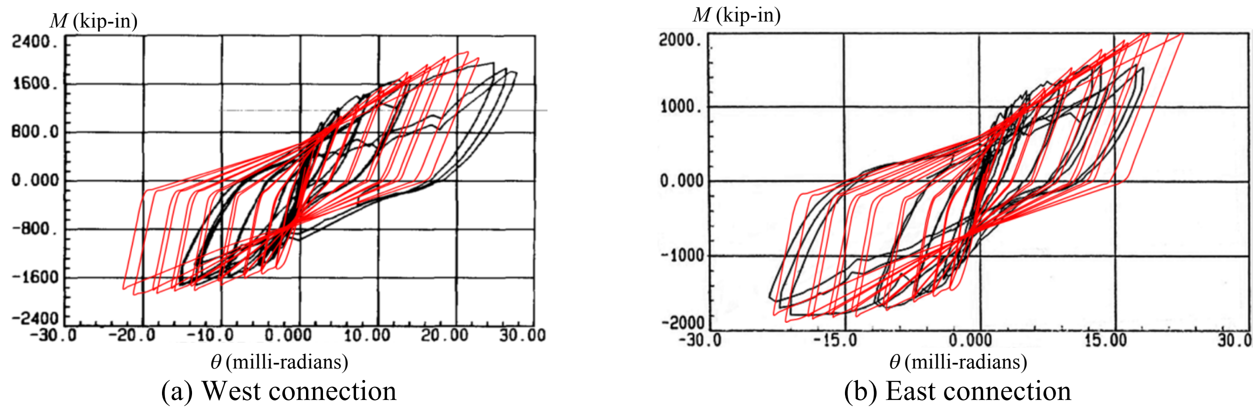


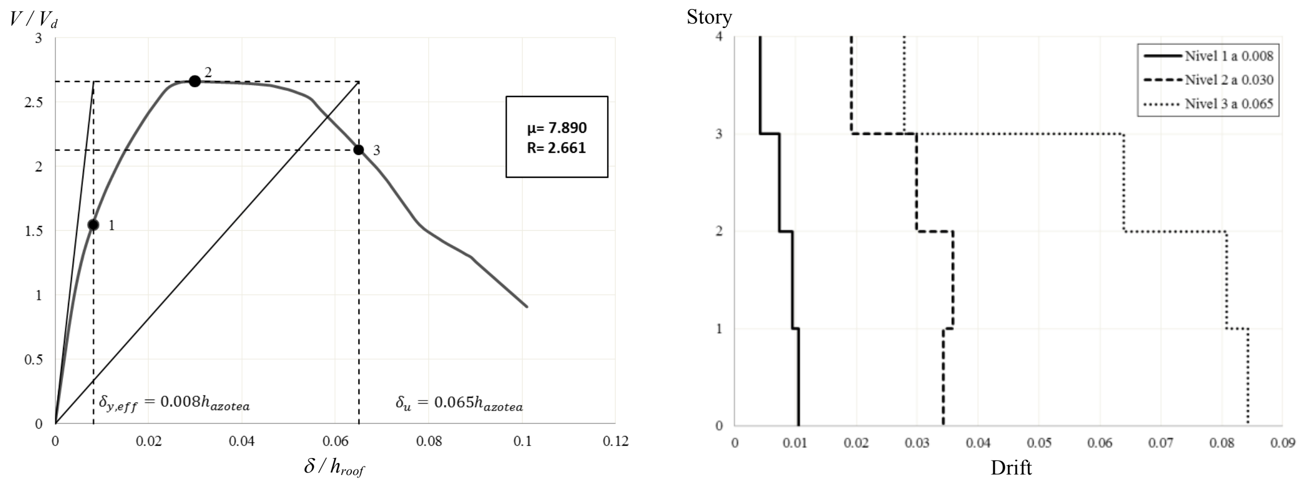
Fig. 7 – Moment-rotation (M-θ) curves of the PRCC: experimental (black) vs. analytical calibration (red)

5 Nonlinear static analysis

This section presents and discusses the results of the static pushover analysis of the proposed C-PRMF, which were modeled as previously described. Note that structural non-linear models are robust since they considered in a comprehensive manner many sources of inelasticity, including plasticity of PRCC calibrated with experimental data, the plasticity of the panel zone, and the possible inelastic behavior of the composite beams and the steel columns. Additionally, second order effects are included in the OpenSees analysis from a geometric transformation type of P-Delta for columns and panel zones. The pushover evaluation of the proposed C-PRMF prototypes was performed using the methodology presented in FEMA P695 [8].

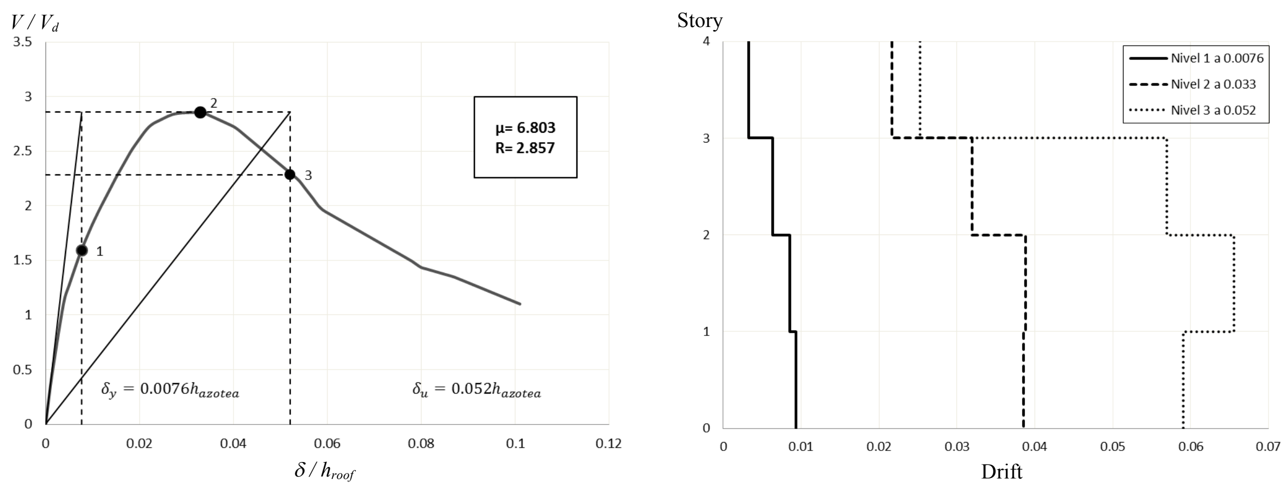
The results of the pushover analysis are presented in Fig. 8 for the frame 4S1, in Fig. 9 for the frame 4O1, and in Fig. 10 for the frame 8S1. The capacity curves for each case are shown in Figs. 8(a), 9(a) and 10(a), respectively. In these figures, the lateral strength (V) is normalized with the design strength (V_d). In these figures, the points indicate: (1) yield, (2) maximum, and (3) ultimate displacements. The numerical values of yield and ultimate displacements normalized with the building height (δ/h_{roof}) are also reported in these figures. Figs. 8(b), 9(b) and 10(b) plot the story drifts corresponding to points (1) yield, (2) maximum and (3) ultimate displacements. From these figures, the following observations are highlighted:

- All beams and columns (except the column bases) remained elastic or underwent only slight inelasticity, at least until the system reach the maximum resistance (point 2).
- Members that developed inelastic behavior after the yielding point (1) were PRCC and panel zones.
- The above two points confirm that the initial design consideration, in which the ductility of the system was mainly provided by the connections and the panel zone, with elastic behavior in the beams and columns (excluding the base), is fulfilled.
- Note that drifts at the maximum strength (point 2) are higher than the drift limit, which is 0.03 in the local building code (NTC-DS [7]) for special or ductile systems. This is attributed to the high ductility of the PRCC and the panel zones. However, because the degradation parameters are not activated in a static analysis, these results are more optimistic than those obtained from a dynamic response analysis.
- The overstrength factors (R in the local building code [7]) for the frames varies from 2.7 to 3.7. These values are higher than the overstrength factor (R=2) in the local building code NTC-DS [7], but close to the overstrength factor (Ω=3) for C-PRMF systems in ASCE/SEI 7-10 [13], and close to the overstrength factor (R_o = 2.5 to 3.0) for special or ductile systems as proposed in an alternative local specification (MDOC-CFE [14]). However, cyclic degradation is not triggered in these static analyses with monotonic loading, and therefore, the structure exhibits greater strength than that developed in cyclic loading.
- The ductility values, obtained as $\delta_u/\delta_{y,eff}$ as per FEMA P695 [8], are 7.890 for the frame 4S1, 6.803 for frame 4O1, and 5.427 for the frame 8S2. These values are also higher than those assumed in the design (Q = 4, 2 and 4, respectively). This highlights the assumption that C-PRMF are ductile systems, including the one designed as an ordinary frame (i.e., Q = 2).



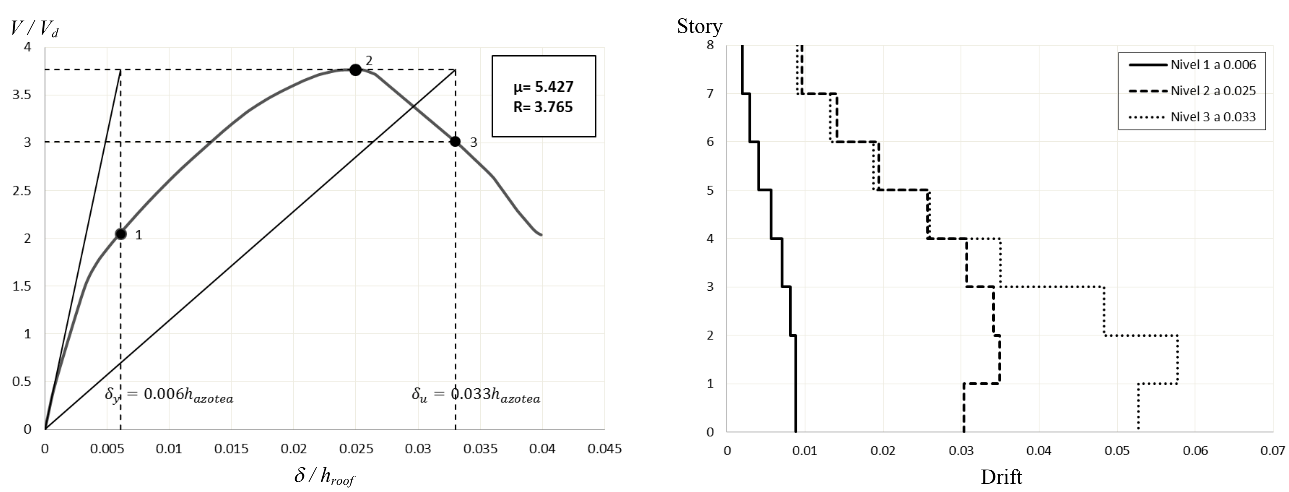
(a) Capacity curve (b) Interstory drifts

Fig. 8 – Frame 4S1



(a) Capacity curve (b) Interstory drifts

Fig. 9 – Frame 4O1



(a) Capacity curve (b) Interstory drifts

Fig. 10 – Frame 8S2

6 Nonlinear dynamic analysis

This section presents the results of nonlinear dynamic analyses for the three frames using eleven ground motion records that are summarized in the Table 3. This dynamic analyses were performed using ground motion records from Mexico, United States (Northridge), and Japan (Kobe). All used records were chosen to fulfill FEMA P695 [8] and other requirements that are believed relevant recommendations. The requirements to be met are:

1. Since the site (Mexico City) is located away from the seismic sources, all chosen records were "far field"; i.e., recorded at a distance greater than 10 km from the fault.
2. According to FEMA P695 [8], the ground motion records must have a peak acceleration (a_{max}) of at least 0.2 g, and a peak velocity (v_{max}) of at least 15 cm/s. In general, these parameters represent the range in which structural damage is generated.
3. The earthquake magnitude (M_w) should be greater than 6.5.

The response spectra can match a target spectrum from: (1) scaling for a specific period the amplitude of the spectrum, or (2) to match multiple periods leveling several spectra with the Spectral Matching technique. The great advantage of spectral matching over scaling is that the spectrum is amplified in the fundamental period and periods corresponding to higher modes. To make the spectral matching respect to the elastic design spectra of the local code (NTC-DS [7]), the software *SeismoMatch* was used, which is a program capable of adjust several records to a target spectrum using an algorithm of frequency manipulation proposed by Abrahamson [15] and Hancock [16].

Table 3 – Ground motion records used in the dynamic analysis

Station	a_{max} (cm/s^2)	v_{max} (cm/s)	Date	Magnitude	Distance (km)
MZ01	384.29	31.37	09/10/1995	8	51
DELS	343.64	32.98	15/10/1979	6.6	35
PTSU	310.98	25.19	11/01/1997	6.5	102
SICC	309.61	23.05	14/03/1979	7.0	114
ZACA	260.90	29.16	19/09/1985	8.1	84
CSER	199.30	15.41	15/06/1999	7.0	90
CHI1	167.10	57.00	19/09/1985	8.1	341
SCT1	158.40	25.30	19/09/1985	8.1	399
TLHB	152.10	54.13	19/09/1985	8.1	406
Beverly Hills - Northridge	492.46	74.308	17/01/1994	6.7	13.25
Canyon Country - Northridge	457.15	53.118	17/01/1994	6.7	11.90
Kobe - Nishi - Akashi	474.80	33.658	17/01/1995	6.9	10.10
Kobe - Shin - Osaka	265.80	41.827	17/01/1995	6.9	19.15

Figs. 11 and 12 show the maximum story drifts extracted from the dynamic analysis for the three evaluated frames. From these analyses, the following observations are highlighted:

- Beams and columns remained elastic or only show slight inelasticity through the time-history records.
- The elements that developed high inelasticity behavior were the PRCC and the panel zones.
- The above two points confirm the initial design consideration, in other words, the ductility for the C-PRMF system is mainly provided by the PRCC connections and the panel zones, while beams and columns (excluding the column bases) shall remain elastic.
- No collapse mechanism compromising the system stability was developed in the analysis. However, results show a considerable damage, especially in the internal panel zones, and in the PRCC for the lower stories.
- The maximum interstory drifts from the dynamic analysis are also higher than the drift limit in the local building code (NTC-DS [7]), which is 0.30 for special or ductile systems, and 0.015 for ordinary ductility systems. This is attributed to a high ductility behavior of the PRCC connections and the panel zones. However, note that the maximum drifts from the dynamic analysis, where the connection response suffered pinching and degradation, are not as large as those developed in the static analysis.

- For the case of the ductile 8-story frame (8S2), the achieved lateral displacements from the dynamic analyzes are high, despite the high rigidity provided by the selection of heavy sections for beams and columns. It is therefore possible to obtain an acceptable behavior for 8-story frames on firm soil (site class or zone I) and ductile detailing (*Q*=4); however, this design is not economical on soft soil (site class or zone III).

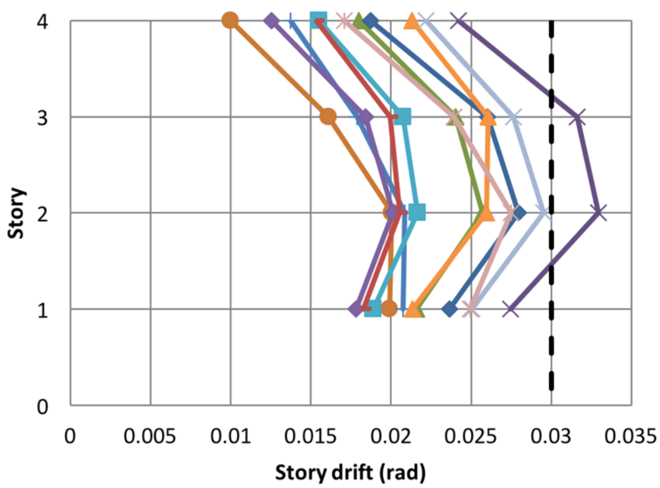


Fig. 11 – Maximum drifts of archetype 4S1

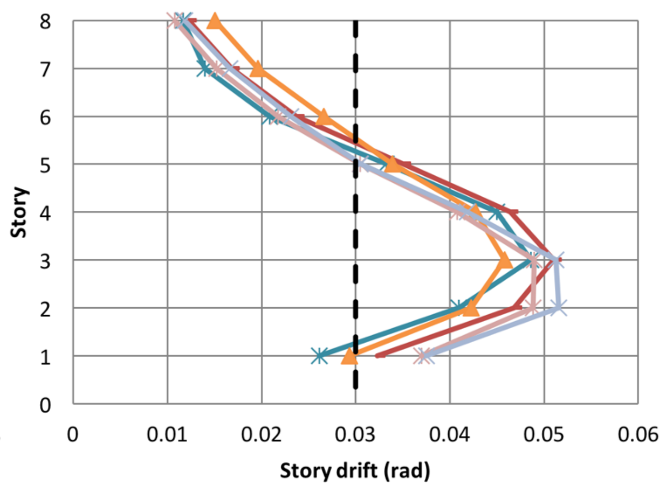


Fig. 12 – Maximum drifts of archetype 8S2

7 Conclusions

The behavior of C-PRMF systems can be properly assessed if the nonlinear behavior of the PRCC connections is included in the analysis. A capacity design criterion is recommended given the ductile nature of the PRCC connections and the panel zones, in which connections and panel zones develop large inelastic behavior, whereas the beams and columns have sufficient capacity to remain essentially elastic.

This paper described some efforts to promote the use of C-PRMF systems and PRCC connections in Mexico City. Even when this system has been implemented in the codes [1, 2] and in real projects of the United States, the adaptation to Mexico City requires the evaluation of the local seismicity, class sites, structural configurations, and the local standards. In order to meet this goal, analytical calibrations to experimental results of PRCC connections tested at the laboratory were performed. In addition, the influence of PRCC connections on the behavior of moment frames under local seismic loads through nonlinear static and dynamic analysis was performed.

The results of this study suggest that, although C-PRMF systems are potentially applicable in Mexico City under certain conditions, these systems are economically convenient when:

(a) Frames are built on firm soils (class site or zone I) or transition soils (class site or zone II) of Mexico City; this is due to lower seismic demands in these sites.

(b) In 4-story frames with up to 12 m. high, a condition that allows continuous columns without splices; 8-story frames with up to 24 m. are possible with acceptable performance only on firm soils (class site or zone I).

(c) Frames are designed as special frames with high ductility (*Q*=4), or as intermediate frames with moderate ductility (*Q*=3); these ductilities are possible due to the great deformation capacity that can be accommodated by the PRCC connections and the panel zones, as long as both are detailed as ductile, which can be achieved by meeting the requirements of the AISC Seismic Provisions [1] and the No. 8 AISC Design Guide [2]. Special care should be taken in the PRCC connections of the lower stories and the panel zones of internal connections.

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