STATIC BEHAVIOR OF CHUANDOU WOODEN FRAMES UNDER LATERAL LOAD

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Abstract

The Chuandou wooden frame is an ancient form of Chinese traditional timber structures. Chuandou timber structure has in recent years, together with other wooden frames, been increasingly used structurally in a range of contemporary buildings in many countries. Though current structural study on the mechanical properties under the horizontal load for Chuandou wooden frames, it is lack of on full-scale model tests under lateral load resistance of and theoretical analysis. In the paper, an experimental investigation of static behavior has been completed, including material tests and the full-scale model prototype test under lateral load.

Material strength was determined from nine samples of thirty specimens including tensile/ compressive/ shearing strength parallel to grain of wood, tensile/ compressive strength perpendicular to grain of wood, modulus of elasticity in compressive parallel to grain of wood, modulus of elasticity in static bending of wood and bending strength of wood. All specimens were fabricated with fir lumber.

The full-scale model prototype test was comprised of the column, Fang, purlin and stone, size about 5.52m wide by 4.4m long, with approximate heights of 6.41 m. The predetermined vertical load was applied at on both sides of the each column top instead of roof weight. The pushover test was carried out using a manual hydraulic jack with a framework composed of two steel bars and steel beams, to ensure model stability and successfully loading. The hydraulic jack had a displacement range of 300mm and was applied at a uniform rate of 2kN/min in the test.

The results of the load lateral displacement curves were comprised of overall lateral displacements for frames, in-plane and out-plane lateral displacements at the bottom of columns, pull-up displacements at the bottom of columns, relative angle displacements. In general, the frame tested failed by slipping out of the stone plane in the bottom of columns. However, there was an obvious difference of the deformation for each column, indicating the failure mode is the large slip of columns, along with the overall overturning.

The tests show good overall performance and coordination performance due to the tightly coupled structural elements, even if the large deformation. The behavior of wooden frames with mortise-and-tenon joints under lateral load showed the slip stage, elastic stage and strengthening stage, when tangent stiffness of the frame was slowly reduced as the displacement was increased. The failure mode of the frame without wall panels and floors was integral sliding with obvious overturning under static lateral load.

Chuandou wooden frame finite element models were established by using the finite element analysis software SAP2000, with the different connection types of nodes. Compared with the test results, the model with semi-rigid and friction isolator units is more reasonable.

Keywords: Chuandou wooden frames, lateral resistance performance, full-scale model test, finite element analysis, Chinese traditional timber frame

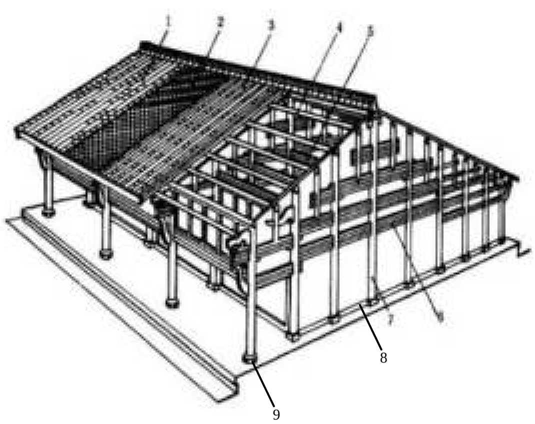
1. Introduction

The history of timber used as structural material in dwelling can be traced back to six thousand years before (Fig.1, Hemudu culture). Till Tang Dynasty, the timber structures in China express her excellent glory not only in buildings, but in bridges, pagodas and palaces (Fig.2). Although timber structures are widely used, the ancient Chinese architecture has numerous similar elements in part, because of the early method of standardizing and prescribing uniform features of structures. The famous book, "Yingzao Fashi" (the Specification of Timber Construction), which written in Song Dynasty (AC 1103) by Jie. Li, is recognized as the first timber construction code in the world. The manuals and drawings have been passed down through generations and dynasties till today.

Fig.1 Imitation Graph of the dwelling at Hemudu site, southeast of China during 5500-3500 B.C.

Fig.2 Great Hall of Foguang Temple on Wutai Mountain, rebuilt in 857 A.D.

Chuandou wooden frames is an ancient type which structural system is formed by columns, purlins and the Fang (a kind of beam) with mortise-and-tenon connections, while walls are just as a role of enclosure and separation (Fig.3). A significant number of dwelling houses with Chuandou wooden frames are used in China, particularly in the southeast and southwest regions.



1 clay tile put on the roof wood plank
2 enclosure wall, made of clay, or brick
3 ruffle
4 purlin
5 Dou-Fang, which inserts to post, is a beam in longitudinal direction of building
6 Chuan-Fang, which inserts to post, is a beam in transversal (main frame plane) direction of building
7 post, only laid up on the stone base without glue, or any other connections
8 Dijiao-Fang, which inserts to post, is a beam at the bottom of post in main frame plane,
9 stone base

Fig.3 The description of the Chuandou Frame wooden building

In the past millennium, with the development of concrete and brick used in low rise building and otherwise lack of structural wood supply, the science and technology of wood structural construction are almost ceased in China. However, in response to the pressure of climate change in this millennium, interest in Chuandou wooden frame, along with other wooden building types, has grown considerably in recent years for its high aesthetic appeal, low energy loss, and good seismic performance.

Structural design regulations for Chuandou wooden frame are very few and only list some suggestion of the maximum height, number of floors and minimum size of post and beam's section in the view of seismic safety. It is difficult to be analyzed by the classic static mechanics to obtain the lateral capacity under earthquake shock, for the Chuandou frame is not a stable system under lateral loads with movable columns all posed on their stones without any connection and mortise-and-tenon joints may rotated, which results that people still use the manuals and drawings followed by “Yingzao Fashi” to build the Chuandou frame. The ancient Chinese design philosophy for resistance of earthquake is energy dissipation rather than increasing of strength and stiffness of structure.

How to make a calculation hypothesis on the mortise-and-tenon joints and the interface of column base and stone foundation is the important and difficult point in traditional wood structure bearing capacity design. Now, the finite element analysis with simplified model or refinement model is the main research method for traditional wooden overall mechanical performance.

In theory, the mortise-and-tenon joint should be considered as a semi-rigid connection, for it can transfer axial force, bending moment and shear force and moment. But in the actual calculation, it is assumed as a hinge connection to make calculation easier which will bring some error. Models of Yingxian wood pagoda with the simplified connection assumption of rigid, hinge and semi-rigid are established by Jue. Wang[1], Ai-lan. Che[2]and Jing-ya. Chang[3]. However, there is no standard on calculating connection stiffness and resistance force for this kind of semi-rigid connection. Even in Japan, there are just some patents of the "construction method" which are not yet in the public domain. Doing experiments is the general method to determine the capacity.

As previously mentioned, wooden columns stand directly on the stone foundation with no embedding in Chuandou wooden frame houses. This way can transfer the vertical load pressure reliably without axial tension. However, wooden columns have to bear the shear force in lateral load resistance design. According to the principle of static equilibrium, the maximum shear which the wooden column can withstand to is limited by the maximum friction force between the bottom of column and the top of the stone foundation. Reference [4] assumes the connection between column base and stone foundation as hinge according to the test vibration mode of the structure, and reference [5] uses the friction isolator unit considering the relative sliding between column base and stone foundation.

Some requirements are listed for the height of the wooden houses in *Code for Seismic Design of Buildings GB 50011-2010* (Standards China 2010)[6], as it should not be more than 2 storeys high and 6 m in total height for a Chuandou wooden frame house in area of 6 ~ 8 degree seismic fortification intensity and it should be single storey building and the total height is not more than 3.3 m in area of 9 degree seismic fortification intensity. A wooden building shall not be more than three storeys, written in specification *Code for design of timber structure GB 50005-2003*[7], according to provisions of fireproof of timber structure. The structural arrangement and structural analysis of the building subjected to vertical loads are relatively simple for wooden house which is 2 ~ 3 storeys high, especially for residential house. In general, the arrangement of column grid and beams is determined according to the architectural requirement. Then the internal force of each member can be deduced by its loading area, and the bearing capacity and deformation can be calculated in accordance with relevant provisions. Due to lack of necessary scientific data, it is difficult to give a specific lateral force resistance fortification for Chuandou wooden frames.

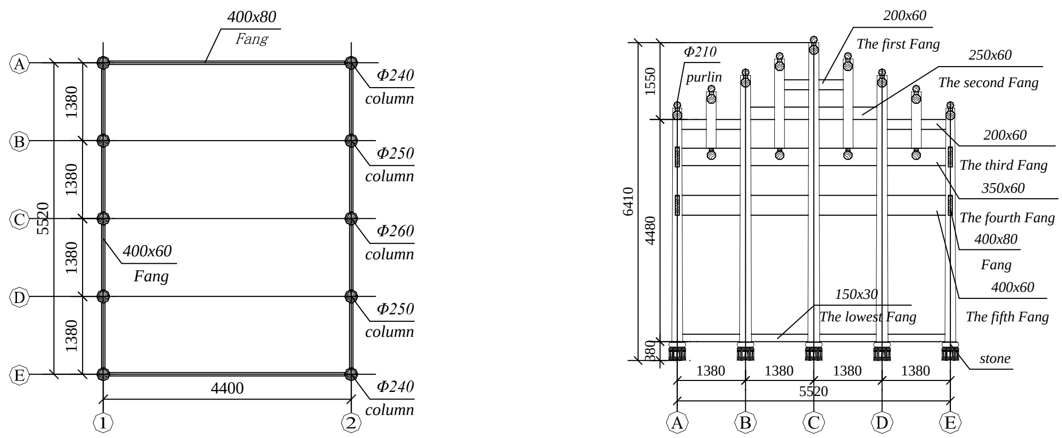
Though current structural study is prevail over the mechanical properties under the horizontal load for Chuandou wooden frames, it is lack of on full-scale model tests under lateral load resistance and of theoretical analysis. The primary aim of the study presented in this paper is to investigate the effects of joint type on the lateral resistance and the failure modes of the frames. The paper presents a novel experimental investigation, comprising material tests and the full-scale model prototype test, focusing on the horizontal bearing capacity, deformation properties, lateral stiffness for Chuandou wooden frames. Chuandou wooden frame finite element models, established by using SAP2000, with the different connection types of nodes, were compared with the test results.

2. Experimental Study

An experimental investigation of static behavior, the full-scale model prototype test under lateral load, has been completed. In this section, test results on the full-scale model prototype test are summarized.

2.1 Test setup and procedure

The model was composed of two piece of Chuandou frames, each one made of five posts, five Chuan-Fangs, one Dijiao-Fang, and laid up on five stone bases. During the procedure of construction, the column and stone were positioned first. Posts and Chuan-Fang were connected by mortise-and-tenon joints to make a frame, secondly. Than a piece of frame was hoisted and laid up on the stone base directly, and supported by temporary braces. After that, another piece of frame was set up with same procedure. Finally, purlins were embedded on the tops of posts. The distributed gravity load in the actual structure was simulated with additional weight hanging on the Dou-Fang closely to the column. It's about 5.52m wide by 4.40m long, with approximate heights of 6.41m (Fig.4). The predetermined vertical load was applied at on both sides of the each column top instead of roof weight (Table 1).



(a) Plan (b) Elevation

Fig.4 – The full-scale model prototype test

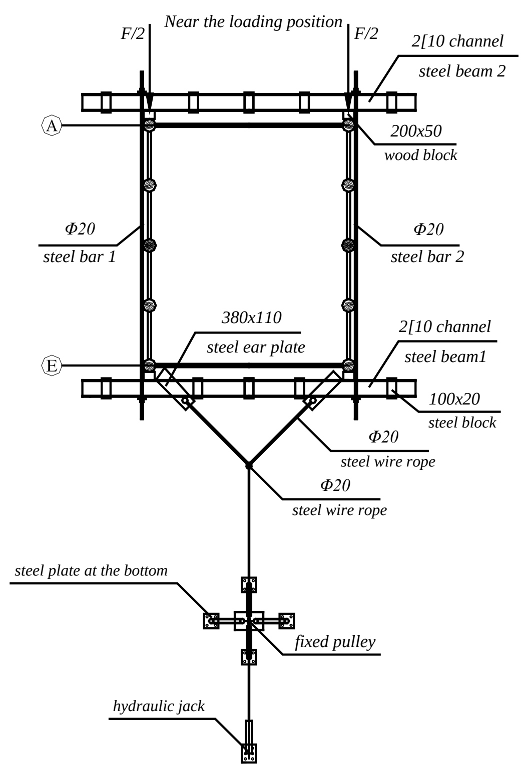


Fig.5 – Loading device design

Table 1 – Roof load for full-scale model pushover test

Load type	Tile	Roof waterproof layer	Sheathing	Rafter	Total
Constant load (kN/m2)	1.1	0.1	0.2	0.15	1.6
Total (kN)	$1.6\times4.4\times5.52/\cos24^{0}=42.5$				

Prototype static test was carried out on site using a manual hydraulic jack with a framework composed of two steel bars and steel beams, to ensure model stability and successfully loading (Fig.5). The position of the horizontal force was act in the middle of the third Fang (3.6m height from the ground). The horizontal force was passed to the steel ear plate through the steel wire rope, and then to the steel beam and the steel bars. The wood block was located between the steel beam and the column, so as to relief the local stress.

The hydraulic Jack and the ground was connected by chemically planted steel bar, and then the horizontal force was passed to the timber frame through the rope and a fixed pulley (Fig.6).The hydraulic jack had a displacement range of 300mm. In testing, a lateral load was applied at a uniform rate of 2kN/min sufficient to obtain a detailed strain profile of the frame before failure. The lateral load and displacement were automatically recorded.

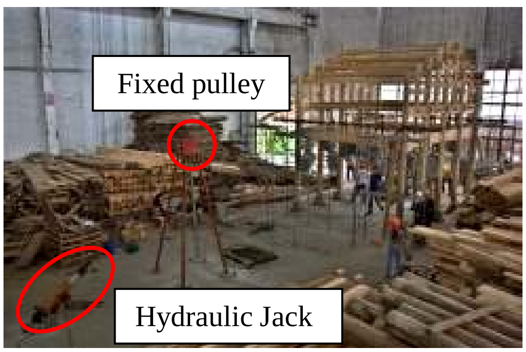


Fig.6 – Loading device arrangement

Fig.7 – Final deformation for full-scale model pushover test

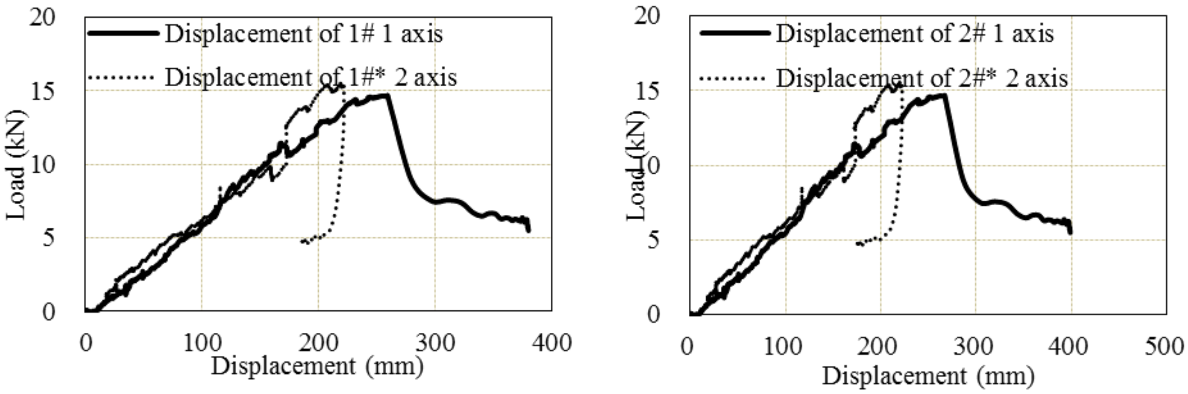
The exactness of the ultimate load estimate and effectiveness of the load device were verified by the full-scale model prototype test. The test phenomenon was similar to that in the earlier stage of the full-scale model prototype test, not repeat them here.

2.2 Experimental results

Fig.8 –The deformation of joints

With the increase of horizontal force, the wooden frame appeared tiny shaking, and mortise-and-tenon joints were gradually squeezed. Columns began to produce significant rotation with tiny cracks between columns and Dijiao-Fang .When the horizontal load 16kN, the slipping of the column reached 8mm, rotation angle of mortise-and-tenon joints reached 2^{o}, and the cracks were increasing between columns and Dijiao-Fang. When the horizontal load 24kN, the tilt of the wooden frame was increasing, seriously cracking at bottom of the column, the maximum slipping of the column reached 22mm (Fig.7 and Fig.8).

When loading by displacement control, the structure was failure with columns completely slipped out (Fig.9).



（a）Load – displacement curve for A axis （b）Load – displacement curve for E axis

Fig.9 – Load – displacement curve for full-scale model pushover test

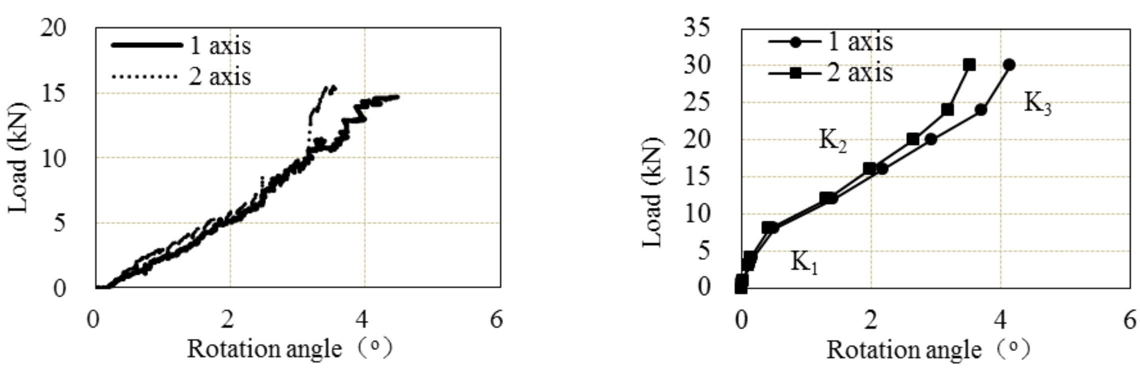
The experimental load - displacement relationships are presented in Fig.9. In the initial, the wood frames appear tiny slip, and then the structural stiffness kept increasing. When load to about 10kN, the structural stiffness was slightly decreased.

The maximum storey displacement angle before the frames failure is:

$$\theta_{\max} = \Delta / h = 206 / 3220 \approx 1/16 \tag{1}$$

The maximum storey drift angle of the timber frame is 1/16, which is much larger than the maximum storey drift angle of the reinforced concrete structures 1/50 under the rare earthquake.

With the help of a wooden pin, the pulling out was small. So the angle deformation was main damage form at the mortise-and-tenon joint. From the load – rotation angle curve, we can see that the slip stage, elastic stage and strengthening stage happened at mortise-and-tenon joins (Fig.10). In the initial, mortise-and-tenon joins squeezed, the relationship of the load and rotation was linear elastic, and the rotation stiffness was K_1.When load to 8kN, the rotation angle was 0.50^{o}, the curve was approximated a straight line, and the rotation stiffness K_2 was smaller than K_1.When the rotation angle was greater than 3^{o}, the rotational stiffness started increasing, and the whole frame was in the stage of strengthening. At the end, the rotation did not reach the limit value, and the plastic was not fully developed.



（a）Load – displacement curve for A axis （b）Load – displacement curve for E axis

Fig.10 – Load – rotation angle curve for the mortise-and-tenon joint

The slip values at the bottom of the column at each stage are shown in Table 2.

Table 2 –The slip values at the bottom of the column at each stage

Loading phase (total ultimate load%)	In-plane slipping at the bottom of columns(mm)									
	A1	B1	C1	D1	E1	A2	B2	C2	D2	E2
2.50%(1kN)	0.09	0.06	-0.03	0.06	0	0.03	0.02	0.09	0	0.03
7.50%(3kN)	0.52	0.49	-0.04	0.46	0.41	0.44	0.38	0.53	0.17	0.48
10%(4kN)	0.69	0.65	0.18	0.61	0.56	0.62	0.52	0.69	0.25	0.69
20%(8kN)	1.84	1.85	0.37	1.6	1.7	1.69	1.56	1.89	0.82	2.29
30%(12kN)	4.84	3.86	1.08	2.68	2.38	3.68	4.47	3.89	1.38	4.05
40%(16kN）	4.54	3.34	1.35	1.61	2.25	8.37	4.76	3.67	0.62	4.29
50%(20kN)	21.73	2.78	1.46	0.71	2.32	14.99	3.93	3.84	0.33	4.3
60%(24kN)	33.38	35.52	1.95	0.32	1.94	22.31	30.92	3.52	0.45	4.24
Maximum load (30kN)	53.9	56.86	2.86	3.06	0.56	30.18	45.02	2.48	0.73	4.05
Before displacement meter removed	56.72	70.37	2.93	52.19	58.77	28.86	46.75	3.43	1.57	4.18

Actually, the load is not evenly distributed among columns of each frame, the stress of the column near the loading position is larger initially. With the increase of load, the deformation of structure increases, the stiffness decreases, and the load is redistributed. Because of the influence of uneven distribution of horizontal load and redistribution of internal force, a specific failure mode will not happen.

The comparison of slip of each column before the detachment of displacement meter is showed in Fig.11. The slip of axis C was small, which can be approximated to zero, but the slip of other columns were much larger. The rotation around C axis in the frame was supposed to happen. Therefore, the failure mode was the large slip of column base, along with the overall overturning of structure.

2.3 Structural capacity

For a more detailed analysis on the test results, the structural performance parameters were defined, including the ultimate load, the ultimate displacement, the peak load, the peak displacement and the lateral stiffness. With reference to *ASTM E2126 (2011)* [8] Based on energy equivalent to the ideal elastic plasticity curve (EEEP), we can determine the lateral force resisting performance indicators of the timber frame through the load displacement curve under monotonic load, as shown in Fig.12.

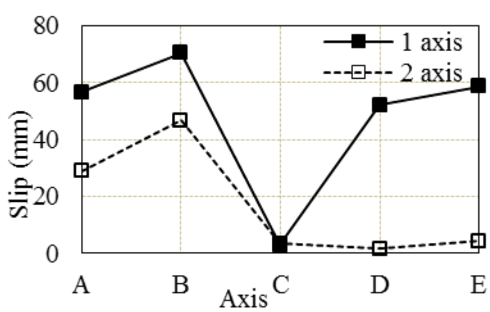


Fig.11 – Slip of each column before the detachment of displacement meter

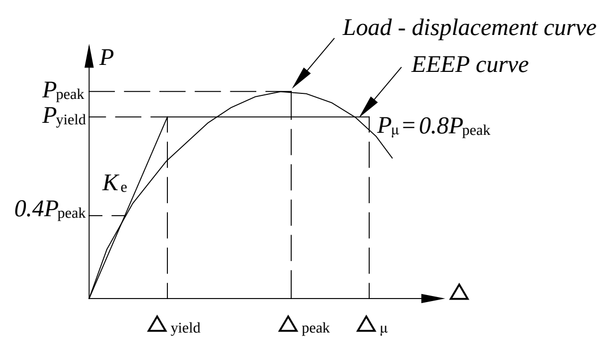


Fig.12 – Definition of performance parameters

The peak load (P_{peak}) refers to the maximum value of the load in the load displacement curve, and the peak displacement (Δ_{peak}) corresponds to the peak load. The ultimate load or failure load (P_{μ}) refers to the 80% of the peak load as load declining, and the ultimate displacement (Δ_{μ}) corresponds to the peak load. K_e , μ , Δ_{yield} and θ refer to the elastic stiffness, ductility coefficient, yield displacement and maximum storey drift angle, where $K_e = 0.4P_{peak} / \Delta_{0.4peak}$, $\mu = \Delta_{\mu} / \Delta_{yield}$, $\theta = \Delta_{\mu} / H$.

The yield load is expressed as $P_{yield} = \left[\Delta_{\mu} - \sqrt{\Delta_{\mu}^2 - 2A / K_e}\right] \cdot K_e$, where A is the area surrounded by the curve from zero point to the point of structural failure. Finally, parameter values are shown in Table 3.

Table 3 – Parameter values for full-scale model pushover test

Displacement meter number	P_{peak}	Δ_{peak}	P_{μ}	Δ_{μ}	P_{yield}	Δ_{yield}	K_e	μ	θ
1 axis 1#	14.69	259.04	11.75	206.35	10.63	175.10	0.06	1.04	1/16
1 axis 2#	14.69	266.84	11.75	212.01	10.68	179.33	0.06	1.04	1/16
2 axis 1#*	15.55	219.49	11.70	192.05	8.93	160.30	0.06	1.01	1/18
2 axis 2#*	15.55	219.78	12.44	217.34	9.01	161.84	0.06	1.01	1/16

Note: the load unit is kN , the displacement unit is mm , the rigidity unit is kN / mm .

The ductility coefficients of the 1 and 2 axis wood frames are 1.04 and 1.01. It does imply that the ductility coefficient of the Chuandou wooden frames is small. In the elastic stage, the deformation of the structure is too large, which leads to the low ductility coefficient of the structure, which is similar to the reference [9]. On the other hand, mortise tenon joints did not reach the ultimate value, no obvious plastic deformation and the energy dissipation capability failed to fully play. So, In order to take advantage of the traditional wooden structure, column slip limit, the structure giving full play to the mortise-and-tenon joint of plastic deformation ability may be necessary in the design.

The maximum storey drift angle of the timber frame is 1/16, which is much larger than the maximum storey drift angle of the reinforced concrete structures 1/50 under the rare earthquake. The results showed that the Chuandou wooden frame has a strong deformation capacity and the Fang between columns can ensure the effective transfer of horizontal force.

3. Finite Element Simulation

On the basis of tests, Chuandou wooden frame finite element models were established by using the finite element analysis software SAP2000, simulating the timber frame with space beam element, simulating roof weight with the equivalent concentrated load on the top of wooden columns.

3.1 Material parameters

In order to provide real and reliable parameters for theoretical analysis and numerical simulation, the mechanical properties of the materials were tested (Fig.13). Displacement control was used for all tests. After each test completed, the moisture content tester was used to test the moisture content of the specimen.

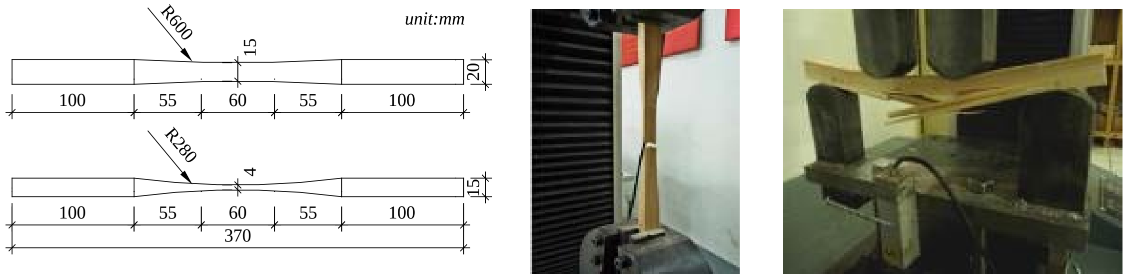


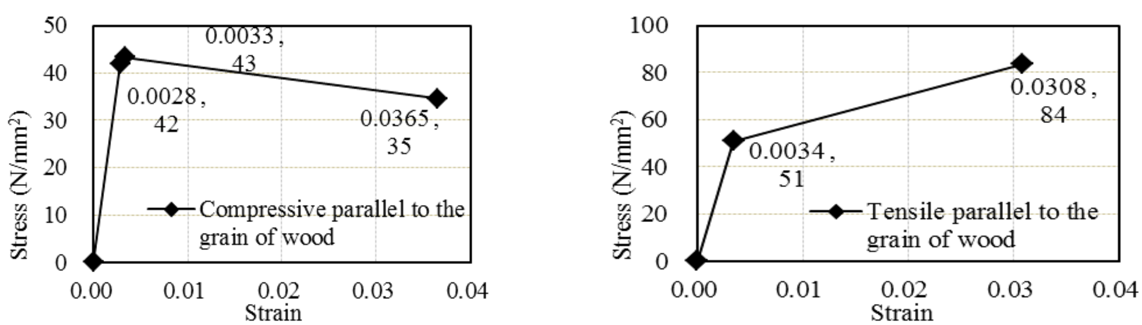
Fig.13 –Test of tensile strength parallel to grain of wood and bending strength of wood

Table 4 – Summary of test results

Property Types		Effective numbers	Mean values/MPa	Average moisture content
Tensile strength parallel to grain of wood		26	86.66	18%
Compressive strength parallel to grain of wood		27	47.25	18%
Modulus of elasticity in compressive parallel to grain of wood		27	14928.81	18%
Shearing strength parallel to grain of wood		29	9.71	18%
Compressive strength perpendicular to grain of wood	Total cross-section compressive strength	27	1.57	19%
	Local cross-section compressive strength	25	1.74	18%
Tensile strength perpendicular to grain of wood		28	1.08	18%
Bending strength of wood		23	62.69	18%
Modulus of elasticity in static bending of wood and bending strength of wood		23	9705.82	18%

Material strength was determined from nine samples of thirty specimens. All specimens were fabricated with fir lumber, and the sizes and numbers were designed according to *Method for physical and mechanical tests of wood (GB1927~1943-91)* and *Standard Test Methods for Small Clear Specimens of Timber (D143 – 09)*. After the unreasonable test data discarded, the values for mechanical properties were calculated, and the results were summarized in Table 4. The mean value is the corresponding strength when the water content is 12%.

The disparate tension and compression model was used considering the different properties of wooden compression and tension. The constitutive model of wood is determined as shown in Fig.14.



（a）Compressive parallel to the grain of wood （b）Tensile parallel to the grain of wood

Fig.14 – Constitutive model of wood

3.2 Assumption for mortise-and-tenon joints

Multi linear elastic connection unit was used for mortise-and-tenon joint according to the test and the study of reference [10]. Stiffness K_1, K_2 and K_3 of moment-rotation curve were $130\,kN\cdot m/rad$, $33\,kN\cdot m/rad$ and $74\,kN\cdot m/rad$ respectively as shown in Fig.15, and its corresponding θ_y and M_y were determined according to the stiffness ratio and angle ratio in each stage as shown in Table 5.

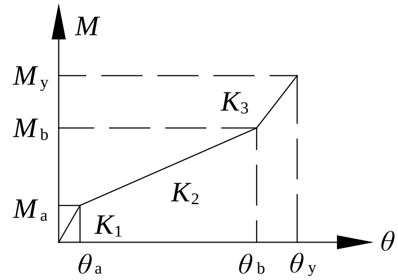


Fig.15 –Test phenomena and constitutive model of mortise-and-tenon joint

Table 5 – Parameters of constitutive model of mortise-and-tenon joint nodes [10]

$M_a / kN\cdot m$	$M_b / kN\cdot m$	$M_y / kN\cdot m$	θ_a / rad	θ_b / rad	θ_y / rad
0.52	2.37	3.85	0.004	0.06	0.08

3.3 Assumption for the connection of column base and stone foundation

To simulate the semi-rigid connection properties, the friction isolation unit was used for the connection of column base and stone foundation according to its friction-sliding from the test (Fig.16), and k was the shear stiffness for vibration isolation unit which equaled to distributive friction force of each column. The roof loads a side column carried was half to those a center column carried, so the friction force and shear stiffness of a side column were 1.9 kN and 190 kN/m respectively, of a center column $3.8\,kN$ and $380\,kN/m$ respectively.

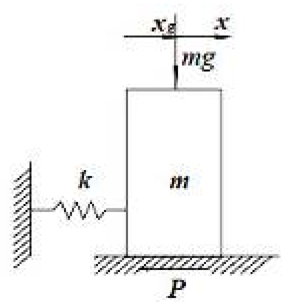


Fig.16 –Test phenomena and friction-sliding model of the connection of column base and stone foundation

3.4 Results analysis and comparison

Some finite element models were established considering the different assumptions of the column foot and mortise-and-tenon joint, and maximum carrying capacity was up to 14 kN for single wooden frame in the test model. The three types of models were calculated and the results were shown in Table 6.

Comparing the model 1 and 3, we can find that the internal force of wooden frame under the action of horizontal force was very close to the model 1 when the connection of column base and stone foundation was assumed as hinge, but the displacement of column cap of model 3 was half of the model 1.When considering the energy dissipation effect and deformation of structure, the column was simplifying the connection of column base and stone foundation as the friction isolation unit.

Comparing model 1 and model 2, when the node of column and Fang was defined as multi linear elastic link unit, the node stiffness was weakened, which led to that most bending moment was carried by column for its end stiffness was relatively larger than Fang's end stiffness. So, the mortise-and-tenon joint nodes can not be assumed to be rigid in calculation of Chuandou wooden frames.

Compared with the test results, nodes assumptions of the model 2 was more reasonable for the displacement at the top of the column was closest.

Table 6 – Model analysis results of different assumptions for the wooden frame

Model	Calculation assumptions		Calculation result						
			Moment at the column / $kN \cdot m$					displacement at the top of the E column/ m	
			A axis	B axis	C axis	D axis	E axis		
Test result	—		—	—	—	—	—	1 axis	0.262
								2 axis	0.222
Model 1	Mortise-and-tenon connection	Rigid	3.8	6.4	8.0	7.1	5.5	0.013	
	Column and stone foundation	Friction isolator							
Model 2	Mortise-and-tenon connection	Multi linear elastic	21.4	25.1	29.5	25.6	22.3	0.107	
	Column and stone foundation	Friction isolator							
Model 3	Mortise-and-tenon connection	Rigid	3.9	6.4	8.0	7.1	5.5	0.006	
	Column and stone foundation	Hinge							

4. Summary and Conclusions

This paper presents an experimental study on the horizontal bearing capacity, deformation properties, lateral stiffness for Chuandou wooden frames. Results from the material test and the full-scale model prototype test are presented, and the horizontal bearing capacity, deformation behavior, lateral stiffness, failure mode and joint deformation have been investigated. Chuandou wooden frame finite element models were established by using the finite element analysis software SAP2000, with the different connection types of nodes.The following conclusions have been made:

- The maximum storey drift angle of the timber frame is 1/16 in the test. It shows good overall performance and coordination performance due to the tightly coupled structural elements, even if the large deformation. The failure mode of the frame without wall panels and floors was integral sliding with obvious overturning under static lateral load. The behavior of wooden frames with mortise-and-tenon joints under lateral load showed the slip stage, elastic stage and strengthening stage, when tangent stiffness of the frame was slowly reduced as the displacement was increased.

- The ductility coefficient of Chuandou wooden frames was small, because the deformation in the elastic stage was too large, which led to the low ductility coefficient of the structure.

- The mortise-and-tenon joint is semi-rigid connection, and can not be simplified to rigid. Column and stone foundation can slip, and can not be simplified to hinge. Compared with the test results, the model, with column and stone foundation as friction isolator unit and the mortise-and-tenon joint as multi linear elastic unit was more reasonable.

5. Acknowledgments

The writers would like to thank the National Science and Technology Support Program "*Key technology research and demonstration for the prevention and control of village geological disasters*"(2014BAL05B01) and the State Key Laboratory of Disaster Prevention in Civil Engineering at Tongji University. The input from various partners is also gratefully acknowledged, in particular, that from Shanghai Guyin real estate Co., Ltd., who prepared the largescale specimens. Finally, Professors and colleagues at the University of Tongji, in particular Professor Zhuhua Liu, technician Miaomiao Yuan, and students, Shi Ruiyuan, Li Benben and Wu Ying, are all thanked for their hard and smart work.

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