Contribution of timber-framed partition walls to the dynamic characteristics of an unreinforced masonry building

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ABSTRACT: Interior timber-framed partition walls are included in almost all buildings, yet because they are a non-structural element, the contribution of these walls is frequently ignored during seismic assessments. Currently there is a lack of large scale in situ testing to confirm this assumption, particularly for unreinforced masonry buildings. In response, a field testing program was performed on a decommissioned building to directly identify the contribution of the partition walls to the dynamic system. The building was a prototypical, three-storey, early 1900's unreinforced clay brick masonry building located in Auckland, New Zealand. The building was subjected to forced vibration testing using two small electro-dynamic shakers, and modal properties were identified using a suite of system identification techniques. Natural periods, mode shapes, and modal damping ratios were first determined for the building with all partition walls in place to establish baseline modal properties. The partition walls were then removed storey by storey, and forced vibration testing was repeated following the removal of the partitions from each storey. Modal properties identified after the removal of partitions at each storey were compared to the baseline modal properties to directly determine the contribution of the partition walls to the overall dynamic response of the building.

1 INTRODUCTION

Interior non-structural lightweight timber framed partition walls are part of almost all unreinforced masonry (URM) buildings and create functionality of the building space. Due to the non-loadbearing nature of such partition walls, the structural contribution of these walls is frequently ignored during assessments and analysis of the global seismic performance of the building. A knowledge gap currently exists to confirm this assumption, particularly for unreinforced masonry buildings. Observations made following the 2011 Christchurch earthquake indicate that non-structural lightweight timber-framed partition walls contributed to the global performance of URM buildings (see Fig. 1), yet the extent of this contribution is as yet undetermined. A team of researchers was presented with a unique opportunity to investigate the effects of partition walls on a full scale vintage URM building that was scheduled for demolition. Natural periods, mode shapes, and modal damping ratios were determined for the building following the removal of the partitions from each storey.

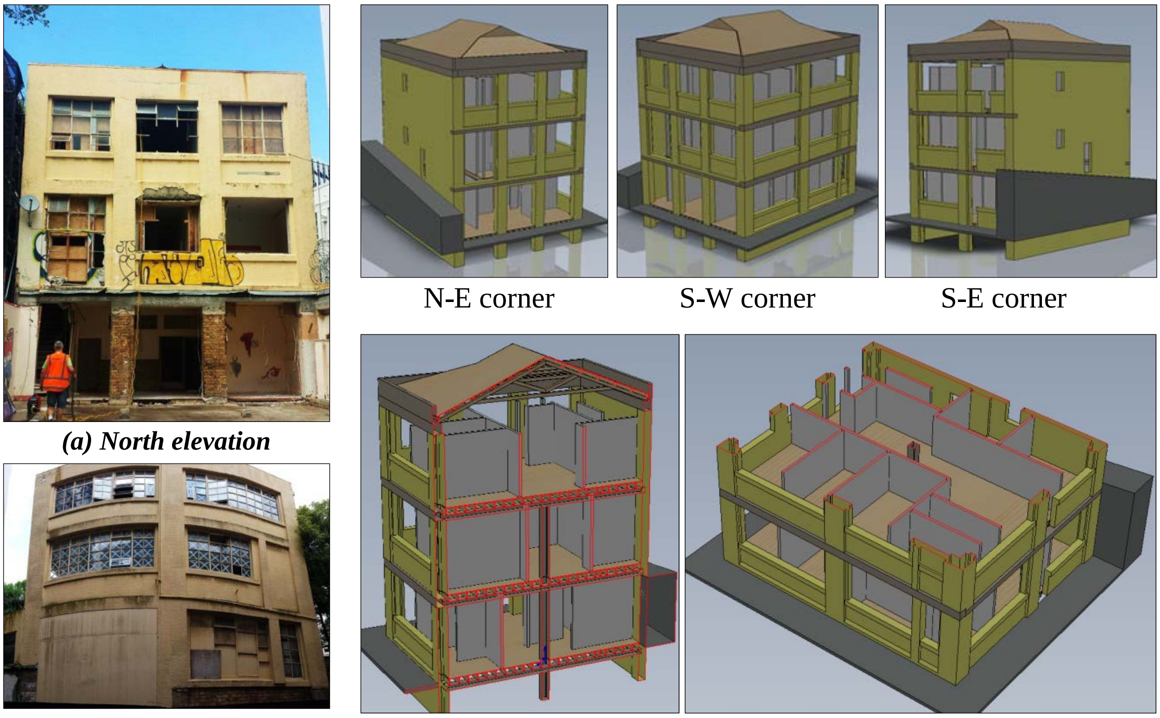
Figure 1. Observations of URM building damage with non-loadbearing partition walls from 2011 Christchurch earthquake.

2 TEST BUILDING DESCRIPTION

The test building, located at 27 Rutland Street in the Auckland CBD, was a relatively prototypical unreinforced clay brick masonry (URM) building (Walsh et al. 2014), that was originally constructed in 1931. The building had floor dimensions of approximately 10 m in the E-W direction and 12 m in the N-S direction (see Fig. 2). The subject building consisted of four stories – three stories above grade (approximately 11.3 m from grade to the top of the reinforced concrete (RC) parapet at the north elevation) and a basement level. The primary gravity loadbearing elements of the building consisted of URM piers and walls. A continuous RC bond beam extended along the full perimeter of the building at the roof level, on which timber roof trusses were supported. RC bond beams at all other storey levels extended around the perimeter of the building (excluding the east elevation), where the RC beams acted as window lintels. Steel angle lintels were used over the few small openings on the east URM wall.

The original floor diaphragm construction consisted of timber joists spanning in the N-S direction, sized primarily 280 mm x 50 mm with an average centre-to-centre spacing of 420 mm. A concrete encased Rolled Steel Joist (RSJ) column was positioned at the centre of the building, supporting a RC beam spanning in the E-W direction which in turn supported the timber joists at all floor levels (except for the roof level). Tongue and groove timber flooring, with an approximate thickness of 20 mm, was used at all floor levels. A ‘false’ floor consisting of timber spacers and medium density fibreboard (MDF) totalling approximately 130 mm in thickness above the original floor level was placed atop the original diaphragm on the first and second floor levels. The roof of the subject building was supported by three large timber trusses spanning E-W and supporting timber purlins and plank sheathing. The building foundation consisted of shallow RC strip and spread footings supporting URM walls and piers.

Interior partition walls on all of the levels of the subject building consisted entirely of lightweight timber framing with plasterboard lining. No interior URM or load-bearing partition walls were present in the test building.



(a) North elevation

(b) West elevation

N-E corner S-W corner S-E corner

Elevation cross-section Floor-plan cross-section

(c) 3D view of the building

Figure 2. The tested building located at 27 Rutland Street, Auckland CBD.

3 TESTING METHODOLOGY

3.1 Demolition Sequence

As the focus of this research was to establish the participation of non-loadbearing timber partition walls in the dynamic response of a URM building, modal properties of the building were established in several phases of interior demolition. The building response was measured in a series of five separate dynamic tests correlating with the following demolition sequence:

1. All partition walls, false ceilings and false floors in place (baseline condition)
2. Partition walls on the first storey removed
3. Partition walls and false ceilings on the top storey removed
4. Partition walls on the ground storey removed
5. False floors on both the first and second storeys removed.

The same excitation protocol was used for each test series and the changes in natural frequency, mode shape, and modal damping ratios were determined for each alteration of the building. Ambient vibrations were also recorded for each of the tests in the series.

3.2 Excitation Protocol

Excitation was provided by two APS ELECTRO-SEIS Model 400 electro-dynamic shakers. Each shaker produced a constant maximum force output of 445 N over the 0.1 to 10 Hz frequency range. Above 10 Hz, amplifier limits reduced the maximum force output to 356 N at 20 Hz and 267 N at 30 Hz. Two excitation types were used being a sweeping sine function between 2 and 25 Hz and random white noise. All test excitations were five minutes long.

(a) Electro-dynamic shakers set up for testing in the N-S direction

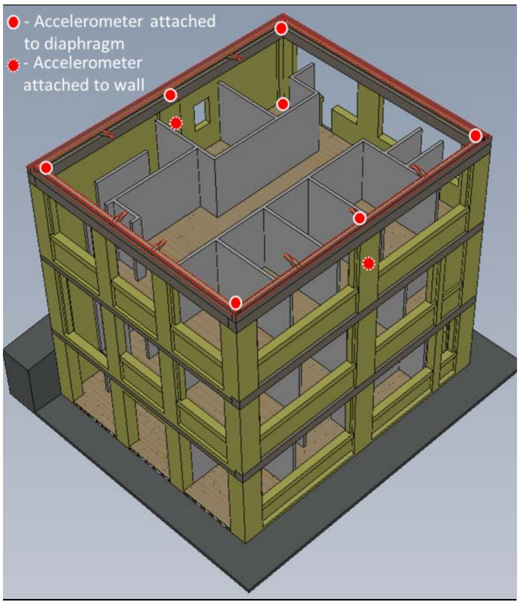
(b) Two types of accelerometers used - standalone (left) and wired (right)

Figure 3. Instrumentation used for testing.

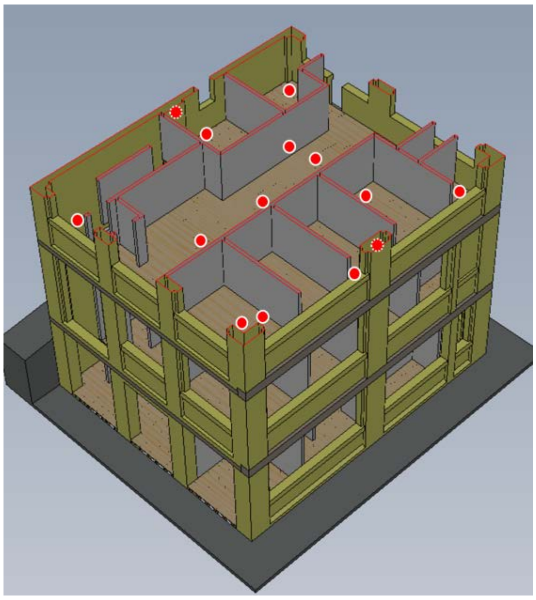
Three configurations of the electro-dynamic shakers were used in order to excite potential modes of the building. In each configuration two shakers were placed on the second storey, first aligned in the N-S direction of the building at the centre of the second storey. The shakers were then rotated to the E-W direction but remained at the centre of the second storey. Finally, one shaker each was placed along the north and south walls and oriented in opposite directions to excite potential torsional modes. The excitation protocol of three sinusoidal frequency sweeps was used in each shaker configuration. Additionally, a series of vertical excitations were induced into the second storey diaphragm to determine the vertical modal response of the floor diaphragms. The vertical excitations were performed at three different locations and were induced by synchronised jumping of two persons.

3.3 Instrumentation

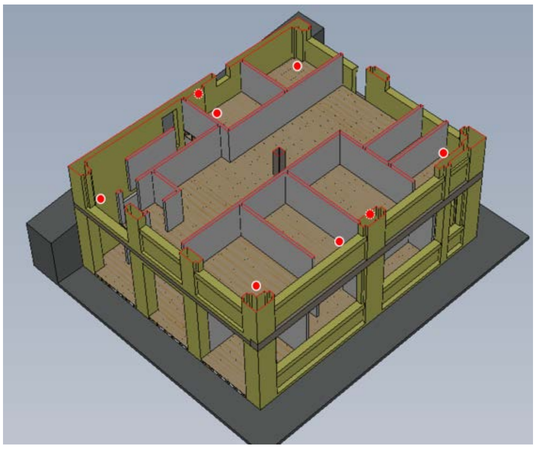
Accelerations were recorded with a network of standalone triaxial ± 2 g Gulf Coast Data Concepts X6-2 accelerometers that recorded measurements to an internal microSD card. A network of 35 of these accelerometers was distributed around the building as shown in Figure 4. Due to battery failure during testing, some of the accelerometers were disregarded in post-processing and as a result the accelerometer network varies slightly from one test to the next.



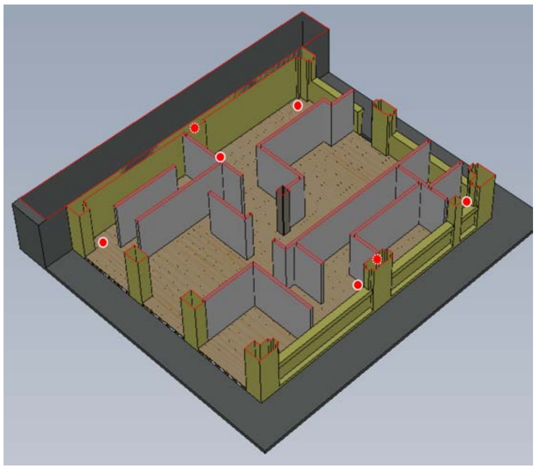
(a) Roof accelerometers



(b) Second storey accelerometers



(c) First storey accelerometers



(d) Ground storey accelerometers

Figure 4. Generalised accelerometer network for Rutland St. dynamic testing.

4 IDENTIFIED MODAL PROPERTIES

The forced vibration data for each test was analysed using a modal property identification toolbox (MPIT) developed at the University of Auckland (Beskhyroun 2011).Five system identification algorithms were used to extract modal properties from recorded acceleration data. Three of the algorithms were frequency domain based and included Peak Picking (PP), Enhanced Frequency Domain Decomposition (EFDD) (Jacobsen et al. 2007), Eigen Realisation Algorithm (Juang and Pappa 1985) (ERA), and Stochastic Subspace Identification (SSI) (van Overschee and De Moor 1996). Natural period was determined using each of these algorithms, and ERA and SSI were used to identify modal damping ratios.

Modal properties were ascertained by finding the mean modal property (e.g. natural period, modal amplitudes, damping ratio) identified by each algorithm for all of the frequency sweep excitations performed in a given direction at each demolition stage. By tracking the change in natural period and damping at each demolition stage, the stiffness and damping participation of the partitions was derived. Due to limitations in length, only the modal property changes in the fundamental mode are discussed herein.

4.1 **Period Shift**

The building exhibited a first mode response in the E-W direction at a period of 0.26 s with all modal amplitudes being in phase (Fig. 5). Modal amplitudes found at the mid-storey heights of the ground storey were lower than expected if the building were to respond with similar storey stiffnesses. Instead, the inter-storey change in modal amplitude was much larger between the first and second storeys than between the ground and first storeys. These small modal amplitudes at the ground storey were likely due to the increased stiffness provided by the storey high retaining wall found along the east side of the building (Fig. 2). Significant stiffness would have been provided by this wall in the eastern direction. However, as the western side of the ground storey was unrestrained, some level of modal amplitude would still be expected up the walls.

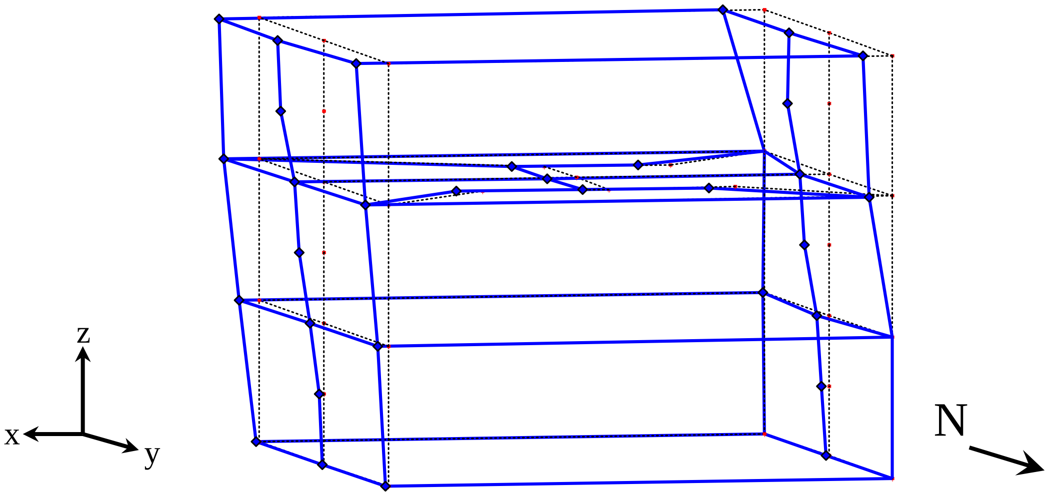


Figure 5. Fundamental mode shape for Rutland St. building. Only locations where accelerometers were located have been displayed in the figure.

The shifts in fundamental period at each demolition stage are shown in Figure 6. As expected there was a period lengthening as partitions and flooring were removed. The total period increase was approximately 12% between the building with all partitions and false floors and the building with no partitions or false floors. In order to determine whether this period shift resulted from simply removing mass from the building or whether it was also caused by a decrease in stiffness caused by the removal of the partition walls, a numerical representation of the building was created. The building was approximated as a single degree of freedom (SDOF) oscillator with dynamic properties corresponding to the first mode response. This approximation was deemed to be acceptable for the first mode because the mode shape approximated a SDOF oscillator (Fig. 5). The lumped mass of the building was approximated using a masonry density of 1580 kg/m^3 measured from specimens taken from the building, in conjunction with published unit weights for concrete, floor joists, partition framing, and gypsum board in NZS 1170.1:2002 Appendix A. The change in mass at each demolition stage was calculated based upon the volume of non-structural material removed during each particular stage of demolition.

Using the relationship between period, mass, and stiffness for a SDOF system (Equation 1) the approximate lumped stiffness of the building was calculated for the corresponding mass and period at each demolition stage. These changes in period, mass, and stiffness at each demolition stage are summarised in Table 1. As shown in the table, there is approximately a 13% change in mass with the removal of partitions or the false floors at all storeys, with approximately a 3-5% change in mass being attributed to the removal of these elements at any given storey. The corresponding changes in building stiffness was almost 35% from the first to the last demolition stage, with a change in stiffness of 7-13% from the removal of the partition walls at a given level and approximately 6% for the removal of the false floors.

From these shifts in period, mass, and stiffness of the building system it is apparent that partition walls did significantly contribute to the stiffness in the E-W direction of the subject building. The large contribution to the stiffness in the subject building is likely due to narrow piers on the north and south side of the building causing each storey to be relatively flexible. While there was a significant stiffness contribution from the partitions walls, the shift in period was insignificant to alter the lateral loading derived using a force-based equivalent static analysis, as base shears are assumed for assessment to be constant for structures with periods of up to 0.4 s for Site Classes A-C (NZS 1170.5:2004). Given this lack of base shear alteration and the likelihood of partition walls being shifted or removed during the life of the building, the current design practice of ignoring the stiffness contribution of these elements appears to be appropriate.

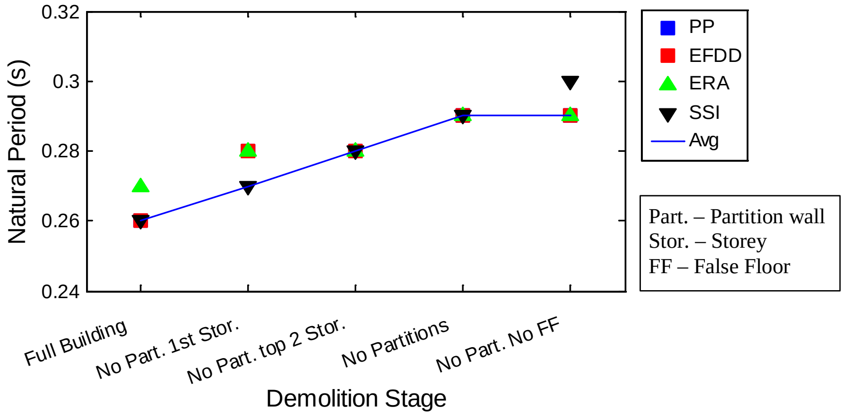


Figure 6. Change in mean fundamental period of Rutland St. building at each demolition state as computed with each system identification algorithm. Blue line indicates mean period at each demolition stage.

$$T_n = 2\pi\sqrt{\frac{M}{K}} = \frac{1}{f_n} \qquad (1)$$

Table 1. Mass and stiffness of the subject building at each demolition state as determined by natural period.

Test	T_n (s)	f_n	T_n cumulative diff. (%)	Mass (Tonnes)	Mass cumulative diff. (%)	Stiffness (kN/m)	Stiffness cumulative diff.(%)
Original building	0.26	3.83	-	145	-	215	
No partitions 1st Floor	0.27	3.64	4.89%	141	2.87%	190	12.63%
No partitions top two stories	0.28	3.54	7.75%	135	7.04%	172	22.45%
No partitions	0.29	3.46	10.2%	132	9.77%	159	29.9%
No partitions and no false floors	0.29	3.41	11.6%	128	12.9%	150	35.8%

T_n – Building natural period in seconds
f_n - Building natural frequency in hertz
diff. – difference

4.2 Damping

The shift in equivalent viscous damping provided in the first mode is shown in Figure 7. The figure includes the maximum and minimum damping ratios determined for all frequency sweeps of a given demolition stage for both ERA and SSI algorithms. The average of these values is represented by the solid line in the figure. The average equivalent viscous damping is approximately 6% for the full building with all partitions and reduces down to approximately 4% for all other demolition states. The shift in damping only occurred at the first demolition stage, the removal of the first floor partitions, and was likely caused by the lower dynamic participation of the partitions on the ground and second floors. As seen in stiffness changes in Table 1, the largest change in total stiffness of the system occurred with the removal of the partitions in the first storey. The ground floor likely had less participation because the retaining wall along the east side of the building limited displacements at that storey and as such the ground floor partitions were unlikely to deform significantly enough to provide additional damping. Likewise, while they provided some stiffness to the overall system, the second storey partitions were unlikely to deform significantly due to the top of the walls not being restrained.

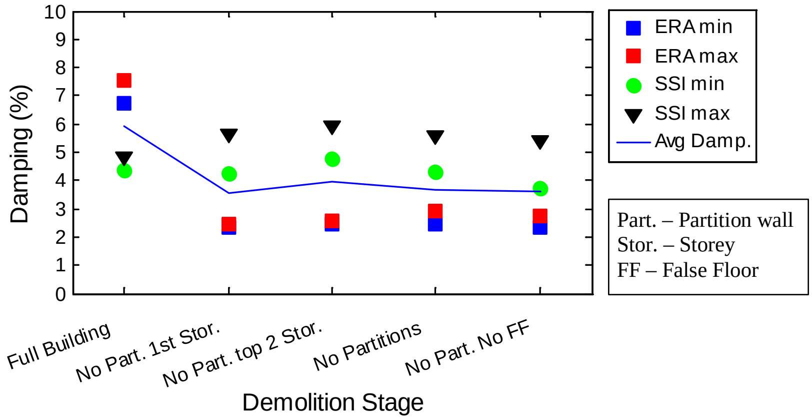


Figure 7. Change in equivalent viscous damping of Rutland St. building at each demolition state as computed with each system identification algorithm. Solid line indicates mean damping at each demolition stage.

From this series of tests it appears that the partition walls provided approximately 1-2% additional equivalent viscous damping per floor for approximately 12 m of wall length in the direction of shaking, if the walls are the full storey height and the storey is allowed to deform significantly. This estimate of increased damping is quite coarse however, as only the first storey partitions in this building met this criterion. The increased damping provided by the partition walls is also suspected to be higher with increased load levels, and the small amplitude shaking completed in this testing would represent the excitation level for lower bound of damping. Based upon these findings, it appears that the current assessment practice of assuming 5% equivalent viscous damping for URM buildings is appropriate.

5 IMPLICATIONS FOR OTHER URM BUILDINGS

The results of the dynamic testing program performed on the Rutland St. building, coupled with observations of URM buildings in the Christchurch earthquake, allow for some generalised guidelines to be proposed for the contribution of non-loadbearing partition walls to the overall seismic response of URM buildings. It should be noted that these guidelines assume that out-of-plane (OOP) failure is restrained, because if not, OOP failure would occur and the only participation the partitions would have is as a secondary gravity system to prevent total collapse of the building (Figure 1). It is also assumed that the partitions are the full storey height and are connected to the diaphragms at both the top and bottom wall. Finally, these guidelines are with respect to building shaking in-plane with the partition walls. The effect of partition walls on the building response subject to shaking oriented OOP, torsionally, or biaxially to the partition walls is not discussed herein.

Non-loadbearing partition walls are likely to have a noticeable effect on the response of buildings when oriented parallel to slender URM pier walls. The greater relative stiffness of the partition walls will likely limit inter-storey drifts and reduce diaphragm displacements. Limiting diaphragm displacements would limit OOP displacement demands on the URM walls in the orthogonal direction and improve their behaviour. When rocking initiates in the slender piers, large deformations and induced damage in the partition walls are likely to provide significant damping to an otherwise lightly damped rocking system.

Non-loadbearing partition walls are unlikely to have a noticeable effect on the response of buildings when oriented parallel to long, stiff URM walls with few perforations. The partitions are expected to be ineffective at providing additional damping or inter-storey stiffness to this systems because the long URM walls limit inter-storey drifts and do not allow the partitions to deform significantly. Partition walls may still help to limit diaphragm displacements and improve OOP behaviour of orthogonal URM walls in this system, especially for wide diaphragms.

6 CONCLUSIONS AND FUTURE WORK

A prototypical 1931 URM building was tested with small amplitude forced vibration testing to determine directly the participation of non-loadbearing partition walls. The building in its as-built state was subjected to a series of sinusoidal sweeps with two small electro-dynamic shakers. Following shaking of the building in this benchmark state, partition walls were removed storey by storey and the forced vibration testing was repeated with the removal of each set of partition walls. A series of system identification algorithms were used to extract modal properties from the forced vibration data collected. Changes in fundamental period indicated that system stiffness was reduced by approximately 35% with the removal of all partition walls. However, the resulting shift in fundamental period was not significant enough to alter the design base shear of the building. Therefore the current practice of ignoring the stiffness participation of the partition walls appears to be appropriate for the building that was investigated.

Equivalent viscous damping ratios were also derived from the forced vibration data, and it was found that one storey of partition walls with approximately 12 m of wall length in the direction of shaking may contribute between 1-2% additional damping from the main structural system at low excitation levels. From the test results, it appears that the current assessment assumption of 5% equivalent viscous damping is appropriate for URM buildings with non-loadbearing partition walls.

The results of this system identification study, including the modal properties of higher modes (although not discussed here), will be used to calibrate computational models of the subject building. This modelling study will be used to determine the participation of the partition walls in these modes both in terms of additional stiffness and damping and will serve as an elastic baseline for further nonlinear analysis.

7 ACKNOWLEDGEMENTS

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