SHAKE TABLE TESTS OF A TWO-STOREY LIGHT WOOD FRAMED HOUSE

Cheng Haijiang[1] Lu Xilin[2] Ni Chun[3] Lu Wensheng[2] Zhu Chunming[4]

[1]*PhD, Shanghai Research Institute of building Sciences & Tongji University, Shanghai, China*
[2]*Professor, Tongji University, Shanghai, China*
[3]*Scientist, Forintek Canada Corp., Vancouver B.C., Canada V6T1W5*
[4] *Professor, Shanghai Research Institute of building Sciences, Shanghai,China*
Email:chenghj@gmail.com, lxlst@mail.tongji.edu.cn

ABSTRACT :

Abstract: Light wood framed house showed good seismic performance in past earthquakes, and it also had some shortage. The specimen for shake table tests is a full size two-storey light wood framed house, 6.0m by 6.0m in plan, and 6.3m in height. Symmetrical configurations with three types opening size and asymmetrical configurations were tested. The test results showed that specimen performed elastic and had no visual damage during the tests of 0.2g PGA. The tested symmetric building with progressively lager opening can withstand successive application of three different seismic ground motions in the order of 0.55g PGA without collapse. For asymmetrical building, the response of acceleration and displacement of specimen were larger than symmetrical building. Interstorey drift in the first story was generally bellow 1/500, 1/250 and 1/80 at nominal levels of 0.1g, 0.2g and 0.4g peak table acceleration. The light wood framed house specimen can meet the requirements for seismic intensity 8 described in China code for Seismic Design of building (GB 50011-2001).

KEYWORDS: Light wood framed structures, Seismic, Shake table test

1. INTRODUCTION

Light Wood-framed construction is by far the most common structural type in North America for single-family houses and low-rise multifamily dwellings. This type of structure is constituted of shear walls, wood floor and wood roof, and is suitable for three or less than three storey houses. Wood structures have been also used in the building history of China for a long time, but most of them are post-beam style. In the latest Chinese "Code for Design of Timber Structures (GB50005-2003)", the light wood-frame house structure was firstly permitted to apply in China. While there exists a relatively large body technical information for the seismic behavior of light wood framed house, many structural engineers and researchers believe that current codes are based on individual component and do not reflect the true behavior of a wood-frame structure in system level.

With the above background, a shake table test of full size two-story wood-frame house has been developed to understand the seismic performance of the structure in state key laboratory for disaster reduction in civil engineering of Tongji University. This project has been sponsored by Canada Wood and Canada Forestry Innovation Investment and is a joint research between Tongji University and Forintek Canada Corp.

2. DESCRIPTION OF TEST SPECIMENS

The test specimen was built in accordance with the prescriptive requirements of the National Building Code of Canada [NBCC, 1995]. The two-story wood frame with studs of 38 × 89 mm No. 2 and better grade SPF lumber spaced at 406 mm (16 in.) on center was sheathed on the outside with 9.5 mm (3/8 in.) oriented strand board (OSB) and finished on the inside with 12.7 mm (1/2 in.) gypsum wall board (GWB), taped and grouted. The interior partition was finished with GWB on both faces. The OSB sheathing was fastened with 65 mm galvanized twisted nails of 3.2 mm diameter, spaced at 150 mm along the perimeter of the sheathing panels and 300 mm elsewhere. The GWB was attached with 3.2 mm diameter screws, 28 mm long at 200 mm spacing. The floor consisted of JSI-20 wood I-joists (241 mm in depth) spaced at 406 mm on center and 19 mm (3/4 in.) T&G OSB as floor sheathing. The roof consisted of pre-engineered trusses spaced at 600 mm on centre, sheathed with 11 mm plywood. Anchor bolts of 12.7 mm diameter at a nominal spacing of 1220 mm fastened the sill plate to the steel grillage.

Figure 1 Specimen of Shake table test

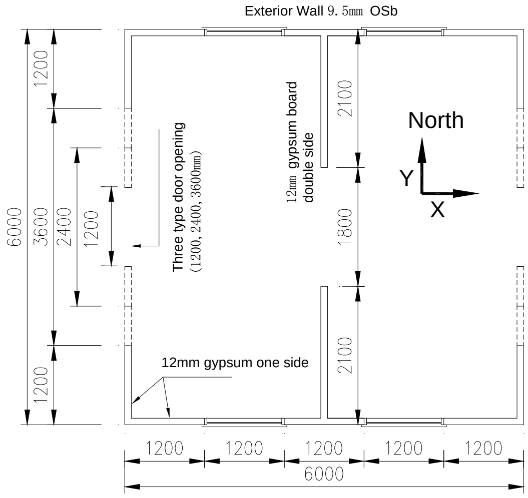

Figure 2 Main floor plan

Fifty percent of design live load was added to floor and roof. With the assumption of 2.0 kPa for residential occupancy and 0.5 kPa for roof loading, a total weight of 6000 kg and 1600 kg were added to the floor and roof, respectively, to simulate a building with a plan dimension of 10 × 6 m. See Figure 2.

Table 1 Main characteristics of five test phases

Phase	Shake Direction	Description	Braced wall length per building side
1	Y	1.2m door width	4.8m (80%)
2	Y	2.4m1.2m door width	3.6m (60%)
3	Y	3.6m1.2m door width	2.4m (40%)
4	X	2.4m1.2m window width	3.6m (60%)
5	Y	1.2m door on East wall, all OSB was removed on west walls.	--

Five phases representing increasing sizes of openings were investigated (Table 1). Three different earthquakes (Pasadena, El Centro, and an artificially generated Shanghai earthquake record) were applied in three progressively larger steps of shaking intensity, from 0.1 g, 0.2 g and 0.4 g peak ground acceleration (PGA). For some tests, additional runs with peak acceleration levels of 0.55g were applied. The sequence of test runs and some main test results of shake table tests are presented in Table 2.

Table 2 Test sequence and main test result.

Phase	Test	Earthquake	Shake Direction	Design PGA[r] (g)	Actual PGA[a] (g)	Shear weight ratio	1st Floor Drift mm	Mode 1 Freq. (Hz)	Mode 1 Damping ratio
1	S1	WN	X	0.07	0.04			4.20	0.049
			Y	0.07	0.10			3.91	0.049
	S2	WN	X	0.07	0.07			4.25	0.048
			Y	0.07	0.09			4.20	0.043
	S3	WN	X	0.07	0.07			4.39	0.050
			Y	0.07	0.09			4.59	0.034
	1	WN	Y	0.07	0.10			4.44	0.051
	3	PA	Y	0.10	0.10	0.14	1.48		
	5	WN	Y	0.07	0.09			4.44	0.053
	7	PA	Y	0.20	0.21	0.26	2.8		
	9	WN	Y	0.07	0.09			4.44	0.054

Phase	Test	Earthquake	Shake Direction	Design PGA^{r} (g)	Actual PGA^{a} (g)	Shear weight ratio	1st Floor Drift mm	Mode 1 Freq. (Hz)	Mode 1 Damping ratio
1	11	PA	Y	0.40	0.49	0.58	6.88		
2	13	WN	Y	0.07	0.09			4.25	0.064
	14	WN	Y	0.07	0.09			4.10	0.054
	15	PA	Y	0.10	0.10	0.13	2.09		
	16	EL	Y	0.10	0.10	0.19	2.33		
	17	SH	Y	0.10	0.08	0.17	1.81		
	18	WN	Y	0.07	0.09			4.10	0.055
	19	PA	Y	0.20	0.25	0.26	3.68		
	20	EL	Y	0.20	0.20	0.35	4.73		
	21	SH	Y	0.20	0.24	0.43	6.93		
	22	WN	Y	0.07	0.09			3.91	0.077
	23	PA	Y	0.40	0.44	0.56	9.16		
	24	EL	Y	0.40	0.37	0.61	10.25		
	25	SH	Y	0.40	0.38	0.67	13.84		
	26	WN	Y	0.07	0.09			3.56	0.104
3	27	WN	Y	0.07	0.09			3.66	0.073
	28	PA	Y	0.10	0.11	0.19	3.25		
	29	EL	Y	0.10	0.10	0.19	2.96		
	30	SH	Y	0.10	0.08	0.20	3.12		
	31	WN	Y	0.07	0.09			3.56	0.078
	32	PA	Y	0.20	0.22	0.34	5.68		
	33	EL	Y	0.20	0.20	0.32	5.45		
	34	SH	Y	0.20	0.19	0.38	7.87		
	35	WN	Y	0.07	0.09			3.32	0.094
	36	PA	Y	0.40	0.63	0.56	13.32		
	37	EL	Y	0.40	0.39	0.60	15.05		
	38	SH	Y	0.40	0.44	0.76	26.21		
	39	WN	Y	0.07	0.10			2.44	0.119
	36a	PA	Y	0.55	0.40	0.76	41.77		
	37a	EL	Y	0.55	0.59	0.91	75.39		
	39a	WN	Y	0.07	0.09			1.46	0.211
4	40	WN	X	0.07	0.09			2.54	0.076
	41	PA	X	0.10	0.10	0.10	1.45		
	42	EL	X	0.10	0.11	0.15	2.45		
	43	SH	X	0.10	0.08	0.16	2.36		
	44	WN	X	0.07	0.10			3.66	0.077
	45	PA	X	0.20	0.18	0.21	3.15		
	46	EL	X	0.20	0.20	0.28	4.95		
	47	SH	X	0.20	0.16	0.25	5.11		
	48	WN	X	0.07	0.09			3.56	0.077
	49	PA	X	0.40	0.37	0.43	10.43		
	50	EL	X	0.40	0.37	0.59	17.44		
	51	SH	X	0.40	0.36	0.58	18.12		
	52	WN	X	0.07	0.10			3.13	0.119
	53	PA	X	0.55	0.48	0.66	23.11		
	54	EL	X	0.55	0.56	0.86	36.36		
	55	SH	X	0.55	0.50	0.72	34.04		
	56	WN	X	0.07	0.10			2.39	0.176
5	57	WN	Y	0.07	0.11			2.10	0.129
	58	PA	Y	0.10	0.10		5.06		
	59	EL	Y	0.10	0.10		7.05		
	60	SH	Y	0.10	0.09		6.15		
	61	WN	Y	0.07	0.12			1.90	0.126

Phase	Test	Earthquake	Shake Direction	Design PGA[r] (g)	Actual PGA[a] (g)	Shear weight ratio	1[st] Floor Drift mm	Mode 1 Freq. (Hz)	Mode 1 Damping ratio
5	62	PA	Y	0.20	0.22		34.22		
	63	EL	Y	0.20	0.21		20.16		
	64	SH	Y	0.20	0.20		20.11		
	65	WN	Y	0.07	0.11			1.81	0.163
	66	PA	Y	0.40	0.45		65.28		
	67	WN	Y	0.07	0.10			1.37	0.174

Note：1. PA--Pasadena, WN--White noise, EL--El Centro, 2. Floor drift of 1[st] floor in Phase 5 was calculated from the data of east wall

3. OVERVIEW OF DAMAGE

After each set of seismic table motions at nominal peak values of 0.1, 0.2, 0.4 and some at around 0.55 g the specimen was inspected and any damage recorded. The main damage in five phases was as following:

Phase 1~3: (1) At level 0.1g, no visual failure was observed. (2) At level 0.2g, several nails were pulled out. (3) At level 0.4g, some nails were pulled out along the wall panels, with local crushing failure which were mainly distributed at the top of first floor, and with wide range of OSB board failure. (4) At level 0.55g of Phase 3, the response of the structure was serious, and with large displacement response and overturning trends. Fatigue failure occurred on the nail joints along the first floor shear walls, with serious damage at the joint parts of OSB walls and first floor bottom. It seems that the damage level has less relation with the width of the opening.

Phase 4: (1) At level 0.1g、0.2g and 0.4g , no serious damage was observed. (2) At level 0.55g, no visual damage was found in shear wall, but serious damage appeared on gypsum board (GWB) in first floor, almost one third of screws on bottom of OSB panel were pulled out. (3) The damage level at level 0.55g in this phase was light than that of Phase three.

Phase 5: (1) At level 0.1g、 0.2g and 0.4g , no serious damage was observed . (2) At level 0.45g, the damage were serious than that in previous four phases, distinct torsion appeared for the asymmetric configuration. Most of the nailed joints along the edge of east elevation of first floor were pulled out or chipped out.

4. TEST RESULT

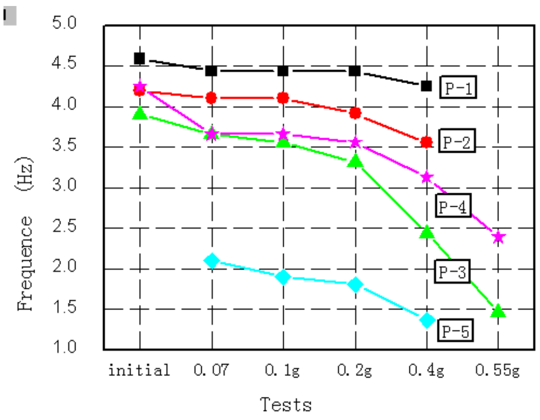

Figure 3 Natural frequency on Y direction

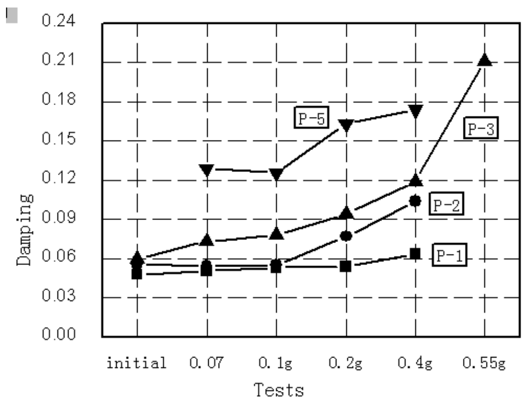

Figure 4 Damping ratio on Y direction

4.1. Nature frequency

The dynamic characteristics such as nature frequency, damping ratio and mode shapes and so on were calculated by mode analysis on white noise tests. Bidirectional white noise test s1, s2, and s3 were carried out in erection stage of shake specimen, and the dynamic characteristics in the three tests are assumed to be the original status of Phase 3, Phase 2 and Phase 1 respectively because of no damage accumulation. While the plan dimension and lateral stiffness in x direction and y direction are very familiar, so the original natural frequency in x and y direction was close. The natural frequency of first order mode shape is between 3.9 to 4.6 Hz, and 11.4 to 3.6 Hz of second order of mode shape.The natural frequency of each white noise test was listed in Table 2 and the changes of natural frequency on Y direction was plotted in Figure 3.

Figure 3 shows that nature frequency goes down with earthquake intensity in the same phase. Because structure stiffness is in proportion with the square of frequency, the changes of natural frequency also indicate the changes of stiffness structure. In Phase 1, the natural frequency has no changes after 0.1g and 0.2g test, which indicates that the stiffness keep unchanged, and still in elastic status; with little reduction after the 0.4g test. In Phase 2 and 3, specimen structure showed nonlinear response with the large reduction of natural frequency caused by the more intensified earthquake input. It can be concluded that with the wider opening in the first floor, the stiffness of structure was cut down greatly.

Figure 3 also shows that the natural frequency of X direction in Phase 4 is less than that in Y direction. In Phase 4 and 2, the effective wall width and natural frequency in erection stage are both close. The natural frequency in Phase 4 is less than that of Phase 2 is because of the influence of continuous test before test four. Although there is no much obvious damage in X direction in Phase 4, but the frequency in X direction was still cut down slightly because of the loosing of nailed joints and so on.

In Phase 5, the specimen stiffness of first floor in Y direction was cut down greatly, specimen configuration was asymmetry, so the natural frequency in Y direction is just half of that in previous four phases. With the increasing values of peak table acceleration, the reduction amount of natural frequency is much higher than that of the previous phase; the specimen stiffness was more lower than that that of the previous phase.

4.2. Damping ratio

The damping ratio was is associated to the natural frequency and was calculated using half-power method by transfer function. Damping ratio reflected the properties of energy dissipation of the specimen structure. The damping ratio on Y direction of specimen in different test phases, as shown in Figure 4, is in the range from 5% to 6% after 0.1g level tests in the previous four phases, and 5%～9% after 0.2g level, 7%～12% after 0.4g level, 18～21% after 0.55g level. In the same phase, the damping ratio grown up with the amplitude of table input earthquake, this is mainly because cumulated damage increase the ability of energy dissipation of specimen. After strong earthquake tests, the specimen changed into elastic-plastic status. In Phase5, with strong torsion response and damage accumulation caused by the previous four phases, the damping ratio was improved one time in comparison with the previous four phases.

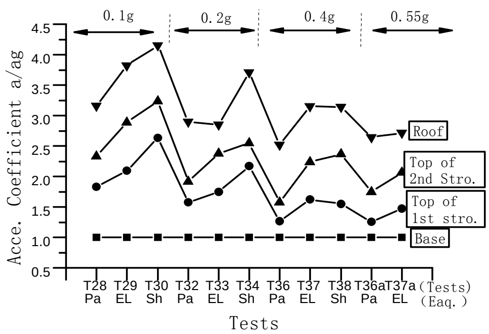

Figure 5 Acceleration coefficient in Phase 3

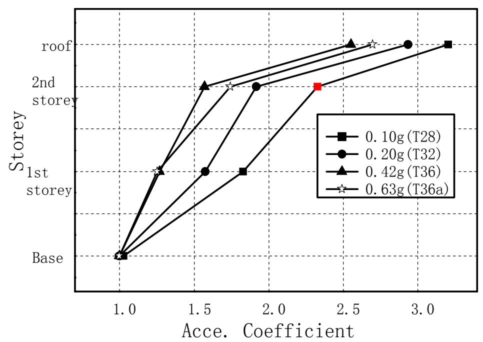

Figure 6 Acceleration coefficient of Pasadena record tests in Phase 3

4.3. Acceleration response of specimen

The acceleration response of specimen is mainly effected by the spectrum character, natural frequency and damping ratio of whole specimen. Figure 5 shows the coefficient of acceleration divided by base input acceleration. Acceleration coefficient in the same stage improved with the height of sensors, with range of 1.25 to 2.63 at the top of first floor, and 1.57 to 3.24 at the top of second floor, and 2.52 to 4.15 at roof. From Figure 5 as far as the three types of earthquake input as concerned, it can be concluded that the order of the acceleration coefficient is SHW2 > El Centrol record> Pasadena record at the same location of sensors .

Figure 6 shows the comparison of acceleration coefficient in Pasadena record test of Phase 3, from which it can be concluded that increment of acceleration coefficient declined with the increase of amplitude of table input. While when it arrive at the 0.63g, the coefficient at the roof and top of the second floor is larger that that of 0.42g stage. Based on the response spectrum theory, the relation of acceleration coefficient and structure period is nonlinear and multi-peak value. With the increasing of the intensity of table input, the stiffness of specimen structure and acceleration response will decrease, and damping ratio will increase. The natural period increased with the failure process. In some extend, such as the 0.63 level tests in Phase 3, the acceleration amplitude coefficient increased with the improvement of structure period

The former discussion is based on the acceleration response in Phase 3 tests, while similar rules can be found in Phase 1, 2 and 4 tests. Figure 6 shows that the acceleration at the top of specimen structure increased greatly. In Phase 4, shake direction is parallel with the roof truss direction, and the acceleration amplitude coefficient at roof is very near that at the top of the second floor because of the higher lateral stiffness of roof. In Phase 5, there exists great difference in acceleration because of the torsion of specimen.

4.4. Displacement response

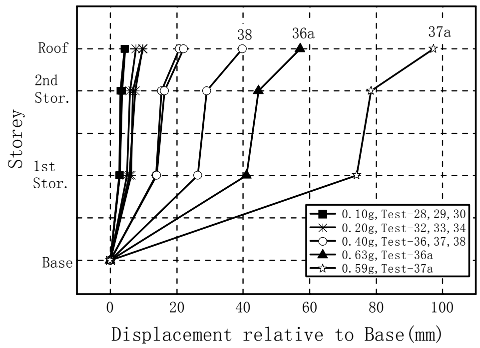

Figure 7 Max. Disp. relative to base in Phase 3

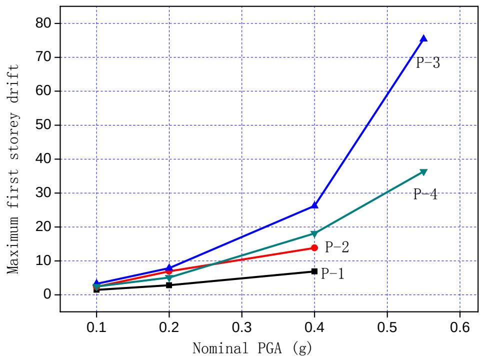

Figure 8 Maximum first storey drift in Phase 1~4

Figure 7 shows the maximum displacement relative to shake table in Phase 3. From which we can see that with the increasing of intensity of table input, the first storey drift is larger than that of the second storey and roof. At Phase 3, the first storey drift increased from 3.3mm at 0.1g level tests to 75.4mm at 0.59g level tests, second storey and roof from 1.7mm to 19.5mm. The first storey drift is more larger because the lateral stiffness of first storey is less than that of second storey, and the base shear force of first storey is also larger than that of second storey. Figure 7 also shows that first storey drift is increased with the order of first storey, roof and second storey.

Figure 8 shows the maximum first storey drift in Phase 1 to 4, from which we can see the first storey drift increase with the increasing of door opening. The door opening at Phase 4 is similar with that of Phase 2, so does the first storey drift. Damage accumulation is also a important reason to the increasing of storey drift from Phase 1 to 3. When the normal PGA of table input is 0.1g、0.2g、0.4g and 0.55g, the maximum first storey drift is 5、10、30 and 80mm respectively, and the first drift ratio was bellow 1/500, 1/250, 1/80 and 1/30 respectively.

4.5. Base shear

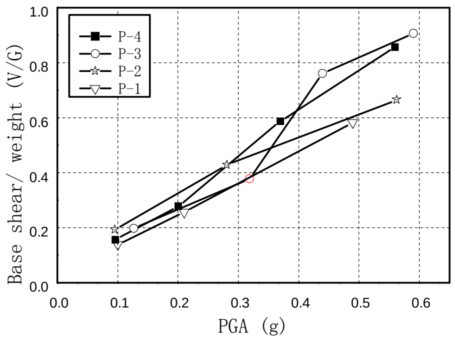

Figure 9 Max. base shear / total weight in Phase 1~4

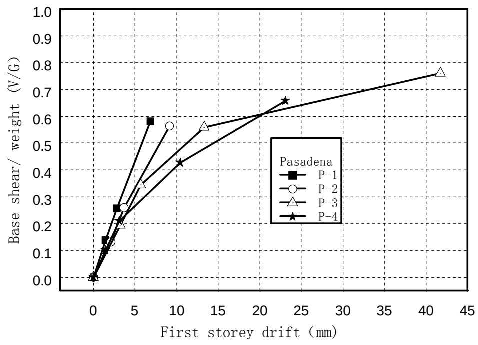

Figure 10 Capacity spectra of Pasa.earthquake tests in Phases 1~4

Figure 9 shows the maximum ratio of base shear to the total weight of specimen in Phase 1 to 4. This figure indicates that the maximum of base shear ratio is in linear relationship with the actual PGA of table input, it also indicated that the stiffness change to some extend has little influence on the maximum base shear ratio. In Phase 3, when actual PGA reaches to 0.3g, the base shear ration increases greatly, that is because of the structure was damaged into elastic-plastic status which caused the reduction of acceleration response. Figure 9 shows that the base shear ratio has no direct relationship with the door opening or the lateral stiffness at 0.1g and 0.2g level tests, while has linear relationship at 0.3g or above level tests.
Form Figure 9, the following rule can be achieved:

$$\gamma_{bs} = PGA + (0.05 \sim 0.01) \qquad (PGA = 0.1 \sim 0.2)$$
$$\gamma_{bs} = PGA + (0.10 \sim 0.15) \qquad (PGA = 0.1 \sim 0.2)$$
$$\gamma_{bs} = PGA + (0.20) \qquad (PGA \geq 0.4)$$

Where γ_{bs} is the base shear ratio, and PGA is number with g as its unit.

4.6. Capacity Spectra

For the first story of the specimen, capacity spectra in the form of plots of maximum base shear ratio against maximum first storey drift are shown in Figure 10. The slop of capacity indicates the lateral stiffness of whole structure. Because the shake tests were continue, the capacity spectrum was also influenced by damage accumulation.

For the Pasadena record the capacity spectrum for Phase 1, the 1.2 m wall opening, is very nearly linear, whereas for Phase 2, with the 2.4 m opening, a lower initial slope and greater softening behaviour at 0.4 g PGA is evident. For Phase 3, initially a further reduction in slope occurs up to 0.2 g PGA, then a more significant reduction at 0.4g. For the 0.63 g PGA, a significant reduction in slope occurs for a maximum first story displacement of 43 mm or story drift of 1.7 %.
In Y direction, Phase 4, the capacity spectrum is nearly linear with a slope that up to 0.2 PGA is comparable to that of Phase 2, but then the slope reduces substantially to 0.4 g and further to 0.55 g PGA as the specimen weakens.

4.7. Seismic Evaluations of specimen

In China code for seismic design of building (GB 50011), the base objective of seismic design is keeping the whole structure no damage in small earthquake, repairable in medium earthquake and no-collapsing in large earthquake. The former discuss indicates that at the 0.1g level tests, the model has no visual damage, and kept in elastic, the maximum first storey drift ratio is less than 1/500, which meets the requirements of small earthquake;

at 0.2g and 0.4g level tests, the specimen run into nonlinear status, damages were slight and focused mainly on nonstructural elements, and which meets the requirements of medium earthquake; at 0.55g levels or above tests, although there exists serious damages, the structure still keeps erect and without collapsing, the maximum first storey drift was 1/30, which meets the requirements of large earthquake. In GB 50011, the maximum storey drift ratio of small and large earthquake for steel structure is 1/300 and 1/30, and has no corresponding limit for wood structure. Taking consideration of the good seismic performance of the wood structure specimen in this shake table tests, the maximum storey drift ratio of small and large earthquake for light wood framed structure were suggested as 1/250 and 1/30 respectively.

One point is that this model analysis only takes consideration of the influence of OSB boards and gypsum board, while the actual wood structure is built with outer wall decorative materials, insulation and thermal protection materials, which proved that strengthened the lateral stiffness of structure.

The above discusses indicated that light wood-framed house specimen can meet the requirements for seismic intensity 8 described in GB 50011.

5. CONCLUSIONS

The following conclusions were obtained from this study:

1) At tests that below 0.2g levels, the specimen had no visual failure and kept in elastic , and keeps in elastic status; at 0.4g level tests, the failure of specimen damages were slight and focused mainly on nonstructural elements; at 0.55g level tests, the specimen had not reached to collapse with continual earthquake tests and 60% door opening.
2) The initial natural frequency was in the range of 3.9 to 4.6Hz. The original damping ratio is in the range of 5～6%. With the damage accumulation, the damping ratio increased slowly, and arrived at 18 to 21% after 0.55g level tests. In Phase 5, the specimen shown distinct torsion because of the asymmetric configuration, and the natural frequency in Y direction is half of that of previous four phases. The damping ratio in Phase 5 is two times of that of previous ones.
3) The acceleration amplitude coefficient increased with the height of location of sensors. After continue shake tests, the acceleration amplitude coefficient decreased with the decreasing of specimen lateral stiffness
4) The inter displacement of first floor is far larger that that of the second floor and roof. In Phase 1 to 4, when the normal PGA of table input is 0.1g、0.2g、0.4g and 0.55g , respectively, and the first drift ratio was bellow 1/500, 1/250 , 1/80 and 1/30 respectively.
5) The light wood framed house specimen can can meet the requirements for seismic intensity 8 described in China code for Seismic Design of building.

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