THE SIGNIFICANCE OF SITE RESPONSE EFFECTS ON PERFORMANCE BASED DESIGN

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SUMMARY

This paper illustrates how site response effects influence the design forces when using performance based design. This is done by showing how the relative force levels vary between the various performance design requirements. Two cases for sites on rock, one in a region of low to moderate seismicity the other in a region of high seismicity are discussed initially. This situation is then used as a reference for comparison when considering other soil conditions. To illustrate how the performance design objectives are influenced by site response effects three soil profiles are used, two being dense sand sites, typical of Class D as defined by UBC (1997) and one being a soft clay site, typical of Class E. We have calculated the effects of site using bedrock ground motions matching three return periods specified in FEMA-273 (1997). These are the 2%, 10% and 50% chance of being exceeded in a 50 year design life corresponding to 'no collapse', 'life safety' and 'immediate occupancy' respectively. In this way the influence of site response on the design process when using performance based design methods is clearly illustrated.

For the rock and the soil sites conforming to Class D the results are as expected with more stringent 'no collapse' force levels being significantly larger than the 'immediate occupancy' force levels. The Class E site however shows that 'no collapse' ground motion is similar in regions of low to moderate seismicity to that of regions of high seismicity. It follows the design of structures on Class E sites in highly seismic regions will often be controlled by the 'immediate occupancy' requirements.

INTRODUCTION

Performance based earthquake design relies on explicitly considering the requirements of the structure when subjected to future earthquake events. For example the performance design code FEMA 273 (1997) requires that life safety must be ensured for occupants of a structure subjected to an earthquake ground motion with a likelihood of a 10% chance of being exceed in a 50 year design life. It must also be demonstrated that the building will still be suitable for immediate occupancy after experiencing an earthquake ground motion with a likelihood of a 50% chance of being exceeded in a 50 year design period. Furthermore, it must not collapse after experiencing an earthquake ground motion with a likelihood of a 2% chance of being exceeded in a 50 year design period. The life safety requirement is the same as that stated in the current operational codes, such as UBC (1997) but the other criteria are additional to what is generally considered by designers at present.

The UBC (1997) has gone a long way to identifying the effects of local soil conditions causing site response effects and shows how these are likely to effect the seismic base shear requirements when considering the life safety design objective. The factors given in UBC (1997) show that as the level of earthquake acceleration increases that soil effects become more non-linear and the amplification effects reduce especially for structures with low fundamental periods. Therefore, for short period structures on soft soil sites and in areas of high seismicity, the adoption of performance based design criteria will mean the design may well be controlled by the operational requirements, since these will give rise to seismic base shears which are similar to those currently associated with life safety requirements.

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OBJECTIVES AND METHODOLOGY

The objective of this paper is to illustrate how site response effects will influence the design forces when using performance based design. This done by showing how the relative force levels vary between the various performance design requirements. Three sites have been selected for this illustration and two sets of input ground motion have been used. One set of input motion is appropriate to Japan to represent a region of high seismicity and the other set is appropriate to Hong Kong to represent a region of low to moderate seismicity. Each set comprises three artificial time histories which match uniform hazard response spectra conforming to the likelihood of 2%, 10% and 50% chance of being exceeded in a 50 year design life respectively.

In this study the same three sites are used for each set of input ground motion. The sites range from dense sand to soft clay and are modelled as one-dimensional profiles assuming non-linear soil behaviour. Time domain site response analyses are carried out to determine the ground motion at the surface and their corresponding response spectra. These are used to illustrate the effects of site response on performance based design both for different types of soil profile and different levels of input seismic ground motion.

DESCRIPTION OF SITES

Three sites A,B and C are included in this study, with properties as given in Table 1. Two sites, A and B, are dominated by extensive sand deposits, whereas Site C mainly comprises normally consolidated clay of low plasticity. Figure 1 provides the depth profile of small strain shear module (G_0) for the three sites. The soil properties for the sites are obtained from the references listed in Table 1.

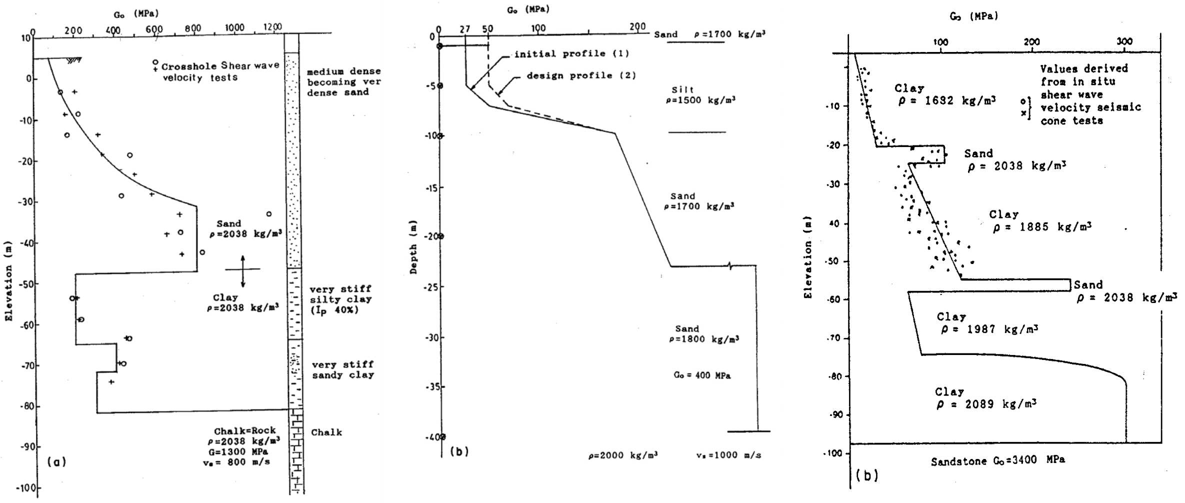
Table 1: Description of Sites

Site	Location Reference	Depth (m)	Period (s)	Description	UBC 30m Shear Wave Velocity (m/s)
A	United Kingdom (Evans et al, 1987)	82	0.95	48m of dense sand over 34m of stiff clay overlying rock	311
B	Chiba. Japan (Katayama and Sato, 1988)	40	0.49	1m of fill over 9m of silt over 30m of fine sand	295
C	United Kingdom (Papastamatiou et al, 1993)	97	2.21	20m of normally consolidated soft clay over 4m sand over 30m stiff clay over 3m sand over 16m stiff clay over 24m boulder clay overlying rock	114

Table 1 also shows the representative shear wave velocity for the upper 30m of the profile calculated using the procedure recommended in UBC (1997). It is seen that sites A and B would both clarify as Class D whereas Site C is a Class E site.

SOIL PROPERTIES

In addition to the soil bulk density, the one-dimensional dynamic site response analysis discussed herein requires a knowledge of the soil shear stiffness and energy dissipation or damping characteristics. Figure 2 shows a variety of G/G_0 (the ratio of the shear modulus G at any particular shear strain amplitude to the small-strain shear modulus G_0) vs. shear strain amplitude curves that have been used for the various materials in this study. Further details of the derivation of the soil properties for these three sites are included in Heidebrecht et al (1990) and Henderson et al (1990).



Site A Site B Site C

Figure 1: Low Strain Shear Modulus Profiles

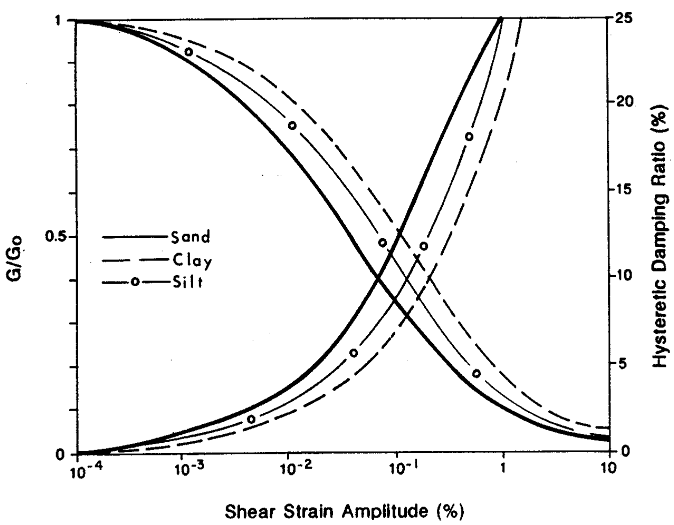


Figure 2: Shear Modulus and Damping Relationships

MODELLING OF SOIL LAYER SYSTEMS

The soil deposits are modelled as one-dimensional layered systems with propagation of shear waves only in the vertical direction. The computer program for the non-linear model, known as SIREN, developed at Ove Arup & Partners (Henderson et al., 1989) solves the problem in the time domain using the explicit finite difference method. The soil is represented as a series of lumped masses connected by non-linear springs. The non-linear springs are formulated as an assemblage of elastic-plastic springs and exhibit hysteretic damping in accordance with the Masing principles (Pyke, 1979). No viscous damping is included in this model. Transmission of energy at the rock-soil interface is accomplished by adjusting the base input velocity in proportion to the shear stress existing at the interface (Papastamatiou, 1973) using the formula

$$v_{bi} = v_{si} + \tau_i/(v_r \rho_r) \qquad (1)$$

where v_{bi} is the base input velocity at time t_i, v_{si} is the specified input velocity at time t_i, τ_i is the shear stress at the soil-rock interface at time t_i, v_r is the shear wave velocity of the rock and Δ_r is the density of the rock.

Henderson et al. (1989) compared SIREN with SHAKE (Schnabel et al, 1972) for a linear elastic soil profile over a transmitting bedrock. Two cases were considered: (i) a soil profile with a constant shear modulus subjected to a single pulse excitation (the solution for this case is known) and (ii) a soil profile with increasing shear modulus subjected to the E1 Centro 1940 north-south component. In both cases very good agreement was observed. In the same investigation (Henderson et al, 1989), a detailed verification of SIREN for non-linear soil profiles was carried out. Strong-motion records obtained on rock and soil deposits during the 1985 Mexico earthquake (Romo and See, 1986; Seed et al., 1987) and records obtained in a borehole at different depths from the 1987 Chibaken-Toho-Oki, Japan earthquake (Katayama and Sato 1988) were used for this verification. The SIREN results were compared with those obtained using SHAKE, which solves the equivalent linear response problem in the frequency domain (Schnabel et al, 1972). The differences between the two approaches are relatively minor, with the non-linear model producing responses approximately 10% below those produced using SHAKE. It is considered that the non-linear model provides a better representation of soil properties and was used for the results presented in this paper.

INPUT GROUND MOTION

FEMA 273 (1997) requires a range of ground motions to be considered for performance based design. For this study we have used ground motions corresponding to likelihood of being exceeded of 2%, 10% and 50% in a 50 year design life. These correspond to ground motions with return period of 2,475 years, 474 years and 72 years respectively.

Ove Arup and Partners have previously carried out seismic hazard assessments for many projects around the world. For the work presented here the results of studies for a project in Hong Kong, reported in Scott et al (1994), and for a site about 50km NE of Tokyo in Japan reported Ove Arup & Partners (1997), have been used. The results of these studies are uniform hazard response spectra for a rock site. The term uniform hazard means that each point on the spectrum has the same probability of being exceeded. The uniform hazard spectra for the Japan site and for Hong Kong are shown in Figure 3.

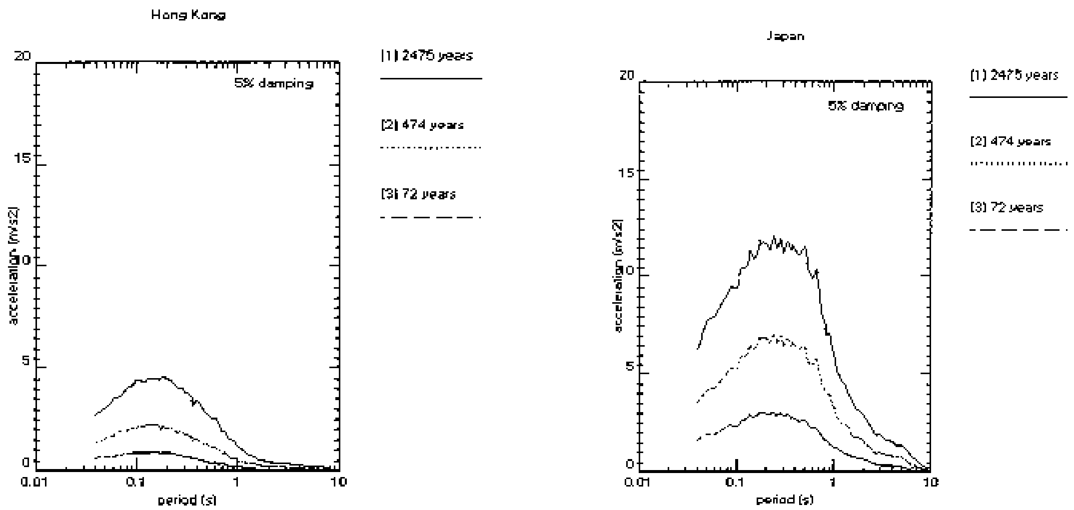


Figure 3: Uniform Hazard Response Spectra for Rock Sites

Figure 3 shows that the Japanese ground motions are about 2.5 times larger than those for Hong Kong. Another feature to note is that the Japanese ground motion has considerably more energy at longer periods. For both areas for ratio of the 474 year return period motion and the 2,475 year return period motion is about 2.5 and 5 times that of the 72 year motion period motion respectively.

Input time histories were artificially generated such that their response spectra matches the uniform hazard spectra in Figure 3. The generation procedure involves selecting real recorded earthquake time histories and then adjusting the Fourier amplitude spectrum until the response spectrum matches that desired. The phase spectrum is not modified in this process. The details of the earthquake time histories are given in Table 2.

Table 2: Input Time Histories

Location	Earthquake	Date	Magnitude	Site Conditions
Hong Kong	Montenegro, Yugoslavia	09/04/79	5.4	Rock
Japan	Hanchinohe Harbour, Honshu, Japan	16/05/68	7.9	Rock

Only one set of input time histories has been used as it has been shown previously that the response spectral ratio of the calculated surface response spectrum to input spectrum are quite consistent over a wide range of input time histories.

CALCULATED SURFACE RESPONSE SPECTRA

The calculated soil surface response spectra resulting from the site response analyses are shown in Figures 4, 5 and 6 for sites A, B and C respectively.

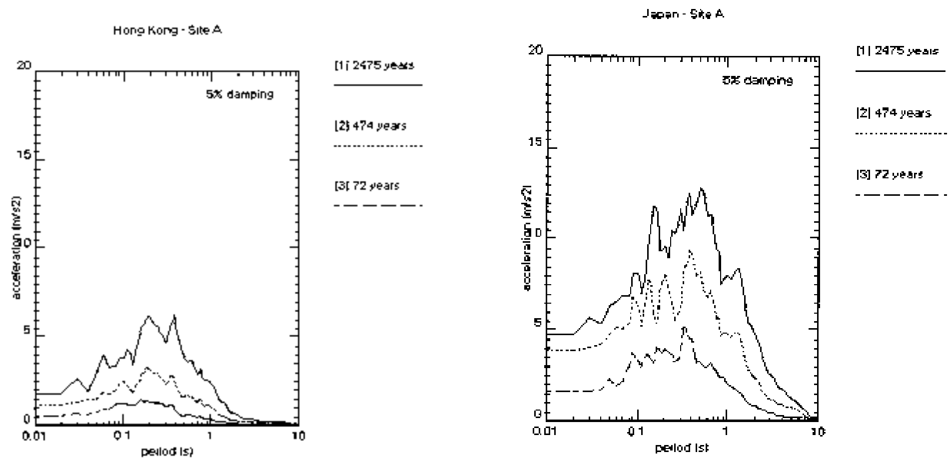


Figure 4: Surface Response Spectra for Site A

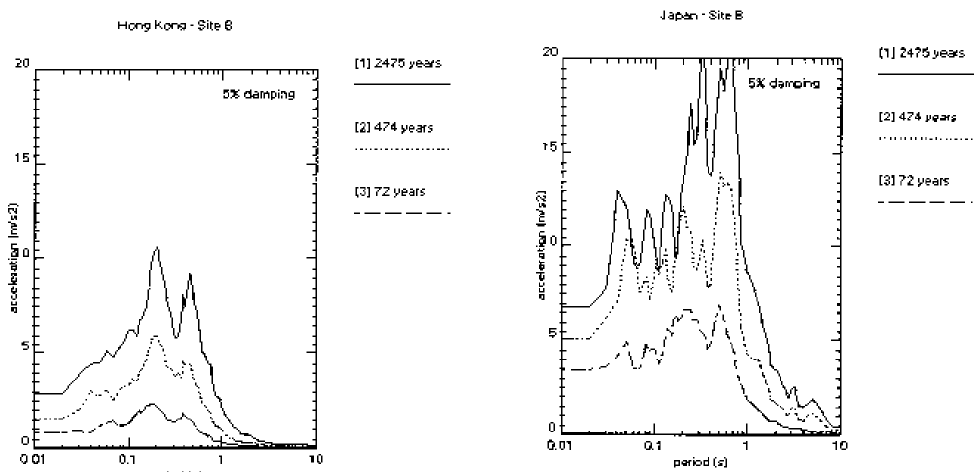


Figure 5: Surface Response Spectra for Site B

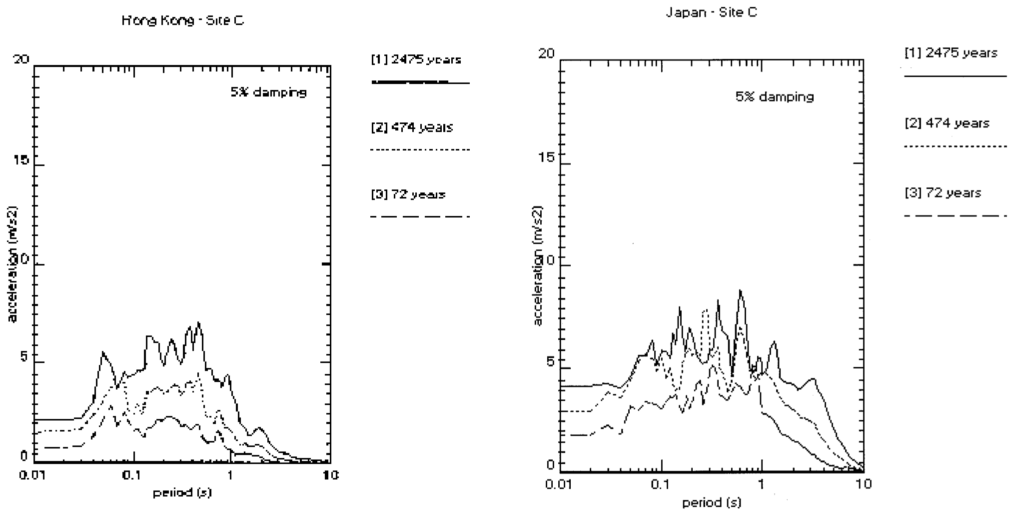


Figure 6: Surface Response Spectra for Site C

For Site A it can be seen that the surface ground motion reflects the input motion quite closely. There is generally some amplification by the soil deposit over the input rock motion especially at periods greater than 0.2 seconds. At the strongest level of ground motion, the 2,475 year return period motion for Japan, the amplification is about zero at all periods. This reflects the increasing non-linearity and consequently soil damping, associated with the larger ground motions.

For Site B there is significant amplification especially at periods between 0.2 and 0.8 seconds. This reflects the natural period of the site. The amplification ranges from about 2.5 for the weaker ground motion to about 2 for the stronger ground motion.

For Site C, the soft clay site, a different trend is observed. Here the site attenuates the stronger ground motions, except at very long periods greater than 1 second but still significantly amplifies the weaker ground motions over the whole period range. The effect of this is to make the 2,475 year return period ground motion, similar for both Japan and Hong Kong, especially at periods less than 1 second.

The non-linearity associated with this calculation is shown in Figure 7 which shows the stress strain curve predicted to be experienced by the clay soil at a depth of 20m below ground level when the site subjects to the 2,475 year return period motion for Japan and Hong Kong.

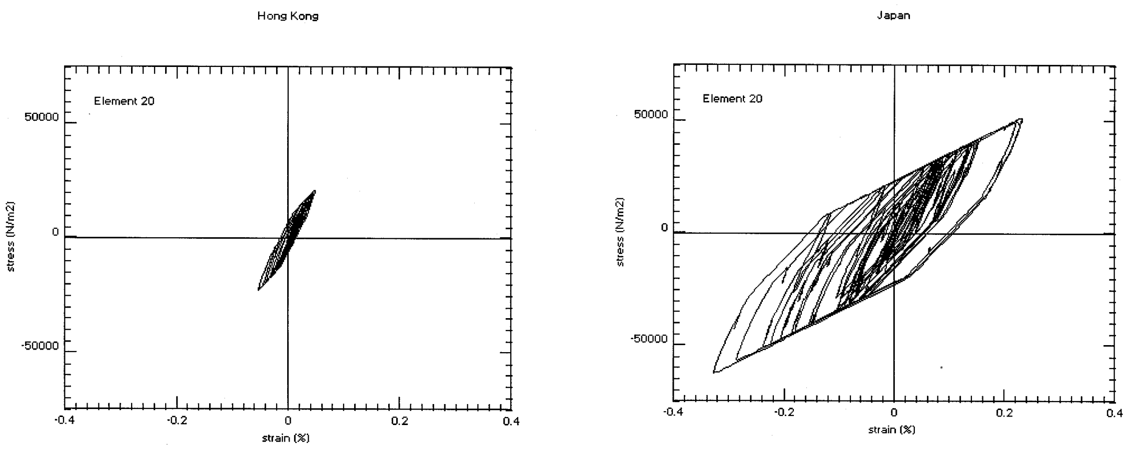


Figure 7: Predicted Shear Stress-Strain Behaviour for a Soil Element at 20m Depth when Subject to the 2,475 Year Return Period Motion

DISCUSSIONS AND CONCLUSIONS

The parametric study presented here shows that Performance Based Design has interesting implications when site response effects are considered. It is shown that, for rock sites with dense sand, the design ground motion are significantly different for an area of high seismicity to those for an area of low to moderate seismicity. This is as expected. For a soft clay site, however, the 'no collapse' level ground motion, corresponding to a return period of 2,475 year (or a 2% chance of being exceeded in a 50 year) design life, is similar in both low and high seismicity areas, especially for periods less than 1 second.

In highly seismic areas it is also shown that the 'immediate occupancy' requirements ground motion for a soft clay site is a similar magnitude as that for 'life safety' or 'no collapse'. For a soft clay site in Japan the 'life safety' or 'no collapse' ground motion is only about 1.3 and 1.8 times the 'immediate occupancy' ground motion respectively. For rock sites these ratios are about 2.5 and 5 times respectively.

The overall conclusion from this study is that performance based design will cause the 'no collapse' requirement, corresponding to a ground motion having a 2% chance of being exceeded in a 50 year design life, to dominate seismic design in regions of low to moderate seismicity. This conclusion is valid regardless of the soil type of the site. In regions of high seismicity, however, the 'immediate occupancy' requirement, corresponding to a ground motion have a 50% chance of being exceeded in a 50 year design life, will dominate the design process for low period structures located on soft or loose soil sites.

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