Beneficial effect of soil-structure interaction to structural response derived from two building arrays in New Zealand

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ABSTRACT

The soil site spectra for site classes D and E in the current NZ design code impose a significant increase in the displacement demands, between 50 and 300% for a structure with a period of 1.0s or longer, compared with a rock site (class A/B) at the same location. However, for many structures on deep or soft soil sites, effects of soil-structure interaction may partly offset the site amplification by means of a filtering effect, and the rocking / translation of the building foundation relative to the free-field ground motion. In this study, results derived from 2 building arrays in New Zealand are presented to provide quantified reductions in the displacement demands for the structures.

1 INTRODUCTION

In a conventional approach for seismic analysis, the base of a structure is assumed to experience the free-field ground motion, i.e., the base has no relative motion to the free field. This assumption is valid for a structure located on a rock or very stiff soil site but not for a heavy or moderately heavy structure on a soft soil site. The motions of the foundation for a structure relative to the free-field are the effects of soil-structure interaction. There are two kinds of soil-structure interactions, namely the kinematic and the inertial interactions. Kinematic interaction refers to the averaging effect of a massless rigid body resting on or embedded in a soil site under a particular wave field. The kinematic interaction can be described by,

$$\mathrm{Re}[\mathrm{T_k}(\omega)] = \frac{\sin(\gamma_o \omega)}{\gamma_o \omega} \qquad \gamma_o = \frac{b}{V_a} \qquad \mathrm{Im}[\mathrm{T_k}(\omega)] = \gamma_1 \frac{\sin(\gamma_o \omega / 4)}{\gamma_o \omega}\left[\frac{\sin(\gamma_o \omega)}{\gamma_o \omega} - \cos(\gamma_o \omega)\right] \quad (1a,b,c)$$

where T_k is the Fourier spectral ratio between the motion of the rigid foundation and that of the free-field and ω is the circular frequency. Re[ ] indicates the real part, and Im[ ] indicates the imaginary part of the Fourier spectral ratio. Parameter γ_o can be interpreted as the ratio of the foundation length and the apparent velocity for seismic waves propagating in the horizontal direction along the foundation slab, and γ_1 is the amplitude scale factor for the imaginary part.

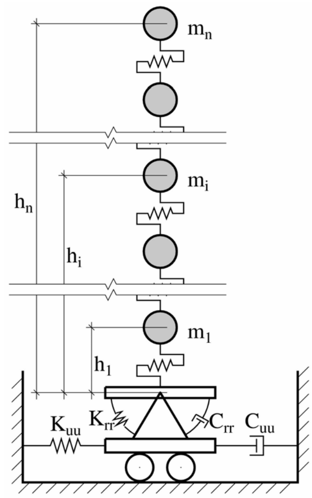

Figure 1: A soil-structure system of a multi-storey structure mounted on soil translation and rocking springs and dampers.

The inertial soil-structure interaction can be established by a simplified model (with a lumped mass for each storey) as shown in Figure 1, where the structure is mounted on translation and rocking soil springs and dampers with coefficients K_{uu}, K_{rr}, C_{uu} and C_{rr}. The subscript u denotes the translational direction and r denotes the rocking direction. The displacement of the structure at the equivalent height (2/3 of the total height for a uniform shear beam) can be derived as

$$\omega_h = \sqrt{\frac{K_{uu}}{M_e}} \qquad \omega_r = \sqrt{\frac{K_{rr}}{M_e h_e^2}} \qquad \xi_h = \frac{\omega C_{uu}}{2 K_{uu}} + \xi_g \qquad \xi_r = \frac{\omega C_{rr}}{2 K_{rr}} + \xi_g$$
$$\frac{1}{\omega_N^2} = \frac{1}{\omega_s^2} + \frac{1}{\omega_h^2} + \frac{1}{\omega_r^2} \qquad \frac{1}{\overline{\omega}_h^2} = \frac{1}{\omega_s^2} + \frac{1}{\omega_r^2} \qquad \xi_N = \xi_s \frac{\omega_N^2}{\omega_s^2} + \xi_h \frac{\omega_N^2}{\omega_h^2} + \xi_r \frac{\omega_N^2}{\omega_r^2} \qquad (2a,b,c,d)$$

where ω_s is natural frequency of the fixed-base structure, ξ_s is the material damping ratio of the structure, the ξ_g is the material damping ratio of the surrounding soil, ξ_h and ξ_r are the radiation damping ratios associated with the foundation translation in the horizontal direction and rocking of the foundation, respectively. The natural frequency of the soil-structure system, ω_N, is always less than ω_h and ω_r and the damping ratio of the soil-structure system, ξ_N, is usually of similar order for the damping ratio of the structure. The natural frequency of a soil-structure system without foundation translational motion is denoted by $\overline{\omega}_h$. The displacement of the structure relative to the base (the deformation of the structure that leads to inter-story-drift) can be solved approximately from the following equation (Wolf 1985, section 3.42),

$$\ddot{u}(t) + 2\xi_N \omega_N \dot{u}(t) + \omega_N^2 u(t) = \frac{\omega_N^2}{\omega_s^2} \ddot{u}_{gk}(t) \qquad \ddot{u}_{gk}(t) = \int_0^\infty T_k(\omega)\, \ddot{U}_g(\omega) e^{i\omega t} d\omega \qquad (3a,b)$$

where U_g is the Fourier amplitude of the free-field accelerations and $|T_k(\omega)|$ is always equal to or less than 1.0. Note that ξ_N is taken as an equivalent viscous damping ratio in Equation (3a). The dot and the double dots for displacement u stand for differentiation of u with respect to time, i.e., the velocity and acceleration in the time domain. Equation (3a) suggests that equivalent input ground motion is that of the filtered ground motion in Equation (3b) (always leading to a reduction in high frequencies) scaled down further by the squared ratio of natural frequency of the soil-structure system over that of the fixed-base structure. The benefits of the soil-structure interaction are evident in Equation (3).

2 BUILDING ARRAYS

One of the building arrays investigated here is the Gisborne Post Office building, a 6-storey reinforced concrete moment-resisting frame building with a 0.38m thick reinforced concrete raft foundation in the basement. The building has two bays in the N44E direction (16.5m) and 10 bays in the S46E direction (50.2m) and the building height above the mat foundation is 25.5m. The embedment depth of the foundation is about 2.4m. The building is located on a soft soil site with about 40m of recent marine and fluvial sediments above a siltstone base. The shear wave velocity of the site has been measured as 98 m/s near the surface, gradually increasing to 240 m/s at a depth of 24m. Three instruments are located inside the building almost in the centre of different levels of the building. A carpark (free-field) site is about 20m away from the building. Two complete sets of earthquake records were obtained in 1982 and 1993. The 1982 earthquake had a local magnitude of 5.1 and an assigned depth of 33 km. The epicentral distance to the Gisborne Post Office was 25 km and the epicentral direction at the building site was in N56W. A maximum horizontal acceleration of 0.09g was recorded at the top floor, 0.04g at the ground floor and 0.06g at the carpark site. The 1993 (Ormond) earthquake was centred at an epicentral distance of 23 km from Gisborne Post Office, and had a moment magnitude of 6.2 a focal depth of 37 km. A maximum horizontal acceleration of 0.25g was recorded at the top floor, 0.14g at the ground floor and 0.26g at the carpark. The building and the soil responded essentially elastically in the 1982 earthquake but nonlinear response possibly occurred in the 1993 earthquake because of apparent reduction in natural period and stiffness (Zhao 1998b).

The Vogel building is a 17-storey reinforced concrete structure in central Wellington, with two storeys embedded in soil. The basement is fully embedded and the lower ground floor is partially embedded. The average embedment depth is about 5-6m. The foundation length is

25.6m in the N02E direction and 32.9m in the N88W direction. The building height from the top of the foundation slab to the roof slab is 59.75m. It has shear walls, the thickness of which reduces slightly with height and there is an additional reinforced concrete stiffener at both the basement and the lower ground floor. The soil comprises completely weathered terrace gravels with a site class D in the current New Zealand loadings code. The topography around the building site is quite complex and the ground surface has a gentle slope in the north-south direction. Soon after construction the Vogel building was instrumented with four strong motion accelerographs. In the 1973 Central North Island earthquake, records were obtained from three of the instruments, those located in the basement, the 14th floor of the building and a carpark (the free field) about 50 m away from the basement instrument. In the 1977 Cook Strait earthquake a complete set of four records was obtained. The 1973 earthquake occurred at a distance of 240km from Wellington with a depth of 173km and moment magnitude of 6.6. A maximum acceleration of 0.10g was recorded at the 14th floor, 0.03g at the basement and 0.04g at the carpark. The 1977 M_w=6.0 earthquake was centred 64km away from Wellington at a depth of 33km. A maximum acceleration of 0.17g was recorded at the 14th floor, 0.07g at the basement and 0.08g at the carpark. The effect of high-frequency attenuation of the foundation motion was reported by McVerry (1984).

3 KINEMATIC INTERACTION

Figure 2 shows transfer functions, which are the Fourier spectral ratios between the translation motion of the foundations and the free-field motions obtained from the carparks for the Gisborne Post Office building and the Vogel building, for two sets of earthquakes. The spectral ratios at frequencies over 6 Hz are smaller than 0.5, i.e., the ground motion amplitudes in the Frequency domain at the foundation slab are less than 50% those in the free field. At low

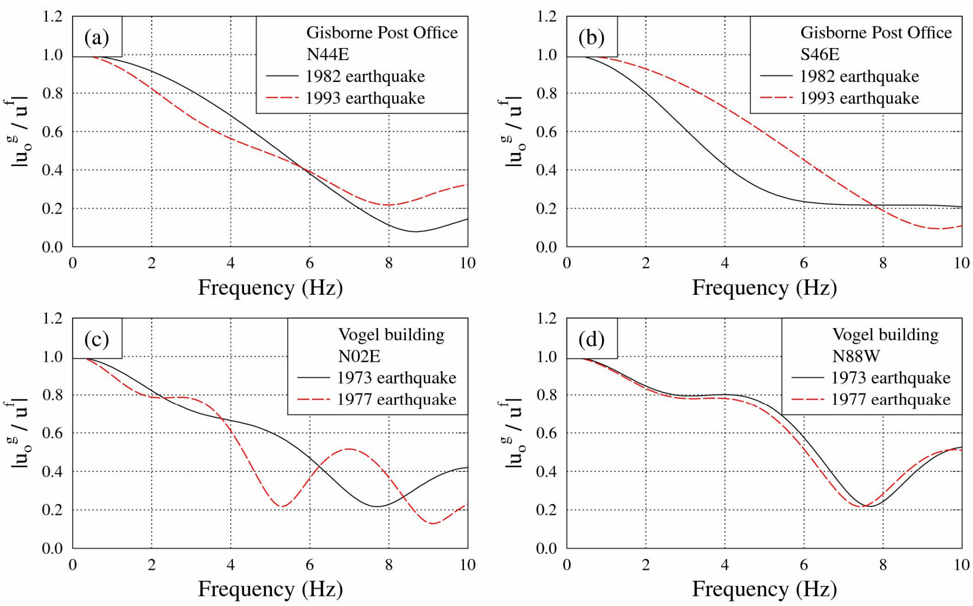

Figure 2: Transfer function derived from recorded accelerograms by using system identification method (Zhao 1998), for the Gisborne Post Office building, (a) N44E, (b) S46E, for the Vogel building, (c) N02E and (d) N88W

frequencies, such as 1.0 Hz, the Fourier spectra of the foundation motion are similar to those of the free-field. The transfer function is a function of foundation dimensions and the propagation velocity of seismic waves in the direction along the foundation slab (Equation 1). The transfer functions differ moderately for ground motions from different earthquakes, but their general trends in the variation with increasing frequency are very similar. The attenuation of high-frequency motions shown in Figure 2 reduces the high-frequency floor accelerations of the building and reduces potential damage to non-structural elements and contents housed by the building. For frequencies up to 2Hz, the effect of kinematic interaction has little effect on the foundation motions and therefore will have little effect on the building base shear and overturning moment for both buildings (Zhao 1998).

Figure 3 compares the response spectra of the ground motions at the free-field and at the building basement locations. The high-frequency attenuation of the foundation motion due to kinematic interaction is evident, with the short period peak spectra of the foundation motion being reduced to nearly 50% those of the free-field ground motion. At periods over 0.6s, the response spectra of the foundation and free-field motions are very similar.

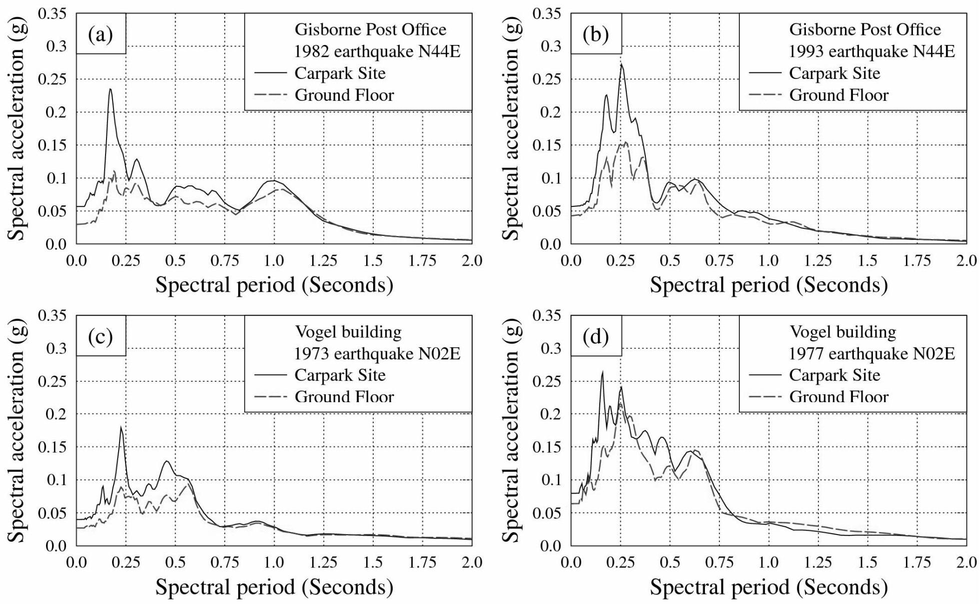

Figure 3: Comparison of response spectra at free field and building basement

4 INERTIAL INTERACTION (ROCKING AND FOUNDATION TRANSLATION)

When a building at a deep/soft soil site experiences a seismic excitation, the foundation of the building will translate and rock relative to the free field ground motion. The structural displacement due to foundation translation and rocking do not induce inter-storey drift and therefore do not need to be accounted for in the design of a structure. However, the displacement due to the foundation motion relative to the free field is part of the displacement demand of the free-field motions. In another word, the foundation motion (including rocking) relative to the free-field motion effectively reduces the damaging displacement demand on the structure as shown in Equation (3a) where the effective excitation is the free-field ground motion scaled by ω_N^2 / ω_s^2.

Table 1 shows the natural frequencies, damping ratios and the frequency ratios. Note that the natural frequency of the soil structure system was reduced from 1.28Hz in both directions for the four records discussed above in the 1982 event to 0.93Hz in the N44E direction and to 1.0Hz in the in the S46E direction in the 1993 event, suggesting possible nonlinear responses in the structure and the surrounding soils because of strong shaking. The foundation translation frequencies derived from the 1993 event are less than 0.5 times those in the 1982 event, a possible result of soil nonlinear response. The values derived for the Vogel building possibly have a considerable error band as the signal/noise ratio in the basement records may be lower than 2.0 (Zhao 1998a).

The distribution of the accelerographs in both buildings does not allow estimation of the foundation rocking frequencies and it is not possible to derive the fixed-base natural frequency of the buildings. Zhao (1998b) estimated the ratio between K_{rr} and K_{uu} for the Gisborne Post Office Building and the fixed-base natural frequency in a range of 1.35-1.4Hz in the N44E direction for the 1982 event, 1.1-1.2 Hz in the same direction for the 1993 event. The reduction of the natural frequencies for the fixed base building is intuitively consistent with the expected structural nonlinear response. Another method for estimating the upper bound of the fixed base natural frequency is to use the deep trough in the transfer functions between the recorded foundation and the free-field motions, as illustrated in Figure 4 where the frequency of the first trough (after the natural frequency ω_N) is 1.24Hz. For an ideal model that has a massless rigid foundation, the frequency of this trough is $\overline{\omega}_h$. However, for a model with a base mass, the trough tends to occur between the fixed base frequency ω_s and $\overline{\omega}_h$, thus 1.2Hz would be an adequate estimate for the fixed-base building in the N44E direction in the 1993 event. The scale factor to compute the displacement demand ω_N^2 / ω_s^2 in Equation (3a) is 0.6, i.e., the actual damaging displacement demand for the Gisborne Post Office building during the 1993 earthquake is only 60% that of the displacement demand of the carpark record in the N44E direction. In the S46E direction, foundation rocking is negligible because of the large foundation length in this direction. The natural frequency of the fixed-base structure is very close to the natural frequency of the soil-structure system. In the 1982 event, ω_s was reported to be 1.34-1.4Hz (Zhao 1998b) leading to ω_N^2 / ω_s^2=0.8-0.9, suggesting that the inertial interaction is negligible, possibly because the extent of soil nonlinear response is small in the 1982 event.

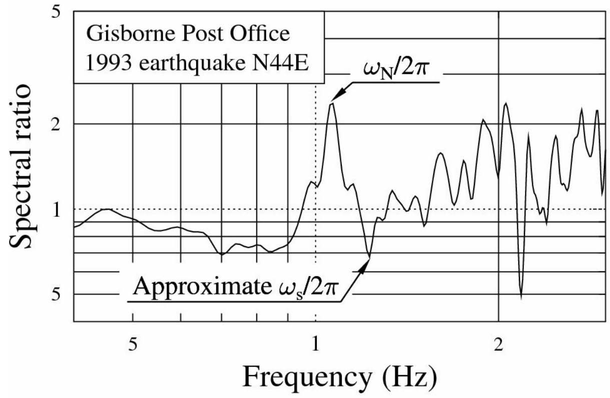

Figure 4: Transfer function between motions of foundation and free field

Table 1: Modal parameters derived from earthquake records

Gisborne Post Office building				
	1982 Earthquake		1993 Earthquake	
	N44E	S46E	N44E	S46E
$\omega_N /2\pi$ (Hz)	1.28	1.28	0.93	1.00
ξ_N (%)	6.38	4.13	5.28	5.36
$\omega_h /2\pi$ (Hz)	5.8	6.7	2.8	3.2
Vogel building				
	1973 earthquake		1977 earthquake	
	N88W	N02E	N88W	N02E
$\omega_N /2\pi$ (Hz)	1.29	1.19	1.26	1.16
ξ_N (%)	2.66	1.31	3.35	2.70
$\omega_h /2\pi$ (Hz)	5.6	6.4	4.4	6.9

Note: ω_h is derived from the effective modal participation factor from Zhao (1998b)

Because of the possible low signal/noise ratio of the basement records from the Vogel building, the fixed-base frequencies will not be estimated here.

A final note is that the displacement demand that causes inter-storey drift cannot be computed by using the ground motions recorded at the foundation of a structure and a conventional structural model that does not allow rocking motion of the roundation. In this case, the horizontal structural displacement that cause structural inter-storey displacement can be computed from

$$\ddot{u}(t) + 2\,\bar{\xi}_h\,\bar{\omega}_h\,\dot{u}(t) + \bar{\omega}_h^2\,u(t) = \frac{\bar{\omega}_h^2}{\omega_s^2}\,\ddot{u}_o(t) \qquad \bar{\xi}_h = \xi_{\mathrm{s}}\frac{\bar{\omega}_h^2}{\omega_{\mathrm{s}}^2} + \xi_{\mathrm{r}}\frac{\bar{\omega}_h^2}{\omega_{\mathrm{r}}^2} \qquad (4)$$

where $\ddot{u}_o$ is the recorded accelerations in the horizontal direction. Note that $\bar{\omega}_h$ is always less than or equal to ω_s. Zhao (1998b) reported $\bar{\omega}_h/2\pi$=0.97 Hz for the Gisborne Post Office building for the N44E component during the 1993 earthquake and the scale factor $\bar{\omega}_h^2/\omega_s^2$=0.65 can be derived. The scale factor $\bar{\omega}_h^2/\omega_s^2$ is only marginally larger than ω_N^2/ω_s^2, suggesting that the major foundation motion is the rocking in the N44E direction.

5 CONCLUSIONS

The modal parameters of the Gisborne Post Office building and the Vogel building in Wellington are presented. Effects of kinematic soil-structure interaction (filtering of free-field ground motion by the foundation slab) and inertial interaction (translation and rocking of the foundation relative to the free field) are significant. At short periods, the kinematic interaction reduces the short period motions of the foundation significantly and the short-period floor accelerations for the buildings, with a potential benefit to reduce the short period response of non-structural elements and the contents housed by the building during very strong ground shaking.

During the 1993 earthquake, the inertial interaction for the Gisborne Post Office building in the N44E direction reduces the displacement demand (that causes inter-storey drift) by as much as 40% of the free-field records. The rocking motion of the foundation in the N44E direction is identified as the major component of the inertial interaction. The soil nonlinear response is also a possible factor for the stronger inertial interaction in the 1993 earthquake for this building. The extent of the inertial interaction is proportional to the squared frequency ratio between the natural frequencies of the soil-structure system and that of the fixed-base building and the soil nonlinear response may lead to the decrease of the soil-structure system. Therefore the effect of inertial interaction may increase with increasing shaking level even when significant nonlinear response developed in the structure.

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