Discrete Model for Dynamic Structure-Soil-Structure Interaction

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SUMMARY:
A simple discrete model is used to treat a problem of dynamic through soil structure to structure interaction via an analytical 2-D formulation that is then numerically solved in time domain. The model includes a frequency independent rotational spring as a key buildings interaction mechanism. Insight into the influence of the geometrical characteristics of the structures and distance between them on the dynamic structure-soil-structure interaction effects is examined.

Keywords: Structure-Soil-Structure Interaction, Discrete Models.

1. INTRODUCTION

The study of the dynamic interaction between several structures with consideration of the underlying or surrounding soil is receiving some considerable attention in recent years, Menglin *et al* (2011). Although experimental in situ investigations, Kitada *et al* (1999), Yano *et al* (2003) and Hans *et al* (2005), provided qualitative evidence of the dynamic interaction effects between adjacent structures, the studies of the dynamic structure-soil-structure interaction phenomena have been explored analytically and also numerically based either on finite element (FE) or boundary element (BE) methods or on coupled FE/BE procedures, for example MacCalden (1969), Luco and Contesse (1973) and Wang and Schmid (1992). The structure-soil-structure interaction analysis of several shear walls erected on an elastic, homogenous half-space conducted by Wong and Trifunac (1975) showed that the scattering, diffraction and interference of waves from and around several foundations with the incident SH waves can lead to significant effects if the structure of interest is smaller and lighter than its neighbours. The complexity of the multi-structural interaction problem has also been emphasised by some recent numerical studies , for example Wirgin and Bard (1996), which showed that the vibration of structures radiates diffracted wave fields into the soil with amplitudes than can be important when heavy structures rest on stratified soft soils and have the same frequency of vibration as the soil. The radiated wave field is energetic enough to be detected up to a distance of 10 times the foundation length and the seismic ground motion may thus be contaminated by the complex contributions from all the buildings, Guéguen, *et al* (2002). In spite of their applicability to complex configurations, the numerical calculations may nonetheless obscure insight into the problem and preclude parametric studies. Therefore, an alternative approach could be based on approximate discrete models. This is particularly attractive for analyses of dynamic interaction between large numbers of buildings typically found in densely populated cities. Extensive research has produced a large variety of approaches for the evaluation of the discrete system constants, Barkan (1962), Lysmer and Richart (1966), Richart *et al* (1970), Gazetas (1991), Wolf and Meek (1993) and Wolf (1994). Among others, Mulliken and Karabalis (1998) illustrated that this kind of modelling can be successfully applied not only to the evaluation of linear but also non-linear systems of massive adjacent surface foundations supported by homogeneous, isotropic, linear elastic half-space.

In this paper, a simple discrete model is used to treat a problem of dynamic through soil structure to structure interaction via an analytical 2-D formulation that is then numerically solved in time domain. Building and the soil are taken to act linearly and elastically and only soils of loose sand have been considered. A study done by Alexander *et al* (2012) has shown that building resting on this type of sand would experience the worst effect of the interaction. Two cases of excitation have been considered that are an artificial Kanai-Tajimi accelerogram and a horizontal ground motion of Westmorland earthquake. The influence of different geometrical characteristics of the structures and distance between them on the dynamic coupling interaction effects is then investigated.

2. THEORY

For this study a case of two buildings is considered. The discrete model is shown in Fig. 2.1; buildings and soil underneath them are coupled by a rotational interaction spring κ . Each structure-soil system is a two degree of freedom (dof) system with one translational dof at the building level (i.e. $x_1= r_1u_1$ and $x_2= r_2u_3$) relative to the ground translation $x_g= r_1u_g$ and one rotational dof at the foundation level (i.e. $\theta_1= u_1$ and $\theta_2= u_2$). All system dofs are non-dimensionalised. m_1 and m_3 are masses; k_1 and k_3 are stiffnesses of building 1 and 2 respectively. $m_2r_1^2$ and $m_2r_2^2$ are polar moments of inertia; k_2 and k_4 are rotational stiffnesses of foundation-soil beneath each building and $r_{1,\,2}$ are radii of gyration of soil semi-cylinders. Using Lagrangian energy formulation a non-dimensional equation of motion of the coupled system is derived.

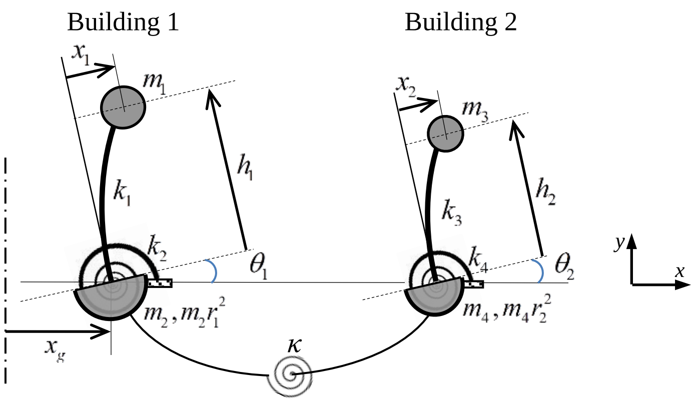

Figure 2.1. Two buildings discrete model

The scope of this analysis is restricted to a special case, Fig. 2.2, by assuming the following:

- i- same loose sand soil profile exists under both buildings, i.e. average soil density ρ_s is identical 1300 kg/m^3.
- ii- both buildings have an identical square plan area of b
- iii- both buildings have the same average density, ρ_b based on typical span and floor loadings and is taken 600 kg/m^3
- iv- buildings can be of different heights, $h_{1,\,2}$
- v- buildings are spaced at some arbitrary distance from each other, zb
- vi- volume of soil mass equals $0.35b^3$ for a square base building of width b which is equivalent to a semi-cylinder of radius $0.47b$, based on Newmark and Rosenblueth (1971).

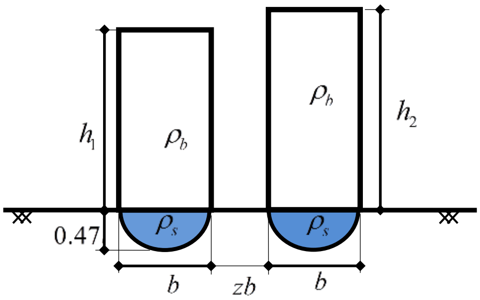

Figure 2.2. Parametric study

The Euler-Lagrange equation of motion expressed in matrix form is as follows

$$\begin{bmatrix} 1 & -3s & 0 & 0 \\ -3s & c_1 s^{-1}+9s^2 & 0 & 0 \\ 0 & 0 & \varepsilon & -3\varepsilon^2 s \\ 0 & 0 & -3\varepsilon^2 s & c_1 s^{-1}+9\varepsilon^3 s^2 \end{bmatrix} \begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \\ \ddot{u}_4 \end{bmatrix} + \cdots$$

$$\cdots + \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & c_1 c_2 q_2 s \bar{V}_s^2 \left(1+q_k\right) & 0 & -c_1 c_2 q_2 q_k s \bar{V}_s^2 \\ 0 & 0 & \varepsilon^{-1} & 0 \\ 0 & -c_1 c_2 q_2 q_k s \bar{V}_s^2 & 0 & c_1 c_2 q_2 s \bar{V}_s^2 \left(1+q_k\right) \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \\ u_4 \end{bmatrix} = \begin{bmatrix} -1 \\ 3s \\ -\varepsilon \\ 3\varepsilon^2 s \end{bmatrix} \ddot{u}_g \qquad (2.1)$$

Eqn. 2.1. is the nondimensional equation of motion of the coupled system and the Newtonian dots signify derivatives with respect to a time-scale $\tau = \omega_1 . t$ where ω_1 is the fixed base natural frequency of building 1. There are four key parameters in Eqn. 2.1. namely, (i) height ratio $\varepsilon = h_2/h_1$ (building 2 to 1); (ii) aspect ratio $s = h_1/b$; (iii) normalised inter-building distance ratio z and *(iv)* soil class (dense, medium or loose). c_1 and c_2 are soil type coefficients written as $c_1 = 0.35(\rho_b / \rho_s)$ and $c_2 = 655(1-\mu)$ where μ is Poisson's ratio . $\bar{V}_s$ is normalised shear wave velocity of the soil. The analysis conducted by Alexander *et al* (2012) showed that an inverse power functional relationship exists between the rotational foundations springs k_2 and k_4 and rotational foundation to foundation interaction spring κ with the inter-building spacing z for any foundation geometry b as follows

$$\kappa = q_\kappa(z) k_2, \quad k_2 = q_2(z) k_S, \quad q_\kappa(z) = \frac{c_3}{(z+1)^3}, \quad q_2(z) = 1 + \frac{c_4}{(z+1)^3} \qquad (2.2)$$

k_s is the soil/foundation rotational spring stiffness in the absence of buildings interaction , Barkan (1962). As κ, k_2 and k_4 are dependent on the same soil properties the coefficients $c_3 \approx -1/4$ and $c_4 \approx 1/2$ are generally independent of soil type.

3. TIME DOMAIN ANALYSIS

Introducing orthogonal damping that is based on a Caughey damping model, Clough and P. J (1993) and Chopra (2000). This is based on all four eigenvalues of the linear unforced system, Eqn. 2.1. Each mode is taken to be damped at 5% of critical damping: thus, Eqn. 2.1. in its general form becomes

$$\mathbf{M}\ddot{\underline{\mathbf{u}}} + \mathbf{C}\dot{\underline{\mathbf{u}}} + \mathbf{K}\underline{\mathbf{u}} = \mathbf{p}\ddot{\underline{u}}_g \qquad (3.1)$$

The second order ODE system in Eqn. 3.1. is solved as a state space model in time domain using the classical Runge-Kutta method with a single integration step and converted to a MATLAB routine. The system is first analysed for an artificial input time history then for excitation of ground motion of an actual earthquake.

The excitation input in the first case is assumed to take the form of generalized stationary Kanai-Tajimi model, Kramer (1996), of an eigen frequency ω_g = $2\pi 3.8$ rad/sec and damping ratio ξ_g = 0.34 and corresponds to Alluvium sites, Fig. 3.1. A horizontal component record of the Westmorland earthquake (California 1981), Fig. 3.2., of magnitude M_L = 5.8 has been obtained from PEER Strong Motion Database (2000), for weak soil conditions which correspond to sites of an average shear wave velocity of less than $180\,m/s$.

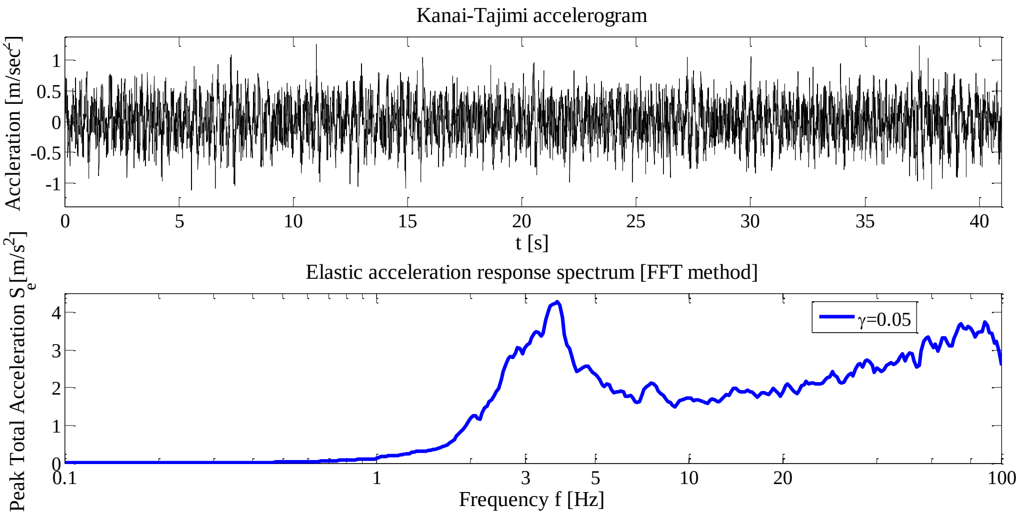

Figure 3.1. Kanai Tajimi artificial accelerogram and elastic acceleration response spectrum (5% damping).

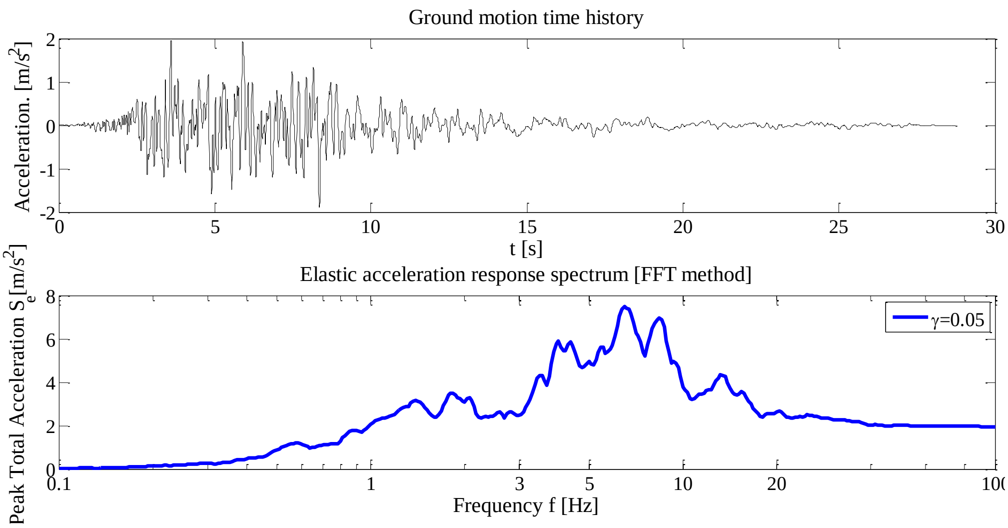

Figure 3.2. Ground motion time history (horizontal component) and its elastic acceleration response spectrum – Westmorland earthquake, PGA=1.95 [m/s²].

The total sway displacement at the top of buildings can be expressed as r_1u_1-h_1u_2=r_1v_1 and r_2u_3-h_2u_4=r_2v_2 where the non-dimensional total sway displacements (sway + rotation) are v_1 and v_2 for buildings 1 and 2 respectively. Based on Newmark and Rosenblueth (1971) $r=0.33b$, hence

$$v_1 = u_1 - 3s\,u_2 \quad \text{and} \quad v_2 = u_3 - 3s\varepsilon\,u_4 \tag{3.2}$$

In order to target the most adverse response, natural frequency of building 1 ω_1 is taken to coincide with the elastic response spectrum peak frequency of the Kanai-Tajimi spectrum, $\omega_g = 2\pi 3.8$ rad/sec, in the first case and of the Westmorland earthquake, $\omega_w = 2\pi 6.48$ rad/sec, in the second case.

4. DISCUSSION

After Fourier transforming v_1 and v_2, Fig. 4.1. and Fig. 4.2. present the response power spectra for buildings 1 and 2 for Kanai-Tajimi and Westmorland earthquake respectively. This corresponds to a specific set of system parameters displayed in the figures; a building of aspect ratio s =3, height ratio ε =1.1 (i.e. building 2 is 1.1 times building 1's height) and the normalised inter-building distance ratio z = 0.1 (i.e. the buildings are very close to one another).

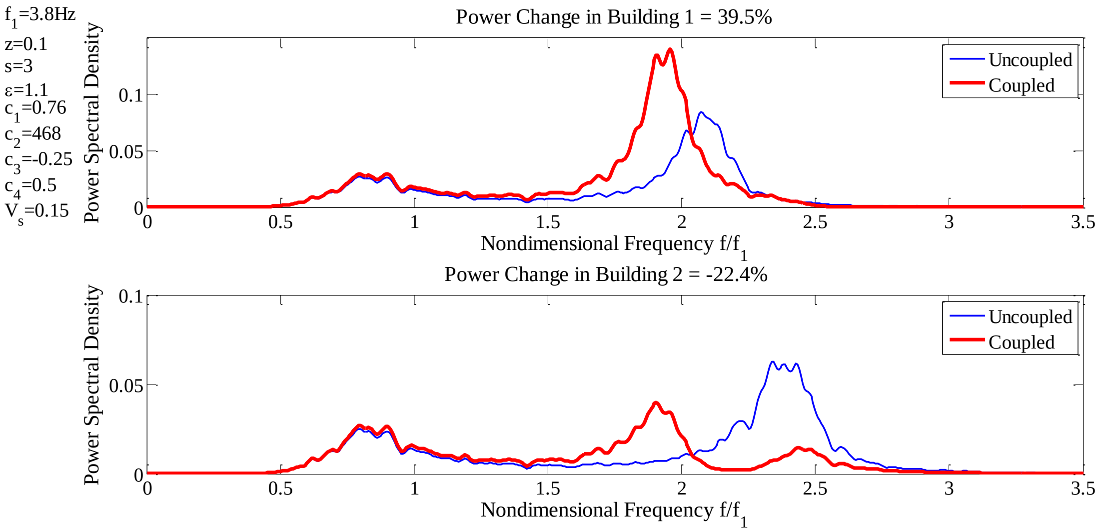

Figure 4.1. Response power spectra for total sway displacements for buildings 1 and 2, due to Kanai-Tajimi excitation.

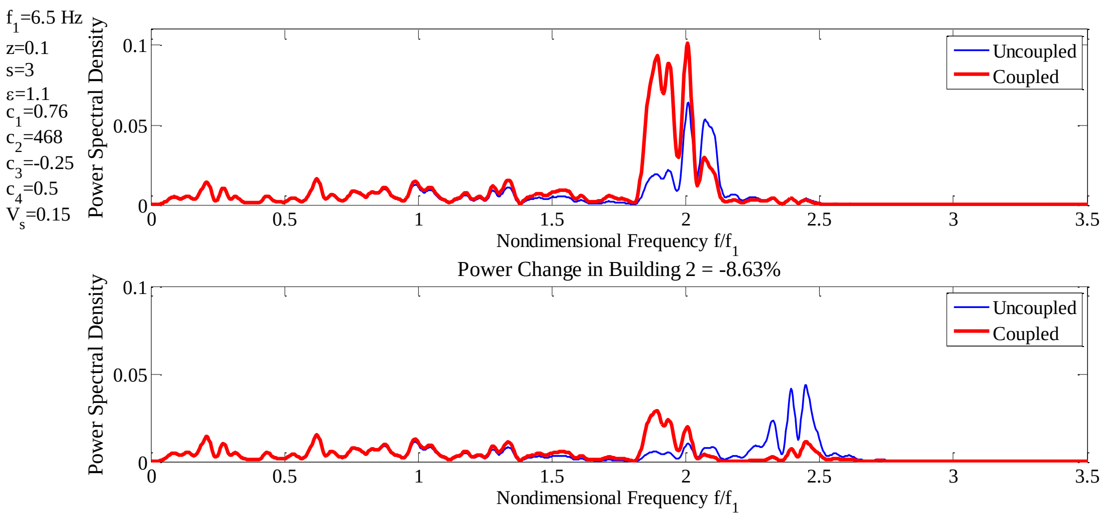

Figure 4.2. Response power spectra for total sway displacements for buildings 1 and 2 due to Westmorland earthquake.

For the aforementioned set of system configurations, it is seen that for both cases of excitation building 1 increases its response and building 2 reduces its response. It is possible to integrate the area under these curves to obtain the total response power of each building for the coupled and uncoupled cases. Then the percentage change in total response power when moving from uncoupled to coupled states can be calculated. For both loading cases respectively, it can be observed that building 1's total response power increases by about 39 % and 47% and building 2's reduces by almost 22% and 8%. It is noted that constructing a second slightly taller building 2 next to an existing building 1 seems to cause the earthquake power to be passed from the taller structure to the shorter one, Alexander and Schilder (2009) and Gourdon *et al* (2007). In other words, building 1 behaves like a tuned mass damper for building 2. It would be interesting to explore how the response power change varies with aspect ratio *s*, height ratio ε and inter-building spacing *z*.

Each of Fig. 4.3. and Fig. 4.4. displays a contour plot of spectral power change of building 1. The critical zones are reds, i.e. where building 1's total response power is amplified by the presence of building 2. Fig. 4.3. shows that the worst possible building parametric configuration resulted from the Kanai-Tajimi loading lies around s = 0.75 and ε =1.25. In this case the second building is 25% taller than the first and the maximum power gain is about 51%. Fig. 4.4. repeats the previous analysis for the case of Westmorland earthquake. In this case the worst parametric configuration is similarly at ε =1.25 but at an aspect ratio of s = 3.25 with maximum power gain of about 63.5%. Retrospectively, in Fig. 4.3. it is seen that for s = 3.25 and ε =1.25 (i.e. parameters that produced the maximum power change due to Westmorland), the power change because of coupling is still high, about 42.4%.

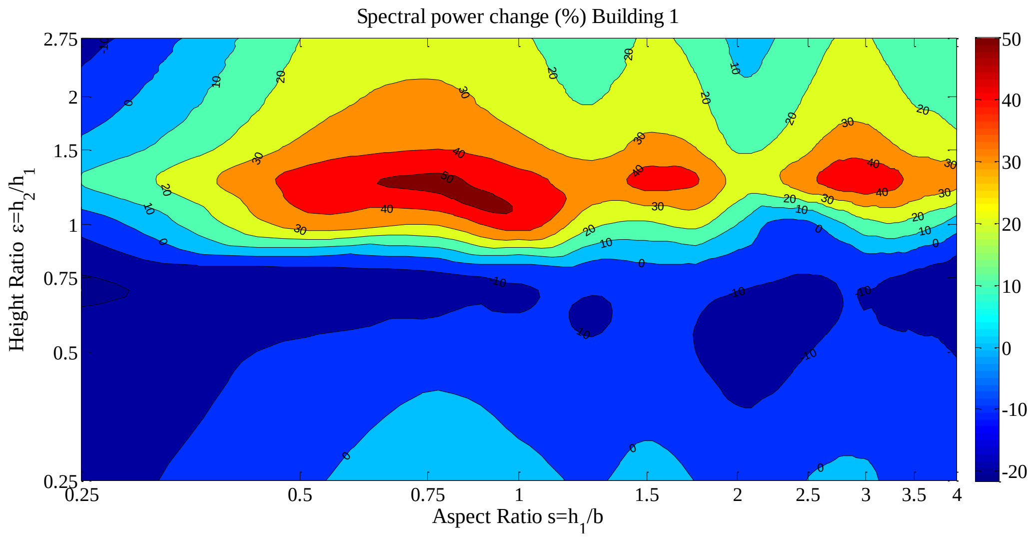

Figure 4.3. Change in power (caused by coupling) vs. aspect and height ratio caused by Kanai-Tajimi.

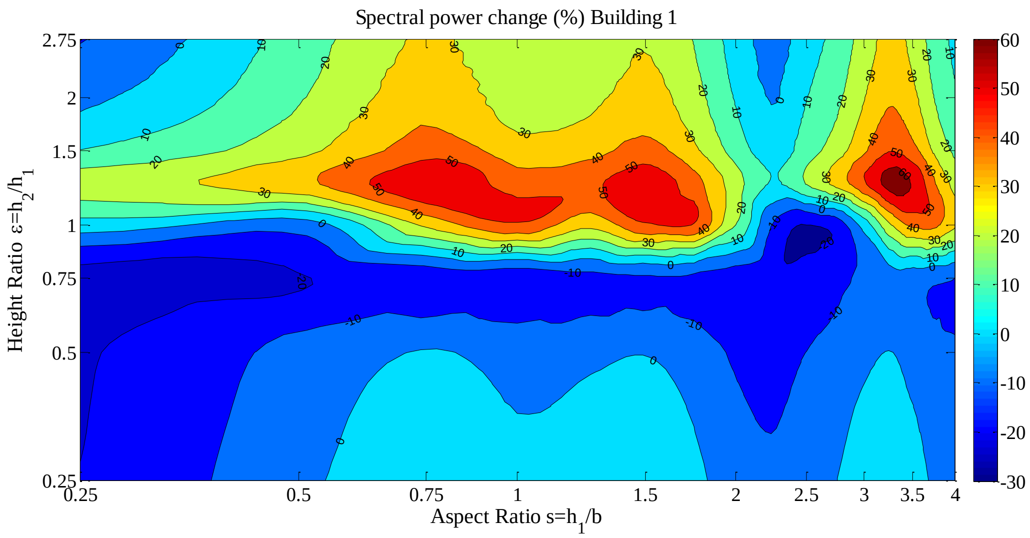

Figure 4.4. Change in power (caused by coupling) vs. aspect and height ratio caused by Westmorland.

Fig. 4.5. and Fig. 4.6. respectively display the variation of power with height ratio ε and inter-building spacing z with fixed aspect ratio s at 3.2. It is obvious that the worst interaction occurs when the building are very closely spaced. At a maximum distance of about 1.7b the worst power change effect has reduced to an insignificant value of 5%.

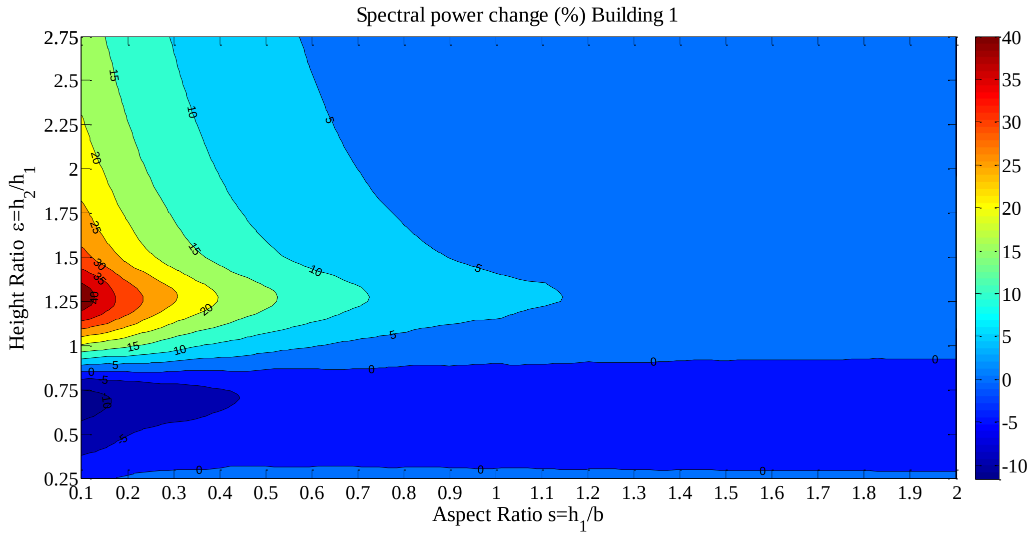

Figure 4.5. Change in power (caused by coupling) vs. height ratio ε and inter-building spacing z, s=3.25 - Kanai-Tajimi

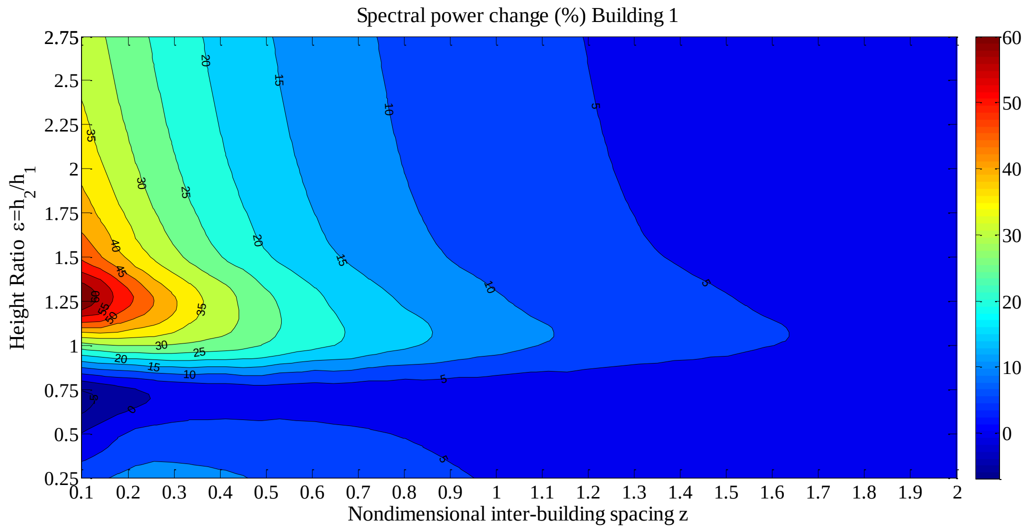

Figure 4.6. Change in power (caused by coupling) vs. height ratio ε and inter-building spacing z, s=3.25 - Westmorland.

5. CONCLUSIONS

An analytical formulation using simple discrete model has been used to model the dynamic 2-D structure-soil-structure interaction problem and has been treated in time domain. Two cases of excitation have been considered. Even though the real earthquake excitation has more rich frequency content than the artificial excitation, the dynamic behaviour of interacting buildings is seen to be qualitatively comparable.

The analyses have been undertaken for loose soils only. Results showed that constructing a second somewhat taller building next to an existing is generally adverse. In this case, construction of the new building increases the seismic risk to the existing one while reducing its own seismic risk. It also appears that the smaller building can act like a tuned mass damper for the larger building. Tall and thin buildings of higher aspect ratios are susceptible to a greater risk. Results also suggest that there is a favourable geometric configuration where risk is reduced, for the existing building, by the construction of a new contiguous building. Results also indicate that a range of -20% to +60% change in spectral power is possible.

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