SIMPLIFIED METHOD FOR PERFORMANCE-BASED DESING OF NON-STRUCTURAL COMPONENTS

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ABSTRACT

Undesirable seismic performance or failure of non-structural components (NSCs) during earthquakes is reported as a major portion of seismic economic loss. To prevent or reduce earthquake induced damage in NCSs sufficient resistance must be provided. In a performance-based design framework, the required seismic design force is determined according to the predefined performance criteria. Depending on the type of NSC, the performance criteria could be either an acceleration or displacement limit. Hence, a floor horizontal acceleration or displacement response spectra is needed to determine the forces generated in the NSCs. This paper describes a novel analytical approach to derive earthquake-induced floor spectra in a regular building. The influence of the nonlinear behaviour of the primary structure on the floor spectra is included in the proposed simple method to provide an improved estimate. The general approach is based on representing the structure as equivalent single degree of freedom (SDOF) systems considering a first non-linear mode and the other predominantly elastic higher modes. Results obtained from nonlinear time-history analysis are compared with the simplified method proposed. The developed floor response spectra is a start point to develop a simplified performance-based design procedure for NSCs .

KEY WORDS: Non-Structural Components, Performance-Based Design, Nonlinear Behaviour, Floor Spectra

1. INTRODUCTION

Non-structural components (NSCs) are those elements attached to the floor, roof, or wall of a supporting building structure and are not part of the main load bearing structural system. However, NSCs are also subjected to large seismic forces depending on the dynamic characteristics of the supporting structure, attachment systems, and their own inherent structural dynamics. The earthquake response of NSCs has particular importance in industrial and power generation plants or hospitals. In fact, the proper function of such places during and after an earthquake is heavily dependant upon on the performance of the critical NSCs. Moreover, the partial or total failure of a component may result in important environmental consequences or service interruption.

Investigation of seismic induced damage to NSCs during pervious earthquakes reports that failure of NSCs causes a major portion of economic losses [1-4]. It may even account for 65%-85% of the total construction costs of commercial building depending on the purpose of the facility [5]. In that sense, proper design of NSCs should constitute an important component of performance-based earthquake engineering.

Peak floor horizontal acceleration is needed as the main parameter to determine forces generated in NSCs supported on these floors and it was the topic of the research work related to design and assessment of NSCs

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Since primary structures are currently designed to have a nonlinear response for design events, any recommendation for floor acceleration or displacement spectra in equivalent design formula should account for this fact. As pointed out by Rodrigues et al. [6], Medina et al. [8], and Politopoulos and Feau [9], nonlinear behaviour of the supporting structure reduces significantly the floor spectrum peak values, which typically occur in the vicinity of the building's natural frequencies.

The main objective of this paper is to present a simplified and rational method to accurately estimate the floor horizontal acceleration that arises during an earthquake, while considering the nonlinear behaviour of the building. Emphasis is given to the interaction between the building's 1st nonlinear mode of vibration and the floor horizontal accelerations. The results from the proposed method will be evaluated by numerical non-linear time-history analysis and also compared with the current seismic design code approaches.

Achieved floor horizontal acceleration and consequently floor response spectra can be a starting point to generate a simplified performance-based seismic design method for NSCs. The philosophy behind this approach is to ensure that NSCs remain functional and minimum economic loss and loss of service is accomplished. Hence, NSC behaviour will be effectively included in structural design procedures

2. Proposed Method for Determining Floor Horizontal Acceleration

2.1. Floor horizontal acceleration in nonlinear structures

In linear elastic systems, floor horizontal acceleration can be defined using modal analysis since the natural modes of vibration can clearly be defined by ensuring orthogonally between different modes. A proper combination of the dynamic response of the building structure in each mode of vibration leads to the total response.

However, as soon as the structure exceeds the elastic limit, the modes are no longer orthogonal to the stiffness matrix. Recently, Rodriguez et al. [6] proposed a first mode reduced method considering the nonlinearity in just the 1st mode utilizing a reduction factor. The main assumption in their approach is that the modes still provide a set of independent vectors that can conventionally be used in the analysis of non-linear systems and that modal acceleration can still be combined to obtain an approximation to the floor acceleration. Their proposed formula is defined:

$$A_n = \sqrt{\left[\Gamma_1 \phi^1{}_n \frac{S_a(T_1,\xi_1)}{R_1}\right]^2 + \sum_{q=2}^{r}\left[\Gamma_q \phi^q{}_n S_a(T_q,\xi_q)\right]^2} \qquad \text{Eq. 1}$$

where Γ_q is the participation factor for mode q, ϕ_n^q is the amplitude of the mode q at level n, S_a is the spectral acceleration, T_q, ξ_q are the period of free vibration and damping ratio, respectively, associated with mode q, and R_q is a reduction factor to account for the effect of nonlinearity on the system. It can be concluded from Eq. 1 that the effect of nonlinearity is addressed by a strength reduction factor in 1st mode, which is an indication of ductility in the building structure.

The effects of nonlinearity in the first mode can be investigated by a more rational approach. As introduced by Priestley [17], an equivalent linear substitute SDOF system can be used as an approximation of the nonlinear response of a MDOF structure in the first mode. Comprehensive nonlinear time-history analysis showed that despite of the simplicity of the substitute SDOF system, it reflected essential features of nonlinear response of real structures.

Utilizing the same concept applied by Rodriguez et al., combining the 1st nonlinear mode with the other elastic higher modes to represent floor horizontal acceleration, and employing the equivalent SDOF system as the 1st nonlinear mode leads to a more rational approach in generating the floor horizontal acceleration. In the following, this proposed methodology is described in more detail and evaluated by means of nonlinear time-history analysis.

2.2. Determining floor horizontal acceleration using "substitute structure" model

In a general modal analysis, the equation of motion of the nth mode representation of a MDOF building is structure expressed by:

$$\ddot{D}_n(t)+2\xi_n\omega_n\dot{D}_n+\omega_n{}^2D_n=-\ddot{u}_g(t) \qquad \text{Eq. 2}$$

The perception of this equation is that a SDOF, representing the nth mode of vibration, having a mass equal to 1 and natural frequency and damping ratio equal to ω_n and ξ_n respectively, is excited by the ground motion $\ddot{u}_g(t)$. Then resulting contribution of this mode in nodal displacement is defined:

$$u_n(t)=\Gamma_n\Phi_nD_n(t) \qquad \text{Eq. 3}$$

where Φ_n is the mode shape and Γ_n is the contribution factor of the nth mode. To define the total response of the structure afterwards all modal responses should be combined:

$$u(t)=\sum_{n=1}^{N}\Phi_n\Gamma_nD_n(t) \qquad \text{Eq. 4}$$

Reminding the possibility of combining the 1st nonlinear mode response with other elastic modes to represent structural response and assuming the response of the structure at 1st mode to be $u_1(t)$, the Eq. 4 can be conveyed in another way:

$$u(t)=u_1(t)+\sum_{n=2}^{N}\Phi_n\Gamma_nD_n(t) \qquad \text{Eq. 5}$$

The analysis procedure described above can also provide the floor acceleration. The floor acceleration can be computed from:

$$\ddot{u}(t)=\ddot{u}_g(t)+\ddot{u}_1(t)+\sum_{n=2}^{N}\Phi_n\Gamma_n\ddot{D}_n(t) \qquad \text{Eq. 6}$$

To define the structural response in the 1st mode, $u_1(t)$, the substitute structure method will be used. The "substitute structure" model is proposed by Shibata and Sozen [18] and is used as a fundamental concept in the

After defining the parameters of the equivalent linear SDOF system, the equation of motion of the 1st nonlinear mode can be represented again using Eq. 2. The response of the equivalent SDOF system can be simply converted to the response of the prototype building structure by means of a scaling factor. As illustrated in Figure 1, the deformation of the equivalent SDOF system Δ_d corresponds to Δ_i at each floor level. Consequently, the scaling factor to express the relative response of the real structure at floor level i will be Δ_i/Δ_d . Thus, the displacement of the real structure at floor level i is defined:

$$u_1^i(t) = \frac{\Delta_i}{\Delta_d} D_1(t) \qquad \text{Eq. 7}$$

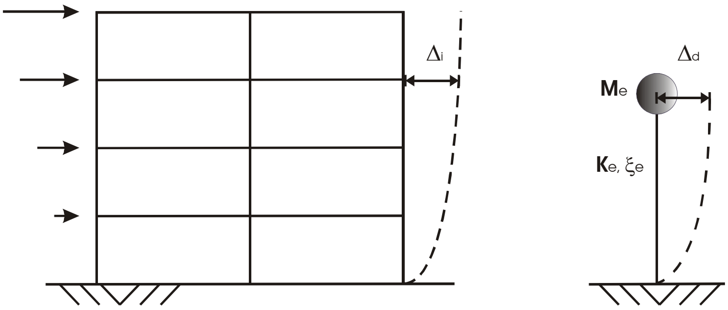


Figure 1 Calculating scaling factor in substitute structure method

To prove the proposed concept, nonlinear time-history analysis for a realistic 5-story building will be presented in the following section.

2.3. Evaluating the Proposed Method in Predicting Floor Horizontal Acceleration

To evaluate the accuracy of the proposed method in predicating floor horizontal acceleration, floor acceleration magnification factor, floor pseudo-acceleration response spectra, and floor acceleration time-history are selected as main representative parameters. Nonlinear time-history analysis is performed on a four-bay five-story moment resisting ductile concrete frame. The resulting responses are compared with the simulated ones from the proposed method.

2.3.1. Frame and records used in this study

The frame analyzed is one of the moment resisting frames in long-direction of a five-story building. The plan view of a typical floor of the buildings is shown in Figure 2. The floor system consists of 200 series precast hollowcore floor units having a 65 mm topping spanning on long direction on every floor. The seismic weight per floor 5180 kN for roof level and 6420 kN for other levels. The story heights are 3.8 m. The frame system is designed according to the New Zealand Concrete Structures Standard [19] using a displacement-based design approach to sustain a target drift level of 2% under a 500 year return period earthquake.

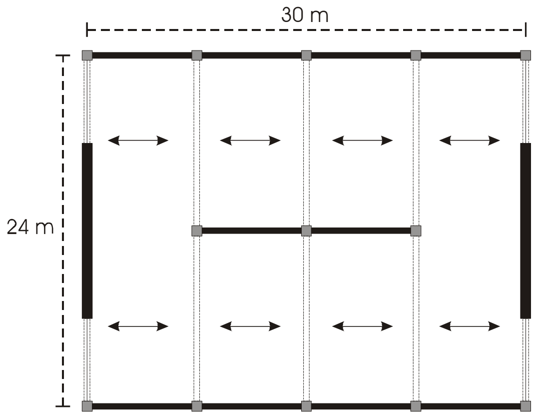


Figure 2 Plan view of the typical story for prototype buildings investigated

Ruaumoko [21] is used for nonlinear time-history dynamic analysis. A Takeda-Type hysteretic model is used for all connections. A standard 5% Raleigh damping formulation proportional to the mass and tangent stiffness matrices is assigned. The first and third mode of vibration is given a 5% damping ratio. Figure 3 shows mode shapes, periods of vibration, damping ratios and participation factors for the first three modes of vibration.

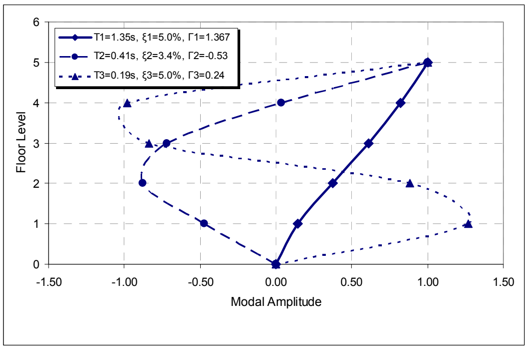


Figure 3 Dynamic characteristics of the 5-story structure with nonlinear first mode

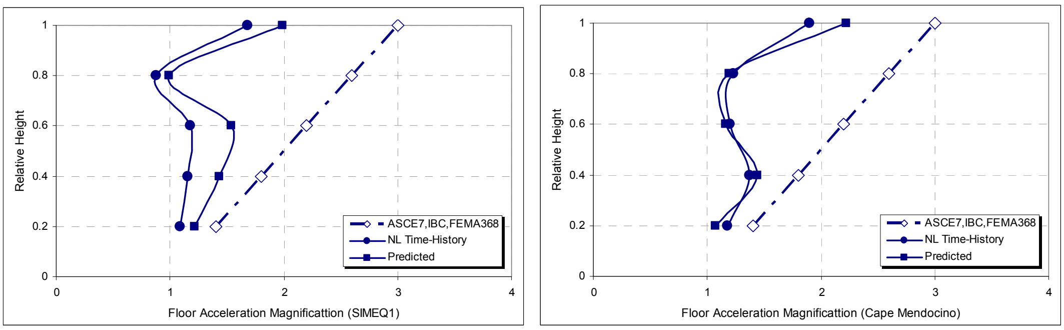
The nonlinear time-history analysis is carried out with two ground motions: (1) a synthetic earthquake ground motion named SIMEQ, and (2) the Cape Mendocino 1992 ground motion recorded on soil type C. SIMEQ is generated to match the target spectra and Cape Mendocino is scaled to match the target spectra, while the target spectra is based on NZS1170.5 [20] for soil type C, annual probability of exceedance of 1/500, and PGA=0.4g..

2.3.2. Analysis Methodology

Nonlinear time-history analysis is used with the 2 aforementioned earthquake events for the prototype frame to generate the benchmark results at each floor level. To determine floor acceleration response utilizing the proposed method, Eq. 6 is used for the first three modes of vibration. In that sense, linear time-history responses of three SDOF systems representing the 1st nonlinear mode (secant stiffness approach), 2nd and 3rd elastic modes are combined together. Based on the resulting acceleration response at each floor level, floor magnification factors and spectra are generated. Since the proposed method deals with just linear elastic SDOF systems, the computational effort involved in time-history analysis of the proposed method is much smaller than what is required to compute the non-linear response of a prototype building frame.

2.3.3. Comparison of floor acceleration magnification factor

Figure 4 presents a comparison of floor acceleration magnification factor simulated for all levels of the prototype structure to those attained from nonlinear time-history analysis and seismic design standards. It can be seen that the floor acceleration magnification factor and consequently floor acceleration demand can be captured by reasonable estimation form the proposed method.



(a) SIMEQ (b) Cape Mendocino

Figure 4 Floor acceleration magnification factor for all levels of the five-story building

The lack of agreement between the actual value and the simulated one in the 2nd and 3rd floor level for SIMEQ seems to be due to presence of nonlinearity in the higher modes of vibration. The frequency content of the simulated earthquake causes the structure to behave nonlinear even in the 2nd and 3rd mode.

Although the proposed simplified method is based on elastic models, it captures the nonlinear response of the structure quite well. The proposed method for both events captures floor acceleration better and more accurately than the highly conservative code approximation.

2.3.4. Comparison of floor pseudo-acceleration response spectra

Acceleration demand in flexible NSCs is usually substantially higher than peak floor acceleration. If the weight of the NSC compared to the weight of the supporting structure is small, the interaction effect between the response of the NSC and the floor can be neglected. As a result, floor spectra can be used to estimate NSC's acceleration demands. However, producing floor spectra ordinates at different floor levels within the structure is more difficult than the estimation of just peak floor acceleration. In particular, results are more sensitive to the nonlinear behaviour of the structure. Using the new proposed method in producing the floor response spectra gives the capability to develop floor response spectra for the expected level of nonlinearity in the structure. In other words, various displacement profiles representing levels of nonlinearity can be utilized to define the 1st nonlinear SDOF system, and consequently different floor response spectra are developed.

Comparisons between the actual and simulated floor pseudo-acceleration response spectra for 1st, 3rd, and 5th floor level are shown in Figure 5. In this figure, dark lines represent the response spectra computed by the use of new method, while the light lines show the spectra obtained by time-history analysis of the prototype frame. It can be seen that the actual and simulated spectra are representing acceptable approximations.

It can be seen that floor spectra can change significantly at different floor levels and from earthquake to earthquake, but the proposed method is able to approximate these changes. The other noticeable point about the floor response spectra, as it is presented in other research work [11,12,6], is that the first mode is amplified more than the other predominant modes along the height. The strong capability of the proposed method, as demonstrated in Figure 5, is that it is able to capture relatively well the peak point of the floor spectra at predominant periods of vibration.

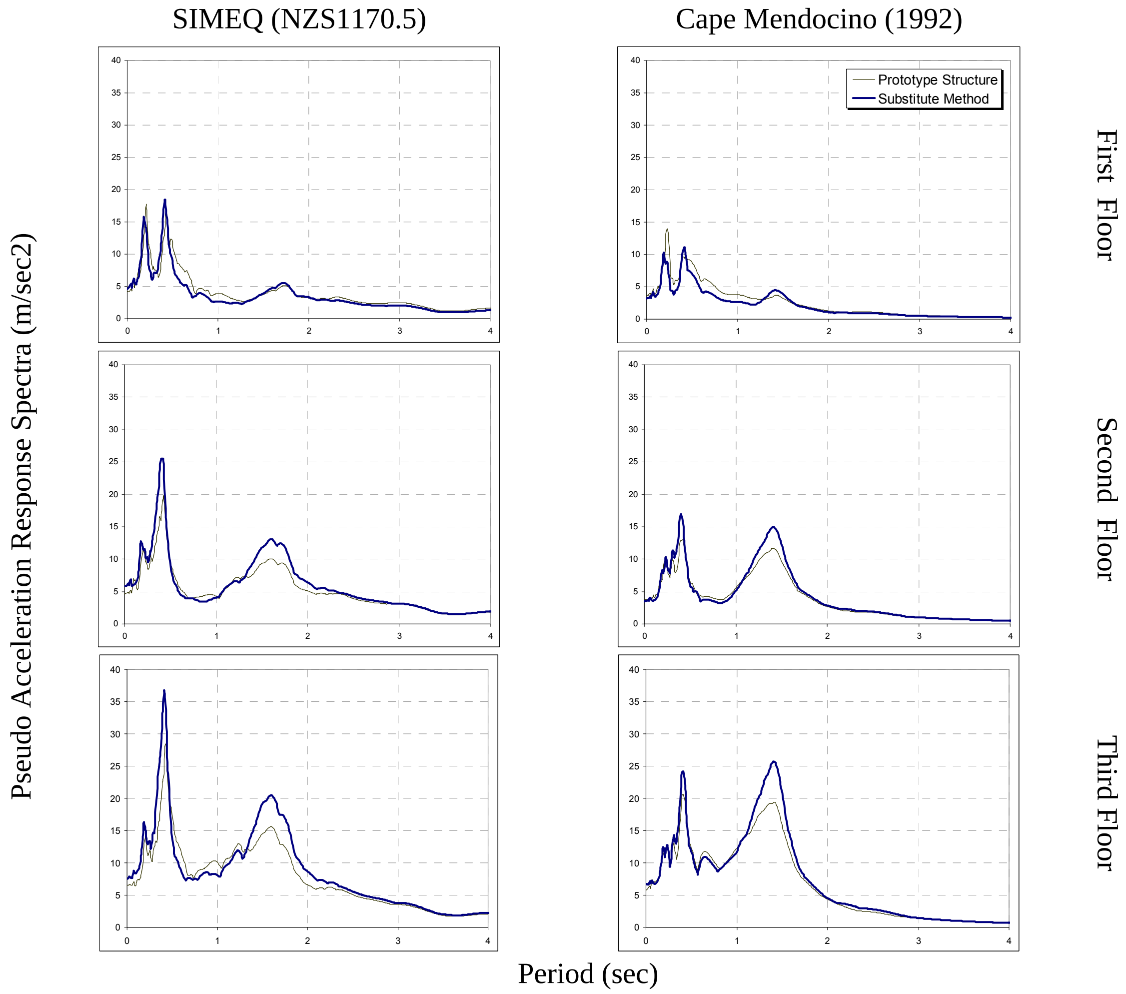


Figure 5 Pseudo-acceleration response spectra using substitute method and prototype structure

Acceptable floor horizontal acceleration response spectra based on the nonlinear behaviour of the supporting structure enables a performance-based seismic design procedure for acceleration-sensitive and/or displacement-sensitive NSCs. A similar method to the displacement-based design procedure can be used for this purpose.

3. Conclusions

An approximate method to simulate floor acceleration response spectra in structural buildings responding inelastically has been presented. The proposed method is based on the assumption that the behaviour of the structure can be simulated by the response of its 1st nonlinear mode in combination with the other predominant higher modes. In the proposed method, elastic time-history response of SDOF systems representing 1st nonlinear mode and other higher modes are combined together to represent the expected time-history response of each floor and based on that response the floor acceleration response spectra can be generated. To obtain the characteristics of the SDOF system representing the 1st nonlinear mode of vibration, the substitute method is used. The stiffness of the system represented by the secant stiffness at maximum expected response and the damping is resulted from the inherent elastic viscous damping and the nonlinear hysteretic damping.

The accuracy of the method has been evaluated by comparing the floor acceleration magnification factor, floor acceleration response spectra, and floor acceleration time-history response computed from the proposed method and nonlinear time-history analysis of the prototype frame. Results show that the new method is able to capture the floor response spectra with a reasonable approximation. It is worth to note that the new method has much less computation time required for a full nonlinear finite element analysis. This simplified method can be easily implemented in small program and can be used by practicing structural engineers.

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