

# Assessment of timber floor diaphragms in historic unreinforced masonry buildings

Aaron Wilson<sup>1</sup>, Pierre Quenneville<sup>2</sup>, Jason Ingham<sup>3</sup>

**Abstract** Unreinforced masonry (URM) buildings outside of Europe were typically constructed with rigid clay brick perimeter walls, and comparatively flexible timber floor diaphragms. URM construction represents the predominant architectural heritage of many nations but the preservation of these buildings in seismically active regions is threatened due their well established inadequacy to withstand earthquakes. Timber floor diaphragms are widely recognized to have significant impact on the overall seismic response of URM structures, and the accurate assessment of diaphragms is therefore crucial during the seismic assessment and retrofit of URM buildings. *NZSEE (2006) - Assessment and improvement of the structural performance of buildings in earthquakes*, and *ASCE 41-06 (2007) - Seismic rehabilitation of existing buildings* represent the current state-of-the-art in seismic assessment but the validity of the procedures associated with timber diaphragm performance remains uncertain, and a review of their application and accuracy is required.

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 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 require updated and representative values, and to include provisions to address the highly orthotropic nature of diaphragms that was identified from testing. It is also believed that these documents should be harmonized to ensure that transparency and consistency exists between international assessment procedures.

**Keywords** timber floor diaphragms, seismic assessment, heritage

## 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

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building stock are testament to their brittle nature and inability to dissipate hysteretic energy. Due to its popularity as a construction practice during early colonial times, many countries worldwide have considerable URM building stocks that remain in service and that often represent a critical component of a nation's architectural heritage. The seismic strengthening of URM buildings to mitigate potential earthquake damage is therefore highly desirable for commercial as well as cultural reasons.

While URM buildings outside of Europe typically comprise rigid clay brick perimeter walls, the floors are usually constructed of comparatively light timber framing. 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 often span continuously between these elements. For larger spans, joists are typically lapped or butted 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 floors in URM buildings act as diaphragms that have routinely demonstrated significant influence on the seismic performance of the complete URM structure due to their flexible nature and often inadequate connection to the perimeter URM walls (Bruneau 1994). The in-plane strength and stiffness of these diaphragms is therefore crucial to the overall performance of the URM building and accurate assessment of their performance is essential for the design of appropriate retrofitting techniques when seismic strengthening is required. Due to a complete lack of experimental data and appropriate analysis (ABK 1981; Peralta 2003; Piazza 2008), the validity of diaphragm assessment procedures offered in current state-of-the-art assessment documents such as *NZSEE (2006) - Assessment and improvement of the structural performance of buildings in earthquakes*, and *ASCE 41-06 (2007) - Seismic rehabilitation of existing buildings* are questionable. Furthermore, communication from engineering practitioners suggests that such documents are difficult to understand and to follow with confidence. Given these shortcomings, significant motivation currently exists to investigate the performance of timber floor diaphragms and the accuracy of assessment procedures. Before non-destructive testing techniques are considered as a viable assessment tool, it is necessary to generate in-plane performance data for timber diaphragms with which to appropriately update and improve current seismic assessment desktop procedures.

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. Performance predictions using the assessment techniques are compared against experimental results and where possible, recommendations to improve their accuracy are suggested.



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



(b) Joists pocketed into URM perimeter wall

**Figure 1** – Typical timber diaphragm configuration

## 2. AS-BUILT DIAPHRAGM TESTING

### 2.1. Test description

A total of four diaphragm specimens labeled 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 centers. 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-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 which replicated in-situ boundary conditions where the edge joists would simply be bolted intermittently to the perimeter URM walls (see Figure 2 (a)). Lateral loading was introduced into the diaphragm using a hinged steel frame ‘whiffle tree’ on castors that distributed the hydraulic actuator point load into four equal point loads. It can be observed in Figure 2 (b) that the loading frame was comprised of 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 loaders together using M16 threaded rods that spanned the depth of the diaphragm (Figure 2 (b)). The diaphragm was supported vertically by Teflon pads that allowed the diaphragm to deform laterally without measurable friction. Diaphragm FS1a shown in Figure 2 (a) was a homogenous diaphragm while diaphragm FS2a shown in Figure 2 (c) included a corner penetration measuring 3.2 m x 1.08 m that represented a typical stairwell present in many timber diaphragms.



(a) Diaphragm FS1a



(b) Loading frame set-up



(c) Penetration in Diaphragm FS2a



(d) Diaphragm FS3a/4a

**Figure 2 – Experimental program**

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, similar to that illustrated in Figure 1 (b). Sliding of the brick walls was prevented by post-tensioning the walls with M16 rods epoxied into the warehouse concrete slab (measurement of wall position during testing showed that no wall displacement was observed). 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.

## 2.2. Instrumentation and loading schedule

During each test, total load ( $F$ ) was recorded using a load cell attached to the actuator, while the diaphragm deformation profile was measured at three locations ( $d_1 - d_3$ ) using string potentiometers. In addition, the in-plane ( $S_{IP1}$  and  $S_{IP2}$ ) and out-of-plane ( $S_{OP1} - S_{OP4}$ ) displacement of the steel side frames was monitored during testing parallel to joists using ‘portal’ strain gauges. Figure 3 illustrates the location of test instrumentation.

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 4. 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.

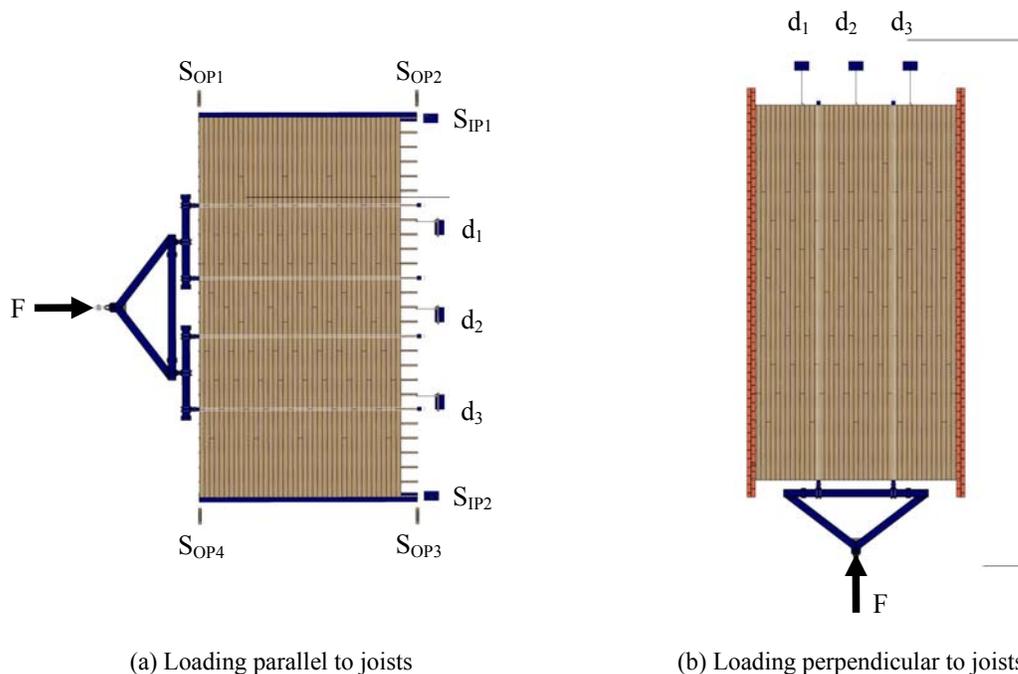
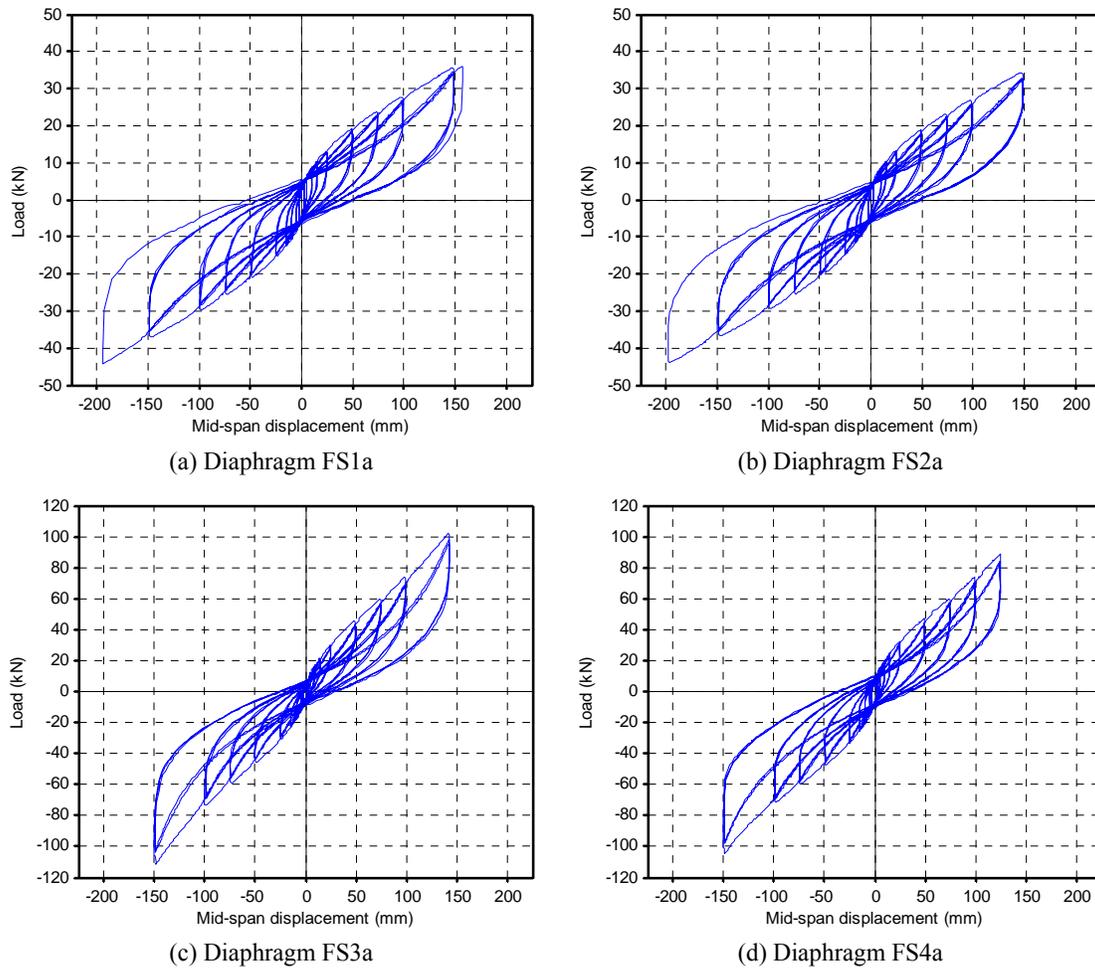


Figure 3 – Instrumentation locations



**Figure 4** – Full-scale diaphragm testing results

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 the diaphragm’s performance, with these responses being largely indistinguishable. It is appreciated that larger penetrations may worsen this effect however. Another interesting observation is that diaphragms FS3a and FS4a behaved analogously despite the presence of a central discontinuity in the joists in diaphragm FS4a. This could be explained by the deformation profile of the diaphragms displaying little amounts of relative displacement between the two point-load locations. With the majority of deformation occurring in the outer regions of the diaphragm, the curvature near the centralized joist connection was low and therefore considerably reduced its effect on diaphragm response.

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

The conventional purpose of desktop assessment procedures in structural engineering are to transform complex loading and response mechanisms into quantifiable performance parameters that can be used in design. In the case of timber diaphragms, designers require stiffness, strength and ductility values of

each floor diaphragm to not only assess the capacity and deformation of the diaphragm, but to also determine in-plane and out-of-plane URM wall loads that rely on diaphragm period, which is a function of diaphragm stiffness. It is clear then that although these procedures should remain simple and transparent, they also need to be accurate.

It is understood that New Zealand structural engineers currently refer to the NZSEE (2006) and ASCE 41-06 (2007) documents to perform desktop seismic assessments of heritage timber floor diaphragms. These documents are considered to be the current state-of-the-art in seismic assessment yet the specific origin of the guidelines published for timber diaphragms is unknown, and with little experimental data available, the validity of such procedures requires review. To verify the accuracy of such procedures, performance predictions of the full-scale diaphragms were determined using the NZSEE and ASCE 41-06 documents and compared against experimentally determined parameters.

### 3.1. NZSEE

#### 3.1.1. 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:

$$R_n = \frac{Q_n s}{\ell b_s} \quad (1)$$

where  $Q_n$  = nominal nail capacity,  $s$  = nail couple spacing,  $\ell$  = 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. The shear strengths calculated for tests FS1a to FS4a using this method are listed in Table 1.

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 grossly exceeds the shear strength determined from Appendix 11B. It is not understood why such a large discrepancy exists.

#### 3.1.2. Stiffness

Diaphragm stiffness  $K_d$  (kN/m) is not directly equated using NZSEE guidelines. Rather, diaphragm deformation is determined first, followed by the calculation of stiffness using Equation (2) below:

$$K_d = \frac{F}{\Delta} \quad (2)$$

where  $F$  = force,  $\Delta$  = midspan diaphragm displacement

Midspan displacement is calculated from first principles using the methodology detailed in Appendix 11A and described by Equation (3) below:

$$\Delta = \frac{L e_n}{2s} \quad (3)$$

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

This seemingly simple equation 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. The stiffness of each diaphragm tested was found using the above procedure and listed in Table 1.

### 3.1.3. 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.

## 3.2. ASCE 41-06

### 3.2.1. Strength

Diaphragm strength ( $R_n$ ) is determined from default shear strength values tabulated for different diaphragm configurations. The published value for single straight sheathed diaphragms is listed in Table 1, and in the absence of additional guidance, applies to all of the tested diaphragms.

### 3.2.2. Stiffness

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:

$$K_d = \frac{4BG_d}{L} \quad (4)$$

where  $B$  = diaphragm depth

Table 1 outlines diaphragm stiffness values calculated using this method for the tested diaphragms. 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.

### 3.2.3. Ductility

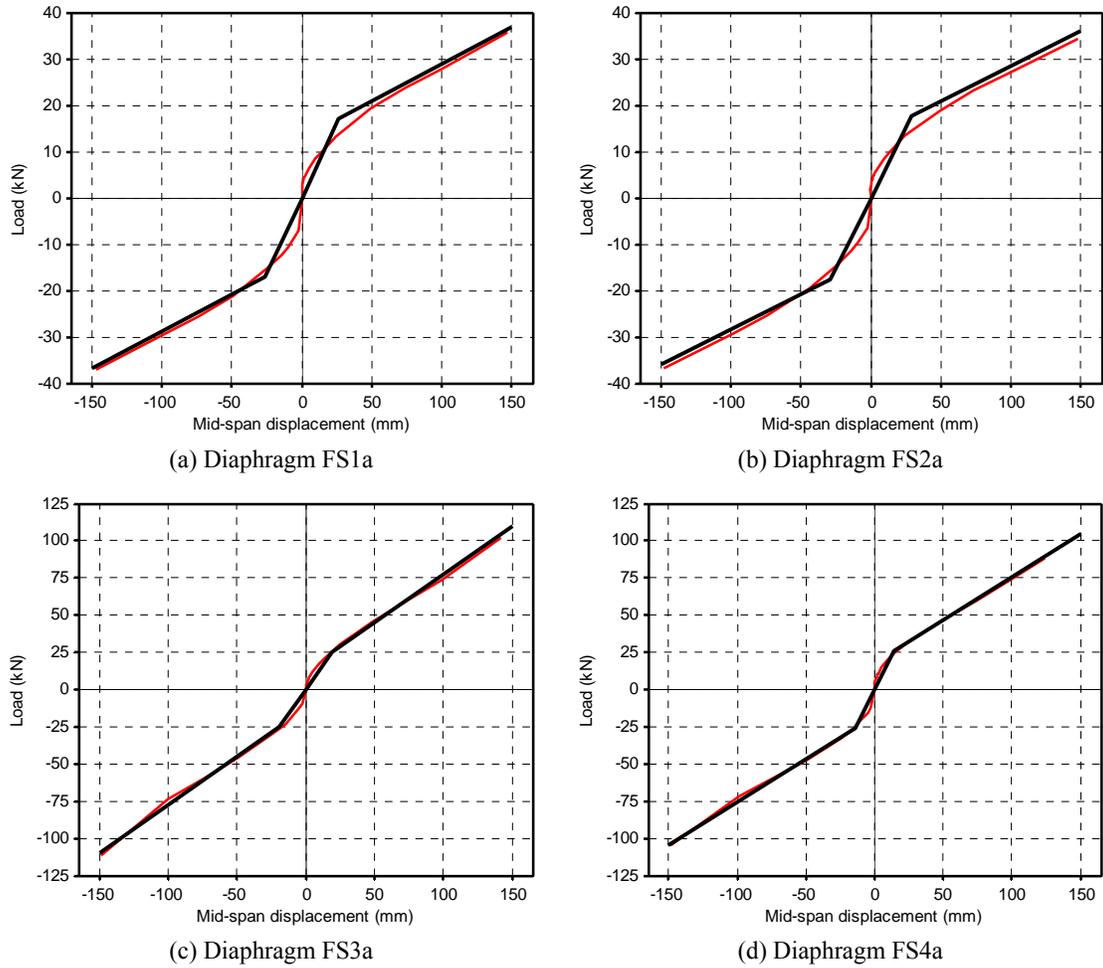
ASCE 41-06 addresses diaphragm ductility capacity using component modification factors ( $m$ -factors) that account for the expected level of ductility at different structural performance limit states. The  $m$ -factors associated with the Life Safety Limit State are analogous to the conventional structural ductility factor  $\mu$  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).

## 3.3. Experimental results

Experimental values of strength, stiffness and ductility were determined for each diaphragm tested and compared in Table 1 against the predicted values using NZSEE and ASCE 41-06 guidelines. To determine these values it was necessary to appropriately characterize the force-displacement data of each test using a consistent and rational methodology that enabled the objective interpretation of diaphragm performance parameters. 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  $\pm 150$  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 5 shows the bilinear curves produced for diaphragm tests FS1a to FS4a. The bilinear curves defined the yield force ( $F_y$ ), yield displacement ( $\Delta_y$ ), maximum force at  $\Delta = 150$  mm ( $F_{max}$ ), and initial and secondary stiffness ( $K_1$  and  $K_2$ ) values presented in Table 2 that were used to calculate the desired



**Figure 5** – Bilinear curves for tested as-built diaphragms

diaphragm performance parameters. Diaphragm stiffness  $K_d$  was defined as initial stiffness  $K_I$  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. These values are presented in Table 1 for comparison.

**Table 1** – Diaphragm performance parameters

	<i>Shear strength, <math>R_n</math></i>				<i>Stiffness <math>K_d</math></i>			<i>Shear stiffness, <math>G_d</math></i>			<i>Ductility, <math>\mu</math></i>		
	[kN/m]				[kN/m]			[kN/m]					
	NZSEE (i)	NZSEE (ii)	ASCE	Exp	NZSEE	ASCE	Exp	NZSEE	ASCE	Exp	NZSEE	ASCE	Exp
<b>FS1a</b>	1.4	6.0	1.75	1.6	207	745	647	97	350	304	-	2.0	5.6
<b>FS2a</b>	1.4	6.0	1.75	1.6	207	745	606	97	350	284	-	2.0	5.2
<b>FS3a</b>	1.4	6.0	1.75	1.2	730	2630	1297	97	350	173	-	2.0	7.7
<b>FS4a</b>	1.4	6.0	1.75	1.2	730	2630	1842	97	350	245	-	2.0	10.8

**Table 2** – Experimental bilinear values

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

#### 4. DISCUSSION

Overall, the guidelines offered in both assessment documents poorly predict diaphragm performance. The values listed in Table 1 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 is considerably under predicted using the methodology in NZSEE, while is over predicted using ASCE 41-06 guidelines. NZSEE offers no explicit guidance for diaphragm ductility while ASCE 41-06 provisions where shown to under estimate diaphragm ductility by up to five times.

An important observation from the experimental performance parameters listed in Table 1 is the highly orthotropic behavior 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 behavior. 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 1 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 testing will provide the necessary information to appropriately modify the performance parameters to ensure 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 include provisions 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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