# PRELIMINARY STRUCTURAL ANALYSIS STUDY OF THE CHINESE COMPLEX BRACKET SYSTEMS

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## ABSTRACT

A series of in-plane structural studies of the traditional timber structures have been conducted since the 1999 Chi-Chi earthquake. Most of these studies were largely based on quasi-static tests, limited modelling studies are found on the dynamic behaviour of these structural types, in particular, the complex bracket systems. Owing to their complexity and the lack of proper seismic evaluation methods for this type of oriental timber structures, structural engineers often assume these timber connections bear no moment resistance, and thus, will tend to simplify them to two idealized extreme forms – fully rigid or fully-hinge/pin joint. Hence, the predicted outcomes often appeared to be unrealistic as they do not truly reflect the actual behaviour of the oriental timber structures. Under-estimation not only gives the public a misguided perception that traditional timber structures are weak and not durable, it also undermines its true seismic capability. From past experimental results, it is understood that oriental timber joints generally behaved like semi-rigid joints than hinges or pin, and the overall stiffness of the global structure is related to vertical loads, friction and partial embedment of wood fibers between contact surfaces. Close-form analytical models have been derived and the predictions fit well with the test results. For verification sake, the above models were subjected to two examinations. Firstly, the models' assumptions and calculated values were cross-validated with past dynamic tests of complex bracket sets with various vertical loads and structural system designs. Next, the calculated assumptions were applied to conventional numerical modelling software and the predictions were cross-referenced with test results. Preliminary results from the above examinations revealed that the predicted models and anticipated weak points of the global structures were generally in good agreement with the dynamic test results, hence the assumptions made generally work well in both static and dynamic tests.
*Keywords: historical timber structures, complex brackets, shaking table tests.*

## 1 INTRODUCTION

Heavy roofs, complex bracket systems, mortise–tenon jointed timber frames and load-bearing walls are characteristic features of the ancient oriental historic timber buildings. Due to the complexity of oriental timber frame construction and the lack of proper seismic evaluation methods for this type of traditional structures, the conventional practice is either to neglect or to set all timber joints as hinged, by assuming that these traditional timber connections bear no moment resistance. From the past related test results [1]–[11], it is observed that the oriental timber joint connections generally performed more like semi-rigid joints than hinges and that, oriental timber joints tend to slip when subjected to lateral force. The overall stiffness of the global structure is also found to be closely related to vertical loads, friction and partial embedment of wood fibres between contact surfaces. Thus, if the above factors are not considered during structural analysis, misunderstanding of the actual deformation pattern might undermine the true seismic capability of these oriental timber structures and subsequently, sending out a misguided notion to the public that traditional oriental timber structures are weak and non-durable.

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Based on our team's past research efforts on the southern Chinese traditional Dieh-Dou type timber structures [2]–[10], a general understanding on the structural behaviour and fracture patterns of the global and critical joint systems has been achieved. Firstly, column restoring force and beam–column connections play a crucial role towards the stability of the global structure. When the frame deformation is small, most of the moment-resisting responsibility falls on the column restoring force. But when the deformation angle gets larger, bending moments from the tie beams tend to take over most of the moment resistance mechanism [7]. Although a traditional timber structure works best when all the mortise–tenon and dowel connections are tightly held in place by heavy roof load [7], the large inertia force resulting from the heavy vertical load also tends to magnify the rocking effect and causes greater deformation to the global structure, particularly around the mortise region of the Dou members. Typical damage patterns, such as widen mortise region of the Dou member, incidentally led to differential rocking behaviour between the front and back end complex brackets. Under such circumstances, the unsynchronised rocking behaviour of the complex brackets indirectly helps to restrain the global structure from further damage till large and/or continued seismic (or lateral) force arises. Friction force between the contact surfaces of the adjoining members is particularly critical for the maintenance of overall structural integrity of the traditional oriental timber structure. When friction between the mortise–tenon connections could no longer withstand the large seismic force, amplified rocking and rotation intensity will eventually lead to inelastic deformation [4], [5].

Although mechanical models derived from the above test observations are in good agreement with the static test results [6]–[9], more verifications need to be carried out to find out if the assumptions made are well validated for dynamic tests. Hence in this study, the above mechanical models were subjected to two types of evaluations. In the first test, the models' assumptions and calculated values were cross-validated with the shaking table tests of complex bracket sets with various vertical loads, wood species and structural system designs. The calculated assumptions were then applied to conventional numerical modelling software to find out if the predicted weak points of the structures are in line with the test observations.

## 2 MATERIALS AND METHODS

### 2.1 Design of the specimens

In this paper, the corridor structure of the Dieh-Dou type timber frame was selected for study. The prototype design originated from an Entrance Hall of the Chung family ancestral complex in southern Taiwan. The entire building was rebuilt in 1930 using Taiwan red cypress (*Chamaecyparis formosensis* Mats.). However, due to rapid urbanization, the Ancestral hall complex was scheduled for demolition in the 1990s. In recognition of its intrinsic value, the entire timber components were dismantled and donated to the NCKU Department's laboratory for structural research purposes. The geometric dimensions of the test specimens were primarily based on the initial design of the front corridor frame section of the Entrance hall. As part of the corridor frame design (Fig. 1, dashed boxed-up region) is similar to other Dieh-Dou type internal main frame design (Fig. 2), Shu members along the dashed box region were reduced into simplified members so that the test results obtained from the revised test specimens could apply to a wider range of structural systems.

Based on the original timber members' geometric dimensions, Specimen 1 was assembled by recycling part of the structural members that had been dismantled from the Entrance Hall.

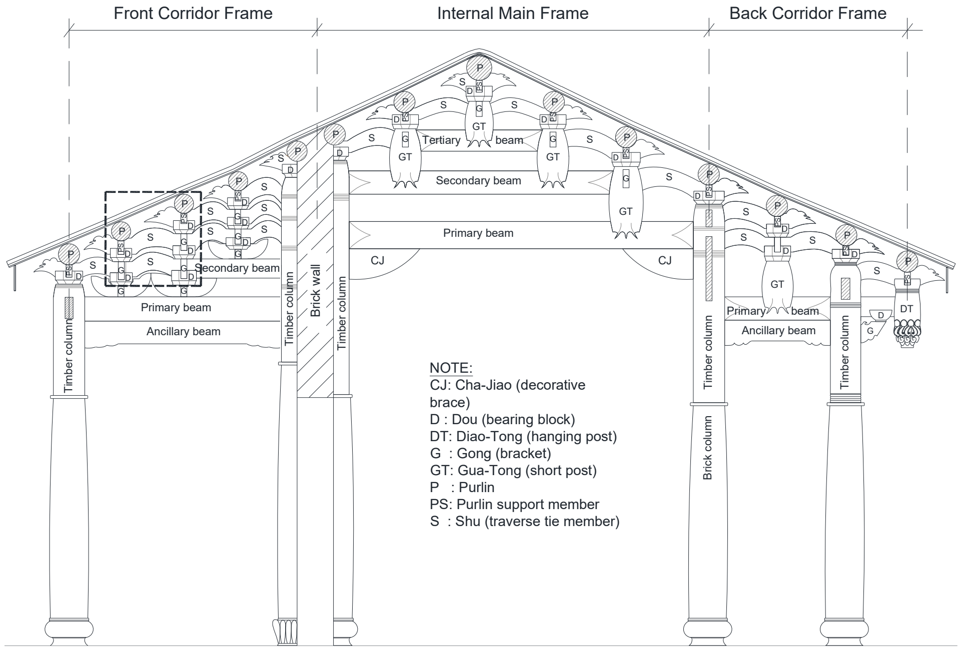



Figure 1: Initial design of the prototype building: Entrance hall of the Chung family ancestral complex.

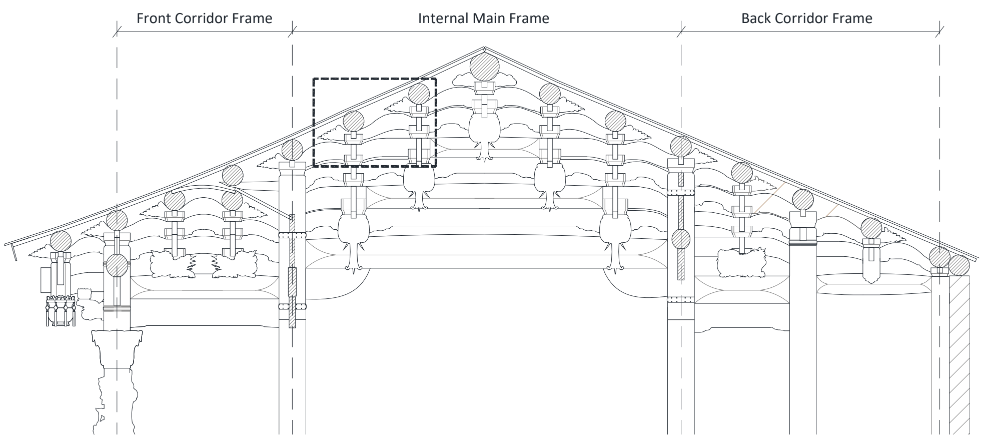



Figure 2: Similar complex bracket design commonly observed in Dieh-Dou type internal main frame.

Apart from the Dou members, all the other members were either repaired or fabricated usingthe same wood material from other parts of the Entrance hall (Fig. 3). The structural form of Specimen 1 was asymmetrically-designed, composed mainly of two identical sub-units of structural members. The geometric design of Specimens 2 and 3 were largely based on Specimen 1. Newly-fabricated using China Fir (*Cunninghamia lanceolata*), Specimen 2 followed an asymmetric design while Specimen 3 took on a symmetrical form (Fig. 4).

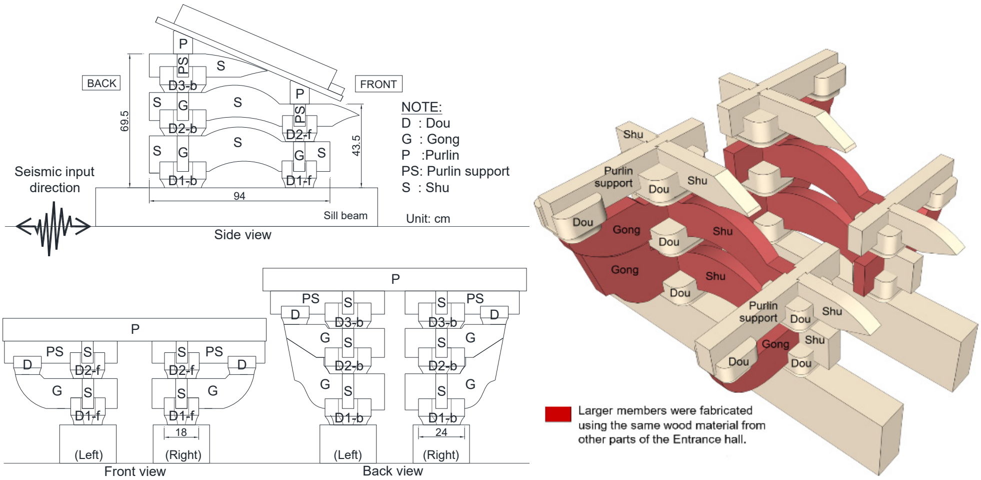



Figure 3: Details of Specimen 1 (asymmetric) [4], [6].

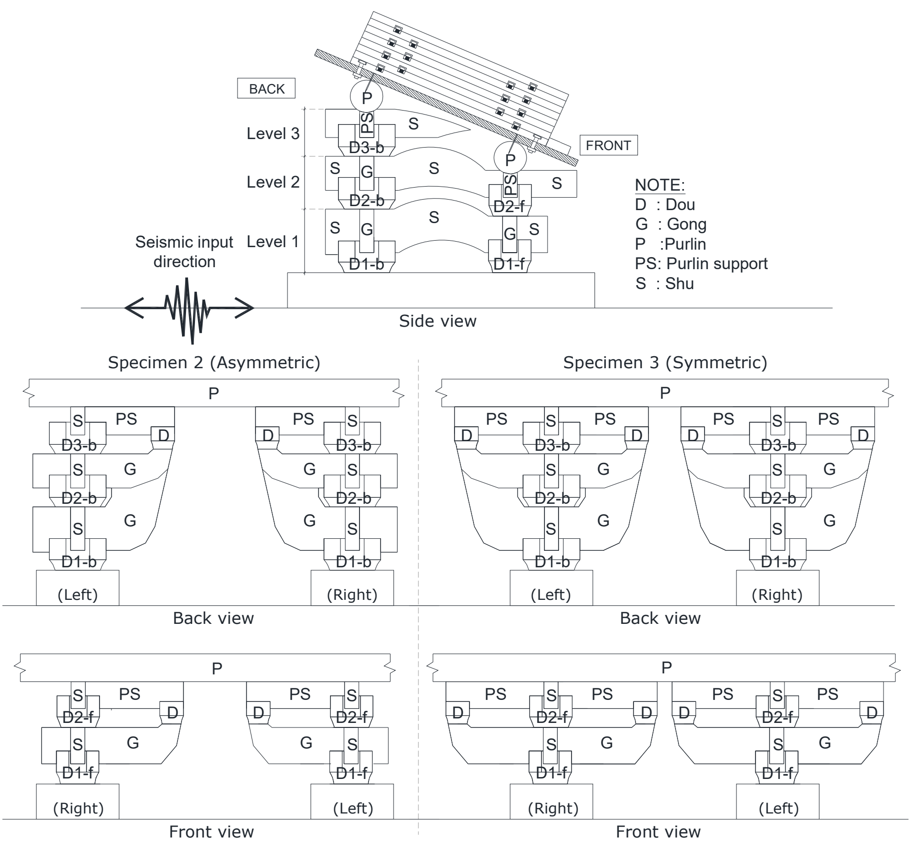



Figure 4: Details of Specimen 2 (asymmetric) and 3 (symmetric) [5].

2.2 Test methods

Although all three specimens were subjected to shaking table tests [4], [5] under various combinations of vertical loads and seismic intensities, the test protocol of Specimen 1 [4] was different from Specimens 2 and 3 [5]. Specimen 1 was the first pilot study that was conducted to understand the dynamic properties of the asymmetric structure under different combinations of vertical loads (5, 10 and 15 kN) at the elastic stage and seismic intensities of 0.11 g, 0.22 g and 0.33 g. With better understanding of the structural behaviour of the complex bracket gathered from the pilot study, another round of shaking table tests was initiated to study the dynamic behaviour of the symmetric and asymmetric forms under different combinations of vertical loads (17, 26 and 35 kN) and seismic intensities. The initial intention was to use the Chi-Chi time history record (TCU 084 EW component) with Peak Ground Acceleration (PGA) of 0.99 g (Fig. 5), however due to on-site facility limitations, the decision was made to downscale the seismic intensities starting from 0.16 g, 0.34 g, 0.48 g, 0.64 g to 0.80 g (Fig. 5). An overview of the test details of the specimens are summarized in Table 1.

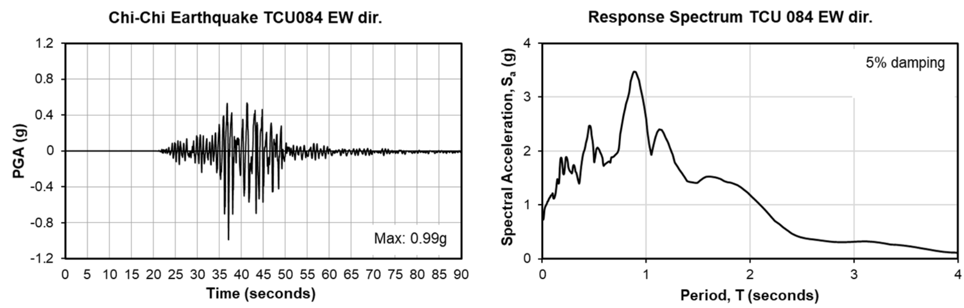



Figure 5: Time history and response spectrum used for the dynamic tests.

Table 1: Basic details of the test specimens for this study.

| Specimen | Structural form | Wood species used | Vertical loads/kN | Seismic inputs/g | | Reference |
|---|---|---|---|---|---|---|
| | | | (Steel + wood) | Previous tests | This study | |
| 1 | Asymmetric | Taiwan Red Cypress | 5.15<br>10.05<br>14.95 | 0.11<br>0.22<br>0.33 | 0.33 | [4] |
| 2 | Asymmetric | China Fir | 17.53<br>26.36<br>35.19 | 0.16<br>0.34<br>0.48<br>0.64<br>0.80 | 0.34 | [5] |
| 3 | Symmetric | China Fir | 17.62<br>26.45<br>35.28 | | | |

In the first part of the study, seismic intensity of around 0.33 g was chosen to evaluate how well the prediction models can fit with existing test results under different combinations of vertical loads (approximate range from 5 to 35 kN), wood species and structural system designs. By applying the various types of spring and embedment concepts [6], the aim of the second part of the study is to investigate how well the calculated values can be used to predict the weak points using conventional structural analysis software. Using Ansys structural analysis software, one sub-unit of the asymmetric specimen was subjected to a uniform vertical loading of 15 kN and a constant feed of lateral force beginning from 0 kN and gradually increasing to a maximum target of 15 kN. Outcomes from the three stages of the lateral force loading, that is, 5, 10 and 15 kN, were extracted to find out if the predicted stress distribution of each structural component coincides with the damage patterns as observed in the previous shaking table tests.

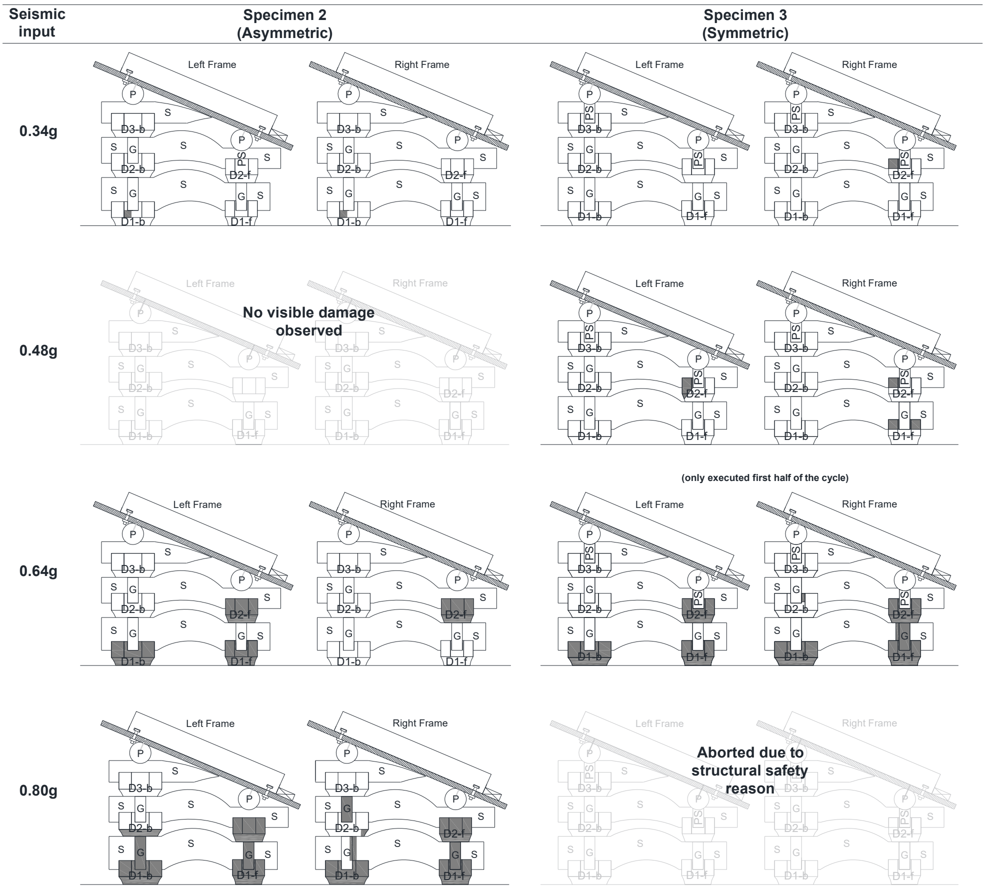
## 3 RESULTS AND DISCUSSION

### 3.1 Evaluation of mechanical models with test results

Table 2 provides an overview of the damage pattern of Specimens 2 and 3 from past shaking tests [5]. The first sign of visible damage was generally observed in the Dou members of Specimens 2 and 3 when subjected to 0.34 g seismic intensity. In the case of asymmetric-designed Specimen 2, the damage pattern began from the back bottom Dou members (D1-b) and as the seismic intensities increased beyond 0.34g , the damage extended to the front row (D1-f) and subsequently moving upwards to all the other members. As for the case of the symmetric-designed Specimen 3, the damage pattern began from the front row upper Dou members (D2-f) instead. As seismic intensities increased, the damage trend moved downwards to the front bottom Dou members (D1-f), extending to the back bottom Dou members (D1-b) and eventually spreading to rest of the upper members. Detailed record of the damage trend can be found in [5] and will not be repeated in this paper. With reference to the deformation patterns of past static and dynamic tests [4]–[9], a mechanical model based on the semi-rigid spring model concept proposed by Yeo et al. [6] was applied onto the shaking table test results.

As noted from the hysteresis loops, increase in vertical load has significant impact on the overall stiffness of the structure. Traditionally, oriental timber joints perform best when they are being subjected to heavy roof load. However, when a relatively lighter roof load is acting on top of the timber structures, the timber components tend to govern the global stiffness of the structure instead, resulting in a “softer” and more ductile frame during shaking. This is particularly seen in all cases of Specimen 1 (Fig. 6) and the 17 kN vertical load cases of Specimens 2 (Fig. 7) and 3 (Fig. 8) where a flatter hysteresis loop was exhibited when the vertical load is 17 kN and below. As the vertical load increases from 26 kN and upward, the heavier roof load started to exert its influence on all the timber joints, subsequently causing the global stiffness of the entire structure to increase significantly, as shown from the steeper hysteresis loops of 26 kN and 35 kN vertical loading cases of Specimens 2 and 3 (Figs 7 and 8). The predicted models presented in Specimen 1 (Fig. 7) appeared to be slightly over-estimated. This could be due to the above-mentioned “lighter roof effect” where the timber joints were not as tight as anticipated, resulting in a softer structure with lowered joint stiffness. Prediction models for Specimen 2 and 3 (Figs 7 and 8) fit relatively well, thus implying that the assumptions made for the two specimens are generally in good agreement.

Table 2: General damage patterns of Specimens 2 and 3 from past dynamic tests [5].




NOTE: D : Dou member; G : Gong member; P : Purlin member; PS: Purlin support member; S : Shu member; : Damage region

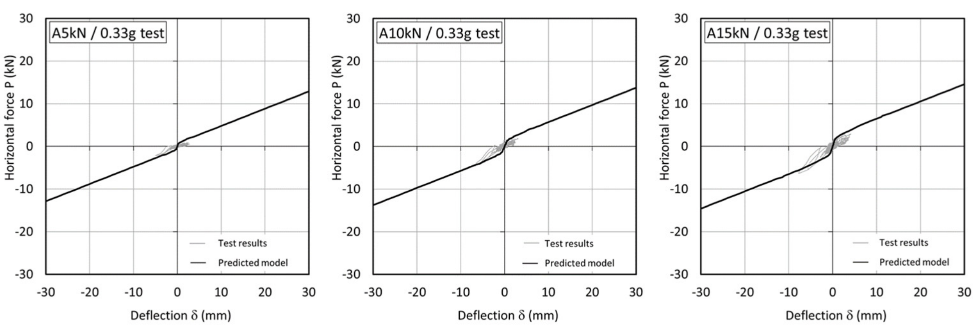



Figure 6: Comparison of predicted models with test results – Specimen 1 (asymmetric).

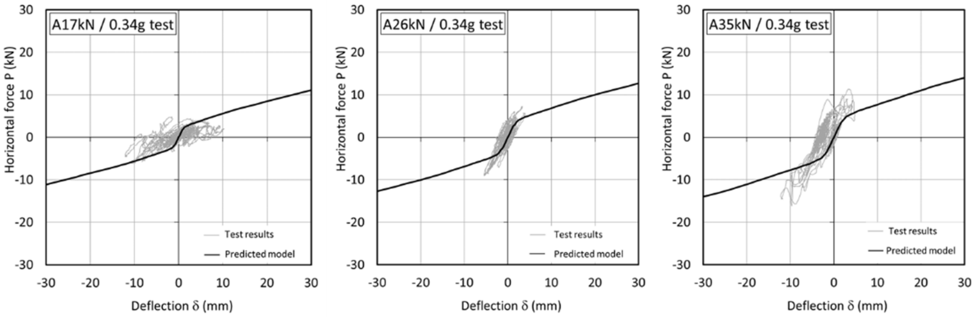



Figure 7: Comparison of predicted models with test results – Specimen 2 (asymmetric).

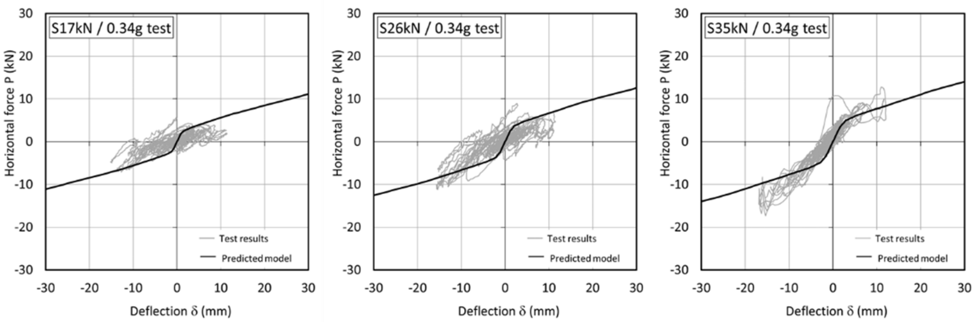
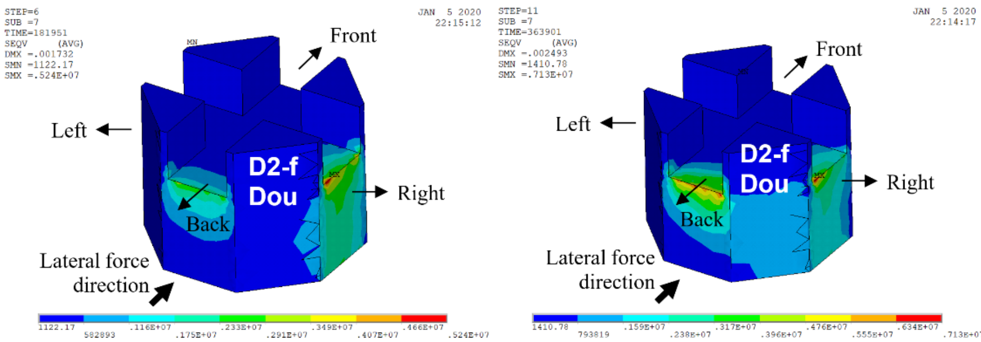



Figure 8: Comparison of predicted models with test results – Specimen 3 (symmetric).

## 3.2 Comparison of anticipated weak points with actual damage observations

With reference to hysteresis loops and test observations of both static and dynamic tests [6], [11], the elastic range of the complex brackets falls around 2 to 5 kN before entering into first yielding stage. The structure then moved on to the elasto-plastic stage with an approximate range between 5 and 10 kN. Signs of visible deformation, either in the form of shear fracture (horizontal, vertical or diagonal types) or friction-induced partial embedment usually will be manifested during this stage [5], [6], [11]. Base on the above observations, the initial plan was to extract three stages of the lateral force loading, namely 5, 10 and 15 kN, to find out if the predicted stress distribution of each structural component will coincide with the up-coming damage patterns as observed in the shaking table tests. However, numerical analysis terminated when lateral force loading reach 14 kN as the Ansys modelling software concluded that the structure has already undergone permanent structural damage. Hence, only results from the 5 and 10 kN lateral loading can be generated.

Fig. 9 presents a typical stress distribution prediction of the Dou structural member when subjected to 5 and 10 kN of lateral loading. As the Dou members are usually the first structural component to be damaged, more emphasis will be placed on the Dou members in this study.




(a) 5 kN lateral force (b) 10 kN lateral force

Figure 9: Stress distribution prediction of the Dou member when subjected to vertical load of 15 kN and lateral force from 5 kN to 10 kN: Front bottom Dou (D2-f)

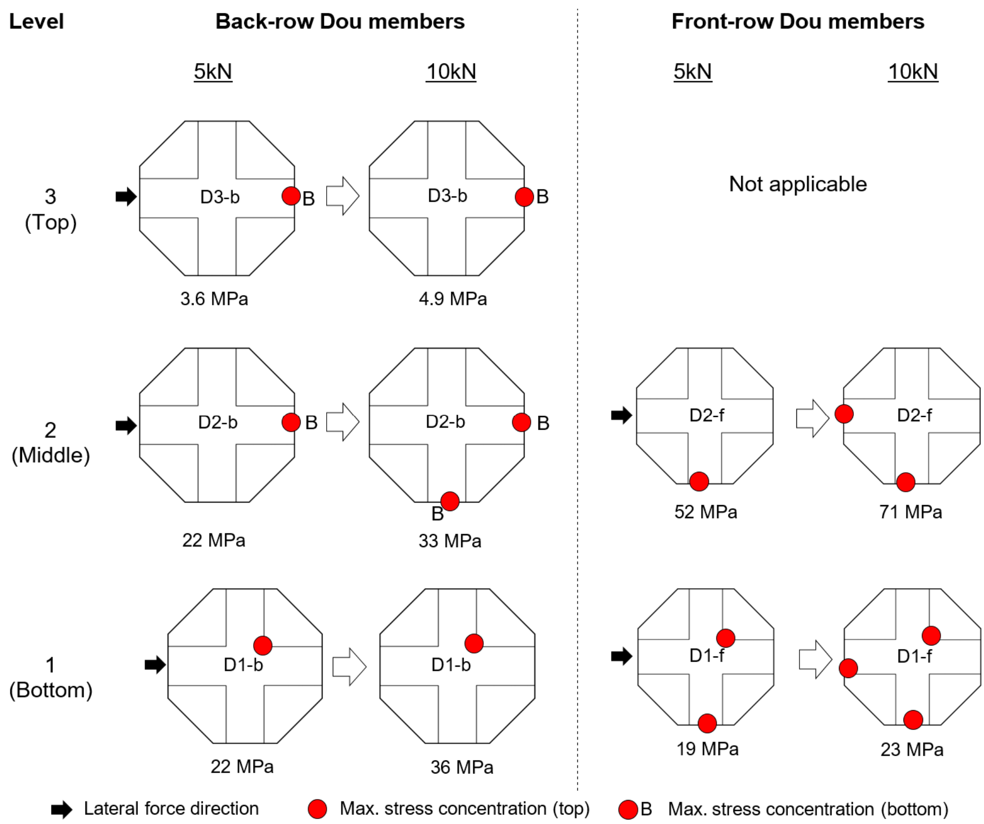



Figure 10: Numerical modelling prediction of maximum stress region of the front and back row Dou members.

The maximum stress concentration prediction for the Dou members ranges from 3.6 to 71 MPa, with Level 1 Back row Dou (D1-b) and Level 2 Front row Dou (D2-f) having a peak value of 36 MPa and 71 MPa at 10 kN, respectively. An overview of the maximum stress distribution patterns of the Dou members when subjected to lateral force loading from 5 to 10 kN is summarized in Fig. 10. From Fig. 10, the first Dou member to have the highest stress value is D2-f. The stress distribution sequence (Figs 9 and 10) also matches relatively well with actual damage pattern of the symmetric specimen (Table 2) where the first deformed region is around the right side and subsequently moving clockwise to the back. Similar damage patterns are also observed in Level 1 Front row Dou (D1-f) and Level 1 Back row Dou (D1-b). In the case of D1-f Dou, stress points first concentrate around the right side and central region before moving clockwise to the back; whilst for the D1-b Dou case, stress regions mainly concentrate along the central region. On the whole, the predicted stress regions are in agreement with the damage patterns observed in the symmetric specimen case; the calculated values and assumptions made are valid in general. As this trial test is only based on the modelling of one sub-unit of the entire specimen, the stress distribution prediction might tend to be skewed towards one side of the specimen. Modelling of a complete two sub-unit specimen will be carried out in future to make the numerical study more comprehensive.

## 4 CONCLUSIONS

This paper attempts to further verify the mechanical models derived from past works to evaluate if the assumptions made are applicable for the dynamic tests. The models were subjected to two types of evaluations and the following conclusions can be drawn:

- Although the mechanical models tend to over-estimate the joint stiffness of Specimen 1 slightly when lighter roof loads of around 15 kN and lower were applied, the mechanical models for Specimens 2 and 3 are in good agreement with the shaking table test results, thus implying that the assumptions made for the specimens are valid in general, but only up to the post-yielding loading level.
- Preliminary results from the above test trial revealed that the assumptions and calculated values applied for the numerical analyses are generally in line with the shaking table test results, and that most of the predicted weak points of the complex bracket structure, mostly concentrating around the Dou members, coincide well with on-site test observations.

## ACKNOWLEDGEMENTS

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