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A STUDY ON THE EARTHQUAKE RESISTANT BEHAVIOR OF COMPOSITE MASONRY BUILDING SUPPORTED ON FRAME-SHEAR STRUCTURE AT THE FIRST STOREY

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ABSTRACT

The sudden change of lateral stiffness between second and first storey the displacement concentration would occurre at the first storey, which deteriorates the aseismic behavior of masory building supported by frame structue at the first storey. In this paper the rational stiffness ratio of the second to first storey is studied, to avoid deformation concentration for the 8-storey composite masonry wall buildings supported by frame-shear structure at the first storey. A 1 : 4 scale 8-storey model composite wall building had been tested on earthquake simulating shaking table to understand the aseismic performance of such buildings. Dynamic analysis and reliability of a typical 8-storey composite masonry building supported by frame-shear structure at the first storey are conducted. It is concluded that if the reliability index $\beta \geqslant 3$ is to be required, for design purposes the rational stiffness ratio for the second to first storey would be 1. 6 and 1. 2 if the erthquake intensities are 7 and 8 respectively. The percentage of vertical load sustained by the supporting frame beam has also been studied by experimental work and nonlinear FEM. It is shown by the test and analysis that due to the arch action and confined columns of composite wall only about 30% of the vertical load is transmitted to the supporting beam. A 60% of the total vertical load from the upper part of the building, uniformly distributed on the supporting beam, is proposed to design the frame beam.

KEYWORDS

Stiffness ratio; Reliability index; Wall on frame-beam structure; Shaking table test; Dynamic response; Nonlinear FEM; Composite wall.

INTRODUCTION

The multi-storey masonry buildings supported by frame structure at the first storey were severely damaged and collapsed during the strong earthquake, as in Haicheng (Feb. 4, 1975, magnitude 7. 3) and Tang shan earthquakes (July 28, 1976, magnitude 7. 8). It was found that the major causes were the large stiffness difference between the second and first storey and insufficient strengths of the first storey. To improve the aseismic behavior of this kind of buildings, therefore, to enhance the strength and siffness of the first storey and a rational stiffness ratio of the second to first storey should be the most important problem for consideration.

By setting several RC shear walls to the frame would be the simplest method to have the strength and stiffness of the frame structure increased, it is named as the frame-shear structure. The masonry upper part of this kind of buildings composite walls system was proposed (Wu, et al., 1996). It has shown that the 7, 8-storey composite wall masonry buildings supported by frame-shear structure at the first storey were of good aseismic performance and a 10—15% of the cost would be cheaper than the frame structure and about one-third of the construction period would be shortened.

Another important problem is the rational design of the beams for the frame. The beam of the frame would support the load of all seven storey walls above it. However, the loading state of the wall-on-beam has still not been suffieciently studied. These two problems would be briefly discussed in this paper.

SIMULATED EARTHQUAKE SHAKING TABLE TEST

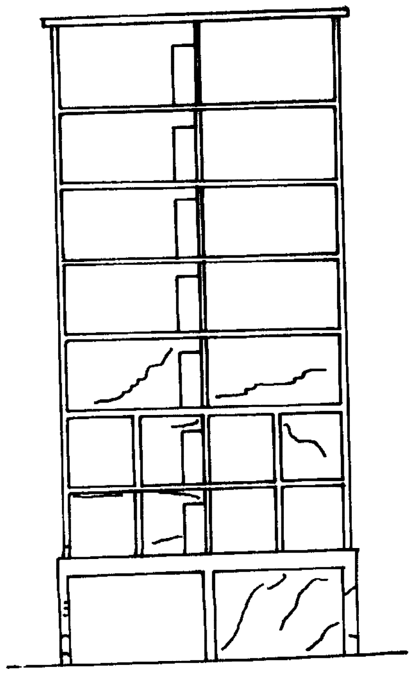
The experimental study of 1 : 4 scale 8-storey composite masonry wall model building on simulated earthquake shaking table was conducted with the aims of evaluation of its aseismic performance. The sketch of the model building is shown in Fig. 1. Its stiffness ratio of the second to first storey is 2.92.

Due to the limited bearing capacity of the shaking table, the applied ballast was made so that the vertical stresses σ_0 in walls and columns were only 50% of σ_0 in prototype. During the elastic stage the Ninghe (1976.11.15 S-N, Type Ⅳ site), Qianan (1976.8.11 S-N, Type Ⅰ site) and El-Centro (1940.5.18 S-N, Type Ⅱ site) recorded accelerograms were inputted and the same intensity of the shaking were controlled by scaling the amplitude of those original records. The test results had shown that the largest response of the model building was in accordance with El-Centro accelerogram, therefore it was used in a sequence of shaking table inputs until the model to be failed. The responses of peak storey drift, storey drift ratio and acceleration were summarized in Table 1.

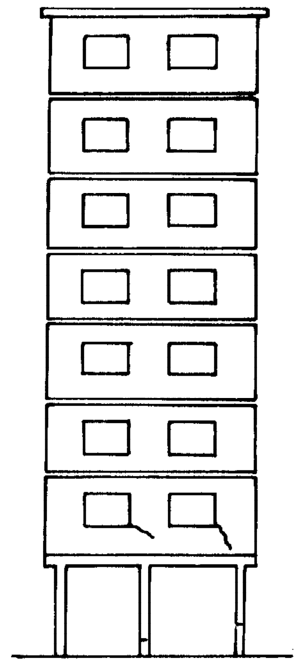
Table 1. Responses of max. storey drift, storey dirift ratio and acceleration

Storey No.	1	2	3	4	5	6	7	8
Peak value of inp. accele.	max. Storey dirft (*mm*)							
0.179*g*	0.22	0.46	0.47	0.54	0.51	0.39	0.41	0.57
0.756*g*	1.66	0.82	0.92	2.32	0.93	1.14	1.10	0.95
1.508*g*	1.87	1.13	1.83	3.21	1.80	19.2	1.96	1.36
	max. Storey drift ratio							
0.179*g*	1/5000	1/1467	1/1436	1/1250	1/1324	1/1730	1/1646	1/1316
0.756*g*	1/663	1/823	1/734	1/291	1/726	1/592	1/614	1/789
1.508*g*	1/588	1/597	1/369	1/210	1/375	1/352	1/344	1/551
	max. aueleration (*g*)							
0.179*g*	0.16	0.18	0.19	0.19	0.17	0.20	0.30	0.37
0.756*g*	0.61	0.67	0.65	0.70	0.70	0.70	1.09	1.36
1.508*g*	1.09	1.26	1.49	0.96	0.78	0.88	1.16	1.68

The inclined crack occurred at the right side on the top of the door opening of the transverse wall on the second storey if the inputting peak acceleration was 0.526*g*. A first inclined crack appeared, if the peak input was 0. 756g in the RC shear wall at the first storey, then the inputting peak acceleration would be increased to 1. 079g, more cracks occurred, the first crack would be prolonged and penetrated in the shear wall. At the same time some horizontal cracks at the ends of three columns on the first storey, and inclined cracks in the walls on the third storey and at the corner of door opening on the fourth storey were observed. Should the peak acceleration be increased up to 1. 508g, then the model building would sustain severe damages. The crack pattern is shown in Fig. 1



a) Transverse direction



b) Longitudinal direction

Fig. 1 Crack pattern at 1. 508g

It had shown that the RC shear wall would resist almost all of the horizontal earthquake action at the first storey, the stress in the steel bar was not large and the maximum stress at the end of the columns was 87MPa. No cracks appeared in the confined beams and columns. The bearing beam was not damaged at all and the maximum stress on the steel bar in the beam was 160MPa. It could be concluded that the model building was severely damaged but not collapsed.

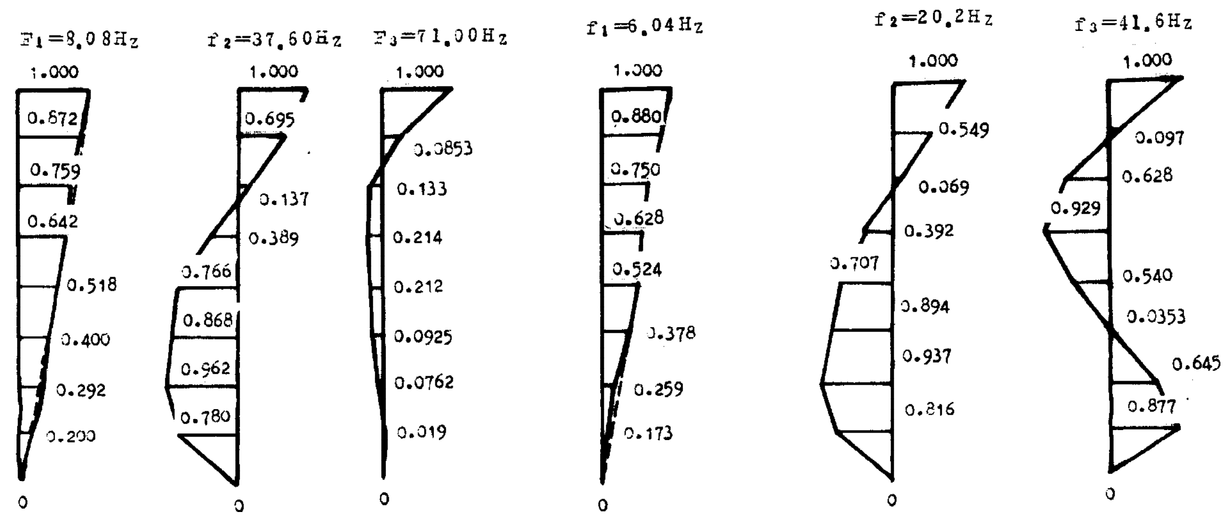
The natural frequencies, damping ratios and the three order mode shapes were measured after several shakings, the natural frequencies were listed in Table 2.

Table 2. The first three natural frequencies (Hz)

Peak accel.	Transwerse			Longitudinal		
	f_1	f_2	f_3	f_1	f_2	f_3
0. 1798g	8. 08	37. 6	71. 0	6. 04	20. 2	41. 6
0. 705g	6. 92	37. 2	67. 2	4. 96	17. 8	37. 0
1. 508g	6. 36	29. 8	50. 8	4. 52	15. 6	32. 6

Table 2 shown that the first natural frequencies were to be a reduction of 14% (transverse) and 18% (longitudinal) respectively than the initial frequencies, if the peak acceleration was increased from 0. 179g to 0. 756g and the RC shear wall would eventually be cracked at this time. When the model building was severly damaged at peak acceleration of 1. 508g a reduction of 21% of the fundamental frequencies would have recorded than the initial ones both at the transverse and longitudinal directions. It is known, the frequency of a structure is related to its stiffness, that is to say the model building was far from the collapsed state, since, in general, a reduction of about 40—50% of the frequency is considered to be failure state.

The first three order mode shapes are shown in Fig. 2, The view of the first order mode shape is more or less like a shear-bending mode but primarily dominated by shear deformation.



a) Transverse direction

b) Longitudinal direction

Fig. 2 First three order natural mode shapes in initial stage.

Owing to the less ballast up to 90% during the shaking table test, it is difficult to evalute directly of earthquake resistance behavior quantitatively. Hence a dynamic analysis is made for the model building in the case of half σ_0 for verifying the precision of the selected physical parameters by the response of the model building. Therefore, the model building would be appropriately analyzed at a full value of σ_0 to evaluate its aseismic behavior.

A primary study on the analogy between scale models with less ballast and their prototypes under shaking table test had been carried out (Huang et al. , 1944), and about 50% of the value for the model building was taken for the peak acceleration of the prototype. This has been approved by dynamic analysis.

EVALUATION OF ASEISMIC BEHAVIOR FOR A 8-STOREY MASONRY BUILDING SUPPORTED ON FRAME-SHEAR STRUCTURE AT FIRST STOREY

Because of the said building is quite difference from the model ones, it is neccessary to evaluate the typical building by a dynamic analysis and the lateral stiffness ratio of the second to first storey is found to be 1. 41.

The peak storey drift is used as the failure criterion in the time history analysis and the accelerogram of El-Centro was inputted. The results are listed in Table 3.

Table 3. Max. Storey drift responses (*mm*)

	1	2	3	4	5	6	7	8
0. 22*g*	1. 65	1. 47	1. 67	2. 30	2. 87	2. 81	2. 50	2. 08
0. 40*g*	3. 47	2. 46	2. 83	3. 88	5. 73	4. 78	4. 15	2. 46
0. 62*g*	6. 89	4. 83	5. 01	6. 50	8. 47	6. 62	4. 62	2. 85
Cracking disp.	2. 26	1. 81	1. 84	2. 52	2. 83	2. 76	2. 64	2. 60
Yield disp.	6. 75	2. 26	2. 30	3. 15	3. 54	3. 45	3. 30	3. 26
Limit disp.	49. 06	5. 37	6. 26	8. 82	10. 91	11. 83	12. 65	13. 40

With the exception of the 5th storey, all the storey drift for each storey did not reach to their cracking displacement when the inputting peak acceleration was of 0. 22*g*. It is to say that this 8-storey building would safely stand at seismic zones with an intensity of 7. When the inputting peak acceleration was increased to 0. 4*g*, all of the walls were cracked and were beyond the yield displacement except the RC shear wall of the first storey; if the peak acceleration reached to 0. 62*g*, all of the walls were beyond the yield displacement, but still far from the ultimate displacement for each storey, then the maximum storey drift ratio at the 5th storey was of 1/319.

PROPOSED LATERAL STIFFNESS RATIO

As mentioned above, this kind of buildings would show good aseismic performance, if the stiffness ratio is 1. 41. To find an appropriate stiffness ratio $k_{\mathrm{I}}/k_{\mathrm{I}}$ the thickness or length of RC shear wall are to be varied in order to get a group of stiffness ratios of 1. 73, 2. 27, 2. 74 and 3. 3. The first Order-Second Moment Method is used to calculate the reliability index β (Wu, et al. , 1981), from which the most appropriate stiffness ratio $k_{\mathrm{I}}/k_{\mathrm{I}}$ could be designated.

The frequencies of 20 recorded accelerograms were adjusted to 2. 2—3. 3*Hz*, close to the fundamental frequency (3. 2*Hz*) of the typical building to obtain greater response. The value of reliability index β at 0. 22 *g* and 0. 4*g* inputting peak accelerations were listed in Tables 4 and 5 respectively.

Table 4. Values of Reliability Index β at 0. 22*g*

No. of storey	$k_{\mathrm{I}}/k_{\mathrm{I}}$	1. 41	1. 73	2. 27	2. 74	3. 33
1st storey		9	9	9	4. 99	4. 20
2—8th storey		9	9	9	9	9

Table 5. Values of reliability index β at 0. 4*g*

$k_{\mathrm{I}}/k_{\mathrm{I}}$	No. of storey	1	2	3	4	5	6	7	8
1. 41		4. 62	2. 57	3. 07	3. 98	3. 54	4. 79	9	9
1. 73		1. 87	1. 22	2. 86	3. 47	3. 76	4. 41	5. 29	9

It is quite obvious to see from Table 4 that β is greater than 4 when the ratio of k_{I}/k_{I} is smaller than 3, and from Table 5 the minimum value β is 2. 57 for the second storey if k_{I}/k_{I} is equal to 1. 41. Conclusion would be drawn that the maximum stiffness ratio of 3 and 1. 4 for k_{I}/k_{I} would be required for the intensities of 7 and 8 respectively. For design purposes, a maximum stiffness ratios of 1. 6 and 1. 2 for intensities of 7 and 8 respectively could be quite appropriate in considerations of other requirements.

STUDY ON THE BEARING STATE OF SUPPORTING BEAM

The bearing beam would sustain the load of upper seven storeys and be tightly combined with the wall. From the mechanics point of view it is a deep beam or a wall-on -beam. If the composite wall system is used for the upper part of the building, the brick masonry and particularly the confined columns which sustain part of the vertical loading would be constructed on te beam. Taking account of the arch action of the wall and the confined columns on the supporting beam, only a part of the vertical load actually exists on the beam.

Experimental Study of the Supporting Beam

Four pieces of transverse model walls and two pieces of longitudinal model in a scale of 1 : 2 with 4 storeys in height were constructed in the following patterns:

A. No opening in the upper part and with frame supporting at the first storey;
B. One door opening in the middle part of the wall and with frame supporting at the first storey;
C. and D. with and without opening in the wall and with frame supporting at the first two storeys;
E. and F. with window opening at the wall and with frame supporting at the first storey and first two storeys.

All of the vertical load for transverse and longitudinal walls of a 8-storey building were loaded on the top of the 4-storey model wall to simulate the vertical stresses in the prototype walls. The scheme of the transverse model walls in case of C and D are shown in Fig. 3. The horizontal forces were acted on the ends of each confined beam. The percentage of vertical load being sustained by masonry (including the portion of vertical load sustained by two middle-side columns) was transmitted to the beam are summarized in Table 6.

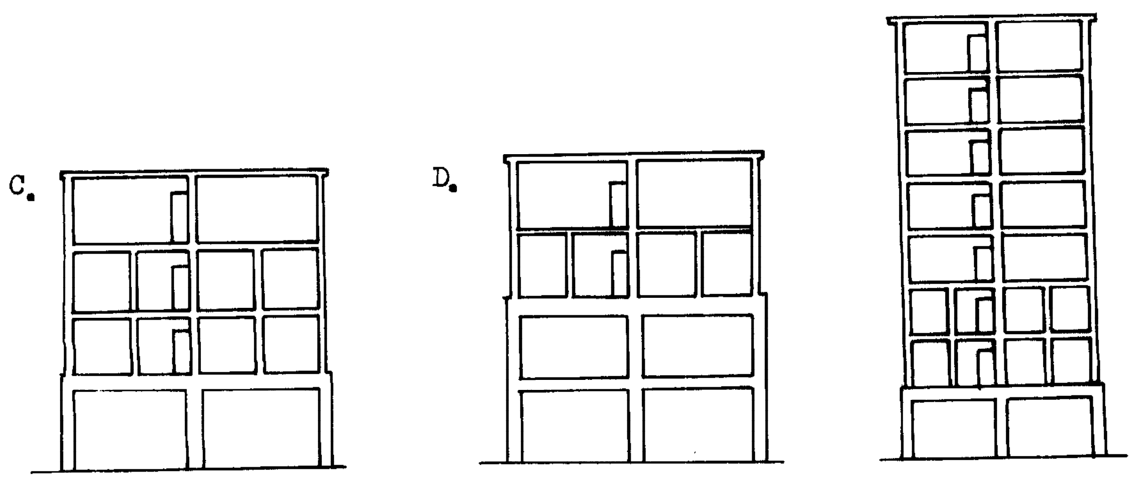


Fig. 3 Sketch of the model walls C and D and wall of 8-storey building

It is shown by Table 6 that the vertical load sustained by supporting beam through masonry and two middle-side columns are approximately 30% and about 70% of the total vertical loads to be directly transmitted to the frame columns.

Table 6. Comparison of test results and FEM

Hori. load (KN)	A (%)		B (%)		C (%)		D (%)		E (%)		F (%)	
	Test	FEM	Test	FEM	Test	FEM	Test	FEM	Test	FEM	Test	FEM
0	24.0	34.4	32.6	36.0	32.4	35.4		34.0	22.5	27.9	21.4	26.2
25		33.0		33.5		34.2		33.2				
50		33.5		33.9		33.5		32.8				
100	26.0	34.3	33.5	34.2	29.3							
65							30.5	32.6				
20											19.5	26.3
22									23.2	28.0		
40											18.9	26.4
44									22.7	28.1		

Loading State of Frame Beam by FEM

The 4-storey and 8-storey (Fig. 6) composite masonry walls supporting at first storey or first two storeys were treated as a plane stress problem by nonlinear FEM (Wu et al., 1979). The calculated results are listed in Table 6 and 7. It could be seen that the comparison is of good agreement and the calculated values are somewhat greater than those by tests. From Table 7 it is seen that the arch action of the wall above the third storey is small and the confined columns which intended to bear the vertical loads with masonry in the wall, a part of vertical load was to be transmitted through the confined columns to the frame columns. But the arch action of the wall on the second or third storey, which would be combined with the supporting beem, are obviously shown. The load transmitted through masonry and middle-side columns to the beam was 29% only.

Table 7. Distribution of vertical load between confined columns and Masonry (%)

No. of Storey	Left end column	Masonry	Middle column	Masonry	Right end column	$\sum$ Masonry
8	14.4	27.3	16.6	27.3	14.4	54.6
7	16.1	25.6	16.6	25.6	16.1	51.2
6	16.4	25.3	16.6	25.3	16.4	50.6
5	17.4	24.3	16.6	24.3	17.4	48.6
4	19.2	21.4	18.8	21.4	19.2	42.8
3	20.4	20.3	22.2	14.5	24.4	29.0

The distribution of vertical load is influenced by many factors, such as the stiffness of the wall and the beam etc. , the more stiffer the beam is, the more vertical load would be sustained by it. The horizontal earthquake action would exert influences on the distribution of the vertical loads, but the analysis shown that only a minor influence would have occurred on the total distributed percentage between the columns and masonry, if the horizontal earthquake action would not exceed the intensity of 9.

The influence of the openings on the wall would produce a concentrated shear force to the beam and should be considered in the design. The existence of distributed shear forces along the top of the beam will develop moment, shear and axial forces if the horizontal earthquake action is to be accounted for.

A simplified Calculating Method for Supporting Beam

A simplified method for calculating the internal forces of the supporting beam is suggested for design purpose. That is a 60% of the total vertical load should be uniformly distributed on the frame beam, the moment, shear and axial forces of the beam in the frame produced and would cover all respective forces from all involving factors.

This proposal had been accepted in design, in which the cross section of the beam had been reduced from 900×400*mm* to 650×350*mm* and several hundred thousands square meters of this kind of buildings were constructed in China.

CONCLUSIONS

Good aseismic behavior of the 8-storey composite masonry wall buildings supporting on frame-shear struture at the first storey had been shown, if the stiffness ratio of the second to first storey is controlled. The appropriate stiffness ratios k_{II}/k_{I} for intensities 7 and 8 respectively are of 1. 6 and 1. 2 are proposed for design purpose.

The base shear method may be used to calculate the earthquake shear forces since the shear-bending mode but primarily dominated by shear deformation.

Only about 30% of the vertical load directly acting on the supporting beam and 70% of the vertical load transmitted to the frame columns through the confined columns and the arch action of walls. A simplified method by assuming a 60% of the total vertical load, uniformly distributed on the supporting beam, is proposed for designing the frame leam.

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