Seismic Behavior of Asymmetric RC Frame Building Systems with One Major Wall

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ABSTRACT:

Traditionally, the seismic torsional resistance of an asymmetric building is evaluated based on the assumption that the torsional resistance of each element can be neglected. However, for some particular asymmetric structural configurations which are usually found in regions of low seismicity, such assumption might not lead to a conservative design. In this paper, a simplified one-story single-wall-frame structural model was adopted to study the effects of the torsional stiffness/resistance of the wall on the overall seismic response of the model. It is found that the torsional resistance of the wall can be substantially mobilized due to the large rotational displacement under ground motions. Simultaneously, the shear resistance of the wall can also be utilized because of the dynamic effect due to the rotary inertia of the mass. As a result, the interaction of shear and torque of the wall should be considered. It is also shown that this special structural system may actually fail under the combined shear-torsion loadings, in which the system is considered to be safe when the torsion stiffness/resistance of the wall is neglected. Moreover, the exclusion of the torsional resistance of the wall does not always lead to a conservative estimation of its shear demand. In some cases, the consideration of the wall torsion stiffness/resistance considerably reduces the torsional twist of the structural system.

KEYWORDS: Seismic behavior, asymmetric structure, torsion of wall, single-wall-frame system,

1. INTRODUCTION

In earthquake engineering design, it is a common practice to provide a multi-walls system for an asymmetric reinforced concrete building to resist the seismic torsional effects. On the other hand, the research study on the seismic performance of an asymmetric reinforced concrete building with one major wall and multi-frames was very limited because this particular structural configuration is usually found in regions of low seismicity like Hong Kong. However, the recent destructive Wenchuan Earthquake which was unexpectedly occurred in a region previously considered to be of low seismicity, caused severe structural damages and tremendous loss of lives. In fact, tracing back to the year 2005, an earthquake of magnitude 5.7 hit Jiujiang of China, a city ranked with a seismic level lower than that of Hong Kong, had already raised concerns on the performances of Hong Kong buildings under a relatively strong earthquake of a low occurrence level but high consequence.

For a multi-walls building system, the torsional resistance comes from the base shear distributed among the wall elements, and ignores the torsional resistance of each individual wall (Christopher L. Kan and Chopra, 1979, De La Llera and Chopra, 1995; Paulay, 2000). However, the effect of the torsional stiffness and resistance of the major wall on the overall seismic behavior of a single-wall-frames building system might be significant. To the authors' best knowledge, no such study so far has been conducted to provide a better understanding on this aspect.

This paper presents a study of a simplified one-story single-wall-frame model under various ground excitations. The material properties and reinforcement details are assigned to the wall and column elements so that the hysteretic flexural and shear models of each element can be calculated. In particular, the hysteretic torsional model of the wall element is estimated, based on the tests conducted by the authors. Thereafter, the analytical study of nonlinear earthquake response of the model is conducted and the results are discussed.

2. STRUCTURAL MODEL

An idealized one-rigid-story asymmetric single-wall-frame model built on a rigid foundation is shown in Figure 2.1. The center of mass CM is located at the geometric center of the rigid slab. The wall and the columns are situated at a distance of b/2 from each side of center of mass CM about the y-axis. The model is symmetric with respect to the x-axis. For simplicity, only the ground motions along the y-axis are considered. Therefore, the deformation of the model can be described by the lateral displacement u_y of the CM, relative to the foundation in the y-axis and the torsional rotation u_θ at the geometric center of the slab (or CM) about the vertical z-axis.

Let k_c and k_w represent the lateral stiffnesses of the column and the wall respectively. $k_{\theta w}$ represents the torsional stiffness of the wall about the vertical z-axis through its shear center. x_c and x_w are defined as the x-coordinates of column and wall with origin at the center of mass. Then the total torsional stiffness of the model about the center of mass $k_{\theta\theta}$, becomes (the torsional stiffness of each column is small enough to be neglected):

$$k_{\theta\theta}=k_{\theta w}+(x_w{}^2k_w+3x_c{}^2k_c) \tag{2.1}$$

As mentioned before, in a multi-walls system, the term $x_w{}^2k_w+3x_c{}^2k_c$ in Equation (2.1) will be dominant, and therefore the torsional stiffness of each individual wall $k_{\theta w}$ is usually not considered. However, in this single-wall-frame model, $k_{\theta w}$ might be large enough to be make a considerable contribution to the total torsional stiffness of the system $k_{\theta\theta}$, and this concern will be quantified in the following sections.

Moreover, the center of stiffness and center of strength of the model need to be clarified as well. The center of stiffness, denoted as CR, is located at distances of e_{rx} (the stiffness eccentricity) from the CM along x-axis, where

$$e_{rx}=\frac{3k_cx_c+k_wx_w}{3k_c+k_w} \tag{2.2}$$

Also considering the nominal flexure shear strengths of the column and the wall F_c and F_w respectively, the location of the center of strength, denoted as CV, can be determined similarly. The distance between CM and CV is the strength eccentricity, e_{vx}, and is given by Equation (2.3):

$$e_{vx}=\frac{3F_cx_c+F_wx_w}{3F_c+F_w} \tag{2.3}$$

Since the stiffness and nominal strength of a wall are much larger than those of a column, the stiffness eccentricity e_{rx}, expressed by the Equation (2.2) and strength eccentricity e_{vx}, obtained from the Equation (2.3) will be very close to the x-coordinate of the wall x_w. Previous studies (Paulay, 2000, Myslimaj and Tso, 2002) have shown that such an asymmetric system with large e_{rx} and e_{vx} will undergo a serious torsional response with a large rotational displacement during a ground motion. Under such circumstance, the torsional resistance of the single wall will definitely be mobilized.

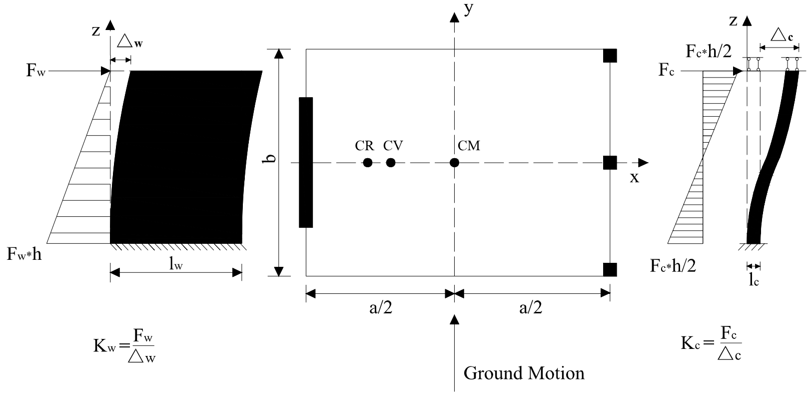
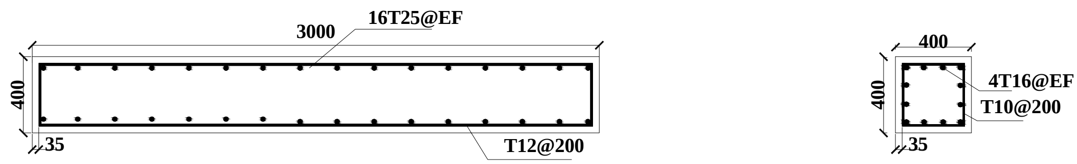


Figure 2.1 Structural Model

3. INELASTIC DYNAMIC ANALYSIS

3.1. Structural Model Considered in Dynamic Analysis

In order to perform a series of dynamic analyses, the dimensions of the rigid reinforced concrete top slab is taken to be 8m by 6m. The story height is 7.5m. The cross-section of the reinforced concrete wall is 3m by 0.4m, and that of each reinforced concrete column is 0.4m by 0.4m. The mass lumped on the top slab is determined from the axial compression ratios of 0.34 for the columns and 0.15 for the wall. Thus, if the concrete cylinder strength $f_c^{'}$ is assumed to be 35MPa, the weight of the slab is approximately 12000kN.



(a) Wall Section (ρ_l =1.3% and ρ_t =0.4%) (b) Column Section (ρ_l =1.7% and ρ_t =0.36%)

Note: ρ_l =Reinforcement ratio of longitudinal rebars, ρ_t = Reinforcement ratio of transverse rebars.

Figure 3.1 Reinforcement Details for Wall and Column Sections

In order to obtain the hysteretic force-displacement relationship, the reinforcement details of the elements are shown in Figure 3.1. These reinforcement ratios are typical values for buildings in Hong Kong. It is also assumed that the reinforcing steel yield strength f_y is 400MPa and its modulus of elasticity E_s is 200GPa. Many researches conducted by Paulay have shown that for the dynamic analysis of reinforced concrete structures under seismic excitations, it is adequate to characterize the moment-curvature relationship of an individual section by an idealized bilinear relationship, as shown by the bold line in Figure 3.2. The first branch of the curve is obtained by extrapolating a straight line from the origin point passing through the point $(M_y, \phi_y^{'})$ to point (M_n, ϕ_y). The second branch is drawn by linking (M_n, ϕ_y) and (M_u, ϕ_u). The definition proposed by Priestley et al (1998) is adopted here to obtain the point (M_n, ϕ_y). The point (M_n, ϕ_y) is selected by constructing a horizontal straight line passing through the full moment-curvature curve at point $(M_n, 5\phi_y)$. The full moment-curvature curves are calculated using Response-2000, a 2D sectional analysis tools developed by Evan Bentz (2001). Figure 3.3 shows the calculated bilinear hysteretic moment-curvature models for the wall and the column.

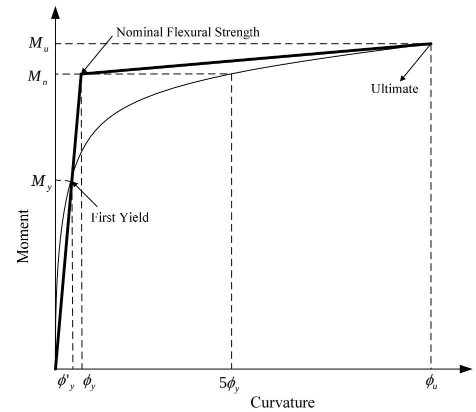
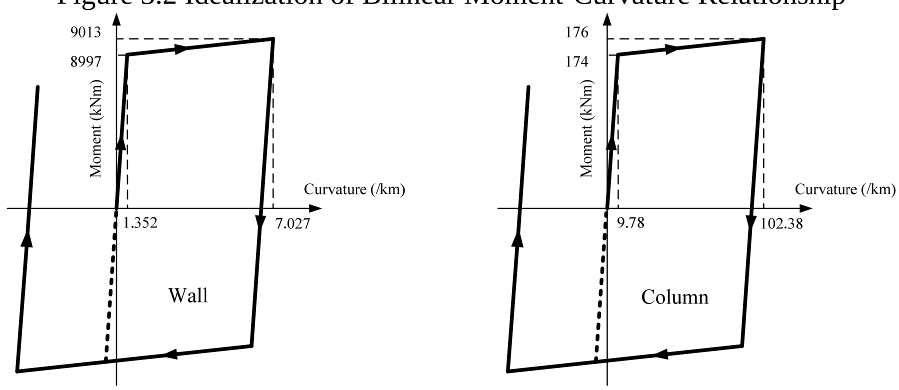
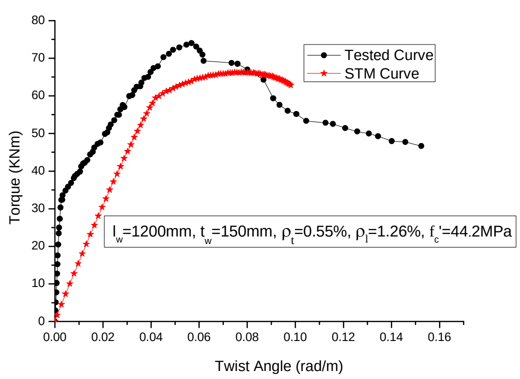


Figure 3.2 Idealization of Bilinear Moment-Curvature Relationship



(a) Wall (b) Column

Figure 3.3 Bilinear Hysteretic Moment-Curvature Relationship of Wall and Column



Note: l_w = length of wall, t_w =thickness of wall

Figure 3.4 Typical Torque-Twist Curve

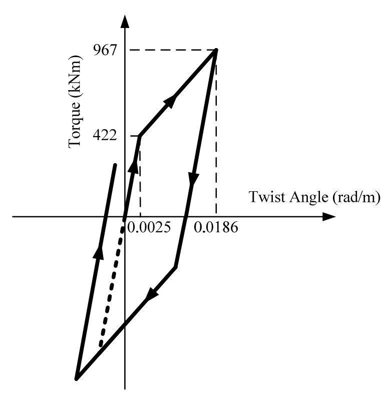


Figure 3.5 Bilinear Hysteretic Torque-Twist Relationship of Wall

One series of tests consisting of eight specimens with different aspect ratios and reinforcement ratios were conducted by the authors to investigate the behavior of RC walls under pure torsion (Peng and Wong, 2007). A typical experimental torque-twist curve and the associated theoretical curve calculated from the softened truss model (STM) developed by Mo and Hsu (1984) for the sectional analysis of a torsional concrete member are shown in Figure 3.4. Several key findings of the tests are herein presented and will be adopted to determine the proposed torque-twist relationship of the wall (see Figure 3.5) in this dynamic study. Firstly, the tested torque-twist curves showed that the overall torsional behavior of wall prior to the maximum torque could be primarily divided into two stages: before cracking and after cracking. Thus, four important parameters should be reasonably predicted to form a bilinear torque-twist curve. They are ① the cracking torque, $T_{w,c}$, ② the twist angle at cracking torque, $\theta_{w,c}$, ③ the maximum torque, $T_{w,\max}$, and ④ the corresponding twist angle at maximum torque, $\theta_{w,\max}$. Secondly, the cracking torque $T_{w,c}$, could be better estimated by the skew-bending theory modified by PCA tests (Hsu, 1968) with a minor modification. Also, the tested twist angles at cracking torque of all the eight specimens were found to be within a range of 0.0022rad/m to 0.0028rad/m. Hence, for the wall element in this study, an average value of 0.0025rad/m is chosen to be the twist at first cracking and the associated torque is 422kNm estimated from the skew-bending theory. Test results also revealed that the maximum measured torque and its corresponding twist angle were relatively comparable to the predictions using the STM. More precisely speaking, for the tested specimens with similar reinforcement ratios to the one used in the current study, the predicted maximum torques were about 90% of the tested values and the predicted twist angle at maximum torques were approximately 150% of the tested values. Consequently, the maximum torque and the associated twist angle of the wall element in Figure 2.1 are calculated as 967kNm and 0.0186 rad/m, respectively. The final hysteretic torque-twist model of the wall element in this study is shown in Figure 3.5.

After all the hysteretic models required in the dynamic analysis have been determined, the lateral stiffness of the wall and the column as well as the torsional stiffness of the wall can be calculated. It is found that the torsional stiffness of the wall $k_{\theta w}$ is 168800kNm, which is about 18% of the total torsional stiffness (950224kNm) of the whole system, and this is denoted as the Reference Model A. For comparison purposes, we also study another Model A' which is same as the reference model 1 except that the term $k_{\theta w}$ is neglected.

Three ground motions are used in the dynamic analysis. The first one is a simple sine wave with a period of 1.5 seconds and the peak ground acceleration (PGA) of 0.15g. The second one is the 1940 El Centro 270 degree component scaled to the PGA of 0.2g. The third input motion is 230 degree component of ground motion recorded at Huston Road, Array 6 during the 1979 Imperial Valley earthquake scaled to the PGA of 0.2g. A damping ratio equal to 5% is adopted. The software CANNY (trail version) (Li, 1999) is used to compute the seismic responses. The analytical results are discussed in the following section.

3.2. Discussions of Analytical Results

The main theme of this study is to investigate the effects of the torsional resistance of the major wall in the Reference Model A. Table 3.1 shows the peak torque of the wall $T_{w,peak}$ developed from the three ground motions. It is evident that the torsional resistance of the wall is mobilized to different extents under various ground motions. The torques of wall under all ground motions exceed the cracking torque and enter into its inelastic range. In particular, under the sine wave, the peak torques reaches 69% of its maximum value. Table 3.1 also shows the peak flexure shear force of the wall $F_{w,peak}$ and the flexure shear capacity of the wall $F_{w,u}$. It is noted that a large portion of the flexure shear capacity can be mobilized under a relatively small PGA in a range of 0.2g.

The effects of the interaction between the shear force and torque of the wall are also of great interest. It is believed that if the torsional capacity of wall is considerably mobilized, the wall may be failed under a relatively small shear or vice versa. Since the peak torque and the peak shear shown in Table 3.1 are not attained simultaneously, it is logical to check the coupled results in the time history. The significance of the interaction effects can be best appreciated by plotting the shear and torque histories superimposed with a torque-shear interaction surface of the wall of the Reference Model A (see Figure 3.6). Based on the author s' best knowledge, the studies on the behaviors of RC wall elements under combined loadings, if any, are very limited. Therefore, a reasonable but simple ultimate strength design Equation (3-1) and (3-2) based on RC beam test results (Klus, 1968) are adopted in this study to determine the interaction surface.

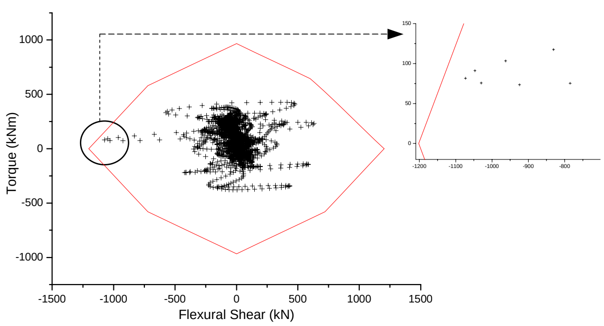
$$\frac{2T_t}{3T_{\max,t}} + \frac{V_u}{V_{\max,u}} \le 1 \quad \text{where} \quad \frac{T_t}{T_{\max,t}} \le 0.6 \tag{3-1}$$

$$\frac{T_t}{T_{\max,t}} + \frac{2V_u}{3V_{\max,u}} \le 1 \quad \text{where} \quad \frac{V_u}{V_{\max,u}} \le 0.6 \tag{3-2}$$

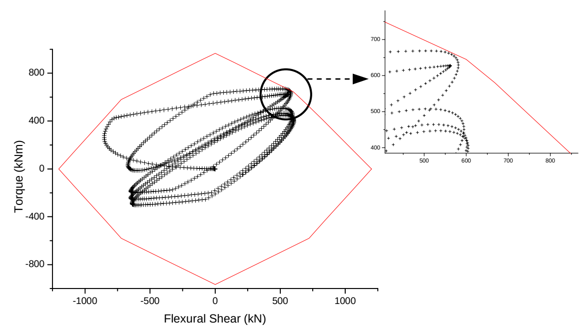
in which T_t =torque applied; $T_{\max,t}$ =torsional capacity; V_u =shear force applied; $V_{\max,u}$ =shear force capacity. The enlargements of the critical time points are also shown in Figure 3.6.

Table 3.1 Peak Response of Reference Model A

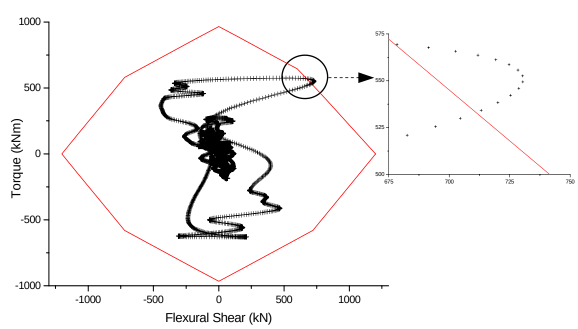
Ground Motions	$T_{w,peak}$ (kNm)	$T_{w,peak}/T_{w,c}$	$T_{w,peak}/T_{w,\max}$	$F_{w,peak}$ (kN)	$F_{w,u}$ (kN)
Sine Wave (T=1.5s)	669	158%	69%	853	1202
1940 El Centro	427	101%	44%	1073	
1979 Imperial Valley	631	150%	65%	731	



(a) 1940 El Centro



(b) Sine Wave (T=1.5s)



(c) 1979 Imperial Valley

Figure 3.6 Shear and Torque Histories of Wall of Reference Model A

As observed from Figure 3.6 (a) and (b), all the shear-torque points of the wall remain inside or close to the interaction surface under the El Centro wave and the sine wave, which indicate that the wall does not fail during the excitation, and the displacement ductility demand of the Columns are less than their capacity. However, for the response histories of Imperial Valley wave (see Figure 3.6 (c)), some shear-torque points are located outside of the failure surface, and these indicate that the wall is severely damaged. It must be noted that once the wall is failed by the combined shear and torque, the analysis should be terminated immediately. However, the dynamic analysis conducted in this paper does not have this termination rule. Nevertheless, the results also indicate that the shear and torque interaction behavior of the wall is very sensitive to the characteristics, in terms of dominant frequency or PGA, of ground motions, and this deserves further examinations. It has to be emphasized that the shear-torque interaction of a structural wall is traditionally not considered in design practice and this may not lead to a conservative design of a single-wall-frame building under seismic attacks.

Table 3.2 Comparison of Peak Responses of Reference Model A and Model A’

Ground Motions	$R_{z,peak}$ (degree)		$F_{w,peak}$ (kN)	
	Reference Model A	Model A’ (Wall torsion neglected)	Reference Model A	Model A’ (Wall torsion neglected)
Sine Wave (T=1.5s)	4.22	5.67	853	808
1940 El Centro	1.15	1.09	1073	1008
1979 Imperial Valley	3.73	3.78	731	794

The torsional responses of the Reference Model A and the Model A’ are also compared. The fundamental period of Model A is found to be 3.15s. By ignoring the torsional stiffness of the wall in the Model A’, its fundamental period becomes 3.49s. Table 3.2 shows the peak slab rotation $R_{z,peak}$ and the peak shear force of the wall $F_{w,\ peak}$ of both models. Under the selected sine wave excitation, the peak shear demand of the wall of the Model A’ slightly reduces by 5% but its peak slab rotation increases by 35% as compared with those of the Reference Model A. Under the case of the El Centro excitations, the peak shear demand of the Model A’ also reduces by 6% as compared with that of the Reference Model A. Although such similar observation is not found under the Imperial Valley motions, it is evident enough to say that the omission of the torsional stiffness/resistance of the wall does not lead to a conservative estimation of the shear demand of the wall in a single-wall-frame building system under seismic attacks. The effect of the torsional stiffness/resistance of the wall will probably reduce the torsional twist of a single-wall-frame building system.

4. CONCLUSIONS

Based on the preliminary analytical study of the seismic responses of one-story single-wall-frame models, the following observations are noted:

1) The torsional resistance of the wall can be substantially mobilized due to the large rotational displacement under ground motions. Simultaneously, the shear resistance of the wall can also be utilized because of the dynamic effect due to the rotary inertia of the mass. As a result, the interaction of shear and torque of the wall should be considered. As shown by the case study presented in this paper, this special structural system may actually fail under the combined shear-torsion loadings, in which the system is considered to be safe when the torsion stiffness/resistance of the wall is neglected.
2) The exclusion of the torsional resistance of the wall does not always lead to a conservative estimation of its

shear demand. In some cases, the consideration of the wall torsion stiffness/resistance considerably reduces the torsional twist of the structural system.

3) Nevertheless, this paper explores uncertainties on the effects of the torsional stiffness/resistance of the wall in a single-wall-frame building system, and further study in this direction is desirable.

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