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Seismic base shear of structures on compliant shallow foundations

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ABSTRACT

A simplified method of assessing soil structure interaction (SSI) using a single degree of freedom representation of the structure is presented. The simplified method uses recommendations from the SEI/ASCE code and implements the response spectra of NZS 1170.5. Three typical structures are analysed, and it was found that SSI effects reduced the base shear in all cases. Reductions in base shear of approximately 30% were found for tall structures with high aspect ratios (h/r=2.8). Structures of a squat nature with aspect ratios of 1.2 yielded a reduction in base shear of 10%. In addition, a nonlinear time domain and linear frequency domain solution is applied in a preliminary manner to the same structures. Using these methods, a reduction in base shear of approximately 50% was found for tall slender structures.

1 INTRODUCTION

In general terms, there are two mechanisms of interaction (SSI) that occur between the foundation of the structure, and the supporting soil under seismic loading. *Inertial interaction* (which is the subject of this paper) arises from forces which are transmitted to the ground through the foundation causing relative motion between the ground and the foundation. *Kinematic interaction* results from wave scattering due to the impedance contrast between the seismic waves and the boundaries they meet. Kinematic interaction will not be discussed further here.

Inertial interaction will lengthen the fundamental period of vibration of the structure, due to the additional flexibility of the foundation soil acting in the system. For many structures, but not all, this will lead to a reduction in inertial forces. The fundamentals of SSI may be studied by considering a single degree of freedom structure (SDoF). Figure 1 shows a simple SDoF structure with SSI resulting in rotation and translation of the foundation.

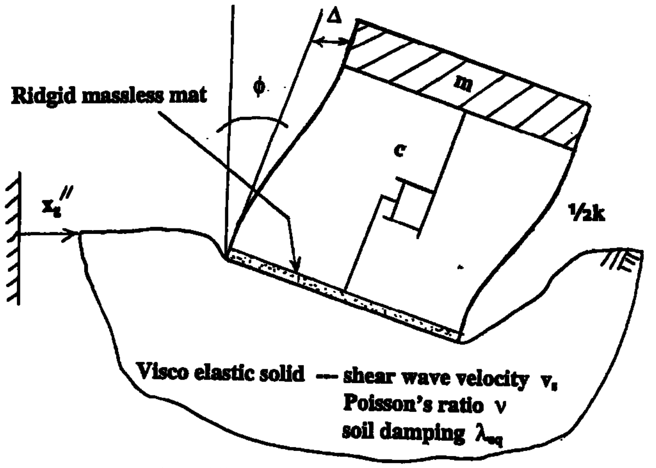

Figure 1 : Inertial SSI model

In addition to altering the vibration properties of the structure, foundation compliance also offers an effective means of energy dissipation. As stress waves propagate away from the foundation, the waves are spread across greater regions of the soil mass. This geometric effect leads to energy loss which is known as *radiation damping.* Furthermore, inertial forces may yield the soil (partial or full) beneath the foundation and as a result further energy is dissipated through *hysteretic soil damping.*

The inclusion of soil – structure interaction implies the structure can no longer be viewed in isolation; rather the "system" comprising the structure, foundations and surrounding soil needs to be considered. Several codes provide methods which approximate the effects of SSI, for example: Eurocode 8 (2002) and SEI/ASCE 7-02 (2002). However the draft New Zealand loadings code (NZS1170.5:2004) ignores the effects of soil-structure interaction unless special studies are carried out. Instead NZS1170 uses a fixed base model, which assumes the foundation and soil are infinitely stiff with no damping. In other words, the structural response is assumed independent of the site geology.

The aims of this paper is to (1) provide commentary on when foundation compliance is important for the design of foundations (2) suggest a method of using the NZS1170 response spectra to approximately incorporate soil-structure interaction effects and (3) compare the results of the suggested method with more rigorous methods in the frequency and time domains.

2 FACTORS INFLUENCING THE IMPORTANCE OF SSI EFFECTS

An important question facing designers is whether a simple fixed base solution, with a period of *T*, is adequate or whether some incorporation of soil-structure effects is needed. Eurocode 8 (2002) suggests that soil-structure interaction may be important where: P-δ effects are significant, massive or deep foundations exist, the structure has a high aspect ratio (height / width) or where the founding soil has an average shear wave velocity less than 100 m/s.

More precise guidance is given by Steward et al (1999), who suggest that soil-structure interaction effects are negligible where the inverse of the stiffness ratio, σ^{-1}, is less than 0.1.

$$\frac{1}{\sigma}=\frac{h}{V_s T}\leq 0.1 \quad \text{where } h \text{ is the height of the seismic mass} \qquad (1)$$

By examining equation (1), it may be seen that tall structures with low natural periods sited on soft soils are more susceptible to soil-structure interaction effects.

3 A SIMPLIFIED METHOD OF APPROXIMATELY INCORPORATING SSI EFFECTS USING NZS1170.5

After making preliminary judgment on the importance of soil-structure effects a further step might involve quantifying the effects of SSI without turning to numerical solutions.

SEI/ASCE 7-02 (2002) contains a detailed section on the assessment of base shear incorporating SSI effects, including an assessment of the compliant period, T^*, the foundation damping factor, ζ_0, and the system damping ζ^*. The variables controlling SSI have been found to be the stiffness ratio, σ, (equation 1), the mass ratio, γ, soil damping, λ_{eq}, and to a lesser extent Poissons ratio, ν. The mass ratio is defined by equation 2, for which a value of 0.15 is recommended for general use.

$$\gamma=\frac{m}{\rho r^2 h \pi} \quad \text{where } \rho \text{ is the density of the foundation} \qquad (2)$$

The system damping ratio, ζ^*, may be found from

$$\zeta^* = \zeta_0 + \frac{\zeta}{(T^*/T)^3} \quad \text{where } \zeta \text{ is the structural damping ratio} \tag{3}$$

The foundation damping factor, ζ_0, may also be found from the work of Kramer and Stewart (2004).

In New Zealand the method from SEI/ASCE could be used in conjunction with NZS1170.5:2004 to approximately assess base shear incorporating SSI using the equivalent static method. However the elastic spectra in NZS 1170.5:2004, shown below in Figure 2, are for conditions of 5% damping. Appropriate scaling of $C_h(T)$ for values of system damping, ζ^*, other than 5% may be achieved using the relationship of Kawashima (1994) , given below as equation 4. It should be noted that while most structures may in principle be modeled as a SDF structure (with varying levels of accuracy) the method described above does not account for effects which may potentially be significant, such as: higher modes, non-uniform soil profiles, embedded foundations, noncircular foundation shapes or multiple isolated footings, flexible structural foundation elements and piles or piers beneath the foundation slab.

$$S_a(\zeta) = \left[\frac{1.5}{(40\zeta + 1)} + 0.5\right] S_a(0.05) \quad \text{where } S_a \text{ is the spectral ordinate} \tag{4}$$

Examples of the application of this method are given in the section below where three different structures are assessed for SSI effects.

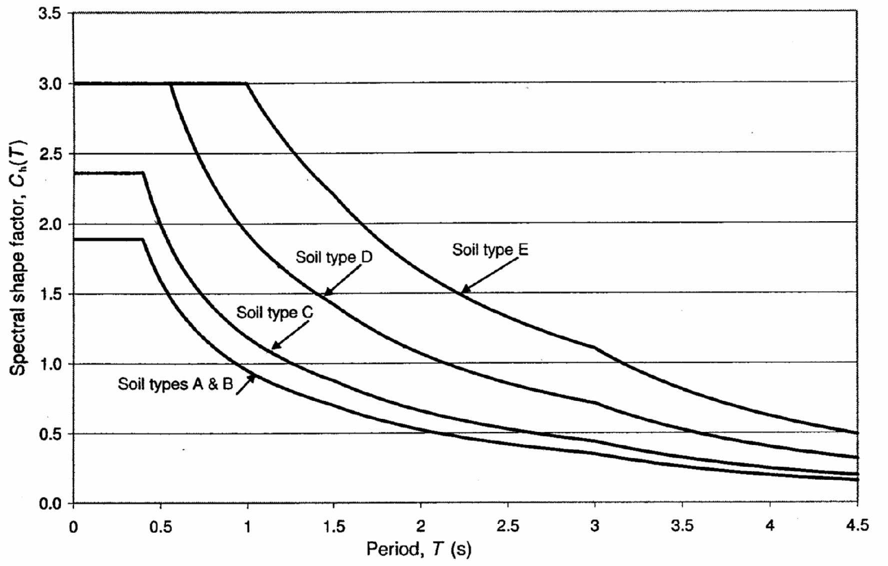

Figure 2: Spectral Shape Factor, C_h *(T)* from NZS 1170.5

The method described above lies at the rudimentary end of the scale of possibilities for assessing SSI effects. At the other end of the scale are the finite element and finite difference methods that are usually accessed through commercial software products. These applications can be expensive and require detailed knowledge of the site and structure for useful results. Two methods of intermediate level of effort are time domain and frequency domain methods applied to single degree of freedom systems. The structure is reduced to its equivalent single degree of freedom structure with mass, stiffness and structural damping and the foundation soil has finite stiffness and damping (both material and radiation damping are incorporated), as shown in Figure 1. These two intermediate level methods are briefly described below and are then applied in a preliminary way to the three example structures.

4 ASSESSING SSI EFFECTS USING A FREQUENCY DOMAIN SOLUTION

The frequency domain method has been used to incorporate SSI effects since the early 1970's. The method treats the incoming seismic wave form as a superposition of a large number of harmonic waves of varying amplitude and phase. Because superposition is used, the system must be assumed linear elastic. The effects of the supporting soil are incorporated through the use of dynamic impedance functions that incorporate stiffness and damping of the foundation soil.

A transfer function of the system in the frequency domain is calculated and used with the Fourier transform of the earthquake record to calculate the system response. The time history response of the system may then be found by performing an inverse Fourier transform. The nonlinear nature of soil response may be approximately incorporated using strain compatible dynamic soil properties for the foundation soil. The method has been further developed and applied to tanks by Larkin (2003).

Because the earthquake record is decomposed into a series of harmonics, the frequency dependent characteristics of the foundation damping may be explicitly incorporated. This is the main advantage of frequency domain solutions.

5 A TIME DOMAIN SOLUTION FOR SSI WITH SOIL YIELD AND FOUNDATION UPLIFT

However, under strong shaking, interactions between the foundation and the soil will not be linear or elastic. Under combined moment and vertical loads, the end regions of footings will become highly stressed and plastic yielding of the soil is likely to occur. In addition, if overturning moments are large, portions of the footing may also lift off the ground surface. These effects are influenced by the static factor of safety against bearing failure. Foundations with smaller factors of safety will tend to yield the subgrade material and settle quickly into the ground. In contrast, foundations with higher factors of safety will tend to uplift and cause less soil yielding.

Soil yielding offers a pathway by which significant energy may be dissipated. The force required to yield the soil multiplied by the resulting settlement (force times distance) is equal to the hysteretic energy dissipated. These kinds of analyses show that it is possible to dissipate large amounts of energy with relatively little settlement or structural damage.

Both soil yielding and uplift will also reduce the rotational, vertical and horizontal stiffness of the foundation. This effect may be understood by imagining the effective area of the foundation being reduced by the uplifted or yielded regions. For instance, in the rotational mode, the stiffness of the foundation is directly proportional to its second moment of area about the rocking axis. For a rectangular foundation, the second moment of area (I) about its width is commonly expressed as:

$$I = \frac{LB^3}{12} \tag{5}$$

Yielding or uplifting at the foundation edges reduces the effective width (B) and hence lowers the rotational stiffness of the footing. It may also be observed that the reduction in I will be proportional to the cubic power of B. Therefore, yielding and uplift of the foundation will cause highly nonlinear effects. In other words, the stiffness of the footing will reduce quickly as regions begin to uplift or yield. The consequence of reduced stiffness is higher displacements and a longer-period vibration response of the overlying structure. These kinds of phenomena have been modelled using a modified Winkler footing in combination with a single degree of freedom oscillator. Winkler models are represented physically by imagining the footing rests on a bed of independent springs. The model aims to capture the effects of footing uplift, soil yielding and the permanent displacements which are accrued beneath the footing. Because the effects of uplift and soil yielding are nonlinear, a linear elastic frequency domain solution cannot be used, and instead the system is solved in the time domain.

6 APPLICATION OF METHODS TO THREE STRUCTURES

Three typical structures have been analysed using the modified NZS1170 method and the frequency and time domain solutions. The structures are reduced to their SDoF equivalent using the principles of structural dynamics and the procedures described above are used to assess the period ratio and system damping. The spectra of NZS 1170.5:2004, for use in the equivalent static method, are used to ascertain the difference in base shear. The equivalent radius, r , is calculated on the basis of a circular foundation of the same area as a square foundation. The structures are assumed to be in the epicentral area of a magnitude 7.5 earthquake, founded on a class C site and to remain elastic.

Case 1: A square four story moment resisting frame of height 17m founded on competent ground, i.e. a class C site. The width of the structure is 17m.
Case 2: A square 14 story moment resisting frame of height 45m and width 20m founded on soft ground, i.e. a class D site.
Case 3: A 20 story square shear frame structure of height 45m and width 20m founded on soft ground (class D) loaded in the plane of the shear walls.

Table 1 shows the results of the simplified method utilising NZS 1170. There are reductions in base shear in all cases. In the case of the 4 story structure the reduction of 10% is within the margin of uncertainty in the calculations and hence on pragmatic grounds may not be included in design. This result is in agreement with the general guideline that structures with a stiffness ratio of 0.1 or less do not experience significant SSI effects. The two structures with higher aspect ratios have significant period lengthening, particularly the shear wall structure. The system damping is 5% in all cases which implies minimal radiation damping from the soil and hence most of the base shear reduction eventuates from period lengthening.

It is important to realize these calculations are approximate only and many assumptions have been made in representing the real structure as a mathematical model which may be analysed by the NZS 1170.5 spectra. A particular limitation is that the structure remains elastic. The ASCE code limits inertial SSI effects to a reduction in base shear of 30%.

Table 1 : Effect of SSI from the simplified method

Dynamic Properties	**Case 1**	**Case 2**	**Case 3**
fixed base period, T	0.5 secs	1.8 secs	1.0 sec
aspect ratio h/r	1.24	2.8	2.8
shear wave velocity V_S	250 m/sec	120 m/sec	120 m/sec
strain compatible shear wave velocity $V_{S\gamma}$	210 m/sec	80 m/sec	80 m/sec
stiffness ratio $1/\sigma$	0.11	0.22	0.39
period ratio	1.05	1.3	1.8
structural damping ratio ζ	0.05	0.05	0.03
foundation damping ratio ζ_0	0.01	0.03	0.04
system damping ratio ζ^*	0.06	0.05	0.05
$S_a(\xi^*)/S_a(.05)$	0.94	1.0	1.0
$C_h(T^*,\zeta^*)/C_h(T,.05)$	$\frac{C_h(.53,.06)}{C_h(.5,.05)} = \frac{1.8}{2.0}$ $= 0.9$	$\frac{C_h(2.34,.05)}{C_h(1.8,.05)} = \frac{.9}{1.15}$ $= 0.78$	$\frac{C_h(1.8,.05)}{C_h(1.0,.05)} = \frac{1.15}{1.7}$ $= 0.68$
change in base shear from SSI	-10%	-22%	-32%

Table 2 shows the reduction in base shear predicted by the frequency and time domain solutions using the earthquake record Takarazu from the Kobe event. The strong motion recorder is sited on a deposit of alluvium and produced a record having a maximum acceleration of 6 m/sec^2 with about 10 seconds of strong ground shaking containing significant near source characteristics.

Table 2 : Comparison of reduction in base shear

Method	Case 1	Case 2	Case 3
Time domain	-37%	-65%	-65%
Frequency domain	-24%	-60%	-41%

7 CONCLUSIONS

Application of a simplified method of assessing SSI using NZS 1170 and the frame work of the code SEI/ASCE has been presented. Significant SSI effects were found for tall slender structures with reductions in base shear of approximately 30%. This is the figure that is specified in SEI/ASCE (2002) as being the maximum permitted reduction in base shear from SSI and thus supports the simplified analyses carried out here. The squat structure assessed for SSI yielded a reduction in base shear of 10% which is within the margin of error of the method.

Application of the two more sophisticated methods yielded significantly increased SSI effects. Only one earthquake record has been applied to the systems and thus variation from code results is to be expected. The earthquake record used has significant near source effects and the long period nature of these characteristics will have influenced the dynamic response of the systems, particularly those with long periods. In these circumstances SSI effects will be enhanced, perhaps significantly. This is possibly a reason for the divergence between the more sophisticated methods and the simplified method. A suite of earthquake records needs to be applied to the time and frequency domain methods before definitive comparison can be made with the simplified method.

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