# Seismic Assessment of Traditional Masonry-Wooden Building in Taiwan



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#### SUMMARY:

In Taiwan, numerous existing traditional buildings are constructed with wooden frames, wooden panels, and masonry walls. The wooden frames are mortise-tenon jointed. The wooden panels infilled in frames are used to partition the interior space. The masonry walls are erected as external walls. It is because the complication of the construction of these old buildings, a procedure for engineer or conservation architect to assess its seismic capacity is still unavailable. In this study, a general procedure for assessment of these old constructions is established. It contains three parts: (1) modeling and nonlinear hinge settings, (2) push-over analysis to determine ultimate base shear and displacement, (3) using capacity spectrum method to determine corresponding PGAs of various performance points. The modeling of nonlinear hinges for masonry walls, wooden wall panels and mortise-tenon joints are based on their corresponding nonlinear behavior. Using this procedure, a 2-story building which is registered as heritage architecture of Tainan City was assessed. The behavior obtained can be divided as two stages. The first stage has high stiffness which is controlled by the in-plane stiffness and strength of masonry wall before the drift of 0.6%. The second stage makes the structure become more flexible after the in-plane failure of masonry wall occurred. When the roof drift reaches 5% in y-direction (axis in building depth), the corresponding PGA is 0.3g. Furthermore, it is observed that in the second stage, the wooden wall panels play an important role in structural stability to prevent collapse of whole structure.

Keywords: Seismic Assessment, Masonry-Wooden Historic Building, Push-over analysis, Capacity Spectrum Method

## **1. INTRODUCTION**

Most of the traditional historic buildings in Taiwan are constructed with mortise-tenon jointed wooden frames and surrounding masonry walls. Due to the shortage of long lumber, in these buildings, the multi-story cases are limited. In structural system, these existing multi-story cases can be divided into following 4 types (Fig. 1.), (a) wooden frames with continual columns confined by the surrounding masonry wall at lower story, (b) wooden frames with continual columns to bear vertical load and the surrounding masonry wall extending to 2-story, (c) wooden frames with story-divided columns and the surrounding load-bearing masonry wall, (d) multi story load-bearing wall and beams clamped. Nowadays in Taiwan, the conservation of these traditional buildings has highly caused attention; however, a reasonable method for the seismic assessment of these building is still unavailable. This study mainly focuses on the seismic assessment of these buildings which resist lateral loads by three main parts: mortise-tenon jointed frames, wooden panel partition walls, and masonry exterior walls. A 2-story case of type (a) which is called Yen-shuei Octagonal Hall had been evaluated in this paper, by push-over analysis with nonlinear hinges of these three main parts. Finally, the push-over curves were evaluated by capacity spectrum method to determine corresponding PGAs of various performance points.



Figure 1. Typical structural systems of traditional 2-story masonry-wooden buildings in Taiwan

### 2. MODELING OF YEN-SHUEI OCTAGONAL HALL

#### 2.1. Structural system and modelling simplification

Octagonal Hall was built in 1847 and located in southern area of Taiwan. The first floor plan of the building is about 13.4m×7.6m, and the height of the 2<sup>nd</sup> floor and the roof ridges are 3.9m and 8.8m (Fig.2., Fig.3.). The structural system consists of surrounding 1-story-height masonry walls and four mortise-tenon wooden frames, as shown in fig. 3. The masonry walls are constructed by bricks in stretch bond for two side walls and dou-chi (box) bond for back wall. The thickness of two side walls is 39cm, and that of back wall is 36cm. The four wooden frames are parallel to the side walls which two sets were half-imbedded in each side wall and two middle frames divide the interior into three spaces. There are several perpendicular wooden tie beams which connect four wooden frames to make system stable. Then the slab beams and purlins are put on the "ta-tung"(girder) and the column tops of wooden frames as simply supported beams. The interior space is partitioned by wooden t&g wall panels whose thickness is about 1.5-3.0cm. And these panels were framed to infill into the wooden structural frames.

A nonlinear 3D model was formed and analyzed by SAP2000 (Fig.7.). In order to assign nonlinear properties as nonlinear hinges, all of the structural elements were modelled as frame elements, also including masonry walls and wooden panel walls. For brick walls, they were modelled as columns and each section is assigned as thickness and width of the wall. And the nonlinear bending and shear hinges were assigned to the wall bottom and mid-height to simulate the out-of-plane bending and in-plane shear resistance (Fig.8., Fig.9.). For wooden partition panel walls, they were modelled as diagonal compressive members. And the cross section and the elasticity were transformed to equivalent values, and the nonlinear axial hinge was assigned to the midpoint to simulate shear resistance of the panel wall (Fig.10). For the mortise-tenon members of wooden frames, the nonlinear bending hinges were assigned to both two ends of major members (Fig.11). The other minor members of wooden frame were simplified to release both ends of moment resistance as tie members as shown in Fig.5. and Fig.6.

The analytical model is shown in Fig.7. and totally there were 1853 elements in this model. The mechanical properties of the brick wall and wood material were showed as Table 1. Some dead loads of roofing material, roof ridges, floor and other finishing were applied to the corresponding bearing members. And no live loads were applied to the model in order to evaluate the limit resistance of this building.



Figure 2. 1F and 2F plan



Figure 3. Southern and eastern(front) elevation



Figure 4. Section of a-a' and b-b'



Figure 5. Moment release of the analytical model



Figure 6. Moment release of floor beams



Figure 7. 3D view of analytical model

	Material	Unit Weight (kgf/m <sup>3</sup> )	Elasticity (kgf/cm <sup>2</sup> )	Poisson's Ratio	
Brick Wall	Isotropic	1900	15000	0.25	
	Longitudinal (Dir 1)		70000	0.378 (Plane12)	
Wood	Radial (Dir 2)	430	2800	0.296 (Plane13)	
	Tangential (Dir 3)		2800	0.484 (Plane23)	

Table 1. Material properties used in the model

1.2

1

8.0 8 9.0 8

0.2





Figure 10. Hinge property of wooden-panel wall



1.2



Figure 11. Hinge property of mortise-tenon joint

### **3. RESULT OF PUSHOVER ANALYSIS**

The pushover lateral load is applied by the referring 0.1g static earthquake coefficient and under displacement control. And Fig.12. and Fig.13. show the results of analysis in y-direction and x-direction. In y-direction, as Fig.12. shows, there are 5 significant peaks in the pushover curve. The first peak is due to out-of-plane failure of the back brick wall, and it occurred at the drift of 0.05%. This failure made quite slight effect to the initial stiffness of structure. The next peak is the maximum base shear about 32tf at the drift of 0.59%. The corresponding behavior is the shear failure of two side brick walls, and then the base shear resistance of structure is down to about 5tf changing to a flexible system. The stiffness of this system is controlled by the behavior of wooden panel walls and mortise-tenon joints. The first failure of wooden panel wall is located under the front window of the middle frame in the first floor until the drift of 4.9%. The coming next two failures of the wooden panel wall occurred in quite larger drift to 7.4% and 9.1%. For the behavior of mortise-tenon joints, the first yielding location is near front and back area in the first floor, and the corresponding drift is about 4.2%. Fig.12. also shows the contributing proportion of lateral resistance after the failure of brick walls. There is about 60% by the mortise-tenon joints and 40% by the wooden panel walls in y-direction.

In x-direction, as Fig.13. shows, the lateral resistance of the structure is much weaker than that in y-direction. The behavior is quite similar as in y-direction. The out-of-plane failure of two side brick walls occurred in the drift of 0.12%, and the in-plane shear failure of the back brick wall occurred in the drift of 0.62%. The corresponding max base shear is about 14tf. For the behavior of the wooden panel wall, the first failure occurred in very large deformation of 9.9% drift. This is resulted in the discontinuity of the partition panel walls between lower and upper floor in x-direction, and the more critical problem is that these partition walls are not connected directly with main columns of wooden frames. Nevertheless, these panel walls still play an important role in x-direction because of the less number and the weakness of mortise-tenon joints in this direction. The lateral stiffness contribution of mortise-tenon joints is only 14% after the disable of brick walls.



Figure 12. Pushover curve of y-direction



Figure 13. Pushover curve of x- direction

### 4. SEISMIC ASSESSMENT BY CAPACITY SPECTRUM METHOD

Based on ATC (1996) and Chung, L. L. et al.(2009), the corresponding PGA of a building's seismic resistance could be obtained by the Capacity Spectrum Method (CSM) which converting base shear and roof displacement pushover curve to Acceleration-Displacement Response Spectra(ADRS) format and this ADRS curve is called as capacity spectrum. Then the capacity spectrum is overlaid on the effective-damping-reduced ADRS design spectrum to find the matching intersection as the performance point.

In order to apply CSM method in this study, we have to simplify the analytic model to a 2-d.o.f story-lumped system as Fig.14. shows. The story displacement is averaged by displacements of eight main columns on the story level (Fig.15.). This displacement simplification yielded the results of 1<sup>st</sup> mode shape as Fig.16. and Table.2. shows. The mode shapes are obviously different before and after the shear failure of brick walls. When the stiffness of 1<sup>st</sup> floor decayed, the displacement of 2F increased and also made the modal mass coefficient  $\alpha_1$  increase, and the modal participation factor PF<sub>1</sub>(displacement relationship between modal and roof) was also changed. The following ADRS conversions of capacity spectrum were calculated by different  $\alpha_1$ s and PF<sub>1</sub>s before and after shear failure of brick wall.



Figure 14. Simplified CSM model of 2 d.o.f system



Figure 15. Mode shapes and corresponding modal period



Figure 16. Mode shapes comparison before and after the failure of brick wall

	E1	Floor Weight (kgf)	Mode Shape Φ				
	Floor Level (m)		1 <sup>st</sup> Mode (x-dir)		2 <sup>nd</sup> Mode (y-dir)		
			Before brick wall	After	Before brick wall	After	
	()		failure (T=0.704s)	(T=1.216s)	failure (T=0.451s)	(T=0.952s)	
RF	7.7	33,918	1	1	1	1	
2F	3.5	87,098	0.227	0.664	0.122	0.752	
Σ		121,016					
Modal participation factor $PF_1$		cipation factor <b>PF</b> <sub>1</sub>	1.398	1.269	1.264	1.195	
Modal mass coef. $\alpha_1$		mass coef. $\alpha_1$	0.620	0.962	0.465	0.982	

Table 2. Mode shape and CSM converting factors of Octagonal Hall 2-d.o.f system

Fig.17. shows the capacity and demand spectrum in y-direction. The demand spectrum is based on the design effective ground acceleration of 475-year return period in this site which is equal to 0.308g. There are 4 points to be discussed in this figure. The point P1 and P2 correspond to out-of-plane and in-plane failure of brick walls; their equivalent periods are 0.45s and 0.58s and equivalent damping ratios are both 5%. These two points could not intersect the 5% demand spectrum, and it means that in-plane shear resistance of two side brick walls could not be enough for the demand. If the strengthening strategy is improving the shear strength of walls, the point P2 has to extend to P2'. After wall in-plane failure, the structure transformed from rigid system to ductile system and its equivalent period extended longer than 2.0s. The point P3 is the matched performance point which could provide the code demand: its equivalent period is about 2.9s, equivalent damping ratio is 9.0%, and the spectral displacement is 38cm. Fig.18 shows the pushover curve and corresponding PGAs which is calculated with the inverse method provided by Chung, L. L. et al.(2009). The result could be adopted easily for strengthening design. The result shows that the roof displacement needs to reach 45cm (5% drift) to have ability to resist PGA=0.308g. Some retrofitting techniques for mortise-tenon joints are needed to improve, in order to prevent tenon joints being pulled out from mortise joints.

Fig.19. and Fig.20 shows the result in x-direction. The point P2 which corresponds to in-plane failure of back brick wall is farther from the demand spectrum than that in y-direction. And the corresponding PGA of the point P2 is only 0.09g as Fig.20. shows. The point P3 matched the demand spectrum of 8.7% damping ratio. The roof displacement reached to 65cm (7.5% drift) which is very large deformation. It is doubted whether the mortise-tenon joints could maintain the connecting ability. On the other hand, the P-Delta effect would also cause the structure deformed seriously. For the strategy of strengthening in x-direction, it is necessary to control the roof displacement and improve the stiffness both for back brick wall and for the wooden frames. To strengthen the in-plane resistance of the back brick wall has an advantage of preventing serious failure in medium scale earthquake. And the strengthening applied on wooden frames such as reformation between partition wooden panel walls and main frames could improve the stiffness in weak x-direction and make the performance point shift to smaller displacement.



Figure 17. Capacity and demand spectrum of y-dir (PGA=0.308g)



Figure 18. Pushover curve and corresponding PGAs (y-dir)







Figure 20. Pushover curve and corresponding PGAs (x-dir)

## **5. CONCLUSION**

In this study, a procedure for the seismic assessment of traditional Taiwanese wood building has been developed and been practiced in a heritage architecture of Yen-shuei Octagonal Hall. The developed procedure needs nonlinear hinge settings for beam-column joint, wooden panel wall, and masonry

wall. The resisting PGA will be obtained from pushover analysis and capacity spectrum method. In the assessed Octagonal Hall, followings are observed:

- 1. The building stiffness and earthquake resistant capacity in x-direction (direction parallel to building width) and in y-direction (direction parallel to building depth) are much different. The difference is related to the layout of surrounding masonry wall and wooden panel wall.
- 2. Under the presupposition that if mortise-tenon joints are not pulled out, the wooden structure would not collapse even in large lateral deformation. The wooden panel wall provides its major contribution to this performance.
- 3. For obtaining the analytical model for capacity spectrum method, the flexible diaphragm of wood structural system could be simplified by story average displacements.

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