FORCE-BASED ELASTIC SEISMIC ASSESSMENT OF NEW ZEALAND UNREINFORCED MASONRY BUILDINGS

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ABSTRACT

Because of the need for a straightforward procedure for assessing the performance of low rise unreinforced masonry structures in New Zealand a force-based, elastic method of analysis for a critical design earthquake is proposed. The comparatively low value of many URM structures in New Zealand justifies a simplified approach and a force-based method has the advantage of familiarity and simplicity. The criteria which the performance of existing buildings must be measured against in New Zealand is defined as one-third the strength of a design earthquake. The procedure for determining the seismic demand to which an existing structure is subjected, as well as how the structure can be analysed to determine the capacity of masonry walls responding in-plane to withstand that demand, are described. It is determined that an equivalent static analysis (ESA) procedure is appropriate for the seismic assessment of low-rise URM structures in New Zealand.

KEYWORDS: Unreinforced masonry, seismic assessment, New Zealand.

INTRODUCTION

Unreinforced masonry (URM) buildings have been shown to perform poorly when subjected to lateral forces which are induced in an earthquake. New Zealand is a seismically active country and lies on the boundary between the Australian tectonic plate to the west, and the Pacific tectonic plate to the east. Recently a collaborative research programme has been undertaken by The University of Auckland and The University of Canterbury to develop seismic retrofit solutions for New Zealand's earthquake risk buildings. Within this programme it has been recognised that buildings constructed of URM pose the greatest risk in terms of safety in an earthquake. Legislation has recently been introduced in New Zealand, which requires that buildings deemed to be earthquake-prone should be managed so that injury or death to people and damage to any other property as a result of an earthquake is prevented [1]. Therefore it is necessary to analyse such structures to determine if they are earthquake prone. The New Zealand Society for Earthquake Engineering (NZSEE) published guidelines in 2006 [2] for assessing the seismic performance of existing structures, but these guidelines are considered overly complex and too detailed to follow. The American Society of Civil Engineers (ASCE) has published similar guidelines [3], but this too is often considered overly complicated and detailed for the majority of URM structures. The need for a simplified procedure has been recognized and such a procedure should clearly identify what demand the structure is required to be subjected to, and how the structure can be analysed to determine its capacity to withstand that demand. It has also been acknowledged that detailed modelling is not a popular technique for assessing low rise URM structures, particularly because of the inherent uncertainty in the material properties of URM, and because of the comparatively low value of the structure. Moreover the criteria against which the performance of the building is to be measured needs to be clearly defined and understood.

BACKGROUND

It has been identified that the New Zealand URM building stock can be classified into seven typologies, which are outlined in Table 1. Buildings are separated according to storey height, and whether they are an isolated stand-alone building or a row building made up of multiple residences joined together in the same overall structure [4].

Table 1: NZ URM building typologies

Typology	Description	Importance	Prevalence (rank)	R_u (design life)	
				50 years	100 years
A	One storey isolated	2	4	1.0	1.3
B	One storey row	2	3	1.0	1.3
C	Two storey isolated	2	2	1.0	1.3
D	Two storey row	2	1	1.0	1.3
E	Three + storey isolated	2	7	1.0	1.3
F	Three + storey row	2	6	1.0	1.3
G	Monumental	2, 3, 4	5	-	-

The table above shows the importance level from AS/NZS.1170.0 [5] and the ranking of typologies in NZ. Most smaller structures are importance level 2, which means "Medium consequence for loss of human life, or considerable economic, social or environmental consequences" [5]. This accounts for most one and two storey buildings, as none have an importance as low as level 1, and few are as high as importance level 3. The ranking shows that the most common URM structures in New Zealand are one and two storey in height. Buildings which are three or more storeys are classified into two groups, isolated or row structures, but no further distinction is drawn on the basis of height. There are too few four, five and six storey buildings to warrant separate typologies. These buildings are less uniform and of greater value than one and two storey URM buildings, and as such will usually require a unique analysis. (Typology G, monumental structures will always require a unique analysis.) Furthermore, the investment associated with assessment and retrofit of such buildings will be greater than for low rise (one and two storey) buildings. Conversely, the value of low rise structures means that a detailed and unique analysis would not be justified for each building, and it would be more appropriate to follow a standardised and simplified approach. Low rise structures make up the majority of the New Zealand building stock and an assessment method familiar to designers and not overly time consuming is required. Thus, a force-based, elastic procedure to analyse a structure to withstand a critical design earthquake is proposed.

NEW ZEALAND EXISTING BUILDING SEISMIC PERFORMANCE CRITERIA

The Building Act 2004 [1] and its associated regulations define the seismic performance criteria which existing buildings must meet. "A building is earthquake prone for the purposes of this Act if the building will have its ultimate capacity exceeded in a moderate earthquake," and a moderate earthquake is defined as "an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one third as strong as, the earthquake shaking that would be used to design a new building at that site." The New Zealand Society for Earthquake Engineering (NZSEE) has interpreted these definitions so that a building which is earthquake-prone would essentially perform in a design earthquake at a level less than 33 % of the standard of a new building, 33 % New Building Standard, or 33 %NBS [2]. This relates to the building's ultimate limit state capacity, as defined by current design standards. As such, an existing building must be assessed to determine whether its ultimate limit state capacity in an earthquake is greater than or less than one-third of the ultimate limit state capacity of a new building constructed on the same site. The limit of 33 %NBS is prescribed by legislation, but it is the view of NZSEE that 67 %NBS would be a more appropriate target. It is considered that 33 %NBS corresponds to approximately 20 times the risk of the building reaching a similar condition to that which a new building would reach in a full design earthquake, and 67 %NBS corresponds to a risk of 3 times. It is the decision of the building owner as to which level the building must reach, with 33 %NBS as a required minimum.

SEISMIC DEMAND

For low rise URM buildings (typology A, B, C and D) a simple and convenient method for analysing the structure is an equivalent static analysis, ESA. In this case a horizontal base shear is determined and distributed as equivalent static forces at each level of the structure. When analysing an existing building to determine if it is earthquake-prone, the horizontal seismic base shear is scaled by one-third, and the capacity of the structure to withstand these forces at its ultimate limit state is assessed. The New Zealand Design Standard NZS:1170.5 [6] for determining earthquake actions sets criteria for when the equivalent static method can be used. The structure must be less than 10 m tall, or the structure should not be classified as irregular and the largest translational period should be less than 2 seconds. NZSEE [2] further notes that the equivalent static method is applicable when during a "design level" earthquake the lateral force resisting systems of the structure respond elastically, or the structure should have a low ductility demand/capacity ($\mu < 2$) provided that there are no significant geometric irregularities. It also states that the building may be up to 30 m tall. All these criteria are widely met in most low rise URM structures typically found in New Zealand and therefore ESA is considered an appropriate method of analysis.

A shown in Equation 1, the base shear, V, is the product of seismic weight, W_t, of the structure and the horizontal design action coefficient, $C_d(T_1)$, which can be determined as shown in Equation 2.

$$V = C_d(T_1) \cdot W_t \tag{1}$$

$$C_d(T_1) = \frac{C(T_1) \cdot S_p}{k_\mu} \tag{2}$$

k_μ is a factor which depends on the ductility factor of the structure. The concept of a ductility factor, μ, defined as the maximum displacement divided by the displacement at yielding, is somewhat meaningless for URM structures. Whether a structure exhibits a ductile response depends on the failure mode. If its ultimate capacity is exceeded because of a shear dominated failure, then the structure is likely to be classified as brittle. A brittle structure is defined in NZS.1170.5 "as a structure with structural components that are not capable of inelastic deformation without undergoing sudden and significant loss of strength. The structural ductility factor, for brittle structures shall be taken as 1.0." As such, a ductility factor of $\mu = 1$ would seem appropriate for a shear failure dominated structure. As a material URM may be brittle, but as a structural form there may be some available "apparent" ductility in the system. If the response of the structure is characterised by rocking or sliding of the in-plane walls, then some inelastic deformation may be possible without collapse. It would seem correct to take $\mu > 1$ for a sliding or rocking dominated structure, but a value is problematic to quantify because of the difficulty associated with determining deflections once inelastic deformations have commenced. Part of the function of a ductility factor is to recognise the damping in the system, from any source, and if $\mu = 1$ then no damping is assumed. It is suggested by NZSEE [2] that a ductility factor of $\mu = 1.5$ would be suitable to take into account the role of any damping. This can be used to determine inertia forces ($\mu > 1$ results in an increased k_μ and consequently a reduction in the horizontal design action coefficient), but that the final deflections should not then be assumed as 1.5 times the elastic deflections. Therefore to account for damping but to disregard any potential for inelastic deformations, a ductility factor of $\mu = 1.5$ is appropriate for URM structures for determining the horizontal design action coefficient, but not for determining inelastic displacements.

S_p, the structural performance factor, represents a number of effects which are not explicitly represented in an analysis. $S_p < 1$ is potentially non-conservative for stiff, short period structures which includes many URM buildings. For determining $C_d(T_1)$, the horizontal seismic design action coefficient, damping is already taken into account with $\mu = 1.5$ and $k_\mu > 1$, and because of the uncertainty in the materials involved, it is not considered appropriate to reduce the design action further, and as such, S_p should be taken as unity.

Thus for determining the horizontal design action coefficient, Equation 1 becomes Equation 3 below:

$$C_d(T_1) = \frac{C(T_1)}{k_\mu} \tag{3}$$

$$C(T_1) = C_h(T) \cdot Z \cdot R \cdot N(T,D) \tag{4}$$

Equation 4 gives the elastic site hazard spectrum ordinate. All factors in Equation 4 can be taken from NZS.1170.5, according to the location and site of the building, its soil type and proximity to a fault line. It should be noted that the return period factor, R_u, is related to the design life and importance level of the structure. The design life when analysing an existing URM structure in

New Zealand should be at least 50 years in most cases, and possibly up to 100 years, particularly because such structures are for the most part more than 80 years old. Such structures maintain an important heritage value which needs to be considered for the future design life. Apart from unique and monumental URM structures (typology G) most other buildings can be considered to have an importance level of 2. A structure of importance level 2 and with a 50 year design life corresponds with an annual probability of exceedance for the ultimate limit state of 1/500, and similarly a 100 year design life corresponds with an annual probability of exceedance of 1/1000. Thus for any given URM structure, the return period factor is either 1.0 for a 50 year design life or 1.3 for a 100 year design life.

When the equivalent static method of analysis is used the fundamental period of the building can be calculated by either an analytical, empirical or approximate method. In an analytical method, a dynamic (eigenvalue) analysis of the mathematical model can be carried out to determine the fundamental period of the building. As a simplified method for assessing a building's performance is desired, this method is not addressed here. NZSEE [2] suggests an empirical method of determining period as per Equation 5.

$$T_1 = \frac{3}{32}\sqrt{A_i\left(0.2+\left(\frac{l_{wi}}{h_n}\right)^2\right)}h_n^{0.75} \tag{5}$$

where, A_i = effective cross-sectional area of shear wall i in the first storey of the building in m^2,
h_n = height from the base of the structure to the uppermost seismic mass,
l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces.
An approximate method is suggested by NZSEE [2] as per Equation 6.

$$T = \sqrt{3.07 \cdot U_d} \tag{6}$$

where, U_d = the maximum in-plane diaphragm displacement in metres, due to a lateral force in the direction under consideration, equal to the weight tributary to the diaphragm.
In this case the diaphragm stiffness must be known, or at least estimated. This is not addressed here. Finally NZS.1170.5 recommends that when the equivalent static method of analysis is used, the largest translational period may be calculated using the Rayleigh method, which may be the method most familiar to designers, and is given in Equation 7.

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n}\left(W_i d_i\right)^2}{g\sum_{i=1}^{n}\left(F_i d_i\right)}} \tag{7}$$

In order to determine the stiffness of the structure, ASCE [3] suggests that the behaviour of solid shear walls can be depicted at low forces levels using conventional principles of mechanics for homogenous materials, and that in such cases, the lateral in-plane stiffness of a solid cantilevered shear wall, k, can be calculated using Equation 8.

$$k = \frac{1}{\frac{h_{eff}^{3}}{3E_m I_g} + \frac{h_{eff}}{A_v G_m}} \tag{8}$$

where, h_{eff} = wall height,

A_v = shear area,

I_g = moment of inertia for the gross section representing uncracked behaviour,

E_m = masonry elastic modulus,

G_m = masonry shear modulus.

It is noted that the condition of a wall pier with full restraint against rotation at its top and bottom is often not present in actual buildings. If this condition is met an equation for determining stiffness is provided in [3], but in most cases Equation 8 is more appropriate.

Once the horizontal design action coefficient and the seismic weight are determined, noting that much of the weight of a URM structure is in the walls, the base shear can be calculated. This base shear force is then applied as equivalent static horizontal forces at different heights on the structure, as per Equation 9. This assumes a shape function obtained from static deflections due to the self-weight of the structure. In modern buildings most of the mass is concentrated at floor levels, particularly in frame type structures with concrete floors. In URM bearing wall construction, (many of which have timber floors) most of the mass is in the walls themselves. For a given storey the mass of the walls contributing to the seismic weight could exceed the contribution of the floors by up to 50 %. Thus it would seems more appropriate to lump the mass at the centre of the wall between floors and to distribute the seismic weight of the floor equally to the node above and below. This will more accurately represent the magnitude and location of the horizontal forces induced by the ground acceleration. When parapets are present on the top of the structure, it would also be appropriate to lump their mass at the roof level. NZS.1170.5 also makes allowance for a lateral force, F_t of 0.08V (Equation 5), to be applied at the top level of the building, which compensates for the effects of higher mode effects in the upper storeys. Once the equivalent static horizontal force at each node is determined, the shear which acts in each storey can also be determined. See Figure 1. NZS.1170.5 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of ± 0.1 times the plan dimension of the structure at right angles to the direction of loading. This results in a torsional action about the centre of resistance (in plan) of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. Moreover, for nominally ductile and brittle structures NZS.1170.5 requires that the structure be analysed with 100 % of the base shear action along one main axis of the building, simultaneously with 30 % of the base shear acting in the orthogonal direction. Both orthogonal axes are to be analysed accordingly. Thus the magnitude of the force which each in-plane wall in the structure must withstand, V_i, can be calculated. The forces transferred into the diaphragms, the walls responding out-of-plane and the parapets can also be determined from the above analysis, but the capacity of these elements to withstand such forces is not addressed here.

$$F_i = F_t + 0.92V \cdot \frac{W_i h_i}{\sum_{i=1}^{n} \left(W_i h_i \right)} \tag{9}$$

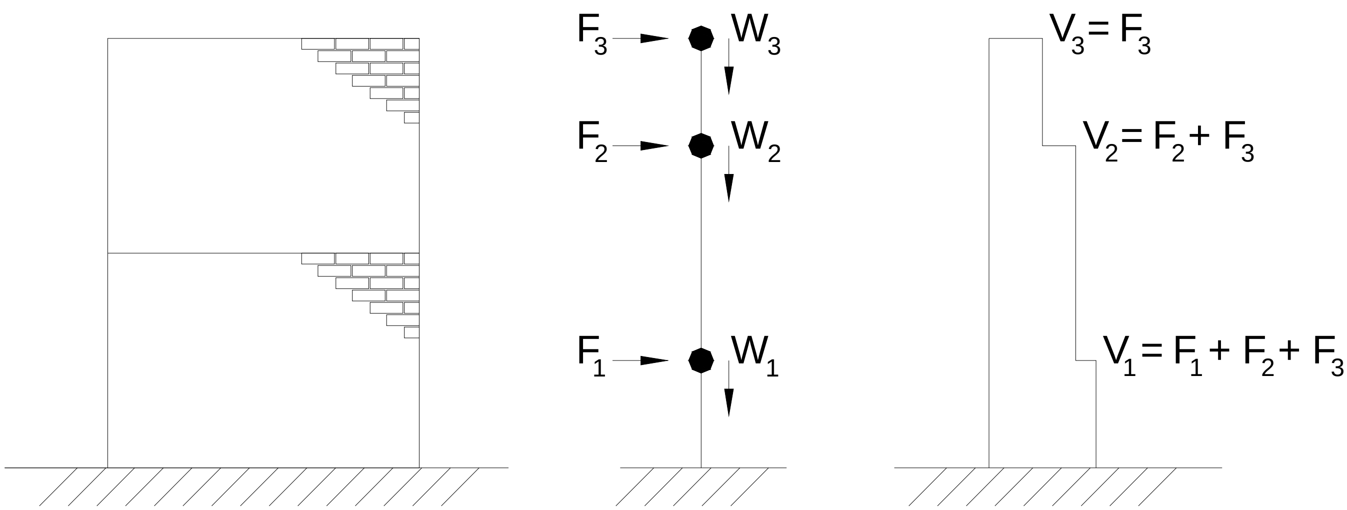

Figure 1: Two storey URM analysis: (a) Building model; (b) Equivalent static horizontal forces; (c) Storey shears

CAPACITY OF IN-PLANE URM WALLS

Generally in an equivalent static analysis the lateral seismic forces corresponding to the development of the first plastic hinge give a lower bound for the probable lateral force capacity of the structure. In URM structures, no plastic hinges develop, and the lower bound capacity corresponds to the elastic limit of the weakest (or most critical) structural element, with some allowance for damping taken into account as detailed above. The elastic limit of masonry walls subjected to seismic actions and gravity actions and responding in-plane depends on the failure mechanism. See Figure 2. The principal in-plane failure mechanisms can be summarised as follows [7, 8]:

- Sliding shear failure: Potential sliding planes can form at the base of walls on cracked bed joints or damp-proof courses (moisture/vapour barriers), and entire parts of the wall move on this plane.
- Diagonal shear failure: A critical combination of principal tensile and compressive stresses as a result of applying combined shear and compression leads to the formation and development of inclined diagonal cracks. Depending on the relative strength of mortar joints, brick-mortar interface, and brick units, two types of cracking may be observed, joint cracking by local sliding along the bed joint and opening of head joints, and diagonal cracking associated with cracks running through the bricks as well as the joints.
- Flexural failure: As the horizontal force and displacement demands increase, the wall behaves as a vertical cantilever, bed joints crack in tension and shear is carried by the compressed masonry. Failure occurs when crushing at the compressed corner and/or cracking in the masonry tension zone limits the wall's bearing capacity. Sometimes flexural failure is distinguished from rocking or toe crushing [2], though in practice there is much overlap and many publications do not make any distinction.

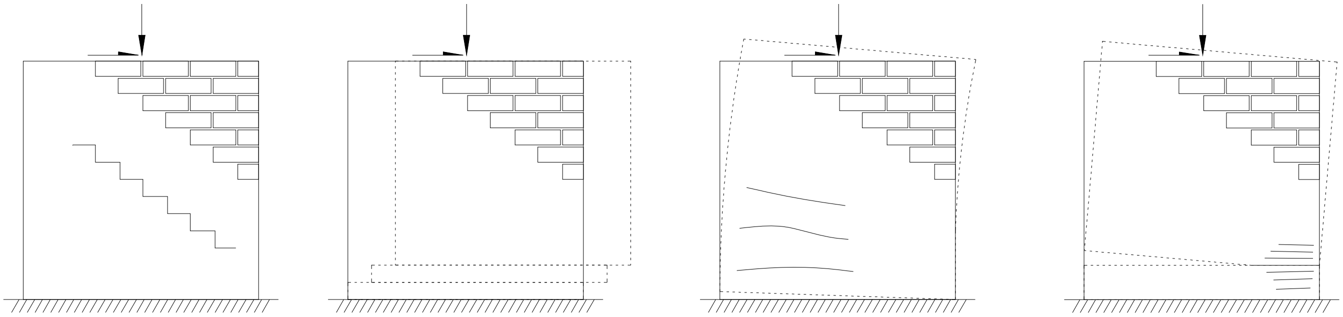

Figure 2: In-plane behaviour modes of URM walls subjected to lateral force: (a) Diagonal shear; (b) Sliding shear; (c) Flexure; (d) Rocking/Toe crushing

A number of publications offer equations for calculating the limiting strength of URM bearing walls [2, 3, 9], but the 2006 International Existing Building Code (IEBC) [10] provides a simplified approach, which is appropriate for assessing the capacity of low-rise New Zealand URM structures. IEBC considers only two behaviour modes for URM walls, a rocking controlled (flexure) mode or a shear controlled mode, with no distinction made between sliding shear and diagonal tension failure. For shear walls with openings, the allowable shear capacity of each pier, V_a, can be calculated according to Equation 10, and the allowable pier rocking capacity, V_r, can be calculated according to Equation 11.

$$V_a = \frac{v_m A}{1.5} \tag{10}$$

$$V_r = \frac{0.9 P_D D}{H} \tag{11}$$

For walls without openings, the allowable rocking capacity can be calculated according to Equation 12, and the allowable shear capacity according to Equation 10, as for walls with openings.

$$V_r = \frac{0.9\left(P_D + 0.5 P_W\right) D}{H} \tag{12}$$

where, v_m = shear strength of URM,

A = cross-sectional area of URM pier or wall,

P_D = superimposed dead load at the location under consideration,

P_W = weight of wall,

D = in-plane width dimension of pier or wall,

H = least clear height of opening on either side of a pier.

The allowable shear capacity of the wall, V_a, is determined using the shear strength of the URM material in the wall, v_m, which is calculated from either direct measurements of mortar shear strength or tensile-splitting strength of the masonry. While there is no substitute for direct material testing in a structure to determine its capacity, IEBC [10] provides some maximum values for v_m based on estimates of f'_m, the compressive strength of the masonry. When the allowable rocking capacity is less than the allowable shear capacity, the wall is considered rocking controlled, and the applied shear force in the wall, V_i, is distributed to each pier, as V_p, in proportion to $P_D D/H$. If $\Sigma V_r < 0.7 V_i$, the wall can be considered able to rock safely, and if not, the wall should be retrofitted. This is applicable also for walls without openings, except V_r is calculated as per Equation 12. When the allowable shear capacity is less than the allowable rocking capacity (in at least one pier in a level), the wall is considered shear controlled, and the applied shear force in the wall, V_i, is distributed to each pier, as V_p, in proportion to D/H. If the applied shear in a wall without openings, V_i, or the shear force in any pier, V_p, exceeds the allowable shear capacity, V_a, the wall is overstressed in shear and should be retrofitted.

CONCLUSIONS

An equivalent static analysis method is appropriate for assessing the existing lateral force capacity of low-rise New Zealand URM structures. NZS.1170.5: (Commentary) states, "There are limits on the applicability of the equivalent static method, but it is likely to remain the most commonly used of the three permitted methods as most buildings are less than three storeys in height." Because of the familiarity among engineers with using an ESA method for assessment, the procedure presented in this paper is considered an appropriate starting point for assessing the performance of URM structures in New Zealand. A force-based, elastic method considering a critical event may involve some conservatism and may be less accurate than more detailed methods, but attention is drawn to the fact that the objective of the legislation is to reduce seismic risk. NZSEE [2] notes that "it may be better that a fairly crude but effective strengthening measure be carried out than for strengthening work to be postponed while the owner saves up to pay for an unnecessarily expensive analysis." The procedure presented is considered the most appropriate method of analysis for the majority of the New Zealand URM building stock.

ACKNOWLEDGEMENTS

The Foundation for Research Science and Technology New Zealand, FRST, provided funding for the research presented here and this is gratefully acknowledged. The suggestions of Stuart Oliver of Holmes Consulting Group are also gratefully acknowledged.

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