Study on the seismic behaviour of upper masonry floor with variable stiffness for multi-story brick structures with bottom-frame based on finite element simulation

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Abstract: The upper masonry layers of multi-story brick structures with bottom-frame were usually severely destroyed in an actual earthquake. But, it is not clear that whether the design method of equal rigidity in vertical direction of upper masonry layer is reasonable. In this paper, four contrastive models of bottom frame structures were established though ABAQUS, and the different design methods of variable vertical stiffness were used for the upper masonry layers. The seismic response results show that the upper masonry layers especially the transitional layer is severely damaged when the design method of equal rigidity in vertical direction is used. According to the principle that the stiffness of the floor in proportion to its seismic force, which can lead to uniform failure of each layer, when the upper masonry layer uses the design method of variable stiffness along vertical direction, the seismic performance of the whole structure is greatly improved.

Keywords: bottom frame structure; variable vertical stiffness; elastic-plastic time-history analysis; seismic behaviour.


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1 Introduction

The multi-story brick structures with bottom-frame is a unique structure form in China. Its bottom is composed of an open frame structure, which can be used to establish shops, restaurants and bathrooms adjacent to street. The upper part is a masonry structure building with dense walls. Due to the low cost, short construction period, flexible layout of the bottom layer space, and the ability to meet the residential and living demands of residents at the same time, this structural form exists in large numbers in China’s medium and small towns, especially in street buildings, and is widely used. However, this structure type is a hybrid structural system composed of two different load-bearing and lateral resistant systems, which is bad for seismic performance. Therefore, the damage of the bottom frame structure in previous earthquakes had been very serious (Sun and Zhang, 2012; Shen and Zhang, 2018).

Chinese scholars had carried out wide and deep research on the seismic performance of bottom-frame houses and had made certain achievements. Kong (2018) designed two bottom frame structures in a 1/5 reduced-scale model of the structures which were tested on shaking table. The results of compared analysis test phenomenon showed that the structural measures could significantly affect the seismic performance of the bottom frame structure. Reinforced concrete anti-seismic wall was installed at the bottom, and its seismic performance was better than that of the brick seismic wall. Wang et al. (2018b) studied the seismic response analysis of the bottom frame structure with eccentricity under bi-directional ground motions, and believed that bi-directional ground motions had a great influence on the lateral displacement, shear force and energy dissipation capacity of the bottom frame structure and should be properly taken into consideration in design.

Wang et al. (2018a) investigated and analysed the collapse of the bottom frame structure in the earthquake. Through simulation analysis and earthquake field experience, author believed that the people facing the collapsed side and the wall with fewer parts had a larger survival space, and the top layer had more living space compared with the bottom frame layer. Zhao et al. (2018) studied the influence of the lateral stiffness ratio of adjacent story on the seismic vulnerability of the bottom frame structure. By controlling the stiffness ratio between the bottom frame layer and the adjacent masonry layer, author studied the effect on the transcendence probability distribution of different failure states of the structure. It was believed that the mutation of stiffness between the bottom frame layer and the adjacent masonry layer had the most significant impact on the holistic vulnerability of the structure.

Most of the existing research results strengthen the seismic capacity of the bottom frame layer by strengthening structural measures of reinforcement, setting reinforced concrete seismic walls, increasing stiffness and other methods. But there is a lack of researches on the rationality of the stiffness design of the upper masonry layer. Disasters due to earthquake shows that in most cases, the damage of the upper masonry layer is greater than that of the bottom frame layer. The reason for the analysis is that each floor of the upper masonry structure residence has the same layout and stiffness. When the stiffness of each layer of the upper masonry layer is the same, the masonry layer adjacent to the bottom frame layer is subjected to the largest shear force. It is located in the stiffness conversion layer that force is complex, and it is easily damage first and thus
makes the overall structure lose its own using function. The design method for the equal stiffness of the upper masonry layer needs further improvement. Therefore, it is necessary to study the design method of the varying stiffness of the upper masonry layer, so that the seismic shear on each layer of the upper masonry layer is proportional to its own stiffness, and guide the upper masonry layer to be destroyed uniformly to avoid weak floors. Based on China’s current development status, the bottom frame structure will continue to be an existence of a great number in medium and small towns in China for a long time, so the research in this article has important guiding significance.

Taking a severely damaged base frame structure in Wenchuan earthquake as an example, the authors established four contrast models by ABAQUS finite element software for time-history analysis of nonlinear seismic response. The influence of the unequal stiffness design of the upper masonry on the overall seismic performance of the local floors and structures was discussed and the design suggestions were given to provide reference for the seismic design of the bottom frame.

2 Earthquake damage example of multi-story brick structures with bottom-frame

2.1 Engineering situation and seismic damage

This paper selects a residential building located in Mianzhu City, Sichuan Province as the research object. The total number of floors of the building is 6, the ground floor is a store, and the floors 2–6 are residential buildings [Figure 1(a)]. It was built in 2001, the building was designed with a seismic degree of 0.1 degree and a class II site. The concrete strength of the first-floor frame column, frame beam and reinforced concrete seismic wall was C30, the material grade of the upper masonry was MU10/M5, and the structural strength of the structural column and ring beam was C20. In the Wenchuan earthquake, Mianzhu City was located at the IX degree area. The overall seismic damage of the bottom frame structure showed as follows: the bottom frame shear layer was basically intact, only partially filled wall appears shear diagonal cracks, frame column, frame beam and reinforced concrete seismic wall damage slightly; The damage of the upper masonry layer, especially the transitional layer wall, was more serious, which was manifested as multiple cracks penetrating diagonal cracks [Figure 1(b)] and horizontal fractures [Figure 1(c)] in the load-bearing wall, in the corner of the corner of the door and Windows, the cracking of the wall was due to stress concentration. The surface mortar dropped [Figure 1(d)]. The seismic damage rating of the building was rated as serious damage. The first floor of the structure and the standard floor plan are shown in Figure 2 and Figure 3. The longitudinal definition of the structure is the X direction and the horizontal direction is the Y direction.
Figure 1  Damage of the instance structure, (a) post-earthquake appearance (b) load-bearing wall cross-cracking (c) failure of masonry load-bearing walls (d) cracking of the walls at the corner of the door (see online version for colours)

Figure 2  The first floor plan
2.2 Analysis of earthquake damage cause

The bottom layer of the multi-story brick structures with bottom-frame is the frame structure. Although the bottom layer has the largest seismic shear force, the frame beam-column system has good deformation and energy dissipation capacity, so the bottom layer of the example structure was basically intact in this earthquake without obvious damage. In the upper layers, the second floor which located in the transition layer was seriously damaged. The main reason is that the interlayer stiffness between each floor in upper masonry layers are the same, because of residence that has the same house type and plane layout. In equal rigidity situation, compared with other upper masonry layers, the second floor has the biggest earthquake shear force which leads to the greatest damage, causing the overall structure out of use function.

2.3 Testing of structural natural frequency and result analysis

In order to better grasp the vibration characteristics of the structure and verify the accuracy of subsequent ABAQUS finite element modelling, a local modal test of the structure was proceeded by author after the earthquake. The 941B sensor and G01NET-1 data acquisition instrument were used as test instrument. The test site was chosen on the fourth floor of the structure, which was roughly symmetrical on the plane and close to the load-bearing member. The gear of the 941B sensor was adjusted to a small speed range, the magnification factor selected 64 and the sampling rate selected 100. The unit was adjusted to m/s. Two horizontal sensors were arranged at the same position to measure the vibration frequency of the structure in two translational directions. Each direction was measured twice consecutively and the average value was taken. The photos of modal testing in field are shown in Figure 4.
Figure 4 Modal testing in field, (a) the position of test point (b) the equipment of testing (see online version for colours)

Through signal analysis of the collected data, the time-history curve of structure velocity \( v \) was obtained by using Bessel filter (Wang et al., 2019). FFT spectrum could be obtained by spectral analysis of the time domain waveform. The FFT spectrums which were measured twice in both directions are shown in Figure 5 and Figure 6. The model testing results of the structure are shown in Table 1.

Figure 5 FFT spectrum of the structure for the first testing, (a) FFT spectrum in longitudinal direction (b) FFT spectrum in transverse direction

Figure 6 FFT spectrum of the structure for the second testing, (a) FFT spectrum in longitudinal direction (b) FFT spectrum in transverse direction
Study on the seismic behaviour of upper masonry floor with variable stiffness

Table 1  Results of the structure of modal testing (Hz)

<table>
<thead>
<tr>
<th></th>
<th>The first order in longitudinal direction</th>
<th>The second order in longitudinal direction</th>
<th>The first order in transverse direction</th>
<th>The second order in transverse direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>The first test</td>
<td>3.420</td>
<td>4.609</td>
<td>4.110</td>
<td>Failed</td>
</tr>
<tr>
<td>The second test</td>
<td>3.430</td>
<td>4.609</td>
<td>4.020</td>
<td>Failed</td>
</tr>
<tr>
<td>Average value</td>
<td>3.425</td>
<td>4.609</td>
<td>4.065</td>
<td>Failed</td>
</tr>
</tbody>
</table>

By taking the average value of the two measurement results, it can be obtained that the first order vibration frequency of the structure in the longitudinal and transverse direction is 3.425 Hz and 4.065 Hz. The second order vibration frequency in the longitudinal direction is 4.609 Hz, and the second order frequency in the transverse direction failed to measure. As the mode superposition method is applied to dynamic analysis of structural response in the finite element simulation, the first order frequencies of the simulation calculation in the direction of longitudinal and transverse should be within the allowable error range with the actual measured values. The actual measurement results here are prepared for the verification of the accuracy of the subsequent finite element modelling by ABAQUS.

Figure 7  Change of the plane layout of the standard floor

3  The design of contrast model

In order to explore the methods to improve the seismic performance of multi-story brick structures with bottom-frame and study the reasonable design method of the inter-layer stiffness of the upper masonry, in addition to the example structure, three more comparative models were designed in this paper. The bottom frame layer remained unchanged, while the upper masonry layer adopted different design methods with
variable stiffness. The example structure was marked as M0. By adding walls to the standard floor plan of the example structure, the total lateral stiffness of the floor was increased as shown in Figure 7. Using the design plane of Figure 7 to replace the second layer of the model M0 and increase the lateral stiffness of the second layer, remembering the model M1. Replacing the second and third layers of the model M0 with the design plane of Figure 7, while increasing the lateral stiffness of layer 2 and layer 3, denoted as model M2. Replacing the 2–5 layers of model M0 with the design plane of Figure 7 while increasing the lateral stiffness of layers 2–5, denoted model M3. As the 6th floor was equipment room, the stiffness of the 6th floor did not change. The models are shown in Figure 8, and red floors represent the increase in the story stiffness of the building.

**Figure 8** Schematic diagram of structure model, (a) model M0 (b) model M1 (c) model M2 (d) model M3 (see online version for colours)

### 4 The establishment of finite element model

#### 4.1 Simulation of the main component

ABAQUS software is relatively mature, which has been widely used in civil engineering field (Genikomsou and Polak, 2015; Riaño and Joliff, 2019). By model building through ABAQUS finite element software, and doing finite element simulation and elastic time-history analysis, for example, structure in this chapter. The modelling process was obtained as follows:

1. **Element type:** columns, girders of frame on ground floor and reinforced concrete anti-seismic wall were simulated by solid elements (C3D20R); floors and upper masonry walls were simulated by shell elements (S4R).

2. **Material properties:** in this engineering, concrete used C20, C30, masonry used MU10/M5, MU10/M7.5, steels used HRB335 and HPB235, materials corresponding density, elastic modulus, Poisson ratio and damping ratio were endowed to corresponding materials attributes.

3. **Simulation on steel:** steels in frame columns, frame girders and reinforced concrete anti-seismic wall used surface elements (SF3D4R) to simulate, and steels in floors used the method of adding rebar orders in the pretreatment process of CAE to simulate.

4. **Connection of structure:** tie connection in constraint was used between floors and walls, steels in frame columns, frame girders and reinforced concrete anti-seismic wall used embedded connection to simulate the coordinated action of steels and concrete.
5 Methods of dynamic analysis: structure modal analysis used frequencies calculation in linear perturbed analysis step, and time-history analysis used dynamic analysis steps (dynamic/implicit) to calculate.

4.2 Simulation of the infill wall

The wall is generally simulated by shell elements (S4R). In this paper, to obtain more refined results, the infill wall of the bottom-frame is simplified to equivalent diagonal strut, and truss rod elements (T3D2) is used to simulate it, as shown in Figure 9. Simulation between truss rod elements (T3D2) in filled walls and framed columns used join connection in connector.

Figure 9 Equivalent strut model for infill panels, (a) infill wall (b) diagonal brace model

Framework filled walls components occur lateral deformation under horizontal earthquake action, which filled walls limit the deformation of framework. As lateral displacement increases further, the interface between frameworks and filled walls gradually separate, and stress between frameworks and filled walls transfer each other only along the diagonal line direction of filled walls compression areas. Therefore, the role of filled walls equal to form a similar to pair angle inclined compressions in temperature frame system along framework diagonal direction, and it changed the transfer mechanism of force (Liberatore et al., 2018). This paper uses calculation formula (1) in Tucker (2017):

\[ W = 0.25d'(\lambda h')^{-1.15} \]

where \( W \) is width of equivalent diagonal brace, \( d' \) is filled wall diagonal length (mm), \( h' \) is height of frame (mm), \( \lambda \) is relative stiffness parameter. The calculation formula (2) of \( \lambda \) is:

\[ \lambda = \frac{\sqrt{\frac{E_m \sin 2\theta}{4E_c I_c h}}} {E_c I_c h} \]

where \( E_m \) is the elastic modulus of filled-wall material, \( E_c \) is the elastic modulus of concrete, \( t \) is infill thickness, \( \theta \) is sloping angle of diagonal of infill, \( I_c \) is moment of inertia of column cross-section, \( h \) is the height of infill.
4.3 Constitutive relation of material

Using concrete damage plasticity model from ABAQUS and defining stress strain constitutive relation for materials under compression and tensile behaviours (Karavelić et al., 2019), simulating nonlinear reaction for the whole structure under seismic action. The constitutive relation of related materials on this engineering is selected as follows.

4.3.1 Constitutive relation of concrete

The constitutive relation of concrete is subjected to concrete uniaxial compression (Figure 10) and tensile stress-strain relations (Figure 11) that is given by GB50010-2010 ‘Code for Design of Concrete Structure’

![Figure 10](image1)

**Figure 10** Concrete uniaxial compression constitutive relation model

![Figure 11](image2)

**Figure 11** Concrete uniaxial tension constitutive relation model

The following formulas (3)–(6) are used to calculate the compression stress-strain curve of single axis.

Curve rising section (when \( x \leq 1 \))

\[
y = \alpha_d x + (3 - 2\alpha_d) x^2 + (\alpha_d - 2) x^3
\]

(3)

Curve down section (when \( x \geq 1 \))

\[
y = \frac{x}{\alpha_d (x-1)^2 + x}
\]

(4)

\[
x = \epsilon / \epsilon_c
\]

(5)
where $\alpha_a$ and $\alpha_d$ are parameters related to the rising and falling of the stress-strain curves of uniaxial compression, $f_c^*$ is uniaxial compressive strength of concrete, $\varepsilon_c$ is the peak compressive strain of concrete corresponding to $f_c^*$.

The following formulas (7)–(10) are used to calculate the tension stress-strain curve of single axis.

Curve rising section (when $x \leq 1$)

$$y = 1.2x - 0.2x^6$$

(7)

Curve down section (when $x \geq 1$)

$$y = \frac{x}{\alpha_t(x-1)^{0.7} + x}$$

(8)

$$x = \frac{\varepsilon}{\varepsilon_t}$$

(9)

$$y = \sigma / f_t^*$$

(10)

where $\alpha_t$ is parameter related to the falling of the stress-strain curves of uniaxial tension, $f_t^*$ is uniaxial tensile strength of concrete, $\varepsilon_t$ is the peak tensile strain of concrete corresponding to $f_t^*$.

4.3.2 Constitutive relation of masonry

The constitutive relationship of masonry is the parabola with the ascending section proposed by Liu (2005) and the linearised masonry compression-tension-stress-strain relationship in the descending section (Figure 12). The following formulas (11)–(12) are used to calculate:

$$\frac{\sigma}{f_m} = 1.96 \left( \frac{\varepsilon}{\varepsilon_0} \right) - 0.96 \left( \frac{\varepsilon}{\varepsilon_0} \right)^2$$

$$0 \leq \frac{\varepsilon}{\varepsilon_0} \leq 1$$

(11)
\[
\frac{\sigma}{f_m} = 1.2 - 0.2 \frac{\varepsilon}{\varepsilon_0} \quad \text{where } 1 \leq \frac{\varepsilon}{\varepsilon_0} \leq 1.6
\] (12)

where \(f_m\) is average compression strength of masonry, \(\varepsilon_0\) is the peak compressive strain of masonry materials.

### 4.3.3 Damage factor of materials

Damage models of concrete materials mainly include macroscopic damage mechanics model, microscopic damage mechanics model and micro-plane model (Karavelić et al., 2019; Casolo, 2004). In the plastic constitutive damage model for concrete of ABAQUS, the damage variable \(D\) is used to describe the stiffness degradation caused by the material damage. The damage variable \(D\) is also called as the damage factor. It is a number greater than 0 and less than 1. The closer to 1, the more serious stiffness degradation will happen. \(D\) is derived based on the assumption of material energy equivalence (Mourlas et al., 2019), and the specific calculation formulas (13)–(14) are as follows:

\[
\sigma = E_0 (1 - D) \varepsilon
\] (13)

\[
D = 1 - \sqrt{\frac{\sigma}{E_0}}
\] (14)

where \(E_0\) is the initial elastic modulus, \(D\) is the damage factor.

### 4.4 Selection of seismic motion record

According to the type of site where the building was located, the Wolong earthquakes recorded in the Wenchuan Earthquake for Class II sites were selected as external ground motion inputs of the structure. The time-history curves of X-direction and Y-direction accelerations of the original Wolong earthquakes are shown in Figure 13. Nonlinear seismic response time history analysis of the example structure using X and Y bidirectional ground motion input. The actual peak acceleration which the example structure was received in the Wenchuan earthquake was 0.6 g. To simulate the real response of structures in earthquake, the acceleration in X direction of the four models was adjusted to 0.6 g and the acceleration in Y direction scaled down.

**Figure 13** Acceleration time history of original Wolong ground motion, (a) X-direction (b) Y-direction
5 Analysis of calculation results

5.1 Analysis of structural frequency

The finite element model is shown in Figure 14(a). Through doing modal analysis on instance structure models, calculation of natural frequencies of the whole vibration type of the first three modes are shown in Figures 14(b)–14(d). From the whole vibration type of the instance structure models, it can be seen that vibration type of the first three modes of this structure all conclude certain torsion frequencies in different degree: the first modal shape is mainly in X direction, and the X-direction translation component accounts for 76%. The second modal shape is mainly in torsion, and the torsional component accounts for 65%. The third modal shape is mainly in Y direction, and the Y-direction translation component accounts for 88%. The comparison of modal calculated results of case frame and measured vibration frequencies were shown in Table 2.

Figure 14  Finite element model and modal calculation results, (a) the finite element model (b) the first modal shape (mainly in X direction) (c) the second modal shape (mainly in torsion) (d) the third modal shape (mainly in Y direction) (see online version for colours)

From Table 2, it can be seen that except for non-measured second-step torsion frequencies, and compared with calculated value, the measured value of structural natural
frequencies is 7.8% and 11.6% lower in X and Y directions respectively. This is because the structure suffered a severe damage under WenChuan Earthquake, and then its whole stiffness decreased, damping increased, and its period became longer. Therefore, the calculated results of structural natural frequencies more than measured vibration frequencies are reasonable, and the finite element modelling method of this structure, and subsequent results are reliable.

Table 2 Comparison the modal calculation results and test value

<table>
<thead>
<tr>
<th>Model shape</th>
<th>Direction</th>
<th>Measured frequency/Hz</th>
<th>Calculated frequency/Hz</th>
<th>Error/%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>X</td>
<td>3.425</td>
<td>3.716</td>
<td>7.8</td>
</tr>
<tr>
<td>2</td>
<td>Torsion</td>
<td>–</td>
<td>4.524</td>
<td>–</td>
</tr>
<tr>
<td>3</td>
<td>Y</td>
<td>4.065</td>
<td>4.598</td>
<td>11.6</td>
</tr>
</tbody>
</table>

5.2 Calculation results and analysis of inter-story displacement angle

The damage of the masonry structure is more sensitive to its displacement, so the degree of damage of the masonry structure can be judged by the change of the relative displacement between two adjacent floors. The inter-story displacement angle refers to the ratio of the inter-story maximal horizontal displacement to the story height $\frac{\Delta u}{h}$ under earthquake action. The $\Delta u/h$ of the $i$th floor refers to the maximum value of the displacement difference $\Delta U_i = U_i - U_{i-1}$ of the $i$th and $(i-1)$th floors at various points of the floor plane. The larger the inter-story displacement angle, the more severe the damage of the floor under the earthquake action.

Figure 15 Inter-story displacement angle under 0.6 g Wolong ground motion, (a) inter-story displacement angle in X-direction (b) inter-story displacement angle in Y-direction (see online version for colours)

Under the input of ground motion with the peak acceleration of 0.6 g, inter-story displacement angle $\theta_i$ of each floor of each model M0–M3 is calculated as shown in Figure 15. Compared with model M0, model M1 increases the story stiffness of the transition layer (the second layer), resulting in a significant reduction in the inter-layer displacement angle. The inter-story displacement angle decreases from 0.00251 to 0.00213 in X-direction which is reduced by 15%. And the inter-story displacement angle decreases from 0.00198 to 0.00166 in Y-direction which is reduced by 16%. While the inter-story displacement angle of 3–5 layers are not changed too much. The third floor
has the largest inter-story displacement angle in the upper masonry layers. The inter-story displacement angle of the third floor X-direction reaches 0.00237, Y-direction reaches 0.00174. Model M2 increases the story stiffness of the transition layer (the second layer) and adjacent transition layer (the third layer), resulting in a decrease in the inter-story displacement angle of both the second and the third layer. The inter-story displacement angle of the second floor decrease from 0.00251 to 0.00206 in X-direction which is reduced by 18%. And the inter-story displacement angle of the second floor decreases from 0.00198 to 0.00166 in Y-direction which is reduced by 16%. The inter-story displacement angle of the third floor decreases from 0.00234 to 0.00189 in X-direction which is reduced by 19%. And the inter-story displacement angle of the second floor decreases from 0.00186 to 0.00151 in Y-direction which is reduced by 19%. A more uniform failure of the upper masonry layer with no apparent weakness. The results of layer 2 to layer 3 inter-story displacement angles in model M3 which the story stiffness from 2 floor to 5 floor are increased at the same time consistent with model M2, and the inter-story displacement angles of layer 4 and layer 5 are further reduced. Since the failure of more than three floors in the example structure is not serious, there is little significance to increase the lateral stiffness of the floors above 3 floors to improve the overall seismic performance of the structure.

5.3 Calculation results and analysis of floor transcendence strength

The inter-layer displacement angle is used to judge the degree of structural failure from the displacement, while the floor transcendence strength is used to judge the structural failure from the point of stress. The floor transcendence strength $E$ is defined as the ratio of the maximum seismic shear force experienced by each floor of the structure during earthquake to the sum of the yield strength of each anti-lateral force member in the floor. The corresponding relationship between the floor transcendence strength $E$ and the damage classification of masonry structure is given by Yin and Yang (2004) as shown in Table 3. The larger the $E$ value, the more serious the structural damage.

<table>
<thead>
<tr>
<th>Damage classification</th>
<th>Basically intact</th>
<th>Slight damage</th>
<th>Medium damage</th>
<th>Severe damage</th>
<th>Devastation</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>&lt; 1.0</td>
<td>1.0~1.35</td>
<td>1.35~2.1</td>
<td>2.1~2.5</td>
<td>&gt; 2.5</td>
</tr>
</tbody>
</table>

Under the input of ground motion with the peak acceleration of 0.6 g, the calculation results of floor transcendence strength $E$ between floors in models M0–M3 are shown in Figure 16. As can be seen from Figure 16, compared with model M0, model M1 increases the story stiffness of the transition layer (the second layer), which makes the $E$ value decreases from 2.31 to 2.10 in X-direction which is reduced by 9%, and the damage classification is reduced from severe damage to medium damage. The $E$ value decreases from 1.68 to 1.54 in Y-direction which is reduced by 8%, damage classification is unchanged, the remaining floors have no significant changes. Model M2 increases the story stiffness of the transition layer (the second layer) and adjacent transition layer (the third layer). The value of $E$ in model M2 decreases from 2.31 to 2.09 in the second layer and from 2.39 to 2.09 in the third layer, and the damage classification reduced from severe damage to medium damage, the second layer decreased from 1.68 to 1.55. The third layer dropped from 1.74 to 1.67, the damage level stays unchanged. In the model
M3, which the story stiffness from the second floor to the fifth floor are increased at the same time, and the $E$ values of both second floor and third floor are basically the same as the values of $E$ of the model M2, but the values of the above three layers slightly decrease.

Figure 16  Floor transcendence strength under 0.6 g Wolong ground motion, (a) floor transcendence strength in X-direction (b) floor transcendence strength in Y-direction (see online version for colours)

Figure 17 shows the tension damage nephogram of simulations results

The damage of masonry is mainly tensile failure. Therefore, the damage grade of the structure can be judged by the tension damage degree of the wall. In ABAQUS, the tension damage nephogram of the structure at the final moment can be output, and different colours represent different damage degrees. The blue represents basically intact and red represents serious damage. The damage degree and location of the structure can be judged by tension damage nephogram. Figure 17 shows the simulation results of the wall damage of the example structure. It can be seen that the transition layer wall (the second layer) is the most severely damaged, which is consistent with the seismic damage phenomenon in the actual earthquake, and also verifies the accuracy of the modelling and simulation method.

Figure 17 shows the tension damage of the whole structure of the four models under the input of ground motion with the peak acceleration of 0.6 g. Figure 17(b) is the calculation result of tension damage nephogram of model M1. Compared with model M0, the model M1 increases the lateral stiffness of the second layer, so that the damage of the second layer is improved, but the weak layer is transferred to the third layer. Figure 17(c) is the calculation result of tension damage nephogram of model M2. Compared with model M0, the model M2 also increases the lateral stiffness of the second to third floors of the structure, so that the damage levels of the second to third floors decrease from severe damage to medium damage, the failure of the upper masonry layers is more uniform and there are no obvious weak floors, and the seismic performance is ideal. Figure 17(d) is the calculation result of tension damage nephogram of model M3. Compared with model M0, model M3 simultaneously improves the lateral stiffness of the second to the fifth floor of the structure. The response of the upper masonry layer is the lowest among all the models, not exceeding the moderate damage. However, considering
of improving the lateral stiffness of the second to the fifth floor of the structure means the high construction cost, the practical application value is not great.

**Figure 17** Nephogram of tensile damage of structure under 0.6 g Wolong ground motion, (a) model M0 (b) model M1 (c) model M2 (d) model M3 (see online version for colours)

To sum up, it is suggested to adopt the variable stiffness design method adopted by the model M2. Increasing the lateral stiffness between the transition layer (the second floor) and the adjacent transition layer (the third floor) in the same time, so that the inter-story stiffness is proportional to the seismic shear force of each floor. Comparing with the design method of constant stiffness, it can effectively improve the seismic capacity of the bottom frame structure.

6 Conclusions

1 The multi-story brick structure with bottom-frame is a unique structural form in China, which adapts to current economic development. The bottom is the frame structure and the upper layers are masonry structure. The masonry layer adjacent to
the frame layer is called the transition layer. The seismic damage example shows that the bottom frame layer of the structure is usually slightly damaged, and the upper masonry layer is especially prone to serious damage. The reason is that each layer of the model and plane layout is the same in upper layers because of residence, inter-story stiffness of each layer is the same too. The second floor suffers the largest inter-story seismic shear force. Compared with other masonry floors, the second floor suffers severe damage under the condition of equal stiffness, resulting in the loss of the overall structure’s functional function. The bottom frame floor has good ductility and energy dissipation capacity, so it is not easy to be destroyed in an earthquake.

2 This paper is based on the general-purpose finite element software ABAQUS, an analysis of dynamic elastic plastic time history of the bottom frame structures under earthquake is taken. Results show that the model of the natural frequency of vibration calculation coincides with the measured values, inter-story displacement angle, floor transcendence strength and the tension damage nephogram shows the damage of the transition layer (the second floor) is most serious, which is consistent with the actual damage. The accuracy of the modelling process and the simulation results is verified.

3 The multi-story brick structures with bottom-frame of the upper masonry with variable stiffness design method could improve the overall structural seismic capacity effectively. When the upper masonry is designed with equal stiffness, the transitional layer would often be severely damaged, which would adversely affect the overall structure. Based on the nonlinear seismic response time history analysis of four contrast models, it is suggested to increase the lateral stiffness of the interlayer between the transitional layer and the adjacent transitional masonry simultaneously at the same time, compared with the design method of constant stiffness, it could effectively improve the seismic capacity of the bottom frame structure. According to the principle that the stiffness of the floor in proportion to its seismic force, which can lead to uniform failure of each layer. The seismic performance of the whole structure is greatly improved.

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