

# What ductility value should be used when assessing unreinforced masonry buildings?

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**ABSTRACT:** There has been much discussion regarding the appropriate values of structural ductility factor, structural performance factor, and equivalent viscous damping coefficient to be used in the assessment of unreinforced masonry (URM) buildings in New Zealand and Australia. The New Zealand Society for Earthquake Engineering 2006 guidelines proposes a ductility factor of 1.0 to be used along with a damping coefficient ratio of 15%. A draft guideline recently developed by researchers from the University of Auckland suggests that a ductility factor of 2.0 and damping ratio of 5% is more suitable. Elsewhere in the world, other researchers have made recommendations regarding these topics. Furthermore, the Canterbury Earthquakes Royal Commission has recently recommended this topic receive further attention. Different alternatives are evaluated by conducting Nonlinear Static (pushover) analyses for a representative URM structure. A comparison is made to previous experimentally and analytically derived equivalents. Following this research, a recommendation is made regarding values which should be used in future design. Results of a simplified plane stress analysis for this building are also presented and compared to the results obtained from the pushover analyses.

## 1 INTRODUCTION

Recent recommendations by the Canterbury Earthquakes Royal Commission (Cooper et al. 2012b) have emphasised the need for further research into the suitability of using values of 2.0 and 0.7 for structural ductility factor and structural performance factor respectively. In response to these recommendations, an investigation into suitable values for structural ductility factor, structural performance factor, and equivalent viscous damping coefficients was undertaken based on previous research and design codes, and also the results from an analysis of an unreinforced masonry building which is representative of typical NZ and Australian URM construction.

## 2 BACKGROUND

### 2.1 Performance of Unreinforced Masonry Buildings in Earthquakes

Unreinforced masonry (URM) buildings have been observed to perform poorly even in moderately sized earthquakes. This poor performance has been demonstrated in the earthquakes of Hawke's Bay (M7.8, 1931), Newcastle (M5.6, 1989), and the Canterbury earthquakes of 2010-2011 (Ingham & Griffith, 2011). URM buildings are New Zealand's most earthquake prone class of building (Russell & Ingham, 2010), and as a result, appropriate legislation has been enacted to reduce the risk from such earthquake prone buildings. The Building Act (2004) requires the seismic strengthening or demolition

of any non-residential building with a strength less than 33% of that required for a new building standard (NBS).

## 2.2 Assessment of URM buildings subjected to earthquake loading

During an earthquake, walls in buildings are simultaneously subjected to both in-plane and out-of-plane excitation. It has been widely observed that satisfactory seismic performance of a structure is only achieved if global behaviour of the structure can be achieved by suppressing local failure mechanisms such as out-of-plane wall failures (Magenes, 2006). Recent research has found that out-of-plane checks of walls can be carried out using a displacement-based approach such as that proposed by Doherty et al. (2002). It is not usually necessary to consider interaction between the in-plane behaviour and out-of-plane behaviour of walls (NZSEE, 2006).

Most seismic design codes specify the permissible methods of analysis for various types of buildings, including URM buildings. The four types of analysis common to most design codes are, in increasing order of sophistication: Linear Static Analysis, Linear Dynamic Analysis, Nonlinear Static (Pushover) Analysis, and Nonlinear Dynamic Analysis.

Although it is a state-of-the art research tool, Nonlinear Dynamic analysis cannot yet be considered a viable tool for the practising engineer due to the lack of reliable models and high level operator skill required to conduct these analyses (Magenes & Penna, 2011). The use of Nonlinear Static analyses is not explicitly referenced in NZS 1170.5 (2004) (Davidson, 2010) and due to the lack of codified procedure for this type of analysis, it is understandable that there is some hesitation among practising engineers in NZ to use nonlinear static analysis as a tool in the design and assessment of buildings. The most common methods of analysis for URM buildings in NZ by practicing engineers are the Linear Static method and the Linear Dynamic method which implicitly model inelastic behaviour by allowing a reduction of the elastic seismic design loads. The general form of the equation for seismic base shear for both of these methods can be expressed by Equation (1):

$$V = C_d \times W_t \Leftrightarrow C_d = V / W_t \quad \text{Where: } C_d = C_e / R \quad (1)$$

Where  $C_e$  is a factor dependent on the period and importance level of the structure, as well as the seismic hazard and subsoil conditions present at the location of the building.  $W_t$  is the seismic weight of the structure which includes the full dead load of the structure as well as a proportion of live load.  $R$  is the force reduction factor which is defined by Equation (2).

## 3 STRUCTURAL DUCTILITY FACTOR

### 3.1 Definition of Structural Ductility Factor and Force Reduction Factors for URM

Experimental evidence and observations from previous earthquakes have shown that with proper construction methods and detailing, URM buildings have a relatively high energy dissipation capacity which allows the elastic earthquake forces to be reduced by a force reduction factor,  $R$  (Tomazevic, 1999). The force reduction factor,  $R$  is an approximation of the ratio of seismic forces that the structure would experience with a completely elastic response to the seismic forces that may be used in the design of the structure (CEN-EN 1998-1, 2004). The definition of the force reduction factor is one of the most important and controversial aspects of seismic engineering. The general form of the structural force reduction factor used by most design codes is presented in (2):

$$R = R_\mu \times \text{OSR} \quad (2)$$

Where  $R_\mu$  is the component of the force reduction factor associated with the inherent energy dissipation capacity, that is, the ductility of the structural system under consideration. The structural ductility factor,  $\mu$  is defined as the ultimate displacement of the structure divided by the yield displacement of the structure. The relationship between structural ductility factor and force reduction factor is shown in *Figure 1*.

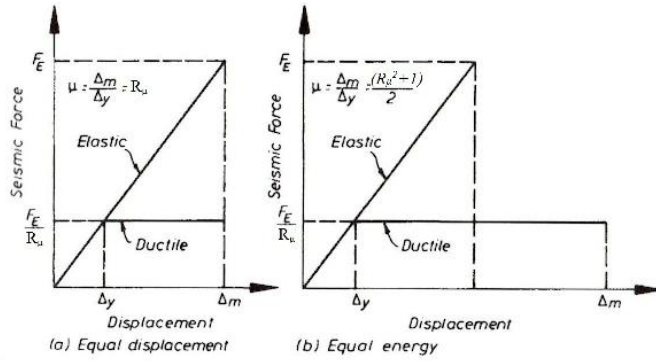


Figure 1: Equal displacement and equal energy principles (Adapted from Paulay & Priestley, 1992)

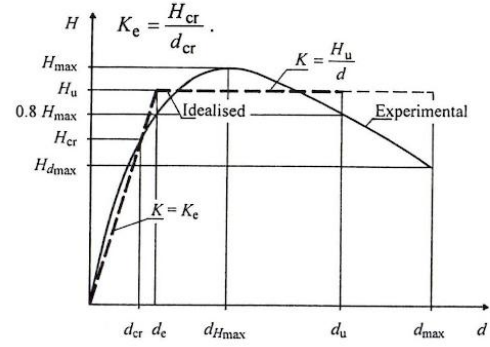


Figure 2: Bilinear approximation of force-displacement curve (After Tomazevic, 1999)

The value of  $R_\mu$  is found by determining the base shear of the structure using a linear elastic model and dividing by the actual measured strength capacity of the structure when subject to the same imposed accelerations.  $R_\mu$  can also be evaluated from the global ductility of the system using the relationship established between force reduction factor  $R_\mu$  and ductility factor  $\mu$  obtained from the equivalent energy or equivalent displacement principles. As most masonry structures fall within the acceleration-sensitive (short period) region of the response spectrum curve, the equivalent energy principle (Equation (3)) is usually used for the calculation of  $R_\mu$ . For longer period structures, the equal displacement principle (Equation (4)) is more appropriate. Figure 1 shows the difference between the equal energy and equal displacement principles.

$$R_\mu = (2\mu - 1)^{1/2} \quad (3)$$

$$R_\mu = \mu \quad (4)$$

It is important to note that for structures with extremely short periods, Equation (3) is non-conservative (Paulay & Priestley, 1992). It is for this reason that the NZS 1170.5 (2004) equation to calculate  $R_\mu$  (called  $k_\mu$  in this standard) is dependent on the structure period. For structures with very low periods, the force reduction factor approaches 1.0 as the structure period tends towards 0. This behaviour is consistent with structural dynamics theory, where an infinitely stiff structure subjected to excitation through ground acceleration must experience the same acceleration as the ground (Chopra, 2007). OSR is the overstrength factor defined as the ratio of the force associated with the yield strength of the entire structure ( $H_u$ ) to the force at which first yield of any component member of the structure occurs ( $H_{cr}$ ). The overstrength factor accounts for favourable effects such as internal force redistribution, higher than specified material strengths, structural redundancy, influence of non-structural components, and strain hardening (Uang, 1991).

One of the difficulties in defining a structural ductility factor for masonry is the highly nonlinear behaviour of masonry. In order to calculate a meaningful value for the structural ductility factor, a bilinear approximation to the force-displacement curve is required. Several methods have been presented to construct this bilinear idealisation including Tomazevic (1999), Park (1989), and Eurocode 8 (CEN-EN 1998-1, 2004). These methods are typically based on an equivalent energy principle, that is, the area under the bilinearised force-displacement curve should be equal to that area under the original force-displacement curve. The bilinearisation procedure proposed by Tomazevic (1999) which is similar to that recommended by the University of Auckland (2011) is shown in Figure 2. Tomazevic (1999) recommends using the secant stiffness from the origin to the point of first cracking as the stiffness for the elastic part of the bilinear force-displacement curve. The effective yield displacement  $d_e$  can then be estimated by dividing the effective yield force  $H_u$  by the stiffness of this initial portion of the curve. The ultimate displacement  $d_u$  is usually defined as the displacement at which the force has decreased to a value that is 80% of the maximum recorded force. If no reduction in force is observed, then the ultimate displacement is defined as the displacement of the structure at the end of the simulation.

Based on the results of quasi-static cyclic tests on a large number of unreinforced masonry walls,

Tomazevic (1999), and OPCM 3274 (2003) recommend using an effective yield force  $H_u = 0.9H_{max}$  as an acceptable approximation to the equal energy method of bilinearising the force-displacement curve for unreinforced masonry. Several authors (Park (1989), Magenes & Calvi (1997)) have also suggested the use of  $H_{cr} = 0.75H_u$  as an approximation to the load at first cracking of masonry. If this simplification is made, then the value of the overstrength ratio, OSR which is defined as  $H_u/H_{cr}$  is implicitly assumed to be equal to  $0.9/0.75 = 1.2$ .

### 3.2 Structural Ductility Factor for URM in Design Codes

Values of force reduction factor  $R$  are prescribed in almost all seismic design codes. The nomenclature of the force reduction factors varies between codes, but the general form of the equation for design base shear follows that presented in *Equation (2)*. In this section, structural ductility factor parameters for unreinforced masonry construction are compared for a range of codes

The New Zealand design standard for seismic actions NZS 1170.5 (2004), expresses base shear in the form of *Equation (1)*. The force reduction factor associated with ductility,  $R_\mu$  is denoted by  $k_\mu$  and is a function of ductility factor  $\mu$ , soil class, and structure period (*Equation (5)*). The overstrength ratio is expressed by NZS 1170.5 as  $(1/S_p)$ . As there is no design standard for URM in NZ, values of  $\mu$  and  $S_p$  must be taken from the NZSEE (2006) guidelines. These guidelines state a ductility factor for URM of  $\mu = 1.5$  with  $S_p = 1.0$  for the standard 5% damping case. Values of  $\mu = 1.0$  with  $S_p = 1.0$  are recommended for a damping ratio of 15%. NZSEE (2006) states that if a damping ratio,  $\xi = 15\%$  is used for unreinforced masonry, then the response of the structure may be reduced by a further 35%. By substituting a value of  $\mu = 1.5$  into *Equation (5)* it can be seen from *Equation (6)* that the force reduction factor is between 1.0 and 1.5 for the case of the standard damping ratio  $\xi = 5\%$ . Similarly, by substituting a value of  $\mu = 1.0$ , and applying the further 35% reduction according to NZSEE (2006), *Equation (7)* indicates that the value of force reduction factor is equal to 1.54 for the case of  $\xi = 15\%$ . It can therefore be concluded that using a structural ductility factor  $\mu = 1.5$  with  $\xi = 5\%$  is slightly more conservative than using a structural ductility factor  $\mu = 1.0$  with  $\xi = 15\%$ .

$$R_\mu = k_\mu = 1 + (\mu - 1)T_1/0.7 \leq \mu \quad (5)$$

$$R_\mu = k_\mu = 1 + (0.5)T_1/0.7 \leq \mu \Rightarrow 1.0 \leq k_\mu \leq 1.5 \quad (6)$$

$$R_\mu = k_\mu / 0.65 = 1/0.65 = 1.54 \quad (7)$$

The Australian earthquake design standard, AS 1170.4 (2007) expresses the seismic base shear in the same format as NZS 1170.5 (2004) but with some simplifications. Values of  $\mu = 1.25$  and  $S_p = 0.77$  are given by AS 1170.4 (2007) with a standard damping ratio of 5%. One major difference between Australian and NZ earthquake design standards is the definition of the force reduction factor associated with ductility,  $R_\mu$ . The value of  $R_\mu$  given by AS 1170.4 (2007) is equal to  $\mu$ , that is, independent of structure period. This assumption may be valid for longer period structures ( $T_1 > 0.7s$ ), but could be non-conservative for short-period structures such as unreinforced masonry buildings.

European codes such as Eurocode 8 (CEN-EN 1998-1, 2004) and the Italian seismic design code OPCM 3274 (2003) express the force reduction factor  $R$  as  $q$ . In, OPCM 3274 (2003), this value of  $q$  is explicitly separated into the ductility and overstrength components  $q'$  and OSR respectively. OPCM 3274 gives a range of  $q'$  values from 1.5-2.0 and overstrength values from 1.4-2.5, giving a range of  $q$  values from 2.1 - 5.0, depending on the structural regularity and number of floors present within the building.

Eurocode 8 does not explicitly decompose  $q$  into the ductility and overstrength components, but instead gives a range of  $q$  values from 1.5-2.5. Both OPCM 3274 and EC8 assume a default viscous damping ratio of 5%, and there are equations in these codes to reduce the structural response for higher values of damping up to 15%.

The North American seismic code ASCE-41 (2007) and its predecessors FEMA-356 (2000) and FEMA-273 (1997) express the force reduction factor  $R$  as  $m \times \kappa$  where  $m$  is the force reduction factor associated with ductility, and  $\kappa$  is a factor between 0.75 and 1.0 accounting for the level of knowledge regarding the material properties and geometry of the structure under consideration.

The major difference between these North American codes and the NZ, Australian, and European codes considered in this study is the fact that the North American codes separate unreinforced masonry failure mechanisms into force-controlled mechanisms which do not allow the use of a structural force reduction factor  $R$ , and displacement-controlled mechanisms which allow the use of a force reduction factor  $R$ . The capacities of force-controlled ( $Q_{CL}$ ) and displacement controlled ( $Q_{CE}$ ) elements subjected to seismic loads ( $Q_{UD}$ ) according to FEMA-273/356 and ASCE-41 are shown in Equations (8) and (9) respectively.

$$Q_{CL} \geq Q_{UD} \quad (8)$$

$$m \times \kappa \times Q_{CE} \geq Q_{UD} \Leftrightarrow Q_{CE} \geq Q_{UD} / R \quad \text{Where } R = m \times \kappa \quad (9)$$

FEMA-356 and FEMA-273 classify diagonal tension and toe crushing as force-controlled failure mechanisms, and bed joint sliding and rocking as displacement-controlled mechanisms. ASCE-41 (2007) classifies only rocking as a displacement-controlled mechanism, and bed joint sliding and toe crushing as force-controlled mechanisms. ASCE-41 (2007) does not consider diagonal tension as a possible failure mechanism. Tremayne et al. (2012) have indicated that the proposed revision of ASCE-41 to be released in 2013 will reinstate diagonal tension as a force-controlled mechanism and re-classify bed joint sliding as a displacement controlled failure mechanism.

ASCE-41, FEMA-356, and FEMA-273 are performance based seismic codes which give different values of structural force reduction factor for the three performance levels presented in these codes; immediate occupancy (IO), life safety (LS), and collapse prevention (CP). For direct comparison with the other design codes considered in this study, life safety is the appropriate limit state to consider.

A summary of structural ductility parameters from various seismic design codes reviewed in this study is presented in Table 1.

**Table 1 – Comparison of Structural Ductility Parameters from Design Codes**

| CODE                               | ORIGIN    | Design Code Nomenclature |                                                                                                     |                     |                   | Recommended Code Values for URM |          |           |                                                                                                           |
|------------------------------------|-----------|--------------------------|-----------------------------------------------------------------------------------------------------|---------------------|-------------------|---------------------------------|----------|-----------|-----------------------------------------------------------------------------------------------------------|
|                                    |           | $\mu$                    | $R_\mu$                                                                                             | OSR                 | $R^\dagger$       | $\mu$                           | $R_\mu$  | OSR       | $R^\dagger$                                                                                               |
| NZS1170.5 (2004)<br>NZSEE (2006)   | NZ        | $\mu$                    | $k_\mu = 1 + (\mu - 1)T_1/0.7 \leq \mu$ ( $\xi = 5\%$ )<br>$k_\mu = 1/0.65 = 1.54$ ( $\xi = 15\%$ ) | $1/S_p$             | $k_\mu/S_p$       | 1.0-1.5                         | 1.0-1.54 | 1.0       | 1.0 - 1.54                                                                                                |
| AS1170.4 (2007)                    | Australia | $\mu$                    | $\mu$                                                                                               | $1/S_p$             | $\mu/S_p$         | 1.25                            | 1.25     | 1.3       | 1.62                                                                                                      |
| OPCM 3274 (2003)                   | Italy     | -                        | $q'$                                                                                                | $\alpha_u/\alpha_l$ | $q$               | -                               | 1.5-2.5  | 1.4 - 2.5 | 2.1 – 5.0                                                                                                 |
| Eurocode 8 (2004)                  | Europe    | -                        | -                                                                                                   | -                   | $q$               | -                               | -        | -         | 1.5 - 2.5                                                                                                 |
| ASCE-41 (2007)                     | USA       | -                        | -                                                                                                   | -                   | $m \times \kappa$ | -                               | -        | -         | $\kappa \times 3h_{eff}/L \geq 1.25$<br>(Rocking)                                                         |
| FEMA 356 (2000)<br>FEMA 273 (1997) | USA       | -                        | -                                                                                                   | -                   | $m \times \kappa$ | -                               | -        | -         | $\kappa \times 3.0 \geq 2.25$<br>(Bed joint sliding)<br>$\kappa \times 3h_{eff}/L \geq 1.25$<br>(Rocking) |

$$^\dagger R = R_\mu \times OSR$$

From Table 1 it is clear that the values of force reduction factor  $R$  recommended for the assessment of NZ unreinforced masonry buildings are lower than the values recommended in Australian and European seismic design codes. The values of  $R$  recommended for the assessment of NZ URM buildings are likely to be less than those recommended by the North American seismic design codes for rocking and bed joint sliding failure modes. As there is no force reduction permitted for diagonal tension or toe-crushing failure modes in the North American seismic codes, the NZ value of force reduction factor will be greater than or equal to the North American values for these failure modes.

### 3.3 Review of URM Structural Ductility Parameters in Published Research

In the past 30 years, much experimental research has been undertaken to determine appropriate values of structural ductility parameters for unreinforced masonry buildings. This research has primarily been focussed on Southern European unreinforced masonry construction, but the results obtained from this

research are applicable to unreinforced masonry structures worldwide (Tomazevic, 2007).

The experimental testing of unreinforced masonry structures can be divided into two main groups:

- Quasi-static in-plane testing of individual walls or pier-spandrel subassemblies
- Dynamic testing of entire buildings

Dynamic testing of buildings consisting of shaking table tests on reduced scale buildings has been undertaken by several researchers including: Tomazevic & Weiss (1994, 2010), Costley & Abrams (1995), and Benedetti et al. (1998). Shaking table tests by Tomazevic & Weiss (1994, 2010) on various configurations of clay and calcium silicate unreinforced masonry buildings found a range of structural ductility factors from 3.2 – 10. It was found that the ductility factor obtained from experimental testing is sensitive to structural configuration. In most cases, it was observed that damage limitation limit states govern the value of ductility factor, and the force reduction factors of 1.5 – 2.5 currently prescribed by Eurocode 8 are adequate and conservative. A range of overstrength factors from 1.42 – 5.27 were calculated for good quality masonry, with an average value of 1.48. Ultimate drift limits in excess of 1% were observed for clay masonry units, whereas ultimate drift limits between 0.4% and 0.9% were observed for calcium silicate units. Tomazevic & Weiss (2010) found an equivalent viscous damping ratio range of 2% - 10%.

Shaking table tests performed by Costley & Abrams (1995) found that pier rocking was a stable, repeatable failure mode and recommended a structural ductility factor of 2.0 for rocking. Ultimate drift limits were found to be 0.9%. Benedetti et al. (1998) conducted shaking table tests on a range of clay and stone masonry buildings and found a range of force reduction factors from 1.5 – 1.8 for clay masonry and 2.0 for stone masonry. An equivalent viscous damping ratio of 6%-10% was found for undamaged brick masonry.

Cyclic in-plane quasi-static tests on a series of brick masonry piers were undertaken by Magenes & Calvi (1997) and the range of structural ductility factor,  $\mu$  calculated was between 1.83 – 7.55 with a mean value of 4.46. All walls tested were found to fail in shear, with ultimate drifts ranging between 0.44% - 0.62% with a mean of 0.53%. The equivalent viscous damping coefficient recommended by this work was 10% for shear failures, and 15% for rocking failure modes. The ESECMaSE project (Fehling et al., 2008) consisted of a large range of both dynamic and quasi-static cyclic experimental tests at various universities throughout Europe and recommended force reduction factors of 1.5 for walls with aspect ratios  $\leq 1.0$ , linearly increasing to a force reduction factor of 2.0 for walls with aspect ratios  $\geq 1.6$ . Cyclic quasi-static testing of walls has been undertaken for walls representative of NZ URM construction by Knox (2012) and Russell et al. (2013). Russell et al. (2013) recommended an ultimate drift capacity of 0.7% for flanged walls and 0.4% for walls without flanges, with a suggested damping factor of 15% for walls failing in diagonal tension failure. Experimental results by Knox (2012) on pier and spandrel subassemblies and a two-storey perforated wall found ductility factors well in excess of 5.0. Knox (2012) also recommended values of structural ductility factor,  $\mu$  of 1.25 and structural performance factor,  $S_p$  of 0.925 for the in-plane assessment of URM walls based on the results of inelastic time-history modelling for a range of representative NZ URM buildings. The evaluation of overstrength ratio was explored further by Magenes (2006) by numerical analyses on a range of two and three storey unreinforced masonry buildings using equivalent frame modelling. Overstrength values ranging from 1.2 – 2.5 with a mean value of 1.7 were determined from this study.

## **4 PUSHOVER ANALYSES OF A REPRESENTATIVE NZ URM BUILDING**

### **4.1 Description of Building**

The representative building studied is a three storey building located in the Auckland CBD. The building was constructed in the early 1920's and consists of two above ground levels and one partially in-ground level due to the sloping site. For the analyses performed, no restraint was assumed to be provided by the ground to the basement walls. Modern and historical photographs of the building are shown in *Figure 3*.



(a) View of Southern wall (2012)



(b) View of Building looking Northwest (Adapted from Richardson, 1920's)

Figure 3: Photographs of building

The building consists of 350 mm thick exterior walls and 230 mm thick interior walls. All walls of the building are of unreinforced solid red clay brick construction. The floors of the building are constructed of timber flooring supported by timber joists spanning between exterior walls with intermediate support provided by internal walls. The roof of the building consists of profiled steel sheeting supported by timber trusses. Dimensions of the building were obtained from architectural drawings and confirmed by site measurement. In order to give a preliminary indication of the seismic resistance of the building, the wall density ratio was calculated as the sum of the cross-sectional walls in each orthogonal direction divided by the total floor area of the building. This information is presented in Table 2.

Table 2 – Wall density ratios

| LEVEL    | X-direction        |                    |              | Y-direction        |                    |             |
|----------|--------------------|--------------------|--------------|--------------------|--------------------|-------------|
|          | 350 mm thick walls | 230 mm thick walls | TOTAL        | 350 mm thick walls | 230 mm thick walls | TOTAL       |
| Level 1  | 4.7%               | 3.0%               | <b>7.8%</b>  | 4.0%               | 2.7%               | <b>6.8%</b> |
| Ground   | 4.5%               | 3.0%               | <b>7.4%</b>  | 2.8%               | 2.6%               | <b>5.4%</b> |
| Basement | 6.2%               | 4.1%               | <b>10.3%</b> | 4.5%               | 2.6%               | <b>7.1%</b> |

Using the rules for “simple masonry buildings” in Section 9.7 of Eurocode 8 (CEN-EN 1998-1, 2004), the wall density ratios calculated for both orthogonal directions at all storeys of the building would classify the building sufficient to resist a peak ground acceleration of 0.10g if the building is considered as a three-storey building, or 0.15g if the building is considered as a two-storey building. Due to the presence of the 230 mm thick internal walls, it is likely that the observed wall density ratios for the building considered are larger than those present in many NZ URM buildings which would use timber stud walling for internal walls. It is for this reason that the pushover analyses were also performed for the case of no internal walls.

Bed joint shear tests were undertaken at 4 locations throughout the structure, and samples of brick and mortar were taken from these locations to determine the compressive strengths of the brick ( $f'_b$ ) and mortar ( $f'_j$ ). Masonry compressive strength ( $f'_m$ ) was calculated according to NZSEE (2006) and masonry density ( $\rho_m$ ) and modulus of elasticity ( $E_m$ ) were estimated using equations in University of Auckland (2011). A summary of these material parameters is presented in Table 3.

Table 3: Elastic material properties

|        |         |                                                |                 |                                 |
|--------|---------|------------------------------------------------|-----------------|---------------------------------|
| $f'_b$ | $f'_j$  | $f'_m = 0.7f'_b{}^{0.75} \times f'_j{}^{0.30}$ | $E_m = 300f'_m$ | $\rho_m = 1578 + 5f'_b + 8f'_j$ |
| 32 MPa | 1.2 MPa | 10 MPa                                         | 3000 MPa        | 1750 kg/m <sup>3</sup>          |

## 4.2 Finite Element Model of Building

The commercial finite element software DIANA (TNO DIANA, 2012) was used to perform a Nonlinear Static (Pushover) analysis of the building. The building geometry was imported into

DIANA from a 3D CAD file prepared from the architect's drawings. In the cases where an effectively rigid diaphragm was assumed, the timber floor and roof diaphragms were modelled as an isotropic linear elastic membrane of uniform thickness of 100 mm and an Elastic Modulus of 10 000 MPa. The density of the floor membranes was adjusted so that the floor mass was equivalent to the dead and reduced live load present on the floors and roof of the actual structure. The external and internal masonry walls of the building were modelled using CQ40S curved shell elements (TNO DIANA, 2012). The smeared crack continuum modelling strategy of the building accounted for both geometrical and material nonlinearities. The cracking model used to replicate the nonlinear material behaviour of unreinforced masonry walls consisted of a total strain fixed-crack constitutive model (TNO DIANA, 2012). This method has successfully been used to model unreinforced masonry in previous work including Ademovic et al. (2013), Lourenco et al. (2011), Pena et al. (2008), and Ramos & Lourenco (2004). Nonlinear material properties used for the unreinforced masonry walls of the building are presented in Table 4.

**Table 4: Material Properties for Nonlinear Static Analysis**

| Tensile Strength                                               | Tensile Fracture Energy | Fracture Energy in Compression | Shear Retention Factor | Crack Bandwidth                  |
|----------------------------------------------------------------|-------------------------|--------------------------------|------------------------|----------------------------------|
| $f_t$                                                          | $G_t$                   | $G_c$                          | $\beta$                | -                                |
| These parameters based on recommendations from Lourenço (2008) |                         |                                | -                      | DIANA default value              |
| 0.5 MPa                                                        | 0.1 N/mm                | 16.0 N/mm                      | 0.05                   | $(\text{Area of element})^{1/2}$ |

In order to check the geometry, meshing, loading and boundary conditions of the nonlinear model, a preliminary linear static analysis was undertaken. The sum of reactions was computed and was verified to match hand calculations.

The Nonlinear Static (Pushover) analyses of the building were undertaken for both effectively rigid and completely flexible (i.e. absent) floor and roof diaphragm assumptions. For the case of the rigid diaphragm analyses, full connection was assumed to be present between walls and all floor and roof diaphragms. The effect of removing the internal walls of the structure was also considered. A summary of the pushover analyses undertaken is presented in Table 5.

**Table 5: Summary of pushover analyses performed**

| Pushover Run ID | Loading Direction | Diaphragm condition | Internal wall condition     |
|-----------------|-------------------|---------------------|-----------------------------|
| RDX+            | +X                | Rigid diaphragm     | Internal walls modelled     |
| RDX-            | -X                | Rigid diaphragm     | Internal walls modelled     |
| RDY+            | +Y                | Rigid diaphragm     | Internal walls modelled     |
| RDY-            | -Y                | Rigid diaphragm     | Internal walls modelled     |
| NDX+            | +X                | No diaphragm        | Internal walls modelled     |
| NDX-            | -X                | No diaphragm        | Internal walls modelled     |
| NDY+            | +Y                | No diaphragm        | Internal walls modelled     |
| NDY-            | -Y                | No diaphragm        | Internal walls modelled     |
| NINTX+          | +X                | Rigid diaphragm     | Internal walls not modelled |
| NINTY+          | +Y                | Rigid diaphragm     | Internal walls not modelled |

For each of the pushover analyses in Table 5, the following procedure was undertaken. Vertical loads consisting of dead load and a reduced live load were applied first. To simplify the analysis of the models with no floor diaphragms, the dead and live loads due to the floor diaphragms were not considered. This is a reasonable simplification, as over 80% of the structure seismic weight is concentrated in the walls of the building. Earthquake loading of the building was then simulated by applying a uniform horizontal acceleration resulting in a loading proportional to the mass of the structure. The horizontal loading was increased in a stepwise fashion. The pushover analyses were terminated when the solution ceased to converge.



In order to assess the global nonlinear behaviour of the structure, force versus displacement was monitored for a control node located at the centre of mass of the roof level. For the case where no floor diaphragm was present, the displacement at each of the four corner points at the roof of the structure were averaged in order to give an equivalent displacement at a central control node. The force versus displacement graphs for the X and Y directions are presented in Figure 4 (a) and (b) respectively.

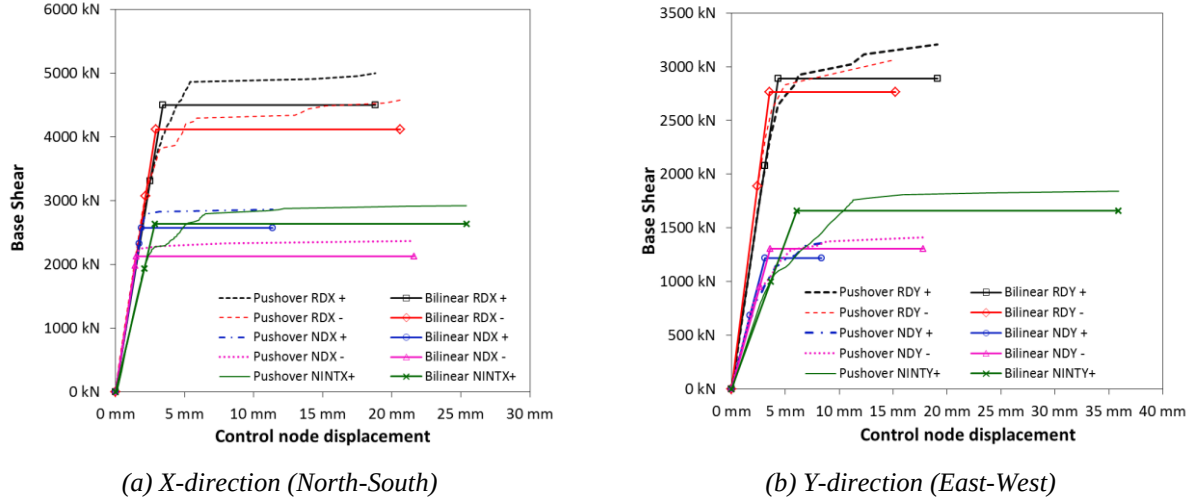


Figure 4: Force vs. displacement graphs for all pushover analyses undertaken

From Figure 4 (a) and (b) it is clear that for the building studied there is no sudden reduction in load carrying capacity of the structure, but rather the behaviour shown is almost perfect elastic-plastic with significant ductility. This force-displacement relationship is consistent with experimental results observed for piers failing by rocking and bed-joint sliding such as those presented by Russell et al. (2009). Using the bilinearisation process outlined in Figure 2, the ductility factor for each of the pushover analyses undertaken was calculated as shown in Table 6.

Table 6: Calculated ductility values from pushover analyses

| PUSHOVER RUN | $W_t$ | $d_{cr}$ | $H_{cr}$ |      | $d_e$ | $H_u$         |      | $H_{max}$ |      | $d_u$ | $\mu = d_u/d_e$ | $S_p = H_{cr}/H_u$ | $\mu_{lim} = 3d_{cr}/d_e$ |
|--------------|-------|----------|----------|------|-------|---------------|------|-----------|------|-------|-----------------|--------------------|---------------------------|
|              | kN    | mm       | $V/W_t$  | kN   | mm    | $a_u = V/W_t$ | kN   | $V/W_t$   | kN   | mm    | -               | -                  | -                         |
| RDX+         | 4720  | 2.5      | 0.70     | 3304 | 3.4   | <b>0.95</b>   | 4503 | 1.06      | 5003 | 18.8  | <b>5.50</b>     | 0.73               | 2.20                      |
| RDX-         | 4720  | 2.2      | 0.65     | 3068 | 2.9   | <b>0.87</b>   | 4121 | 0.97      | 4578 | 20.6  | <b>7.07</b>     | 0.74               | 2.23                      |
| RDY+         | 4720  | 3.1      | 0.44     | 2077 | 4.3   | <b>0.61</b>   | 2889 | 0.68      | 3210 | 19.1  | <b>4.40</b>     | 0.72               | 2.16                      |
| RDY-         | 4720  | 2.4      | 0.40     | 1888 | 3.6   | <b>0.59</b>   | 2761 | 0.65      | 3068 | 15.2  | <b>4.28</b>     | 0.68               | 2.05                      |
| NDX+         | 3815  | 1.7      | 0.61     | 2327 | 1.9   | <b>0.68</b>   | 2575 | 0.75      | 2861 | 11.4  | <b>6.05</b>     | 0.90               | 2.71                      |
| NDX-         | 3815  | 1.4      | 0.52     | 1984 | 1.5   | <b>0.56</b>   | 2129 | 0.62      | 2365 | 21.6  | <b>14.35</b>    | 0.93               | 2.80                      |
| NDY+         | 3815  | 1.7      | 0.18     | 687  | 3.1   | <b>0.32</b>   | 1219 | 0.36      | 1354 | 8.4   | <b>2.71</b>     | 0.56               | 1.69                      |
| NDY-         | 3815  | 2.6      | 0.25     | 954  | 3.6   | <b>0.34</b>   | 1305 | 0.38      | 1450 | 17.8  | <b>4.93</b>     | 0.73               | 2.19                      |
| NINTX+       | 3231  | 2.1      | 0.60     | 1939 | 2.8   | <b>0.81</b>   | 2632 | 0.91      | 2924 | 25.4  | <b>9.00</b>     | 0.74               | 2.21                      |
| NINTY+       | 3231  | 3.7      | 0.31     | 1002 | 6.1   | <b>0.51</b>   | 1658 | 0.57      | 1842 | 35.9  | <b>5.93</b>     | 0.60               | 1.81                      |

| Average | Average | Average |
|---------|---------|---------|
| 6.42    | 0.74    | 2.21    |
| C.O.V   | C.O.V   | C.O.V   |
| 0.51    | 0.15    | 0.15    |

From Table 6 it can be seen that the ultimate base shear coefficient,  $a_u = H_u/W_t$  ranges from 0.32 – 0.61 for the weaker Y direction of the building, and from 0.56 – 0.95 for the stronger X direction of the building. In order to give an indication of the strength of the building, we can compare these ultimate base shear coefficients, that is, accelerations to those recorded in the Christchurch CBD during the February 22<sup>nd</sup> 2011 earthquake. The maximum ground acceleration recorded in the Christchurch CBD during this earthquake was approximately 0.7g (Cooper et al., 2012a), that is, a base shear coefficient of 0.7. From the results presented in Table 6 it is obvious that the weaker Y direction of the building would likely suffer significant damage if subjected to this earthquake, regardless of the diaphragm configuration. For the stronger X direction of the building, the level of damage during such earthquake would range from minor to severe depending on the actual diaphragm configuration present.

From Table 6 it can also be seen that the structural ductility factor,  $\mu$  for all loading directions is in excess of 2.0 for all cases considered. The calculated structural performance factor  $S_p$  is also in reasonable agreement with the value of 0.7 which would be obtained from the NZS 1170.5 (2004) equations using a ductility factor,  $\mu$  of 2.0. For the analyses presented in Table 6, a limiting ductility factor  $\mu_{lim}$  has also been computed using the recommendation of three times the displacement at first cracking as proposed by Tomazevic (2007) to reduce the damage to non-structural components. The limiting ductility factor is also greater than 2.0 for all but one of the cases considered.

In order to better understand the failure modes of the building, the maximum principal strain contours occurring at the ultimate displacement were plotted on the deformed shape of the building in *Figure 5*, *Figure 6*, and *Figure 7*, giving an indication of the locations where significant damage is likely to occur.

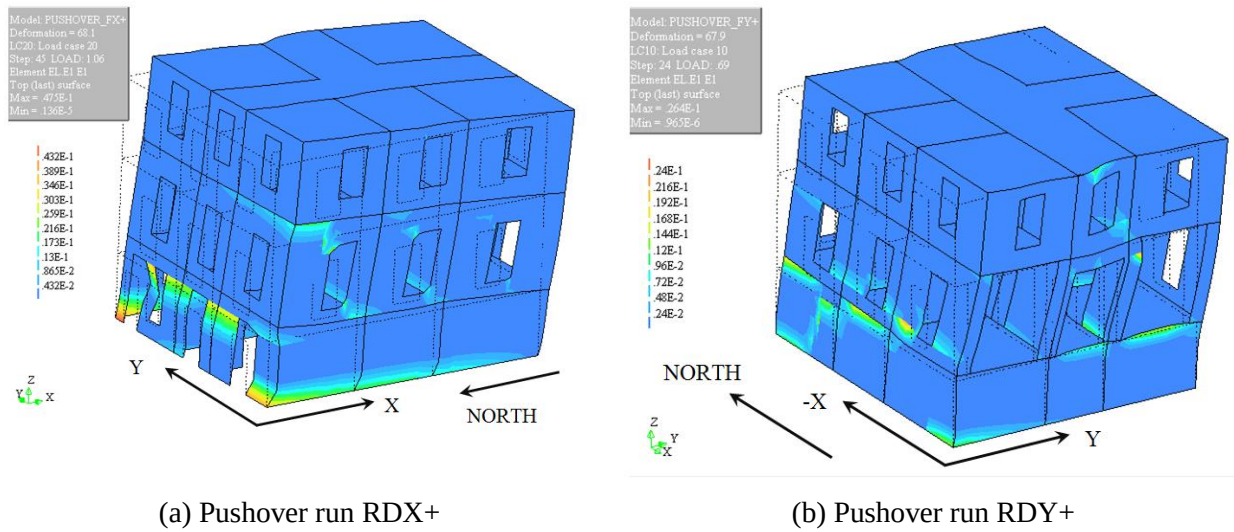


Figure 5: Principal strain contours identifying damage at the ultimate displacement for models with rigid diaphragms and all walls included

From *Figure 5 (a)* it can be seen that failure in the X-direction is dominated by a rocking / sliding response of piers at the lower levels of the building and that flanges perpendicular to the direction of seismic action are also engaged. From *Figure 5 (b)*, failure in the Y-direction can be seen to consist of rocking failure at the ground level of the building. Significant engagement of flanges can also be observed from the damage pattern in the Y-direction.

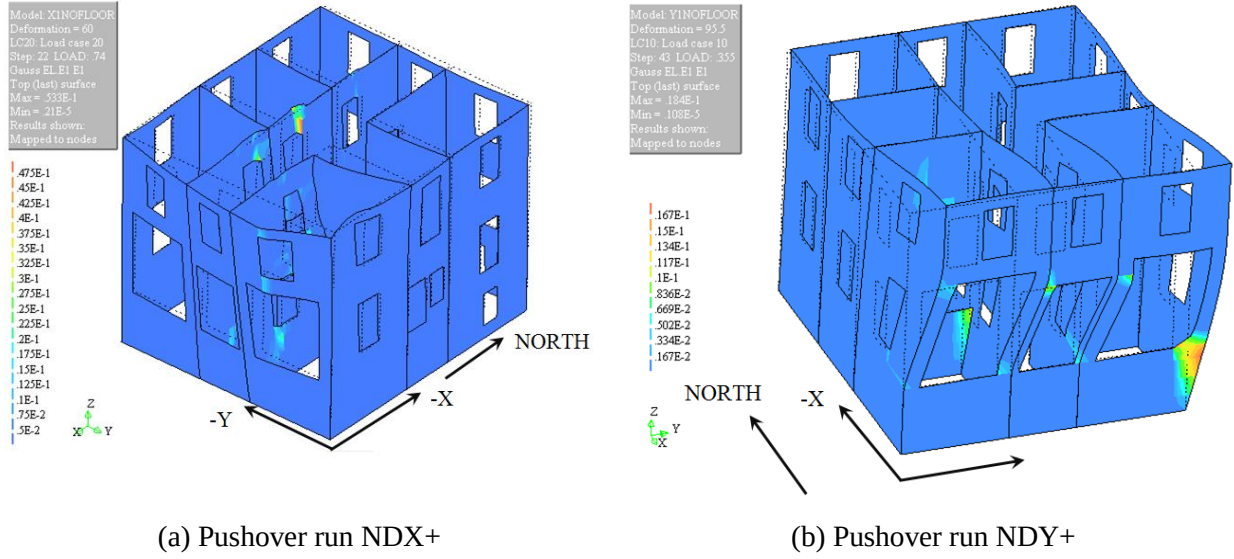


Figure 6: Principal strain contours identifying damage at the ultimate displacement for models with no diaphragms and all walls included

From Figure 6 (a), it can be seen that there is significant out-of-plane displacement of the Southern wall of the building due to the lack of restraint that would otherwise be provided from floor diaphragms. Signs of damage due to out-of-plane horizontal bending can also be observed on the Southern wall. Figure 6 (b) shows that without a rigid diaphragm to tie the walls together, the structural response is dominated by in-plane deformation of the Southern wall of the building, with the Northern wall remaining almost elastic with very little displacement or damage. From both Figure 6 (a) and (b) it is clear that the full in-plane strength of the walls cannot be developed due to the presence of localised failure mechanisms. This reduction in capacity is shown in Figure 4 where it can be seen that the presence of rigid floor diaphragms results in approximately double the ultimate base shear than that obtained with no floor diaphragm. The presence of a rigid diaphragm also results in less variation of ductility factor,  $\mu$  with respect to direction of loading due to the suppression of local failure effects which are present for the no-diaphragm case.

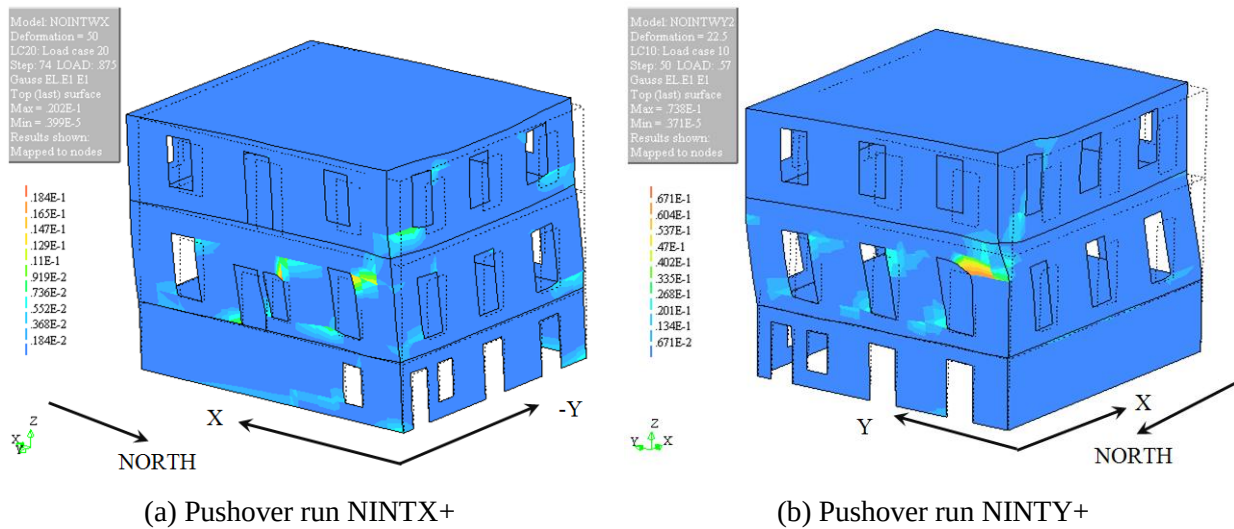


Figure 7: Principal strain contours identifying damage at the ultimate displacement for models with rigid diaphragms and internal walls not included.

For the case with internal walls removed, it can be seen from *Figure 7 (a) and (b)* that pier rocking at ground level is the predominant failure mode in both directions. From *Figure 4* it can be observed that, for each direction, the removal of internal walls results in an ultimate base shear that is approximately 60% of the value obtained for the case with all internal walls present. It is interesting to note that this ratio is approximately equal to the ratio of the area of external walls to total area of walls for each direction.

## 5 SIMPLIFIED PLANE STRESS ANALYSIS OF BUILDING TO NZSEE 2006

To assist in the interpretation of the results from the Nonlinear Static analysis, a simplified plane stress analysis of the structure was undertaken. The shear capacities of the piers of all walls of the building were calculated in accordance with NZSEE (2006) plane stress analysis equations which account for rocking, sliding, and diagonal tension wall failure modes. A cohesion of 0.3 MPa as obtained from bed joint shear tests was used in the NZSEE equations. Vertical compressive stresses acting on the piers were obtained from the preliminary linear static analysis of the building. To assess the global behaviour of the structure, the storey shear capacities in each direction were determined by summing the capacities of the individual pier elements in each direction. The capacity calculated using the NZSEE (2006) simplified equations was then compared with the storey shear capacities obtained from the Nonlinear Static analysis of the building. A comparison of these values is presented in Table 7.

**Table 7: Comparison of NZSEE Simplified Plane Stress Analysis to Nonlinear Static Analysis**

| LEVEL    | Storey Shear Capacities Using NZSEE (2006) Equations |               | Storey Shear Capacities Obtained from Nonlinear Static Analyses with Rigid Diaphragm Condition |               | Relative Difference |               |
|----------|------------------------------------------------------|---------------|------------------------------------------------------------------------------------------------|---------------|---------------------|---------------|
|          | $\sum V_{ix}$                                        | $\sum V_{iv}$ | $\sum V_{ix}$                                                                                  | $\sum V_{iv}$ | $\sum V_{ix}$       | $\sum V_{iv}$ |
| Level 1  | 565 kN                                               | 490 kN        | 720 kN                                                                                         | 480 kN        | 27%                 | -2%           |
| Ground   | 1276 kN                                              | 716 kN        | 2313 kN                                                                                        | 1542 kN       | 81%                 | 115%          |
| Basement | 3789 kN                                              | 2079 kN       | 3807 kN                                                                                        | 2538 kN       | 0%                  | 22%           |

The observed failure modes of the piers at the basement level of the building consisted of bed joint sliding for most walls with low aspect ratios with rocking failure modes predicted for basement level walls with aspect ratios  $H/L > 2.5$ . In the upper levels of the building, sliding shear failures were also predicted for most walls, with rocking failures predicted for walls with higher aspect ratios of  $H/L > 2.5$ .

The storey shear capacities obtained from the simplified NZSEE plane stress method are in good agreement with the values predicted from the Nonlinear Static Analysis for Level 1 and Basement Levels. However, there is a significant difference between the capacities of walls at Ground Level, with the shear capacities obtained from the Nonlinear Static Analysis significantly larger than those predicted using NZSEE equations. This difference may be attributed to several factors, such as the beneficial effect of flange action which can be observed based on the damage patterns in *Figure 5*. The failure modes predicted by the simplified plane stress analysis were also in reasonable agreement with those predicted by the pushover analysis. One obvious deficiency in the NZSEE method is the inability to predict damage to spandrels as was observed in *Figure 5 (c) and (d)*. Another limitation of the simplified plane stress analysis is the fact that flange action is not considered. It is obvious from *Figure 5* that flanges perpendicular to the direction of seismic action are being utilised in the resistance of the structure in both directions.

## 6 CONCLUSIONS AND FURTHER WORK

Based on the literature reviewed in this study, a structural ductility factor,  $\mu$  greater than 1.0 is certainly justified for unreinforced masonry buildings in New Zealand and Australia. A structural ductility factor,  $\mu = 1.25$  with a corresponding structural performance factor,  $S_p = 0.925$  calculated to NZS1170.5, and an equivalent viscous damping coefficient  $\xi = 5\%$  would be a conservative set of values to use for the assessment of unreinforced masonry buildings in NZ and Australia.

The results of the Nonlinear Static analysis of a representative NZ unreinforced masonry building suggest that values of  $\mu = 2.0$ ,  $S_p = 0.7$ , and  $\xi = 5\%$  are reasonable for the assessment of URM buildings. These values also match previous analytical and experimental results well, but should be justified further by nonlinear static and time history analyses of a wide range of NZ unreinforced masonry buildings. The influence of intermediate diaphragm flexibility between the fully rigid and fully flexible case, and loading distribution will also be considered in future studies.

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