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ANALYTICAL EVALUATION OF SEISMIC RESPONSES OF A POWER-PLANT FRAME STRUCTURE WITH ENERGY DISSIPATION DAMPER

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SUMMARY

Case-study is carried out on a large-scale thermal power plant structure with and without energy-dissipating bracing system. The plant is designed for construction in a region of high seismicity in China. The project is 8-story 40.77 meters high, 6-bay RC frame in the longitudinal direction. The seismic design proposed to add X-type steel bracing in the longitudinal frame and to adopt a new type of lead-alloy damper of energy-dissipation bracing system. The study is to carry out elastic and nonlinear time history analysis to variable levels of earthquake input so to provide the design with the information of the effectiveness of the damper installation. The paper presents the results of analytical evaluation on the longitudinal frame's seismic responses. In order to compare the damping effects of different installations, four structural models with and without bracing and dampers are used to conduct nonlinear dynamic response analysis subjected to three different levels of ground motion input, respectively. The peak acceleration levels correspond to the most probable earthquake (or minor earthquake, expected once in the lifespan of 50 years return period), design fortification earthquake (or moderate earthquake, in the area of 8-degree intensity in MM-scale with return period of 475 years), and the rare-intensive earthquake (with return period of 1970 years roughly) according to Chinese Code for seismic design of buildings [1]. The seismic performance of the four structural models is evaluated based on the analysis results (such as the vibration period, earthquake force, story shear force, floor displacements, dynamic amplification factors, etc). It is found that the installation of the lead-alloy dampers are effective to the steel bracing frame structure subjected to the rare-intensive earthquake and it reduces significantly the stress in the frame structure. Setting up the dual seismic resistant system, it can be effectively controlled the deformation of the frame structure and the damage of main structural elements. Based on the investigation, some suggestions are given in this paper towards the optimal structural design.

INTRODUCTION

In conventional earthquake-resistant design, the earthquake action is designed to be resisted by means of strengthening the structural components, such as enlarging the cross sections of structural elements and

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increasing the steel ratio of reinforcement are commonly adopted. However, it does certainly increase the construction cost, and it may not be very effective owing to the uncertainty of earthquake actions and complexity of structural seismic responses. In contrast, energy-dissipation technology, which has developed rapidly in recent years, produces more rational, effective, secure and economical seismic fortification measures for structures. The new approach of the seismic design is different from the traditional way of strengthening main structure. It makes some of the non-load-bearing members or secondary structural members into energy-dissipating elements. Such elements dissipate the energy during the earthquake motion by means of friction, viscosity and plastic deformation, so that the seismic response is controlled and the seismic damage of the main structure is reduced or avoided.

According to the principle of "three-level seismic fortification goals, two-stage seismic design procedures" introduced by Chinese Seismic Design Code [1], most buildings in regular configuration are calculated and designed at the first seismic design stage, in which only the most probable earthquake (minor earthquake, with return period of 50 years) actions are concerned. While for the main workshop of a large-scale power plant, the seismic problems need to be dealt more carefully and seriously, because this kind of structure is tall and large span, and bears considerable vertical load. This kind of structures is usually in long rectangle plan and the lateral stiffness in the longitudinal frame direction provided by the beam-column frame system (moment-resistant frame) is often insufficient, so that it is difficult to limit the story drift and displacement of the structure within allowable deformation in an economical manner. Enhancing the earthquake resistant strength of the structure is usually by installing steel braces between the columns along the longitudinal direction of the workshop. To the minor and moderate seismic action, such braced frame structure has sufficient stiffness and spare strength. To the intensive earthquake, however, the steel bracing in compression yields and the stiffness and strength degrade severely, as pointed by Higginbotham [2], so that the capability of dissipating energy of the structure is poor. In the recent years, the energy-dissipating dampers have been developed and equipped with steel bracing to improve the seismic behavior of frame structures.

The study presented in this paper makes use of the data of a large-scale power plant, which is designed to construct in the Northwest region of China, and has been proposed to adopt a kind of lead-alloy damper, Zhang [3]. The study employs nonlinear dynamic analysis methods to investigate the seismic performance of the structures with the energy-dissipating dampers in different installation and subjected to the input of minor, moderate, and intensive earthquake actions (with return period of 50, 475, and roughly 1970 years, respectively). The results prove that the energy-dissipating damper installed at the joint of the steel bracings results in high-capacity of dissipating energy with reasonable lateral stiffness of the structure. It indicates the prospective application of the energy-dissipating system in the seismic design for this kind of structures.

The concept of seismic performance based design is encouraged to be used into critical buildings and facilities in most seismic design codes. Important electric power plant being as a key facility in life line system is required in Chinese seismic design code to enhance earthquake resistant defense standard. Installing high performance dampers into structures is an important measure for improving the seismic performance of the structure. The main target of seismic performance design is to make the structure against interrupt of power supply in design fortification earthquake and eventually to protect the structure from serious damage and collapse during extreme earthquake. In order to fit these requirements the option and application of dampers becomes attractive topics. In this paper, the performance based design method is discussed combined with a real thermo-electric power plant, and the effectiveness of lead-alloy damper is verified by time history analyses of earthquake responses of the exemplary structure.

GERNERAL INFORMATION OF THE STUDY CASES

The power-plant project is designed for the area with seismic fortification intensity in 8-degree. The construction site is classified in type II according to the Chinese Code. The structure is 8-story in different story height, total 40.77 meters high. Its longitudinal frame has 6 bays in 12 meters of span, as show in Fig.1. The cross section of the reinforcement concrete columns is 700×600 mm, of the I-shaped steel beam is 300×600 mm. The steel bracing is double channel-steel, size of 2-360×100×13 mm, 2-320×90×10 mm and 2-200×75×9 mm in the first story, the second through the 4th story, and the 5th through the 7th story, respectively. The analysis is conducted in the following four structural models to compare the seismic performances

Case 1: Moment-Resistant Frame (MRF);
Case 2: Ordinary Braced Moment-Resistant Frame (BMRF);
Case 3: Energy-Dissipation Braced Frame (EDBF), as shown in Fig. 1 of the EDBF typical frame;
Case 4: Partial Energy-Dissipation Braced Frame (PEDBF)

The PEDBF has the energy-dissipation bracing installed in the first through the 3rd stories and the ordinary steel bracing for all upper stories.

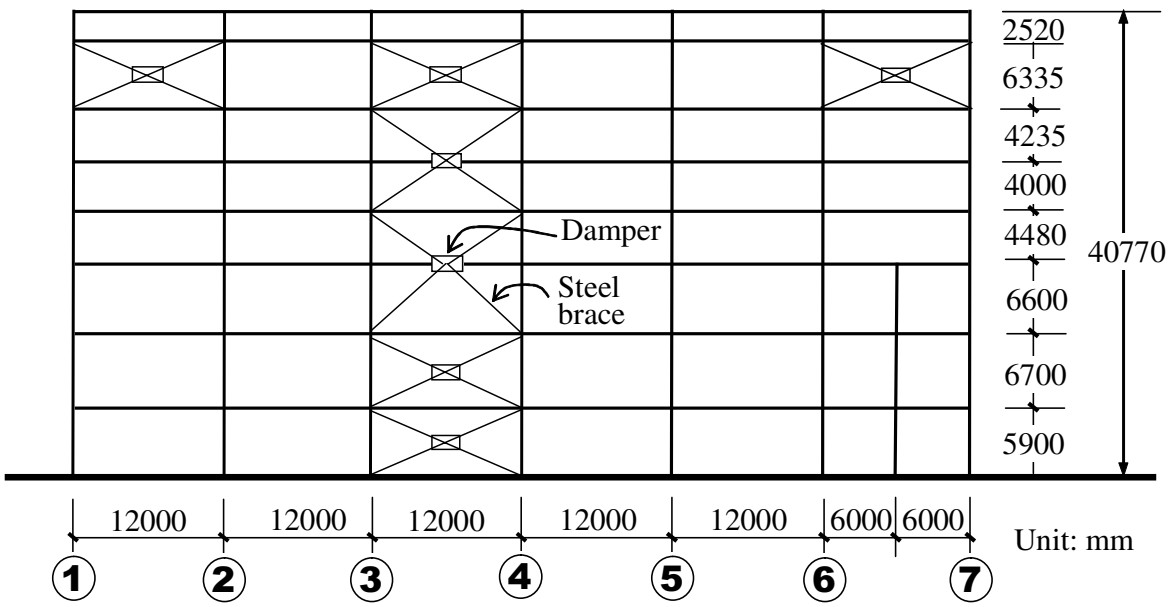

Figure 1 Typical frame model of the EDBF frame in longitudinal direction (the pattern of the steel brace installation are the same of the BMRF, EDBF, PEDBF structure models)

ANLYSIS MODELS AND ASSUMPTIONS

Plane frame structural model is used in analysis, and each beam-column joint is treated as rigid node, having three displacement degrees of freedom (horizontal and vertical translations and in-frame-plane rotation). It is assumed that cast-in-site reinforcement concrete slabs have infinite stiffness in its plane. So the horizontal displacement DOF is reduced to be one at each floor level. The base of the first-story column is fixed to the foundation (ignoring the structure-soil interaction).

The steel beams and RC columns are idealized as line element with inelastic bending deformation. Inelastic shear and axial deformation is considered with vertical column elements. The element-end has rigid zone to approximate the beam-column joint effect. The bracing is treated as tension-compression bar element, so the node of the brace-joint has two translational DOFs only. All elements are treated mass-less and their weights are concentrated at nodes to form the mass matrix.

Beam element is simulated using one-component model, i.e. two elasto-plastic rotational springs at the elastic element ends to represent nonlinear flexural deformation. For the RC column element, which has the interaction between axial load and bending moment, is idealized by multiple-axial-spring model (MS model), Li [4]. It has a line element with two MS elements at the column-end. The line element is elastic in flexural behavior and axial deformation. The MS element consists of a number of uniaxial springs. The number of the spring depends on material properties, size of column cross section, and reinforcing bar arrangement. Each single steel bar is replaced by a steel spring, and the concrete is properly discretized into small concrete portions and represented by concrete springs. The deformation of each spring conforms to the "plane section assumption". The MS element bears moment and axial force but not shear force.

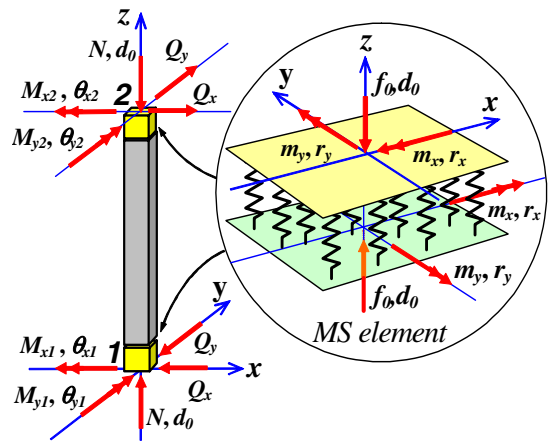

Figure 2 Multiple axial spring model

For the tension-compression steel bracing element, the tension yielding strength f_y is calculated based on the section size and the material strength, while the compression strength is approximately evaluated as one-fifth of the tension strength ($f_y^{'} = 0.2 f_y$) in consideration of buckling. As the results, the tension strength is 3200kN, 2550 kN, 1540 kN for the steel braces in the first story, the second through the 4th story, and the 5th to 7th story, respectively. The energy-dissipation damping device (lead alloy damper) is represented by an inelastic shear spring interacting between the upper and lower portions of steel bracing (Fig. 3). For shear-force and the displacement relations of the shear spring has a trilinear skeleton curve as shown in Fig. 4.

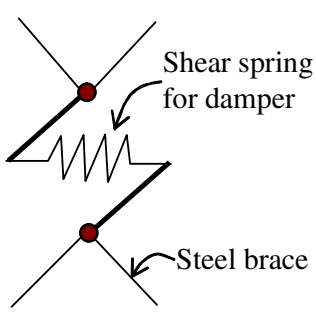

Figure 3 Damper as spring model

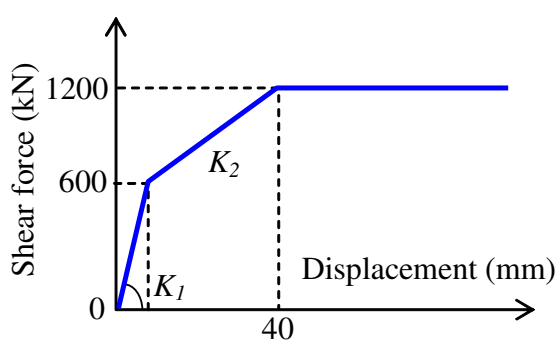

Figure 4 Hysteresis skeleton curve of damper

The seismic response analysis is performed using CANNY program, a 3-dimensional static/dynamic structural analysis computer program, Li [5]. The step-by-step analysis solves the equation of motion using Newmark β-method (β=0.25) in a time interval of 1/100 second. Damping is assumed proportional to the stiffness matrix at 5 % of damping constant.

RESULTS OF NONLINEAR SEISMIC RESPONSE ANALYSIS

The response analysis is carried out to the input of two typical earthquake records, the 1940 El Centro (NS, the turning-point period or characteristic period of the smoothed spectrum is 0.56 second) and 1952 Taft (NS, the characteristic period is 0.44 second) records, and of one artificial acceleration wave. The artificial wave is synthesized according to the site parameter of the building (seismic fortification intensity 8 and site classification II). The input peak accelerations of the earthquake records are scaled in 70 GAL, 200 GAL and 400 GAL, corresponding to the minor, moderate and intensive earthquakes for the area of seismic fortification intensity 8, respectively. The fundamental period of the four structure models are found in the Table 1.

Table 1 Fundamental period (sec) of the four structural models

MRF	1.81	BMRF	0.81	EDBF	0.90	PEDBF	0.87

The natural period of MRF structure is much longer than the other three structures. It demonstrates that both the steel bracing with and without the energy-dissipation dampers increase the lateral stiffness efficiently. It is reasonable that the bracing with damper (EDBF and PEDBF structures) resulted in slightly lower stiffness than that of the BMRF structure.

Results of the Response to the Input Level of Minor Earthquake

The responses of the four structural models are summarized in Table 2. Similar results are found of the BMRF, EDBF and PEDBF structures response to the input corresponding to minor earthquake. The top displacement of the three structural models is reduced in 50% compared with the response of the MRF structure, while the maximum base shear force is about double of that of the MRF structure. The maximum story drift angles of the BMRF, EDBF and PEDBF structure are less than 1/1000. The structure remains in the initial elastic state. In the EDBF structure, damping devices in lower stories just develop into the stiffness declining state with their internal force slightly more than 600kN, and those in upper stories have the force less than 600kN, so are in the initial elastic state. The results indicate that the energy-dissipating bracing system has the same behavior with the ordinary steel bracing when subjected to minor earthquake actions. However, the MRF structure has more responses and some of the frame elements develop in to cracks.

Results of the Response to the Input Level of Moderate Earthquake

The responses of the four structural models to the input level of moderate earthquake are summarized in Table 3. It still can be found the similar responses of the top displacements and maximum story drift angles of the BMRF, EDBF and PEDBF structures subjected to the input level of moderate earthquake. The ratio of the top displacement of the three structures to that of the MRF structure remains the same with the response to minor earthquake. However, the maximum base shear force of the EDBF and PEDBF structures is about 40% less than that of the BMRF structure, or becomes near to that of the MRF structure. The damping devices in the 1st through the 5th stories of the EDBF structure resist in more than 600kN force, that is, developed fully in to the stiffness declining state with some sliding in the damping devices.

The results show that the energy-dissipating bracing does not only provide resistance to decrease the top displacement and the story drift angle but also plays the role of damping effect to reduce the seismic responses of the structure.

Table 2 Maximum Responses to the Input Level of Minor Earthquake

Input (PA=70Gal)	Structural model	Top displacement (m)	Story drift angle (story number)	Base shear factor (W%)
El Centro record	MRF	0.037	1/714 (2nd story)	3.06
	BMRF	0.025	1/1282 (3rd story)	8.46
	EDBF	0.026	1/1000 (2nd story)	7.58
	PEDBF	0.025	1/1052 (2nd story)	7.63
Taft record	MRF	0.052	1/598 (3rd story)	3.85
	BMRF	0.024	1/1123 (3rd story)	8.33
	EDBF	0.025	1/1149 (2nd story)	7.67
	PEDBF	0.024	1/1030 (2nd story)	7.51
Artificial wave	MRF	0.040	1/588 (2nd story)	3.38
	BMRF	0.021	1/1298 (3rd story)	7.32
	EDBF	0.024	1/1294 (2nd story)	6.12
	PEDBF	0.023	1/1298 (2nd story)	6.24

Table 3 Maximum Responses to the Input Level of Moderate Earthquake

Input (PA=200Gal)	Structural model	Top displacement (m)	Story drift angle (story number)	Base shear factor (W%)
El Centro record	MRF	0.108	1/238 (2nd story)	8.54
	BMRF	0.066	1/434 (3rd story)	21.81
	EDBF	0.056	1/417 (2nd story)	10.54
	PEDBF	0.058	1/370 (2nd story)	12.72
Taft record	MRF	0.137	1/213 (3rd story)	10.80
	BMRF	0.068	1/370 (3rd story)	20.25
	EDBF	0.060	1/400 (3rd story)	12.56
	PEDBF	0.062	1/400 (3rd story)	12.61
Artificial wave	MRF	0.114	1/204 (2nd story)	9.64
	BMRF	0.049	1/526 (3rd story)	18.22
	EDBF	0.057	1/500 (2nd story)	11.62
	PEDBF	0.053	1/476 (2nd story)	11.79

Results of the Responses to the Input Level of Rare Intensive Earthquake

In the responses to the input level of intensive earthquake, the maximum top displacement and the ratio to the structural total height are shown in Table 4 in the average of the responses to the three input waves.

The results of the BMRF, EDBF and PEDBF structures are very close to each other, and are about 45% than that of the MRF structure.

The maximum responses of story drift angle are shown in Fig 5. Except the MRF structure, which has much larger responses than others, the responses of the BMRF, EDBF and PEDBF structures are close to each other. The distribution pattern of the maximum inter-story displacement is quite similar among all four structural models, with the maximum inter-story drift angle in the 2nd or 3rd story. It attributed to the different story height (the greater height, the less lateral stiffness).

Fig. 6 shows the maximum responses of story shear force. The response of the BMRF structure is greatest, while the others are very close to each other. Counting the average responses of the story shear force to all the three input waves, the result of the MRF structure is about 52%, and the results of the EDBF and PEDBF structure are about 63% to that of the BMRF structure, respectively.

The damage of flexural yielding subjected to the input level of rare-intensive earthquake is shown in Fig. 7, of the results from the response to the EI Centro input. Counting the number of yielding hinges in the frame elements, it is less damage of the BMRF, EDBF and PEDBF structures than that of the MRF structure, and further reduced damage in the EDBF and PEDBF structures compared with that in the BMRF structure. The damage pattern from the responses to the input of Taft wave and artificial wave are similar with that to the EI Centro input.

To compare the steel brace stress of the structure with and without damper, Table 5 lists the maximum tension stress of steel braces in the structure model BMRF and EDBF response to the input level of rare intensive earthquake. The list in the table is in the ratio of the maximum stress to the yielding strength. The brace yielding strength is given in the parentheses in the column of story number. The ratio equal to or greater than 1.0 means yielding occurred. The results show that the structural model BMRF has almost all braces yielded or near yielding except those in the top story. While in the structural model EDBF, all the steel braces from base to top have the tension stress within 15 ~ 30 % of the yielding strength. This is because the strength of the energy-dissipating damper is designed at 1200kN (see Fig. 4). Obviously, the tension stress has caused the damper yielding and large sliding. That is, the stress developed in the steel brace depends on the damper capacity. In other word, the design can control the stress in steel brace to prevent both the steel bracing system and the frame from damage. In the case of the structure equipped with the energy-dissipating damper encounters rare intensive earthquake shock, the damage could be limited in the damper devices. However, this kind of lead-alloy damper installing in the steel brace is easier to repair end replace, so it makes the total reparation cost of the structure more economic. Though the lower stress in steel brace, it may not be suggested to reduce the steel brace size in design, because it may weak the frame lateral stiffness too much and less the effectiveness of the damper. To conclude on this point may need further investigation.

Table 4 Average Displacement Responses to the Input Level of Intensive Earthquake (PA = 400 Gal)

Structural model	Top displacement (m)	Ratio of the top displacement to the structure height, H
MRF	0.229	H/178
BMRF	0.101	H/404
EDBF	0.106	H/385
PEDBF	0.109	H/374

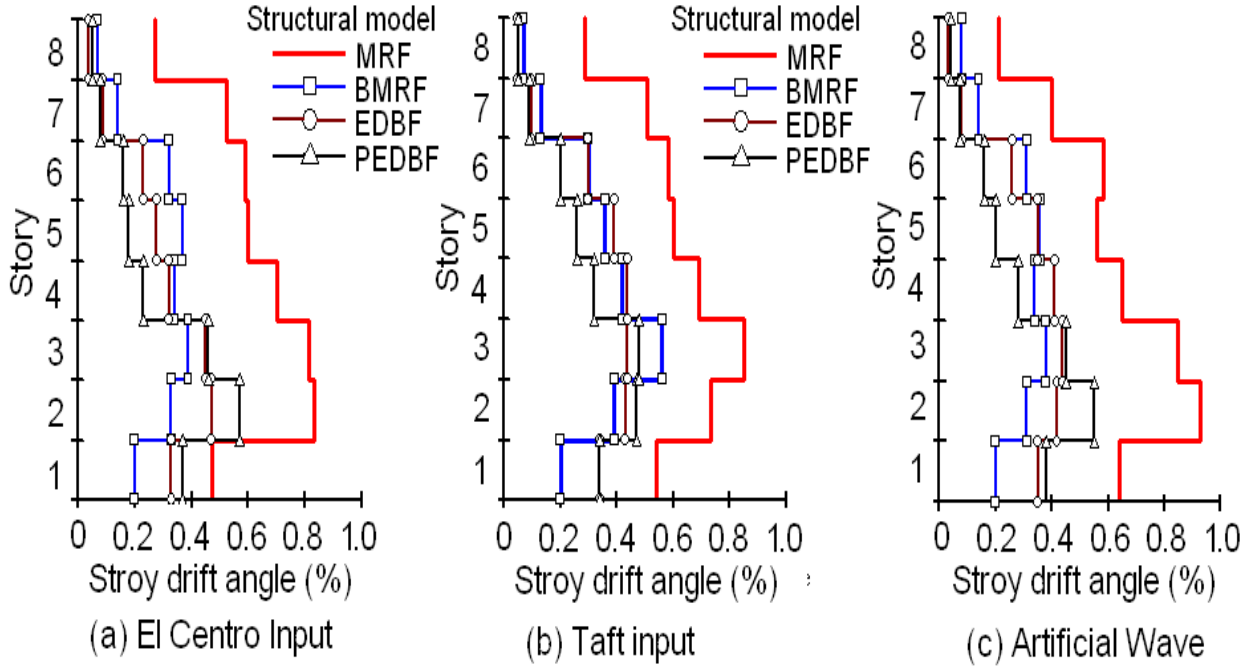

Figure 5. Maximum story drift angle (response to the input level of rare-intensive earthquake, PA = 400 Gal)

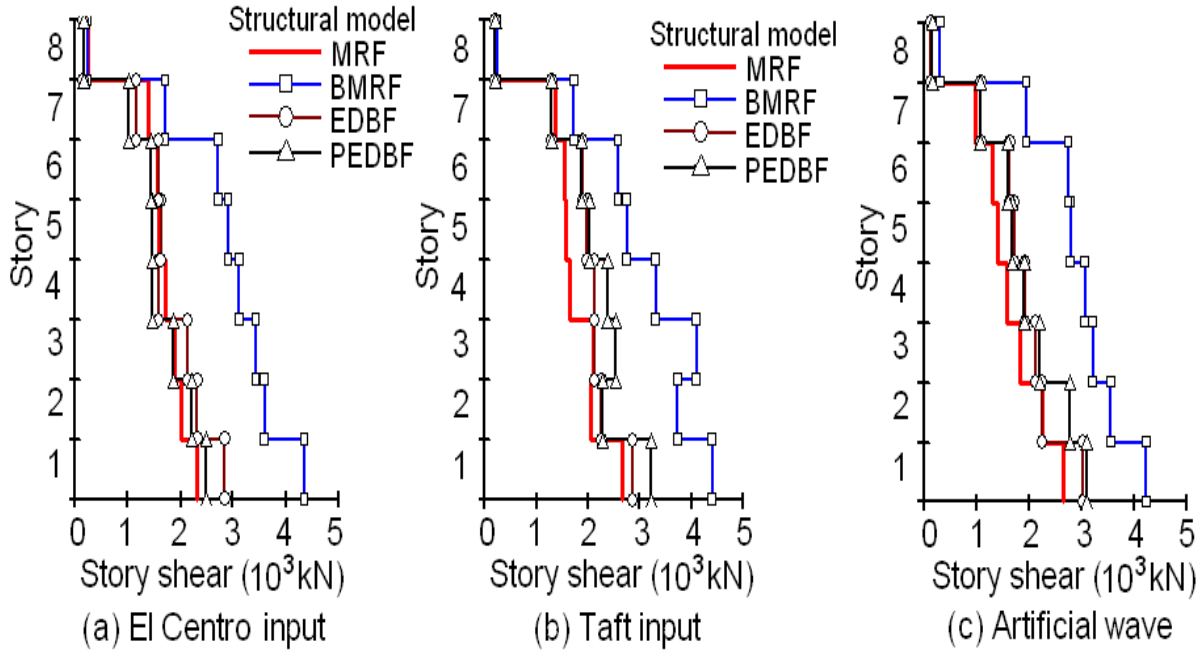

Figure 6. Maximum story shear force (response to the input level of rare-intensive earthquake, PA = 400 Gal)

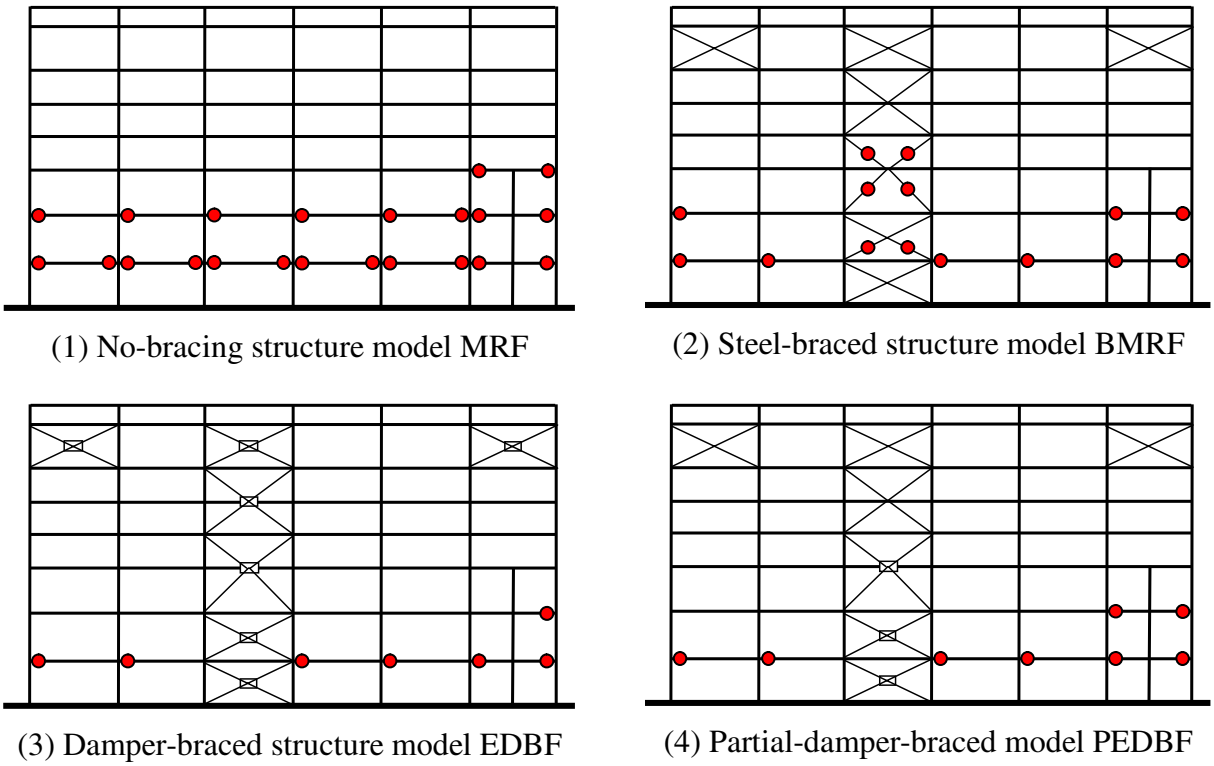

Figure 7 (Cont'd) Damage distribution of the frame flexural yielding and brace tension yielding (response to El Centro input, rare-intensive earthquake, PA=400 Gal)

Table 5 Maximum Tension Force Rate to Yielding Strength in Steel Brace of the BMRF and EDBF Structures Response to Rare Intensive Earthquake (PA = 400 Gal)

Structure model	Ordinary steel-braced BMRF			Damper steel-braced EDBF		
Input wave	El Centro	Taft	Artificial	El Centro	Taft	Artificial
7th story (1540)	0.4117	0.4487	0.4481	0.2065	0.2065	0.1760
6th story (1540)	0.9818	0.8409	0.8740	0.2643	0.2760	0.2513
5th story (1540)	0.9825	1.0045*	0.9532	0.2818	0.2942	0.2734
4th story (2550)	1.0275*	1.0561*	1.0259*	0.1929	0.2153	0.2106
3rd story (2550)	1.0173*	1.0447*	0.9780	0.2855	0.2871	0.2706
2nd story (2550)	1.0157*	1.0224*	1.0161*	0.2380	0.2290	0.2655
1st story (3200)	0.8503	0.8338	0.7869	0.1575	0.1597	0.1609

*steel brace tension yielded (brace tension strength in kN shown in parentheses after story number).

CONCLUDING REMARKS

The concept of seismic performance design is a powerful approach to improve the structural performance at lowest cost. A large-scale thermal power-plant is used as example in the study to verify the effectiveness of installing energy-dissipating bracing system to the frame structure. Analytical evaluation is carried out on the seismic responses of different structural models to various input levels of earthquake excitation. Comparing the structural displacement responses, it is found that adding steel bracing to the frame is effective to enhance the frame lateral stiffness and reduce the displacement response. Installing damper at the brace joints does not weaken too much the stiffness.

Adding ordinary steel bracing enhances the lateral stiffness of the frame structure, at same time it makes the structure bear more earthquake action or causes more stress in the frame elements. In contrast, installing the energy-dissipation bracing system can add the frame stiffness reasonably and meanwhile it reduces the seismic responses of the structure to satisfactory seismic behavior. Through the design of the damper devices, the seismic behavior and responses of the structure can be controlled.

The energy-dissipation damping device selected for the project in study is expected to work as normal steel brace when considering the structure responses to lower level input of the so called minor earthquakes and make sure in thus stage the structure remains in elastic state. To the input of moderate and intensive earthquake, it is effective in reducing the structural responses and controlling the plastic deformation induced in structure within allowable range. The numerical analyses indicate that the effect of the device depends on its mechanical properties and its force-displacement developed during the seismic responses. Therefore, it shall be designed carefully according to the seismic capacity and the demand in particular seismic area.

The steel brace tension stress of the structure with the damper installation is significantly reduced compared with the ordinary steel-braced structure. The information is obtained for design that the energy-dissipating damping device prevents the steel brace from yielding and damage as well. However, it needs more careful investigation to determine whether in design it can do to reduce the steel brace size while to maintain the effectiveness of the damper device.

In addition, it is convenient replacing the lead alloy damper, and it is easier to repair the structure equipped with energy-dissipating damping device for minor seismic damage in the main frame.

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