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Experimental testing of full-scale timber floor diaphragms in unreinforced masonry buildings

A.W. Wilson, P.J.H Quenneville, & J.M. Ingham

The University of Auckland, Auckland, New Zealand.

ABSTRACT: Unreinforced masonry (URM) buildings in New Zealand are typically constructed with rigid clay brick perimeter walls and comparatively flexible timber floor diaphragms. URM construction represents the predominant architectural heritage of our nation but the preservation of these buildings is threatened due their well established inadequacy to withstand earthquakes. Timber floor diaphragms are widely recognized to significantly influence the seismic response of URM structures and the accurate assessment of diaphragms is therefore crucial in any seismic assessment and retrofit of URM buildings. As part of a wider research program, a series of full-scale diaphragm tests were performed to generate the much needed data to critique the current state-of-the-art desktop procedures. In this contribution, the NZSEE and ASCE 41-06 procedures are used to predict full-scale diaphragm performance and are compared against experimentally determined values. It was found that inconsistency exists between the two assessment documents with considerable differences found in strength, stiffness and ductility predictions. The procedures published in NZSEE and ASCE 41-06 poorly predicted diaphragm response and require updated and representative values that include provisions to address the highly orthotropic nature of timber diaphragms. These documents should also be harmonized to ensure that transparency and consistency exists between international assessment procedures.

1 INTRODUCTION

Unreinforced masonry (URM) structures are widely-recognized to perform poorly in earthquakes. The 1931 Hawkes Bay earthquake (Dowrick 1998) and the more recent 2010 Darfield earthquake (Dizhur et al. 2010; Ingham and Griffith 2011) in New Zealand that severely damaged surrounding URM building stock are testament to their brittle nature and inability to dissipate hysteretic energy. URM construction remains prevalent throughout New Zealand and forms a considerable percentage of the building stock. The seismic strengthening of these structures to mitigate potential earthquake damage is therefore highly desirable for commercial as well as cultural reasons.

URM buildings in New Zealand are typically constructed with rigid clay brick perimeter walls and comparatively light timber floor diaphragms. These floors are generally made up of either straight-edge or tongue & groove floorboards nailed perpendicular to joists that span between the URM walls. When perimeter walls are close enough (approximately less than 6 m) joists sometimes span continuously between these elements. For larger spans, joists are typically lapped or butted, with or without a mechanical connection, over intermediate steel or timber cross-beams supported on columns. Diaphragm blocking and chord elements are almost never present, and timber cross-bracing is typically fitted intermittently between joists to prevent out-of-plane buckling. Joist ends are typically either simply supported on a brick ledge resulting from the perimeter walls reducing in width at each storey height, or pocketed into the wall to a depth equal to one brick width. Examples of timber floor configurations are given in Figure 1.

Timber floor diaphragms have routinely demonstrated significant influence on the seismic performance of URM structures (Bruneau 1994) but due a complete lack of experimental data and appropriate analysis (ABK 1981; Peralta 2003; Piazza 2008), the validity of diaphragm assessment

(a) Underside showing joists, sheathing and cross-bracing

(b) Joists pocketed into URM perimeter wall

Figure 1 – Typical timber diaphragm configuration (Australis House, Britomart, Auckland)

procedures offered in the current state-of-the-art assessment documents are questionable and require formal review. Results of a series of full-scale diaphragm tests are summarized and are used to review the validity of the current desktop assessment procedures published in NZSEE and ASCE 41-06.

2 FULL-SCALE DIAPHRAGM TESTING

2.1 Test description

A total of four diaphragm specimens labelled FS1a to FS4a, constructed with new pine timber, were tested. Each specimen measured 10.4 m x 5.535 m and was comprised of 135 mm x 18 mm straight edge floorboards nailed perpendicular to 45 mm x 290 mm joists spaced at 400 mm centres. Joists were orientated in the 5.535 m dimension. Cross-bracing was fitted between the joists at 1/3 joist length locations using 45 mm x 75 mm framing. Every floorboard-to-joist connection was fastened using two 75 mm x 3.15 mm bright power driven nails spaced at approximately 95 mm.

Diaphragms FS1a and FS2a were tested in the direction parallel to joists so that the diaphragm span to depth ratio was 1.88 to 1. In this direction the two side joists were bolted to steel frames that were fastened to the warehouse concrete slab, replicating in-situ boundary conditions where the edge joists would simply be bolted intermittently to the perimeter URM walls (Fig. 2a). Lateral loading was introduced into the diaphragm using a hinged steel frame on castors that distributed the hydraulic actuator point load into four equal point loads. Figure 2b shows that the loading frame comprised a main truss and two secondary beams that were capable of rotating with the deforming diaphragm. Reversed cyclic loading was achieved by positioning loaders on both ends of the loaded joists, and post-tensioning these together using threaded rods that spanned the depth of the diaphragm. The diaphragm was supported vertically on Teflon pads that allowed the diaphragm to deform laterally without measurable friction. Diaphragm FS1a (Fig. 2a) was a homogenous diaphragm while diaphragm FS2a (Fig. 2c) included a corner penetration measuring 3.2 m x 1.08 m that represented a typical stairwell present in many timber diaphragms.

Diaphragms FS3a and FS4a were tested in the direction perpendicular to joists so that the diaphragm span to depth ratio was 1 to 1.88. Realistic conditions were created for this set-up by constructing URM walls on each side of the diaphragm for the joists to pocket into (Fig. 2d), similar to that illustrated in Figure 1b. Sliding of the brick walls was prevented by post-tensioning the walls with threaded rods epoxied into the warehouse concrete slab. Loading was introduced into the diaphragm using the same hinged steel frame as for the previous set-up but with only two points of loading. Diaphragms FS3a and FS4a were equivalent in construction except that diaphragm FS3a had continuous joists spanning between the brick walls while diaphragm FS4a had discontinuous joists with a two-bolt lapped central connection.

(a) Diaphragm FS1a

(b) Loading frame set-up

(c) Penetration in Diaphragm FS2a

(d) Diaphragm FS3a/4a

Figure 2 – Experimental program

2.2 Instrumentation and loading schedule

During each test, total load was recorded using a load cell attached to the actuator, while the diaphragm deformation profile was measured at three locations using string potentiometers. In addition, the in-plane and out-of-plane displacement of the steel side frames was monitored during testing parallel to joists using strain ‘portal’ gauges. Each diaphragm was subjected to pseudo-static reversed-cyclic loading to midspan displacement amplitudes of 2.5 mm, 5 mm, 10 mm, 15 mm, 25 mm, 50 mm, 75 mm, 100 mm, and 150 mm. Each displacement amplitude was repeated three times to investigate the cyclic degradation of diaphragm performance.

2.3 Test results

Overall the diaphragms displayed flexible and highly nonlinear characteristics with low levels of hysteretic pinching, as illustrated in the force versus midspan displacement plots in Figure 3. The open hysteretic loops demonstrate that these diaphragms are capable of dissipating considerable amounts of energy when subject to lateral loading. No splitting of timber, nail tear-out or other failure mechanisms were observed during testing, even at displacements in excess of 150 mm, and all diaphragms appeared to remain completely serviceable at the conclusion of each test.

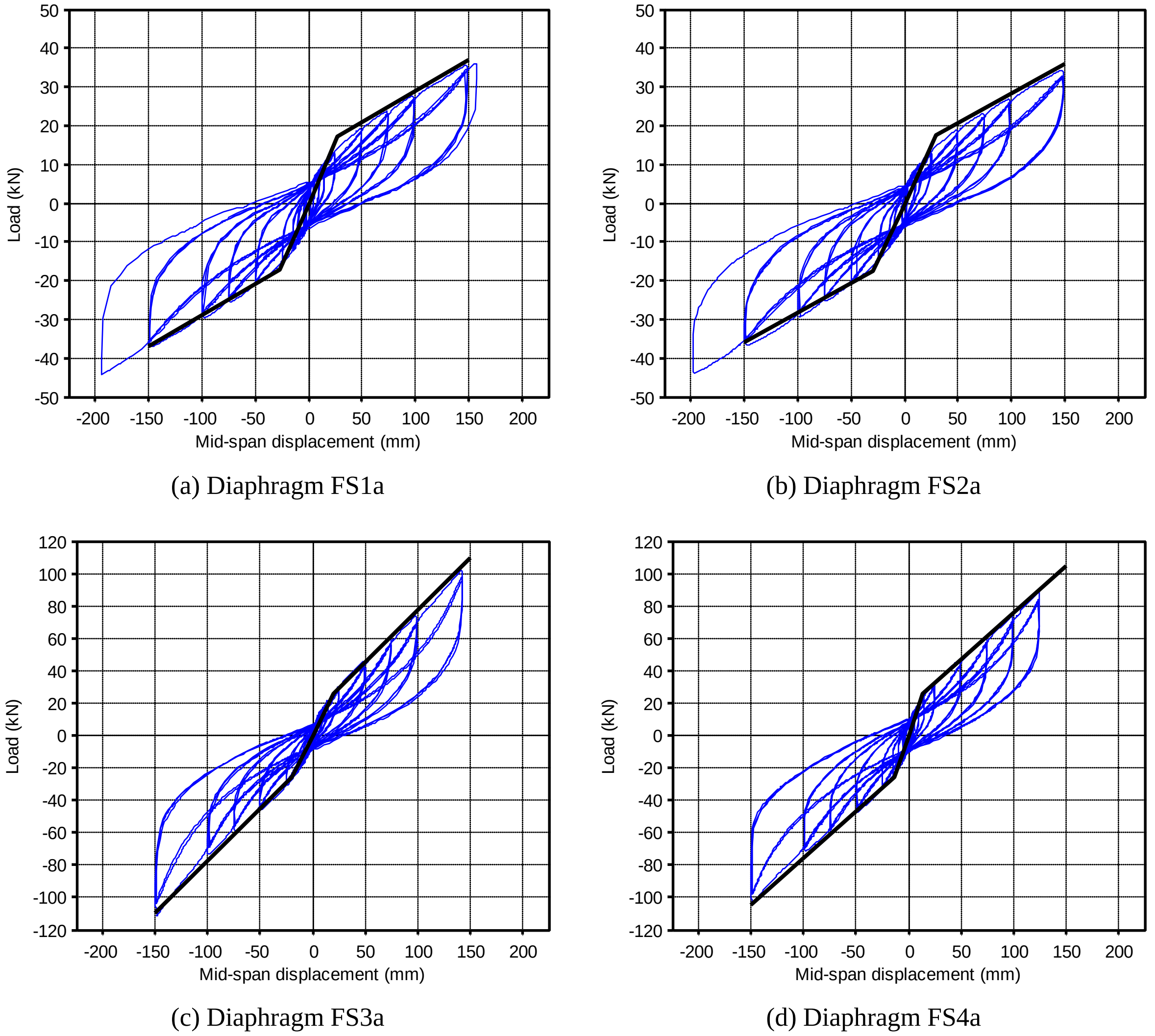
In order to determine comparable performance parameters it was necessary to appropriately characterize the force-displacement data of each test using a consistent and rational methodology. In the absence of a universally accepted method to characterize diaphragm response, this was achieved by using the principle of hysteretic energy conservation (Mahin and Bertero 1981) to develop a bilinear representation of the backbone force-displacement data.

In order to obtain a unique solution, the following constraints were applied to the bilinear curves:

- Must pass through zero load and zero displacement.
- Final displacement set to ±150 mm with corresponding load taken from the relevant backbone curve.
- Secondary stiffness (K_2) taken as the average gradient of the lines joining displacement amplitudes 75 mm, 100 mm and 150 mm on the relevant backbone curve. This portion of the backbone curves was essentially linear for all diaphragms tested, therefore justifying the constraint of its stiffness for the bilinear curve.

Figure 3 shows the bilinear curves produced for the tested diaphragms. The bilinear curves defined the yield force (F_y), yield displacement (Δ_y), maximum force at Δ = 150 mm (F_{max}), and initial and secondary stiffness (K_1 and K_2) values presented in Table 1 that were used to calculate the desired diaphragm performance parameters.

A comparison of the force-displacement responses of specimens FS2a and FS1a suggests that a small diaphragm opening such as a stairwell does not significantly affect diaphragm performance, with these responses being largely indistinguishable. It is appreciated that larger penetrations may worsen this effect however. Interestingly, FS4a was found to have higher initial stiffness than FS3a despite the



(a) Diaphragm FS1a

(b) Diaphragm FS2a

(c) Diaphragm FS3a

(d) Diaphragm FS4a

Figure 3 – Full-scale diaphragm test results with bilinear idealisations

Table 1 – Experimental bilinear values

	F_y [kN]	Δ_y [mm]	F_{max} [kN]	Δ_{max} [mm]	K_1 [kN/m]	K_2 [kN/m]
FS1a	17.2	26.6	36.8	150	647	159
FS2a	17.6	29.1	35.9	150	606	151
FS3a	25.4	19.6	110.0	150	1297	649
FS4a	25.7	13.9	104.3	150	1842	578

presence of joist discontinuities. Either the bolted connections added additional strength to the diaphragm or the effect of the joist discontinuities was minimised by zero shear force existing within the central region of the diaphragm, and the increased stiffness was a function of construction variation between tests. Further analysis of these results is currently being performed.

It is difficult to directly compare force-displacement response between the two principal loading directions due to the influence of diaphragm geometry. Diaphragms FS3a and FS4a demonstrated greater stiffness and strength than FS1a and FS2a, but this is clearly due to a considerably lower span to depth ratio. As a general observation, the hysteretic loops are larger in the direction parallel to joists which could result from greater engagement of the yielding nail couples in this direction as opposed to loading in the direction perpendicular to joists, where the nail couples are less engaged and the out-of-plane bending of the joists has greater influence on response.

3 DESKTOP ASSESSMENT PROCEDURES

Desktop assessment procedures should aid structural engineers by transforming complex loading and response mechanisms into quantifiable performance parameters that can used for design. It is understood that New Zealand practitioners currently refer to the NZSEE (2006) and ASCE 41-06 (2007) documents to perform seismic assessments of heritage timber floor diaphragms. To verify the accuracy of the assessment procedures published in these documents, predicted values of diaphragm stiffness, strength and ductility were compared against experimentally determined values, and are summarised in Table 2.

3.1 Strength

Diaphragm strength is conventionally calculated as shear strength per lineal meter depth of diaphragm, R_n. Appendix 11B of NZSEE describes a methodology to determine diaphragm shear strength from first principles using Equation (1) below:

$$R_n = \frac{Q_n s}{\ell b_s} \tag{1}$$

where Q_n = nominal nail capacity, s = nail couple spacing, ℓ= joist spacing, b_s = floorboard width.

Using the New Zealand Timber Structures Standard NZS 3603 (1993) or similar (ENV 1995-1-1 2004) to determine Q_n, the value of R_n can be readily found using diaphragm configuration parameters. A simple alternative to the above method is offered in the form of default shear strength values corresponding to different diaphragm configurations. The most relevant value for the tested diaphragms is suggested to be 6 kN/m, which considerably exceeds values calculated using Equation 1. It is not understood why such a large discrepancy exists.

Default shear strength values are tabulated for different diaphragm configurations in ASCE 41-06. The published value for straight-sheathed diaphragms is 350 kN/m, and in the absence of additional guidance, applies to all of the tested diaphragms.

3.2 **Stiffness**

Diaphragm stiffness K_d (kN/m) is typically presented as total diaphragm load per midspan displacement. NZSEE recommends that midspan displacement be calculated from first principles using the methodology detailed in Appendix 11A, as described by Equation (2):

$$\Delta = \frac{Le_n}{2s} \tag{2}$$

where L = diaphragm span, e_n = nail slip resulting from applied shear force V, s = nail couple spacing

From which diaphragm stiffness can be calculated using Equation (3):

$$K_d = \frac{F}{\Delta} \tag{3}$$

where F = force, Δ = midspan diaphragm displacement

This seemingly simple procedure is complicated by the determination of e_n which is unclear, and for which no explicit guidelines are provided. It is understood that engineering practitioners typically calculate e_n at the nominal nail capacity using NZS 3603 or similar, and relate this back to applied diaphragm shear force using Equation (1) above. The displacement and corresponding stiffness of the diaphragm at its strength capacity can subsequently be calculated. It is recognized that K_d is sensitive to the selection of V and is thus subjective to the designer's interpretation, further complicating the application and consistency of this procedure.

Rather than providing methodology to calculate diaphragm stiffness (K_d) directly from first principles, ASCE 41-06 has published default shear strength (G_d) values which are independent of diaphragm geometry and that are subsequently used to calculate diaphragm stiffness using Equation (4) below:

$$K_d = \frac{4BG_d}{L} \tag{2}$$

where B = diaphragm depth

For the sake of comparison, values of G_d have also been back calculated using Equation (4) for the diaphragm stiffness values determined using the NZSEE methodology.

3.3 **Ductility**

The NZSEE document provides no specific guidance for timber floor diaphragm ductility. It is understood that engineering practitioners typically apply a nominal ductility of $\mu = 4$ that is suggested in NZS 3603 for new timber structures.

ASCE 41-06 addresses diaphragm ductility capacity using component modification factors (*m*-factors) that account for the expected level of ductility at different structural performance limit states. The *m*-factors associated with the Life Safety Limit State are analogous to the conventional structural ductility factor μ used in most design procedures. For single straight-sheathed diaphragms, the published *m*-factor is 2.0 for span to width ratios less than three (which applies to all typical diaphragm geometries).

3.4 **Experimental values**

Diaphragm stiffness K_d was defined as initial stiffness K_1 while shear strength R_n was calculated by simply dividing yield force by two times the diaphragm depth. Ductility was determined using the conventional assumption of elastic-perfectly plastic response and calculating the ratio between maximum displacement and yield displacement.

Table 2 – Diaphragm performance parameters

	Shear strength, R_n [kN/m]				*Stiffness K_d* [kN/m]			*Shear stiffness, G_d* [kN/m]			*Ductility, μ*		
	NZSEE (i)	NZSEE (ii)	ASCE	Exp	NZSEE	ASCE	Exp	NZSEE	ASCE	Exp	NZSEE	ASCE	Exp
FS1a	1.4	6.0	1.75	1.6	207	745	647	97	350	304	-	2.0	7.4
FS2a	1.4	6.0	1.75	1.6	207	745	606	97	350	284	-	2.0	7.4
FS3a	1.4	6.0	1.75	1.2	730	2630	1297	97	350	173	-	2.0	7.7
FS4a	1.4	6.0	1.75	1.2	730	2630	1842	97	350	245	-	2.0	10.8

4 DISCUSSION

Overall, the guidelines offered in both assessment documents poorly predict diaphragm performance. The values listed in Table 2 illustrate that diaphragm shear strength, stiffness, shear stiffness and ductility are either under predicted or over predicted using the NZSEE and ASCE 41-06 assessment procedures. Shear strength is the most accurately predicted parameter with approximately 10% discrepancy from experimentally determined values, with the exception of the alternative default value offered by NZSEE that grossly over estimates strength. The reason for this large discrepancy remains unknown. Diaphragm stiffness and shear stiffness are considerably under predicted using the methodology in NZSEE, while are over predicted using ASCE 41-06 guidelines. NZSEE offers no explicit guidance for diaphragm ductility while ASCE 41-06 provisions were shown to under estimate diaphragm ductility by up to five times.

An important observation from the experimental performance parameters listed in Table 2 is the highly orthotropic behaviour demonstrated by the timber diaphragms. The shear strength and shear stiffness values, which are independent of diaphragm geometry, are significantly different in each principal direction of the diaphragm, yet the current assessment documents offer no provisions to address this behaviour. In order to improve the transparency and accuracy of the assessment procedures, diaphragm performance parameters should be explicitly provided for in each principal direction.

It is recognized that heritage diaphragm performance may differ from the experimental performance values presented in Table 2 due to out-dated construction materials and the effects of age and decay. A component of the current research program involves testing extracted floor sections and nail connections from ~100 year old timber floor diaphragms. It is hoped that the data generated from this testing will provide the necessary information to appropriately modify the performance parameters to ensure that they are representative for heritage construction. For the interim, the considerable difference observed between predicted and measured diaphragm performance suggests that the current procedures offered in NZSEE and ASCE 41-06 require updating. In addition, it is believed that these documents should be harmonized so that international assessment procedures are consistent with one another.

5 CONCLUSIONS AND RECOMMENDATIONS

Timber floor diaphragms have proven to significantly influence the overall seismic performance of URM buildings. Despite a long established demand for greater desktop assessment accuracy, communication with engineering practitioners indicates that confidence remains low with the procedures offered in the current state-of-the-art assessment documents NZSEE and ASCE 41-06 to predict diaphragm performance. The data generated from a series of full-scale diaphragm tests was analyzed and used to review the application and accuracy of these procedures. A comparison of predicted diaphragm strength, stiffness and ductility indicates that the NZSEE and ASCE 41-06 procedures are inconsistent and both poorly predict diaphragm performance. To improve accuracy, it is recommended that the current assessment procedures be updated with representative values and that provisions are included for each principal direction to address the highly orthotropic nature of timber diaphragms. It is also believed that international assessment documents should be harmonized to improve transparency and consistency.

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