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Direct displacement-based seismic design of flexible-base structures subjected to pulse-like ground motions

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ABSTRACT

In this paper, a practical displacement-based framework is presented for seismic design of flexible-base structures in near-fault regions. Particular attention is given to pulse-like motions that may cause significant damage to building structures. The proposed design methodology utilises displacement response spectra constructed using a new procedure, which takes into account the effect of pulse period. An equivalent fixed-base single-degree-of-freedom oscillator is adopted to capture the salient features of an actual soil-structure interaction (SSI) system in order to facilitate the design process. Two step-by-step direct-displacement based design (DDBD) procedures based on compatible inelastic spectra and equivalent linearisation are introduced. The effectiveness of the stated design procedures is examined using results of nonlinear response history analysis of two example SSI systems subjected to a set of sixteen spectrum-compatible near-fault pulse-like ground motions. The results of this study suggest that the procedure based on inelastic design spectra, in general, provides a better design solution than using an elastic linearisation method, especially when structures are designed with a higher ductility demand.

1. Introduction

Structures located near causative faults are prone to extensive damage in strong earthquakes, especially when ground motions contain distinguishable pulses whose periods are close to those of the buildings. A well-known phenomenon that may lead to velocity pulses is the "forward directivity" effect due to the propagation of fault rupture at a velocity close to the local site shear wave velocity. Compared to far-field ground motions that build up energy more gradually within structures, pulse-like motions (e.g. the fault-normal components of ground motions, in particular) can expose structures to high input energy at the beginning of shaking [1]. Note that in addition to the velocity pulses, high-frequency components of near-fault ground motions may also cause adverse effects on short stiff structures [2,3].

The effects of pulse-like motions on building response have been studied by a number of researchers. Alavi and Krawinkler [4] demonstrated that the storey shear force distribution within a 20-storey moment-resisting steel frame varied significantly with the pulse period T_P (i.e. duration of the distinct ground velocity pulse). In their study, the pulse period was defined based on three mathematical square-wave acceleration pulses that represented a set of 15 near-fault earthquake records. Akkar et al. [5] showed that the maximum inter-storey drift ratio of steel frame systems exhibited noticeably higher values when the fundamental period of the building T_s approached the pulse period T_P, which was measured directly from the ground velocity time series. The results of the stated studies were confirmed by Kalkan and Kunnath [6] by utilising sinusoidal wave shapes to simulate pulse-like motions. Baker [7] developed a method to identify and extract pulses in near-fault ground motions by decomposing velocity time series into wavelets. He suggested that if the largest extracted velocity pulse in a ground motion is "large" enough compared to the remaining motion, the ground motion is classified as pulse-like, and the associated pseudo-period is regarded as the pulse period. Using the pulse-like motions identified by Baker [7], Champion and Liel [8] showed that buildings may have a substantially lower potential for collapse under pulse-like motions (especially when $T_s/T_P \le 0.5$)) than under far-field motions. The results of their study indicated that this phenomenon is more prominent for ductile buildings.

While the above mentioned studies were restricted to fixed-base buildings, it is well known that soil-structure interaction (SSI) may have a pronounced effect on the seismic performance of buildings [9–13]. SSI can change the response of a building by altering the foundation input

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motion and the dynamic properties of the interacting system. The former could even occur for massless foundations without the presence of any superstructure, referred to as the "kinematic interaction", where a stiff foundation cannot follow the pattern of free-field motion. The latter is a direct result of the "inertial interaction" where a shaking superstructure dissipates seismic energy into its foundation through soil deformation and wave radiation, which, in turn, affects the vibration of the structure. More recently, the effects of pulse-like motions on flexible-base structures have received considerable attention (e.g. [14–18]). Similar to the observations stated earlier for fixed-base structures, these studies in general showed a strong dependence of seismic response of flexible-base structures on the fundamental period of the interacting system T_{ssi} relative to the pulse period T_P [16,17].

Although much work has been devoted to the investigation of the effects of pulse-like records on fixed and flexible-base buildings, less effort has been made on the development of a comprehensive design framework that includes these effects [19,20]. Procedures for seismic design or assessment of flexible-base structures have been proposed by a number of researchers [21–25], but as yet few studies have introduced the combined SSI and pulse effects in the design process. SSI procedures in current seismic provisions and standards (e.g. [26,27]) for design of new buildings adopt a force-based approach, which does not address the effects of velocity pulses. This study attempts to incorporate, for the first time, the effects of near-fault pulse motion into a practical direct displacement-based design (DDBD) procedure for seismic design of flexible-base structures. Note that not all near-fault ground motions exhibit intensive velocity pulses, which are also not necessarily due to forward-directivity (depends on orientation of the site relative to the fault plane). The current work deals with near-fault impulsive motions without permanent ground displacements. The proposed design procedure is suitable for buildings supported by raft foundations that are bonded to a homogeneous soil half-space (i.e. for relatively heavy buildings on softer soils). Foundation input motion is assumed to be due to coherent vertically propagating shear waves, in which case kinematic interaction effects are not present. Soil nonlinearity is approximated using equivalent-linear shear modulus and damping ratio values that are compatible with the strain levels associated with the design scenario.

The paper is organised into five main parts (Sections 2–6). Section 2 suggests a novel method to construct compatible design response spectra for near-fault earthquakes which accounts for the pulse period. Section 3 describes a practical approach for substituting an actual SSI system by an equivalent fixed-base single-degree-of-freedom (SDOF) oscillator. Section 4 integrates methods introduced in Sections 2 and 3 into a DDBD framework considering both SSI and near-fault pulse effects. Two step-by-step design procedures based on, respectively, inelastic displacement spectra and equivalent linearisation are then presented in Section 4, with their effectiveness verified and compared using results of response history analysis for two design examples. Finally, the applicability of the proposed design methodology is discussed in Section 5 and conclusions are provided in Section 6.

2. Design response spectrum

2.1. Bi-normalised response spectrum

Currently the seismic design of new structures (or evaluation of existing structures) is usually based on a design response spectrum, which is representative of the response of a series of SDOF oscillators subjected to an ensemble of ground motions. The design response spectrum can be determined empirically by averaging the individual response spectra corresponding to each ground motion in the ensemble with peak response parameters (i.e. acceleration, relative velocity or relative displacement) normalised by the corresponding peak ground values (i.e. peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD)). However, the response spectra of real earthquake records usually exhibit peaks at predominant periods (T_g) that may vary significantly from one record to another. For pulse-like motions, T_P is a predominant period around which spectral shapes can be significantly different from those without impulsive characteristics [19]. Previous studies showed that code-specified spectral shapes based on averaged spectra using various ground motion records failed to reflect realistic spectral ordinates around predominant periods, which may lead to non-conservative design solutions (e.g. reduced spectral accelerations due to irrational averaging) [28–32].

We may define T_{ga}, T_{gv} and T_{gd} as the predominant periods (T_g) at which the pseudo spectral acceleration (PSA), pseudo spectral relative velocity (PSV), and spectral relative displacement (SD) attain their maximum values, respectively. The spectral values are related to one another by the following relations [33]:

$$\mathrm{PSA} = \mathrm{PSV}\left(\frac{2\pi}{T}\right) = \mathrm{SD}\left(\frac{2\pi}{T}\right)^2 \tag{1}$$

where T is the natural period of the vibrating system. Normalising T by the stated predominant periods T_g has been shown to produce more realistic design spectra (referred to as the "bi-normalised spectra" [30]) than the conventional normalised response spectra for near-fault regions [28,29,31]. The idea of such an approach is to preserve the peaks that exist in the individual response spectra at the predominant period T_g. A common practice to develop a bi-normalised design response spectrum is to fit the design spectrum to the statistical mean spectrum through a least-square analysis. The mean spectrum is usually obtained using a large number of ground motion records with the stated normalisation methods [28–31]. For example, Maniatakis and Spyrakos [28] derived such a bi-normalised displacement response spectrum (i.e. SD/PGD vs. T/T_{gd}) as a more accurate means of estimating design displacement demands of structures located in near-fault regions, compared with the current code spectrum. However, as pointed out by Malhotra [34] and Xu et al. [35], the response spectrum of a real earthquake record usually only correlates well with PGA, PGV and PGD in the short, intermediate and long-period regions, respectively. In this sense, a bi-normalised displacement spectrum obtained by averaging SD/PGD for a large number of earthquake records may not be reliable for predicting the maximum displacement of short and intermediate-period structural systems (i.e. those with $T_s < T_{gd}$), which is crucial for direct displacement-based design procedures utilising displacement spectra.

It is a simple matter to show that the pseudo-acceleration calculated using the Maniatakis and Spyrakos [28] displacement spectrum tends to infinity for near-rigid systems, which is contradictory to the fact that the PSA of short-period systems should be close to PGA. A possible solution to the incompatibility between SD and PSA may be in using the Newmark-Hall spectrum [36] based on scaling spectral ordinates from PGA, PGV and PGD. The next subsection adopts the Newmark and Hall procedure to suggest new compatible design response spectra for near-fault sites that can efficiently retain the spectral peaks.

2.2. Compatible response spectrum

Fig. 1(a) illustrates the suggested compatible design response spectrum constructed on a four-way logarithmic chart. Note that values of PSV, PSA and SD are read along the vertical, $-45°$, and $+45°$ axes, respectively, on a log scale. The spectral ordinates at control points a-e are scaled from peak ground parameters by various amplification factors at the corresponding control periods T_a-T_e. It is assumed that PSA = PGA and SD = PGD for $T \le T_a$ and $T \ge T_e$, respectively. Periods T_b, T_c and T_d correspond to the predominant periods T_{ga}, T_{gv} and T_{gd} at which peak ordinates of PSA, PSV and SD are scaled from PGA, PGV and PGD by amplification factors α_a, α_v and α_d, respectively. The design pseudo-velocity spectrum is constructed by connecting straight lines between the control points on the four-way logarithmic chart, while the pseudo-acceleration and the displacement spectra are derived