

## The Seismic Design of Shallow Foundations: A State of the Art Exploration

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**Abstract:** The engineering understanding of the behaviour shallow foundation in earthquakes has developed piecemeal over the past sixty years with the emergence and refinement of solutions to several separate aspects of the overall problem, prompted and provoked by field observations. The past few years have seen more rapid development as the insights and techniques developed for the separate aspects of seismic foundation behaviour have been integrated into more complete solutions. Unsurprisingly the more complete solutions require numerical solution techniques which make them less transparent than their progenitors. The review commences by considering relevant aspects of soil and rock behaviour, not all of which have been thoroughly investigated to date. Elastic solutions are reviewed followed by a review of seismic bearing capacity solutions, consideration of foundation rocking and solutions for permanent displacements and the combination of all of these aspects into macro-elements suitable for combined numerical analysis of the soil-foundation-structure system. The understanding of shallow foundation seismic behaviour continues to develop with new issues coming to the fore. The paper considers some of the areas of current development and some of the issues which need to be addressed going forwards.

### 1. Introduction

#### 1.1 Seismic Foundation Design Considerations

The normal role of foundations is to transmit loads into the ground with modest levels of deflection which are tolerable to the form of structure being supported. The foundation will need to perform this role with a level of reserve in order to impose modest demands on the supported structure and also to accommodate the variability in ground conditions which may be encountered at the site. It is desirable that the soil-foundation-structure system should work together in a coherent manner. In particular, if the site is exposed to high transient environmental loadings (including earthquakes) it is highly desirable that the soil-foundation part of the system should play an appropriate role in delivering the required overall performance.

In practice the static design of pad or strip foundations is usually undertaken by assessing the static bearing capacity under various potential load combinations. Acceptable performance with regard to deflections is usually obtained by dividing the ultimate bearing capacity by a substantial factor (typically 2.5 to 3) to determine allowable working loads. It would be more intellectually satisfactory to calculate the deflections of these foundations directly. However, it is generally more straightforward to obtain a measure of the strength of the soil rather than its stiffness and hence there is some rationality in a strength-based design approach. For larger slab or raft foundations the common approach is to calculate the deflections directly allowing for short term and longer term soil behaviour under the applied loadings. In this class of foundation, the differential deflections across the foundation often dictate the structural design of the raft.

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The seismic design of foundations has largely evolved from static design considerations. However, the performance requirements for foundations under design seismic loadings differ from the static requirements in several important aspects. The loadings (or more accurately, the displacements) imposed by the earthquake are of very short duration and hence some permanent soil-structure deflections under the peak actions may be permissible and even desirable. The foundation must continue to support the structure during and after the seismic event and foundation deflections must remain within the range that the structure can withstand. In order to limit the transient deflections it is very desirable that the stiffness of the combined building and foundation system does not cause the structure to respond within the peak frequency range of the seismic motion. These considerations require different analytical approaches to those used for static design.

The designer of shallow foundations subject to seismic loading ideally seeks robust solutions to the following questions:

- What transient and permanent foundation deflections will occur? Knowledge of the seismic bearing capacity will assist in answering this question but will not provide the complete solution.
- What transient accelerations, forces and/or displacements will be transmitted through the foundation and into the structure?
- Will the foundation continue to support the structure following the seismic event?
- How can economic foundation solutions be provided for new and existing structures?

The paper seeks to address these issues in more detail, highlight gaps in our knowledge which may be useful areas for further research.

## **1.2 Shallow Foundations Seismic Design Overview**

The engineer has a variety of approaches which may be applicable to the seismic design of shallow foundations. Indeed it is quite likely that on any major project several of the different classes of analysis will be used to assess different aspects of the foundation behaviour as the project proceeds. An overview of the types of analysis which may typically be used is illustrated in Figure 1 and are discussed in more detail in the following sections.

The assessment of pseudostatic seismic bearing capacity (Figure 1a) developed from static foundation bearing capacity theory. Solutions exist for bearing capacity, sliding and rocking behaviours. If the behaviour of the foundation remains within the soil's yield envelope then elastic impedance methods (Figure 1b) may be used to model the soil-structure interaction, idealising the soil behaviour as a spring and dashpot for each mode of behaviour examined. Heavy and stiff foundations may impose kinematic constraints and inertial loadings on the soil which affect the ground's stiffness and modify the earthquake motions transmitted to the structure. These effects are conveniently studied by 2D and 3D FE or FD models (Figure 1c) which may use equivalent linear idealisations of the soil behaviour or full non-linear hysteretic stress-strain models. It may be necessary and/or desirable to permit the foundation to displace under the seismic actions (Figure 1d). Various models enable differing modes of displacement to be assessed. Finally the elastic and plastic behaviour of the foundation may be brought together using a macro-element (Figure 1e) and combined with the structural finite element model. This approach enables more realistic behaviours of the foundation to be incorporated into the design with a reasonably modest computing overhead and may be particularly useful for design using smaller pad or strip foundations.

It may be noted at this point that the design of large raft foundations will need to address rather different issues to those addressed for strip or pad footings. While sliding of a raft may be possible, bearing capacity or rotational "failure" is unlikely. Large rafts may flex during

seismic loading and the seismic actions across the raft may become incoherent. These issues are unlikely to require evaluation for most designs using pads and strip footings.

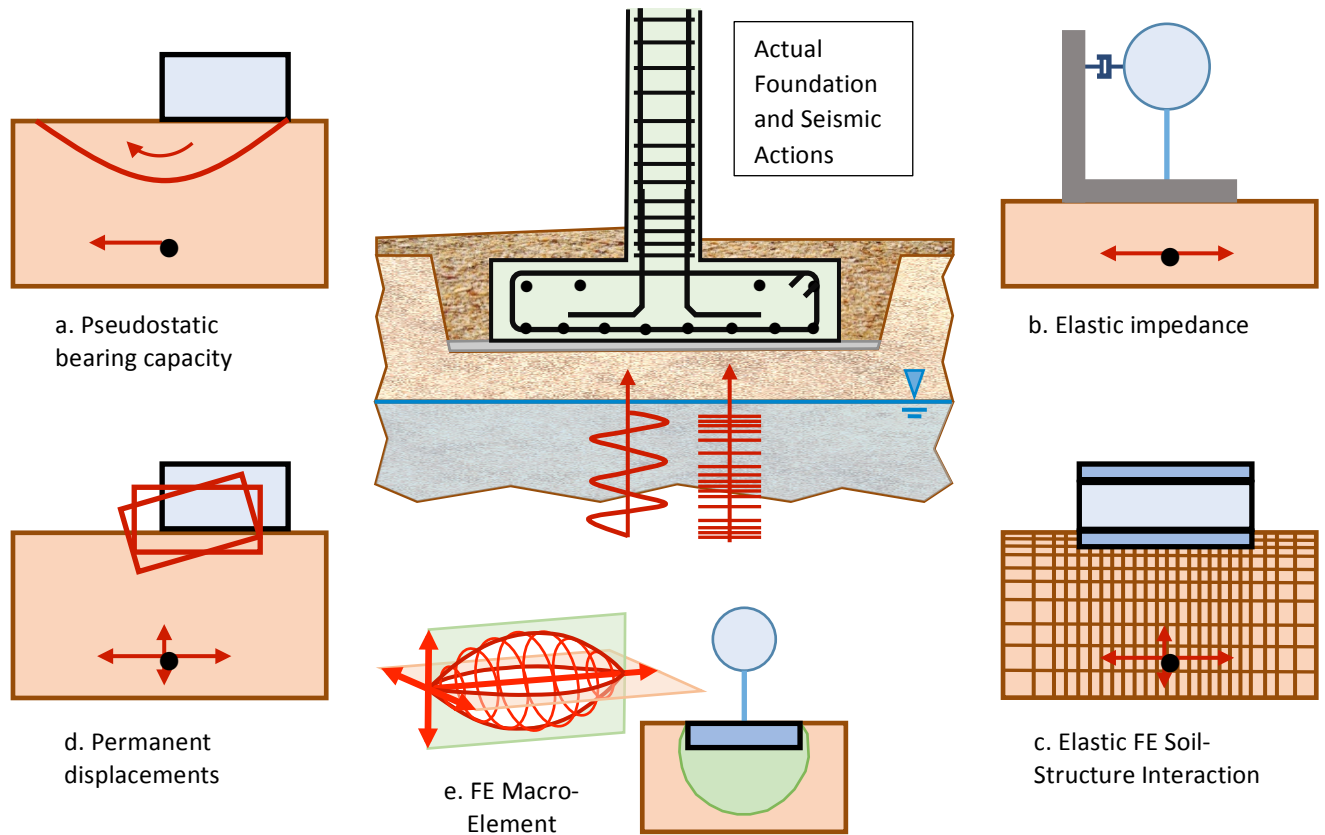


Figure 1. Analytical methods for seismic design of shallow foundations

### 1.3 Use of Models in Engineering Design

As in all fields of engineering there is a fundamental difference between seismic foundation design and seismic foundation analysis. The design aspect involves assessment of the requirements; selection of an appropriate foundation solution (conventional or novel); and definition of a process for evaluating and refining the chosen solution. No amount of analysis will change a fundamentally poor foundation solution into a good solution. The evaluation and refinement will normally require the use of models which may be conceptual and expressed mathematically; physical experiments; or prototype observations or a combination of these. At different stages of a typical large project the engineer may need to use a series of different models to validate and refine the design concept. This paper primarily focusses the various classes of conceptual model available for seismic foundation design. All such models involve simplifications and assumptions of which the designer must be fully aware.

Figure 2 illustrates the use of some of these models in the various stages of an actual seismic design project. This example illustrates *inter alia* that the seismic foundation designer needs to be conversant with a wide range of analytical solutions in order to deliver a robust and appropriate design. The models used to validate initial design choices in this example were highly simplified but, used carefully, sufficient to enable the structural design to proceed in parallel with the more detailed assessment of soil-structure interaction. More sophisticated modelling was used to refine the design in its later stages and was able to make use of the detailed site investigation data which was not available at the outset.

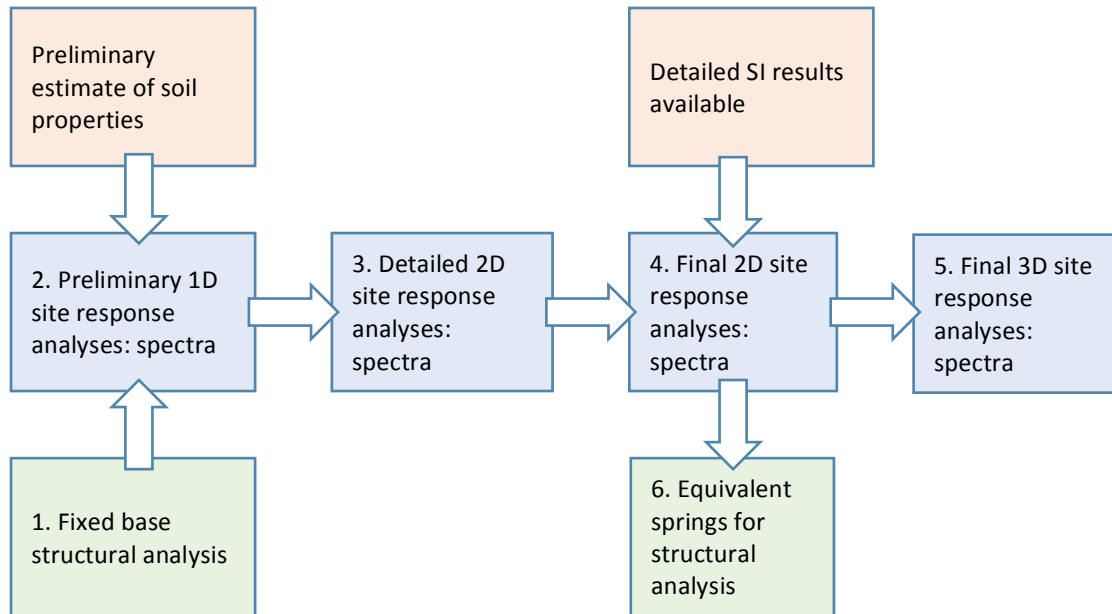


Figure 2. Typical flowchart for soil-structure interaction analysis in seismic foundation design

## 2. Aspects of Soil and Rock Behaviour

### 2.1 Introduction

The use of shallow rather than deep foundations is usually dependant on the loads to be supported, the strength of the ground and sometimes by the value and/or risk associated with the supported structure. For common types of structure, shallow foundations may be appropriate in firm to stiff (or stronger) cohesive soils or granular soils which are medium dense or denser. The foundation designer will need to address various potential geohazards (e.g. shrink-swell behaviour, collapsible soils, voids, slope instability etc). It is not uncommon for the designer to find that the soils are stronger at the surface and weaker at depth. If shallow foundations are used in such conditions then the performance of both the stronger and the weaker strata may be critical to the overall foundation performance.

When considering the seismic performance of shallow foundations under seismic loading a range of issues should be borne in mind by the designer. These include:

- The properties of the soil beneath the foundation (including its strength and stiffness) will change over time due to consolidation, creep, the effects of low level cyclic loading and moisture content change in partially saturated conditions. Hence the foundation response at the start of a severe or design earthquake may be affected by soil conditions which differ appreciably from those determined from a pre-construction green-field site investigation.
- The stiffness of soil and (to a lesser extent) weak rock is highly non-linear as discussed below. The soil is also anisotropic in its stiffness and strength properties which means that considerable care is required when interpreting measured ground properties for use in simplified linear or non-linear isotropic analytical models.
- Under cyclic loading the stiffness of the soil is “reset” to its current small strain value on each abrupt change in direction of the stress path. Degradation of stiffness due to increasing strain occurs along the most recent stress path until the next abrupt change

occurs. Degradation of stiffness also occurs under continued cyclic loading due to the build-up of pore pressure in the soil. This occurs more rapidly when the cyclic loading includes substantial rotations of principal stresses (as is common in seismic loading).

- Considerable energy is dissipated by foundations due to hysteretic damping in the soil and by radiation damping. The hysteretic component is strain-dependent. Radiation damping may be affected by the position of the ground water level (Section 3.5).
- If the soil yields under cyclic loading the strains will become localised on shear surfaces. On these surfaces the strength of the soil will tend to be reduced from the peak values of shearing resistance. In granular soils the strength is likely to approach the critical state values while in cohesive soils the undrained strength may approach remoulded values. It should be noted that the shear surfaces generated due to seismic loading of a foundation are likely to be in substantially different planes to those relevant to static (i.e. pre- and post-seismic) loading conditions.

### 2.2 Non-linear behaviour of soil and rock

A fundamental challenge when attempting SSI analyses is the highly non-linear behaviour of soil and also of rock masses.

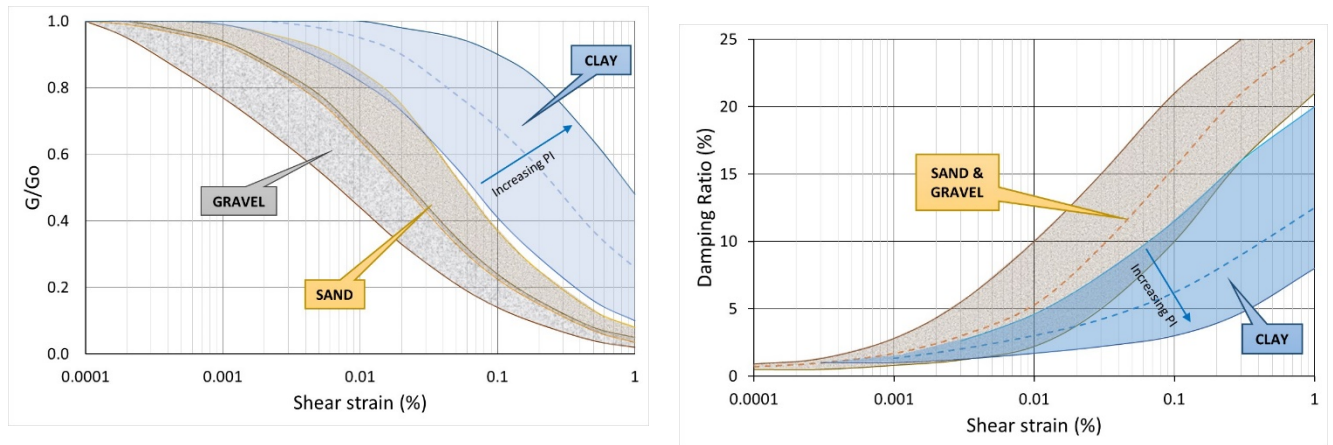


Figure 3. Stiffness and damping for sands and clays

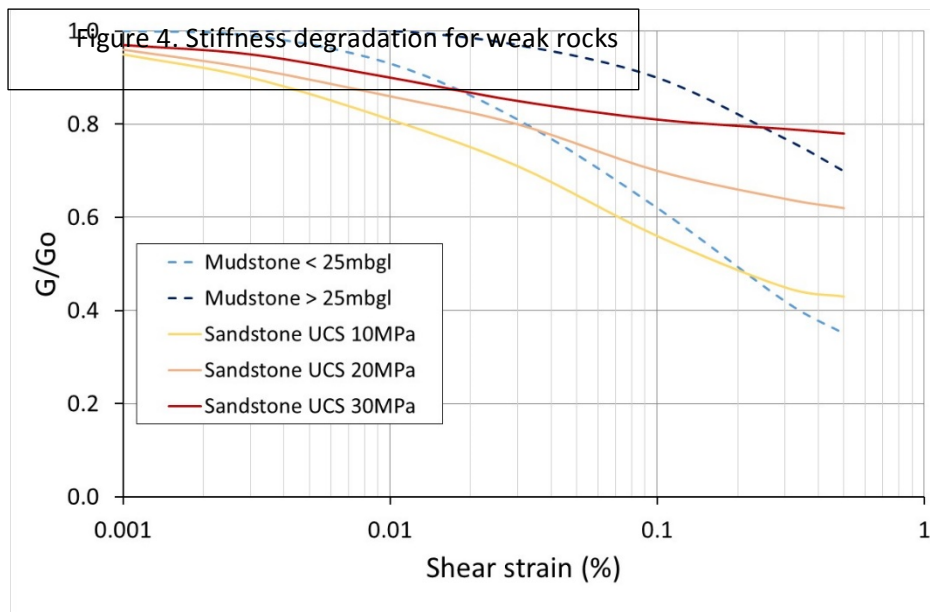


Figure 4. Stiffness degradation for weak rocks

Figures 3 and 4 show the degradation of normalised shear modulus in various types of soil and rock mass using published relationships from Seed et al. (1986), Vucetic and Dobry (1991), Darendeli and Stokoe (2001), Thomson and Leach (1985) and Davis et al. (1996). Also shown on these figures is the increase in material damping with increasing strain. These changes in stiffness and damping with changing strain levels have a profound effect on the SSI response in soils and weak rocks. Hence appropriate values of stiffness and damping of the soil must either be selected *a priori* (which is a significant challenge) or the degradation relationships must be incorporated into the SSI analysis.

Vucetic (1994) identified two limiting cyclic shear strains on the modulus degradation vs strain and damping vs strain relationships. For strains below ( $\gamma_{tl}$ ) the small strain soil behaviour has a constant stiffness. For strains between ( $\gamma_{tl}$ ) and ( $\gamma_{tv}$ ) the stiffness of the soil decreases with increasing strain but no degradation is seen with increasing numbers of cycles. For strains above ( $\gamma_{tv}$ ) volumetric change or pore pressure increases occurs with increasing numbers of cycles. Vucetic related the two threshold values to soil plasticity as

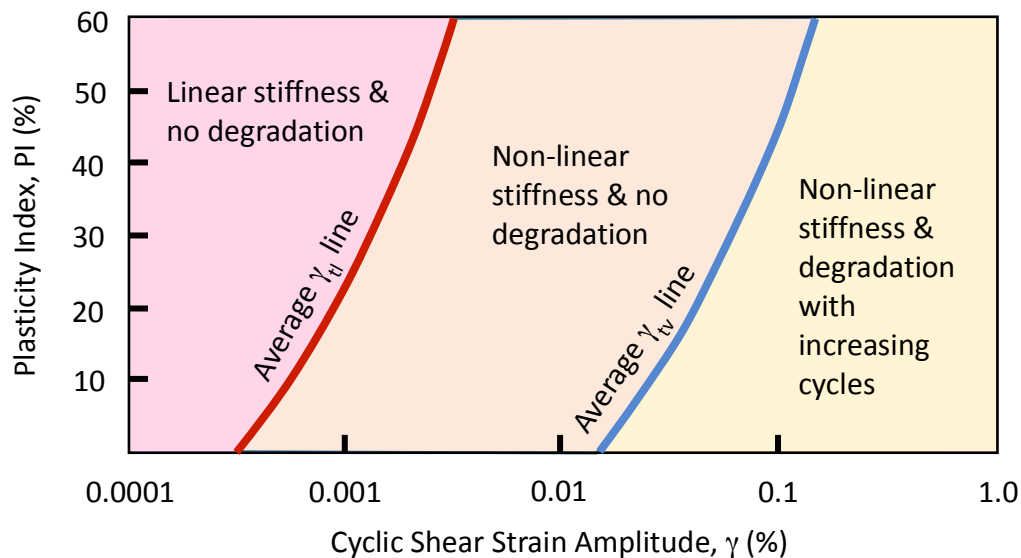


Figure 5. Soil stiffness characteristics with cyclic strain and Plasticity Index, after Vucetic (1994)

shown in Figure 5.

### 2.3 Partially saturated soil

In many cases the soils immediately below shallow foundations may be partially saturated either at the time of an initial saturation and/or when the foundation is subject to earthquake shaking. This applies as much in temperate climates (such as the UK) as to foundations in hotter and drier climates. For critical structures the effects of potential climate change need to be considered within the life of the structure. The effects include potential changes in ground water levels outside of their present-day ranges (see HSE, 2008).

The behaviour of partially saturated soils under seismic loading is less well understood than that of saturated soil. Fredlund et al (2012) provides an excellent treatise on the static behaviour of partially saturated soils, explaining *inter alia* the usefulness of Soil-Water Characteristic Curves and the role of soil suctions in the observed strength and stiffness of soils in this state. Relationships between moisture content and suction are shown in Figure 6 for a range of soil types.

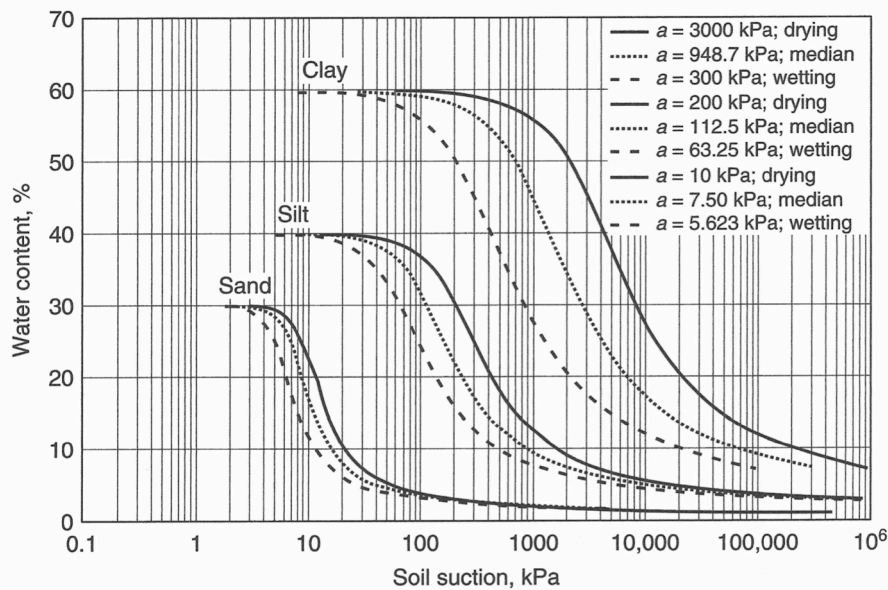


Figure 6. Typical Soil-water Characteristic Curves for various soil types, Fredlund et al (2012)

The static and cyclic tests conducted by Schneider et al (2008) in Perth Sand provide important insights into the brittle nature of the enhanced strength and stiffness of partially saturated granular soil. In particular they demonstrate that the ratio of the cyclic resilient modulus (determined by careful self-boring pressuremeter testing) to the small strain shear modulus (determined from shear wave velocities) is significantly lower for the partially saturated sand compared to the ratios typically observed for saturated sands. This appears to be due to a rapid breakdown in the bonding of the partially saturated soil with increasing strain. For partially saturated clay soils much higher suctions may develop than can be sustained in sands (see Figure 6). Evidence from Vucetic (1994) suggests that the cyclic strain limit  $\gamma_{TV}$  can be an order of magnitude higher in partially saturated clay soils than for the same soils in a saturated condition.

Further useful information on the dynamic Poisson's ratio of partially saturated sands at various confining pressures has been provided by Kumar and Madhusudhan (2012). The value of the dynamic Poisson's ratio (with the shear wave or compression wave velocity) are required inputs for evaluating "elastic" soil-structure response. Variations in the soil's dynamic Poisson's ratio can have a substantial effect on the vertical response of the system as described below.

The loss of full saturation in soils typically effects a dramatic reduction in the p-wave velocity. By contrast the s-wave velocity is practically unaffected unless high soil suctions develop. In saturated soils the p-wave travels at approximately 1500 m/s through the water phase. But in partially saturated soils with strengths in the range firm to very stiff for clays or medium dense to dense for sands the p-wave velocities will typically be in the range 400 - 750 m/s.

### **3. Seismic Soil Structure Interaction**

#### **3.1 Introduction**

Seismic soil structure interaction (SSI) arises from the kinematic interaction of the soil with the structure and the inertial interaction of the structure with the soil. (Note that although the term “soil” is used, SSI can also occur between a stiff and heavy structure and weak or fissured rock.) The kinematic interaction of the soil with the structure occurs when the stiffness of the structure prevents the free field soil displacements from occurring. A common example occurs for stiff buried structures which are sufficiently stiff to restrain the seismic free field horizontal ground motions. The kinematic interaction generates additional lateral loads and overturning moments acting on the structure. Inertial interaction of the structure with the soil occurs when the inertial loads from the acceleration of the structure generate significant additional deformations on the soil. Hence both the mass and the stiffness of the structure relative to the ground dictate the significance of SSI effects.

Elastic interaction analysis of light and relatively flexible structures (e.g. typical reinforced concrete moment resisting frame structures) with shallow foundations will typically indicate that the structure has little effect on the seismic actions at foundation level. However, if the foundations slide or rock under the applied actions there will usually be a substantial reduction in the peak actions transmitted into the structure.

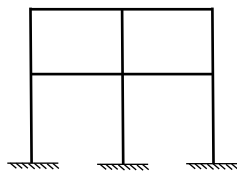
The effect of SSI is to lengthen the fundamental period of the soil-structure system by comparison to the values calculated using a fixed-base model of the structure. For large stiff structures on raft foundations the main purpose for assessing SSI is to obtain modified base slab seismic acceleration spectra and/or to determine an equivalent simplified foundation model (e.g. springs and dampers) for use in the seismic structural analyses.

#### **3.2 Options for SSI Analysis**

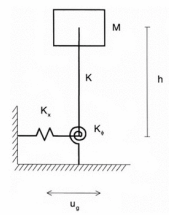
Potential direct analysis methods for considering the interaction of the structure with the ground are shown on Figure 7. They include consideration of the ground as rigid (fixed-base model), the characterisation of the ground by a set of springs and dashpots to represent its stiffness and damping characteristics, and 1D, 2D and 3D SSI numerical analysis models. It is likely that major structures in seismic locations will be analysed using detailed 3D structural finite element models. So it may be supposed that such models should be extended to incorporate SSI by inclusion of additional elements to model the soil surrounding the structure. However, in practice it is often more practical and more transparent to separate the detailed structural modelling from the modelling of the soil-structure interaction. The various available SSI models can all have a useful role to play in a robust assessment of the seismic interaction between the ground and the structure.

Fixed base analysis of the structure assumes a rigid foundation. For structures founded on hard rock this is likely to be an appropriate assumption. For structures on less stiff foundation materials the fixed base analysis will underestimate deformations and in-structure seismic amplification. However, a fixed base analysis is often a necessary starting point for SSI analyses on softer foundations. The fixed base fundamental structural frequencies (horizontal and vertical) and the respective mass participations are very useful for deriving simplified models of the structure for use in SSI analyses.

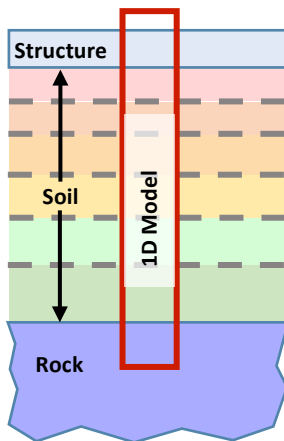
The next class of models represent the foundation behaviour using springs and dampers. For a typical shallow foundation 6 degrees of freedom can be defined: vertical, 2 horizontal, 2 moment and torsion. Solutions are in the form:



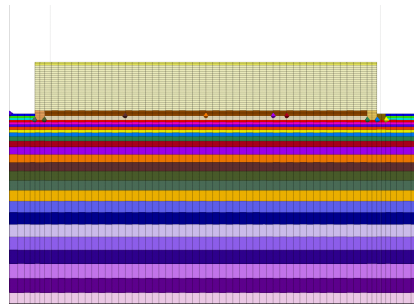
a. Fixed base



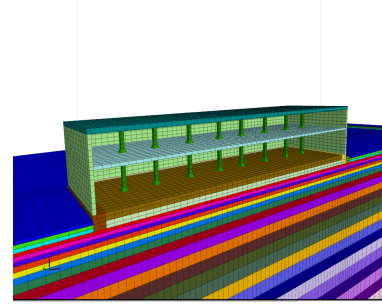
b. Spring & dashpot



c. 1D SSI



d. 2D SSI



e. 3D SSI

Figure 7. Options for SSI Analysis

$$\bar{K}_x + i\omega C_x = \frac{F_x}{u_x}$$

Where  $K_x$  is an equivalent dynamic spring

$i$ ,  $\omega$  and  $C_x$  are  $v(-1)$ , frequency and dashpot coefficient respectively

$F_x$  (or  $M$ ) is a force or moment and  $u_x$  (or  $\theta$ ) is a displacement (or rotation)

Solutions for the values of  $K$  and  $C$  have been published by various authors, those by Dobry and Gazetas (1986) and Gazetas (1991) are particularly well known. These simple models are useful for gaining an understanding of an elastic SSI problem (such as the response of a large raft foundation).

For 1D and 2D SSI analysis it is common to derive “three layer sandwich” of low  $H/B$  ratio shear wall structures. A stiff base slab and roof slab are modelled with a soft middle section representing the mass and stiffness of the walls. Closed form shear beam solutions are used to obtain the necessary stiffnesses for the building models tuned to the fundamental sway and bounce modes. These simplified structural models are then applied in appropriate SSI analyses.

It might be supposed that the simple spring and damper representations of the ground would provide a rapid and adequate way to incorporate SSI into the structural analysis. However, in practice these models are difficult to derive without the assistance of more sophisticated

modelling. The springs and dampers are formulated in terms of a single value of soil shear stiffness and Poisson's ratio. In practice the soil's stiffness and material damping is highly dependent on the shear strains (which will vary temporally throughout the earthquake and spatially throughout the foundation) and on the static small strain stiffness distribution which will almost certainly vary strongly with depth below ground level. Allowance could be made for these major variations by adopting wide bounds on the values of stiffness used in the analysis. However, a more satisfactory approach is usually to return to the derivation of these springs once the effect of SSI on the overall behaviour has been established by other means. It remains useful in the seismic structural analysis to be able to represent the foundation by means of appropriate springs and dampers. This is sometimes effected using a single set of horizontal, vertical, moment and torsion springs (e.g. as in ASCE 4-98). However, it is common for the structure to be sufficiently flexible to require a distributed spring bed or modulus of subgrade reaction approach. Spring beds, although convenient for structural analyses, are rather poor models of soil behaviour and it can be difficult to define a single spring bed which simultaneously provides adequate vertical and rocking responses.

One dimensional SSI numerical analyses can be undertaken using any one of a number of different site response codes. The most commonly used is SHAKE (Schnabel et al., 1972) and its various derivatives e.g. WESHAK, SHAKE91 and SHAKE2000. This code transforms seismic acceleration time histories into the frequency domain and uses Green's functions to calculate the transfer of the motion between soil strata of varying stiffness. An iterative solution is used to account for the strain dependency of stiffness and damping on shear strain for each soil layer. This produces an equivalent stiffness and damping (usually equal to the values at 2/3 of the peak shear strain) which is effectively applied to the complete seismic event. This comparatively simple approach to modelling the soil behaviour is termed "equivalent linear analysis" and can be applied to various forms of SSI calculation. The standard use of the code is to calculate free field acceleration time histories at the ground surface using the expected seismic motion at buried rockhead as the input. For SSI analysis additional layers are added to represent the mass and stiffness of the structure. This is clearly a highly simplified representation of the building and the ground which, among other limitations, does not allow for moment-rotation effects. Nevertheless, for large structures with low H/B ratios on mat or raft foundations it can be surprisingly useful. Not only can a reasonable preliminary estimate of the spectral response at foundation level be obtained but also the likely design impact of different seismic spectra and different strata profiles and properties can quickly be evaluated. Examples using this approach for the assessment of the effects of varying groundwater levels on the vertical response are shown below.

Modelling the both the soil and the structure together in two dimensions is a step forward in complexity from the 1D analyses. The 2D approach allows a finite building to be placed on the soil, as opposed to the 1D assumption of infinite lateral building extents. The 2D model of the building is able to sway, rock and flex more realistically. The stiffness and mass distribution of the building can be fine-tuned to ensure that these modes occur at the correct frequencies. The distribution of mass through the building slice can be apportioned more realistically, usually with the most mass located in the base and roof slabs. Embedment of the building foundations in the ground may be modelled. Incorporating these factors all produce a more realistic modified response spectrum at foundation level of the building.

However, there are still modes of the structure that cannot be adequately captured in 2D. For example twisting of the building, corner amplifications and roof 'drum' modes cannot be captured in 2D. Modelling in 3D also allows an increase in complexity of the structural model. Walls, columns and internal floors can be modelled explicitly increasing the realism of the response.

Finite element and finite difference codes are used to carry out the 2D and 3D SSI analyses. For 2D analyses specialist codes QUAD4M and FLUSH are available. These codes use equivalent linear soil models and an iterative solution approach with similarities to that described for the 1D SHAKE analyses. More recent finite element and finite difference codes such as DYNA3D and FLAC have been developed which incorporate the soil behaviour using a hysteretic model.

Using the hysteretic soil model a time-history analysis can be run without requiring the iterations of the equivalent linear method. When an element of soil is subjected to cyclic loading the stress-strain behaviour follows a curve. The loading and unloading curves form a loop. Under cyclic loading, the hysteretic model updates the stiffness of the soil during the analysis to move around the loop. The area enclosed by the loop represents the material damping of the system. An example of a hysteresis loop obtained from an analysis in which a single element is strained at a constant rate is shown in Figure 8. Hysteresis loops are produced at several different constant strain rates for an element of soil, and modulus reduction and damping are plotted with strain which are compared with the empirical backbone curves for that material. In this way, the hysteretic model can approximate the backbone curves. The hysteretic model was calibrated to match as closely as possible both the stiffness and damping backbone curves.

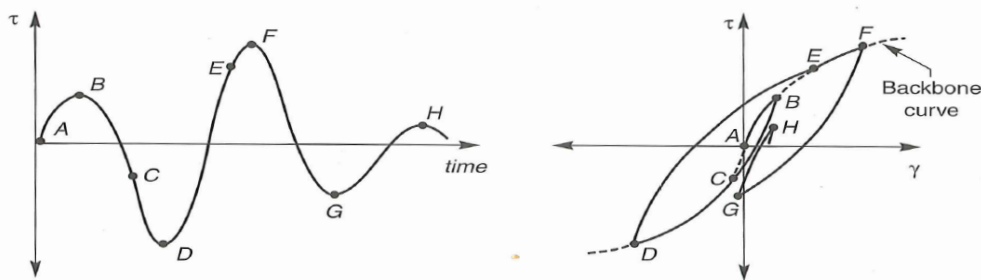


Figure 8. Single element hysteresis loop

### 3.3 Comparison of 1D, 2D and 3D SSI Approaches

Figures 9 to 11 show the horizontal and vertical SSI spectral responses at the centre of the foundation slab for a stiff and heavy nuclear structure from 1D, 2D and 3D analyses. This example is for a structure with its long axis aligned N-S and a length to width ratio of 2.5 to 1.

On these figures the 1D response is an envelope of results from analyses using the equivalent linear WESHAK code. This analysis has been performed using a PML earthquake spectrum. The structure was represented by a single element with a mass chosen to match the average vertical stress imposed by the structure on the soil column and stiffnesses chosen to give match with the fundamental sway and bounce modes of the structure in separate horizontal and vertical analyses.

The 2D results were produced using equivalent linear QUAD4M analyses and the 3D results were run in FLAC3D using a hysteretic soil model. Careful calibration demonstrated that the equivalent linear and hysteretic models gave closely comparable results for 2D slices.

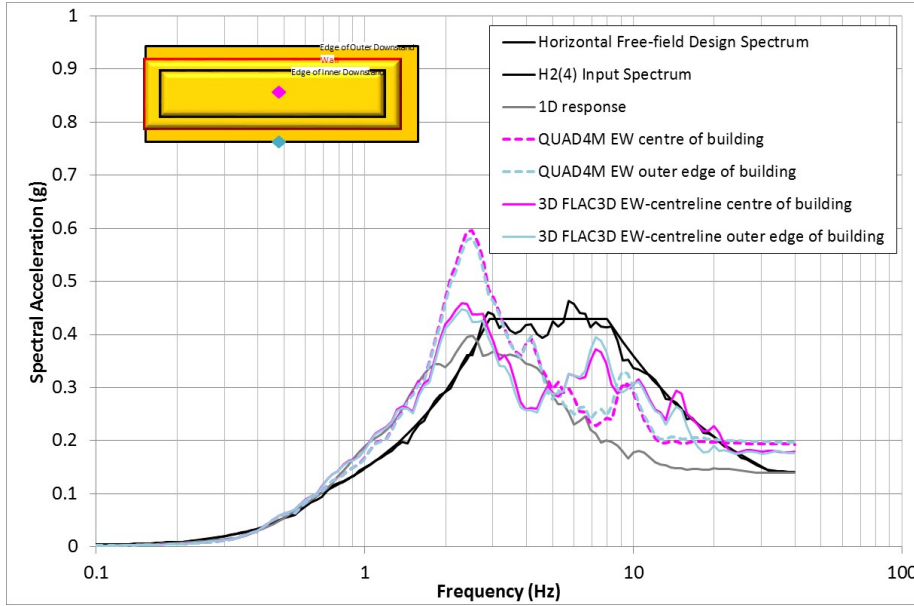


Figure 9. Horizontal East-West SSI response

Figure 10. Horizontal North-South SSI response

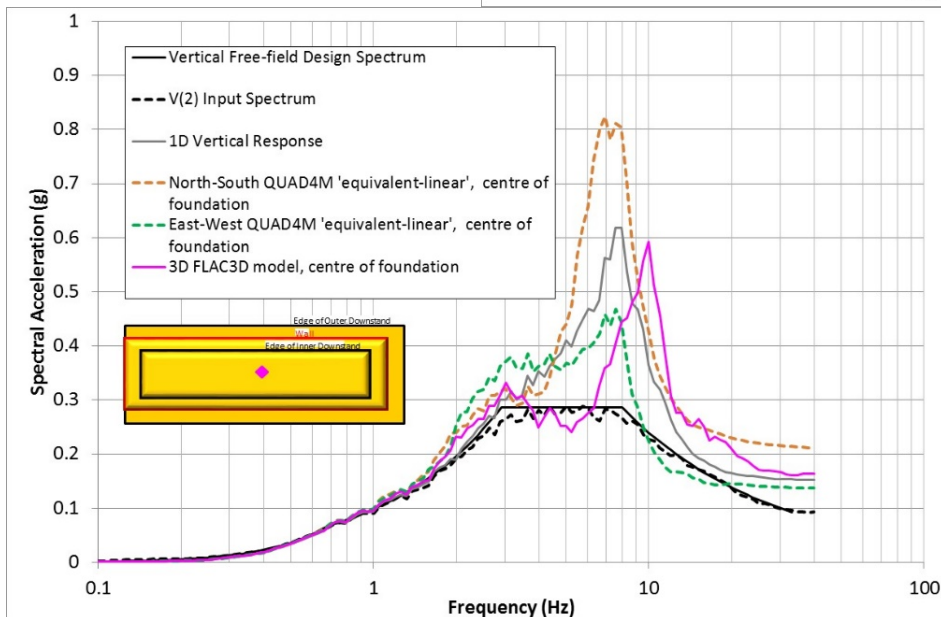
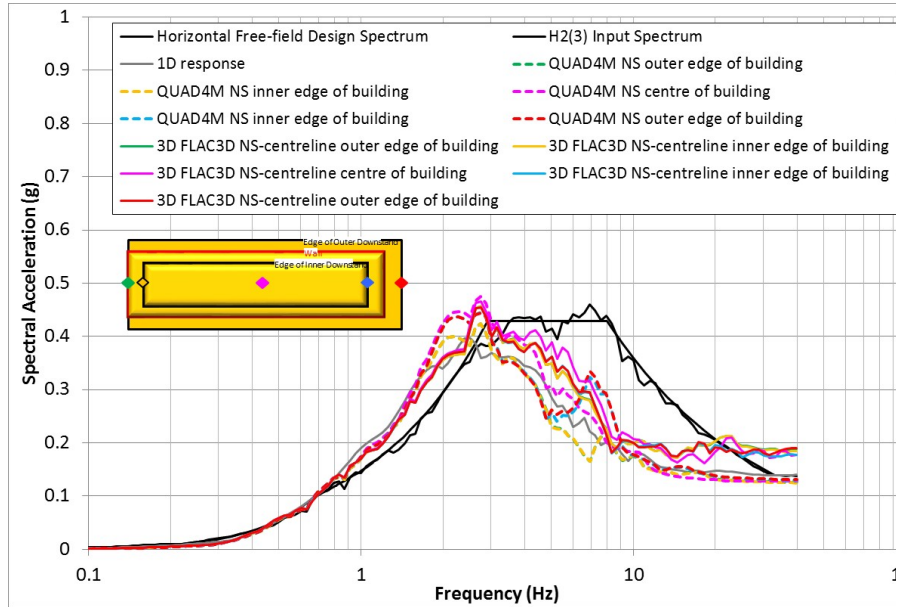


Figure 11. Vertical SSI response – centre of slab

Figures 9 and 10 show the horizontal response at the centre node in the 3D hysteretic FLAC3D model, compared with the north-south and east-west equivalent-linear QUAD4M models. This comparison shows a 30% reduction in peak horizontal response from the QUAD4M East-West response to that of the equivalent FLAC3D model. For the North-South response the 2D and 3D analyses give similar peak horizontal responses.

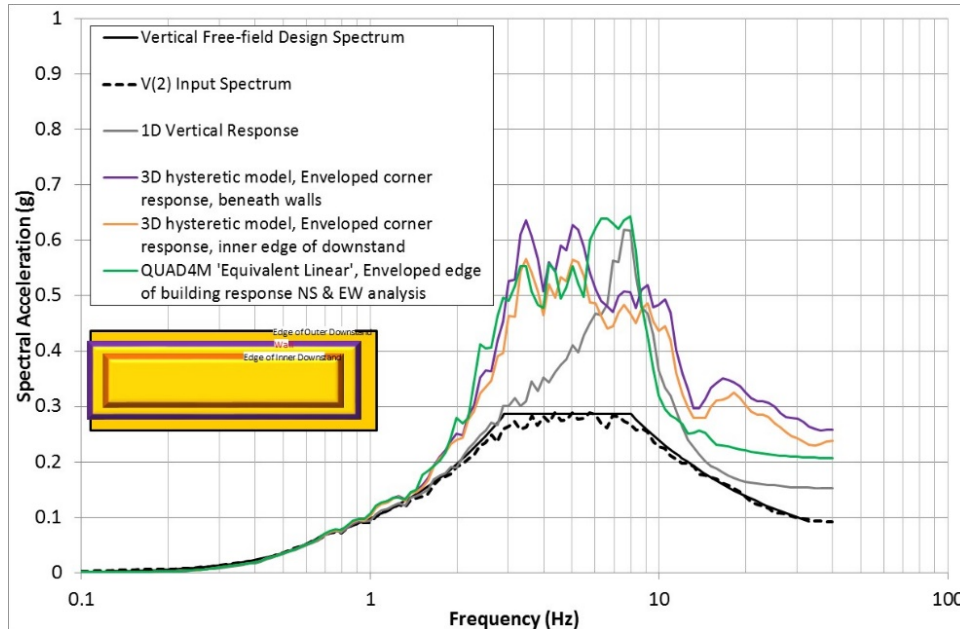


Figure 12. Vertical SSI response – edges & corners of slab

Figures 11 and 12 show the vertical response of the base slab at its centre and an envelope of the edge and corner responses respectively. The peak spectral response is seen in the centre of the slab for the 2D analysis with the longer North-South section providing a much larger response than the East-West section (Figure 11). The peak response from the 3D analysis lies mid-way between those of the two 2D sections. Vertical responses at the mid points of the edge of the slab are produced by both the 2D and the 3D analyses (Figure 12) and can be seen to be broadly similar. The 3D analysis also enables the responses of the corners of the slab to be examined. In this case the peak vertical spectral responses at the corners of the slab are about 10% higher than those at the mid side points of the slab. It may be noted that the frequencies of the peak responses at the corners of the slab are considerably lower than that of the response at the centre.

It can be seen from Figures 9 to 11 that the 1D approach has produced a generally conservative spectral envelope in comparison to the 2D and 3D analyses. The degree of conservatism is not substantial between the 1D and 2D models. However, the more refined 3D analysis shows appreciably lower responses for this building, especially for the vertical response mode.

### 3.4 Effect of groundwater levels on SSI

#### General Considerations

For buildings founded at relatively shallow depth on raft foundations it is quite likely that the water table will lie below the base of the underside of the foundation for some or all of the life of the structure. In the UK the ONR Safety Assessment Principles (HSE, 2008) require the effects of varying groundwater levels to be considered including the potential effects of climate change which may cause greater fluctuations in groundwater levels than occur under current climatic conditions. The position of the water table has only minor influence on the shear wave velocity ( $v_s$ ) of the soil. However, the velocity of compression waves (P-waves) is strongly influenced by the water table and more specifically by the degree of saturation of

the soil (Kontoe et al., 2013). In fully saturated soils the p-wave velocity ( $v_p$ ) tends towards 1450m/s, the typical  $v_p$  for water. In partially saturated soil Biot's Theorem (Biot, 1956) indicates that air content of fractions of a percent result in significantly lower  $v_p$  values, potentially less than 500m/s.

The presence of air in the soil pore space, even in small quantities, thus dramatically reduces the constrained modulus of the soil and is reflected in a large reduction in the dynamic Poisson's ratio ( $v_{dyn}$ ) of the soil. The relationship between these parameters is given by the following equations:

$$v_{dyn} = \frac{0.5v_p^2 - v_s^2}{(v_p^2 - v_s^2)} \quad \text{Eqn. 1}$$

$$G_0 = \rho v_s^2 \quad \text{Eqn. 2}$$

$$K_0 = \rho v_p^2 \quad \text{Eqn. 3}$$

Where  $G_0$  is the small strain shear modulus,  $K_0$  is the small strain constrained modulus of the soil and  $\rho$  is the bulk density of the soil. It can be seen from these relationships that if the value of  $v_p$  in partially saturated soil is reduced to say 30% of its value in saturated soil then the constrained modulus will be reduced by an order of magnitude. This change in stiffness can have a significant effect on the vertical amplification seen at foundation level.

A second effect also occurs due to the sharp change in  $v_p$  at the transition from partially saturated to fully saturated conditions. The velocity contrast can cause the p-wave to reflect between the underside of the foundation and the top of the water table also potentially enhancing the vertical amplification at foundation level.

It is likely that in granular soils the transition between the partially saturated and the fully saturated state occurs at the groundwater table. However, in cohesive strata the situation is not so straightforward. In these strata capillary action can draw pore water up several tens of metres above the water table. Depending on the air entry value of the clay, the soil may remain saturated for many metres above the water table. There is very little published data on the effects of ground water levels on  $v_p$  values in clay strata. Our own tentative observations base on in situ geophysical measurements suggest that there may be a zone of several metres where the value of  $v_p$  transitions from the fully saturated to the partially saturated value. This also is of significance to SSI analyses because the existence of a transition zone in the  $v_p$  profile reduces the potential for sharp reflections of the p-waves.

### **Site-specific study**

Studies have been carried out to determine the potential effect on the SSI response of variations in the groundwater levels at a site with a 4m thick sand and gravel formation overlying a clay formation. Parametric studies were carried out using a 1D SSI approach. At an early stage in the project design no data were available on the value of the dynamic Poisson's ratio in the granular stratum. In addition no analyses were available to constrain the credible range of the groundwater table under future climate change. Piezometer data were available that showed the water table remaining within the granular stratum and no lower than 1m above its base.

Two preliminary parametric studies were carried out using a 1D SSI approach. In the first of these the sensitivity to values of  $v_{dyn}$  was investigated and in the second the sensitivity to large changes in groundwater level was investigated.

For the sensitivity study on dynamic Poisson's ratio examination of some preliminary geophysical data suggested an upper bound to  $v_{dyn}$  of 0.38. A literature review suggested a

likely value of around 0.30 with a lower bound value of 0.20. The parametric study was carrying out using a PML free field seismic input spectrum and the results are shown on Figure 13. The reduction in  $\nu_{\text{dyn}}$  from 0.38 to 0.25 results in a 50% increase in in vertical peak spectral acceleration and a modest reduction in the associated frequency. Further reductions in  $\nu_{\text{dyn}}$  result in further reductions in peak frequency but no increase in peak spectral acceleration. Using the results from this study a value of  $\nu_{\text{dyn}}$  of 0.25 was adopted for the detailed design analyses pending results from the field testing. When the results of the field test became available they demonstrated a value for  $\nu_{\text{dyn}}$  of  $0.38 \pm 0.03$ . A final value of  $\nu_{\text{dyn}} = 0.35$  was considered to be appropriate for design and the analyses already carried out using  $\nu_{\text{dyn}} = 0.25$  were adopted as conservative.

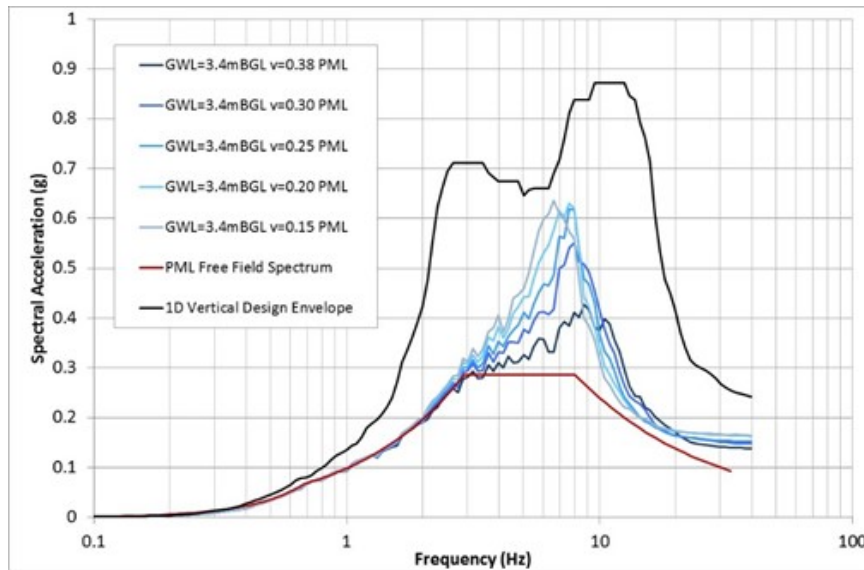


Figure 13 Vertical SSI response with change in dynamic Poisson's ratio

The sensitivity study on groundwater levels considered variations in groundwater table above and below the design basis level at the base of the granular stratum. Again 1D SSI analyses were undertaken. In these studies a PML and a UHS free field seismic input motion were used. The results of raising the groundwater level showed that vertical amplification decreased in this case. A small and a large decrease in the groundwater level were therefore also considered. The small decrease of 0.5m was judged to be a plausible decrease of the water table into the low permeability clay stratum. A much larger fall in the groundwater level of 12.6m was also considered as shown in Figure 14. The results show that small changes in groundwater levels have minimal effects on the seismic response. However, reducing the groundwater level by several metres has a substantial effect on the frequencies at which peak vertical responses occur. Subsequent hydrogeological studies on the behaviour of the sand and gravel aquifer under the most extreme predicted climate change demonstrated that the water table will remain within the granular stratum.

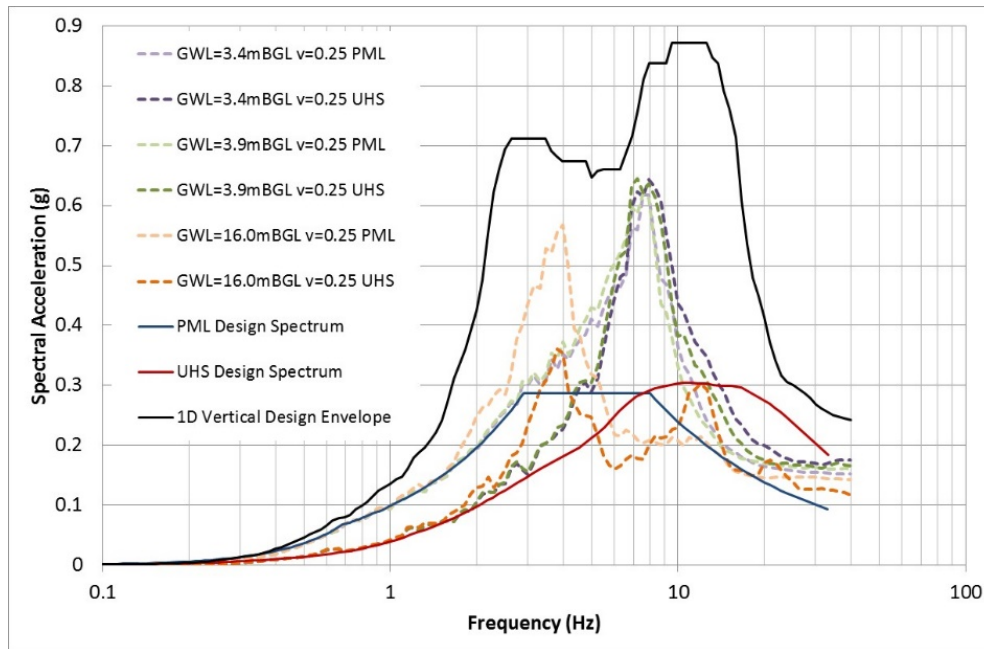


Figure 14  
Vertical SSI  
response with  
change of gwl

### 3.5 Rocking on Rigid or Elastic Foundations

The examples discussed in Sections 3.3 and 3.4 relate to structures with a relatively low height to width ratio and low centre of inertia. Some rocking effects can be seen in the SSI response across the foundation but these are relatively minor. However, for structures with greater height to width ratios the rocking behaviour may be a major design consideration.

It has long been recognised that the rocking of some types of structure in earthquakes has been a key factor in limiting seismic damage. Housner (1963) provided an early assessment of this behaviour, prompted by his observations of the performance of inverted pendulum structures (golf-ball water tanks) in the 1960 Chilean Earthquake. He made the important observation that a dynamic analysis rather than a pseudo-static analysis was required to explain the observed behaviour. Using a rigid soil model with consideration of separation between the foundation and the soil, he developed the equations of motion for analysing rocking blocks. Among his observations were explanations of why larger blocks of a given geometry should be more stable than smaller blocks (the rocking scale effect) and the significant energy dissipation generated in rocking with lift-off and subsequent impact. Psycharis and Jennings (1983) developed Housner's analysis with direct consideration of the foundation compressibility and damping. They proposed a Winkler spring model for the foundation response which they simplified to an equivalent pair of springs and dampers located towards the edges of the block. The apparent rocking period was shown to increase with the amount of lift-off while the apparent critical damping decreased. Numerous further studies of the rocking block problem have been presented (e.g. Makris and Roussos (2000), Palmeri and Makros (2008)) considering *inter alia* different aspects of ground motions, irregularity in the form of the rocking block, the effect of anchorages on the rocking block and the influence of soil stiffness and coefficients of restitution or damping.

## 4. Seismic Bearing Capacity

### 4.1 Introduction

Most seismic standards and other guides to good practice require that shallow foundations meet minimum pseudo-static factors of safety on bearing capacity. Such factors of safety are intended to prevent yield in the soil beneath the foundation. However, there is a growing recognition that the criteria which fundamentally determine the adequate performance of shallow foundations in earthquakes are the foundation displacements during and after the earthquake (e.g. Paolucci (1997), Priestley et al (2007), Pender (2014)).

### 4.2 Conventional Seismic Bearing Capacity

The current nuclear codes generally say rather little on the methods used to assess seismic bearing capacity leaving the designer to make a suitable choice. IAEA NS-G-3.6 provides a general expectation that the static factors of safety against bearing capacity failure should be high (not less than 3.0) and therefore there should also be “reasonable safety margins” under the design basis ( $10^{-4}$  p.a.) seismic loading. The guide also recommends that a minimum factor of safety of 1.5 should be achieved for the bearing capacity under seismic loadings. However, the definition of the factor of safety, the appropriate seismic loading and most details of the appropriate assessment method are not specified apart from the requirement to consider overturning. ASCE 4-98 provides extensive consideration of SSI but is silent on foundation bearing capacity. The more recent ASCE/SEI 43-05 addresses foundation sliding and overturning/rocking providing details of appropriate displacement-based methods of analysis. However, bearing capacity requirements are not addressed in detail. The code does indicate that limited (permanent) settlement may be considered in design and suggests that soil strength may be increased by  $\frac{1}{3}$  above the static strength for combined static and seismic loading.

Conventional bearing capacity theory for the static loading of soils dates back to the limit equilibrium analysis Terzaghi (1943). Notable developments of this theory were published by Meyerhof (1953) and Vesic (1975). Of particular relevance for seismic bearing capacity considerations, Meyerhof’s work included factors for the effect of inclined loading and an equivalent foundation area theorem for considering the effect of moments acting on the foundation. Thus the ultimate bearing capacity is given by:

$$q_{ult} = cN_c s_c i_c d_c + p_o N_q s_q i_q d_q + \frac{1}{2} \gamma B N_\gamma s_\gamma i_\gamma d_\gamma \quad \text{Eqn. 4}$$

Where:

- $cN_c$  is a component due to cohesion and friction
- $p_o N_q$  is a component due to surcharge and friction
- $\frac{1}{2} \gamma B N_\gamma$  is a component due to soil self-weight and friction
- $s_c, s_q$  and  $s_\gamma$  are shape factors
- $i_c, i_q$  and  $i_\gamma$  are inclination factors dependent on H/V
- $d_c, d_q$  and  $d_\gamma$  are depth factors dependent on foundation embedment
- H and V are the horizontal and vertical loading respectively

For foundations on clay soils subjected to rapid undrained loading the soil cohesion may be taken as the undrained shear strength ( $s_u$ ) and Eqn. 1 may be simplified as:

$$q_{ult} = s_u N_c s_c i_c d_c + p_o N_q \quad \text{Eqn. 5}$$

Meyerhof’s practical and conservative approach to incorporating the effects of moment loading (M) is to adjust the actual foundation width and length (B x L) to effective values (B’ x L’) to which the vertical and horizontal loading are applied:

$$B' = B - 2e$$

$$L' = L - 2e$$

Eqn. 6

Where the load eccentricity,  $e = M/V$

Potentially the above equations, Eqn. 4, 5 and 6, provide a solution for the assessment of the bearing capacity of shallow spread foundations under seismic loading. The seismic excitation will generate inertial forces in the structure, commonly imposing significant horizontal and moment loadings on the foundation together with increased or decreased vertical loadings.

However, considering the conventional bearing capacity equations more closely, it is apparent that the solutions for given values of  $H$  and  $M$  are expressed in terms of the ultimate vertical bearing capacity ( $q_{ult} = V/A'$ ) where  $A' = B' \times L'$ . For the seismic loading of foundations this does not provide a satisfactory reference to the factor of safety as typically the vertical load on the foundation does not vary greatly from the static value while the horizontal load and moment may for short periods increase very substantially. If a factor of safety is required it could be considered more appropriate to define it in terms of  $H$  and  $M$  rather than in terms of  $V$  (but see below on factors of safety).

Further consideration of the nature of the seismic loading indicates some problems with the application of conventional bearing capacity theory. Firstly the seismic motion will impart inertial forces to the soil as well as to the structure which are not accounted for in the conventional theory. Secondly the seismic component of the forces will vary rapidly during the earthquake with the time histories being independent in orthogonal directions. Hence the maximum seismic forces will not be applied simultaneously to the foundation. Thirdly the duration of the seismic loadings is very short with peak loading pulses of typically around 0.1 to 0.2 sec duration. This raises the question as to whether the foundation design should resist the peak seismic loadings or whether some limited foundation displacement can be permitted. If small foundation displacements can be permitted then a lower or factored value of the peak seismic loading may be appropriate for a pseudostatic design evaluation.

Various authors have published solutions which extend the classical bearing capacity theory with the inclusion of pseudostatic soil inertia forces including Sarma & Iossifelis (1990), Richards et al. (1993) and Kumar & Mohan Rao (2002). These solutions treat the foundation loading as consisting of the vertical load  $V$  and a horizontal load  $H = k_h V$  where  $k_h$  is the horizontal seismic coefficient. The same value of  $k_h$  is used to calculate the soil inertia force. This convention used in numerous papers on seismic bearing capacity is unfortunate. For most foundation problems the horizontal inertia loads imposed on the foundation by the supported structure will not be adequately represented as being equal to  $k_h V$ . An approach is required which treats the soil inertia independently from the foundation loads.

The common analytical approach to seismic bearing capacity suffers from not including the effects of applied moments which will usually accompany the horizontal loading. The moments may substantially reduce the ultimate seismic bearing capacity. The conventional approach also has the drawback that bearing capacity is formulated in terms of the ultimate vertical load ( $V_{ult}$ ). Under typical seismic loading conditions the vertical loads are known with a greater degree of certainty than the dynamic horizontal loads and moments. Hence solutions which enable the factor of safety on  $H$  and  $M$  to be evaluated directly are potentially more useful.

### 4.3 VHM formulation of bearing capacity

Salençon and Pecker (1995), Pecker and Paolucci (1996) and Paolucci and Pecker (1997a & b) adopted an alternative slip line approach to the determination of seismic bearing capacity. This work formed the basis of the seismic bearing capacity approach suggested in Eurocode 8-5. With this approach the inertial acceleration in the soil mass could be considered independently of the applied foundation loadings and hence the significance of these two aspects could be evaluated separately. An interesting finding was that soil inertia could have a substantial destabilising effect on foundation bearing capacity but only for cases where the initial static bearing capacity was abnormally low. For most normal cases where foundations are designed with a static bearing capacity factor of safety of 2.5 or higher, the soil inertia effects have minimal influence on the seismic bearing capacity. Pecker and Paolucci (1996) produced a bearing capacity formulation incorporating vertical load (V), horizontal load (H) and moment (M) similar to that developed for static bearing capacity by Gottardi and Butterfield (1994). Using VHM notation the Pecker and Paolucci (1996) formula is as follows for purely cohesive soils:

$$\frac{(\beta \bar{H})^2}{(\alpha \bar{V})^a (1 - \alpha \bar{V})^b} + \frac{(\gamma \bar{M})^2}{(\alpha \bar{V})^c (1 - \alpha \bar{V})^d} = 1 \quad \text{Eqn. 7}$$

Where:  $\bar{V} = \frac{V}{s_u B}$        $\bar{H} = \frac{H}{s_u B}$        $\bar{M} = \frac{M}{s_u B^2}$

a, b, c, d and  $\alpha, \beta, \gamma$ , are curve fitting functions

B = width of the foundation

$s_u$  = undrained shear strength

Note that the meaning of the terms H and V (i.e. horizontal and vertical load) in Eqn. 4 are not the same as those used in the original paper (and in Eurocode 8). The denominators and numerators in Eqn 4 can be modified to incorporate the effects of horizontal seismic soil inertia as in the expression given in Appendix F of Eurocode 8-5. However, Pecker and Paolucci (1996) indicate that this modification is unnecessary where:

$$\bar{F} = \frac{k_h \gamma_b B}{s_u} \leq 2 \quad \text{and} \quad \bar{V} \leq 2.5 \quad \text{Eqn. 8}$$

Where:  $k_h$  = the horizontal seismic coefficient

$\gamma_b$  = bulk unit weight of soil

Thus for many practical cases the seismic bearing capacity could be evaluated using static bearing capacity theory provided that the actions V, H and M are all adequately considered. Considerable effort has been expended in developing bearing capacity theory for offshore structures in recent years. It is therefore interesting to compare Eqn. 7 with a recent VHM formulation of the bearing capacity derived from finite element analyses primarily for offshore applications by Randolph and Gourvenec (2011):

$$\left( \frac{h}{h^*} \right)^2 + \left( \frac{m}{m^*} \right)^2 = 1 \quad \text{Eqn. 9}$$

Where:  $h = H/H_{ult}$        $m = M/M_{ult}$        $v = V/V_{ult}$

$h^* = 4(v - v^2)$  for  $0.5 < v < 1$        $h^* = 1$  for  $0 < v < 0.5$

$$m^* = 4(v - v^2)$$

The similarities in the basic form of Eqns. 4 and 6 are immediately apparent although the more recent form is undoubtedly more elegant. The advantage of using this type of VHM formulation is that the application of a load factor to the H and M terms provides an assessment of the factor of safety for a given value of V whereas this is more difficult to determine from classical bearing capacity theory.

#### **4.4 Acceptance Criteria for Seismic Bearing Capacity**

Equations 4, 7 and 9 have all been used by the authors for seismic bearing capacity evaluations of the foundations of nuclear structures. Despite the improved consideration of the effects of moments and horizontal loading in the VHM formulations there remains a lack of guidance on appropriate methods for incorporating the rapidly fluctuating loading resulting from an earthquake into the pseudostatic bearing capacity evaluation for nuclear structures. As noted by Paolucci and Pecker (1997b), if the peak transient loadings acting on the foundation are considered (including especially the moment effects) strong reductions in bearing capacity are predicted for many typical foundations under seismic loading. However, observations of shallow foundation performance in earthquakes shows that in general their performance is satisfactory even when back-analysis indicates that “failure” should have occurred. The explanation appears to lie mainly in the transient nature of the loading. The foundation may start to displace under the transient loading but the duration of the loading is too short to generate damaging permanent displacements. In many structures sliding displacements or settlements of a few centimetres would be tolerable.

The Eurocode 8-5 formulae for seismic bearing capacity permit a limited amount of sliding displacement to be considered and implicitly allow for some permanent deformation in the partial model factors ( $\gamma_{Rd}$ ) suggested for seismic bearing capacity analysis, see Pecker and Pender (2000). (It may be noted that Eurocode 8-2 explicitly prohibits designing for this type of foundation behaviour). Methods for undertaking such displacement assessments are not yet well established. Richards et al. (1993) provided an approximate method of assessment based on a two wedge idealisation of the bearing capacity failure surfaces leading to a seismic settlement calculated on the basis of a Newmark sliding block analogy similar to that used for assessing the displacements of gravity retaining walls. While this represents a good start, more work needs to be done on rocking foundations (e.g. Knappett et. al., 2006) supported with experimental data before a reliable method of assessing foundation displacements under seismic loading can be fully established.

The IAEA (2004) requirements for static and pseudo-static factors of safety provide a conventional and conservative set of acceptance criteria for the foundations of nuclear structures. A minimum factor of safety of 1.5 is required under a pseudo-static seismic load. This is presumably intended to ensure non-plastic behaviour of the soil. Further comments on the appropriateness of this requirement are given in the following section.

## 5. Foundation Displacements

### 5.1 Introduction

Most codified approaches to the design of shallow foundations under seismic loading seek to provide a factor of safety against failure for the foundation under pseudostatic loading conditions. Examples include Eurocode 8 (BSI 2004) which requires foundations to remain elastic under the applied seismic action and NS-G-3.6 (IAEA, 2004) which requires a minimum factor of safety of 1.5 against bearing capacity failure under the design seismic loading. However, there is a substantial body of evidence that indicates that small permanent displacements of shallow foundations can safely be accommodated in structures and also that designing for such displacements may increase the resilience of the supported structure. Indeed it is routine to design slopes affected by seismic loading on the basis of their permanent displacement. Contributions in this field include Newmark (1965), Ambraseys and Menu (1988), Rathje and Bray (2000) and Bray and Travararou (2007). The design of gravity retaining walls also routinely incorporates allowance for translation and rotation of the wall. Assessment of the sliding of gravity retaining walls were made by Richards and Elms (1979), Whitman & Liao (1985), Steedman and Zeng (1990 and 1996) and Steedman (1998).

Deterministic assessments of wall displacements using time histories, amplification of ground motions and rotation together with translation have been performed for the design of various major projects including for example the seismic retrofit of Devonport Dockyard, Ibrahim and McCarthy (2002) and Taylor et al (2011).

For some major structures such as the 90 m base diameter main piers of the Rion-Antirion Bridge, sliding and rocking behaviour has been deliberately incorporated into the foundation design, Pecker & Teyssandier (1998). However, it is not yet routine to account for limited foundation displacements in the seismic design of structures and for the most part seismic codes do not encourage such considerations.

### 5.2 Sliding Displacements

In general sliding behaviour is most likely to be of interest in the case of shallow raft foundations supporting stiff structures with a low height (H) to width (B) ratio (say  $H/B < 1$ ). Such structures typically have a very high static factor of safety against vertical bearing capacity failure. Moment effects on such foundations are rather small and hence the load path generated by the transient horizontal loading is likely to reach the foundation VHM yield surface in the sliding section (Figure 15). However, some care is required to calculate an appropriate nominal vertical capacity ( $V_{ult}$ ) and to confirm the angle of shearing resistance  $\delta$  between the foundation and the soil in order to avoid significant over- or under-estimates of the horizontal resistance.

The case of foundation sliding may be simply addressed by treating a shallow foundation as a sliding block and using the Newmark (1965) approach to calculate displacements using a time-stepping analysis when the transient lateral accelerations induce shear forces which exceed the base resistance of the foundation. The practical application is somewhat complicated by the embedment of most real foundations which give rise to passive resistance ahead of the foundation and lateral resistance on the sides of the foundation. There is also a likelihood that the resistance on the sliding surface will reduce due to remoulding or post-peak contractile behaviour. Once a gap has opened on one side of the foundation there will be less resistance to the foundation sliding back in that direction than for it to continue in the direction of increasing passive resistance. This will induce some tendency for the embedded foundation to self-centre as it displaces under the cyclic earthquake excitation.

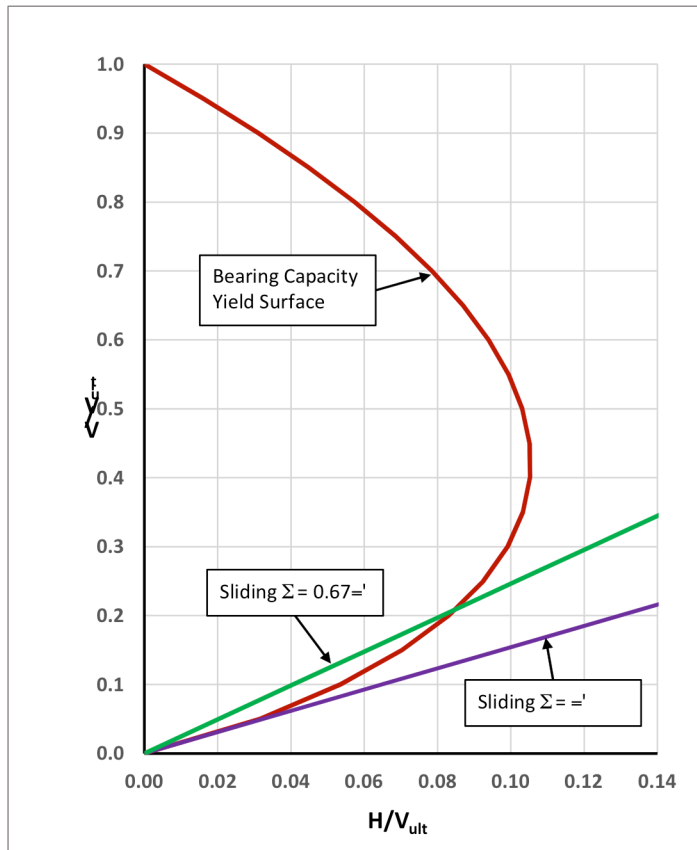
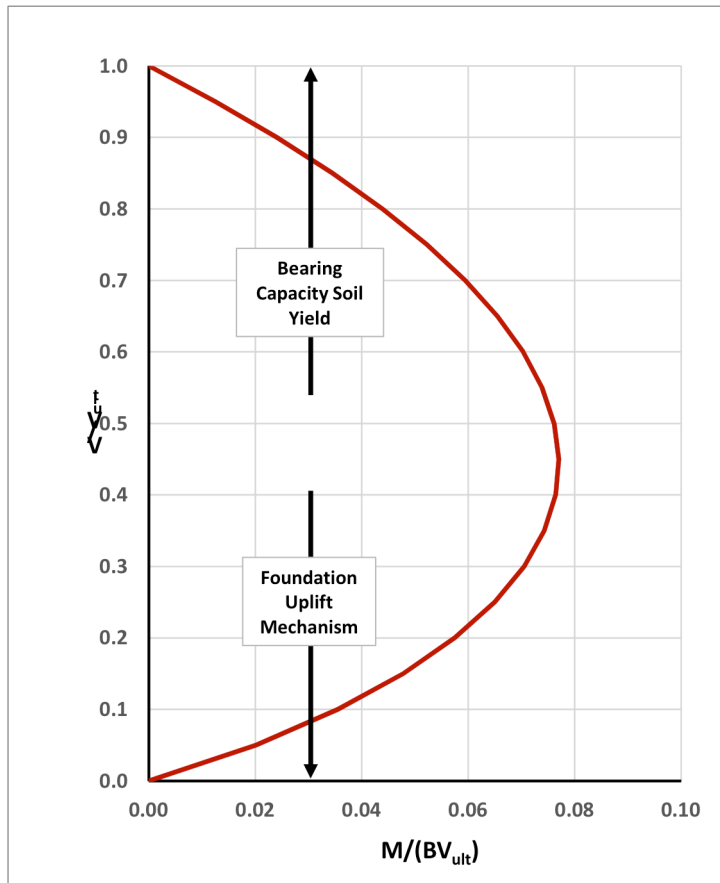


Figure 15. Sliding and Bearing Capacity Yield Surface for varying V & H (M constant)

Undertaking this type of analysis it becomes clear that the calculated displacements are likely to be sensitive to the details of the exciting time history. Significantly different displacements may be generated by artificial time histories each of which comply with the same target spectrum and duration envelope. When performing time domain analyses of structures it used to be commonplace to generate “white noise” time histories which had little resemblance to actual earthquake motions other than their duration and spectral content. More recent work has used suites of actual time histories of the required magnitude and scaled zero period acceleration. Selection procedures are discussed by Katsanos et al (2010). An alternative is to modify real records of the appropriate magnitude to match the design spectra, e.g. Hancock et al (2006).

### 5.3 Bearing Capacity and Rocking Displacements

In the case of pad footings or strip footings supporting light structures it is likely that the vertical static factor of safety ( $F_s$ ) against bearing capacity failure will be in excess of 2.5 in order to limit settlements. The maximum moment resistance in the soil for such foundations is realised at a vertical load ( $V$ ) of about half  $V_{ult}$  (i.e. at  $F_s = 2$ ) as shown in Figure 16. Gazetas and Apostolou (2004) and Gazetas (2015) show that for the usual case of  $F_s > 2$  uplift is likely to occur (i.e. the foundation rocks with temporary detachment at the edges). If  $F_s < 2$  a full rotational bearing capacity “failure” mechanism is likely to be mobilised without foundation uplift.



A considerable number of numerical and experimental studies have shown that the rocking behaviour with uplift tends to self-centre due to  $P-\delta$  effects at the end of each cycle of uplift thus resulting in small residual rotations.

The experimental evidence indicates that foundation rocking on granular soils takes place without appreciable loss of strength for the limited number of cycles applicable to earthquake loading. Analytical and field testing of rocking foundations show that substantial damping is mobilised on each cycle. Pender (2014) includes a summary of the snap-back testing which he and his team have developed for dynamic testing of full-scale shallow foundations. These field tests demonstrate several interesting features including the degradation of rocking stiffness with increasing numbers of cycles. The observed damping behaviour was not typical of classical viscous systems but was better described by Housner's (1963) concept of energy loss on reversal of rocking direction.

Additional advantages in designing shallow foundations to rock are that the forces transmitted to the structure are substantially reduced, potentially reducing the moment demand at the column-foundation joint to about half that of a non-rocking foundation. Also the rotation of a rocking foundation significantly lengthens the period of the soil-foundation-structure system (Ton and Pender, 2008), even for quite modest levels of rotation. This is usually advantageous for light flexible structures moving their fundamental period away from that of the earthquake motion.

However, foundation rocking does induce settlement (Pender, 2014 and Gazetas 2015). For foundations with a rather low initial  $F_s$  such settlements can be quite large. Building damage can be severe if large differential settlements develop between footings. Hence it will normally be prudent to limit total (and therefore differential) settlements by providing an initial static value of  $F_s \geq$  about 4 for shallow footings.

#### **5.4 Plastic Foundation-Soil Macro-elements**

Although the concepts and studies described in Section 5.3 are useful for preliminary sizing of shallow foundations, for detailed seismic design of structures supported on pad or strip footings it is necessary to incorporate the plastic soil-foundation behaviour adequately into the structural design. This can be undertaken with sophisticated 3D finite element representations of the structure, foundation elements and soil but this is computationally demanding and costly making it unattractive for routine or even quite sophisticated design-office work.

A more pragmatic approach is to incorporate the elasto-plastic behaviour of the soil and foundation into a user-defined macro-element. Typically this can be achieved using the VHM bearing capacity concepts discussed in Section 4 with assessment of the plastic displacements generated when the load path has reached the VHM yield surface. In order to calculate the displacements both the yield surface and the flow rule or plastic potential have to be defined. Most such models have considered moments and horizontal forces acting on one plane through the problem whereas for the prototype foundation overturning moments and horizontal forces may act in two planes simultaneously and a torque may also be applied. (For a 6 degree of freedom model see Bienen et al 2006.) Solutions typically consider the soil behaviour to be linear elastic when the load path lies inside the yield envelope and consider that no hardening occurs when the load path lies on the yield envelope (i.e. the soil response is considered to be linear elastic-perfectly plastic). These are both significant simplifications which may nevertheless be justified by the capture of the major elements of the foundation response.

Paolucci (1997) presented a VHM model for modelling shallow foundation response under seismic loading. No allowance was included for foundation uplift (rocking). Several authors have developed Paolucci's approach including Cremer et al (2001) and Toh and Pender (2008). The selection of an appropriate flow rule is important in these models because the flow rule controls the plastic displacement vectors. Alternative flow rules were discussed by Toh and Pender (2008) who concluded that the flow rule chosen by Cremer produced more realistic patterns of behaviour than that adopted by Paolucci (1997). In particular Cremer's definition results in yield producing a component of settlement rather than sometimes generating net uplift.

It is possible to capture some of the aspects of the VHM plasticity models using non-linear spring models. For example Anastasopoulos and Kontroupi (2014) demonstrate that good results for rocking foundations can be achieved by the use of a non-linear rotational spring together with linear horizontal and vertical springs and a rotational dashpot. Pender (2014) illustrates spring bed analyses for the assessment of rocking buildings and suggests amendments to the FEMA 356 approach which appears to produce excessively stiff foundation moment-rotation curves. These models are certainly useful, however for detailed design the VHM-based plasticity models offer a much fuller and more realistic representation of soil-foundation behaviour.

#### **5.5 Displacement Criteria**

There are various reasons why the engineering profession has been slow to embrace displacement-based seismic design for structures. One of these is the lack of agreed criteria for acceptable or limiting permanent displacements. This shortcoming was recognised by the Canterbury Earthquakes Royal Commission (2012) which called for the establishment of acceptable levels of post seismic foundation deformations. Pender (2014) provided a tentative response, suggesting the following limiting residual foundation deformations:

- residual building tilt of 0.2 degrees (approx. 3 mrad)
- residual settlement increment of up to 50% total static settlement
- residual horizontal displacement equal to residual settlement

It could be observed that the criteria would be different depending on the required outcomes. Those suggested by Pender might be suitable for the serviceability limit state (perhaps the OBE). If life-safety is the concern (for example at the MCE) then it is likely that tilting in excess of 3 mrad would be tolerable. Limits would be required on the differential settlement of footings and these might be defined by tolerable differential displacements for beam supports with properly detailed plastic hinges. It might also be prudent to limit the differential lateral displacement of foundation elements, noting that codes such as Eurocode 8 include a pragmatic requirement for tie beams.

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