

# LARGE-SCALE IN-PLANE TESTING OF TIMBER-FRAME DIAPHRAGMS AND COMPARISON OF PERFORMANCE WITH DESIGN MODELS

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**ABSTRACT:** Large-scale diaphragm tests representative of current Australian practice for timber-framed buildings are undertaken to determine the in-plane ultimate capacity, midspan displacement, and failure mechanism. Tests are conducted for two diaphragm specimens with a span of 6.3 m and a width of 2.4 m. The specimens are constructed using particleboards fastened to timber trusses with metal webs using screws. The observed failure mode included splitting of the top chord of the timber trusses, shear failure of screws, tearing-out failure of particleboard near edge screws, and flexural failure of the particleboard. The shear capacity and displacement of the diaphragm specimens are compared with existing design models, including principles of mechanics using the sheathing-to-framing connection shear strength. The mean strength prediction using the connection shear strength values over predict the ultimate load recorded in the experiments. Further, the midspan displacement predicted by the design equation developed for blocked diaphragms needs to be amplified to match the displacements observed in the experiment.

KEYWORDS: timber, frame, diaphragm, in-plane

## **1 – INTRODUCTION**

Timber-framed floor and roof diaphragms consists of a sheathing material, such as plywood and particleboard, fastened to timber framing members. They are designed to resist vertical loads and to transfer lateral loads caused by wind, seismic, impact and other actions, to their supporting elements.

The in-plane load transfer mechanism of timber-frame diaphragms is dependent on the shear flow around the framing. The two key design methods for analysing diaphragms are the deep beam analogy and the equivalent truss method. Most design codes and guidelines adopt the deep beam analogy, where the sheathing acts as the web and the edge framing (referred to as chords) are the flanges of the beam. The beam analogy is suitable for regular geometries, The equivalent truss method is more challenging and sensitive to assumptions, however, it may be more suitable for complex geometries. Furthermore, finite element modelling methods may also be used to predict the performance of timber diaphragms although it can be very computationally time consuming. While the design methodologies are useful, some design standards rely on large scale diaphragm in-plane experimental test results as it ensures greater confidence in determining the diaphragm capacity and stiffness. This is because the response of diaphragms is dependent on numerous factors which are not necessarily sufficiently accounted for by design models. Some of the key factors include diaphragm sheathing panel type, thickness and layout, fastener type and spacing, blocking members between joists, joist type, especially width and depth of joist members where the fasteners are installed, diaphragm aspect ratio, boundary conditions, configuration and loading.

Most design standards that use experimental tests results for determining the diaphragm performance are based on tests that are not representative of modern-day construction materials and methods and therefore are not applicable. This paper presents the results of large-scale testing of typically constructed diaphragms in Australia with particleboard sheathing fastened using screws to the timber trusses with metal webs. Two series of tests are conducted for a 2.4 m by 6.3 m diaphragm with 19 mm thick particleboard sheathing.

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The shear capacity of the diaphragms obtained from the experiments is compared with design models, including the capacity predicted using principles of mechanics where the sheathing-to-framing shear connection values are obtained from connection tests.

Furthermore, the midspan displacement of the diaphragms recorded during the experiment is compared with predicted values from design guidelines which have been developed for blocked diaphragms.

The results suggest that current mechanic-based design models need to be modified to more accurately predict the inplane performance of diaphragms. This is the on-going aim and work of the authors.

The accurate prediction of timber diaphragm performance is becoming more important as the use of timber is being extended for mid-rise residential and commercial buildings.

# 2 - BACKGROUND

Significant amount of research about the in-plane performance of diaphragms initiated from the 1950s in America [1-4]. The work undertaken by many researchers has led to the development and expansion of the shear capacity tables and stiffness values for diaphragms with various parameters and configuration in the International Building Code [5] and the Special Design Provisions for Wind and Seismic (SPDWS) by the American Wood Council [6]. Similar design tables are available in Australia in the Engineered Wood Products and Association of Australasia (EWPAA) design guide [7] which is based on American research and suitable modifications for Australian conditions. Currently, the design tables in the EWPAA guide are limited to plywood sheathing nailed to solid-sawn lumber joists.

Some design standards (such as Eurocode 5 [8]) allow the shear capacity of the diaphragms to be determined using the shear capacity of the sheathing-to-framing connection capacity, shear strength of the sheathing panels, and the axial capacity of the chord members, with the assumption that other failure modes will not take place. Furthermore, they require full blocking between joists, such that all of the edges of the sheathing panels are connected to framings members (joists or blocking members).

Further, design standards and guidelines (such as SPDWS [6], NZS AS 1720.1 [9] and WS TDG 35[10]) provide equations for obtaining the in-plane displacement of diaphragms. However, these design models have certain assumptions, including fully blocked and chorded diaphragms, geometric limitations, loading and boundary conditions which are not always applicable.

In Australia, typical timber-frame construction for floors can include open-web joists such as trusses with timber or metal webs, as well I-joists. The sheathing material is typically particleboard, and it is becoming more common to use selftapping screws instead of nails to fasten the sheathing to the framing.

It is necessary to evaluate the performance of current timberframed diaphragms. In particular, there are concerns about the potential of splitting of the top chords of the trusses and the Ijoists. Further, diaphragms in Australia are typically designed without any blocking, thus they are weaker and more flexible, and the sheathing panel may be vulnerable to buckling.

## **3 – PROJECT DESCRIPTION**

Two large-scale diaphragm tests representative of current Australian practice for timber-framed buildings are undertaken to determine the in-plane ultimate capacity, midspan displacement, and failure mechanism.

The performance obtained from the test results is then compared with the predicted response in accordance with current design models. Since the in-plane performance of the diaphragms is highly dependent on the sheathing-to-framing connections, shear connection tests using the same material as the diaphragm specimen were also conducted to obtain the load-deformation response. Details of the connection testing are not provided in this paper; however, the results are used to determine the load-deformation response of the diaphragms.

#### 4 – EXPERIMENTAL SETUP

The diaphragm specimens are 2.4 m in width and 6.3 m in length. Yellow tongue particleboards which are 19 mm thick are used as the floor sheathing in a staggered configuration with the length of the boards running perpendicular to the direction of loading. Two sheathing sizes are investigated: 600 mm x 1800 mm and 600 mm x 2700 mm which have length-to-width aspect ratios typically used in Australian buildings.

The sheathing is fastened to the joists using fully threaded #10 screws 50 mm long. The screws were installed using a drill. The installation of the particleboards, including the spacing of the screws was done in accordance with the manufacturer's installation guide [11].

The joists consist of timber trusses made with 90 x 35 MGP10 timber with metal webs and a total depth of 290 mm. The joists are positioned at 450 mm spacing. 90 x 35 MGP10 timber was also used as the chords of the diaphragms which represent the wall top plates.

ASTM E455-19, the *Standard Test Method for Static Loading* of Framed Floor or Roof Diaphragm Construction for Buildings [12] was used as guidance to develop the test setup for the four-point static bending test.

Lateral restraints were provided at the support and at midspan to minimise out-of-plane actions.

The applied monotonic load and midspan displacement of the diaphragm is continuously measured during the test. The schematic of the test setup for specimen 1 is shown in





Figure 1. Schematic of diaphragm test setup.



Figure 2. Photographs of test setup, front and back view.

## 5 – RESULTS

#### **2.1 EXPERIMENTAL RESULTS**

Specimen 1 (with 600 mm x 1800 mm sheathing panels) reached a maximum load of 41 kN, and Specimen 2 (with 600 mm x 2700 mm sheathing panels) reached a maximum load and 44 kN. The observed failure at the end of the testing included: (i) splitting of the top chord of the timber trusses, (ii) shear failure of the screws which were visible near where splitting of the top chord had taken place as well as other places, also, many more became apparent during disassembly of the specimen (i.e., unscrewing the particleboards from the

joists), (iii) tearing-out failure of particleboards near edge screws, and (iv) flexural failure/rupture of particleboards.

The load deformation response of the diaphragm specimens is shown in Figure 3 with photographs of the failure observed.

From the load-deformation response, it is apparent that the two diaphragm have a similar initial stiffness. At an applied load of approximately 15 kN and larger, specimen 2 displays a stiffer response compared to specimen 1. This may be due to the larger size sheathing panels and hence less joints between the panels resulting in a stiffer response. Specimen 2 also fails at a lower midspan displacement. At maximum load,



specimen 1 reached a midspan displacement of 66 mm and specimen 2 reached 47 mm.

Figure 3. Load-deformation response of the diaphragm specimens and photographs of the observed failure

#### 2.1 PREDICTIONS BY DESIGN MODELS

The shear strength of the diaphragm can be approximated using principles of mechanics based on the deep beam analogy. The shear strength is typically taken as the smaller of the shear strength calculated based on the (i) sheathing-toframing connection strength, (ii) sheathing panel shear strength, and (iii) the tensile strength of the chords (which is lower than the compressive strength of the chords). The expressions for the above are provided below and further details can be found in [13, 14].

The equations provided below to obtain the diaphragm shear strength assume that the diaphragm is simply supported and subjected to a uniformly distributed load (w), where the total load on the diaphragm ( $F_d$ ) is given by the product of wL, where L is the length of the diaphragm:

• The shear strength based on the sheathing-toframing connection strength  $(F_{d,l})$  is given by (1), where *B* is the diaphragm depth, i.e., the dimension of the diaphragm parallel to the direction of the load,  $R_{d,f}$  is the shear design resistance per fastener, and  $s_f$ is the fastener spacing at the edges of the panels.

$$F_{d,l} = (2B) R_{d,f} / s_f$$
 (1)

• The shear strength based on the sheathing panel shear strength ( $F_{d,2}$ ) is given by (2), where *t* is the thickness of the panel, and  $f_{d,v}$  is the design panel shear strength.

$$F_{d,2} = (2tB) f_{d,\nu} \tag{2}$$

• The shear strength based on the chord member strength in tension is given by (3), where  $f_{d,t}$  is the tensile design strength of the chord members, and *A* is the cross-sectional area of the chord member in tension.

$$F_{d,3} = (f_{d,t}A)(8B) / L$$
(3)

Using the equations (1) to (3), the shear strength is predicted for the diaphragms tested, with suitable modifications to represent a diaphragm which is simply supported and subjected to two-point loads. The relevant diaphragm properties and results are shown in Table 1. Since, the aim here is to compare the calculated results with experimental results, where suitable, the mean properties (instead of design properties) of the diaphragm members are used. The mean resistance for the fastener  $(R_f)$  is calculated based on sheathing-to-framing experimental tests conducted by the authors. The mean tensile strength for the chords is approximated based on the characteristic value given in AS 1170.1:2010 [15] for MGP10 multiplied by a factor of 1.5 to convert from characteristic to mean strength. Furthermore, the shear strength of the particleboard is taken as 3.4 MPa based on the lower limit of the reported values in the literature for particleboards [16, 17]. Further testing is required to verify the values, especially for the particleboards that are used in Australia.

The strength predictions for the two diaphragm specimens tested is the same as the capacity equations do not take into account the dimensions of the sheathing panels. The results in Table 1 suggest that the expected failure mode is the sheathing-to-framing connections, with a total shear capacity of 77 kN. Hence, the maximum load observed in the test is only approximately 53% to 57% of the calculated capacity. The overprediction of the capacity is likely due to the splitting of the top chords of the trusses governing the response of the diaphragm. Furthermore, the proposed equation for calculating the shear capacity based on the sheathing-toframing connection is most suitable for diaphragms which are blocked.

Table 1: Diaphragm properties and calculated shear strength based on design models

Diaphragm properties		
B, diaphragm depth	2400	mm
L, diaphragm length	6300	mm
t, sheathing panel thickness	19	mm
a, distance of point load from support	2250	mm
R <sub>f</sub> , mean shear resistance perfastener	2400	Ν
$s_{f}$ , fastener spacing at the edges of the sheahitng panels	150	mm
$f_{\rm v},$ mean panel sheathing panel strength	3.4	MPa
ft,mean tensile strength of the chord members	11.6	MPa
A, coss-sectional area of the chord member in tension	3150	mm <sup>2</sup>
Shear strength of the diahragm		
F <sub>d,1</sub> , shear strength based on the sheathing-to- framing connection strength	77	kN
$F_{d,2}$ , shear strength based on the sheathing panel shear strength	310	kN
$F_{d,3}$ , shear strength based on the chord member strength in tension	78	kN

Equation (4) provides the expression for calculating the midspan displacement for a simply supported diaphragm subjected to uniformly distributed load, where v is the unit shear force induced by the design load, E is the modulus of elasticity of the chord member, A is the cross-sectional area of the chord member, G is the modulus of rigidity of the sheathing panel, t is the thickness of the sheathing panel,  $\alpha$  is the aspect ratio of the sheathing panel (i.e., the dimension of the panel perpendicular to the load divided by the dimension of the panel parallel to the load), *m* is the number of sheathing panels along the length of the chord member,  $e_n$  is the fastener slip at the unit shear force,  $\delta_c$  is the diaphragm chord splice at the unit shear force, and X is the distance of the chord splice to the nearest support. The equation can be derived based on the equation provided in NZS AS 11720.1:2022 [9]. It is similar to the four term equations provided in the IBC [5] and SPDWS [6], except that the fastener slip has been written as dependent to the aspect ratio of the sheathing panel, rather than assuming a constant ratio based on typical US sheathing panel size of 1.2m x 2.4m. The equation accounts for four sources of deflection, including flexural deformation in the chord members, shear deformation due to the sheathing

panels, shear deformation due to fastener slip, and deformation due to slip in chord-splices (if applicable).

Furthermore, equation (4) assumes that the diaphragm is blocked. It is expected that an unblocked diaphragm will experience 2.5 to 4 times the deflection of the same diaphragm with blocking [18, 19, Kessel and Schönhoff 2001 in 20].

$$\Delta_{dia} = \frac{5vL^3}{96EAB} + \frac{vL}{4Gt} + \frac{(1+\alpha)me_n}{2} + \frac{\sum \delta_c X}{2B}$$
(4)

The first two terms in (4) are modified so that it is suitable for a diaphragm subjected to two-point loads at a distance of a from the supports, as shown in (5).

$$\Delta_{dia} = \frac{v(3L^2 - 4a^2)}{12EAB} + \frac{va}{Gt} + \frac{(1+\alpha)me_n}{2} + \frac{\sum \delta_c X}{2B}$$
(5)

Hence, (5) is used to calculate the midspan displacement of the two diaphragm specimens. The fourth term is ignored as it did not significantly contribute to the midspan deflection. The fastener slip is obtained from the sheathing-to-framing connection tests undertaken by the authors. For the modulus of rigidity for the particleboards a value of 1000 MPa is adopted based on limited values in the literature [17, 21], and for the elastic modulus of the chord members a value of 10,000MPa is adopted based on the reported value in AS 1720.1:2010 [15] for MGP10. The other parameters are calculated based on the properties of the diaphragm specimens.

The comparison of the load-displacement response obtained during the experiment for the two specimens and the calculated response using (1) for predicting the load and (5)for the midspan displacement is provided in Figure 4. It can be seen that the calculated response significantly overpredicts the strength and stiffness of the two specimens tested. This is likely due to the limitations of the expressions, making them unsuitable for the specimens tested. Firstly, the equation for predicting the force capacity is based on a diaphragm where the failure mechanism is governed by the shear response of the sheathing-to-framing connections. While this is likely to have been the case at lower loads, it appears that the governing failure mechanism in the experiment was likely due to splitting of the top chords of the trusses, especially since a brittle failure was observed after the maximum load was reached. Secondly, the equations used are suitable for blocked diaphragms. The two specimens tested had no blocking members between the joists, hence a weaker and softer response was expected.

For comparison, the calculated deflection was amplified until the predicted response matched the load-deformation obtained from the test up until maximum load. The calculated deflection was multiplied by a factor of 5.5 for specimen 1, and 4.5 for specimen 2, in order to provide a better prediction of the load-deformation response.



Figure 4. Comparison of load-deformation response between experimental results and design models

## **6 - CONCLUSION**

This paper presents the test results for two-large scale diaphragms representative of Australian construction to determine the in-plane capacity, midspan displacement, and failure mode. The performance obtained from the test results is compared with the predicted response in accordance with current design models based on principles of mechanics. There are critical assumptions in the models, including that the failure mode is governed by the preferred failure mechanism due to shear failure of the sheathing-to-framing connections, and that full blocking is provided between the joist members ensuring that all four edges of the sheathing panel are connected to framing members (i.e., the joists or blocking members).

The results showed that the predicted diaphragm capacity, which also considered the shear strength of the panels and the axial capacity of the chord members, was significantly higher than the observed capacity in the experiment. The maximum load obtained in the experiment for the two specimens was 53% to 57% of the calculated capacity.

Furthermore, the deflection equations, which are also based on blocked diaphragms seem to underpredict the midspan displacement of the diaphragm specimens. For comparison, the calculated displacement was amplified until the predicted response matched the load-deformation obtained from the experiment up until maximum load. The calculated displacement was multiplied by a factor of 5.5 for specimen 1, and 4.5 for specimen 2, to provide a better prediction of the load-deformation response. The findings from this study suggest that a review of design models is necessary to ensure accurate determination of the performance of diaphragms representative of today's construction materials and methods. In particular, it is important that the design models take into account all possible sources of failure, including splitting of top chord of trusses and joists, and the design of unblocked diaphragms as they are typically constructed due to their ease of construction.

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