

SEISMIC ASSESSMENT OF NON-DUCTILE MULTI-STOREY BUILDINGS

TUAN NGO, NELSON LAM, JOHN WILSON AND PRIYAN MENDIS
UNIVERSITY OF MELBOURNE

AUTHORS:

Tuan Ngo is a PhD student at the University of Melbourne. He got his Master degree in Civil Engineering at the same university in 2000. He worked in Vietnam for 3 years as structural engineer. He is currently investigating the seismic behaviour of limited-ductile multi-storey buildings.

Nelson Lam is Senior Lecturer at The University of Melbourne. He joined Scott Wilson Kirkpatrick & Partners in 1982 and was bridge engineer with the firm until 1989 when he began his academic career at The University of Melbourne specialising in the field of earthquake engineering.

John Wilson is Senior Lecturer at the University of Melbourne. He was with Ove Arup and Partners before becoming Senior Lecturer at The University of Melbourne in 1992. He is co-author of a book and numerous publications in many different areas of earthquake engineering and structural dynamics.

Priyan Mendis is Associate Professor and Reader in Civil Engineering at the University of Melbourne. He is author of a book and more than one hundred papers on High-strength /High-performance Concrete, Seismic Design of Reinforced and Prestressed Concrete Structures and Design of Tall Buildings.

ABSTRACT:

The current Australian Earthquake Loading Standard allows the use of the Equivalent Static Load procedure or the Dynamic Analysis procedure, to assess the seismic performance of structural systems. While these conventional force-based (FB) procedures are reasonable design tools, there are difficulties in applying them to assess existing buildings. More rational displacement-based (DB) assessment procedures such as the Capacity Spectrum Method have been developed recently in the United States to provide the seismic assessment for a range of structures, particularly structures which develop well defined collapse mechanisms and possess reasonable amount of ductility. However, such static procedures may not be reliable and effective in assessing limited-ductile buildings in which the potential failure mechanism is uncertain.

This paper introduces an alternative DB procedure which can effectively identify the likely potential failure mechanism in existing limited-ductile multi-storey buildings. The procedure estimates the building dynamic deflection profile which is then translated into curvature and flexural strain demand, based on a bi-linear displacement spectrum. The formation of plastic hinge is predicted at the location where the strain demand exceeds the material yield strain. Contributions by the fundamental and the higher modes of vibration are accounted for. The procedure is both simple and accurate enough to be carried out in normal practice, as is demonstrated in the paper using three multi-storey buildings (12, 15 and 20-storeys) as examples.

1. INTRODUCTION

In the conventional equivalent static force procedure specified by design codes of practice, the design base shear for the building is first determined in accordance with the acceleration response spectrum. Thus, the procedure begins with the determination of the building's natural period (T), which is normally estimated by some simple rule-of-the-thumb formulae. The estimated T is then used to define the response spectral acceleration (RSA) and the seismic inertia force, that is typically distributed triangularly up the building's height. The estimated seismic force is then used to analyse the internal forces, which includes the bending moment and shear forces, using a suitable analytical model for the structure. The condition of yielding is identified from comparison of the moment demand with the moment capacity at yield.

Whilst such a procedure has been used in building design codes, there are recognized uncertainties in the estimates of T . It should be noted that the design spectral acceleration (and hence the base shear) is actually proportional to $1/T^2$ in the region of constant displacement demand when the building period exceeds a dependable limit. For rock sites, constant displacement demand occurs when the building period exceeds between 0.5 and 1.5secs depending on the earthquake magnitude (Koo, 2000; Lam, 2001b). For soil sites prone to resonance, the limit of constant displacement is simply the site natural period which is approximately 1sec for a 50m deep sediment possessing an average shear wave velocity of 200m/sec (Lam 2000, 2001a). Whilst displacement demand is insensitive to the building period in this constant displacement region, the spectral acceleration (and hence seismic force) is ultra sensitive to the building period. In this period sensitive range of the acceleration spectrum, uncertainties in the estimation of T can be translated into significant errors in the estimated seismic forces. In such situations, the force based (FB) approach is not entirely satisfactory particularly when making retrofit decisions, since conservatism in the FB assessment can lead to significant economical and social implications.

The alternative displacement based (DB) approach appears to eliminate the problems associated with the natural period uncertainties since the displacement spectrum is approximately constant (i.e. period independent) in the high period range. Thus, the seismic response in high-rise buildings can be conveniently predicted by the displacement spectrum, whilst the response in medium-rise buildings can be conservatively predicted if the peak displacement demand defined for the flat part of the spectrum is used. However, recent studies using different approaches indicated that this peak displacement demand based on very onerous site conditions in Australia was only in the order of 100mm-200mm. Consequently, the DB method based on the peak displacement limit (constant region of RSD) may be used as a rapid assessment tool for both high-rise and medium-rise buildings.

Currently developed DB assessment procedures (Priestley, 1995 & 2000; ATC-40) are typically associated with an inelastic quasi-static analysis (also known as "push-over analysis") in which the applied load is consistent with the observed static displacement profile. Whilst these DB procedures are gaining wide acceptance, it is also recognised that the static analysis cannot adequately cater for dynamic (or higher mode) effects. Limited-ductile buildings, in particular, can develop unexpected mechanism which cannot always be identified by the push-over analysis procedure. Thus, further development of the DB method is required for adaptations to low and moderate seismic regions like Australia (where buildings are generally limited-ductile and the

likely collapse mechanism is not considered in design). It is proposed that buildings must first be assessed to identify the likely locations of plastic hinge formation based on analysis which accounts for the higher mode effects.

Higher-mode (dynamic) effects have been accounted for in the conventional force-based (FB) procedure in various ways including artificially augmenting the response spectrum in the high period range, introducing a fictitious point force at the roof level and altering the static load profile. Whilst these empirically based provisions are already part of the day-to-day design process, their effectiveness as an assessment procedure is questionable.

This paper describes a new DB procedure which forms part of the long term research objective to address non-ductile behaviour in buildings. The proposed procedure predicts the dynamic deflection profile which is then translated directly into curvature demand and hence flexural strain demand at the critical cross-section of a structural wall. Thus, the condition for yielding can be detected without necessarily involving bending moment and flexural stiffness calculations. Clearly, the procedure is much more direct and versatile than existing FB and DB procedures. Details are described in Section 2.

The approach of predicting the curvature from a discretised dynamic deflection profile has not been tested, until now. In theory, the accuracy could be highly sensitive to the discretisation of the profile. To address this, a comparative study was carried out to obtain the curvature demands using different methods. The outcome from the study is very promising as the estimated curvature is shown to be reasonably accurate (refer Section 3).

Further investigations are currently being undertaken by the authors to study the curvature characteristics of a wide range of buildings. The objective is to develop a simplified model based on a generalised dynamic deflection profile to predict the member curvature and strain demand in a structural wall.

2. CONCEPT OF DISPLACEMENT BASED PROCEDURE

The curvature (ϕ_i) of a vertical element can be obtained from the discretised deflection profile (Fig. 1) using the following expression:

$$\phi_i = \frac{d^2 y}{dx^2} \cong \frac{1}{h^2} (y_{i+1} - 2y_i + y_{i-1}) = \frac{n^2}{H^2} (y_{i+1} - 2y_i + y_{i-1}) \quad (1)$$

where H is the building height.

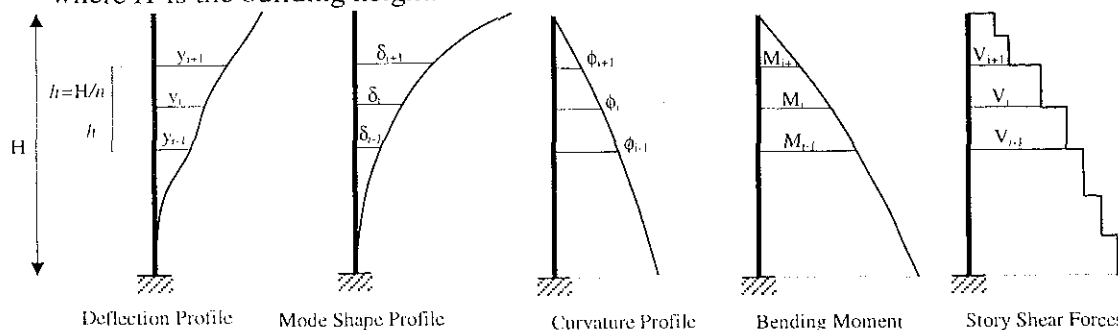


Figure 1. Relationship between the storey drift, modal displacement, curvature and bending moment at level "i", contributed by mode of vibration "j".

Thus, the curvature ($\phi_{i,j}$) at level "i" due to mode of vibration "j" is defined by Eq.2a-2d:

$$\phi_{i,j} = PF_j \left(\frac{(\delta_{i+1} - 2\delta_i + \delta_{i-1})_j}{H^2/n^2} \right) \times RSD_j = \Gamma_{i,j} \times RSD_j \quad (2a)$$

where: PF_j - modal participation factor. $PF_j = \frac{\{\delta\}_j^T \{M\} \{I\}}{\{\delta\}_j^T \{M\} \{\delta\}} = \frac{\sum_{i=1}^n m_i \delta_i}{\sum_{i=1}^n m_i \delta_i^2}$

RSD_j - modal response spectral displacement, and

$$\Gamma_{i,j} = PF_j \left(\frac{(\delta_{i+1} - 2\delta_i + \delta_{i-1})_j}{H^2/n^2} \right) = \left(\frac{\sum_{i=1}^n m_i \delta_i}{\sum_{i=1}^n m_i \delta_i^2} \right) \times \left(\frac{(\delta_{i+1} - 2\delta_i + \delta_{i-1})_j}{H^2/n^2} \right) \quad (2b)$$

The resultant curvature ϕ_i at story level "i" can be estimated by combining the contributions of the first three modes of vibration. The combined curvature can be calculated by the "square-root-of-the-sum-of-the-squares" (SRSS) method and the "absolute summation" (ABS) methods as represented by Eqs.2c & 2d, respectively.

$$\phi_i = \sqrt{\phi_{i,1}^2 + \phi_{i,2}^2 + \phi_{i,3}^2} = RSD_1 \sqrt{\Gamma_{i,1}^2 + \Gamma_{i,2}^2 \left(\frac{RSD_2}{RSD_1} \right)^2 + \Gamma_{i,3}^2 \left(\frac{RSD_3}{RSD_1} \right)^2} \quad (2c)$$

or $\phi_i = |\phi_{i,1}| + |\phi_{i,2}| + |\phi_{i,3}| = RSD_1 \left(|\Gamma_{i,1}| + |\Gamma_{i,2}| \times \frac{RSD_2}{RSD_1} + |\Gamma_{i,3}| \times \frac{RSD_3}{RSD_1} \right) \quad (2d)$

The flexural strain at the critical cross-section can be calculated using the following relationship:

$$\epsilon_j = \frac{\phi_j \times l_w}{c} \quad (3a)$$

The yield curvature ϕ_y is hence expressed as:

$$\phi_y = \frac{\epsilon_y c}{l_w} \quad (3b)$$

where: - l_w = section depth

- c = depth of the neutral axis which is insensitive to the reinforcement content and axial load level, and is typically in the order of 2 for structural walls (Priestley, 1998a).

and - ϵ_y = yield strain of the longitudinal reinforcement.

The DB procedure is summarised in the following steps:

- (i) The modal displacement profiles, $\{\delta\}_j$, are first obtained from modal analyses. Interestingly, the normalised mode shapes for the two buildings supported by cantilever walls have been found to be very similar as shown in Fig. 2 (which shows the measured normalised mode shapes for a 20-storey and a 26-storey building in Singapore as reported by Brownjohn (2000)).

- (ii) The response spectral displacement, RSD_i , are then obtained from the displacement response spectrum (Fig.3).
- (iii) The Γ_{ij} factors are then determined from Eq.2b, and the curvature demand $\{\Phi\}$ from Eqs. 2c and 2d.
- (iv) The wall yield curvature computed using Eq.3b is then compared with the curvature demand to identify the occurrence of yielding, and hence the potential collapse mechanism.

The notable advantage of this DB procedure over the conventional procedure is that yielding can be identified without necessarily involving bending moment and flexural stiffness calculations which can lead to significant errors (Priestley, 1998b).

3. EXAMPLES DEMONSTRATING THE ASSESSMENT PROCEDURE

Three multi-storey buildings (12, 15 and 20 storeys) supported by structural walls (Fig. 4), are used as examples in this section to evaluate the proposed DB procedure. The buildings are each subject to the unilateral earthquake excitation of "El Centro" and a simulated motion for a Magnitude 7 event (Quake1) on a flexible soil site using program GENQKE (Lam, 2001c) and SHAKE (1992) (Fig. 3). The response spectral displacement demands for the fundamental mode of vibration (RSD_1) are listed in Table 1.

Table 1. Computed RSD_1 values for example buildings modelled for this study (Fig.3)

Building \ Ground motion	12 storey	15 storey	20 storey
Elcentro	90 mm	149 mm	274 mm
Quake I	250 mm	204 mm	175 mm

Time-history analyses (THA) of the two accelerograms were carried out (using computer program RUAUMOKO (1998)) to compute the curvature demand for the three building models for comparison with results obtained from the DB procedure (i.e. using Eqs.1-2). Results from the two methods are shown to be in very good agreement (Fig.5). In all cases, the exact results from the THA are bounded in between the DB(ABS) and the DB (SRSS) envelopes, as represented by Eq.2c & 2d respectively.

In the case of the "El Centro" analysis, the maximum curvature is largely controlled by the fundamental (1^{st}) mode of vibration, although there are notable contributions by the higher modes particularly for the taller buildings. Thus, the conventional equivalent static load procedure (which is based primarily on the actions of the 1^{st} mode), or a static push-over analysis procedure, will also provide reasonable predictions. It is further shown in Fig.5 that the higher mode contributions are much more significant with the "Quake 1" analysis in which the effects of soil resonance have been modelled. Clearly, the maximum curvature demand could only be reliably estimated from THA or from the DB procedure. Of particular interest is the higher mode amplification of the curvature demand at the mid-height levels, which may result in plastic hinge forming at "unexpected" locations up the building height. The resulting collapse mechanism would be very different to that predicted by a push-over analysis.

Strictly speaking, the FB would also be capable of predicting the correct curvature and strain demand provided that the flexural stiffness of the building has been accurately represented. The potential advantage of the DB procedure lies in its simplicity and its capability to relate seismic hazard to strain demand much more directly.

4. CONCLUSIONS

A simple DB procedure is presented in this paper to predict the curvature demand in multi storey buildings in order that occurrence of yielding and the potential failure mechanism can be identified. The notable advantage of this DB procedure over the conventional procedure is that yielding can be identified without necessarily involving bending moment and flexural stiffness calculations. There is good agreement between the curvature demand estimated from the procedure and from time-history analyses which account for the higher-mode effects. Further investigation is being undertaken to further develop the procedure for practical applications.

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