Non-linear Time History Response Analysis of Low Masonry Structure with tie-columns

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SUMMARY:
Low masonry structure is widely used in rural area in China because of its convenience and low cost. In order to study the seismic behaviour of low masonry structures, the method of non-linear dynamic time history response analyses of masonry structure model by ABAQUS is studied. The model is two stories, and restrained by four core-tie-columns. Firstly, the constitutive relationship and the calculation method of plastic damage parameters applied to ABAQUS are put forward. The relationship of column and walls, slab and walls in the model are determined by different way. Then, time-history analysis is carried out. The acceleration responses, deformations, and the plastic strains in the masonry walls of the model are analyzed. At the same time, the research results are contrasted on the results from the shaking table test of the same model. The contrasts show that, there is 25% deviation in the story deformations by analyzed and by test. The damage parameter nephogram can simulate the failure mode of masonry structure to some extend.

Keywords: Masonry Structure; Non-linear Time History Response Analysis; ABAQUS

1. INTRODUCTION

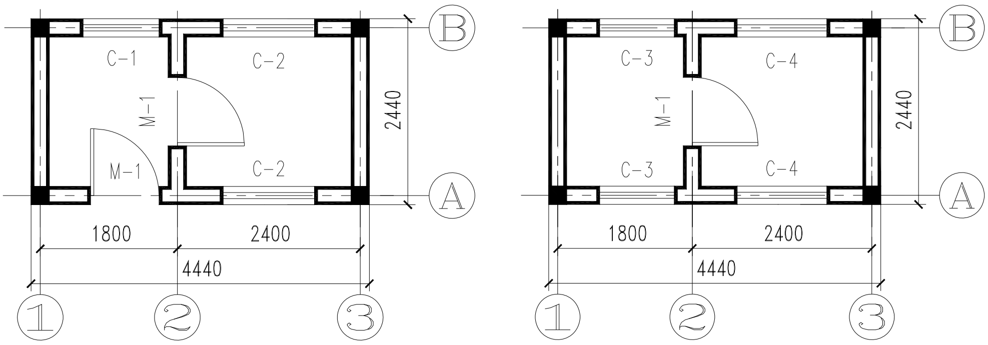
Low masonry structures are the main structural style for buildings in small or medium-sized cities, villages, towns and the countryside in China. The non-linear behaviours of masonry structures under seismic actions are an important issue. In previous studies, Ruifeng et al. (1979), Feng et al. (2000), Quanbiao et al. (2005) and Yingmin et al. (2006) analyzed the non-linear seismic response of masonry walls at element level, Haixu et al. (2009) surveyed the non-linear behaviours of masonry structures based on story model. In recent research, Nina (2011) studied the seismic response of whole structure. To investigate the seismic performance of masonry structures, non-linear history analysis of a masonry model in a shaking table test with core-tie-columns, which was defined by Yingmin (2010), are carried out using the software ABAQUS. The non-linear dynamic time history response analysis of the masonry model are carried out by using the implicit dynamic finite element analysis in the software ABAQUS. The analysis results are compared with those measured from the shaking table test.

2. MODEL

2.1. Model introduction

The two-story analysis model is as same as the masonry structures model with core-tie-columns in the shaking table test, the height of both floors is 2.8m, and the width of all masonry wall in the model is 240mm. The floor in the model is made by some precast hollow concrete slabs, and the thickness of the slab is 120mm. the transversal wall are all solid and the two longitudinal walls are with doors and windows in. The layout of the model is shown in Fig. 1. The masonry walls are built by MU15 fired perforated brick and M1.5 cement mortar. The live uniform loads of the first and second floors are 1.8

KPa and 1.0 KPa, respectively. There are four core-tie-columns at the four corners of the model. And there are reinforced brick ring beams, which's elevation is 5.6m, in the second floor of the model.



(a) First story (b) Second story

Figure 1. Layouts of the model

The floors in the test model are made by precast hollow concrete slabs with the thickness of 120mm. In the analysis model, floors are equivalent to solid plates with the thickness of 80mm. The equivalent elastic modulus of core-tie-column material is calculated according to the core area. The ccompressive and tensional stress and strain relationship curves are made according to the equivalent elastic modulus.

2.2. Model in finite element analysis

The model was built and analyzed by using the software ABAQUS. All the elements such as masonry walls, reinforced concrete floors and core-tie-columns were separation simulated using the three-dimensional reduced integration solid elements named as C3D8R and T3D2. The steels are embedded into the concrete entity. The wall are meshed into two units along the wall's thickness direction and the grid size are 120mm. while the wall are meshed into rectangular shape along the length and height of the wall and the grid sizes are not greater than 240mm. The three-dimension finite analysis model is shown in Fig. 2. All the components, such as the walls, the core-tie-columns and the floors are combined together through assembly type named tie. The final model's weight is 29.27 tons, and the test model's dead-weight is 29.19 tons. Error between is 0.27 per cent.

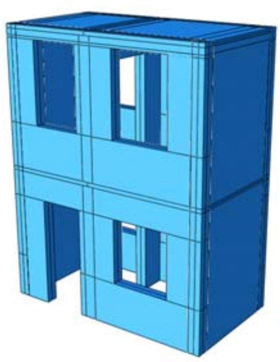


Figure 2. Analysis model

3. PARAMETER OF THE MATERIAL

3.1. Concrete

In order to simulate concrete compressive and tensile damage equation, damaged plasticity material model were used. The single axial compressive stress and strain equation consists with the one given by "code for design of concrete structures GB50010-2010" and the single axial tensile and compressive damage factors can be derived according to the energy equivalent hypothesis. The single axial compressive damage factor is given by Eqn. 3.1.

$$D = 1 - \sqrt{\frac{1}{[\alpha_t (x-1)^{1.7} + x]}} \quad x \geq 1, \quad x = \frac{\varepsilon}{\varepsilon_t}, \quad \alpha_t = 0.312 f_t^2 \tag{3.1}$$

The single axial compressive damage factor is given by Eqn. 3.2.

$$D = 1 - \sqrt{\frac{1}{[\alpha_d (x-1)^2 + x]}} \quad x \geq 1, \quad x = \frac{\varepsilon}{\varepsilon_c}, \quad \alpha_d = 0.157 f_c^{0.785} - 0.905 \tag{3.2}$$

Where D is damage factor, f_t is the average value of axial tensile strength, f_c is the average value of axial compressive strength, and it is taken as 20MPa in the analysis model. ε is the compressive or tensile damaged plasticity strain. ε_c is the compressive strain corresponded f_c, and it is taken as 0.002 in the analysis model. ε_t is the tensile strain corresponded f_t.

3.2. Masonry wall

The material of masonry walls are taken as same as them in the shaking table test, the compressive stress constitution relation in the analysis adopts the one given by Weizhong Yang, which is defined by Eqn.3.3 and shown in fig.3. Specific parameters in the formula are determined by material mechanics test.

$$\frac{\sigma}{\mathrm{f}_m} = \frac{\eta}{1 + (\eta - 1)(\varepsilon / \varepsilon_m)^{\eta/(\eta-1)}} \frac{\varepsilon}{\varepsilon_m} \tag{3.3}$$

Where σ is the compressive stress and ε is the strain of a point on the curve, f_m is the average value of axial compressive strength which is determined according to the masonry material such as MU15 fired perforated brick and M1.5 cement mortar; ε_m is the corresponded strain when the stress is f_m, which is determined by test generally. In this paper ε_m is taken as 0.003. The ultimate stress is 1.6 times of ε_m. η equals to 1.633.

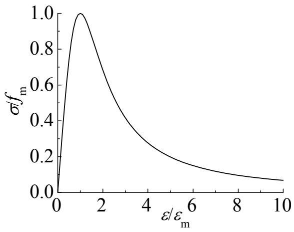


Figure 3. Compressive stress and strain curve

The single axial compressive damage factor of masonry material can be derived according to the energy equivalent hypothesis, which is defined by Eqn. 3.4.

$$D_c = 1 - \sqrt{\frac{1}{0.612 + 0.388 x^{2.58}}}, \quad x = \frac{\varepsilon}{\varepsilon_m}, \quad x \geq 1 \tag{3.4}$$

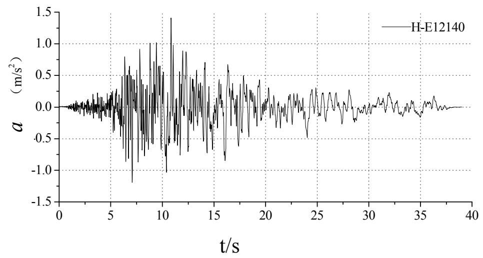
Where D_c is damage factor of masonry wall, ε is the plastic strain of the masonry material, ε_m is the strain corresponding to the stress f_m.

The tensile constitutive relation of masonry material is taken as same as the concrete tensile constitution relation for studies about tensile constitution relation of masonry material are few. The single axial tensile damage equation of masonry material is as same as the concrete's one.

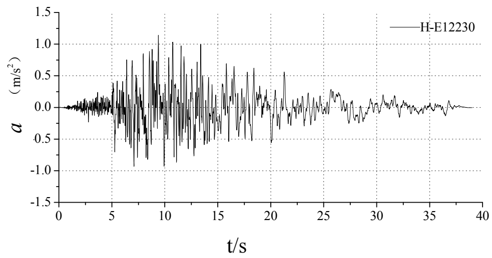
4. NON-LINEAR TIME HISTORY RESPONSE ANALYSIS

4.1. Analysis conditions

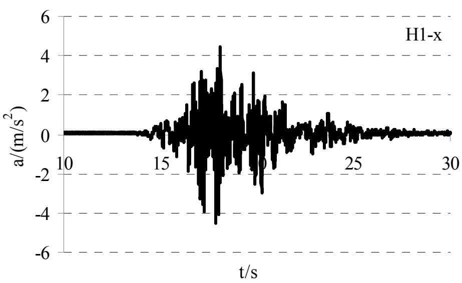
The input waves in dynamical time-history analysis, including natural waves and artificial waves are same as the shaking table test, which are shown in Fig.4. Eight conditions in the shaking table test, which input are mainly by x direction, are taken to non-linear time-history response analysis. The input direction and amplitudes of the waves are listed in Table4.1.



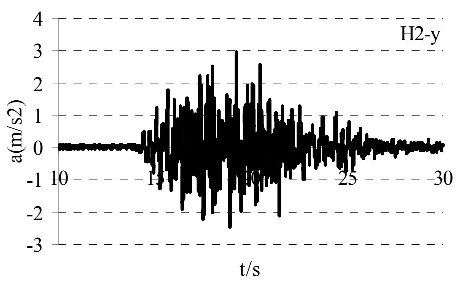
(a) Natural ground motion H-E12410



(b) Natural ground motion H-E12230



(c) Artificial ground motion H1



(d) Artificial ground motion H2

Figure 4. Acceleration waves

Table 4.1. The input direction and amplitudes of the waves

Abbr of the analysis condition	Input wave's types	The amplitude of input acceleration (g)	
		X direction	Y direction
Sx6N	H-E12410	0.051	0
Sx9M	H1	0.053	0
Sx13N	H-E12410	0.92	0
Sx16M	H1	0.088	0
Sx32N	H-E12410	0.129	0
Sx39M	H1	0.177	0
Sxy40M	H1 and H2	0.15	0.135
Sxy42M	H1 and H2	0.235	0.194

4.2 Analysis results

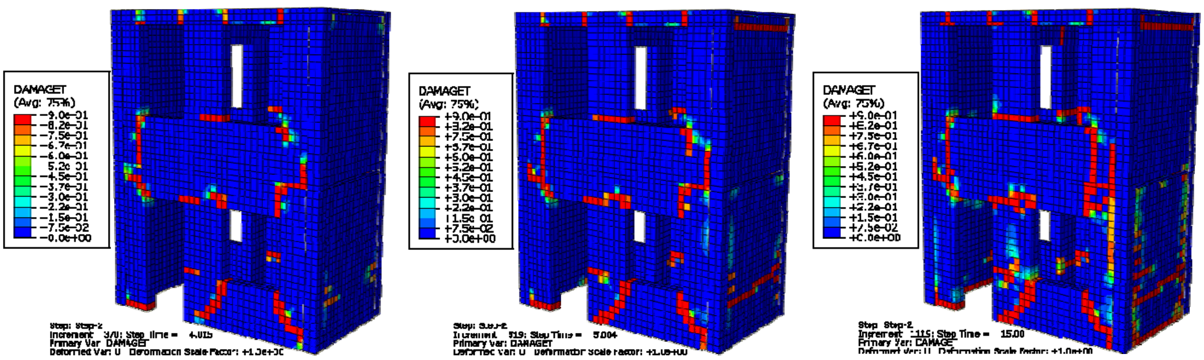
The contrasts of the maximum inner-story displacement in X direction for the first and the second story of the model structure calculated by non-linear time history analysis and the ones recorded in shaking table test are shown in Table 4.2. From table 4.2 we can see that, the first-inner-story displacement of the average difference is no more than 25% and the second-inner-story displacement of the average difference is nearly 37%, which meet the requirements of the normal accuracy by finite element simulation. It can also be seen that, when the input acceleration is relatively small, the inner-story displacement simulation error is relatively small. With the input acceleration increasing, the inner-story displacement error increase to some extent. This may be related to the cumulative damage of the structure. With the increase of the input waves, the damage of the test model increases, and the actual stiffness of the test model become smaller, while the stiffness of the finite element analysis model remains unchanged, so the larger gap between the calculation and experimental results emerges.

Table 4.2. The maximum inner-story displacement contrasts of X direction in the first and the second floor

Abbr. of the analysis condition	D_{T1}(mm)	D_{A1} (mm)	Error $[(D_{A1} - D_{T1})/D_{T1}]\times 100(\%)$	D_{T2}(mm)	D_{A2} (mm)	Error $[(D_{A2} - D_{T2})/D_{T2}]\times 100(\%)$
Sx6N	0.345	0.279	19%	0.36	0.265	-26%
Sx9M	0.361	0.22	39%	0.282	0.227	-20%
Sx13N	0.434	0.552	-27%	0.425	0.548	29%
Sx16M	0.699	0.418	40%	-	0.445	-
Sx32N	0.808	0.691	14%	0.627	0.604	-4%
Sx39M	1.008	0.99	2%	0.655	0.812	24%
Sxy40M	1.217	0.747	39%	0.82	0.772	-6%
Sxy42M	1.814	1.5	17%	1.023	1.38	35%
Average			25%			18%

Note: D_{T1} is the maximum inner-story displacement in x direction of the first story by test, D_{A1} is the maximum inner-story displacement in x direction of the first story by analysis. D_{T2} and D_{A2} are the corresponding values for the second story.

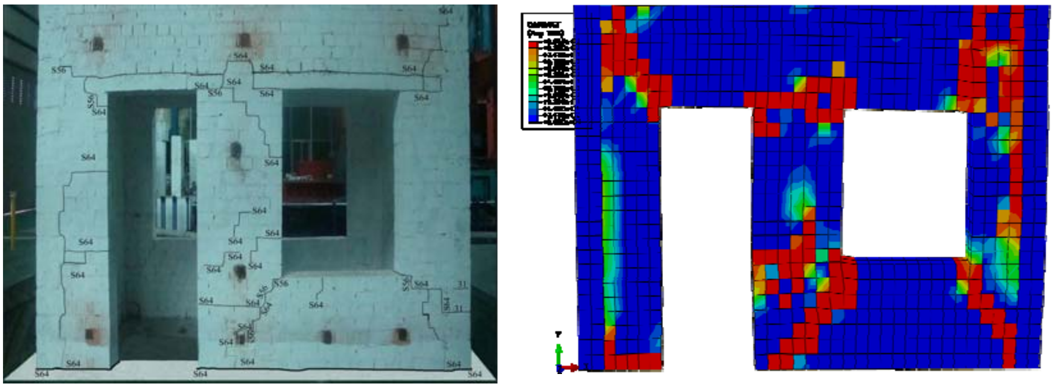
Fig.5 shows the model's damage parameter cloud images in tension in different moments in the 42nd conditions, in which the inputs are by bi-directional artificial waves, and amplitudes of input waves' in x and y direction are 0.235g and 0.194g, each. From Fig.5 we can see that, the structural damages are worsening with the increase in seismic waves hold, the maximum tensile damages generally occur in the seismic wave peaks, and it is unrecoverable. The tensile damages on the window and door openings corner are severely, and the compression injuries are minor.



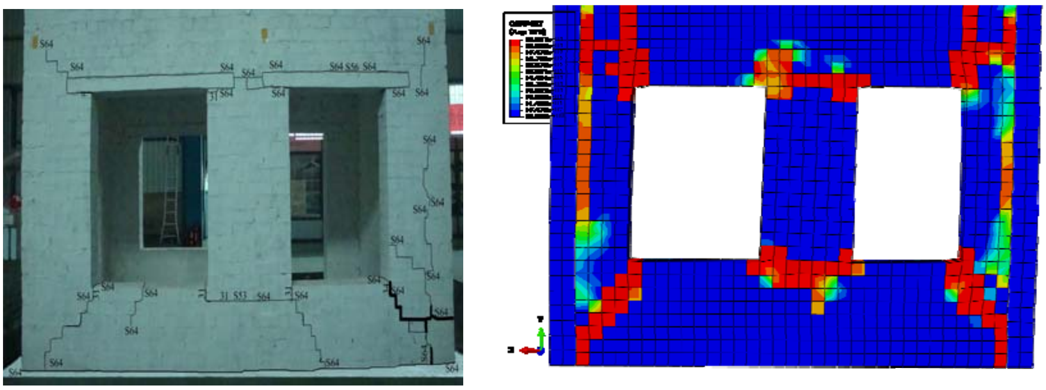
(a) At the 1st second (b) At the 5th second (c) At the 15th second

Figure 5. Damage phenomenon

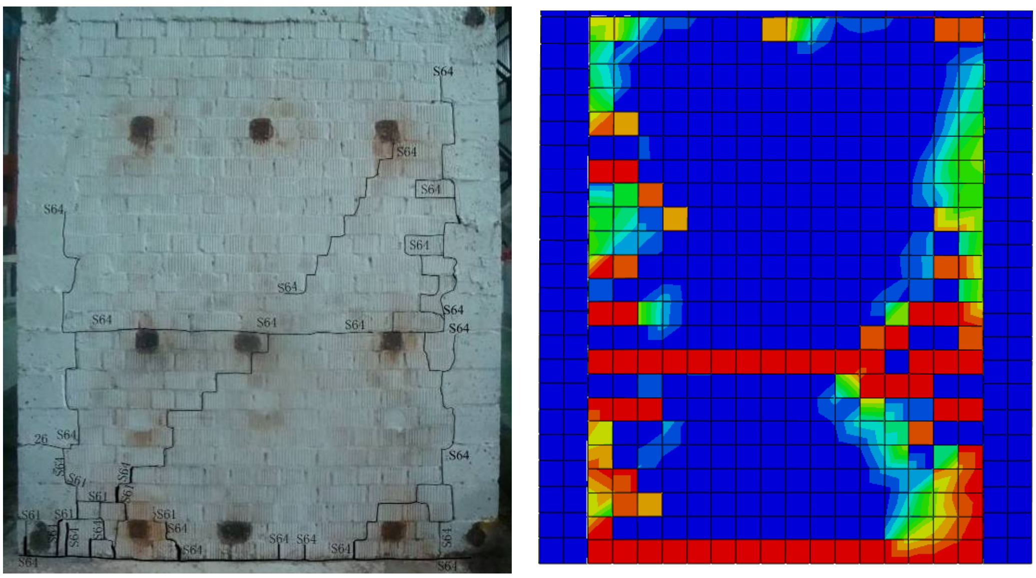
Figure 6 shows the comparison of distribution of tensile damage in longitudinal walls along A axis and B axis and in transversal wall along 1 axis in the first story. It can be seen from Fig.6 that, the tensile damages obtained by the finite element analysis can coincide with the cracks of the model in the test one–to-one, such as, the incline cracks at the window corners and between the wall and window in longitudinal wall along A axis, shown in Fig.6(a), inclined cracks at the window corners of the longitudinal wall along B axis, shown in Fig.6(b), and vertical cracks between the wall and core-tie-columns, shown in Fig.6(c), are all consistent with the actual cracks by tests.



(a) Longitudinal wall along A axis



(b) Longitudinal wall along B axis



(c) Transversal wall along 1 axis

Figure 6. Comparison of tensile damage distribution by analysis and test

Fig.7 shows the contrast of average tensile damage factors of the walls in the model's first story on the walls in the model's second story. It can be seen from Fig.7 that average tensile damages of the first story walls are more severe than the second story, and the results coincide with the damages found mainly in the first floor in the test.

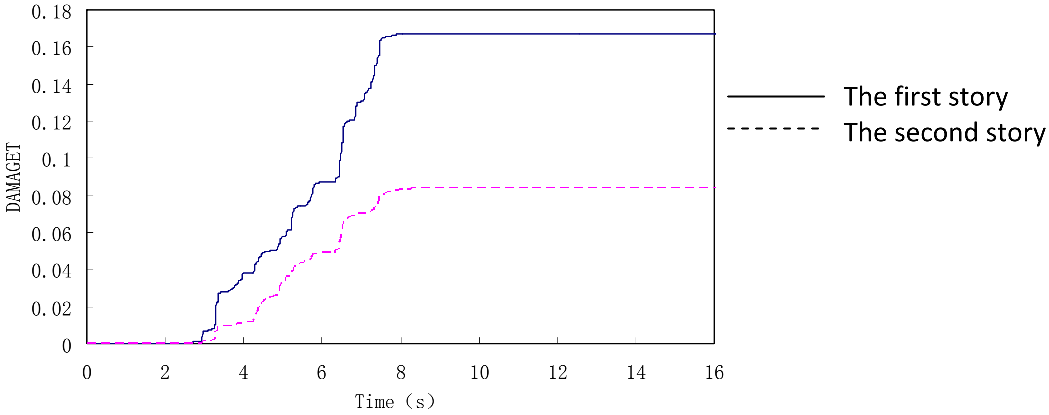


Figure 7. Comparison of average tensile damage parameter in the walls on the first floor and the second floor

5. CONCLUSIONS

Based on the non-linear time history analyses of the masonry model and contrasts on the test, some conclusions can be obtained as follows. The non-linear seismic response analysis of masonry structure under severe earthquake action can be simulated by choosing simple plastic kinematic model and defining appropriate failure criteria with the aid of ABAQUS, and the analysis accuracy is satisfied to some extent. The damage occurs at the corners of doors and windows where the stress concentrations are big. With the earthquake action continuing, the bottom of masonry structure would be damaged severely. Tensile failure images of the model about concrete damage plasticity can simulate the failure characteristics of the masonry structure, constitutively.

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