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Typical Collapse Modes of Confined Masonry Buildings under Strong Earthquake Loads

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Abstract: Confined masonry structures are a widely applied structural system in many developing countries. During the past Wenchuan Earthquake in 2008, numerous confined masonry buildings collapsed, while many others suffered damage. This study reviews the construction practices of confined masonry buildings in China. Simple models and hand calculation methods are proposed for quantifying the tearing failure of diaphragms, the tensile failure of tie-columns and the sway-mode strength of masonry buildings. The results indicate very good agreement with field observations. The seismic measures that are stipulated in the seismic design codes are very effective for increasing the strength and integrity, but not the ductility of masonry buildings. For those buildings that survived the earthquake, strength rather than ductility protected the confined masonry from collapse or serious damage. Design recommendations are suggested for preventing various types of premature failures and enhancing the lateral strength of masonry buildings.

Keywords: Collapse, ductility, infill walls, integrity, masonry, spectral acceleration, strength, tie-system.

1. INTRODUCTION

The past 2008 Wenchuan earthquake occurred in Sichuan province in China (see Fig. **1**) led to around seventy thousand deaths and numerous injuries. The majority of the deaths and injuries were in Dujiangyan, Mianyang and Deyang. More than 216,000 buildings collapsed, including 6898 schools, and countless buildings were damaged to varying extents.

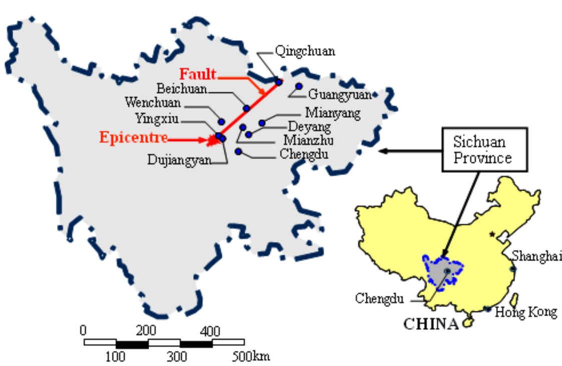

Fig. (1). Location of the epicentre and major affected cities.

Confined masonry structures, which are widely used in China, consist of tie-columns, reinforced concrete (RC) beams and precast concrete hollow floor planks in the upper floors and RC infill frames on the bottom floor. Approximately 70% of residential buildings in China are made of confined masonry (see Fig. **2**). During the Wenchuan Earthquake, numerous confined masonry buildings failed and caused a large number of casualties. Table **1** shows the proportion of masonry buildings destroyed during the earthquake [1]. In Dujiangyan, about 55% of the masonry buildings collapsed or were seriously damaged.

Fig. (2). Typical confined masonry buildings in Sichuan.

In the study reported herein, different collapse models of masonry buildings were proposed. The predicted results are compared with field observations to validate the proposed models. The key findings and main design considerations for the collapse prevention of masonry buildings are presented in this paper. Some misconceptions of the Chinese seismic design codes in the design of seismic resistant masonry buildings are highlighted.

2. TYPICAL CONFINED MASONRY STRUCTURES IN CHINA

In China, confined masonry structures are widely used in the construction of low-rise residential buildings in the regions with the basic design peak ground acceleration (PGA) less than or equal to 0.3g. Various types of brickwork, such as fired clay bricks, fired clay perforated bricks, and small hollow concrete blocks, are commonly used in building con-

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Table 1. The Proportions of the Destruction of Masonry and Bottom Frame Masonry Buildings in the Wenchuan Earthquake

City / Town	Intact	Slightly to Moderately Damaged	Severely Damaged to Collapsed
Cheungdu	96.8	3.21	0.033
Deyang	50.3	47.2	2.5
Mianyang	55.9	33.8	10.3
Mianzhu	20.4	47.0	25.6
Dujiangyan	27.7	24.1	55.2

struction. Most of the collapsed buildings were constructed from fired clay bricks. According to the Chinese design code GB50011-2001 [2], masonry brickwork must be restrained by cast in-situ RC ring-beams and tie-columns. The required member sizes and RC details are dependent on the basic design PGA. In the earthquake affected regions, where the basic design PGA is 0.05g to 0.1g, the minimum size of the tie-columns is 240 mm × 180 mm and the minimum depth of the tie beams is 180 mm. A typical construction detail of the brick walls, ring-beams, and tie-columns is shown in Fig. (**3**). The beams and columns are required to be reinforced by four longitudinal high yield steel bars (nominal yield stress $f_y \geq 335$ MPa) with diameters of 12 mm. Horizontal tie-bars (two 6 mm diameter bars, 1 m long at 500 mm vertical spacing) are used to improve the structural integrity between the tie-columns and the brickwork. To further enhance the bonding between the tie-system and the masonry walls, the tie-columns and beams have to be cast after laying the adjacent brickwork. The concrete used should have a cube strength of at least 20 MPa. Because the small section of the columns often impedes the compaction of the concrete during construction, smaller sized coarse aggregates with better flowability and high workability concrete should be used in construction. However, our field investigations revealed that the quality of concrete was generally poor. Alluvial pebbles, with diameters as large as 50 mm, were used as coarse aggregates in the concrete. The proportion of concrete mix was improper, with the cement content generally low. Honeycombs were commonly found in concrete tie members, indicating that the concrete compaction was insufficient.

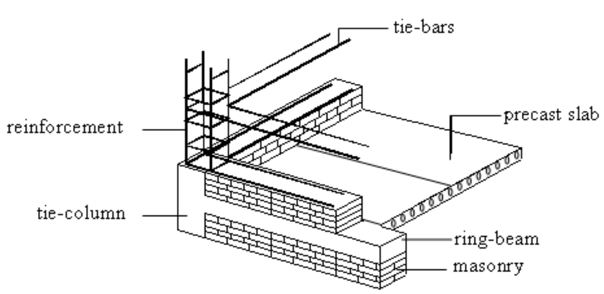

Fig. (3). A typical construction detail of confined masonry structures.

The Chinese design code GB50011-2001 (clause 7.3) [2] stipulates that tie-columns must be provided at the four corners of the exterior walls, at intersections of the transverse wall in the slit-level portion and the exterior longitudinal wall, at both sides of large openings, and at intersections of interior walls and exterior longitudinal walls in large rooms. The code also states that columns should be provided at the four corners of staircases and elevator shafts, and at intersections of each 15 m or unit transverse wall and exterior longitudinal wall. Wang [3] reported that the buildings which strictly adhered to the design code performed satisfactorily under the Wenchuan earthquake load.

The structural arrangements of a typical collapsed residential masonry building in Dujiangyan are illustrated in Fig. (**4**). Hollow precast panels were adopted for the floor supporting system. Infilled reinforced concrete frames were used in the first storey, while confined masonry was used in the second storey and above to resist gravity and lateral loads. The field investigation conducted by the University of Hong Kong revealed that many of the collapsed buildings did not follow the design code requirements. Tie-columns were not usually provided at the intersections of transverse and longitudinal walls, and neither at both sides of bigger openings (e.g., window and door openings). Without sufficient tie-columns and adequate confinement to masonry walls, the load-carrying capacity, the energy dissipation ability and the strength retention of masonry walls were substantially impaired.

3. COLLAPSE MODES OF CONFINED MASONRY STRUCTURES

3.1. Estimation of Basic Design Earthquake Load

The basic design spectral acceleration and the characteristic periods of the regions affected by the earthquake can be found in the Chinese Seismic Zoning Maps GB18306-2001 [4], which are shown in Figs. (**5a**) and (**5b**), respectively. It can be observed that the basic design PGA in the affected areas is relatively low: 50 gal for Mianyang and Deyang and 100 gal for Chengdu, Dujiangyan, Yingxiu, Wenchuan, Beichuan, and Qingchuan.

The Chinese seismic design code adopts the widely used concepts of designing structures to allow no damage during frequent earthquakes, repairable damage during occasional earthquakes and no collapse during rare earthquakes. The frequent, occasional (basic design), and rare earthquakes correspond to earthquakes with a 63%, 10%, and 2% probability of exceedance in 50 years, respectively. Ultimate design earthquake forces are obtained by multiplying the loading demand of frequent earthquakes by a partial safety factor of 1.3.

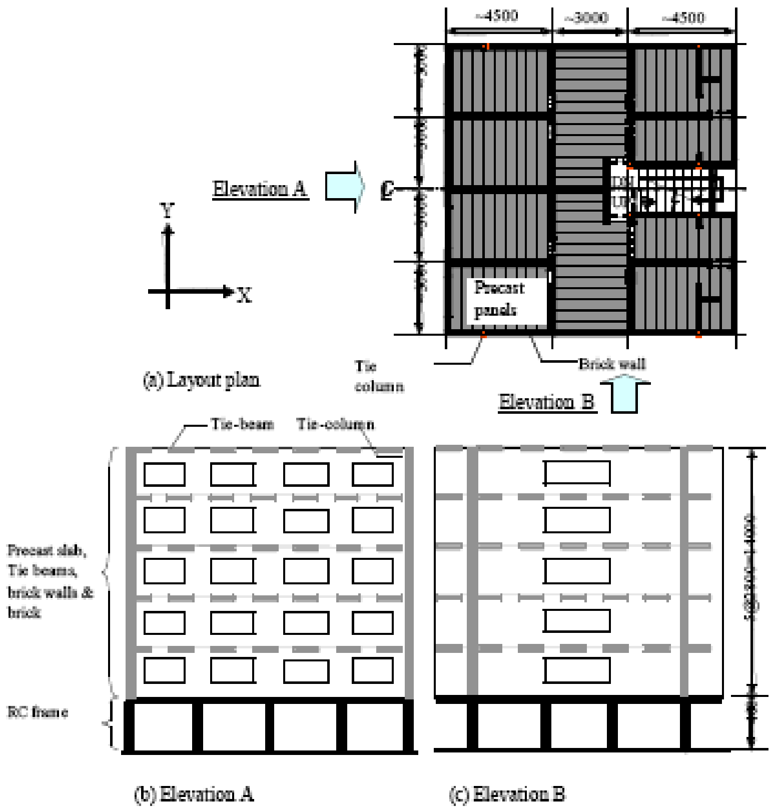

Fig. (4). Typical structural arrangements of masonry residential buildings.

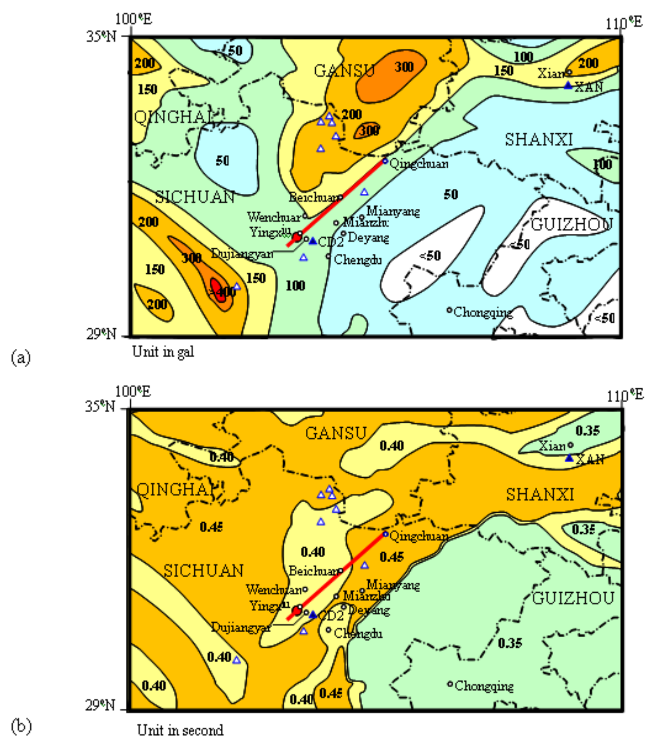

Fig. (5). Distributions of (**a**) basic design *PGA* and (**b**) characteristic period in the earthquake affected areas.

The spatial distributions of recorded PGAs in both E-W and N-S directions of the Wenchuan Earthquake [1] were combined and are plotted in Fig. (**6**). The PGA in the affected area ranged from 100 gal to 958 gal and the maximum PGA of the Wenchuan Earthquake was located near Yingxiu and Beichuan.

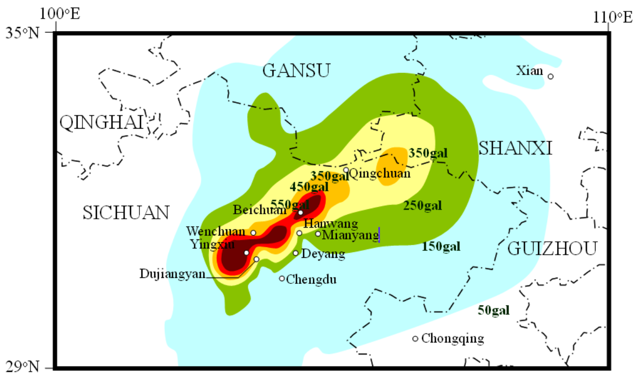

Fig. (6). Spatial distribution of PGAs of the Wenchuan Earthquake.

Table **2** compares the ultimate design PGA and the measured PGA in various affected regions. The measured PGA is found to be about 8 times higher than the ultimate design PGA in most of the affected regions, except at Chengdu, which is only 2.2. It is worth noting that although the design overstrength ratios, which are defined as dividing the measured PGA by the design ultimate PGA, are quite similar for Deyang, Mianyang, Mianzhu, and Dujiangyan, serious damage to buildings (see Table **1**) was only observed at Dujiangyan and Mianzhu (as well as Yingxiu), where the PGA demands were higher than 290 gal. The design overstrength ratio, which can be related to the global ductility demand, was not a good indicator for quantifying earthquake damage. The high PGA demand was the major factor accounting for the serious damage and widespread collapse of the buildings in the affected areas. Further explanations will be given in Sections 3.2 to 3.4.

3.2. Tearing Failure of Floor Diaphragms

Under earthquake load, cyclic tensile forces are generated in floor diaphragms. When masonry infill walls are added in a frame building, the structure resists the lateral earthquake load mainly by the strut and tie action. Very high tensile forces can be induced in the diaphragms, in particular at the first floor of the building. Sufficient reinforcements have to be provided to resist the forces. Horizontal ties should not yield earlier than the occurrence of the ductile beam-sway mechanism for the building. Fig. (**7**) shows a strut and tie model for simulating an infilled frame building of N bays and J storeys subjected to lateral earthquake loads. In this model, the length of each bay and the floor to floor height are both taken as 3m. The member sizes and material properties are presented in Fig. (**7**). This model should be applicable to rigid buildings of which the higher mode effect is not significant. Hence, assuming the fundamental mode shape is approximately by horizontal displacements increasing linearly along the height, the seismic shear distribution can be determined by Eq. (1).

Table 2. A Comparison of the Ultimate Design and Recorded PGAs in Various Affected Regions

City / Town	Ultimate Design PGA (gal)	Measured PGA* (gal)	Design Overstrength Ratio+
Cheungdu	35.6	80	2.2
Deyang	17.8	150	8.4
Mianyang	17.8	160	9.0
Mianzhu (Hanwang City)	35.6	290	8.1
Dujiangyan	35.6	320	9.0
Yingxiu	35.6	550	15.1

Note: * The values were interpolated from Fig. (**6**).
+ Design overstrength ratio is obtained by dividing the design ultimate PGA with the measured PGA.

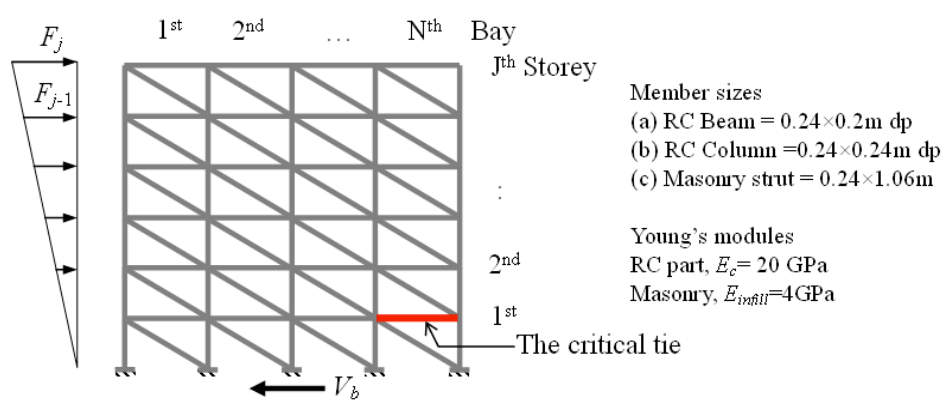

Fig. (7). Strut and tie model for an infilled frame building subjected to lateral earthquake loads.

$$F_i = V_b \frac{h_i \cdot w_i}{\sum_{j=1}^{J} h_j \cdot w_j} \quad (1)$$

where h_i is the level at the i^{th} floor, V_b is the seismic base shear, w_i is the weight at the typical floor and w_J is the weight at the roof which is taken as $w_i/2$. The effective uniformly distributed weight on the diaphragm may be taken as 10 kPa for the masonry buildings in China. The lateral earthquake load V_b, which is same as the base shear force, is expressed in Eq. (2):

$$V_b = W_{eq} S_a \quad (2)$$

where W_{eq} is the equivalent weight of the building, which is equal to $0.85W$ according to the Chinese seismic code GB50011-2001 (clause 5.2.1) [2], and S_a is the spectral acceleration.

By considering that the number of storeys of the building varied from two to ten (J =2 to 10) and the number of bays was either two or four (N = 2 or 4), a finite element analysis using the finite element package ETABS [5] was conducted. The internal forces (F_t) in the critical tie as highlighted in Fig. (**7**) were determined. The normalized tie forces ($F_t N/V_b$) are presented in Fig. (**8**). Using the regression analysis, the normalized tie force was expressed in terms of the number of storeys. The normalized tie force can be estimated by Eq. (3).

$$\frac{N F_t}{V_b} = 0.417 + 0.14J - 0.0066J^2 \quad (3)$$

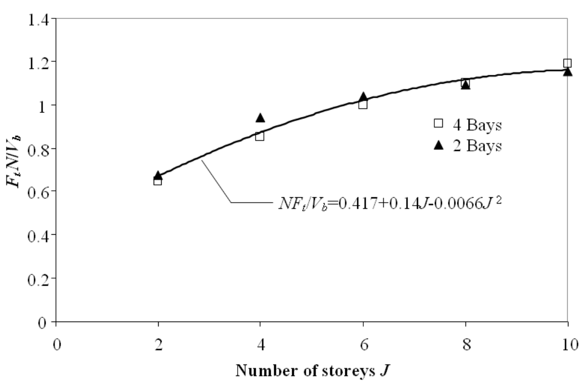

Fig. (8). Normalized tensile force in the critical tie.

For a six-storey building with two bays only, the maximum tie force F_t can be as high as $0.51V_b$. For cast-in-situ floor slabs, the flexural reinforcement provided is often sufficient for serving as ties. However, for restraining precast floor systems, sufficient ties or tie beams have to be installed to prevent the premature tearing failure of the floor diaphragm.

Considering the horizontal force equilibrium, the total required steel area A_s in the diaphragm at the first floor can be estimated as,

$$A_s = F_t / f_y \quad (4)$$

where f_y is the yield strength of the tie reinforcement.

The field investigation revealed that tearing failures of floor diaphragms are not uncommon. An example collapsed building in Dujiangyan as shown in Fig. (**9**) was resulted from a lack of tie beams. Pre-cast slabs together with tie beams have been pulled down at the failure zone.

Although the Chinese design code GB50011-2001 (clause 3.5.5) [2] mentions that precast structures should be properly and firmly connected to the other structural components to ensure the integrity of the structure, our field investigations revealed that many simply supported precast slabs did not comply with this design regulation and were directly seated on tie-beams without any mechanical connections. Only tie beams were provided to resist the tensile force generated by the earthquake loads.

Fig. (9). A 4-storey building without sufficient tie beams collapsed at Dujiangyan.

To demonstrate the significance of the tearing failure of pre-cast floor slabs, the critical spectral acceleration required to tear up the pre-cast floors of the six-storey masonry building with four bays as shown in Fig. (**4**) is estimated based on the proposed equations. As the building has four tie beams along the Y-direction and each tie beam contains four reinforcing bars with a diameter of 12 mm, the total area of tie reinforcement provided on each floor is 1809 mm^2 (=$6^2\pi\times4\times4$). The tie force capacity, according to Eq. (4), is 606 kN (=1809mm$^2\times$335Nmm^{-2}/1000). By Eqs. (2) and (3), the spectral acceleration capacity S_a is found to be 0.3 g which is much less than the peak spectral acceleration demand of around 0.9 g in Dujiangyan. Although the estimated value should be on the conservative side, as the tension stiffening effect of the concrete and brickwork has not been considered in the calculation, the large difference between the demand and capacity demonstrates that the installation of tie beams alone is not sufficient for preventing tearing failure of the floor diaphragm. Precast slabs have to be firmly connected to the other structural components to increase the tension capacity in high seismicity regions.

3.3. Tensile Failure of the Columns

Under the seismic actions, reversed cyclic axial forces are generated in tie columns. Such actions could cause the yielding of vertical ties and debonding between tie-beams and infill masonry walls. More importantly, the loss of confinement for the infill masonry walls could significantly weaken the lateral stability of the load-bearing walls. The extent and importance of such an effect could be investigated through the following simple analysis.

The masonry building with a bottom frame, as shown in Fig. (**4**), is considered again. Knowing that the bottom reinforced concrete frame, which is provided with more tension reinforcement, is much stronger than the masonry superstructure on top of the frame, tension failure usually occurs at the interface between the reinforced concrete frame and the confined masonry structure (i.e., at the top of the first floor). Fig. (**10**) shows a free body diagram of the masonry structure above the bottom frame. The average bearing stress developed in the load-bearing masonry walls at the first floor can be obtained from Eq. (5).

$$\sigma_{avg} = \frac{W_m + T_c}{A_w} \tag{5}$$

where W_m is the effective weight of the superstructure above the first floor, T_c is the total tensile forces developed in the tension ties, and A_w is the total sectional area of load-bearing walls.

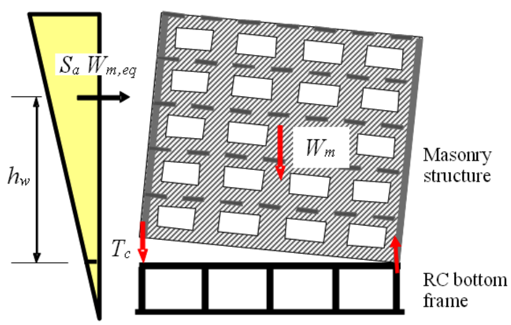

Fig. (10). Free body diagram of the masonry structure above the RC bottom frame.

Under strong lateral earthquake loads, uplifting could occur at the exterior masonry walls. Assuming that the masonry body deforms rigidly, when the tension stress $\sigma_{b,}$ developed by the overturning earthquake moment, just balances the average bearing stress, one can have,

$$\sigma_{avg} = \sigma_b = \frac{\left(M - T_c D/2\right)D}{2I_W} \tag{6}$$

where I_w is the second moment of the area of load-bearing walls, D is the depth of the building and M is the critical earthquake moment, above which could lead to tension failure of the confined masonry walls. Under reversed cyclic earthquake loads, debonding of the mortar between the bricks and the tie beams, could easily and quickly take place. Hence, the tensile strength provided by the masonry walls is conservatively ignored in the calculations. The critical moment M is expressed in terms of spectral acceleration capacity in Eq. (7).

$$M = S_a W_{m,eq} h_w \tag{7}$$

where $W_{m,eq} \approx 0.85\ W_m$ is the equivalent building weight above the first floor and h_w is the moment arm of the resultant lateral forces measured from the first floor. Assuming that the seismic lateral force is triangularly distributed for the low-rise masonry building considered, and h_1 and h_2 are the levels at the first and roof floors, respectively, the moment arm can be expressed as,

$$h_w = \frac{\left(h_2 - h_1\right)\left(h_1 + 2h_2\right)}{3\left(h_2 + h_1\right)} \tag{8}$$

Taking the thickness of the masonry walls to be 240 mm, and following the building arrangements as shown in Fig. (**4**), the total sectional area A_w, second moment of area I_w and effective weight above the first floor W_m are estimated to be 17 m^2, 380 m^4 and 7200 kN, respectively. When h_1 and h_2 are 4 m and 18 m, respectively (see Fig. **4**), Eq. (8) gives h_w = 8.48 m. Furthermore, when the two edge tie columns are both yielded, the total tie force T_c provided is 303 kN (=$6^2\pi\times8$mm$^2\times335$N/mm^2). Using Eq. (5), the average bearing stress is 0.44MPa, which is much less than the nominal compressive brickwork strength of 10MPa and the nominal compressive mortar strength of 5 MPa. Hence, the compressive failure of the masonry walls was unlikely when the walls were properly restrained. By using Eqs. (6) and (7), the spectral acceleration capacity S_a is found to be 0.57 g. It is worth noting that under strong reversed cyclic earthquake loads, tension cracking (see Fig. **11a**) or even out-of-plane dislocations of masonry walls could occur. Those types of damage could severely weaken the integrity and stability of the masonry walls, and hence the gravity load-bearing system. Fig. (**11b**) shows a nearly collapsed 6-storey building that suffered from out-of-plane failure of the masonry walls above the first floor of the concrete frame.

(a)

(b)

Fig. (11). Damage to exterior masonry walls on top the first floors (**a**) extensive cracks and (**b**) out-of-plane dislocations.

Fig. (12). Damage of confined masonry buildings (**a**), (**b**), and (**c**) partial collapses, (**d**) collapse of simply supported hollow precast slabs, and (**e**) pullout failure of a beam.

As evidenced by the extensive collapse of multi-storey masonry buildings in Dujiangyan and Yingxiu, the peak spectral acceleration demands for buildings are likely to be higher than 1.0 g (further details will be given in Section 4). The small tie-columns or ring-beams were too weak to contribute to the resistance of lateral and gravity loads. Thus, the collapse of the masonry walls triggered the progressive collapse of the buildings (see Figs. **12a** to **12d**). Fig. (**12e**) shows a pullout failure of a tie-beam from a beam-column joint. The lack of redundancy of simply supported precast slabs and the insufficient strength of the tie-beam and column systems are believed to be the two major causes of the widespread collapse of confined masonry residential buildings. To improve the integrity and hence the load-bearing capacity of the masonry walls, it is strongly recommended adding vertical ties (two 6mm diameter bars, 1 m long at 500 mm spacing) to attach the masonry infill to the top and bottom tie beams to restrain all four edges of the infill walls and prevent out-of-plane dislocations. Furthermore, larger diameter bars (e.g., T16) with higher yield stress (e.g., 460 MPa) should be used in tie beams and tie-columns to improve the integrity of the building and to avoid the premature tearing failure of buildings.

3.4. Sway Mode Failure Strength

Previous earthquakes and extensive studies [6-8] have revealed that masonry buildings, even those designed with ductility considerations and with uniform storey stiffness, create a soft storey at the bottom of the building if the ground motion is strong enough. When the strength is inadequate, deformations of the building will concentrate in the weakest storey and will increase rapidly. The PGA increasing ratio from the beginning of the formation of a soft storey to the collapse of the structure is only around 1.1 to 1.2 [6,7], which is much smaller than the strength reduction factor of 4 or higher that is typically used in the seismic design of buildings in China. Although it is difficult to accurately determine when a structure collapses, a masonry structure with a soft storey is dangerous enough in a normal design sense. Hence, the non-collapse design of low-rise masonry buildings should make sure that the structure has sufficient lateral strength.

To ensure the whole building resists the seismic induced loading, good structural integrity is vitally important. When premature failures such as those mentioned in Sections 3.2 and 3.3 have been effectively controlled, the entire building can undergo swaying deformations. The ultimate lateral strength associated with swaying is defined as the inherent lateral strength. The inherent strength considers the strengths from both non-structural and structural components, and can be estimated at peak loading (or yielding) status.

The inherent strength might be several times higher than the ultimate design lateral strength due to the following factors. First, the design of structural members for low-rise buildings located in low-seismicity areas is often controlled by gravity rather than lateral wind or earthquake loads. A very high safety margin is reserved for resisting additional lateral loads. Second, bare frame models are usually employed in the design. The stiffening and strengthening effects from non-structural components, such as infill walls and partitions, are conservatively ignored in earthquake design calculations. Thirdly, the actual strengths of construction materials are much higher than the design characteristic strengths.

In this section, a simple model proposed by Su *et al*. [9] was adopted to estimate the inherent strength of masonry buildings. The key element of the model is that stiffness and damping of the structure are characterized by secant properties at maximum response, rather than based on initial elastic properties. The building is assumed to fail in shear mode and the structure is represented as an equivalent single degree of freedom (SDOF) system under seismic attack (See Fig. **13**). In such case, the spectral acceleration capacity S_a can be calculated using Eq. (9):

$$S_a = S_d\left(\frac{2\pi}{T}\right)^2 \tag{9}$$

where S_d is the spectral displacement at the peak load of the structure and T is the lengthened structural period at the peak load.

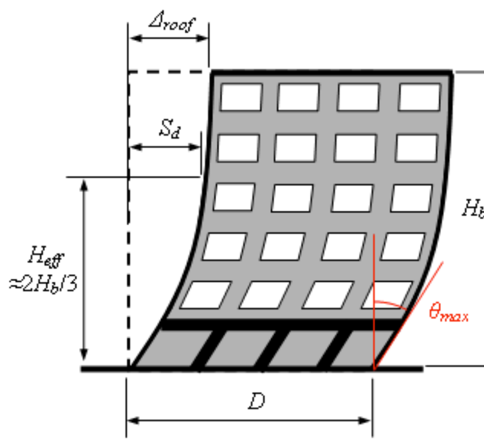

Fig. (13). Lateral deformation of a low-rise building.

The structural period of a building can be conveniently estimated from ambient vibration test data. Based on 12 dynamic test results of low-rise masonry buildings in China, Liang and Chen [10] reported that the structural period T of masonry buildings may be expressed as,

$$T = 0.0463\beta H_b / \sqrt{D} \tag{10}$$

where D is the depth of the building and β is the period shift factor. Under seismic actions, masonry buildings undergo inelastic deformations due to, for instance, cracking and sliding brickwork. The reduction in stiffness leads to lengthening of the structural period under strong shakings. As the structural period can be related to the structural stiffness through the well-known relationship ($T=2\pi\sqrt{M/K}$), the period shift factor can be obtained from the stiffness degradation factors. According to the shaking table analyses [7,11] of confined masonry structures, the period shift factor (β) of infilled frames is found to range from 1.66 to 2.23 with a mean around 1.9.

As the vibration shapes of various shear-mode dominant low-rise buildings are very similar under earthquake excitations, S_d can be related to the maximum inter-storey drift ratio θ_{max} by Eq. (11).

$$S_d = \frac{H_b\theta_{max}}{\lambda} \tag{11}$$

where H_b is the height of the building and λ is the drift factor depending mainly on the height and type of the building. Miranda and his co-worker [12,13] have determined the drift factor explicitly. By substituting Eq. (11) into Eq. (9), one can obtain Eq. (12):

$$S_a = \frac{H_b\theta_{max}}{\lambda}\left(\frac{2\pi}{T}\right)^2 \tag{12}$$

Eq. (12) summarizes the key parameters that affect the lateral inherent strength of a building. Under the peak loading condition, θ_{max} is equal to the yield inter-story drift ratio. Previous experimental studies [7,11,14,15] of confined infill walls reported that the yield drift ratio primarily depends on the geometry and configuration of walls, the confinement steel content and the construction methods, and typically ranges between 0.5% and 0.9%.

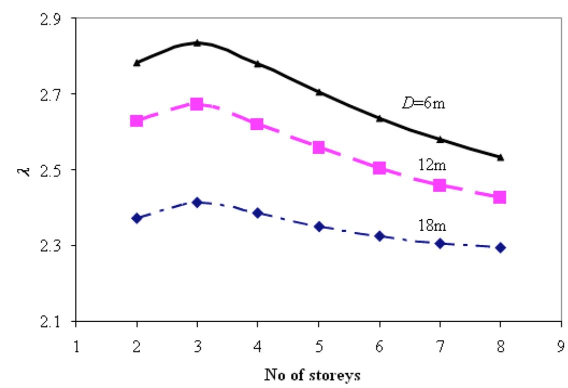

Fig. (14). Variations of drift factor λ against the number of storeys.

To illustrate the application of the abovementioned theory, the masonry building as depicted in Fig. (**4**) is considered again. Assuming a global ductility capacity of 2.5 for brittle masonry buildings, the variations of drift factor to the number of storeys with building depth varying from 6m to 18m determined according to references [12,13] are shown in Fig. (**14**). For the building with a depth of 12m and 6 storeys, λ is found to be 2.50. The corresponding structural period T is found to be 0.457 sec (=0.0463×1.9×18/√12) according to Eq. (10). Assuming the yield inter-storey drift θ_{max} is equal to 0.7% and using Eq. (12), the spectral acceleration capacity S_a is equal to 0.97 g (or 9.52 ms^{-2} =18×0.007×(2π/0.457)^2/2.5) is obtained. This implies that when stronger tie-columns and tie-beam systems are provided, a more deformable swaying failure mode could be enhanced. The lateral strength, in terms of spectral acceleration capacity, could be increased remarkably from 0.57 g to around 1.0 g for 6-storey masonry buildings.

4. ESTIMATED RESPONSE SPECTRA

The response acceleration spectra of some of the affected areas, such as Zengjia, Bajiao, Qingping and Wolong, were

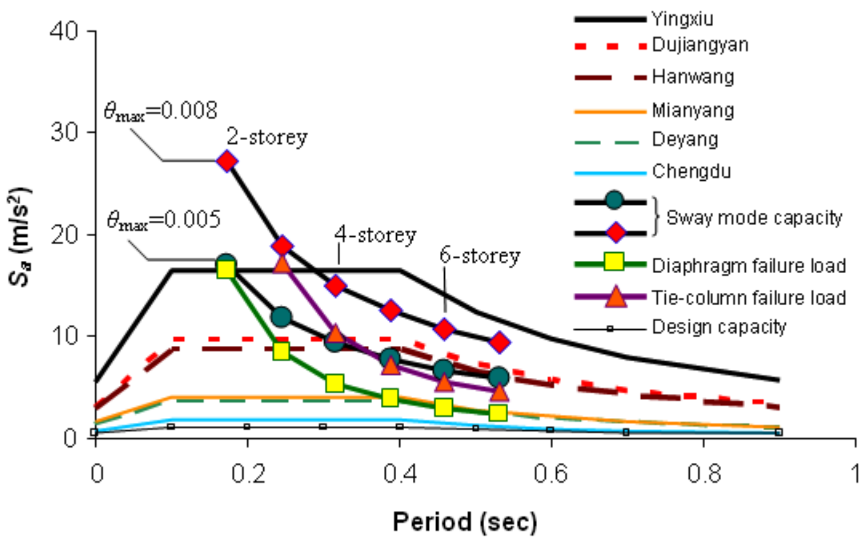

Fig. (15). A comparison of spectra acceleration demands and capacities of masonry buildings in various earthquake affected areas.

given by Li *et al*. [16]. From these spectra, one can find that the dominant characteristic period of the ground motions was at 0.4 sec, the acceleration spectral ratio ranged from 2.7 to 5.4 with an average of 3.5, and the descending branch of the acceleration spectra in the range of 0.4 sec to 1 sec was roughly proportional to the function $T^{1.4}$. Based on these parameters, together with the recorded PGA as shown in Fig. (**6**), the response acceleration spectra at various earthquake affected areas were estimated and presented in Fig. (**15**).

By considering the masonry building as shown in Fig. (**4**) and assuming the number of storeys varies from 2 to 7, and β=1.9, the structural period and spectral acceleration capacity of multi-storey masonry buildings were calculated. Fig. (**15**) shows the comparison between the failure strengths, design capacity and the estimated seismic loads (all in terms of spectral accelerations). To take into account the variations of material properties, yield rotations of 0.5% and 0.8% were considered. The design capacity was calculated according to the Chinese seismic design code GB 50011-2001 Clauses 5.1.4 and 5.1.5 [2]. The results show that buildings generally have sufficient lateral strengths in Chengdu, Deyang and Mianyang, while three- to seven-storey buildings in Dujiangyan, Hanwang and Yingxiu could have insufficient strength. Furthermore, the results show that the lateral strength of masonry buildings is very sensitive to building heights. Low-rise masonry buildings with three or fewer storeys are more robust under seismic attacks. Diaphragm failure of these buildings is the most common failure mode. The tie-columns of taller (and usually more slender) masonry buildings are likely to fail under tension due to the high over-turning effect. The loss of confinement to the infill walls could trigger the walls to detach from the main structures and the subsequent out-of-plane dislocations. The structural period of an unrestrained infill wall could be a few seconds, and the displacement demands for long-period structural components in the affected areas could be much higher than 200 mm. Bearing in mind that the actual displacement demands experienced by the infill walls are calculated by adding up the spectral floor displacements and the ground displacements, the peak ground displacements were 200 mm in Chengdu and Mianyang, more than 500 mm in Dujiangyan and around 1000 mm in Yingxiu [17].

It should be noted that this paper has no intention of using the aforementioned simple analyses to explain all the possible causes of the collapse of a large variety of masonry buildings in the Wenchuan Earthquake. It is very difficult, but not impossible, to obtain all the essential material and geometric parameters to define the pre-earthquake conditions of individual buildings for conducting detailed collapse analyses. However, the above simple analyses did explain the damage and collapse of the buildings in the Wenchuan Earthquake very well, and are worth considering in future seismic resistant design.

There are a number of possible ways to prevent the premature failure of masonry walls: for example, one can (1) reduce the height-to-depth ratio of the buildings, (2) add more tie-columns and tie-reinforcement, and (3) connect the infill walls to the tie-beams using tie-bars. The above provisions are very effective for improving the integrity of masonry buildings under seismic loads.

Finally, if the premature failures could be controlled for a multi-storey masonry building, a more deformable sway mode failure would probably happen. To further increase the lateral strength of the sway mode, one could (1) increase the yield inter-storey drift ratio of the building by using higher strength construction materials, (2) increase the initial stiffness (or shorten the initial period) of the buildings by increasing the size of structural members, and (3) avoid having structural irregularities so that the drift factor λ would not be larger than 2.5. Based on the above discussions, providing lateral strength should be a viable solution for the seismic resistant design of masonry buildings.

5. FIELD OBSERVATIONS

In our post-earthquake field investigations, we visited the Chengdu and Mianyang urban areas, as well as Dujiangyan and Yingxiu cities. Our observations broadly agreed with the damage data presented in Table **1**. The main findings are,

1) No building collapses were observed in the Chengdu urban area;

2) Only one partially collapsed 6-storey building resulting from the failure of load-bearing masonry walls was found at the Mianyang urban centre (see Fig. **16**);

3) Numerous partial collapses of 5- to 6-storey buildings occurred in Dujiangyan; and

4) Widespread collapse of buildings occurred in Yingxiu.

Fig. (**15**) shows that the proposed simple strength calculations explain our field observations very well. Apparently, strength, rather than ductility, protected the confined masonry buildings from collapse or serious damage.

Fig. (16). A partial collapsed 6-storey building due to collapse of load-bearing masonry walls at Mianyang city.

6. THE EFFECTIVENESS OF CHINESE SEISMIC DESIGN CODE ON PREVENTION OF BUILDING COLLAPSE

The effectiveness and importance of the seismic measures in the Chinese seismic design code have been evidenced in the Wenchuan earthquake. However, there is a different understanding regarding the real effect of the seismic measures, in particular the efficacy of tie-systems. Wang [3] reported that, for masonry structures designed and constructed according to the Chinese design code, tie-columns and tie-beams succeeded in acting as the second line of defence after the damage of the masonry walls. However, as demonstrated by the present study, many tie-systems were too weak to contribute to the resistance of gravity and earthquake loads in high seismic intensity areas. We believe that the real effect of the tie-columns and tie-beams system is to restrain infill walls and resist the tensions generated in the global lateral load structural systems. The tie-system can enhance the integrity of the entire building and effectively avoided premature failures under cyclic earthquake loads. However, the restraining frame of tie-columns and tie-beams cannot be considered as a second defence line.

In Section 3.1, earthquake load demands were found to be about 8 times higher than the corresponding design ultimate strength in many seismic affected areas. In the cities such as Chengdu and Mianyang, despite the high PGA demands and design overstrength ratios, almost all of the buildings could avoid collapse under the Wenchuan Earthquake. As mentioned in previous sections, the good performance of the buildings in these areas is mainly attributed to the high inherent strengths of the masonry buildings (see Fig. **15**), but not the ductility capacity. In fact, the actual overstrength ratio (using the inherent lateral strength as the denominator) of masonry buildings is very small, around 1.15 according to [6,7]. The seismic measures presented in the Chinese design code show the efficacy of increasing the lateral strength, but not ductility. Hence, the damage to buildings has a better correlation with PGA demands, but not the design overstrength ratios. When the seismic loading demands in terms of PGA is higher than the inherent spectra acceleration capacity of the buildings, extensive damage or even collapse of the buildings occurs. The seismic measures defined in the Chinese seismic design code are quite effective in ascertaining the required integrity (as demonstrated in Section 3.2), but not the ductility of structures. Nonetheless, the tie-columns and tie-ring system, and other ductile detailing requirements, are still considered to be of vital importance for increasing the yield rotation and hence the inherent strength of a building.

Our post-earthquake field investigations further revealed that the ductile swaying failure mode of masonry buildings was rarely seen in the earthquake-affected areas. This is because in real construction of masonry buildings, reinforced concrete beams were often strengthened and stiffened by the adjacent slabs, masonry walls, and other non-structural components. The principle of strong column-weak beam was difficult to implement in real masonry buildings. If ductility design is difficult to implement, should we look for an alternative? Sufficient strength rather than ductility should be considered as the decisive factor in the design of masonry buildings. If rare earthquake loads are used for the non-collapse design of low-rise masonry buildings– judging from the variations between inherent strength and the rare earthquake loads the increase in the construction costs for masonry buildings should not be substantial.

Furthermore, many current seismic design codes adopted component-based seismic design without adequate considerations of the global integrity of the entire structures, and the limited ductility capacity of brittle construction materials and global collapse mechanism should be thoroughly reviewed. The proposed simple analyses in light of global collapse mechanisms warrant further refinement and improvement.

7. CONCLUSIONS

Based on field studies of the areas affected by the Wenchuan Earthquake and the subsequent seismic assessments of masonry buildings, the major findings are summarised as follows. The major factors influencing the inherent strength and preventing premature failures have been briefly discussed. Various failure models have been presented to quantify the tearing failure of floor diaphragms, the tension failure of tie-columns and the sway-mode failure of masonry buildings. The strength calculations explain our field observations very well. The substantial under-design of strength is found to be the primary reason for the widespread collapse of masonry buildings. For those masonry buildings that survived in this earthquake, their inherent strength, rather than their ductility, protected them from collapse. Moreover, the seismic measures stipulated in the Chinese seismic design code are very effective for increasing the strength and integrity of structures (but not ductility), and should be followed during design and construction. Furthermore, design recommendations have been given for preventing premature failures and increasing the inherent lateral strength of masonry buildings. The strength-based approach should be used in the design of low-rise masonry buildings. However, the ductility design approach is preferable for the design of medium and high-rise buildings. Rare earthquake loads could be used directly in the design of masonry buildings to achieve the objective of “no collapse in rare earthquakes”. Because the inherent strength of the buildings constructed according to the current Chinese seismic code is already very high, the increase in construction costs should not be significant.

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