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EARTHQUAKE RESISTANCE OF UNREINFORCED MASONRY BUILDINGS

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SUMMARY

Unreinfored masonry buildings (URM) present a severe hazard in earthquake prone regions. However, for commerical and historical preservation reasons many of these buildings must be retained. These buildings require structural engineering assessment and strengthening to avoid life safety hazard and minimize property damage. For historic buildings, any strengthening must be as unobtrusive as possible to avoid damaging the architectural fabric.

This paper presents a methodology which provides an accurate assessment of the seismic strength of existing URM buildings and which ensures that any added strengthening elements were compatible with the original structural system.

The methodology is based on the ANSR-II nonlinear analysis computer program. Face loaded walls are represented by a rocking block formulation with crack opening and geometric stability effects included. In-plane load resistance is modelled by either shear panels with degrading strength and stiffness and very limited ductility or a rocking pier formulation, depending on the configuration. Explicit modelling is used for other components such as flexible diaphragms, added concrete walls or strengthening trusses.

KEYWORDS

Unreinforced masonry; nonlinear analysis; degrading strength; geometric stability; cracking

INTRODUCTION

For most unreinforced masonry buildings, features such as the lack or diaphragms, irregular layout of the resisting walls and uncertain material properties make assessment of the building strength difficult. For this reason, many strengthening designs for historic buildings have assumed that the existing building has no strength and provide a new lateral load resisting system to support all the earthquake loads. This has two main disadvantages:

1. Ignoring the building strength makes the new system more intrusive than it needs to be.

2. The new system may not be compatible with the old system. For example, ductile systems may deform 25 mm or more per storey under full earthquake load. If unreinforced brick has to deform this far it will be badly damaged and may lose its ability to support the gravity loads.

Guidelines developed in New Zealand [Reference 1] provide some assistance for evaluating strength but the allowable stresses are largely empirical and detailed analysis procedures are not provided.

This paper discusses the development of computer modelling techniques to allow the strength of the existing building to be assessed more accurately than current procedures permit.

RESPONSE CHARACTERISTICS OF UNREINFORCED MASONRY

Masonry strength and stiffness depend on the properties of the blocks and mortar and also the interaction between the two. Modelling at the "micro" level, incorporating each component into a computer model, has not been very successful and the problem is even more complex when discontinuities are included, for example, after cracking.

To produce a procedure capable of modelling complete buildings, the methodology developed here models the structure at the "macro" level where complete components are modelled as a single element described by its overall force deformation function. For example, a complete wall segment between windows is modelled as a single 4 node element with a shear force - shear strain relationship based on experimental tests on complete walls.

The response characteristics of each segment are determined before the model is formulated so that elements with appropriate characteristics can be used to represent it. These are characterised according to the load amplitude, load orientation and aspect ratio of the element into a number of mechanisms:

Face Loading

Under face loading an unreinforced masonry wall will respond elastically until tensile stresses due to flexure exceed the tensile strength of the masonry plus the compression due to gravity loads. At this level a crack will form at the location of maximum moment, generally about mid-height. After cracking, the response mechanism will depend on the boundary conditions:

1. For infill masonry walls, a shallow arching action will develop. Wall collapse will occur due to either compression failure or from geometric instability ("snapthrough" of the arch). [Reference 2].

2. For load bearing walls the portions of wall above and below the crack will act as rocking blocks and the wall collapse will be due to geometric instability once the central displacements exceed the stability limit. [Reference 3].

For rocking or arching walls, the moment capacity of the section increases with increasing deflection, but the vertical load acting on the wall provides a negative P-Δ moment which is linear with deflection. The net moment capacity of the wall is the sum of these two effects. As the slope of the moment capacity reduces below the P-Δ slope the net stiffness become negative. Complete wall instability will eventually occur at a displacement less than the wall thickness.

Figure 1 compares the capacity of an arching wall with a rocking wall at three levels of a 5 storey building. As the axial load on the rocking walls increases the moment capacity tends towards the arching case. The curves do not converge completely because the rocking mechanism is based on a pinned end column analogy whereas the arching wall has moment fixity at the top and bottom.

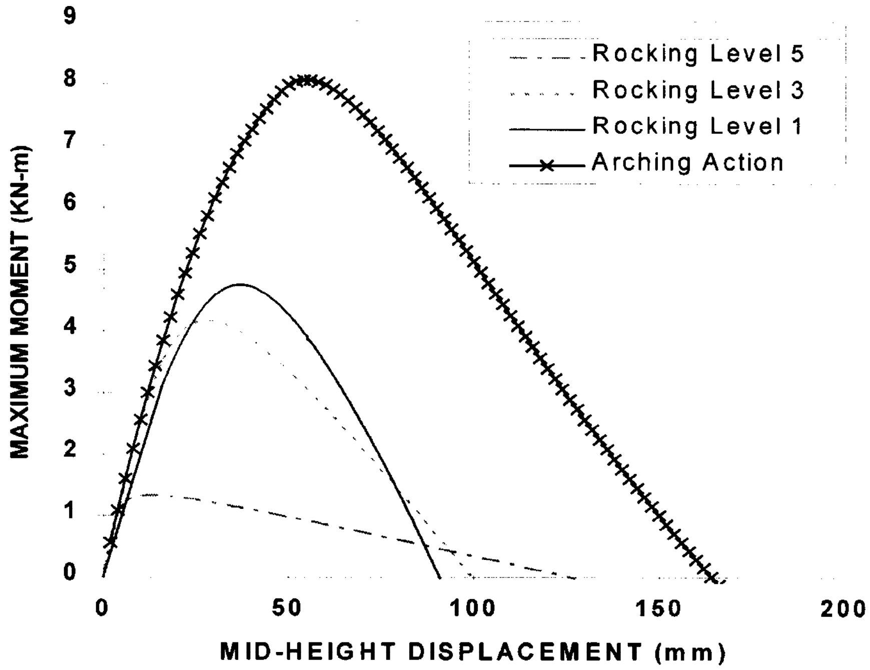

Fig. 1. Theoretical Capacity of Face Loaded Walls

<u>In-Plane Loads</u>

Under in-plane loading shear and bending stresses will develop in the walls. As the load amplitude increases cracking will occur in either a flexural or shear mode depending on the aspect ratio of the wall:

1. If the wall segment is slender, flexural cracks will occur and the wall will rock. The shear force resisted by the wall will remain constant at the value required to initiate cracking and collapse will be through geometric instability.

2. For squat segments the masonry will crack due to shear stresses. Cracking will be in the form of diagonal cracks. Once cracking occurs the strength and stiffness will reduce rapidly under cyclic loading. As the cracks widen the wall will lose the ability to support vertical loads and collapse will occur due to material instability. [References 4,5,6].

References [5 & 6] report the results of static cyclic tests on brick panels. Reference [4] reports tests on a number of brick piers under simulated earthquake motions on a shake table. The earthquake motions were gradually increased and the shear modulus and damping computed from the measured response. Figure 2 shows the envelope curve of shear stress versus shear strain measured from these tests.

1. The shear stress is approximately linear with strain for shear strains up to about 0.001 and then increases at a slower rate until brittle failure occurs at shear strains of between 0.0025 and 0.005.

2. The dynamic shear modulus is much lower than would be expected from prism test results.

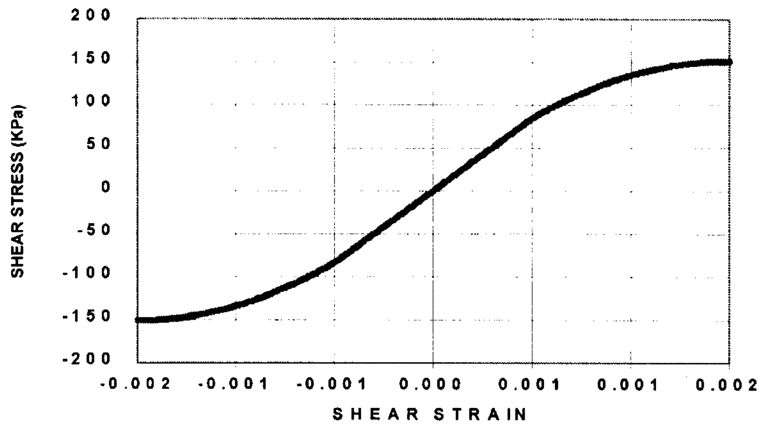

Fig. 2. EERC Tests : Measured Shear Modulus

COMPUTER MODELLING

A number of previous studies have examined methods of modelling aspects of unreinforced masonry response. (Reference [7]). These previous studies have been used to develop a comprehensive analysis procedure based on the ANSR-II general purpose computer program [Reference 8].

<u>Face Loads</u>

For face loads, the wall is modelled as two components (1) elastic portions of wall where cracking is assumed not to occur and (2) cracking regions. The crack is modelled as gap elements located at each side of the wall. Rigid links are used to provide the correct wall width. This is a similar approach to that used in Reference [7].

Cracks are generally assumed to occur at mid-height and the base for rocking walls. For arching action models the top support is confined and a crack model is used at this location also. The gap elements in the model are initially in compression due to the self weight of the wall and any imposed gravity loads. As lateral loads are applied the gaps open when the tension due to flexure overcomes the compressive load. The geometric stiffness option of ANSR-II is used to model the P-Δ effects.

Figure 3 compares the moment-displacement function from an ANSR-II model with the theoretical curve. The model correctly models the initial stiffness and also the negative geometric stiffness. However, it does this as a bi-linear function and does not model the transition curve. This leads to an overestimate of the moment capacities for deflections exceeding about 25 mm in this example.

A structural element with a force displacement function as shown in Figure 3 is in unstable equilibrium once displacements exceed the point of maximum moment. Wall survival past this point is reliant on inertia force acting in a direction to restore the wall. This characteristic of rocking blocks makes it very difficult to predict the load which will cause failure. In these conditions a deterministic formulation of wall capacity is difficult to achieve and so probabilistic techniques are used.

In the application of this methodology, this is accounted for in two ways:

1. Using multiple input earthquake motions to represent a given event.

2. Applying a factor of safety of at least 2 to the response, that is, ensuring survival at levels of input two times the design level.

The model is relatively simple and multiple analyses do not require excessive resources. A future aim is to rationalise the evaluation process by analysing a full matrix of wall dimensions, overburden ratios and input motions to obtain sufficient results to formulate a probabilistic wall capacity.

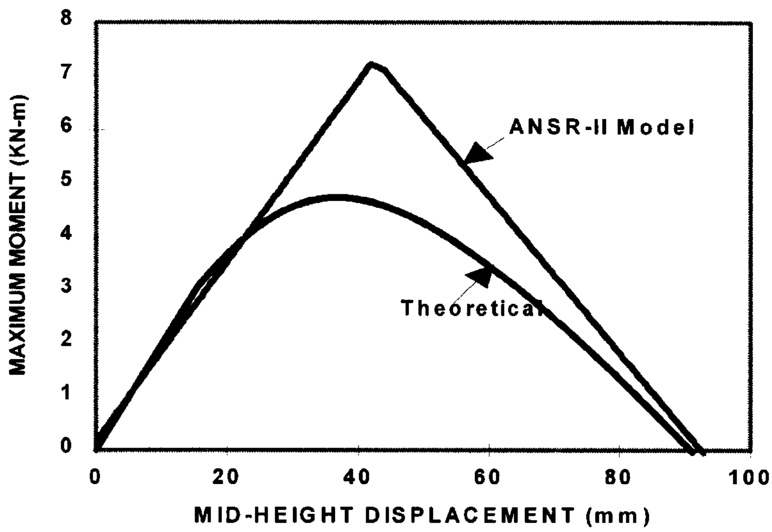

Fig. 3. Moment Capacity of Rocking Walls

In-Plane Loads

The computer model for in-plane response is based on a 4 node plane stress finite element. The element is nonlinear in shear with both degrading strength and degrading stiffness with the stiffness at any time a function of the maximum shear strain attained in previous cycles.

The stiffness is modelled in piecewise fashion as a series of straight lines, defined by shear stress and shear strain, which define the envelope curve. Based on the results from Reference [4] the wall is modelled to reach a peak shear stress at a shear strain of 0.0015 and complete failure at 1.5 times this value, 0.00225. Once the failure strain has been reached the stiffness and strength reduce to zero for the remainder of the analysis. If the load cannot be redistributed to other elements then failure will occur at this level.

The model was validated by developing a computer model of the brick walls tested in [4]. For the tests, the El Centro 1940 earthquake input was used and the amplitude increased incrementally. For each test the maximum wall top displacement and shear stress was recorded. The same process was followed using an ANSR-II model of the wall, represented as a single element.

Figure 4 shows the maximum wall displacements from the analyses compared to the test displacements for increasing earthquake amplitude. For earthquake motions less than 2 times El Centro the model provides a good prediction of displacement, although generally about 10% higher than the test values. The analysis showed failure at a factor of 2.07 on El Centro whereas the test specimen survived 2.27 El Centro.