Performance evaluation of a precast beam-to-beam concrete connection using pushover analysis

Hashem Shariatmadar

Assistant Professor, Civil Engineering Department, Semnan University, Semnan, Iran

hamed Asgari

M.Eng. Graduate Student, Civil Engineering Department, Semnan University, Semnan, Iran

Abstract

This research is performed to compare behavior of a specific precast moment resisting frame to an emulative monolithic one's. In this study, the experimental data are used which are based on the precast connection tests conducted in the Middle East Technical University [1-2]. Six beam-to-beam connection subassemblies were tested under reversed cyclic loading simulating severe earthquake action. The first tested specimen was a monolithic specimen (MR1) and used as a reference specimen to define cast-in-place connection. Other specimens were precast elements that each one had its unique properties. The first precast specimen was the original specimen (PO1) designed and detailed by the company in cooperation. Others had some modification to improve the response behavior. In this research, the modified specimen PM1 and the original specimen PO1 are compared with reference monolithic specimen MR1. The hinge properties are derived based on the experimental hysteresis curves, and then implemented in nonlinear computer program. The nonlinear push-over analysis are preformed for a three stories-two bays frame designed using each mentioned connections. To investigate the behavior of the specimens, the results are discussed and compared to each other. Finally, based on the evaluation of all the results conclusion are made. In order to suggest the best substitution for monolithic cast in place connection, several recommendations are also given in the context of the paper.

Key words: Beam-to-beam connection; Precast Concrete; Performance-Base-Design; Nonlinear Push-over Analysis; Hinge Properties.

1. Introduction

All structures in the case of industrials, parking, officials, hotels, schools or bridges and other structures can be making by precast concrete. This type of structure has many advantages, such as reduction of construction time, best quality of construction, increase speed of construction, totally reduction of costs and etc. Connections between precast members normally constitute the weakest link in the structure, therefore, satisfactory performance and economy of precast concrete structures depends to a great extent on the proper selection and design of connections. Test results on full scale structures have shown that the connections start yielding even when steel in the precast panels is in the elastic range. This is because the strength of a connection is normally less than that of the surrounding panel, and this strength can decrease with cyclic loading. Connections can be used to dissipate energy if they show stable elasto-plastic behavior. In recent years many of codes and specifications prefer to study and design structures by considered plastic region and level of performance which this method called "Performance-Base-Design "[3-4]. In this method we allow to structures to have a plastic behavior from yielding point up to end of strength hardening and divided this region in to three levels from immediate occupancy to collapse prevention and study behavior of a structure after performing of a nonlinear static analysis. FEMA 356 divided moment resisting frame into two sections: Precast concrete frames that emulate cast-in-place moment frames and precast concrete moment frames constructed with dry joints in this study we are going to discuss about the precast joint that emulated cast in place. Joints can rightly be asserted as the weakest and the most critical points of a precast concrete structure. Precast concrete frame construction is not used extensively in high-seismic regions of most countries [5–8]. Iran is located at around one of the most active fault zones in the world and is exposed frequently to destructive earthquakes [9–11]. In recent years Iran going to become an industrial country and the demand of prefabricating structures going to increase on the other hand the government going to retrofit or replace old and masonry buildings with new ones that make according to the fresh building codes. Because of lack of time and big amount of old buildings prefabricating is important and because of Iran position moment resisting system is the lower costs and the best icon.

2. Experimental data

In this section, the details of test specimens are explained. The middle beam was connected to the rest of cantilever beams which were extended from columns with wet in-situ concrete above of connection (Figure 1). Within the connection region, the top reinforcement is continued by lap splicing.

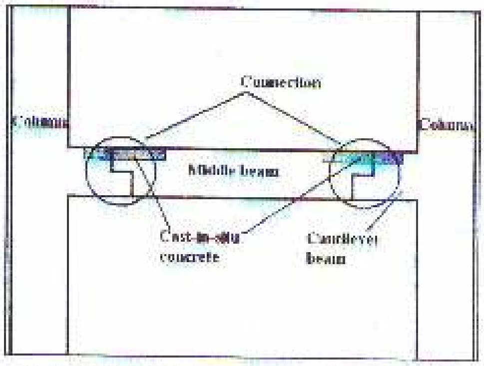

Figure 1: Precast frame and connection under study.

Bottom reinforcement is continued by welding the two steel plates together which are anchored to the bottom of the middle and cantilever beams. All details of beam-to-beam connection are shown in Figures 2 and 3.

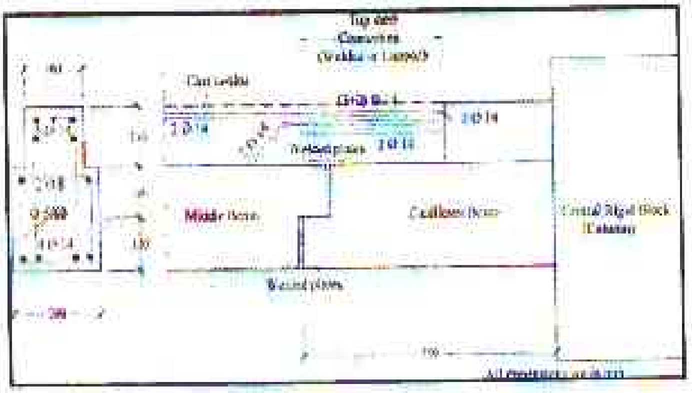

Figure 2: Dimensions and reinforcement details of the specimen.

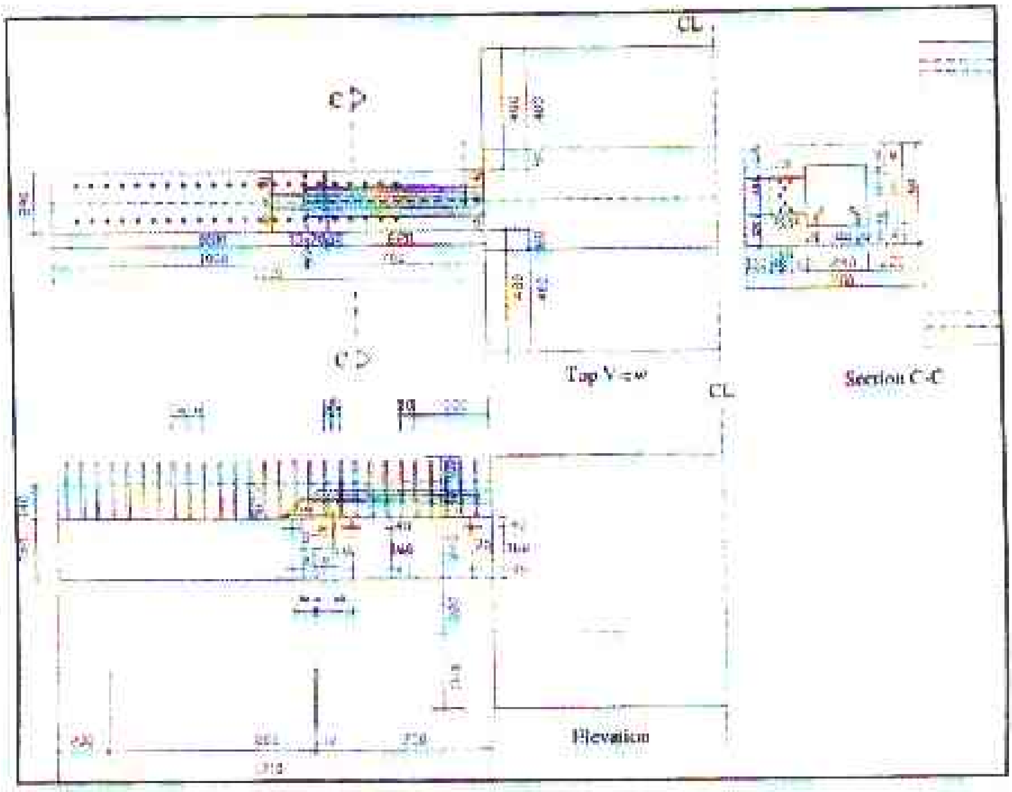

Figure 3: Details of precast specimens.

The first specimen (MR1), cast monolithically, was assigned as the reference specimen, designed for comparison purposes. This specimen was tested to figure out the cast-in-place connection as a reference behavior. The second tested specimen (PO1) was the original precast specimen designed and detailed by the company in cooperation. The specimen PM1 had some modification to improve the behavior of original precast specimen PO1. Details of specimen PO1 as the base of used specimen PM1 are shown in Figure 4 and the properties of these three specimens are given in Table 1.

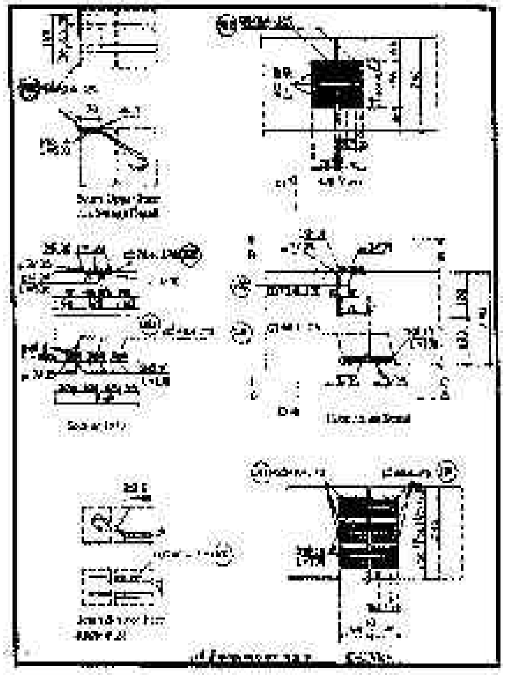

Figure 4: Connection detail of precast original specimen PO1.

Table 1: Properties of test specimens

Specimen	Longitudinal steel		Top steel connection		Bottom steel connection		
	Top	Bottom	Type	Length	Bars	Weld length	Weld plate anchorage
MR1	4φ14 (+2 Φ12)	2φ14	Continuous				
PO1	4φ14	2φ14	Lapped	350 mm (25db)	[illegible]	35 mm	3 x 2φ (L = 400 mm)
PM1	4φ14	2φ14	Welded		[illegible]	40 mm	3 x 2φ (L = 400 mm)

2.1. Monolithic specimen MR1

The monolithic specimen MR1 was designed for comparison the behavior of cast-in-place connection with precast connections. The dimensions of the beam and reinforcement detaillings were identical with other precast connection specimens.

2.2. Precast original specimen PO1

The precast specimen PO1 was designed with a connection detail proposed by the company in corporation. In specimen PO1, where the top bars were lap spliced, premature failure was observed due to rapid anchorage deterioration in the lap splice. Thus it is deduce that the lap splice length and the bottom beam-to-beam connection details were unsatisfactory. Contrary to the expectation, significant deformation and hinging formed in the connection region where maximum moment values were foreseen.

2.3. Precast modified specimen PM1

In specimen PM1 the top bars of the middle beam were welded to those extending from the central block. Besides, one half of the bottom bars (two bars) in the middle beam were bent up and welded with "L" shaped bars in the connection region at the top. The welded top steel connection was introduced as an alternative to the lap splice. The result of hysteresis response curves for connection region and root for all three specimens are presented in Figures 5 and 6, respectively.

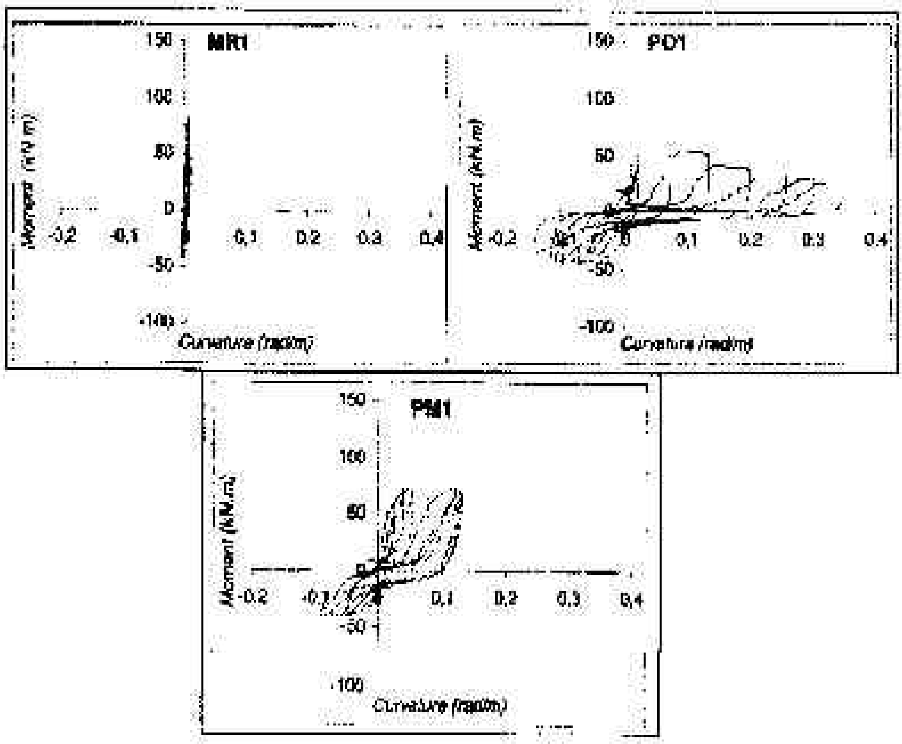

Figure 5: Moment-curvature hysteresis responses of specimens for connection region.

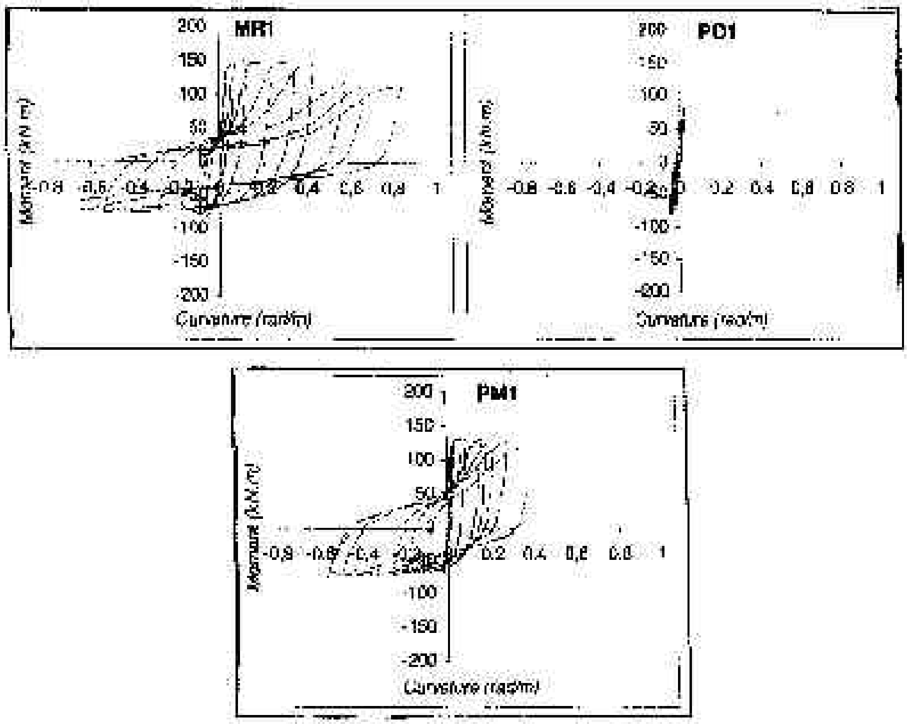

Figure 6: Moment-curvature hysteresis responses of specimens for root region.

3. Connection modeling and Push-over analysis

Three stories-two bays frame is selected for this study is shown in Figure 7. The lateral load resisting system is ordinary moment resisting frame with structural response factor of R=7. The dead and live load of 2.5 ton/m and 0.8 ton/m are considered, respectively. It is assumed that the structure is located in the high seismic region on the soil property Type II. All these data extract from Iranian code of practice for seismic resistant design of building, standard No. 2800-05, 3rd Edition [15]. The importance building coefficient is selected from third group given I=1. The structure was designed by ACI-318-99 code. The columns dimension were 400×400 after designing and include 8φ20 with cross tie and the beam details are presented in Figure 2. Concrete compressive strength and reinforcement yielding strength were 30 MPa and 400 MPa, respectively.

The hinge properties are derived from experimental hysteresis loops by the methods and commands given in FEMA 356, Chapter 2 [13]. The backbone curves for the discussed specimens MR1, PM1 and PO1 for both connection region and root are computed and presented in Figures 8 to 10. Then numerical values extracted from backbone curves and listed for all specimens in Tables 2 to 4.

All hinges properties are modeled and implemented into beam root and beam connection in the frames, consequently, the structure with all calculated characteristics are modeled in nonlinear computer program [14]. Finally, the nonlinear push-over analyses are performed and the analytical results are compared for precast and cast in place frames.

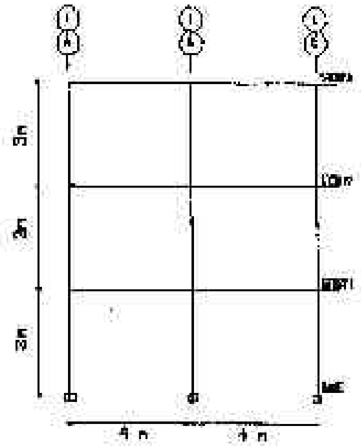

Figure 7: Elevation of the frame

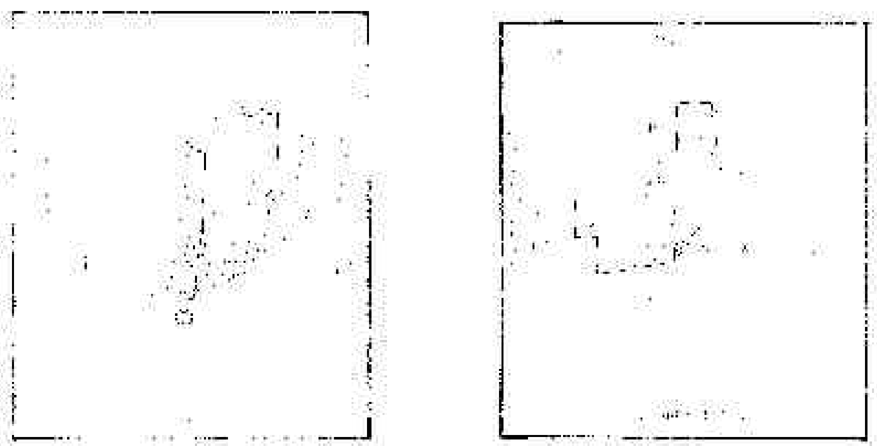

Figure 8: Backbone curves for specimens in root region

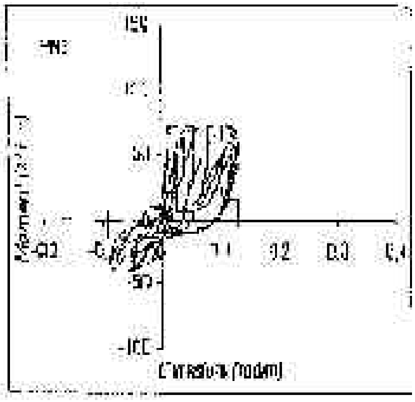

Figure 9. Backbone curve for specimen PM1 in connection region

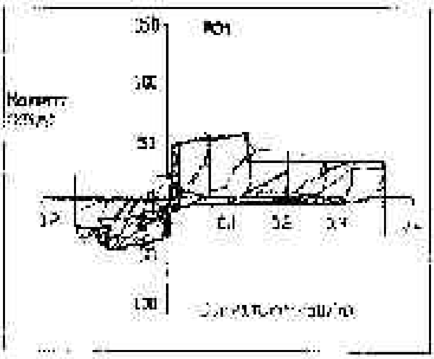

Figure 10. Backbone curve for specimen PO1 in connection region

Table 2. Numerical values for backbone curve in root region for specimen MR1

point	Moment (KN.m)	Curvature (rad/m)
E	80.2	0.82
D	80.2	0.41
C	145.4	0.39
B	131.2	0.02
A	0	0
B'	81.1	0.08
C'	81.1	0.62
D'	30.1	0.62
E'	30.1	0.75

Table 3. Numerical values for backbone curve in root region for specimen PM1

point	Moment (KN.m)	Curvature (rad/m)
E	52.2	0.42
D	52.2	0.25
C	127.3	0.22
B	126.4	0.015
A	0	0
B'	53.3	0.035
C'	57.2	0.43
D'	31.2	0.43
E'	31.2	0.57

Table 4. Numerical values for backbone curve in connection region for specimen PM1

point	Moment (KN.m)	Curvature (rad/m)
E	10.2	0.13
D	10.2	0.11
C	74.3	0.11
B	74.3	0.01
A	0	0
B'	40.1	0.02
C'	43.2	0.56
D'	12.2	0.56
E'	12.2	0.92

Table 5. Numerical values for backbone curve in connection region for specimen PO1

point	Moment (KN.m)	Curvature (rad/m)
E	38	0.35
D	38	0.13
C	55	0.13
B	50	0.03
A	0	0
B'	43	0.05
C'	51	0.1
D'	32	0.1
E'	32	0.13

4. Discussion of analytical results:

In this section we selected one of the lateral load groups to assign on the frame for push over analysis. Because the frame was short and the mode one was the dominated mode we selected group one which was gravity load as initial load and model, uniform load and triangle load. The triangle load is calculated by redistribution of lateral load using Standard 2800-05, 3[rd] Edition [15]. The result of pushover analysis for each frame for triangle is drawn in the Figures 11 to 13.

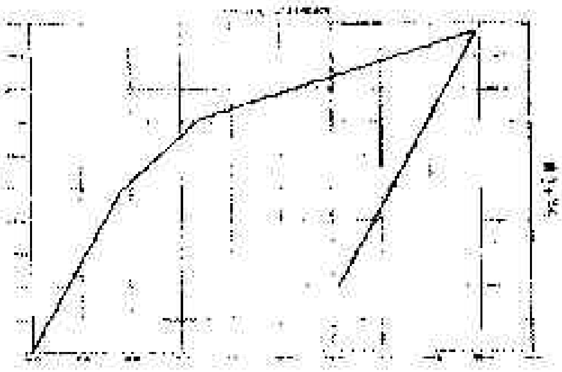

Figure11: base shear versus deformation for specimen MR1 (ton-cm)

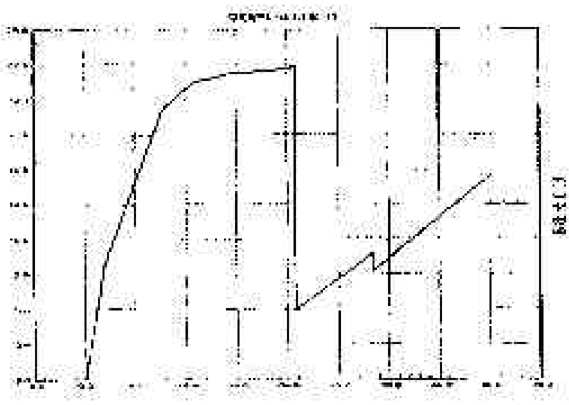

Figure12. base shear versus deformation for specimen PM1 (ton-cm)

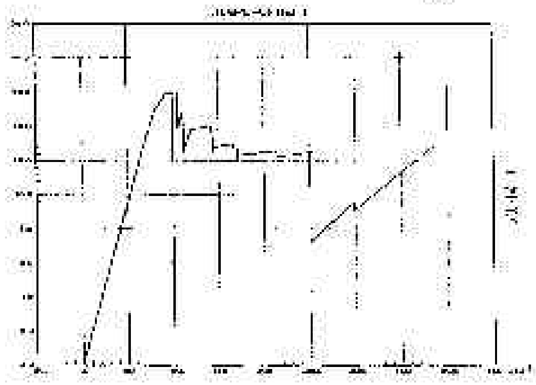

Figure13: base shear versus deformation for specimen PO1 (ton-cm)

5. Conclusion

The design recommendations and conclusions are based on analytical results. The conclusions and design recommendations are summarized below:

- The original connection details PO1 is not suitable for high seismic zone and it is not recommended to be used instead of cast in place connection. As it is showed in the above figure, the structure that is made by this connection doesn't have a nonlinear region, and actually acts as an elastic code designed structures.

- Significant improvements are achieved by introducing the modifications proposed as given for connection type PM1.

- All response curves show that after occurring one new moment hinge in the beams the structural ductility increases, where, the total strength decreases.

- In the frame modeled with PM1 connections as is presented in the Fig.12 the lateral load increase up to 22 ton without decrease in total strength and deformation in this point is approximately 21 cm.

• In the frame analyzed with MR1 connections as is presented in the Fig.11 the lateral load increase up to 29 ton without decrease in total strength and deformation in this point is approximately 18 cm.

• From these two results that mentioned above we can understand that the structure with PM1 connections has a weaker behavior than the structure with MR1 connections in total strengths, but has a better ductility and energy dissipation.

6. References

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