Effects of wave passage on torsional response of symmetric buildings subjected to near-fault pulse-like ground motions

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Abstract: This article investigates the effects of wave passage on the torsional response of elastic buildings in the near-fault region. The model of the soil-foundation-structure system is a symmetric cylinder placed on a rigid circular foundation supported on an elastic halfspace. The idealized model is subjected to obliquely incident plane SH waves simulating the action of near-fault pulse-like motions. The response of the structure is assessed in terms of the relative twist between the top and the base of the superstructure. A parametric analysis of the maximum relative twist as a function of the input parameters of the seismic excitation and soil-foundation-structure system is performed to identify the parameters that control the torsional response of buildings due to wave passage in the near-fault region. It is shown that the torsional response is most sensitive to a key parameter of the near-fault ground motion referred to as “pulse period”. Specifically, large rotations are observed when the pulse period is close to the torsional period of the structure. It is also demonstrated that the wave passage effects are controlled by the wave apparent velocity, rather than the local site conditions. Furthermore, broadband near-fault ground motions from three hypothetical earthquakes of different magnitude are generated, and the torsional responses due to the simplified pulse-like and broadband ground motions are compared against each other. The results show that the simplified pulse model that describes the coherent seismic radiation is able to represent the main features of the near-fault ground motions that cause large torsional response. The maximum relative twist at resonance is found to be $\sim 10^{-3}$ rad, a value that is consistent with the upper bound of rotations in structures reported in the literature.

Keywords: wave passage effects; near-fault ground motion; pulse period; wave apparent velocity; torsional response; relative twist; soil-structure interaction

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1. Introduction

During an earthquake, buildings may undergo torsional response in addition to translational response. For buildings with inherent eccentricity (i.e. non-symmetric distribution of masses or load-resisting elements), torsion is induced by the geometrical separation of the centers of mass and stiffness, resulting in coupled lateral-torsional response. On the other hand, spatially varying seismic excitations due to ground motion incoherence and wave passage effects contribute to the torsional response of both symmetric and asymmetric buildings. The ground motion incoherence, which arises from the scattering effect of seismic waves traveling through an inhomogeneous medium, is typically investigated using random-vibration-based procedures, whereas the wave passage effect considers deterministically ground motions that differ in phase from point to point due to the seismic wave front impinging the foundation-soil interface obliquely. Several studies have investigated the effects of wave passage and ground motion incoherence on the torsional response of buildings, but only a few of them are summarized in this section. The interested reader is referred to Anagnostopoulos et al. (2015) for a comprehensive literature review of earthquake-induced torsion in structures.

Newmark (1969) made the first rational attempt to investigate the torsional response of symmetric buildings due to base rotation arising from earthquake wave motions. Luco (1976a,b) presented the complete representation of the torsional steady-state response of a symmetric, elastic structure placed on surface-supported or embedded foundation under the action of obliquely incident plane SH waves accounting for soil-structure interaction effects. It was demonstrated that large displacements associated with torsional response may be generated even for symmetric structures. Luco and Wong (1982) presented the earthquake response of symmetric, elastic structures subjected to SH-wave excitation with different angles of incidence and to Rayleigh waves. The results revealed that the response to non-vertically incident waves is significantly different from that obtained on the basis of the usual assumption of vertically incident SH waves.

Luco and Wong (1986) and Luco and Mita (1987) investigated the dynamic response of rigid foundations supported on an elastic halfspace when subjected to a spatially varying ground motion. The results indicated that the spatial randomness of ground motion produces effects similar to the deterministic effects of wave passage, including reduction of the translational components of the response at high frequencies and generation of rocking and torsional response components. Veletsos and Prasad (1989) arrived at a similar conclusion, and further indicated that the kinematic interaction effects due to ground motion incoherence and wave passage may be interrelated. It should be noted that although the studies by Luco and Wong (1986) and Luco and Mita (1987) focused exclusively on the dynamic response of the foundation and did not examine the response of the superstructure, the calculation of the

torsional response at the foundation level is an essential step for further investigating the dynamic response of the superstructure.

Heredia-Zavoni and Barranco (1996) examined the torsional response of symmetric structural systems subjected to spatially varying ground motions due to loss of coherence, wave passage, and local soil conditions. Torsional effects were found to be significant depending on the aspect ratio of the system, the soil conditions, and the times for the seismic waves to travel across the base of the system, especially for structural systems with fundamental periods close to the predominant period of the ground. De la Llera and Chopra (1994) extracted base torsional excitations, associated with spatially varying ground motions, from translational ground motions recorded at the foundation level of actual buildings. It was shown that accidental torsion increases the building displacements by less than 5% for systems that are torsionally stiff or have lateral vibration periods longer than 0.5 s, whereas short period (less than 0.5 s) and torsionally flexible systems may experience significant increases in response.

Juarez and Aviles (2008) examined the combined torsional effects of structural asymmetry and foundation rotation induced by wave passage in flexibly supported structures. The foundation was considered to be embedded into an elastic halfspace and the earthquake excitation was assumed to be composed of obliquely incident SH waves. An effective eccentricity was defined by matching the equivalent static displacement at the flexible edge of the structure with the peak dynamic displacement. For a low-rise structure, the effects of foundation rotation were found to be detrimental and even more important than those of structural asymmetry, whereas the reverse was observed for a midrise structure. The effects of rocking excitation were found to be insignificant compared to those of the torsional excitation.

Near-fault ground motions are frequently characterized by intense velocity and displacement pulses of relatively long duration, which clearly distinguish them from typical far-field ground motions. Even though intermediate- and long-period structures are particular susceptible to near-fault pulse-like seismic excitations, the attention regarding the effects of such motions on structures has almost exclusively focused on translational vibrations. Only a few studies have touched upon the effects of impulsive motions on the torsional response of buildings due to wave passage or ground motion incoherence. For instance, Heredia-Zavoni and Barranco (1996), although not focusing explicitly on pulse-like seismic excitations, modeled the ground motion input as a narrowband process with a dominant frequency, and concluded that torsional response due to spatially varying ground motions could be significant when the torsional period of the structure is close to the predominant period of the ground motion. This finding implies that near-fault ground motions could potentially induce large torsional response when the period of the pulse-like motion is approximately equal to the torsional period of the structure. In a recent study, Gicev et al. (2015) examined the translation, torsion and wave excitation of a

two-dimensional building model supported by a nonlinear rectangular, flexible foundation embedded in nonlinear soil. The building model was excited by an earthquake half-sine pulse of a plane SH wave that was intended to model pulse-like motions observed in the near-fault region. Their results showed that the part of energy due to non-translational motion increases rapidly as the angle of incidence (relative to vertical) increases. Gicev et al. (2015) also concluded that the building response becomes spatially more complex as the ratio of building width and wavelength of incident waves increases.

Building upon the methodology and results presented by Meza-Fajardo (2004) (see also Meza-Fajardo and Papageorgiou, 2004), this article aims to identify the key parameters of the soil-foundation-structure system and ground motion input that control the effects of wave passage on the torsional response of symmetric, elastic buildings under the action of near-fault pulse-like motions. The soil-foundation-structure model proposed by Luco (1976a) is adopted to calculate the torsional response of the buildings, whereas the mathematical model proposed by Mavroeidis and Papageorgiou (2003) is used to describe the coherent component of the near-fault ground motions. The building torsional response is assessed in terms of the relative twist between the top and the base of the superstructure. The characteristics of the transfer function, ground excitation model, and torsional response are investigated through a detailed parametric analysis using realistic physical properties of the soil-foundation-structure system. Finally, broadband near-fault ground motions from hypothetical earthquakes are generated, and the torsional response due to simplified and broadband ground motions are compared to assess the effectiveness of idealized pulse models to estimate accurately the building torsional response due to wave passage in the near-fault region.

2. Soil-Foundation-Structure System

2.1 Model Configuration

Figure 1 illustrates the soil-foundation-structure model under the action of an obliquely incident plane SH wave proposed by Luco (1976a). This model is adopted in the present study to investigate the effects of wave passage on the torsional response of symmetric, elastic buildings subjected to near-fault pulse-like ground motions. The superstructure is modeled by a uniform elastic bar of height H and radius a with mass moment of inertia about the z -axis I_b , hysteretic damping factor ξ , and fixed-base fundamental frequency in torsion ω_1 . The foundation is represented by a rigid circular disc with the same radius as that of the superstructure and mass moment of inertia with respect to the z -axis I_0 . The soil is assumed to be an elastic, homogeneous and isotropic halfspace with density ρ_s , shear modulus μ , and shear wave velocity β .

As discussed by Luco (1976a), the frequency response of the soil-foundation-structure system illustrated in Figure 1 can analytically be calculated by considering the ground motion input as an

obliquely incident plane SH wave of amplitude $u_{g0}/2$ and frequency of steady-state excitation ω , which would lead to a free-field motion of amplitude u_{g0} on the ground surface. The plane SH wave is assumed to propagate at an angle θ with respect to the x axis (i.e. angle of incidence) and the particle movement is parallel to the y axis, as shown in Figure 1.

It should be pointed out that the soil-foundation-structure model proposed by Luco (1976a) may oversimplify certain aspects of the problem under investigation. For instance, soils and structures subjected to strong ground shaking in the near-fault region may undergo nonlinear response, which cannot be captured by the adopted soil-foundation-structure model that assumes linear response. However, the soil-foundation-structure model proposed by Luco (1976a) is considered to be a suitable model for this study because it facilitates the extensive parametric analysis that is required to identify the input parameters of the seismic excitation and soil-foundation-structure system that control the torsional response of buildings due to wave passage in the near-fault region. It is clear that more realistic models, describing the detailed geometry of the building and taking into account the nonlinear structural response, are required to capture more accurately the behavior of the soil-foundation-structure system for design purposes.

2.2 Transfer Function

The torsional response in the frequency domain (also known as transfer function) of the soil-foundation-structure system shown in Figure 1 has been derived analytically by Luco (1976a) and is briefly presented in this section. The average value of the tangential component of displacement $\bar{u}_\theta$ of the rigid circular foundation is decomposed into three parts:

$$\bar{u}_\theta = \bar{u}_\theta^{i+r} + \bar{u}_\theta^R + \bar{u}_\theta^S \quad (1)$$

where $\bar{u}_\theta^{i+r}$ is the average tangential component of the free-field ground motion, $\bar{u}_\theta^R$ is the motion generated by the foundation twist in the absence of seismic excitation, and $\bar{u}_\theta^S$ is the average tangential component of motion that needs to be added to the free-field motion when the foundation is kept fixed under the action of seismic excitation. The average tangential component of the free-field motion $\bar{u}_\theta^{i+r}$ on the surface of the ground can be obtained by considering the free-field ground motion resulting from the incident and reflected plane SH wave. The mixed boundary value problems corresponding to $\bar{u}_\theta^R$ and $\bar{u}_\theta^S$ are reduced to the solution of Fredholm integral equations associated with the numerical evaluation of related functions θ_R and θ_S . Once the average tangential displacement field $\bar{u}_\theta$ is obtained, the average tangential stress field $\bar{\sigma}_{z\theta}$ may be evaluated by

$$\bar{\sigma}_{z\theta}(r, z) = \frac{1}{2\pi} \int_0^{2\pi} \sigma_{z\theta}(r, \theta, z) d\theta = \mu \frac{\partial \bar{u}_\theta}{\partial z} \quad (2)$$

The total torque $T_s e^{i\omega t}$ that the rigid foundation exerts on the soil can then be calculated by integrating the average tangential stress field $\bar{\sigma}_{z\theta}$ along the radius a and by setting $z=0$:

$$T_s e^{i\omega t} = 2\pi \int_0^a \bar{\sigma}_{z\theta}(r, 0) r^2 dr \quad (3)$$

By combining Equations (1)-(3) and after lengthy derivations, T_s can be expressed by

$$T_s = K_{TT}(a_0)(\alpha_b - \alpha^*) \quad (4)$$

where $K_{TT}(a_0)$ is the torsional impedance function for the foundation calculated as

$$K_{TT}(a_0) = 16\mu a^3 \int_0^1 t \theta_R(t) dt; \quad a_0 = a \omega / \beta \quad (5)$$

α_b is the twist angle of the foundation (also known as base twist), and α^* is the input twist that corresponds to the rotation of the rigid foundation under the action of seismic excitation when no external forces are acting on the foundation ($T_s = 0$):

$$\alpha^* = \left(8i\mu a^3 a_0 \cos \theta \int_0^1 t \theta_s(t) dt / K_{TT}(a_0) \right) u_{g0} / a \quad (6)$$

By considering the torque-twist relationship for the superstructure and the coupling of superstructure and soil through the foundation (i.e. equation of motion for the rigid foundation assuming there is no slippage between the soil and the foundation), the base twist α_b can be expressed as

$$\frac{a\alpha_b(\omega)}{u_{g0}} = \frac{S\left(a_1 \frac{\omega}{\omega_1}, \theta\right)}{1 - \frac{\pi}{20} \left(a_1 \frac{\omega}{\omega_1}\right)^2 \left\{ \frac{I_0}{I_s} + \frac{I_b}{I_s} \left[\frac{\tan \left[\frac{\pi}{2} \left(\frac{\omega}{\omega_1} \right) \frac{1}{\sqrt{1+2i\xi}} \right]}{\frac{\pi}{2} \left(\frac{\omega}{\omega_1} \right) \frac{1}{\sqrt{1+2i\xi}}} \right] \right\} \frac{K_{TT}(0)}{K_{TT}\left(a_1 \frac{\omega}{\omega_1}\right)}} \quad (7)$$

where

$$S\left(a_1 \frac{\omega}{\omega_1}, \theta\right) = a a^* / u_{g0}, \quad a_1 = a\omega_1/\beta = a_0\omega_1/\omega, \quad (8a-e)$$

$$I_s = \frac{4\pi}{15} \rho_s a^5, \quad I_b = \frac{1}{2} m_b a^2, \quad I_0 = \frac{1}{2} m_0 a^2$$

with m_b and m_0 denoting the masses of the superstructure and the foundation, respectively. The twist at the top of the superstructure α_t can then be expressed as a function of the base twist α_b by

$$\alpha_t(\omega) = \alpha_b(\omega) \sec \left[\frac{\pi}{2} \left(\frac{\omega}{\omega_1} \right) \frac{1}{\sqrt{1 + 2i\xi}} \right] \quad (9)$$

whereas the relative twist between the top and the base of the superstructure α_r can be defined as

$$\alpha_r(\omega) = \alpha_t(\omega) - \alpha_b(\omega) \quad (10)$$

In summary, the transfer function of the relative twist of the soil-foundation-structure system shown in Figure 1 may be described in terms of the following dimensionless parameters: normalized frequency ω/ω_1 , stiffness of the superstructure relative to that of the soil a_1 (also known as relative stiffness parameter), ratios of moments of inertia I_b/I_s and I_0/I_s , hysteretic damping factor ξ , and angle of incidence θ (see also Luco, 1976a).

2.3 Input Parameters

Table 1 summarizes the basic characteristics of 40 buildings along with their site conditions considered in this study. This information has been obtained from the empirical evaluation of soil-structure interaction effects conducted by Stewart et al. (1998, 1999). The foundation radius adopted herein is the radius of a circular disc whose area matches the area of the actual foundation (see Stewart et al., 1999). Consistent with findings of previous studies on the evaluation of vibration properties of buildings (e.g. Satake et al., 2003), the torsional fundamental period T_1 is assumed to be equal to 80% of the fixed-base translational period of the fundamental mode as that was reported by Stewart et al. (1999).

The relative stiffness parameter a_1 for each building listed in Table 1 can be calculated by substituting the radius a , the torsional fundamental frequency $\omega_1 = 2\pi/T_1$, and the shear wave velocity β in Equation (8b). Figure 2a presents the distribution of the calculated relative stiffness parameter a_1 for

all buildings considered in this study. This histogram suggests that the majority of buildings have a_1 values between 0.1 and 1.75, whereas a few stiff buildings (occasionally with large radius) on soft soil conditions (i.e. small shear wave velocity) are characterized by values of a_1 between 2.0 and 5.5.

The ratios of moments of inertia, I_b/I_s and I_0/I_s , can be calculated by using Equation (8c-e) in conjunction with the foundation radius a and typical values of ρ_s , m_b and m_0 . For instance, the soil density ρ_s ranges from 1.5-2.4 Mg/m³ depending on the type of soil (Holtz et al., 2011). The mass of the superstructure m_b for typical residential and office buildings may be calculated as follows:

$$m_b = \rho_b H A \quad (11a)$$

with

$$\rho_b \approx \frac{q}{gh_0} \quad (11b)$$

where ρ_b is the mass density of the superstructure, H and $A = \pi a^2$ are the height and floor area of the building, h_0 is the interstory height, and q is the gravity load consisting of dead and live loads. Once the mass of the superstructure m_b has been calculated using Equation (11), the mass of the foundation m_0 can be estimated for typical buildings (e.g. Veletsos and Meek, 1974; Juarez and Aviles, 2008; Zhang and Tang, 2008) as:

$$m_0 \approx (0.05 - 0.35)m_b \quad (12)$$

The interstory height for residential and office buildings is about 3.5 m (11.5 ft). Average gross dead loads are 4.8-7.2 kN/m² (100-150 lb/ft²) for reinforced concrete buildings and 2.9-3.8 kN/m² (60-80 lb/ft²) for steel framed buildings (e.g. Saouma, 2004), whereas the minimum uniformly distributed live loads typically vary between 1.9-4.8 kN/m² (40-100 lb/ft²) depending on the occupancy characterization of the building (ASCE, 2010; CTBUH, 1995). By considering the aforementioned values of interstory height, dead load and live load, the mass density of the superstructure ρ_b of Equation (11b) is estimated in the range of 150-350 kg/m³. Typical values of the mass density of the entire building block (including the foundation system) reported in the literature are between 250-350 kg/m³ (e.g. Tsogka and Wirgin, 2003; Kham et al., 2006). It should be noted though that the mass density of the entire building is typically greater than ρ_b .

Representative values of I_b/I_s can be calculated by combining Equations (8c,d) and (11) and by setting ρ_s and ρ_b equal to their average values (i.e. $\rho_s = 1.95$ Mg/m³, $\rho_b = 250$ kg/m³). Figure 2b illustrates the distribution of I_b/I_s for all 40 buildings listed in Table 1 with the majority of these values

being between 0.1 and 0.7. Buildings with $I_b/I_s > 0.7$ are typically characterized by large H/a ratios. Similarly, I_0/I_s can be computed by combining Equations (8c,e), (11) and (12). Provided that the superstructure and the foundation have the same radius a , the ratio of I_0/I_s can alternatively be calculated by multiplying I_b/I_s with the foundation-to-superstructure mass ratio m_0/m_b , which has been shown to vary between 0.05 and 0.35 according to Equation (12).

To obtain a complete set of structural input parameters, the hysteretic damping factor ξ is approximated by the equivalent viscous damping ratio. In general, viscous damping ratios are spread over a relatively wide range (0.5-9.0%) depending on the material (steel vs. reinforced concrete) and height of the building (e.g. Satake et al., 2003; PEER/ATC, 2010; Bernal et al., 2015). Viscous damping ratios are also sensitive to the system identification algorithm utilized in the analysis, as well as the type and level of excitation used as an input (i.e. earthquake-, wind- or forced-induced vibration). For buildings subjected to earthquake loads, values of $\xi = 3.0\%$ and 5% are typically assumed for steel and concrete buildings, respectively. For very tall steel buildings, even smaller values of ξ may be considered (e.g. Bernal et al., 2015).

For the parametric analyses presented in this study, the soil density ρ_s , the mass density of the superstructure ρ_b , the foundation-to-superstructure mass ratio m_0/m_b , and the hysteretic damping factor ξ are set equal to their average values (i.e. $\rho_s = 1.95 \text{ Mg/m}^3$, $\rho_b = 250 \text{ kg/m}^3$, $m_0/m_b = 0.2$, and $\xi = 4\%$). In Section 4, the effect of these four parameters on the torsional response of buildings due to wave passage in the near-fault region will further be investigated and shown to be insignificant.

2.4 Parametric Analysis

This section presents a parametric analysis of the transfer function of the relative twist between the top and the base of the superstructure α_r , expressed by Equation (10), for the soil-foundation-structure system shown in Figure 1 subjected to an obliquely incident plane SH wave. In an effort to model realistic configurations of buildings and soil conditions, input parameters a_1 and I_b/I_s are selected to be consistent with the histogram distributions presented in Figure 2. Namely, the selected values of the relative stiffness parameter a_1 are 0.2, 0.7, 1.2, 2.2, 3.2 and 5.0, whereas the values of the ratio of moments of inertia I_b/I_s are 0.1, 0.8 and 1.7. Following the discussion in Section 2.3, the hysteretic damping factor ξ and the foundation-to-structure mass ratio m_0/m_b are set equal to 4% and 0.2, respectively. Finally, three angles of incidence θ are considered: 0° , 40° and 80° .

The parametric analysis results in Figure 3 are presented in terms of the modulus of normalized relative twist $|a\alpha_r/u_{g0}|$ and the normalized frequency ω/ω_1 for distinct values of a_1 , I_b/I_s , and θ . Parameter $|a\alpha_r/u_{g0}|$ may be interpreted as the ratio of the tangential component of relative motion on the

perimeter associated with the torsional response generated by obliquely incident seismic waves over the amplitude of the free-field motion. For the vast majority of cases, the twist at the top has much larger amplitude than the twist at the base, and therefore the relative twist is principally dominated by the twist at the top. Only for cases characterized by large a_1 and I_b/I_s values (e.g. $a_1 > 3$ and $I_b/I_s > 1$), the base twist is as large as the top twist and thus the amplitude of the relative twist is small. In the remainder of this article, the relative twist between the top and the base of the superstructure α_r (i.e. termed “relative twist” for simplicity) is designated as the single response quantity of interest. In Section 4.1, it will be shown that α_r is indeed a suitable response quantity for investigating the effects of wave passage on the torsional response of buildings subjected to near-fault pulse-like motions.

As shown in Figure 3, the transfer function of the relative twist of the superstructure, expressed in normalized form by $|a\alpha_r/u_{g0}|$, is dependent on a_1 , I_b/I_s , θ , and ω/ω_1 . The relative stiffness parameter a_1 has a significant effect on the transfer function of the relative twist. For very flexible buildings or very stiff soils (e.g. $a_1 = 0.2$) and for very stiff buildings or very soft soils (e.g. $a_1 = 5.0$), the amplitude of the tangential component of relative motion is small and generally less than the amplitude of the free-field motion. For $a_1 = 1.2$, the amplification factor $|a\alpha_r/u_{g0}|$ could reach peak values of up to ~ 8 depending on the particular values of I_b/I_s and θ . Furthermore, it should be noted that, as a_1 increases from 0.2 to 5.0, the resonance frequency of the first peak response tends to decrease, especially for larger values of I_b/I_s .

Inspection of Figure 3 also reveals that the angle of incidence θ largely affects the transfer function of the relative twist. For smaller values of a_1 (e.g. $a_1 < 2.5$), the maximum response occurs for horizontal incidence ($\theta = 0$) and decreases monotonically to zero as θ increases to 90° regardless of the I_b/I_s value. For larger values of a_1 (e.g. $a_1 > 2.5$), the effect of θ on the maximum response is more complex and depends on the value of I_b/I_s . Finally, Figure 3 illustrates that larger values of I_b/I_s tend to reduce the amplitude of the first peak response and shift the corresponding resonance frequency of the soil-foundation-structure system to values smaller than the fixed-base fundamental frequency in torsion ω_1 .

Figure 4a shows the variation of the amplitude of the first peak response with respect to a_1 and θ for characteristic values of I_b/I_s . The maximum amplitude is obtained for $I_b/I_s = 0.1$ and $a_1 \approx 1.5$ under the action of horizontal incidence ($\theta = 0$). Figures 4b and 4c illustrate cross sections of Figure 4a for $a_1 = 1.5$ and 4.5, whereas Figures 4d and 4e illustrate cross sections of Figure 4a for $\theta = 40^\circ$ and 80° . As shown in Figures 4d and 4e, the amplitude of the first peak response varies linearly with a_1 for $a_1 < 0.8$ and is not dependent on the variation of I_b/I_s .

3. Near-Fault Ground Motions

In this study, the soil-foundation-structure system shown in Figure 1 is subjected to near-fault pulse-like ground motions which are assumed to arise from the superposition of incident non-vertical plane SH waves and their total reflections on the free surface. Near-fault seismic ground motions are frequently characterized by intense velocity and displacement pulses of relatively long duration that clearly distinguish them from typical far-field ground motions. The existence of pulse-like motions in near-fault records primarily depends on the relative position of the station that recorded the motion with respect to the direction of propagation of rupture on the causative fault plane and on the magnitude and direction of slip on that segment of the fault that is located in the vicinity of the station. Whenever these ground motion pulses do occur, they are typically caused by the forward directivity and/or permanent translation (fling) effects (e.g. Mavroeidis and Papageorgiou, 2003) and are strongly correlated with large slip on the fault plane locally driven by high stress drop (e.g. Mavroeidis and Papageorgiou, 2010).

3.1 Mathematical Model

For evaluating the effects of wave passage on the torsional response of buildings subjected to near-fault pulse-like motions, the mathematical model proposed by Mavroeidis and Papageorgiou (2003) is adopted to describe the coherent component of the seismic excitation input in the near-fault region. The mathematical formulation proposed by Mavroeidis and Papageorgiou (2003) for the representation of the near-fault velocity pulses is the product of a harmonic oscillation and a bell-shaped function, that is:

$$v(t) = \begin{cases} \frac{A}{2} \left[1 + \cos\left(\frac{2\pi f_P}{\gamma}(t - t_0)\right) \right] \cos[2\pi f_P(t - t_0) + \nu], & t_0 - \frac{\gamma}{2f_P} \leq t \leq t_0 + \frac{\gamma}{2f_P} \text{ with } \gamma > 1 \\ 0, & \text{otherwise} \end{cases} \quad (13)$$

where A controls the amplitude of the signal, f_P is the prevailing frequency of the signal, ν is the phase of the amplitude-modulated harmonic, γ is a parameter that defines the oscillatory character of the signal, and t_0 specifies the epoch of the envelope's peak. The pulse period T_P is defined as the inverse of the prevailing frequency f_P , thus providing an "objective" assessment of this important parameter:

$$T_P = \frac{1}{f_P} \quad (14)$$

The model input parameters have a clear physical meaning as they coincide with the key features that determine the waveform characteristics of the near-fault pulses (i.e. amplitude, duration, phase and

number of half cycles). In addition, the closed-form expression of Equation (13) has the potential to facilitate the parametric investigation of the dynamic response of structural models (similar to the one shown in Figure 1) to variations of the ground motion input parameters (i.e. A , T_p , γ , and ν) that affect near-fault pulse-like motions.

3.2 Input Parameters

In previous studies, the scaling characteristics of the model input parameters were investigated using a large set of actual near-fault ground motion records affected by forward directivity, and simple empirical relationships were proposed (Mavroeidis and Papageorgiou, 2003; Halldorsson et al., 2011; Cork et al., 2016). By performing least-squares analysis, Mavroeidis and Papageorgiou (2003) (see also Cork et al., 2016) obtained the following relationship between the pulse period T_p and the earthquake magnitude M_w with the requirement to satisfy the self-similarity condition:

$$\log T_p = -2.9 + 0.5M_w \quad (15)$$

The amplitude of the near-fault velocity records appears to be a fairly stable parameter. A value of 100 cm/s effectively represents peak ground velocities within a few kilometers from the causative fault regardless of the earthquake magnitude (Mavroeidis and Papageorgiou 2003). This observation is in good agreement with the typical slip velocity value of 100 cm/s frequently considered by seismologists. As indicated by Mavroeidis et al. (2004), there are solid physical reasons that explain the stability of the velocity amplitude close to the fault. More recently, Halldorsson et al. (2011) and Cork et al. (2016) proposed attenuation relationships for peak ground velocity (PGV) with rupture distance R . Finally, parameter γ varies from a value slightly larger than 1 up to a maximum value of 3, whereas the phase angle ν varies from 0 to 360°. Halldorsson et al. (2011) provided the probability density functions for γ and ν , assuming that these two parameters are normally distributed.

In this study, T_p ranges from 0.5 to 10 s with an increment of 0.05 s. According to Equation (15), this range of values for T_p corresponds to earthquake magnitudes M_w between 5.2 and 7.8. Parameter A is fixed to 100 cm/s based on the recommendation made by Mavroeidis and Papageorgiou (2003) and Mavroeidis et al. (2004) for sites located within a few kilometers from the causative fault. The effect of γ is taken into account by considering four characteristic values: 1.0, 1.5, 2.0, and 3.0. Finally, parameter ν is set equal to 90° to ensure that the displacement offset vanishes for all generated motions in agreement with the waveform characteristics of pure forward directivity motions. It should also be pointed out that the variation of ν has little effect on the Fourier amplitude spectrum of the ground velocity (see Mavroeidis and Papageorgiou, 2003), a quantity used in Section 4 to investigate the effects of wave

passage on the torsional response of symmetric, elastic buildings under the action of near-fault pulse-like motions. Mavroeidis et al. (2004) have also shown that the effect of ν on the response of elastic and inelastic single-degree-of-freedom (SDOF) systems is insignificant.

3.3 Parametric Analysis

Mavroeidis and Papageorgiou (2003) derived a closed-form expression for the Fourier transform $V(\omega)$ of the velocity signal provided by Equation (13) as a function of A , T_p , γ , and ν . The Fourier transform $D(\omega)$ of the corresponding displacement signal can readily be derived by the following equation:

$$D(\omega) = \frac{V(\omega)}{i\omega} \quad (16)$$

Figure 5 illustrates the variation of the normalized Fourier amplitude of displacement $|D(\omega)|\omega_p^2/A$ as a function of the normalized frequency ω/ω_p (where $\omega_p = 2\pi f_p$) for characteristic values of γ . Inspection of this figure reveals that the Fourier amplitude spectrum of displacement is a relatively narrowband spectrum with its peak amplitude depending on γ , A and ω_p . For constant values of ω_p and A , the effect of γ on the Fourier amplitude spectrum depends on the frequency range of interest. For $0.5 \leq \omega/\omega_p \leq 1.5$, the Fourier amplitude increases with γ , while this trend is reversed for $\omega/\omega_p \geq 1.5$. Overall the peak amplitude of the spectrum is increased by a factor of ~ 2 as γ increases from 1 to 3. Also for a constant value of ω_p , a larger value of γ gives rise to a narrower bandwidth, with the frequency of the peak amplitude being closer to the prevailing frequency ω_p of the pulse. In addition, as was explained in Section 3.2, parameter A that scales the spectral ordinates is a stable parameter within a few kilometers from the causative fault. On the other hand, ω_p (or equivalently $T_p = 2\pi/\omega_p$) turns out to be the most important parameter. Note that the displacement Fourier amplitude spectrum is normalized with respect to ω_p^2 , meaning that the Fourier amplitude increases quadratically with T_p . For the range of pulse periods considered in this study ($T_p = 0.5$ -10 s), the Fourier amplitude would vary by a factor of 400. Hence, it is reasonable to conclude that, although γ and A contribute to the variability of the Fourier amplitude spectrum of displacement, the dominant parameter is the pulse period T_p .

4. Torsional Response due to Simplified Near-Fault Pulse-Like Ground Motions

In this section, a detailed investigation is performed to identify the key parameters of the soil-foundation-structure system and ground motion input that control the effects of wave passage on the torsional response of symmetric, elastic buildings under the action of simplified near-fault pulse-like

motions. The torsional response of the soil-foundation-structure system in the frequency domain is first calculated by multiplying the transfer function of the relative twist $\alpha_r(\omega)$ of the superstructure subjected to an obliquely incident plane SH wave with the Fourier transform $D(\omega)$ of the simplified ground displacement. The transfer function $\alpha_r(\omega)$ can be calculated by combining Equations (10), (9) and (7) for $u_{g0} = 1$, whereas $D(\omega)$ can be computed from Equation (16). The relative twist of the superstructure in the time domain is then obtained by applying the inverse Fast Fourier transform. The response quantity of interest is the maximum relative twist expressed in radians.

As was mentioned in Section 2.3, the soil density ρ_s , the mass density of the superstructure ρ_b , the foundation-to-superstructure mass ratio m_0/m_b , and the hysteretic damping factor ξ are set equal to their average values (i.e. $\rho_s = 1.95 \text{ Mg/m}^3$, $\rho_b = 250 \text{ kg/m}^3$, $m_0/m_b = 0.2$, and $\xi = 4\%$). By varying these parameters over their potential range of values presented in Section 2.3 (i.e. $\rho_s \in [1.5, 2.4] \text{ Mg/m}^3$, $\rho_b \in [150, 350] \text{ kg/m}^3$, $m_0/m_b \in [0.05, 0.35]$, and $\xi \in [3\%, 5\%]$), the building torsional response due to wave passage changes by less than 20% as compared to the torsional response computed based on the average values of these parameters, and thus their overall effect on the torsional response is considered to be relatively small.

4.1 Effect of Ground Motion Parameters

Three buildings with short, intermediate and long periods are selected from Table 1 to examine the effect of ground motion parameters on torsional response: Building A10 (3 stories, $H = 14.4 \text{ m}$, $a = 22.9 \text{ m}$, $T_1 = 0.22 \text{ s}$ and $\beta = 234.1 \text{ m/s}$), Building A36 (6 stories, $H = 23.1 \text{ m}$, $a = 15.2 \text{ m}$, $T_1 = 0.86 \text{ s}$ and $\beta = 362.1 \text{ m/s}$), and Building A28 (19 stories, $H = 95.8 \text{ m}$, $a = 28.0 \text{ m}$, $T_1 = 2.76 \text{ s}$ and $\beta = 298.7 \text{ m/s}$). Figure 6 presents the maximum relative twist of the three buildings as a function of the normalized period T_p/T_1 for different values of γ and θ . Since the T_p values considered in this study range from 0.5 to 10 s (corresponding to M_W 5.2-7.8), the effective range of T_p/T_1 over which Figure 6 is plotted differs for each of the three buildings because of their distinct torsional periods T_1 . With reference to Figure 6, the following observations are made:

- The global peak of the maximum relative twist is attained when T_p/T_1 approaches a value slightly larger than one (see Buildings A28 and A36), indicating that resonance occurs when the pulse period approaches the torsional period of the building. This is in agreement with observations reported previously by Mavroeidis et al. (2004) and Hubbard and Mavroeidis (2011) with regard to the resonance response of SDOF systems subjected to near-fault pulse-like excitations. When T_p/T_1 becomes equal to ~ 0.6 (see Building A28), a local peak amplification is observed associated with the excitation of the second mode which is evident in Figure 3. As shown in Figure 6, the maximum

relative twist that corresponds to the first mode is greater than the one of the second mode. This can be attributed to the fact that (a) the Fourier amplitude spectrum of the ground displacement decays quadratically as ω_p increases (or equivalently T_p decreases) (see Figure 5), and (b) the amplitude of the transfer function of the relative twist for the first mode is typically greater than that of the second mode (see Figure 3). Finally, Building A10 is not subjected to resonance response due to the range of T_p/T_1 values considered in this study, and thus the response tends to be significantly smaller than the response of Buildings A28 and A36.

- As γ increases from 1 to 3, the global peak of the maximum relative twist (resonance response) increases by a factor of ~ 2 and the shape of the response curve changes gradually from “flattened” to “peaked”. It is also worth mentioning that for $T_p/T_1 > 1.5$, the maximum relative twist decreases as γ increases from 1 to 3. These observations are consistent with the observed influence of γ on the Fourier amplitude spectrum of the ground displacement (see Section 3.3 and Figure 5).
- Comparison of Figures 6a and 6b indicates that the maximum relative twist decreases significantly as θ increases from 40° to 80° . This observation is consistent with the effect of θ on the transfer function of the relative twist for $a_1 < 2.5$ discussed previously (see Section 2.4 and Figure 3; for reference, a_1 values for Buildings A10, A36, and A28 are less than 2.5).
- The maximum relative twists for the short-, intermediate- and long-period buildings shown in Figure 6a or Figure 6b cannot be compared to one another in a meaningful way, even for fixed values of γ and θ . In addition to γ and θ , the maximum relative twist depends on various parameters, including a , I_b/I_s , a_1 (which also incorporates the effect of β), as well as ω_p (or equivalently T_p) and ω_1 (or equivalently T_1). Finally, it should be reiterated that Figure 6 does not incorporate the effect of A on torsional response since this ground motion parameter was fixed to 100 cm/s for reasons that are outlined in Section 3.2. Nevertheless, the effect of A on the maximum relative twist can be taken into account by considering the attenuation relationships for peak ground velocity (PGV) with rupture distance R proposed by Halldorsson et al. (2011) or Cork et al. (2016) and by scaling the ordinates of the response curves accordingly.

Figure 7a presents the same parametric analysis as the one shown in Figure 6a for Building A28 with the exception that the response quantity of interest is the overall maximum relative twist between any point along the height of the building and its base. Comparison of Figures 7a and 6a for Building A28 indicates that the maximum relative twist between the top and the base of the building is identical to the computed overall maximum relative twist between any point along the height of the building and its base with the exception of points lying on the tail of the response curve for $T_p/T_1 \leq 0.4$ (i.e. circled by dashed line in Figure 7a). This can be attributed to higher mode effects as shown in Figure 7b where the variation

of the maximum relative twist along the height of the building is plotted for $T_P/T_1 = 0.2, 1.0$ and 1.8 . For $T_P/T_1 = 1.0$ and 1.8 , the maximum relative twist along height increases monotonically as z/H increases from 0 to 1 indicating the dominant effect of the first mode and that the overall maximum relative twist along height is identical to the maximum relative twist between top and base. However, in the case of $T_P/T_1 = 0.2$, the maximum relative twist between top and base is about 20% smaller than the overall maximum relative twist along height, which occurs at ~30%-40% of the building height depending on the value of γ . It should parenthetically be noted that for Buildings A10 and A36, the maximum relative twist between top and base is identical to the overall maximum relative twist along height for all values of T_P/T_1 considered in the analysis. Therefore, although the maximum relative twist between top and base would slightly underestimate the overall maximum relative twist along height for small values of T_P/T_1 due to higher mode effects, it is deemed an appropriate quantity for representing the largest torsional response for the vast majority of cases and certainly for $T_P/T_1 \approx 1$ where the global peak of the maximum torsional response is attained.

4.2 Physical Constraints on Angle of Incidence

As was discussed in Sections 2.4 and 4.1, the transfer function and thus the maximum relative twist depend on the angle of incidence θ . In reality, the impinging directions of the traveling seismic waves are considerably close to vertical. The primary reason is that soils are layered with shear wave velocities increasing with depth and therefore the refraction of waves at the layer interfaces cause the waves to travel in a more vertical direction as they approach the ground surface (e.g. O'Rourke et al., 1982).

The angle of incidence θ of a plane SH wave and the shear wave velocity β at the top soil layer are related through the following equation:

$$c_H = \frac{\beta}{\cos \theta} \quad (17)$$

where c_H is the wave apparent horizontal velocity. Previous studies published in the literature have estimated the range of c_H values of body waves in the near-fault region using empirical or numerical approaches. For instance, O'Rourke et al. (1982) approximated c_H at a particular site based on the three acceleration components of ground motion and the material properties of the top layer. The method was applied to several sites that recorded the 1971 San Fernando and 1979 Imperial Valley earthquakes, and the median c_H values reported by O'Rourke et al. (1982) for the two earthquakes were 2.1 and 3.8 km/s, respectively. Through numerical simulations of ground motion in the near-fault region, Bouchon and Aki

(1982) concluded that c_H is controlled by the rupture velocity on the fault plane or the shear wave velocity of the basement rock with values ranging between 2.2 and 3.5 km/s. The numerical values of c_H computed by Luco and Sotiropoulos (1980) for different epicentral distances and source depths using a simple model of the seismic source and propagation medium are also in good agreement with the c_H values reported by O'Rourke et al. (1982) and Bouchon and Aki (1982).

Three representative values of c_H (i.e. 2.0, 2.9 and 3.8 km/s) and four representative values of β (i.e. 1130, 560, 270 and 170 m/s) corresponding to the National Earthquake Hazards Reduction Program (NEHRP) Site Classes B, C, D and E (BSSC, 2004) are selected for the parametric analysis presented in Sections 4.2 and 4.3. By substituting these representative c_H and β values into Equation (17), the angle of incidence θ is estimated to vary between 55.6° and 87.4° , suggesting that the impinging directions of the traveling seismic waves are mostly close to vertical. Note that the adopted values of β aim to examine the effect of different soil types on torsional response and do not necessarily reflect the site conditions of any particular building listed in Table 1.

Figure 8 illustrates the maximum relative twist of the three buildings shown in Figure 6 (Buildings A10, A28 and A36) as a function of the normalized period T_P/T_1 for $\gamma = 2.0$ and distinct values of $c_H = 2.0, 2.9, 3.8$ km/s and $\beta = 170, 270, 560, 1130$ m/s. Each line style represents a certain wave apparent velocity, whereas the four shear wave velocities are not specified individually since their impact on the torsional response for a particular c_H value is insignificant. Figure 8 shows that the maximum relative twist increases as c_H decreases. This observation is consistent with findings reported in the literature indicating that a lower value of c_H would induce a larger torsional response (e.g. Luco and Sotiropoulos, 1980; Li et al., 1993; Aviles and Suarez, 2006) and that the maximum rotation of the ground about the vertical axis is inversely proportional to c_H (Newmark, 1969). More importantly, although the shear wave velocity β (through the relative stiffness parameter a_1) and the angle of incidence θ were found to be important parameters for the transfer function (see Section 2.4) and thus for the relative twist (see Section 4.1), Figure 8 suggests that it is actually the wave apparent velocity c_H that determines the relative torsional response of the superstructure. These numerical results are not surprising, since careful inspection of the mathematical formulation of the transfer function reveals that the foundation input twist a^* depends on $\cos \theta / \beta$, which is exactly the inverse of c_H .

4.3 Effect of Building Parameters

In this section, the effect of building parameters on torsional response due to wave passage in the near-fault region is investigated. Figure 9 presents the maximum relative twist of all 40 buildings listed in Table 1 as a function of T_P/T_1 for $\beta = 170, 270, 560, 1130$ m/s and $\gamma = 2.0$, whereas c_H is selected as 2.0 km/s to obtain the most critical response. Note that the effective range of T_P/T_1 over which each

response curve is plotted is different for each building because of its distinct torsional period T_1 . Figure 9 shows that both the shape and amplitude of the response curves are remarkably similar to one another, especially as the shear wave velocity increases.

This trend is schematically explained in Figure 10 where Buildings A36 ($T_1 = 0.86$ s) and A28 ($T_1 = 2.76$ s) are excited by near-fault pulses with T_p values of 1.03 s and 3.31 s, respectively. These pulse periods were selected so that $T_p/T_1 = 1.2$ for both buildings. Without loss of generality, the shear wave velocity is considered to be equal to 270 m/s, whereas γ is set equal to 2. Figure 10a presents a direct comparison between the Fourier amplitude spectra of the relative twist $|\alpha_r(\omega) \cdot D(\omega)|$ for Buildings A36 and A28. It can be seen that both buildings vibrate in their fundamental mode and that their Fourier amplitude spectra are of similar shape. As was discussed in Section 2.4 and shown in Figures 4d and 4e, the amplitude of the first peak response of the transfer function $|a\alpha_r/u_{g0}|$ increases linearly with a_1 (or equivalently, in the case study presented in Figure 10 where β is fixed, $|\alpha_r/u_{g0}|$ increases linearly with ω_1) in the range of $a_1 < 0.8$. For reference, the a_1 values for Buildings A36 and A28 are 0.41 and 0.24, respectively. On the other hand, the Fourier amplitude of the ground displacement decreases quadratically with ω_p as was discussed in Section 3.3 and shown in Figure 5. As a consequence, for a fixed value of ω_1/ω_p (or equivalently T_p/T_1), an increase in ω_1 (e.g. from Building A28 to Building A36) leads to a linear decrease of the peak of the Fourier amplitude spectrum of the relative twist, as shown in Figure 10a. Figure 10b illustrates the same information, but now the frequency ω and the Fourier amplitude of the relative twist $|\alpha_r(\omega) \cdot D(\omega)|$ for Building A28 have been normalized with respect to $k = \omega_1^{A36}/\omega_1^{A28} = 3.21$, yielding a normalized Fourier amplitude spectrum that is identical to the Fourier amplitude spectrum of Building A36. According to the time scaling property of the Fourier transform (e.g. Papoulis, 1962), the frequency responses of Buildings A36 and A28 shown in Figure 10a correspond to time signals of the same amplitude and waveform characteristics, but with a scaled time vector. Figure 10c shows the time histories of the relative twist for Buildings A36 and A28 confirming the time scaling property of the Fourier transform. Furthermore, Figure 10d shows that the Fourier amplitude spectra of Buildings A36 and A28 become identical, if properly normalized with ω_1 (i.e. $|\alpha_r(\omega/\omega_1) \cdot D(\omega/\omega_1)|\omega_1$ vs. ω/ω_1). It should be noted that the features shown in Figure 10 apply to all those buildings listed in Table 1 characterized by $a_1 < 0.8$. If $a_1 > 0.8$, relatively small differences in the amplitude of the maximum relative twist are observed.

On the other hand, the small variation in torsional response caused by different shear wave velocities (see Figure 9) confirms that local site condition have an insignificant effect on relative twist. This effect is even less pronounced for larger values of the shear wave velocity. This observation can be explained by considering that increased values of shear wave velocity would result in more sites within

the range of $a_1 < 0.8$, thus satisfying the condition of the linearly increasing peak amplitude of the transfer function mentioned in the previous paragraph. Finally, the case study presented in Figure 10 indicates that building parameters such as height and radius have relatively insignificant effect on the torsional response, especially for stiff soil conditions, whereas the period ratio T_p/T_1 is clearly the dominant parameter.

5. Torsional Response due to Broadband Near-Fault Pulse-Like Ground Motions

In this section, broadband near-fault pulse-like ground motions are simulated and the effects of wave passage on the torsional response of symmetric, elastic buildings subjected to these synthetic motions is investigated. The objective is twofold: (a) estimate the amplitude of the relative twist that buildings may experience under broadband near-fault pulse-like excitations, and (b) investigate the effectiveness of idealized pulse models to estimate the building torsional response.

5.1 Synthesis of Broadband Near-Fault Pulse-Like Ground Motions

Mavroeidis and Papageorgiou (2003) proposed a simplified methodology for generating broadband near-fault pulse-like motions adequate for engineering analysis and design. Based on this technique, the coherent (long-period) ground motion component is simulated using the mathematical model presented in Section 3.1, whereas the incoherent (high-frequency) seismic radiation is synthesized using the specific barrier model (SBM) (Papageorgiou and Aki, 1983a,b; Papageorgiou, 2003). This simplified methodology has been applied to both hypothetical and actual earthquakes (e.g. Mavroeidis and Papageorgiou, 2003; Halldorsson et al., 2011).

The SBM is a physical model of the seismic source that applies both to the “near-fault” and “far-field” regions, allowing for consistent ground motion simulations over the entire frequency range and for all distances of engineering interest. In the context of the SBM, the fault is visualized as an ensemble of non-overlapping circular subevents of equal diameter $2\rho_0$ that cover a rectangular fault with length L and width W (Figure 11). As the rupture front sweeps the fault plane with a rupture velocity V , a local stress drop $\Delta\sigma_L$ occurs on each subevent. The subevent rupture starts from its center and spreads radially outward with a constant spreading velocity until it is arrested by the barriers, denoted by the shaded area between the cracks in Figure 11. The SBM has been calibrated to shallow crustal earthquakes of three different tectonic regions: interplate, intraplate, and extensional regimes (Halldorsson and Papageorgiou 2005). Given an earthquake magnitude and the tectonic region, the interdependence of other source parameters on the local stress drop $\Delta\sigma_L$ and the barrier interval $2\rho_0$ allows the causative earthquake fault to be constructed. Since the site of interest is considered to be in the “near field” of the ruptured fault, it is necessary to simulate time histories for each individual subevent of the SBM, rather than for the entire

seismic event as an aggregate of subevents. As explained by Halldorsson et al. (2011), the subevent time histories are subsequently summed up at the site, appropriately lagged in time accounting for the time it takes the sweeping rupture front to reach the subevent and for the travel time of the seismic radiation from the subevent to the site. The high-frequency time histories simulated in this manner account for the directivity effects that the fault geometry and rupture progression have on the high-frequency strong motion expected at the given site.

In what follows, broadband near-fault pulse-like motions for three hypothetical earthquakes of M_W 5.8, 6.4 and 7.0 are simulated, representing moderate, moderate-to-large and large seismic events, respectively. The causative fault is assumed to be a vertical strike-slip fault in an interplate region. Figure 11 illustrates the fault-station geometry considered herein for the high-frequency ground motion simulations using the SBM. The diameter of the subevents $2\rho_0$ is 2.1, 4.1, 8.2 km for M_W 5.8, 6.4 and 7.0, respectively. According to the calibration of the SBM for interplate regimes (Halldorsson and Papageorgiou, 2005), the total number of subevents that make up the SBM is typically 15. For consistency and simplicity in our simulations, the 15 subevents are arranged in a 5x3 pattern for all three earthquakes, even though this subevent arrangement may not necessarily be realistic for the M_W 7.0 earthquake because of the implied fault width. The station is located in the forward direction with respect to the propagation of rupture and the site characterization is assumed to be NEHRP site class D. The parameters for the simulation of the long-period component using the mathematical model proposed by Mavroeidis and Papageorgiou (2003) are chosen as $A = 100$ cm/s, $\gamma = 2.0$, and $\nu = 90^\circ$. Based on Equation (15), the pulse period T_p is calculated as 1.0, 2.0 and 4.0 s for M_W 5.8, 6.4 and 7.0, respectively. Figure 12 illustrates the high-frequency, long-period and broadband time histories of ground acceleration, velocity and displacement generated at the site of interest for the three seismic events.

5.2 Results and Discussion

The computational procedure for estimating the torsional response due to the synthetic high-frequency, long-period and broadband ground motions shown in Figure 12 is similar to the one outlined and applied in Section 4, except that the Fourier transform of the ground displacement is now obtained using the Fast Fourier transform for frequencies up to 15 Hz. Figure 13 presents the variation of the maximum relative twist as a function of T_1 for all 40 buildings listed in Table 1 due to the hypothetical earthquakes described in Section 5.1. It is assumed that $c_H = 2.0$ km/s and $\gamma = 2.0$, whereas the pulse period T_p corresponding to each earthquake is indicated by a dotted vertical line. With reference to Figure 13, the following observations are made:

- For all three earthquakes, the maximum relative twist due to the high-frequency component of ground motion varies within the range of 10^{-6} to $6 * 10^{-4}$ rad, and tends to increase with earthquake magnitude due to the increasing amplitude of the high-frequency motion.
- For the M_W 5.8, 6.4 and 7.0 earthquakes, the agreement between the values of maximum relative twist due to long-period and broadband ground motions is very good for $T_1 > 0.5$ s, 1.0 s and 2.5 s, respectively. For smaller values of T_1 , the maximum relative twist due to the broadband motion is clearly controlled by the high-frequency component. It should also be noted that the maximum relative twist due to the broadband motion is consistently greater than that of the long-period or high-frequency component.
- The maximum relative twist of short-period buildings ($T_1 < 0.5$ s) appears to be relatively small for all three earthquakes and is controlled by the high-frequency component of ground motion, thus implying that torsional response of short-period buildings due to wave passage is relatively insensitive to near-fault pulse-like motions.
- For both the long-period and broadband ground motions, the global peaks of the maximum relative twist are of comparable magnitude and occur for buildings whose torsional period T_1 is close to the pulse period T_p of the seismic excitation. This implies that torsional response at resonance is associated with near-fault pulse-like motions. In addition, the maximum relative twist at resonance exceeds 10^{-3} rad, a value that is consistent with the relative twist of a 13-story steel moment-resisting frame building computed by Maison and Ventura (1991) and with the upper bound of rotations in structures reported by Trifunac (2009).

It should be noted that for symmetric buildings, torsional response may also be caused by ground motion incoherence (associated with the high-frequency component of the near-fault ground motions), in addition to the wave passage effects considered herein. Therefore, the actual maximum relative twist of symmetric buildings could potentially be greater than the values obtained in the present study. Moreover, nonlinear soil response, ground failure, and soil liquefaction may induce additional transient and permanent rotations on ground surface near faults, which in turn may further affect the building torsional response (Trifunac, 2009).

6. Conclusions

The effects of wave passage on the torsional response of symmetric, elastic buildings subjected to near-fault pulse-like ground motions were investigated by considering soil-structure interaction effects and realistic configurations of buildings and soil conditions. The following conclusions are drawn:

- Results of parametric analysis have shown that (a) the relative stiffness parameter a_1 , the angle of incidence θ , the ratio of moments of inertia I_b/I_s , and the normalized frequency ω/ω_1 affect the amplitude of the transfer function of the relative twist, and (b) the pulse period T_p is the dominant parameter of the Fourier amplitude spectrum of ground displacement, whereas parameters γ and A contribute to its variability to a lesser extent.
- The peak value of the maximum relative twist is attained when the pulse period T_p of the ground excitation approaches the torsional period T_1 of the building, and increases by a factor of ~ 2 as γ increases from 1 to 3. In addition, the maximum relative twist strongly depends on the wave apparent velocity c_H , rather than the local site conditions. Finally, building parameters such as height and radius have relatively insignificant effect on the relative twist, especially for stiff soil conditions.
- The maximum relative twist of short-period buildings appears to be relatively small and is controlled by the high-frequency component of ground motion, thus implying that torsional response of short-period buildings due to wave passage is relatively insensitive to near-fault pulse-like motions. The maximum relative twist at resonance is controlled by the long-period component of ground motion and exceeds 10^{-3} rad, a value that is consistent with the upper bound of rotations in structures reported in the literature.

As stated by Jennings (1997), simplified structural models can be useful in assessing the potential earthquake response of buildings when their detailed properties are not known and in estimating the demands placed on buildings subjected to strong ground motions under the assumption that the building response, although nonlinear, is describable by linear models with appropriate period and equivalent damping. This assumption implies that structural yielding is not excessive and is well distributed over the structure. In addition, simplified structural models have the potential to facilitate the extensive parametric analysis that is frequently required to identify the parameters of the seismic excitation and soil-foundation-structure system that control structural response. Nevertheless, it should be noted that approaches based on simplified structural models are not intended to replace standard practices in the final stages of the design of buildings.

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TABLE	TITLE
1	Buildings considered in present study (modified after Stewart et al., 1998, 1999).

Table 1. Buildings considered in present study (modified after Stewart et al., 1998, 1999)

Site index ^a	Building	Structural system ^b	No. stories	Height ^c (m)	Foundation radius ^d (m)	Fixed-base fundamental translational period (s)	Soil shear wave velocity (m/s)
“A” Sites							
1	Eureka Silvercrest Apts.	SW	5	13.5	17.4	0.15	213.7
4	Emeryville Pacific Pk. Plaza	CF	31	94.9	26.5	2.45	136.6
5	Hayward City Hall	DWF	11	36.6	20.1	1.11	673.6
8	Piedmont Jr. High School	SW	3	10.9	15.8	0.16	554.7
10	Richmond City Hall	DWF	3	14.4	22.9	0.28	234.1
11	San Jose 3-St. Offc. Bldg.	SF	3	15.2	26.2	0.67	805.3
12	El Centro Imp. Co. Ser. Bldg.	DWF	6	23.5	18.6	0.50	141.4
13	Indio 4-St. Gov’t Offc. Bldg.	DWF	4	24.4	21.0	0.67	211.8
14	Lancaster 3-St. Offc. Bldg.	SW	3	11.3	16.5	0.20	276.8
15	Lancaster 5-St. Hospital	SF	5	17.4	30.2	0.69	305.1
17	Loma Linda VA Hos.	SW	4	21.8	75.0	0.25	431.3
20	Long Beach VA Hospital	SW	11	42.7	25.6	0.51	348.4
23	LA 6-St. Offc. Bldg.	SF	5	24.4	6.5	0.82	192.0
24	LA 6-St. Pkg. Garage	SW	6	17.4	48.5	0.51	265.2
26	LA 7-St. UCLA Bldg.	DWF	7	28.7	10.1	0.63	167.0
27	LA 15-St. Offc. Bldg.	SF	17	75.8	39.9	3.20	353.9
28	LA 19-St. Offc. Bldg.	SF	19	95.8	28.0	3.45	298.7
29	LA Hollywood Storage Bldg.	CF	14	41.8	18.0	1.77	283.5
30	LA Wadsworth VA Hospital	SF	6	34.0	57.6	0.92	299.0
31	Newport Beach Hoag Hospital	SW	11	40.9	18.6	0.70	307.5
33	Norwalk 12440 Imp. Hwy.	SF	7	31.4	43.3	1.30	276.2
34	Palmdale 4-St. Hotel	SW	4	10.5	21.0	0.12	480.1
35	Pomona 2-St. Bldg.	CF	2	12.2	18.0	0.25	379.8
36	Pomona 6-St. Bldg.	CF	6	23.1	15.2	1.07	362.1
38	San Bernardino 3-St.	SF	3	12.6	23.8	0.52	269.1
39	San Bernardino 5-St.	SW	5	22.6	29.0	0.65	375.8
40	San Bernardino Vanir Tower	SF	9	32.2	16.8	2.01	258.5
41	San Bernardino Co. Govt Cntr	SF	5	16.5	34.7	0.51	308.2
44	Sylmar Olive View Med. Cen	SW	6	27.4	38.4	0.27	459.0
45	Ventura 12-St. Hotel	SW	12	30.0	18.9	0.53	270.1
“B” Sites							
2	San Bruno 9-St. Offc. Bldg.	SW	9	28.7	21.9	0.97	279.2
3	San Fran. 47-St. Offc. Bldg.	SF	47	180.3	26.2	5.03	145.7
5	San Jose 10-St. Resid. Bldg.	SW	10	26.6	19.8	0.29	234.1
6	San Jose 13-St. Gov’t Offc.	SF	12	47.5	25.3	2.13	221.0
7	Walnut Creek 10-St. Offc.	DWF	10	38.8	9.8	0.66	428.2
10	LA 9-St. Offc. Bldg.	DWF	9	38.8	15.2	1.25	267.6
11	LA 17-St. Resid. Bldg.	SW	17	39.6	23.2	0.85	362.7
12	LA 32-St. Offc. Bldg.	DWF	32	130.6	28.3	1.84	408.1
13	LA 54-St. Offc. Bldg.	SF	54	180.3	29.3	5.70	401.4
14	Whittier 8-St. Hotel	SW	8	20.9	19.5	0.49	256.6

^a Site classes “A” and “B” correspond to sites with or without a free-field accelerograph in the system identification study conducted by Stewart et al. (1999).

^b Lateral force resisting system: SW=masonry or concrete shear wall, DWF=dual wall/frame, CF=concrete frame, SF=steel frame.

^c Full height of structure derived from the effective height reported by Stewart et al. (1999) (i.e. effective height = 0.7 * full height).

^d Foundation radius matching the area of the actual foundation.

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Fig. 1. Model of the soil-foundation-structure system (modified after Luco, 1976a).

Fig. 2. Histograms of dimensionless parameters a_1 and I_b/I_s for all buildings listed in Table 1.

Fig. 3. Modulus of normalized relative twist $|a_r / u_{g0}|$ vs. normalized frequency ω / ω_1 for $a_1 = 0.2, 0.7, 1.2, 2.2, 3.2, 5.0$, $\theta = 0^\circ, 40^\circ, 80^\circ$, and $I_b / I_s = 0.1, 0.8, 1.7$. The soil density ρ_s , the mass density of the superstructure ρ_b , the foundation-to-superstructure mass ratio m_0 / m_b , and the hysteretic damping factor ξ are equal to their average values (i.e. $\rho_s = 1.95 \text{ Mg/m}^3$, $\rho_b = 250 \text{ kg/m}^3$, $m_0 / m_b = 0.2$, and $\xi = 4\%$).

Fig. 4. (a) Amplitude of first peak response of $|a\alpha_r/u_{g0}|$ vs. a_1 and θ for $I_b/I_s = 0.1, 0.8, 1.7$. (b,c) Cross sections of Figure 4a for $a_1 = 1.5$ and 4.5 . (d,e) Cross sections of Figure 4a for $\theta = 40^\circ$ and 80° .

Fig. 5. Normalized Fourier amplitude spectrum of ground displacement vs. normalized frequency for $\gamma = 1.0, 1.5, 2.0, 3.0$.

a

b

Fig. 6. Maximum relative twist between top and base of Buildings A10, A36 and A28 vs. T_p/T_1 for $\gamma = 1.0, 1.5, 2.0, 3.0$. The angle of incidence is considered to be $\theta = 40^\circ$ and 80° for (a) and (b), respectively. The dotted vertical line specifies the lower limit of the effective range of T_p/T_1 for each building.

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Fig. 12. Synthetic high-frequency (HF), long-period (LP) and broadband (BB) time histories of ground acceleration, velocity and displacement for three hypothetical earthquakes of M_w 5.8, 6.4 and 7.0 and the fault-station geometry shown in Figure 11.

Fig. 13. Maximum relative twist between top and base of all 40 buildings listed in Table 1 as a function of T_1 due to the high-frequency (HF), long-period (LP) and broadband (BB) ground motions shown in Figure 12. The pulse period T_p for each earthquake is indicated by a dotted vertical line, whereas $c_H = 2.0$ km/s and $\gamma = 2.0$.