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Seismic study of cable-cylinder bracing under near field records

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Abstract

Recently, the researchers have growingly focused on the cable bracing systems, among which new cable-cylinder bracing can decrease the drift and increase the ductility of structures, comparing to cross-cable bracing. Directivity pulse and fling-step are of exclusive specifications of near filed records. For the first time, overstrength factor, ductility factor and response modification factor of cable-cylinder bracing system are computed by two-dimensional model, in this research. Accordingly, the cable-cylinder bracing system is subjected to 10 near filed earthquake records for conducting incremental dynamic analyses. Moreover, the values of response modification factor are calculated and compared for moment frames with cross-cable bracing and cable-cylinder bracing. Based on the results, cable-cylinder bracing works better than cable bracing, regarding its higher response modification factor, in comparison with that of cross-cable bracing.

Keywords:

cable-cylinder bracing, near field records, incremental dynamic analysis, response modification factor, overstrength factor.

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Introduction

A new bracing system has been presented by Tagawa and Hou (2007). This system has two cables and one hollow cylinder through which the cables pass at their crossing region, Figure 1. (Tagawa and Hou 2007). The pipes, used for the cylinder, can be steel with high stiffness or PVC with low stiffness.

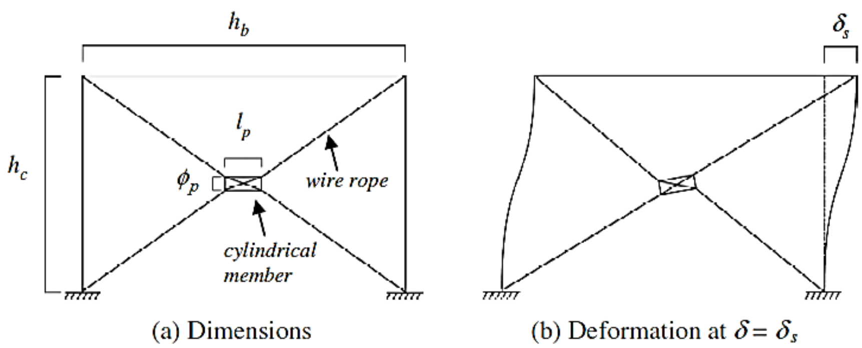


Figure 1. Cable-cylinder bracing system: a) dimensions; and b) deformed frame (Tagawa and Hou, 2007).

In the cable-cylinder bracing systems:

1. The drifts of storeys are limited without increasing their base shears. The compression force of the columns less increases comparing to cross-cable bracing system (Fanaie and Aghajani, 2012).
2. The stiffness of system is almost zero in the low drifts. It means that moment frame with cable-cylinder bracing and moment frame have the same periods in the low drifts (Fanaie et al., 2016a).
3. The energy dissipation is higher, comparing to that of cross-cable bracing system.
4. The damage is prevented from being concentrated in a certain storey (soft storey) (Fanaie et al., 2016a).

As mentioned earlier the pipe with high stiffness can be used as the cylinder member. In this case, both bracing members are under tension as the cylinder rotates before the displacement reaches δ_{sr}. Tensile force (T_b) is also appeared in member b, shown in Figure 2. (Tagawa and Hou, 2009).

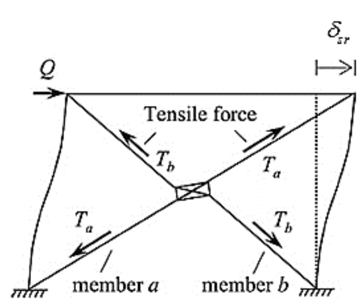
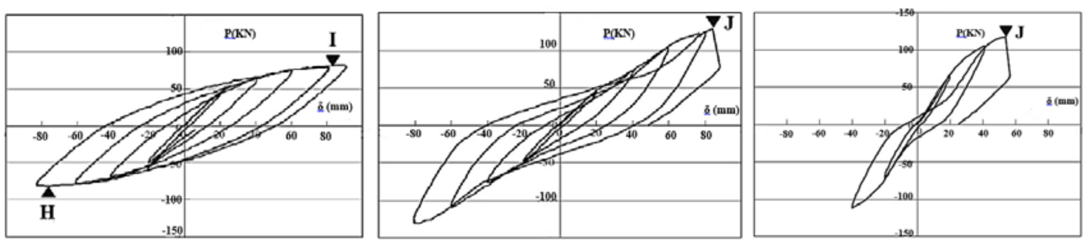


Figure 2. Bracing system with stiff cylinder (Tagawa and Hou, 2009).

In the laboratory investigation, Tagawa and Hou have applied cyclic loadings to the moment frame, moment frame with cross-cable bracing and moment frame with cable-cylinder bracing (Tagawa and Hou, 2009). They plotted hysteresis curves for the mentioned structures, presented in Figure 3.



(a). Specimen A (b). Specimen B (c). Specimen C

Figure 3. Hysteresis curves for laboratory samples (Tagawa and Hou, 2009).

Based on the figure, braced frames present lower ultimate displacements comparing to the frame without bracing. Moreover, shear storey of the frame with cable-cylinder bracing is lower comparing to that of cross-cable braced one. The capability of energy absorption of cable-cylinder bracing is higher than that of cross-cable bracing (Tagawa and Hou, 2009). Cable-bracing system with stiff cylinder has two advantages, considering the conducted investigations. Firstly, its cable reaches the ultimate strength in higher storey drift; therefore, the ductility of the frame increases, compensating the cable's ductility weakness. Secondly, under a considerable range of loading, both cables are in tension; under lateral displacement, loosening occurs in none of the cables; hence, the impact caused by cables loosening is not observed.

Fanaie et al. (2016a) showed that the needed cross-section area is lower in cable-cylinder bracing comparing to cross-cable bracing, considering the fact as one of the advantages of cable-cylinder bracing system. They also presented the equations which govern the behavior of stiff cable-cylinder bracing (like steel cylinder) in another research, Fanaie et al. (2016b). In this research they assessed the effects of cylinder dimensions and prestressing of cables on the behavior of cable-cylinder bracing.

It is assumed that the frame is displaced in one direction and one of the cables becomes straight. Then, 1) the cables, inside and outside the cylinder, are placed in the same directions; 2) the strain becomes zero in the opposite cable (Aghajani, 2011).

Definition and characteristics of near-fault ground motion

It is assumed that near-fault ground motion is typically restricted to the distance of 20 km from a fault (Li and Xie, 2007). The effects of near-fault motions are attenuated with the increase of their distances from the fault, resulting in the higher effects of magnitude and local site conditions on the ground motion. Near-fault ground motion is distinguished with the pulses generated by directivity and fling-step effects (Bolt and Abrahamson, 2003). Such pulse-type ground motion usually has one or more distinct pulses in the acceleration, velocity (most frequently) and displacement time histories.

The directivity effects are categorized into forward, backward and neutral, considering the relative position between rupture direction and site location. If the fault rupture is propagated toward a site with a velocity close to shear wave velocity, causes most of the seismic energy from the rupture to arrive in a single large pulse of the motion, occurred at the beginning of the record. This phenomenon is called forward directivity (Somerville, 2003). This large pulse of the motion is oriented normal to the fault plane by the radiation pattern of shear dislocation on the fault. Consequently, strike-normal component of the ground motion will be larger than its strike-parallel component at the periods higher than about 0.5 s (Somerville, 2003). Backward directivity effects, occurred when the rupture is propagated far from the site result in long duration motions of low amplitude at long periods. (Howard et al., 2005; Davoodi and Hadiani, 2010).

Fling-step is another important specification of near-fault ground motions. This property is the permanent static displacement in the fault-parallel and fault-normal directions for strike-slip and dip-slip faults, respectively. Fling-step is resulted from permanent ground displacement and generates one sided velocity pulses. However, forward directivity is a dynamic phenomenon that produces no permanent ground displacement (Bray and Rodriguez-Marek, 2004).

Large velocity pulses can be produced by directivity or fling-step. However, the latter typically involves the motions with longer periods and lower acceleration spectra (Graves, 2003). Near-fault velocity pulses, caused by the earthquakes with the magnitudes (Mw) lower than 7.5–8, have the periods typically lower than 3–4sec. The acceleration spectrum, produced by a strong velocity pulse, has the predominant period approximately 0.75 times the pulse period (Somerville, 2003). Based on the regression analysis, the period of a near-fault velocity pulse depends on the earthquake magnitude and soil-site conditions. The response spectrum has a peak whose period increases with the magnitude, considering the magnitude dependence of pulse. By the way, near-fault ground motions caused by moderate-magnitude earthquakes may exceed those of larger earthquakes at the intermediate periods about 1 sec. (Somerville, 2002; Panza et al., 2011).

Incremental Dynamic Analysis (IDA)

Random intrinsic nature of earthquakes is one of the main uncertainties that should be considered in assessing the seismic behavior of structures. Seismic responses of structures should be determined to quantify such uncertainties. For this purpose, different dynamic analyses are performed in the course of different ground motions. In the present research, Incremental Dynamic Analysis (IDA) is used to investigate earthquake uncertainty. Accordingly, sufficient numbers of records are used to consider the uncertainties in the frequency content and earthquake records spectra shapes (Vamvatsikos and Cornell, 2002). Each earthquake record is scaled in such a way to cover proper ranges of seismic intensities as well as elastic limit to collapse responses of structure. The intensity measure (IM) (eg: PGA or Sa (T1)) is scaled with a proper algorithm for IDA analysis. This scaling is started from a very low amount to a certain level, for motivating elastic response and target collapse state, respectively, in the considered structural model. Time history analysis is performed in IDA, applying different records, obtained from various scale factors. The values of DM (Damage Measure), corresponding to the IMs, used in the dynamic analysis, are determined at the end of each analysis.

It is crucial to select the parameters appropriately for IM and DM in IDA analysis. These parameters should be scalable to be chosen for a proper seismic intensity. In this study, IM is considered as the spectral acceleration of the first mode, including the principal period of structure and taking the earthquake duration and damping parameters into account. Collapse criteria of structures, applied in this study, are joint rotation, inter-storey drift, roof displacement and axial deformation of elements. Also, DM is considered as maximum inter-storey drift to meet appropriate structural response against the records.

Calculating the seismic parameters of structure

Almost all universal codes take response modification factor into account to reduce the calculated earthquake loads and consider inelastic behaviour. Accordingly, the designers are allowed to conduct elastic analysis on the reduced loads and design the structures based on the obtained results. Response modification factor depends on different items such as ductility of structure, material properties, damping characteristics, cooperation of non- structural members, overstrength, and etc.

In the present research, Uang's ductility factor method is used to calculate response modification factor (Uang, 1991). In this approach, a bilinear elastic perfectly plastic relation is applied to idealize real nonlinear behaviour (Figure 4) (Zahrai and Jalali, 2014).

The base shears, shown in Figure 4, are used to express several parameters for calculating response modification factor. Different structures have different values of overstrength under different earthquake records. Overstrength factor which is very important in the behaviour of structure against earthquake is computed by IDA in this research. Maximum base shear is obtained by IDA applying the method presented by Mwafy and Elnashai (2002). Besides, a structural model is subjected to one (or more) ground motion record(s), scaling each of which to multiple intensity levels (Vamvatsikos and Cornell, 2003). Overstrength factor is expressed as follows:

$$R_s = \frac{V_{b(Dyn,u)}}{V_{b(st,y)}} \quad (1)$$

where, $V_{b(Dyn,u)}$ is maximum dynamic base shear corresponding to the collapse or considered damage of structure; and $V_{b(st,y)}$ is static base shear corresponding to the first plastic hinge formation. Actual lateral strength against design lateral strength of the structure is considered by overstrength factor.

The results of IDA and linear dynamic analysis are used directly in the method presented by Mwafy and Elnashai (2002), to obtain ductility factor, defined as follows:

$$R_\mu = \frac{V_{b(Dyn,el)}}{V_{b(Dyn,u)}} \quad (2)$$

where, $V_{b(Dyn,el)}$ is maximum dynamic base shear corresponding to the elastic behavior of structure causes its collapse under the earthquake. $V_{b(Dyn,u)}$, is obtained by increasing the spectral acceleration of earthquake record (the intensity measure applied in this study) until forming the mechanism in the structure or meeting the considered damage. The spectral acceleration that results in the mechanism or damage is basically accepted as the ultimate limit in which the corresponding base shear is obtained. Moreover, dynamic analysis is used also to calculate maximum linear base shear of the structure, assuming its elastic behaviour under the same spectral acceleration. The base shear, corresponding to the first plastic hinge, is computed by nonlinear static analysis. The obtained shear base is used to calculate overstrength factor. In both static and dynamic analyses, the end of linear zones, corresponded to the first plastic hinge, are considered the same (Mwafy and Elnashai, 2002). The ductility factor is a function of structural system type, quality of connections, number of stories, etc. V_s is reduced to V_w by allowable stress factor (Y) in the design codes, defined as follows (Uang, 1991):

$$Y = \frac{V_s}{V_w} \quad (3)$$

where, V_s is the maximum base shear corresponded to the first significant yield of structure; and V_w is the base shear presented in allowable stress design method. In this research, allowable stress factor (Y) is considered as 1.44. Actually, response modification factor is originated from strength reduction factor due to ductility (R_μ) and overstrength factor (R_s).

Response modification factor with ultimate strength method is defined as follows:

$$R_u = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_\mu \times R_s \qquad (4)$$

Response modification factor with allowable stress design method is expressed as follows:

$$R_w = \frac{V_e}{V_y} \times \frac{V_y}{V_s} \times \frac{V_s}{V_w} = R_\mu \times R_s \times Y \qquad (5)$$

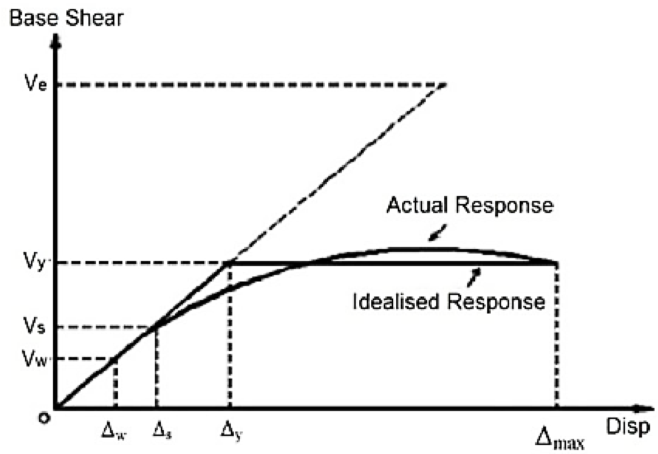


Figure 4. Elastic and inelastic responses of structure (Uang, 1991).

Applied models

In this research, three frame models are considered with the same specifications, excluding the sections of beam and column, the first and second ones for verification in OpenSees software. In these two models, 3- and one-storey frames are used, respectively, assuming rigid beam and box section for columns. The third model is considered for pushover and incremental dynamic analyses, using one-storey frame with the beam of IPE section, equivalent to the box section. The dimensions of box section are 200×200×8; and its plastic section modulus is 442.6 cm3. The nearest plastic section modulus to this value is corresponded to IPE270. As box sections are not ordinarily applied in the one-storey buildings, IPB200 section has been used for the columns. The heights of storey and span of the studied frames are 3.5m and 5m, respectively. Bracing complex, the cables and cylinder, are modeled in the form of truss. The axial stiffness of cylinder and its inner cables is 1500 times that of outside cables (α and β= 1500) (Tagawa and Hou, 2009), Figure 5. The specifications of cable-cylinder bracing system are presented in Table 1. (Tagawa and Hou, 2009; Nolan and Domenico, 1995).

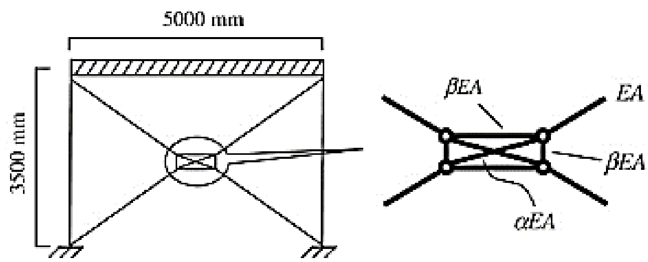


Figure 5. Modeling cable and cylinder (Tagawa and Hou, 2009).

Table 1. Specifications of cable-cylinder bracing system.

Modulus of elasticity of cable (MPa)	Cable's section area (mm^2)	Ultimate tensile strain of cable	Diameter of cable (mm)	Internal diameter of cylinder (mm)	Length of cylinder (mm)
137000	374	0.015	28	200	703

Modeling in OpenSees software

In this research, nonlinear static and incremental dynamic analyses are modeled and performed in OpenSees software ver.2.4.5 (Mazzoni et al., 2007). Beam and column are modeled using nonlinear beam-column element and fiber section. Steel02 bi-linear material with 1% hardening has been applied to model nonlinear behaviors of the elements. The cylinder is defined in the performed modeling in the two-dimensional form, using four cable elements. The cables are modeled using corrotational truss element, two- end hinged element; they have no compressive strengths and work only in tension. Therefore, this specification is expressed using elastic-perfectly plastic material. InitStressMaterial with initial stress is considered for applying prestressing in the cables. The masses of storeys are concentrated in the nodes.

Model verification in OpenSees software

OpenSees software is used to analyze two-dimensional 3-storey frame with the specifications of one-storey frame (the first model). The purpose is to verify the accuracy of modeling and appropriateness of selecting the material and elements for beam, column, cable and cylinder. The value of fundamental period, calculated in this research, is 0.69 sec., exactly equal to that of ANSIS in Tagawa model. Moreover, pushover curves have been plotted for one-storey frame based on the results of nonlinear static analysis, Figure 6.

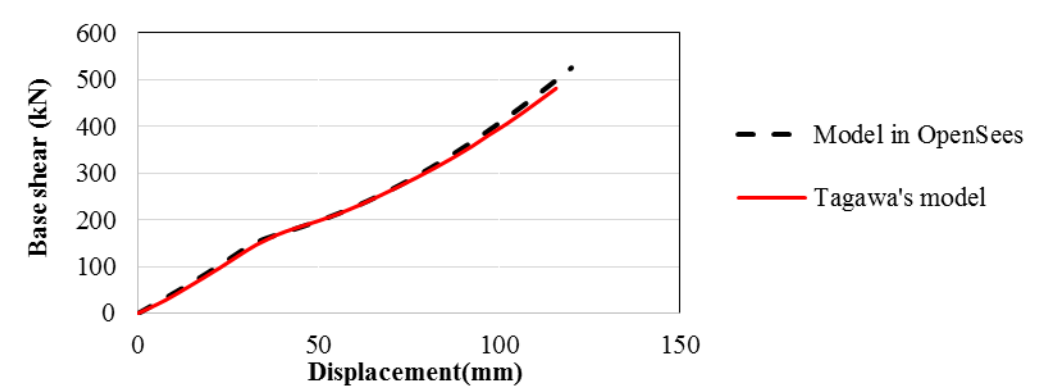


Figure 6. Pushover curves of the frame modeled in OpenSees and Tagawa's model.

Based on the figure, the pushover curves, obtained from OpenSees and Tagawa's model are coincided in the nonlinear region with very slight difference. This fact verifies geometric modeling, selecting parameters for material modeling, conditions of modeling the connections of structural elements and most particularly bracing elements in OpenSees software.

Analysis of the results

Nonlinear static analysis

Figure 7. presents the pushover curves, plotted for moment frame with cable-cylinder bracing system and moment frame with cross-cable bracing. Target displacement has been calculated in the braced frames, using 0.02h value (ASCE 2007).

The values of base shear corresponding to the first yielding are given in Table 2.

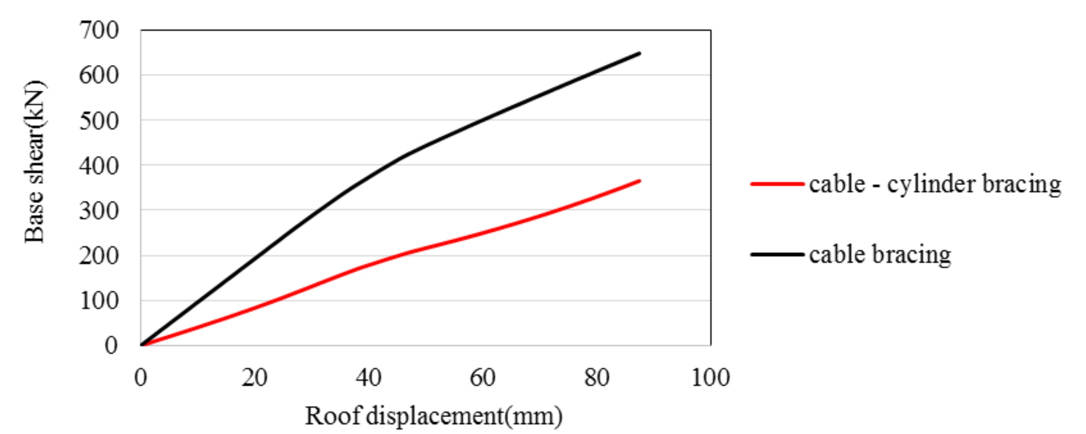


Figure 7. Pushover curves.

Table 2. Shear base corresponding to the first yielding.

System Type	Base Shear Corresponding to the First Yield (kN)
MRF+ Cross-Cable Bracing	361.64
MRF+ Cylinder-Cable Bracing	155.89

Selected records for IDA

Incremental dynamic analysis has been conducted on the considered frames, using 10 near filed records of the world well-known earthquakes (Davoodi et al., 2013; Dimakopoulou et al., 2013). Table 3. presents the specifications of these records. IDA curves, plotted for cable-cylinder bracing system, have been represented in Figure 8.

Table 3. Specifications of the near field records used in IDA.

Record No.	Record	Station Name	Date of Occurrence	PGA(g)
1	chi chi -Taiwan	TCU052	09/20/1999	0.35
2	Northridge	USGS 655 Jensen Filter Plant	01/17/1994	0.57
3	Coyote Lake	Gilroy Array	08/06/1979	0.43
4	Parkfield	Temblor	06/28/1966	0.27
5	Mammoth Lakes	Long Valley Dam	05/27/1980	0.95
6	Morgan Hill	Coyote Lake Dam	04/24/1984	1.30
7	San Salvador	Geotech Investig Center	10/10/1986	0.70
8	Bam Iran	Bam	12/26/2003	0.81
9	San Fernando	Pacoima Dam	02/09/1971	1.23
10	Tabas	Tabas	09/16/1978	0.85

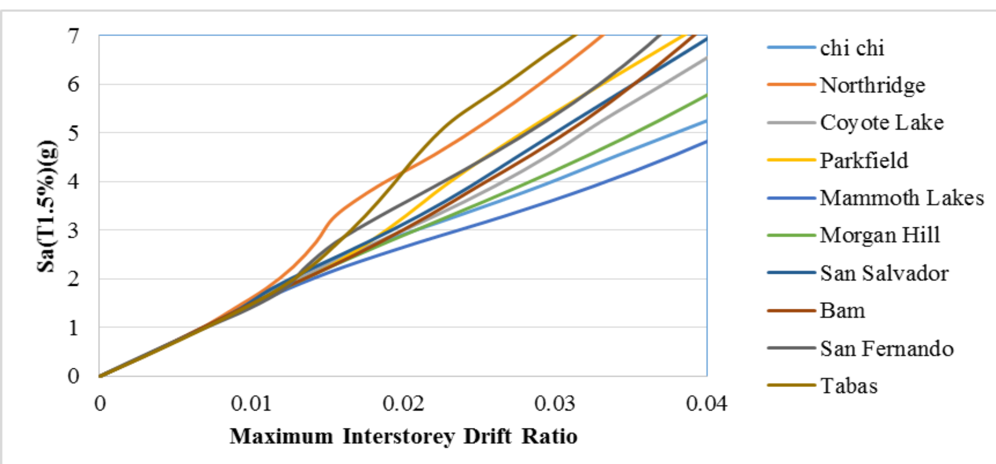


Figure 8. IDA curves of the moment frame with cable-cylinder bracing system.

Calculating response modification factor

Moment frame with cross-cable bracing and moment frame with cable-cylinder bracing are investigated through nonlinear static, linear and nonlinear dynamic analyses. Ductility, overstrength and response modification factors are calculated for the mentioned frames. In this regard, 10 records are considered in the ultimate limit and allowable stress design methods. Tables 4-5 present the relevant obtained results. These factors are calculated for two kinds of frames, averaging the results, and illustrated in Table 6. According to the tables, the value of ductility factor is higher in the frame with cable-cylinder bracing due to the presence of cylinder, comparing to that of cross-cable bracing. The lengths of bracing cables increase as they are deviated from their diameter directions by the cylinder. Consequently, they reach their ultimate strengths in the higher drifts, causing the increase of ductility. Finally, the response modification factor of cable-cylinder bracing system is higher, comparing to that of cross-cable bracing system.

Table 4. Oversterngth, ductility and response modification factors of moment frame with cross-cable bracing system.

records	DM Max Drift	IM $S_{a(T1,5\%)}$	$V_{b(Dyn,u)}$ (kN)	$V_{b(st,y)}$ (kN)	$V_{b(Dyn,e)}$ (kN)	R_s	R_μ	R_{LRFD}	R_{ASD}
chi chi -Taiwan	0.02	0.47	801.00	361.64	1066.59	2.21	1.33	2.95	4.25
Northridge	0.02	0.87	806.24	361.64	1107.23	2.23	1.37	3.06	4.41
Coyote Lake	0.02	0.59	803.17	361.64	1062.54	2.22	1.32	2.94	4.23
Parkfield	0.02	0.58	800.55	361.64	949.93	2.21	1.19	2.63	3.78
Mammoth Lakes	0.02	1.02	802.54	361.64	1059.32	2.22	1.32	2.93	4.22
Morgan Hill	0.02	1.72	779.19	361.64	877.15	2.15	1.13	2.43	3.49
San Salvador	0.02	1.35	803.21	361.64	1101.63	2.22	1.37	3.05	4.39
Bam Iran	0.02	2.20	806.54	361.64	1240.96	2.23	1.54	3.43	4.94
San Fernando	0.02	1.90	803.53	361.64	1136.84	2.22	1.41	3.14	4.53
Tabas	0.02	2.40	804.72	361.64	1169.27	2.23	1.45	3.23	4.66
average						2.22	1.34	2.98	4.29
sigma						1.49	1.50	0.23	0.30
C.V						0.67	1.12	0.08	0.07

Table 5. Oversterngth, ductility and response modification factors of moment frame with cable-cylinder bracing system.

records	DM Max Drift	IM $S_{a(T1,5\%)}$	$V_{b(Dyn,u)}$ (kN)	$V_{b(st,y)}$ (kN)	$V_{b(Dyn,e)}$ (kN)	R_s	R_μ	R_{LRFD}	R_{ASD}
chi chi -Taiwan	0.02	0.49	360.48	155.89	435.48	2.31	1.21	2.79	4.02
Northridge	0.02	1.68	364.36	155.89	500.76	2.34	1.37	3.21	4.63
Coyote Lake	0.02	0.63	371.59	155.89	435.48	2.38	1.17	2.79	4.02
Parkfield	0.02	0.77	360.48	155.89	599.87	2.31	1.66	3.85	5.54
Mammoth Lakes	0.02	0.72	363.07	155.89	435.48	2.33	1.20	2.79	4.02
Morgan Hill	0.02	1.87	362.94	155.89	442.83	2.33	1.22	2.84	4.09
San Salvador	0.02	1.89	361.22	155.89	478.17	2.32	1.32	3.07	4.42
Bam Iran	0.02	1.45	369.22	155.89	514.26	2.37	1.39	3.30	4.75
San Fernando	0.02	1.87	364.94	155.89	640.88	2.34	1.76	4.11	5.92
Tabas	0.02	2.02	367.07	155.89	1169.90	2.35	3.19	7.50	10.81
average						2.34	1.55	3.63	5.22
sigma						0.58	1.96	1.36	0.63
C.V						0.25	1.27	0.38	0.12

Table 6. Oversterngth, ductility and response modification factors of different structures.

System Type	R_S	R_μ	R_{LRFD}	R_{ASD}
MRF+ Cross-Cable Bracing	2.22	1.34	2.98	4.29
MRF+ Cylinder-Cable Bracing	2.34	1.55	3.63	5.22

Conclusion

In this research cable-cylinder bracing system has been studied under near field records. The results obtained for one-storey moment frame with cable-cylinder bracing are compared with those of moment frame with cross-cable bracing. According to the following findings, seismic performance of the moment frame with cable-cylinder is better, comparing to that of cross-cable bracing:

1. Overstrength factor value is 2.34 for one-storey moment frame with cable-cylinder bracing, slightly higher than that of cross-cable bracing (2.22);
2. Ductility factor value is 1.55 for one-storey moment frame with cable-cylinder bracing, higher than that of cross-cable bracing (1.34). That is cable-cylinder bracing system is more ductile comparing to cross-cable bracing system;
3. Response modification factor value is 5.22, obtained from allowable stress design method for one-storey moment frame with cable-cylinder bracing. This value is higher than that of cross-cable bracing (4.29);

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