13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1461

TEST STUDY OF ADJACENT BUILDINGS LINKED BY HIGH EFFICIENT DAMPING SYSTEM

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SUMMARY

The seismic responses of two adjacent buildings linked by high efficient damper for multi-structure system (HEDMS) are investigated in this paper based on the shaking table tests. Experimental and analytical results confirm that the HEDMS could effectively reduce the seismic responses of sub-structures, and the investigation points out some problems on which emphases should be placed when the HEDMS are used.

INTRODUCTION

Adjacent buildings widely exist in most metropolises, which mainly include two conditions. First, due to the high cost of the land in some areas, the separation distance of some neighboring buildings are very small. Secondly, to meet certain functional requirements, some buildings are designed as main and appendix configuration. These adjacent buildings are susceptible to pounding damage during a strong earthquake, which were observed in the Mexico City earthquake in 1985 [1] and in Loma Prieta earthquake in 1989 [2]. Some researches on this problem have been done [3]-[6]. General countermeasures are to keep the separation distance enough between buildings or to link these buildings with rigid components. The two means both have shortage. For the former, a secure and economical separation distance is very difficult to decide. For the latter, the rigid jointers will remarkable change the dynamic characteristics of buildings, which maybe introduce negative effects during earthquake. A more reasonable way is to install damping equipments between adjacent buildings and utilizing the relative movement between these buildings to drive the damping equipments to dissipate the seismic energy. Some researchers have done lots of work in this field. Kobori. T developed bell-shaped hollow connectors to link very closely buildings in a complex [7], Y.L Xu carried out an investigation on aseismic performance of adjacent buildings connected by viscoelastic dampers [8]. G.Q Li advised to employ a kind of vibration absorbers to restrain the vibration of adjacent frames [9].

A high efficient damper for multi-structures system (HEDMS) [10] is investigated in this paper. The nonlinear numerical analyses on adjacent structures linked by HEMDS have confirmed that this device has excellent effect on reducing the seismic response of structures [11]. To research father more, Shaking table tests on an adjacent structural models connected by HEDMS were conducted in Beijing key lab of earthquake engineering and structural retrofit. The experiment proved the HEDMS's great capabilities as energy dissipaters.

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DAMPING DEVICES AND EXPERIMENTAL MODELS

The HEDMS comprised of a lever and a damper (Fig.1). The damper might employ triangular plate damper or friction damper, and so on. In this experiment, triangular plate damper (Fig.2) is adopted. It's obviously that HEDMS would dissipate more energy than the same triangular plate damper when the relative displacements between the adjacent structures are equal. So, the HEDMS can add more damping to buildings. The restoring force model of the HEDMS is the same as the damper equipped in that, but the hysteretic parameters are different, which can expediently be changed by altering the lever's multiple.

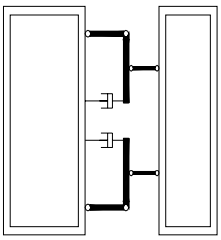

Fig.1 the HEDMS

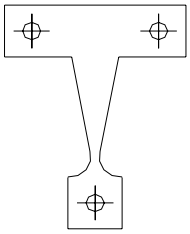

Fig.2. Triangular Plate Damper

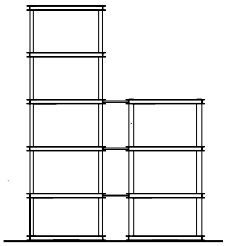

Fig.3. Adjacent Buildings

The structural models consist of two 1/3-scale steel frame structure (Fig.3). The taller one is five storeys, and labeled as "sub-structure 1". The other is three storeys, and labeled as "sub-structure 2". The two frames both are 1.196m length, 1.096m width, 1.1m high, and 155kg additional mass per storey. An acceleration recorder is fixed in every storey of both sub-structures (included the shaking table), a strain gauge placed on one of columns of every storey and embedded in every damper.

Table.1 listed the analytical and experimental results of the periods of structural modes. It could be seen that, if considering the base joints as half-rigid of moment other than rigid; the analysis is more closed to the experimental results. The reason is the base joints, which fixed the structural models to the shaking table, maybe relax more or less during vibrating. It should be placed emphasis that, to attain more accurate analytical results, the slack of the base joints of the system should be taken into account.

Table 1. The Mode Period

	Sub-structure 1					Sub-structure 2		
mode	1	2	3	4	5	1	2	3
experimental(s)	0.3175	0.1024	0.0634	0.0488	0.0433	0.1472	0.0504	0.0350
Rigid base(s)	0.2929	0.0990	0.0614	0.0467	0.0403	0.1275	0.0433	0.0285
Error	7.75%	3.32%	3.15%	4.30%	6.93%	13.38%	14.09%	18.57%
Half-rigid(s)	0.3204	0.1063	0.0643	0.0478	0.0405	0.1439	0.0469	0.0293
error	-0.91%	-3.81%	-1.42%	2.05%	6.47%	2.24%	6.94%	16.29%

In order to decide the parameters and the layout of the damping devices, Sap2000 is adopted to perform a series nonlinear time history analyses. The earthquake record of El Centro NS is chosen as the input motion. The PGA (peak ground acceleration) is adjusted to 0.30g ("g" means gravity acceleration), and the hysteretic performance of triangular plate damper is simulated with the bouc-wen model. The HEDMS's optimum parameters are picked out from 50 group of parameters which included 10 triangular plate dampers and 5 multiples of the lever.

There are seven choices to install damping devices between two sub-structures: in all three storeys, in two storeys, or only in one storey. The analytical results show that the effect is very desired when all three

storeys are linked (Fig.4). But the closely effect also can be achieved when damping devices are only installed in the third storey (Fig.5). So, these two layouts are studied in this experiment. To be easier for relation, the “layout 1” would refer to the former, the “layout 2” refer to the other.

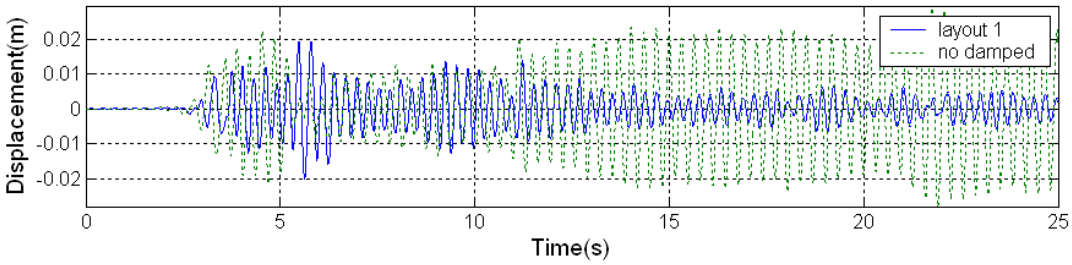

Fig. 4. Analytical Roof’s Relative Displacement Time Histories of Sub-Structure 1 Under Layout 1

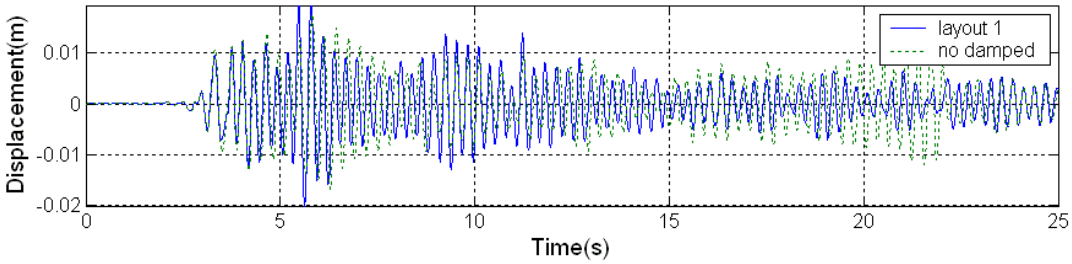

Fig. 5. Analytical Roof’s Displacement Time Histories of Sub-Structure 1 under Two Layouts

4 different earthquake inputs are adopted, which are EL Centro NS, Taft SE, Fujialou SE and an artificial signal. Each signal is adjusted to three different magnitude grades ranked by the PGA, which are 0.07g, 0.20g and 0.30g. The layout 2 only inputted records of PGA=0.30g.

EXPERIMENTAL RESULTS

In the condition of layout 1, the four earthquake records of three magnitude grades are input one by one. The dynamic responses of structural system all have obviously decreased. This paper only gives the experimental observation of EL Centro NS. The acceleration time histories of the roof of sub-structure 1 are showed in Fig.6, Fig.7 and Fig.8. It could be seen that, before being linked, the acceleration amplitude is considerably high all the time during vibration. And it decreases at very slow speed after vibrations of the shaking table ceases after about 50 second. After being linked with damping devices, although the amplitude has no obviously change in the beginning, it falls to a very low level rapidly, especially for the input motions PGA=0.20g and 0.30g. These indicate the structural self damping is too low to suppress vibrations. The damping devices provide plenty of additive damping for the system, which restricts the vibration efficiently.

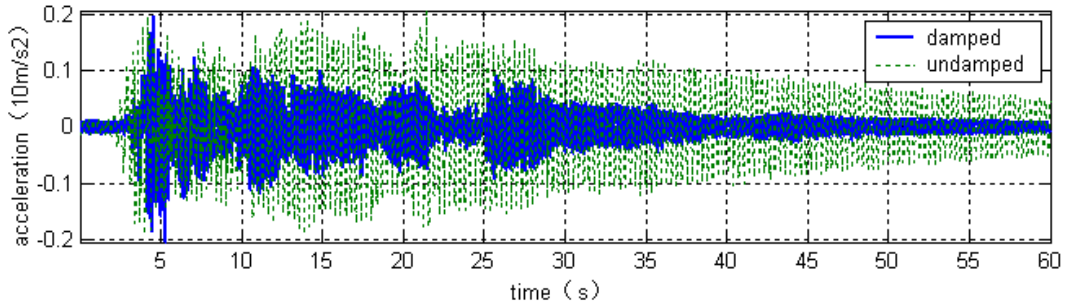

Fig. 6. PGA=0.07g Roof’s Relative Acceleration Time Histories of Sub-Structure 1

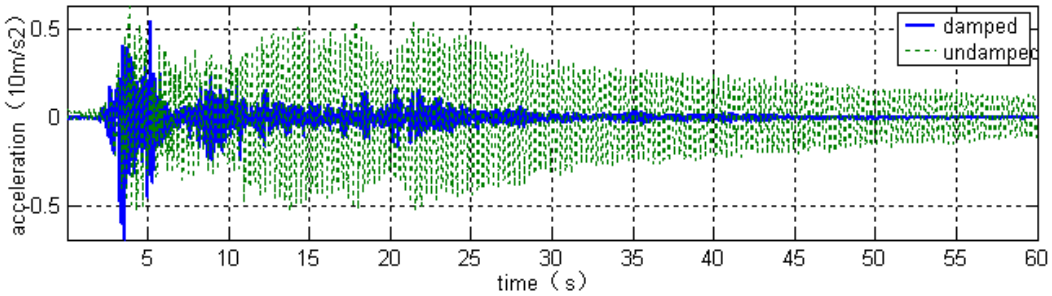

Fig. 7. PGA=0.20g Roof's Acceleration Time Histories of Sub-Structure 1

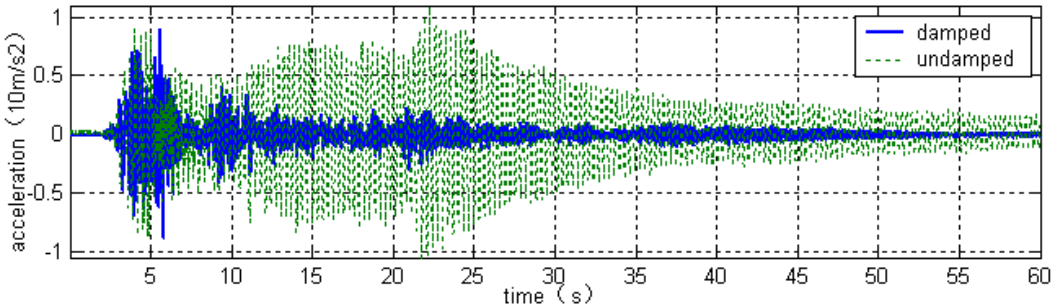

Fig. 8. PGA=0.30g Roof's Acceleration Time Histories of Sub-Structure 1

The peak strain of bottom columns of sub-structure 1 is showed in Fig.9, Fig.10 and Fig.11, which has obviously decrease when structure is with heavy damping. And the amplitude has kept a low level during all the time history. The peak strains of the columns are listed in table 2, which indicates that all the columns remained elastic. Table 2 also shows that the peak deformations of most columns are reduced except for the third level of sub-structure 2 under PGA=0.07g.

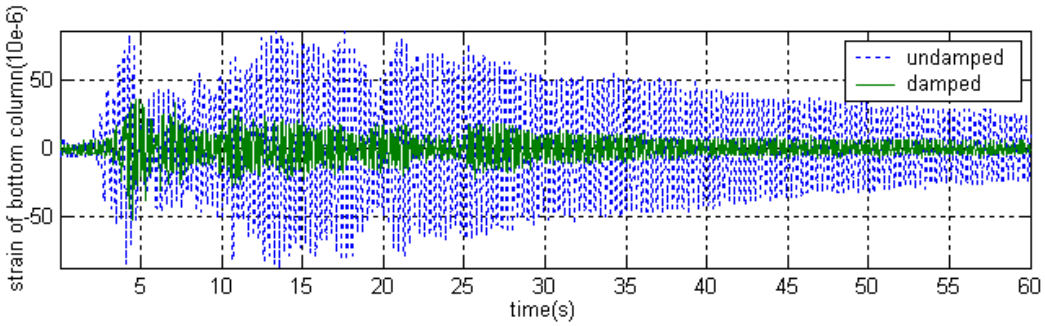

Fig.9. PGA=0.07g Bottom Column's Strain Time Histories of Sub-Structure 1

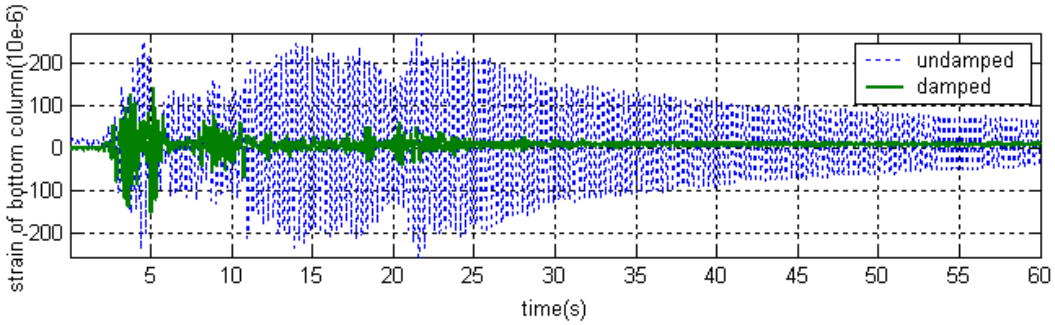

Fig.10. PGA=0.20g Bottom Column's Strain Time Histories of Sub-Structure 1

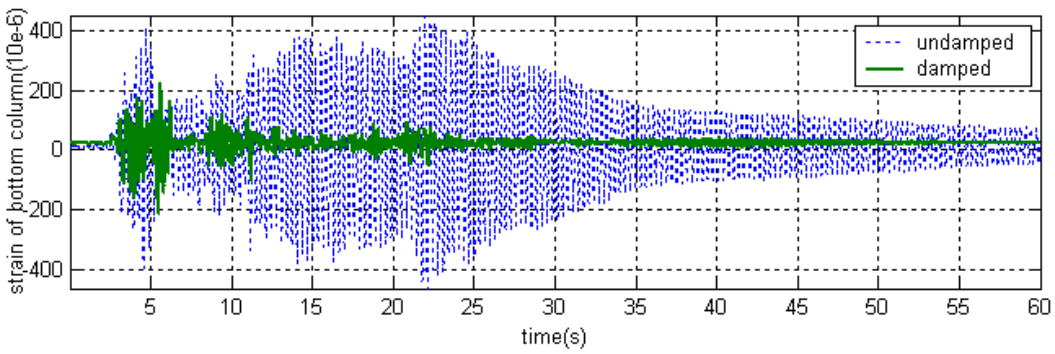

Fig.11. PGA=0.30g Bottom Column's Strain Time Histories of Sub-Structure 1

Table 2. Peak Strains of Columns

El Centro NS		Sub-structure 1（10^{-6}）					Sub-structure 2 (10^{-6})		
PGA		Storey 1	Storey 2	Storey 3	Storey 4	Storey 5	Storey 1	Storey 2	Storey 3
	undamped	87.02	97.4	71.33	48.77	56.71	54.18	28	28.72
0.07g	damped	52.76	55.08	46.15	39.1	18.92	39.8	24.43	29.29
	effect	39.4%	43.4%	35.3%	19.8%	66.6%	26.5%	12.8%	-2.0%
	undamped	291.50	316.06	241.29	164.28	85.81	106.74	62.90	63.70
0.20g	damped	154.5	166.1	131.8	142.0	52.9	82.0	49.3	41.2
	effect	47.0%	47.5%	45.4%	13.6%	38.3%	23.2%	21.6%	35.3%
	undamped	465.41	513.75	360.25	265.54	111.6	198.65	108.91	109.65
0.30g	damped	226.75	261.48	189.61	159.33	62.67	125.33	81.09	75.87
	effect	51.3%	49.1%	47.4%	40.0%	43.8%	36.9%	25.5%	30.8%

Table 2 reveals another important phenomenon that the damping devices work more effective as the magnitude of the disturbing force increases. The same conclusion also could be drawn from the Fig.9~Fig.11 (10s~40s). The reason is that the damping devices performed effective only when the triangular plate dampers behave in inelastic fashion. When the magnitude of input motion is very small, the deformations were too little to have the damper dissipate energy because they remain elastic. So the damping devices only work as some link springs. However, when the input motion is severe enough, for example PGA=0.30, the devices' deformation would be big enough to make the mild steel yielded. Then most of dampers could effectively dissipate seismic energy through hysteretic behavior, which provide the structures a significant damping, and improved the aseismic performances of the system greatly.

The data collected from tests and the analytical results both demonstrate this idea. The experimental results of the maximum strain time histories of triangular plate damper are showed in Fig.12 and Fig.13. It could be found out that all the triangular plate damper remain elastic in most time during vibrating when PGA =0.07g, and when PGA=0.30G even the first layer damper has yielded during 3s to 6s and 25s to 26s. Because the strain gauge adhered to second layer triangular plate damper are destroyed during vibrating, only two layer strain time histories are provided herein. The analytical hysteretic loops of the third and first layer damping devices are showed in Fig.14, Fig.15, Fig.16 and Fig.17. When PGA=0.07g both layer damping devices dissipated little energy. When GPA=0.30g, the hysteretic loops of the third damping devices are very plump, and it could be seen that the first layer also dissipates some energy.

Consequently, a conclusion could be derived that, in order to make the damping devices more efficient, optimization designs of the parameters of the devices should be performed so that all the devices behave in inelastic fashion in majority of vibrating duration when the earthquake of predictive magnitude is occurred.

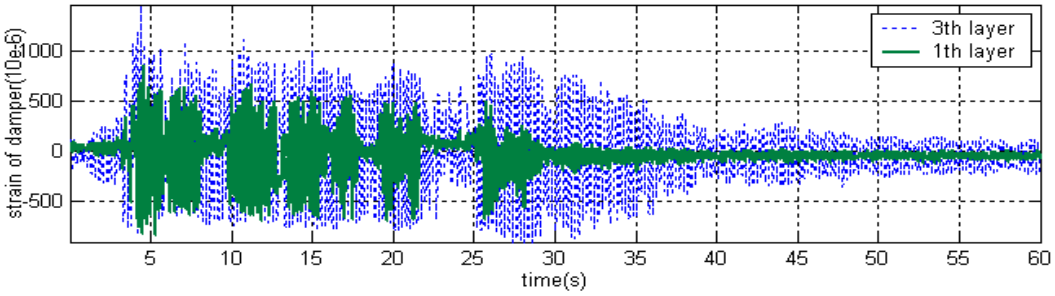

Fig.12. Triangular Plate Damper's Maximum Strain Time Histories PGA=0.07g

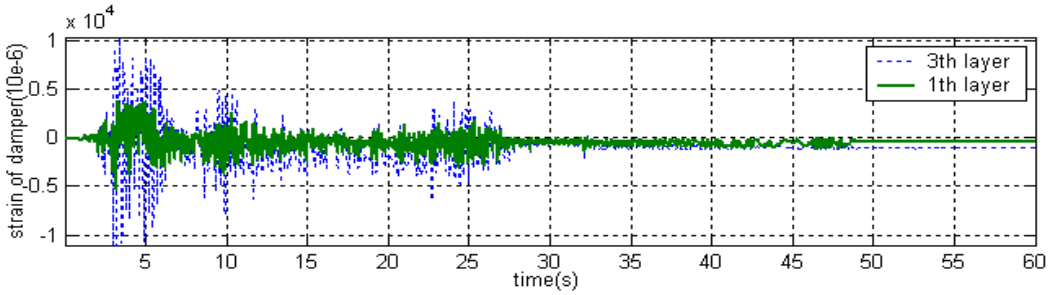

Fig.13. Triangular Plate Damper's Maximum Strain Time Histories PGA=0.30g

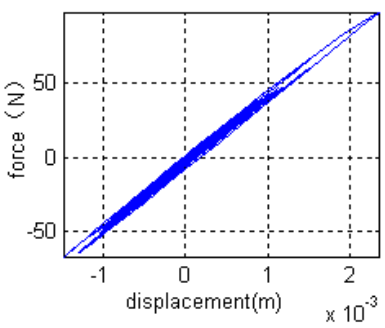

Fig. 14. Hysteresis Loops Of the Third Layer Devices PGA=0.07g

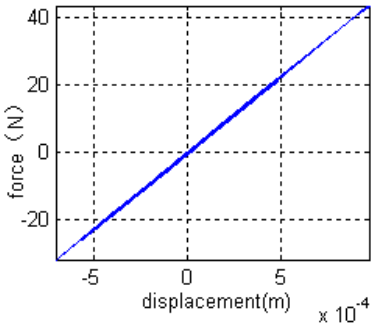

Fig.15. Hysteresis Loops Of the First Layer Devices PGA=0.07g

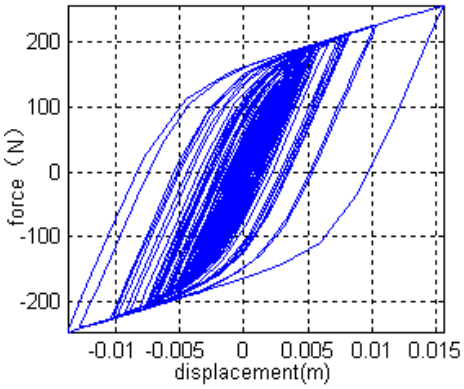

Fig.16. Hysteretic Loops of the Third Layer Devices PGA=0.30g

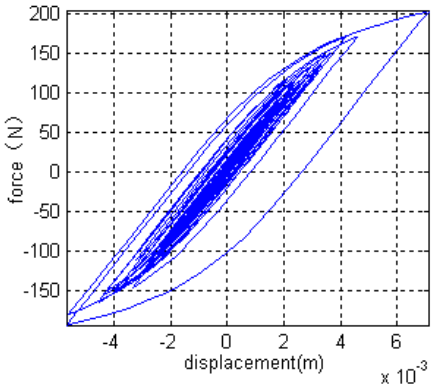

Fig.17. Hysteretic Loops of the First Layer Devices PGA=0.30g

In the condition of layout 2, the response characteristics of the sub-structures are similar to layout 1. Table 3 indicates that, as prediction as analytical results, the two layouts achieves almost the same effect on reducing the amplitude of columns' strain. In order to testify that further, the acceleration and columns' strain time histories of two layouts are compared. The results show that the two layouts are very closely as displayed in Fig.18 and Fig.19. The reason would be presented in the following parts of this article.

Table 3. Peak Strain of Columns

El Centro PGA=0.30g	Sub-structure 1（10^{-6}）					Sub-structure 2(10^{-6})		
	Storey 1	Storey 2	Storey 3	Storey 4	Storey 5	Storey 1	Storey 2	Storey 3
Undamped	465.41	513.75	360.25	265.54	111.6	198.65	108.91	109.65
Layout 1	226.75	261.48	189.61	159.33	62.67	125.33	81.09	75.87
Effect	51.3%	49.1%	47.4%	40.0%	43.8%	36.9%	25.5%	30.8%
Layout 2	215.19	226.79	212.72	194.01	70.25	128.22	83.98	75.87
Effect	53.8%	55.9%	41.0%	26.9%	37.1%	35.5%	22.9%	30.8%

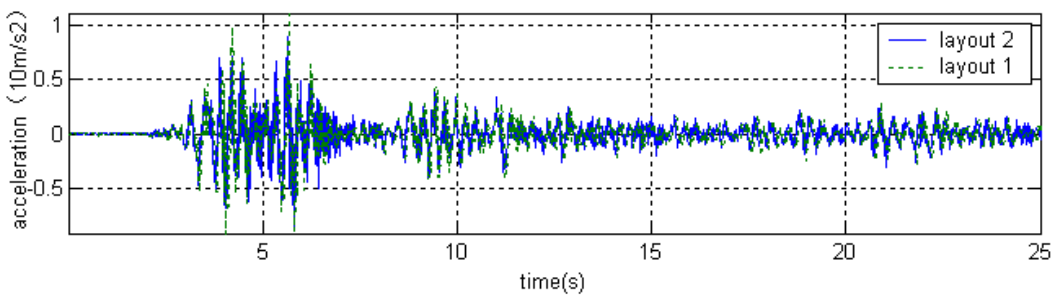

Fig.18. Roof's Acceleration of Sub-Structure 1 Of Two Layouts PGA=0.30g

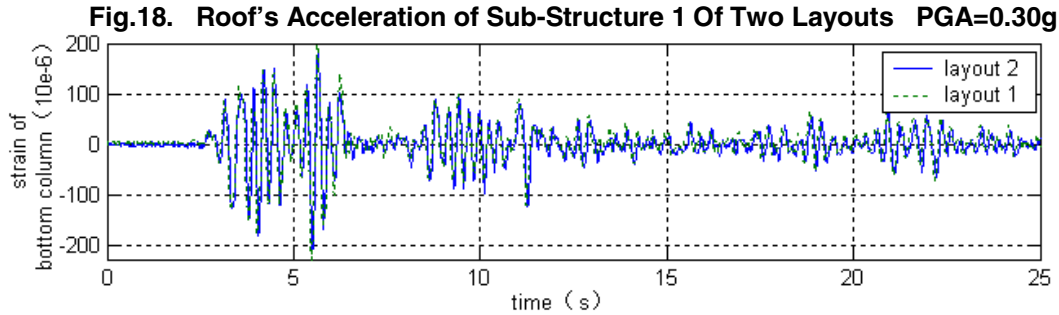

Fig.19. Bottom Column's Strain of Sub-Structure 1 Of Two Layouts PGA=0.30g

In order to find out the reason, the energy dissipation under two layouts is compared, which yields a profound insight to this phenomenon. The energy time histories of two layouts are displayed in Fig.20 and Fig.21. Three indexes are calculated: (1) .the energy dissipating ratio (the ratio of energy dissipated by damping devices to input energy). Though the damping devices of layout 1 dissipates more energy (158.81Nm vs. 151.69Nm), the input energy of layout 1 is also more than layout 2 (175.76Nm vs. 169.59 Nm). The energy dissipating ratios are 90.36% vs. 89.45%, which are very closely. (2). Potential energy. Due to the potential energy represents the deformation of structures; it could present directly evaluation of restraining effect on relative displacement. The result is 57Nm vs. 47Nm. It seems that the layout 2 yields a lower maximum potential. However, taken account into the system under layout 1 incorporates two times more dampers than layout 2, and the dampers would store some potential energy, the sub-structures' maximum potential of two layouts are very close in fact. (3) Energy dissipates by single damping devices. Fig.22 shows the hysteretic loops of the third layer damping devices of layout 2. It could be seen that the device of layout 2 dissipates more energy than layout 1(Fig.16), which indicates that the damping devices of layout 2 worked more efficiently. Consequently, it could be seen that more dampers do not always means better control effect. The spatial layout of dampers is very important. If the relative movement is

too little, it is unessential to install damping device in that storey (such as bottom storey in this experiment).

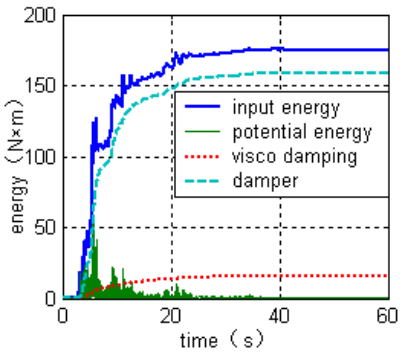

Fig.20. Energy Time Histories of Layout 1

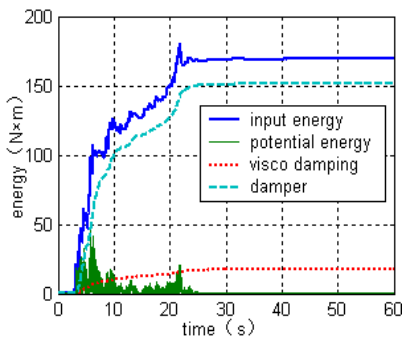

Fig.21. Energy Time Histories of Layout 2

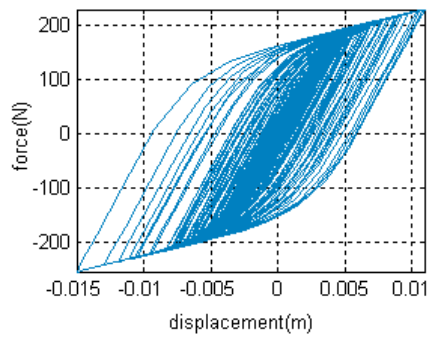

Fig.22. Hysteretic Loops of Third Layer Devices of Layout2

CONCLUSIONS

Some conclusions can be drawn from this experiment.

1). with reasonable design, HEDMS can significant increase the available damping within the structures, which can restrain the seismic response of structure obviously. To have the HEDMS work more efficiently, its parameters should be adjusted to ensure it behave in inelastic fashion during earthquake.

2). Optimization design is required to decide the number and the spatial layout of the HEDMS, because installing only a few HEDMS in certain locations can achieve very well effect.

3). When performing the time history analysis to simulate the shaking table test, it must be taken into account that the base joints of structures are not rigid absolutely because they maybe slack during vibrating. To set the base as rigidity maybe result that the stiffness is on the high side.

ACKNOWLEDGEMENTS

This study is supported by the National Science Foundation under Grant No. 50078003 and Beijing Municipal Education Commission. The supports are gratefully acknowledged.

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