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Applicability of nonlinear static procedures for low-rise unreinforced masonry buildings with flexible diaphragms

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ABSTRACT

The applicability of the N2 method, the modal pushover analysis (MPA) and an adaptive pushover analysis method are investigated for estimating the peak seismic responses of unreinforced masonry buildings with flexible diaphragms. The performance of nonlinear static procedures are compared against the nonlinear time-history analyses of three low-rise building models with various levels of stiffness eccentricity, type of failure mechanisms of piers (rocking or shear), and a range of diaphragm stiffness representing timber floor and roof systems. The results indicate that the MPA is unsuitable for unreinforced masonry buildings with flexible diaphragms, if the building response is shear-dominated. The adaptive method provides the most accurate estimates when the diaphragms are relatively stiff. When the diaphragms are relatively flexible, none of these methods can provide accurate predictions of peak seismic demands. However, conservative results may be obtained with the N2 method, by taking the envelope of pushover analyses carried out using the force distributions proportional to the uniform and linear displacement shapes along the height of the building. The present study has also identified the most suitable parameters/methods for the use of various nonlinear static procedures, such as the location of the control node used in the N2 method and the modal combination rule used in the MPA, for unreinforced masonry buildings with flexible diaphragms.

1. Introduction

An essential component of the performance-based seismic assessment of a building is the prediction of the peak inelastic seismic response of the building subjected to a predetermined level of earthquake shaking. For the prediction of seismic response, it is well recognised that the linear elastic analysis used in traditional force-based assessment is inadequate in capturing the redistribution of internal forces, as well as the distribution of damage, after the onset of inelastic deformation [1]. On the other hand, the rigorous nonlinear time-history analysis (NTHA) of a multi degree of freedom (MDOF) building model remains unfeasible for typical design/assessment tasks, due to the requirement for developing complex numerical models, appropriate selection of ground motions and the large computational efforts.

In order to address these issues, nonlinear static procedures (NSPs) based on pushover analysis have been developed in the past decades, with the aim of attaining a balance between the accuracy of analysis and the suitability for practical use.

The simplest form of NSPs (e.g. the N2 method [2,3]) considers the building to respond in an invariant displacement shape (or in a single mode) throughout the entire range of excitation. By considering a single-mode response, the dynamic response of the MDOF structure can be reduced to that of an equivalent single degree of freedom (SDOF) system. This simpler equivalent SDOF is used to estimate the peak inelastic displacement (target displacement) at a selected location, which is the control node of the MDOF system. The pushover analysis at the computed target displacement is considered to approximate the peak inelastic seismic response of the building.

A limitation of the single-mode pushover analysis method is that it cannot capture the responses of buildings containing multiple dominant modes. This limitation has led some researchers to propose multi-mode NSP, an example of which is the Modal Pushover Analysis (MPA) [4]. In the MPA, the multi-mode effects are accounted for in an approximate manner by conducting separate pushover analyses for each significant elastic mode and the results of such "modal" pushover analyses are combined to obtain the

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total peak dynamic response. While the MPA considers the contribution of multiple modes, the elastic mode shapes are considered to remain unmodified throughout the excitation, even if an inelastic damage alters the dynamic property of the building.

The adaptive pushover methods were developed with the aim of capturing the changing characteristics of the structure as it enters into the inelastic range. Various proposals have been made for modifying the pushover forces based on the instantaneous damage state of the structure [5–8]. An adaptive pushover procedure has also been developed for unreinforced masonry (URM) buildings [9], with the primary aim of making the analysis independent of the location of the control node. According to the procedure developed by Galasco et al. [9], the pushover force distribution at the i^{th} step ($\mathbf{p}_i$) is constructed using the computed displacement shape of the previous analysis step ($\boldsymbol{\psi}_{i-1}$),

$$\mathbf{p}_i = \mathbf{m}\boldsymbol{\psi}_{i-1} \tag{1}$$

where $\mathbf{m}$ is the mass matrix of the structure.

It was observed during the initial application of the adaptive method that constraints were needed on the force distribution, $\mathbf{p}_i$, in order to obtain realistic responses for buildings with flexible diaphragms [9]. While promising results are reported, this adaptive pushover procedure has not been investigated in the context of a NSP.

The seismic behaviour of URM buildings with flexible diaphragms have been investigated in the past, both experimentally and analytically [10–14], which highlighted the presence of multiple dominant modes and weak coupling of in-plane loaded walls. However, past studies have not addressed the applicability of NSPs for the analysis of URM buildings with flexible diaphragms in detail. In particular, two broad issues require further studies:

- The first issue is the uncertainties in the selection of analysis parameters when using the NSPs. These uncertainties include the suitable hysteresis rule to be used in calculating the target displacement, the suitable location of the control node, the modal combination method used in MPA, and the procedure to convert the pushover curve to an equivalent SDOF system definition for the adaptive method, considering the continuously changing lateral force distribution pattern. Some of these uncertainties arise because the NSPs were originally developed for buildings with rigid diaphragms. For example, while the location of the control node can logically be placed at the centre of mass of the roof for buildings if the diaphragms are rigid, the most suitable location is not immediately apparent when the diaphragms are flexible. Other issues concern the nonlinear static analysis of URM buildings more generally. For example, while the use of equivalent SDOF systems with idealised hysteresis models for estimating the target displacement have been studied for RC or steel frame buildings [15,16], specific studies for URM buildings have been limited, with a notable exception [17]. If a suitable idealised hysteresis rule can be identified, inelastic displacement ratios derived on the basis of extensive statistical studies for modern construction systems [18–20] can be adopted also for URM buildings.
- The second issue requiring further studies is the identification of the applicable ranges of the NSPs. While intuition suggests that more advanced methods (e.g. MPA and the adaptive NSP) are able to provide better estimates of seismic responses than the single-mode N2 method for a wider range of diaphragm stiffness values, systematic evaluations have not been undertaken to verify the accuracies of various methods. Furthermore, no studies have been conducted to identify the factors affecting the accuracies of NSPs for URM buildings with flexible diaphragms.

The study reported herein aims to address the above issues associated with the NSPs for URM buildings with flexible diaphragms. The applicability of the N2 method, the MPA and an adaptive NSP utilising the pushover algorithm of Galasco et al. [9] are investigated for URM buildings with flexible diaphragms, using three building models with different levels of stiffness eccentricity and predominant failure mechanisms (by rocking or shear). A wide range of diaphragm stiffnesses representative of timber floor systems are considered. The geometrical and engineering properties of these buildings and the earthquake loading scenarios are reported in Section 2. The most suitable analysis parameters for each NSP are identified in Section 3, by comparing the estimated control node displacements with the "exact" results obtained from the NTHA of the MDOF model. Finally, utilising the best analysis parameters for each NSP identified in Section 3, the relative accuracies of the NSPs are investigated in Section 4, followed by concluding remarks in Section 5.

2. Descriptions of buildings, numerical models and ground motions

2.1. Description of building models

Three building models, broadly representing low-rise isolated URM buildings commonly found in New Zealand and Australia [21,22] were analysed. The models differed in the number of storeys (2 or 3), number of bays (1 or 2), stiffness eccentricity and the predominant failure mechanisms. Table 1 summarises the key properties with Fig. 1 showing the plan views of the buildings and the elevations of the in-plane loaded walls. The stiffness eccentricities were calculated for the first floor, considering the piers of the ground storey to be fixed at top and bottom (i.e. rigid spandrels). The densities of masonry and timber materials were 1800 kg/m^3 and 660 kg/m^3 respectively. Uniform floor live load of 4 kPa was taken, with the seismic load factor of 0.3 in accordance with AS 1170.1 [23]. The analyses were conducted under unidirectional loading, applied in the X directions of the models.

The floor and roof diaphragms were representative of flexible (or non-rigid) timber systems with identical constructions at all floor levels. Six different configurations of these systems were considered for each model. They corresponded to single straight sheathing (D1), single diagonal sheathing (D2), double straight sheathing (D3), double layered panels (D4), and two additional levels representing hypothetical retrofits (D5 and D6). The

Table 1
Structural properties of the building models.

Building	Number of storey	Number of bays	T_{rig}[a] (s)	Normalised stiffness eccentricity[b]	Dominant failure mechanism[c]
Model 1	2	1	0.231	0.36	Rocking
Model 2	2	1	0.151	0.1	Shear
Model 3	3	2	0.253	0.2	Shear

[a] Fundamental period of the building with rigid diaphragms.
[b] Normalised eccentricity in the direction of excitation.
[c] The shear failure was considered to be dominant if more than half the piers of the critical storey failed in shear.