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SHAKING TABLE TEST AND ANALYSIS OF A NEW TYPE OF SHEAR WALL WITH SEISMIC CONTROL DEVICE

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ABSTRACT

A new seismic energy dissipation shear wall structure is proposed in this paper. The new shear wall is one with purposely built-in vertical slits within the wall panel, and various seismic energy dissipation devices are installed in the vertical slits so that the dynamic characteristics of the structure (for instance, lateral stiffness, ductility and fundamental period) can be controlled. In order to verify this concept, shaking table tests of two 10-story shear wall models with the scale of 1/5 were carried out, and the seismic behavior of the two models was studied by analyzing the test data and computing the nonlinear seismic response of the models.

KEYWORDS

High-rise building; Shear wall; Structural control; Nonlinear analysis; Finite element method; Seismic behavior; Time-history analysis; Shaking table test.

INTRODUCTION

Reinforced concrete shear walls are widely used in the design and construction of multi-story buildings in seismic regions. They can be designed to have very good structural efficiency and , since they are more effective in restricting inter-story distortions, can provide much better damage control than frame structures. However, due to their relatively high lateral stiffness, they also tend to attract large amounts of seismic energy, resulting in larger seismic loads to be resisted. Furthermore, after the earthquake attacks, the damages on the shear walls, which mostly occur at the bases, are generally very difficult to repair. To help overcome these problems, the idea of converting the solid shear wall into slit shear walls has recently been proposed. A slit shear wall is a shear wall with purposely built-in vertical slits which divide the shear wall into two narrower wall units, as shown in Fig.1. The shear transfer interfaces between adjacent wall units are generally of three types: (1) reinforced concrete connecting beams, Fig.1 (a); (2) some elastic materials with low elastic modulus and high deformation capacity such as rubber belts, Fig.1 (b); (3) various dampers, such as rubber dampers, spring dampers, etc. Under normal wind loading conditions when the structure remains elastic, the slit shear wall behaves like a solid shear wall. When attacked by a strong earthquake, however, the shear transfer interfaces will first crack and yield leading to significant reduction in their lateral stiffness, and then act as damping devices to dissipate the excessive seismic energy, thereby the premature failure of the wall and the foundation can be protected. It is hoped that by sacrificing the shear transfer interfaces which are relatively easier to repair, the ductility and seismic performance of the overall structure could be improved.

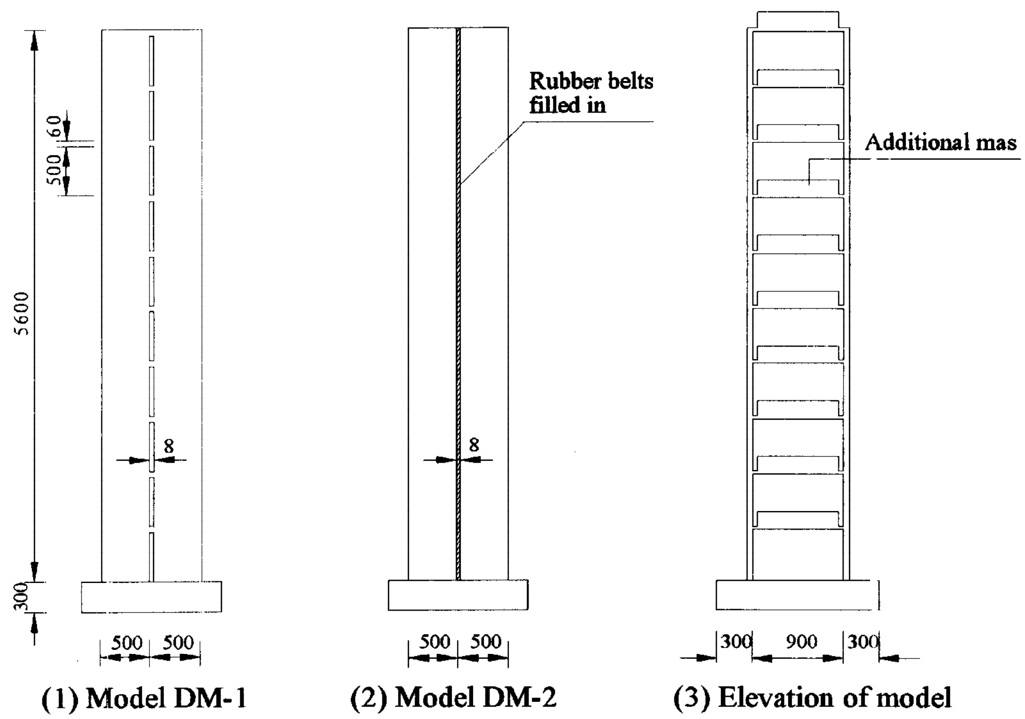

Fig.1. Slit shear walls and various shear transfer interfaces.

The innovative idea of casting vertical slits into infilled wall panels so as to increase their ductility and energy dissipation capacity was introduced by Muto in 1960s. He even put his idea into practice in the design of a 36-story seismic resistant building in Japan. The slit shear wall system proposed herein is not the same as that developed by Muto, albeit there are some similarities in the structural concept. Muto's slit walls are really concrete infilled walls with vertical slits cast purposely which were bounded by and connected to a ductile steel frame, therefore they are not directly applicable to ordinary reinforced concrete shear walls. On the other hand, the Author's slit shear wall system is basically a reinforced concrete shear wall with vertical slits cast along the central axis and is therefore not an infilled frame structure. The idea of casting vertical slits into the walls so that the behavior including cracking and failure of the walls follows certain preferred patterns with the ductility of the wall structures increased is the same as that of Muto. However, the mechanism of dissipating seismic energy is different between the Muto's slit shear wall and the Author's. The mechanism of dissipating seismic energy of the Muto's slit shear wall depends on the shear wall damage more or less, while that of Author's slit shear wall depends on crack and plastic deformation of the connecting element between the adjacent walls. Therefore the Author's object is to prevent the shear wall, the main support component, from damage and collapse.

Except for the difference that the slits are built in purposely, the slit shear walls are also very similar to shear walls constructed by joining precast wall panels at their vertical edges. The influence of the vertical joints on the static behavior of precast shear walls had been studied by Bhatt[1973] who assumed that the shear connections at the vertical joints are continuous and employed what he called an "incomplete interaction theory" for the analysis. Pekau[1981], on the other hand, evaluated the influence of the vertical joints on the earthquake response of precast shear walls by modeling the vertical joints with discrete connector elements and using the finite element method to analyze the structure. However for the past 10 years, very little research work, especially the dynamic tests of the structure, had so far been carried out to study the dynamic behavior (such as crack and failure pattern, seismic energy dissipation mechanism) of this kind of structures. In this paper, based on the shaking table tests of two 10-story shear wall models, the crack pattern, the failure mode and the seismic energy dissipation mechanism of the new shear wall structure were studied, and the numerical models were developed for the nonlinear seismic response analysis.

SHAKING TABLE TESTS

Two reinforced concrete slit shear wall models were constructed and tested. They were both 1/5 scale models of typical 10-story shear wall structure having a height/width ratio of 5.6. The two models were identical to each other except the details of the shear transfer interfaces. The first model (named as DM-1) has reinforced concrete connecting beams with depth of 60 mm as shear transfer interfaces, while the second model (named as DM-2) has rubber belts filled in the vertical slits as shear transfer interfaces. Fig.1 shows the overall dimensions of the models, and Fig.2 the details of reinforcements provided. Each model consists of two pairs of walls which were 40 mm thick and four foundation beams for fixing the model onto the shaking table. Micro-concrete and mild steel wires were used for the construction of the models. The models were cast vertically one story by one story. The compressive strengths of concrete as determined by testing cubes cast of

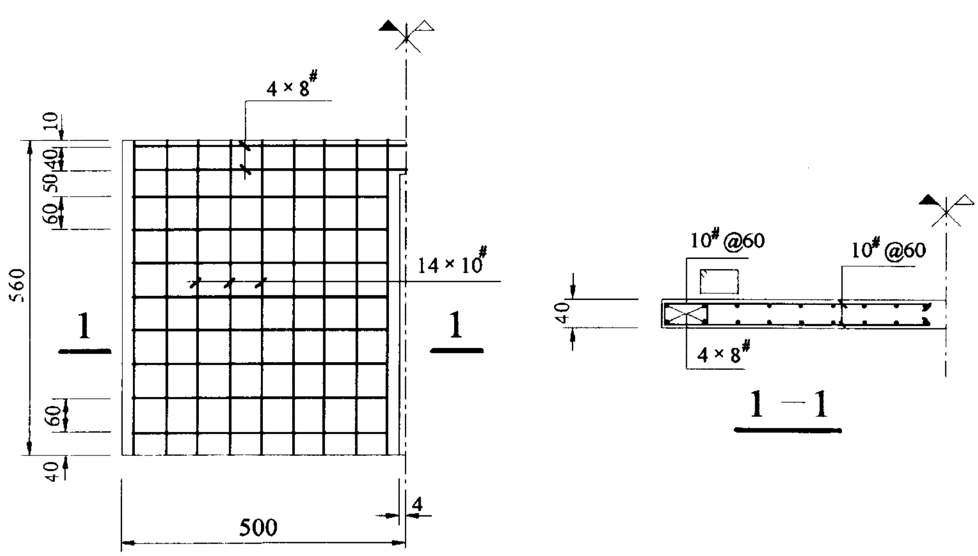

Fig.2. Details of walls at each interstory.

the same batches of materials and cured alongside the models were 11.46 Mpa and 13.00 Mpa for models DM-1 and DM-2 respectively. The mild steel wires (Chinese Standard No.8 with diameter 4 mm and No.10 with diameter 3.48 mm) were used as reinforcement for the walls throughout. The yield strengths of the steel reinforcements were 283 Mpa for No.8 wires and 342 Mpa for No.10 wires respectively. The modulus of elasticity for the steel wires is 189000 Mpa. All vertical rebars in the walls were properly anchored into the foundation beams. The ratio of vertical reinforcements in each wall was approximately 0.92 %.

The design and manufacture of the models basically meet the requirements of similitude theory. Additional masses were applied to each floor to simulate the vertical loads and to reduce the errors caused by the distortion of similitude requirements of gravitational acceleration. The total weight of each model is approximately 150 kN which reached the maximum capacity of the shaking table.

The acceleration record of El-centro N-S (1940) was used as input data of the shaking table, and the time interval and duration of the acceleration data were scaled according to the similitude requirements. The peak value of table acceleration (PTA) was increased by the following sequences: 0.08g, 0.233g, 0.466g, 0.932g, 1.200g and 1.600g which excite the models from elastic state to failure state. The acceleration time histories at each floor level and the displacement at roof level relative to the foundation beam were recorded by the data acquisition system of the shaking table.

TEST RESULTS

The detailed test results can be read elsewhere (Lu et al., 1995). Table 1 lists the major test results which will be used in the following parts of the paper. In Table 1, a_t stands for the actual achieved PTA, a_r the PTA at the roof level, and f the natural frequency of the model.

Table 1. The major test results of shaking table tests

Designed PTA	Model DM-1				Model DM-2			
(g)	a_t (g)	a_r (g)	f (Hz)	deformation	a_t (g)	a_r (g)	f (Hz)	deformation
0.080	0.083	0.291	3.833	no cracks	0.080	---	3.833	no cracks
0.233	0.213	0.547	2.876	no cracks	0.200	0.477	3.000	no cracks
0.466	0.401	0.635	2.500	micro cracks	0.414	0.672	2.500	no cracks
0.932	0.897	1.253	2.166	cracks	0.892	1.330	2.066	micro cracks
1.200	1.280	1.611	1.666	failure	1.280	1.579	1.666	cracks
1.600	---	---	---	---	1.770	2.015	1.167	failure

By observing the test procedure and analyzing the test data, the following primary conclusions were found out. (1) The effect of vertical slits on the initial stiffness of the wall system is very small which is verified by the evidence that the frequencies of the two models are identical. (2) The energy dissipation mechanism of the two models is different. Model DM-1 dissipated the seismic energy by concrete crushing and reinforcement yielding while model DM-2 by the friction of the interfaces between concrete and rubber belts, elastic deformation of the rubber belts and the yielding of reinforcement. Therefore model DM-2 does have good energy dissipation capacity and ductility. (3) Under the same inputs of PTA, the deformation of model DM-2 was smaller than model DM-1 while the cracking and failure condition of model DM-1 was severer than that of model DM-2. Under the same cracking and failure conditions the input PTA that the structure can resist for model DM-2 was larger than model DM-1, which means the seismic resistant capacity of model DM-2 is better than model DM-1.

NUMERICAL ANALYSIS

Based on the structural characteristics of the slitted shear wall models, the plane F.E. approach is utilized for nonlinear seismic response analysis of structure. For finite element representation, an isoparametric quadrilateral element was used. The dynamic equation are integrated by the Wilson-θ method in a step by step way. The analysis model of DM-2 include 960 R.C. finite elements ,180 rubber finite elements and 360 contact elements. The pattern of combination of each kinds of elements around the slit range is shown in Fig.3. For R.C. finite element the composite element model is adopted. In this model the steel is considered to be smeared in the global element area, and the element material is regarded as continuous and homogeneous in one direction. The constitutive model of concrete developed by Darwin and Pecknold was used which was an orthogonal anisotropic model based on equivalent uniaxial strain. The stress-strain curve of concrete under cyclic loading is modeled as Fig.4. The nonlinear behavior of concrete, such as tension stiffening, local contact effect, bond effect between concrete and steel, is taken into account. The steel is assumed to be ideal elasto-plastic. The rubber is modeled as elastic isoparametric element.

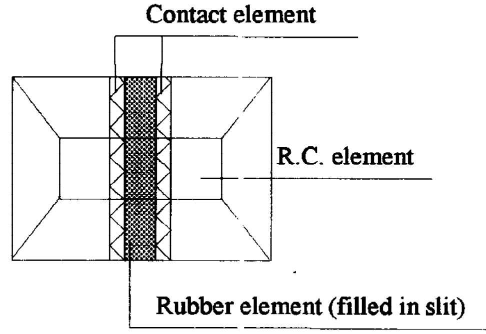

Fig.3. The Combination relationship of each kinds of element around the slit range

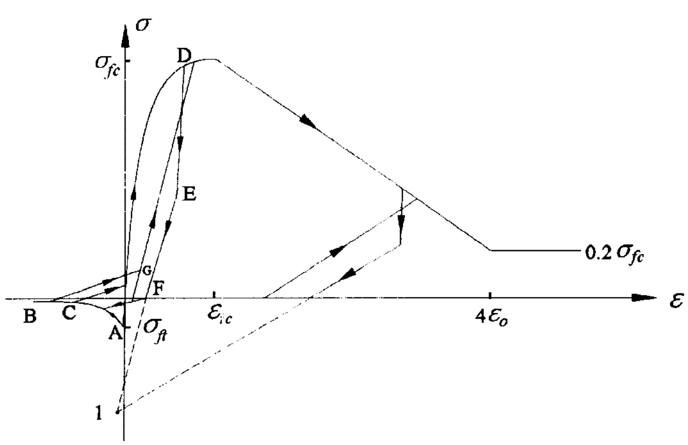

Fig.4. Stress-strain curves for concrete under cyclic loadings.

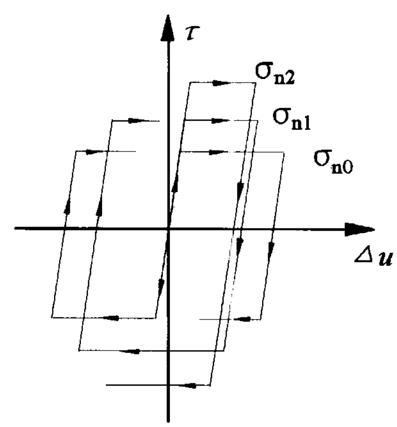

Fig.5. Shear-sliding curves for the interface between concrete and rubber.

In order to reflect the seismic energy dissipation mechanism on the sliding friction interfaces between concrete and rubber, a contact element is derived from the joint element which was conventionally used in the rock mechanics. The contact element is composed of two sliding surfaces, and the interface shear-sliding curve is shown in Fig.5. When the shear stress is larger than the friction stress which obstructs sliding on the interface, the sliding on the interfaces will take place.

By using the FEM, the nonlinear responses of DM-2 under the exciting of 0.233g, 0.466g, 0.932g, 1.2g, 1.6g El-centeo N-S (1940) are studied. The deformation, the procedure of model cracking and the time histories of acceleration and displacement are obtained. Fig.6~Fig.9 are the time histories of acceleration and displacement on the roof of DM-2. The deformation of the upper part of DM-2 is shown in Fig.10. It can be seen that the rubber and concrete slid relatively on the interface, which is the same as that of experiment.

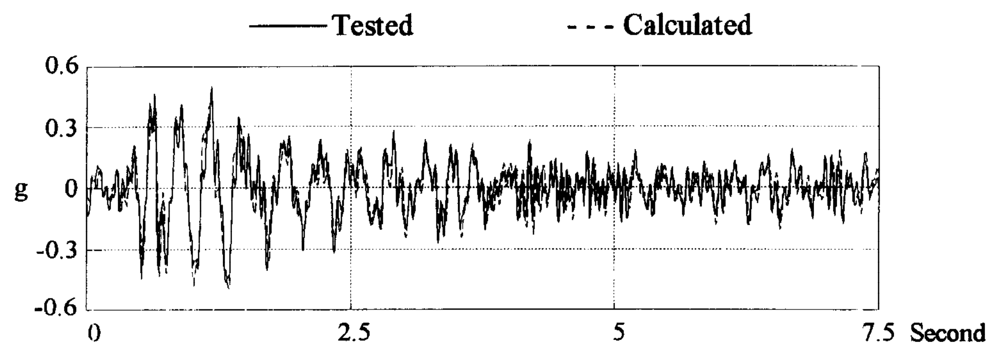

Fig.6. Acceleration time histories of model DM-2 under input PTA of 0.233g

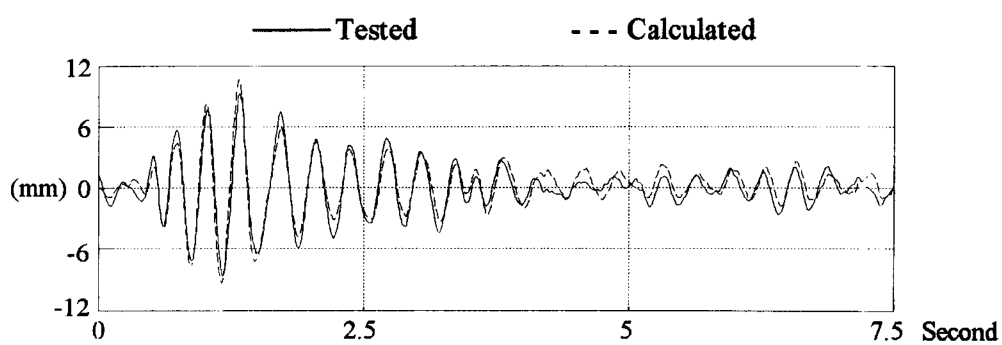

Fig.7 Displacement time histories of model DM-2 under input PTA of 0.233g

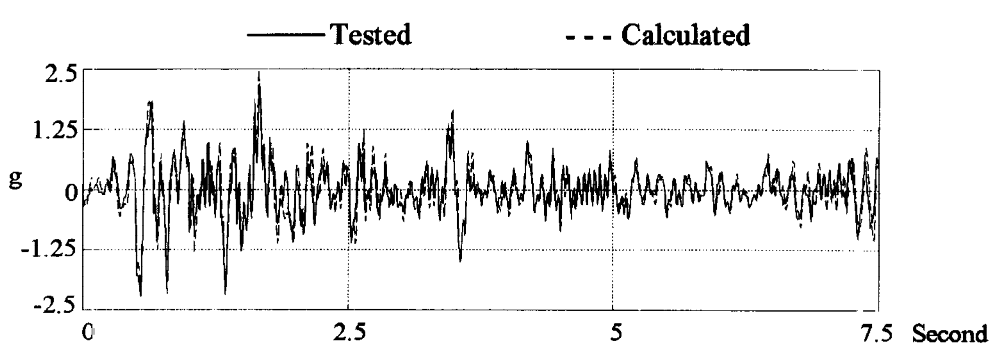

Fig.8. Acceleration time histories of model DM-2 under input PTA of 1.6g

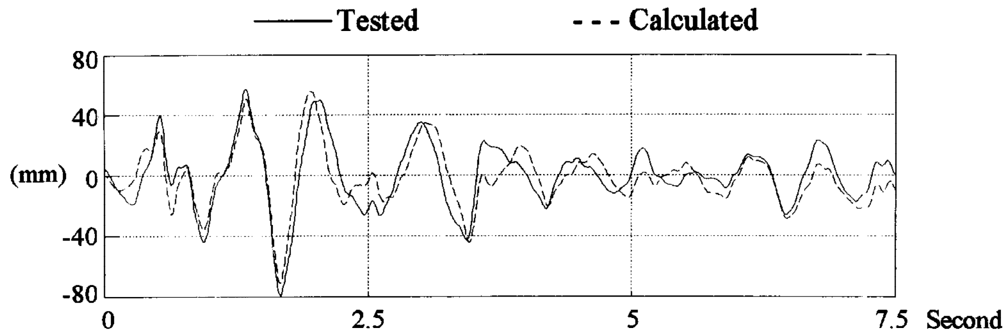

Fig.9 Displacement time histories of model DM-2 under input PTA of 1.6g

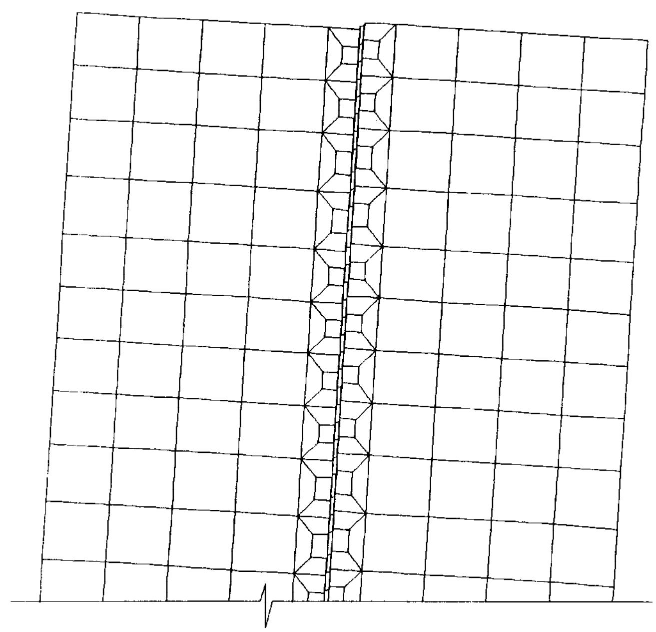

Fig.10 . The deformation of the upper part of DM-2 when the input wave during 1.3 second.

CONCLUSIONS

Based on the shaking table tests and numerical analysis, the following conclusions can be drawn.

(1). The crack and failure pattern of the two models is different: model DM-1 failed due to concrete spalling of the main crack at the bottom of the wall; model DM-2 failed due to the distributed cracks along the height of the wall and the relative deformation between the two wall piers.

(2). The energy dissipation mechanism of the two models is different: model DM-1 dissipated seismic energy by concrete cracking and crushing; model DM-2 dissipated seismic energy by rubber's deformation and friction between concrete and rubber belts.

(3). The good damping behavior of rubber belts can reduce the seismic response of the system, and the good deformation ability of rubber belts filled in the vertical slits can protect the concrete wall from large deformation and crush.

(4). Under the same seismic inputs, the response of model DM-2 is smaller than that of model DM-1. Therefore the seismic resistant capacity of model DM-2 is better than model DM-1.

(5). The dynamic analysis method with nonlinear R.C. finite element proposed in this paper can simulate the nonlinear behavior and cracking procedure of the new shear wall. The numerical results, including acceleration time history, displacement time history, cracking patterns, are in good agreement with those of the shaking table tests.

(6). The contact element can fairly reflect the seismic energy dissipation mechanism on the sliding interfaces between concrete and rubber.

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