8. SEISMIC RESPONSE ASPECTS FOR DESIGN AND ASSESSMENT

8.1 Introduction (Jim)

8.2 Soil modelling

As explained in chapter 3 of the document soil characterization is a complex task and, depending on the choice of the soil constitutive model used for the analyses, the number of parameters to determine may vary to a large extent and degree of complexity (see table 3.3). Therefore, it is important that the level of efforts put in the determination of the soil characteristics be adapted to the needs without overshadowing the essential features of soil behaviour. In any case, it is essential that soil characteristics be determined by site specific investigations including field tests and laboratory tests, which should, as far as possible, yield coherent soil characteristics; any incoherence should be analysed and explained. Laboratory tests and field tests shall not be opposed to each other but used in combination since each of them has its own merit, limitation and range of applicability (see Figure 3.10). Special attention must be paid to the characterization of manmade backfills for which the characteristics can only be measured and determined provided enough specifications are available in terms of material source, identification, and compaction.

In several regions of the world the design earthquake may represent a moderate level earthquake which will induce only small to moderate strains in the soil profile. Typically, an earthquake with PGA of the order of 0.2–0.3g may be considered as a moderate event; however, as explained below and later in section 8.3.2.1, PGA should only be regarded as a rough proxy for classifying the earthquake as moderate or high and induced strains should definitely be considered. In others, highly seismic, regions the design earthquake may represent a strong motion event. These features should be considered when defining the soils parameters and associated investigations needed for design. In the first instance (moderate event), and as indicated in section 3.3, the most appropriate constitutive model will be the equivalent viscoelastic linear model (EQL); as pointed out above the decision to use an EQL model should not be based only on PGA and the right indicator to use is the shear strain. This model, which represents the state of practice, is simple enough to be amenable to the large number of parametric and sensitivity analyses required to account for the variability of soil properties (see section 8.6.2). It must be remembered that the uncertainty on the elastic properties is not the single parameter that needs to be considered: large uncertainties exist in the determination of the nonlinear shear stress–shear strain curves (or equivalently $G/G_{\text{max}}$ and damping ratio curves used to define the equivalent linear model). It is strongly recommended to measure these curves on undisturbed samples retrieved from the site and not to rely exclusively on published results; however, comparisons with published results are useful to define the possible variation of the curves and to assess the possible impact of such variations on the site response. It would be very uncertain to attempt to relate the domain of validity of the EQL model to some earthquake parameter (like PGA) since the strains also strongly depend on the material: some materials are “more linear” than others (e.g. highly plastic clay). However, for a preliminary estimate, PGA’s less than 0.2 –0.30g may be considered as moderate earthquakes for which the EQL model is relevant. However, in general, validity of the equivalent linear model has to be checked at the end of the analyses by comparing the induced shear strain to a threshold strain beyond which the constitutive
model is no longer valid. Chapter 3 has proposed to fix that threshold strain to twice the
reference shear strain (see chapter 3 for definition of the reference shear strain) and one
example in the appendix on site response illustrates this aspect (see also section 8.3.2); note
also that, if in a 1D model the definition of the shear strain is straightforward, in a 3D situation
it is proposed to define the “shear strain”, to be compared to the threshold strain, as the
second invariant of the deviatoric strain tensor.

In highly seismic areas, it is most likely that the induced motions will be large enough to induce
moderate to large strains in the soil profile. Therefore, the EQL model may not be appropriate to
represent the soil behaviour. True nonlinear soil models are required to analyse soil structure
interaction response. As indicated in chapter 3, numerous nonlinear models exist and the choice
cannot be unique; it is strongly recommended that at least two different constitutive models be
used by possibly two different analysts. The models should be validated for different stress paths,
not only with respect to shear strain–shear stress behaviour but also with respect to volumetric
behaviour, and their limitations should be fully understood by the analysts. Furthermore, it is
highly desirable, although not mandatory, that the models possess a limited number of
parameters easily amenable to determination and be based on physical backgrounds. As it is well
known that soil response is highly sensitive to the chosen model, it is essential that uncertainty in
the parameters, especially those with no physical meaning, be assessed through parametric
studies. An example of a nonlinear constitutive model is described in the appendix along with the
examples on site response analyses.

8.3 Free field ground motions

8.3.1 Approaches 1, 2, 3 (Jim)

8.3.2 1D model

A 1D soil column is used to develop examples, presented in appendix 1, illustrating the differences
between the various approaches to 1D site response analyses. The soil profile consists of 30.0m
of sandy gravel overlying a 20m thick layer of stiff, overconsolidated, clay on top of a rock layer
considered as a homogeneous halfspace (Figure 8-1). The water table is located at a depth of
10.0m below the ground surface. The incident motion is imposed at an outcrop of the halfspace
in the form of an acceleration time history. The soil constitutive models include an equivalent
linear model, a nonlinear model for 1–phase medium and a nonlinear model for 2–phase
(saturated) medium.

Under the assumption of vertically propagating shear waves, the numerical model is a one-
dimensional geometric model; however, to reflect the coupling between the shear strain and
volumetric strains each node of the model possesses two (1-phase medium or 2-phase undrained
layer) or four (2-phase medium pervious layer) degrees of freedom corresponding to the vertical
and horizontal displacements (respectively vertical and horizontal translations of solid skeleton
and vertical and horizontal velocities of the fluid).

The purposes of the analyses are to:

- Compare equivalent linear and nonlinear constitutive models;
- Show the differences between total vs effective stress analyses;
Show for a 2-phase medium the impact of the soil permeability;
Compare the predicted vertical motion assuming P-wave propagation to the motion calculated from the horizontal motion with V/H GMPEs (Gülerce & Abrahamson).

Figure 8.1: Soil profile for illustrative examples on 1D site response analyses

Results are compared in terms of 5% damped ground surface response spectra, pore pressure evolution in time at mid depth, horizontal and vertical displacements at the ground surface.

The following Table 8–1 summarizes the different analysed cases. All the nonlinear analyses are run with the software Dynaflow; the equivalent linear analyses are run with SHAKE.

<table>
<thead>
<tr>
<th>Case</th>
<th>1a – 1b</th>
<th>2a – 2b</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum</td>
<td>1-Phase</td>
<td>1-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
</tr>
<tr>
<td>Model</td>
<td>Total stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
</tr>
<tr>
<td>Constitutive relationship</td>
<td>Equivalent linear/ nonlinear</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>-</td>
<td>0</td>
<td>$10^3$</td>
<td>$10^4$</td>
<td>$10^3$</td>
<td>$10^2$</td>
</tr>
<tr>
<td>Input motion</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
<td>Horizontal</td>
</tr>
<tr>
<td>Software</td>
<td>Shake</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
</tr>
</tbody>
</table>
Table 8-1: Summary of analysed cases

<table>
<thead>
<tr>
<th>Case</th>
<th>6a – 6b</th>
<th>7a – 7b</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum</td>
<td>1-Phase</td>
<td>1-Phase</td>
<td>2-Phase</td>
<td>2-Phase</td>
</tr>
<tr>
<td>Model</td>
<td>Total stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
<td>Effective stresses</td>
</tr>
<tr>
<td>Constitutive relationship</td>
<td>Equivalent linear/linear</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
<td>Elastoplastic</td>
</tr>
<tr>
<td>Permeability (m/s)</td>
<td>-</td>
<td>0</td>
<td>$10^{-5}$</td>
<td>$10^{-2}$</td>
</tr>
<tr>
<td>Input motion</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
<td>Horizontal + vertical</td>
</tr>
<tr>
<td>Software</td>
<td>Shake Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
<td>Dynaflow</td>
</tr>
</tbody>
</table>

8.3.2.1 Comparison EQL / NL (cases 1a–1b)

The example is used to point out that, beyond some level of shaking, EQL solutions are not valid. Figure 8-2 illustrates the comparison in terms of ground surface response spectra calculated for 3 increasing amplitudes of the input motion. This figure and additional figures presented in the appendix show that, as long as the input motion is not too strong (here $p_{ga} \sim 0.20g$), the EQL and NL solutions do not differ significantly. For $0.25g$ differences start to appear in the acceleration response spectra: high frequencies are filtered out in the EQL solution and a peak appears at 3Hz corresponding to the natural frequency of the soil column. At $0.5g$ the phenomena are amplified with a sharp peak at 2.8Hz in the EQL solution. Filtering of the high frequencies by the EQL analysis has been explained in chapter 3.4: they are dumped because the damping ratio and shear modulus are based on the strain, which is controlled by low frequencies, and the same damping is assigned to all frequencies. High frequency motions induce smaller strains and therefore should be assigned less damping.
The value of the pga threshold should not be considered as a universal value: it depends on the material behaviour; as explained in the main document (section 3.3.1), the fundamental parameter to look at is the induced shear strain, or better the reference shear strain $\gamma_r$. The maximum shear strain calculated as a function of depth for each run is plotted in Figure 8-3 below. When the amplitude of the input motion is smaller than 0.20g, the maximum induced shear strain remains smaller than $10^{-3}$, which was indicated as the upper bound value for which equivalent linear analyses remain valid and, indeed, equivalent linear and nonlinear analyses give similar results. When the amplitude of the input motion is equal to 0.50g, the maximum induced shear strain raises up to $3.4 \times 10^{-3}$ at 20m depth; at that depth the reference shear strain is equal to $10^{-6}$ (calculated from table 3 in the appendix) and therefore the induced shear strain is larger than two times the reference shear strain. For an input motion of 0.25g, the maximum shear strain at 20m depth is approximately equal to $2\gamma_r$ and equivalent linear analyses and nonlinear analyses start to diverge.
8.3.2.2 Total vs effective stress analyses (cases 1a–2a)

The analyses presented in the appendix show that at low level of excitation (pga ≤0.25g) both solutions (total or effective stress analyses) are comparable except of course for the pore pressure build up which cannot be predicted by the total stress analysis. At pga =0.50g differences appear in the acceleration response spectra and in the vertical displacements (Figure 8-4, left).

It can be concluded that effective stress analyses are not needed for low level of excitation but are important for high levels, when the excess pore pressure becomes significant.

For an impervious material the effective stress analyses carried out assuming either a 1-phase medium or a 2-phase medium (case 2a–2b) do not show any significant difference (Figure 8-4, right). Based on the results of other analyses, not presented herein, this statement holds as long as the permeability is smaller than approximately 10⁻⁴ m/s. Therefore 2-phase analyses are not required for those permeabilities.

The impact of the value of the permeability appears to be minor on all parameters except the excess pore pressure (Figure 8-5). It is only for high permeabilities (10⁻²m/s) and high input excitations that differences appear in the acceleration response spectrum and, to a minor extent, in the vertical displacement.
Figure 8-4: Comparison of ground surface response spectra for effective and total stress analyses (left) and 1–Phase versus 2–Phase medium (right)
8.3.2.3 Vertical motion

The previous analyses are run with a single, horizontal, component for the input motion. The vertical component of the ground motion is often assumed to be caused by the vertical propagation of P waves (section 5.3.1); as explained in section 6.3.1 and 6.3.3 the vertical motion cannot usually be assumed to be created only by the vertical propagation of P waves: Rayleigh waves, diffracted P–SV waves also contribute to the response. In the present analysis the two (horizontal and vertical) components of motion are input simultaneously in the model since the material behaviour is nonlinear. From the calculated spectra at the surface, $V/H$ ratios have been computed and compared to Gülerce–Abrahamson prediction equation for a magnitude 6.9 event recorded at 15km from the source (Joyner–Boore distance). Results, presented in Figure 8-6 clear show that for frequencies less than 10Hz the vertical motion can be predicted assuming vertical propagation of P waves; however, for higher frequencies the vertical motion cannot be assumed to be created only by the vertical propagation of P waves: Rayleigh waves, diffracted P–SV waves also contribute to the response.
### 8.3.2.4 Lightly damped profiles

The purpose of this example is to show that for nearly elastic materials, the choice and modelling (hysteretic, Rayleigh) of damping is critical for the site response. The “soil” column is composed of one layer of elastic rock material (2km thick with $V_S=2000\text{m/s}$) overlying a halfspace with $V_S=3000\text{m/s}$. An elastic model is used for the rock because, for such a high shear wave velocity, significant nonlinear degradation of the shear modulus cannot be expected; furthermore, there is no experimental evidence nor reliable and well documented curves for the modulus degradation curve and damping curve of hard rocks. It is subjected to a real hard rock record recorded in Canada and provided by Gail Atkinson: record OT012-HNG with a duration of 60s and a maximum acceleration equal to $0.03\text{g}$; although pga is very small it has not been scaled up since all calculations presented in this example are linear. The record, its 5% damped response spectrum, and its Fourier amplitude spectrum are shown in the appendix.

Several methods are used for the calculations and illustrated on the rock column:

- Pure elastic calculation with a time domain solution obtained with Dynaflow and two meshes: one with ten elements per wave length (element size 5m) and one with 20 elements per wave length (element size 2.5m); differences between both meshes are shown to be negligible and only the results with 10 elements per wave length are presented;

- Pure elastic calculation in the frequency domain with the Exponential Window Method (EWM) developed by Eduardo Kausel and introduced in the TECDOC (section 3.4);

- Viscoelastic calculation with 0.1% damping in the rock layer (the halfspace is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM;
Viscoelastic calculation with 1% damping in the rock layer (the halfspace is still undamped) with a frequency domain solution: classical FFT (SHAKE) and EWM;

Numerical damping in the time domain analysis with Dynaflow (Newmark’s parameter $\gamma$ set equal to 0.55 instead of 0.50 for no numerical damping);

Rayleigh damping (stiffness proportional) in the time domain analysis calibrated to yield 1% damping at two times the fundamental frequency of the layer, i.e. 0.5Hz.

Results are presented in terms of 5% damped ground surface response spectra (Figure 8-7).

Figure 8-7: Influence of damping modelling and numerical integration method on ground surface response spectra

Surface motions are very sensitive to low damping values, in the range 0% to 0.1%. For pure elastic calculation either the time domain solution (without numerical damping) or the EWM should be used; the agreement is good up to 25Hz; above that value they slightly differ but this may be due to filtering by the mesh in the time domain solution. For very lightly damped systems (0.1%), there is only one reliable method, the EWM; damping cannot be controlled in the time domain solution and the classical FFT overdamps the frequencies above 8Hz (in that case). For lightly damped systems (1%), the classical FFT and the EWM perform equally well. However, the EWM is much faster and does not require trailing zeroes to be added to the input motion; the duration of this quiet zone might be a cause of errors in FFT calculations if not properly chosen. Finally, Rayleigh damping should never be used for damped systems in time domain analyses; it might even be better to rely on numerical damping, but the exact damping value implied by the choice of the Newmark integration parameter is not known because it is frequency dependent (proportional to frequency for the present analysis).

Last but not least, from a practical standpoint, soils or rock with very low damping represent a very critical situation because the exact amount of damping in very stiff rock (0%, 0.1%, 1% ?) will never be known (or measured) with sufficient accuracy and the results are very sensitive to this choice above 1Hz.
### 8.3.3 2D models

This example is presented to outline the importance of topographic effect. Motions are calculated at the location in the middle of a valley (see figure in the appendix) where a marked topography exists. Calculations are made assuming:

a/ a 1D model, extracted from the soil column at the examined location, and an equivalent linear constitutive model,

b/ a 2D model, including the whole valley shown in the figure, with the strain compatible soil properties retrieved from the 1D–EQL analyses,

c/ the same 2D model as above but with a fully nonlinear constitutive model for the soil.

Ground surface response spectra calculated for these three assumptions are depicted in Figure 8-8.
The calculated surface spectra clearly indicate that the 1D model is unable to predict the correct answer except for long periods, above 1.5s; at these periods, scattering of the incoming wave by the topography is insignificant. They also indicate that the main difference between the spectra arise from the geometric model rather than from the constitutive model, although the 2D linear model should be regarded with caution because damping is modelled as Rayleigh damping while in the 2 other analyses frequency independent damping is considered.

### 8.3.4 Three Dimensional Seismic Motions and Their Use for 3D and 1D SSI-Modeling

A set of simulations is developed to address differences between results if full 3D (three dimensional geometry) and 3C (three components) and 3D, 1C (one component) motions are used. These differences are addressed as common practice is to use 3D/1C motions, while in reality all the motions are 3D/3C, and even 3D/6C, where 6 components include three translations and three rotations. Use of 3D/1C motions can create a problem if vertical motions are modelled.

A generic NPP model is used to illustrate those differences. A more detailed analysis is provided by Abell et al., 2018.

Assume that a small scale regional models is developed in two dimensions (2D). Similar model can of course be developed in 3D, but for the purpose of this example, 2D is sufficient to show differences in response. Model consists of three layers with stiffness increasing with depth. Model extends for 2000m in the horizontal direction and 750m in depth. A point source is at the depth of 400m, slightly off center to the left. Location of interest (location of an NPP) is at the surface, slightly to the right of center.

Figure 8.96.9 shows a snapshot of a full wave field, resulting from a small scale regional simulation, from a point source (simplified), propagating P and S waves through layers. Wave field is 2D in this case, however all the conclusion will apply to the 3D case as well.

It should be noted that regional simulation model shown in Figure 8.96.9 is rather simple, consisting of a point source at shallow depth in a 3 layer elastic media. Waves propagate, refract at layer boundaries (turn more “vertical”) and, upon hitting the surface, create surface waves (in this case, Rayleigh waves). In our case (as shown), out of plane translations and out of plane
rotations are not developed, however this simplification will not affect conclusions that will be drawn. A seismic wave field with full 3 translations and 3 rotations will only emphasize differences that will be shown later.

Assume now that a developed wave field, which in this case is a 2D wave field, with horizontal and vertical translations and in plane rotations, are only recorded in one horizontal direction. From recorded 1D motions, one can develop a vertically propagating shear wave in 1D, that exactly models 1D recorded motion. This is usually done using de-convolution (Kramer 1996).

It is important to note that the horizontal motions (1D) at the surface are the same as horizontal component of 6D motions from the original wave field (full 3D translations and 3D rotations).
1. **Figure 8-9**: Snapshot of a full 3D wave field (left: global model; right: local site response) (live links to animations)

**Figure 8-9**: (Left) Snapshot of a wave field, with body and surface waves, resulting from a point source at 45° off the point of interest, marked with a vertical line, down-left. This is a regional scale model of a (simplified) point source (fault) with three soil layers. (Right) is a zoom in to a location of interest. Figures are linked to animation of full wave propagation.

*Figure 8-10 presents the idea of using a 3D wave field to develop a 1D wave field.*
Figure 6.10: Illustration of the idea of using a full wave field (in this case 2D) to develop a 1D seismic wavefield.

Two seismic wave fields, the original wave field and a subset 1D wave field now exist. The original wave field includes body and surface waves, and features translational and rotational motions. On the other hand, subset 1D wave field only has one component of motions, in this case an SV component (vertically polarized component of S (Secondary) body waves).

Figure 6.11 shows local free field model where both 3D (left side) and 1D (right side) motions are input. It is important to note that seismic motions are input in an exact way, using the Domain Reduction Method (Bielak et al., 2003; Yoshimura et al., 2003) (described in chapter 6.4.10) and how there are no waves leaving the model out of DRM element layer (4th layer from side and lower boundaries). It is also important to note that horizontal motions in one direction at the location of interest (in the middle of the model) are exactly the same for both 2D case and for the 1D case.

If a full NPP model is now used, and a full 3D and a 1D wave fields are input, differences in response become obvious. Figure 6.12 shows a snapshot of an animation (available through a link within a figure) of difference in response of an NPP excited with full seismic wave field (in this case 2D), and a response of the same NPP to 1D seismic wave field.
2. Figure 86.1: Left: Snapshot of a full 3D wave field, and Right: Snapshot of a reduced 1D wave field. Wave fields are at the location of interest. Motions from a large scale model were input using DRM, described in chapter 6 (6.4.10). Note body and surface waves in the full wave field (left) and just body waves (vertically propagating) on the right. Also note that horizontal component in both wave fields (left, 2D and right 1D), are exactly the same. Each figure is linked to an animation of the wave propagation.

3. Figure 86.12: Snapshot of a 3D vs 1D response of an NPP, upper left side is the response of the NPP to full 3D wave field, lower right side is a response of an NPP to 1D wave field.

In addition to observing differences in response through animation, time histories of displacements and accelerations reveal the full extent of differences.

Figures 86.13 and 86.14 show displacement and acceleration response on top of containment building for both 3D and 1D seismic wave fields.
A number of remarks can be made:

- Accelerations and displacements (motions, NPP response) of 3D and 1D cases are quite different. In some cases 1D case gives bigger influences, while in other, 3D case gives bigger influences.
- Differences are particularly obvious in vertical direction, which are much bigger in 3D case.
- Some accelerations of 3D case are larger than those of a 1D case. On the other hand, some displacements of 1D case are larger than those of a 3D case. This just happens to be the case for given source motions (a Ricker wavelet), for given geologic layering and for a given wave speed (and length). There might (will) be cases (combinations of model parameters) where 1D motions model will produce larger influences than 3D motions model, however motions will certainly again be quite different. There will also be cases where 3D motions will produce larger influences than 1D motions. These differences will have to be analysed on a case by case basis.

In conclusion, response of an NPP will be quite different when realistic 3D seismic motions are used, as opposed to a case when 1D, simplified seismic motions are used. Recent paper, by Abell et al. (2018) shows differences in dynamic behaviour of NPPs same wave fields is used in full 3D, 1D and 3×1D configurations.
4.3. **Figure 8.13**: Displacements response on top of a containment building for 3D and 1D seismic input.
5.4. **Figure 8.14**: Acceleration response on top of a containment building for 3D and 1D seismic input.
8.4 SSI models

8.4.1 Structure (Jim)
- Stick models
- Plane (2D) models
- 3D models
- Structural properties: Cracked inertia for beams, shear walls, piles

8.4.2 Foundations

Foundation modelling is separated into conventional foundation/structure systems (surface founded and shallow embedded), deep foundation (piles), and deeply embedded foundation/structure systems.

8.4.2.1 Foundation modelling for conventional foundation/structure systems

Modelling of surface foundations in a global direct SSI analysis, or in substructure analyses provided the analyses are carried out with the same software for all individual steps, does not pose any difficulty: software like PLAXIS, ABAQUS, GEFDYN, DYNAFLOW, SASSI, CLASSI, MISS3D, Real ESSI, etc... can account for foundations with any stiffness. In a conventional substructure approach, however, the usual assumption is to consider the foundation as infinitely stiff to define the foundation impedances and the foundation input motion. The question then arises of the validity of this assumption which depends on the relative stiffness of the foundation and of the underlying soil. Stiffness ratios \( SR_v \) for the vertical and rocking modes, and \( SR_h \) for the horizontal and torsional modes can be introduced to this end. These stiffness ratios depend on the foundations characteristics (axial stiffness \( E_S S_b \) in kN/ml or bending stiffness \( E_b I_b \) in kN.m\(^2\)/ml) and on the soil shear modulus \( G \) or Young’s modulus \( E \). For a circular foundation with diameter \( B \) these stiffness ratios are given by

\[
SR_v = \frac{1}{B} \left( \frac{E_b I_b B}{E} \right) \quad SR_h = \frac{1}{B} \left( \frac{E_b S_b B}{G} \right)
\] (1)

The foundation can be assumed stiff with respect to the soil when:
- \( SR_v > 1 \) for vertical and rocking modes
- \( SR_h > 5 \) for horizontal and torsion modes

Usually the condition on \( SR_v \) is always satisfied. For nuclear reactors and buildings with numerous shear walls the condition on \( SR_v \) is also satisfied considering the stiffening effect of the walls; for moment resisting frame buildings with a mat foundation, the condition on \( SR_v \) is hardly satisfied and either a complete analysis taking into account the raft flexibility shall be carried out, or the stiffness of fictitious rigid foundations around the columns shall be computed and specified at each column base, assuming that no coupling exists between the individual footings.

The impedance functions are then introduced in the structural model as springs \( K \) (real part of the impedance function) and dashpot \( C \) related to the imaginary part \( K \) of the impedance function. Alternatively, the damping ratio of each SSI mode can be computed as
The usual practice limits the damping ratio to 30%, but some standards allow for higher values, if properly justified. The main difficulty with the impedance functions is their dependence on frequency, which cannot be easily considered in time domain analyses or modal spectral analyses. Several possibilities exist to approximately take the frequency dependence into account:

- To implement an iterative process which, for each SSI mode, determines the stiffness compatible with the frequency of the corresponding undamped SSI mode; the SSI mode can be identified as the mode with the maximum strain energy stored in the spring.
- To develop a rheological model which accounts for the frequency dependence by addition of masses connected to the foundation with springs and dashpots (De Barros & Luco 1990, Wolf 1994, Saitoh 2012). The parameters of the rheological are simply determined by curve fitting of the model response to the impedance function. An example of such a model is shown in Figure 8-15 (Pecker 2006).

It should be noted that when the soil profile becomes significantly layered with sharp contrasts in rigidity between layers, the impedances functions become jagged and either of the two procedures described above may become difficult to implement; the only possibility is then to resort to frequency domain solutions.

Finally, if for surface foundations the coupling term between horizontal translation and rocking around the transverse horizontal axis may be neglected, this not true for embedded foundations; in the first instance, the impedance matrix is diagonal and springs and dashpots can be assigned independently to each degree of freedom; in the second one, the impedance matrix contains off-diagonal terms, which makes the rheological model more tricky to develop: if the software does offer the possibility of adding a full stiffness matrix to the foundation, alternatives may consist in connecting the spring at a distance \( h \) from the foundation with a rigid beam element (see for instance Koliás et al. 2012).

\[
\beta = \frac{K_i}{2K_s} = \frac{\alpha C}{2K_s}
\]

Figure 8-15: Example of a rheological model and model prediction for the horizontal mode of vibration
The substructure approach, on which the concept of foundation impedances is based, assumes linearity of the system. However, it is well recognized that this is a strong assumption, since non-linearities are present in the soil itself (section 3.2) and at the soil foundation interface (sliding, uplift, section 7.4.6). Soil non-linearities may be partly accounted for by choosing, for the calculation of the impedance matrix, reduced values of the soil properties that reflect the soil nonlinear behaviour in the free field (Section 7.4.4). This implicitly assumes that additional nonlinearities taking place at the soil foundation interface do not affect significantly the overall seismic response.

### 8.4.2.2 Deep foundations

As opposed to shallow, or slightly embedded, foundations, modelling of piles foundations is a more complex task because it usually involves a large number of piles and soft soil layers; a direct (3D) analysis becomes demanding, especially for nonlinear solutions. The substructure approach, in which the piles and the soil are represented through impedance functions, becomes more attractive provided the system can be assumed to remain linear; note however that, as opposed to shallow foundations, the impedance matrix always contains off-diagonal terms representing the coupling between the horizontal translation and rocking. An example of the substructuring approach is shown in Figure 8-10 for the case of a bridge pier (Kolias et al., 2012).

![Figure 8-10: Typical substructure approach for pile foundation](image)

Another modelling concept has been traditionally widely used for piles foundations: the so-called Winkler models, based on the concept of (linear or nonlinear) springs and dashpots to model the effect of the soil on the piles. It is illustrated in Figure 8-11 for a bridge pier (Kolias et al., 2012). The springs and dashpots, distributed along the pile shaft, represent the interaction with the soil. Although conceptually the soil reaction forces are still represented by the action of springs and dashpots, unlike for the impedance matrix approach, there is no rational or scientifically sound method for the definition of these springs and dashpots. They are usually based on standards or field experiments under static conditions. Their values, but more importantly their distribution along the pile, vary with frequency; there is no unique distribution reproducing the global foundation stiffness for all degrees of freedom. Furthermore, two additional difficulties arise for pile foundations:

- the choice of the springs and dashpots should reflect the pile group effect and,
as the seismic motion varies with depth, different input motions should be defined at all nodes shared between the piles and the soil; one should resort to a separate analysis for the determination of these input motions.

Therefore, in view of all the uncertainties underlying the choice of their parameters, global Winkler-type models, although attractive because nonlinearities between the shaft and the soil can be approximated, should not be favoured. The substructure approach, with its limitations described below, should be preferred.

Modelling of pile foundations in a substructure method raises several issues:
- Can full contact or full separation between the pile cap and the soil be assumed?
- Should the piles be considered clamped or hinged in the pile cap?

There is no definite answer to each of this question and the situation is very likely to evolve during earthquake shaking. As full consideration of this evolution is incompatible with the substructure approach which assumes linearity of the analysed system, only approximate solutions can be handled.

Contact between pile cap and soil: the contact condition may evolve during the lifetime of the structure due to settlement of the soft layers caused by consolidation of clayey strata, construction around the existing structure, … It may also evolve during the earthquake due to soil compaction. Results presented in the appendix for horizontal sway, vertical and rocking impedances show that the impact is negligible for vertical and rocking impedances and is only marginally important for the horizontal impedance. These conclusions apply to pile group with a large number of piles and may be different when few piles are considered. Given the insignificant difference between the two assumptions, it is recommended for design purpose to retain the no contact condition, which is the most likely situation in soft soils.

Fixity at the pile cap connection: during seismic loading the connection may deteriorate and evolve, due to reduction of the connection stiffness, from perfectly clamped piles to a condition where a plastic hinge forms at the connection. The results presented in the appendix compare the dynamic impedances for both extreme conditions: the clamped condition affects the horizontal stiffness but does not affect the vertical or rocking
ones. Note however that the reduction in the horizontal stiffness, almost a factor of 2, is certainly much larger than the reduction that would be obtained under the formation of a plastic hinge which, as opposed to the hinged condition, exhibits a residual moment capacity. Regarding the kinematic interaction forces, if the maximum values are not significantly affected, the distribution of the forces along the pile is totally different with maximum values occurring at deeper locations along the pile when the stiffness deteriorates. Based on the obtained results recommendation would be either to run both types of analyses (clamped and hinged connection) and to take the envelope of both conditions, or for a more conservative approach, to take the maximum forces from the hinged analysis and to extend the maximum values upwards to the pile cap connection.

8.4.2.3 Embedded foundation

An example is provided in the appendix illustrating the implementation of the substructure method for an embedded foundation (indeed similar to the implementation for a surface foundation) but which details all the intermediate steps and, more importantly, illustrates how the embedment can affect the foundation input motion due to kinematic interaction. These steps have been described in details in section 7.3.2. The structural model used for this purpose is the Small Modular Reactor model, described in section 8.4.2.4; the soil profile is identical to the one used for the 1D site response analysis with the same input motion scaled to 0.25g, for the EQL analysis to be valid. First, the site response analysis is run to determine:

- The surface motion, which serves as input motion in the following steps;
- The strain compatible soil properties (shear modulus and damping ratio) used in the SSI analyses carried out with SASSI.

Note that the site response analysis only provides the (strain compatible) shear modulus (or shear wave velocity) and not the bulk modulus (or P–wave velocity); however, both parameters are needed for a 2D or 3D SSI analysis. The reduction factor to be applied to the P–wave velocity should not be taken equal to the reduction factor calculated for the S–wave velocity. This would be equivalent to assuming that Poisson's ratio remains constant regardless of the strain amplitude; this assumption is obviously false. The parameter which is the most likely to remain constant is the soil bulk modulus $B$; in total stress analyses, like those performed with SHAKE for site response analyses, this assumption is true in saturated soils and not extremely stiff soils (like hard rock), because $B$ is practically equal to the water bulk modulus. For unsaturated soils, this assumption is only approximate but is still reasonable and probably represents the most realistic one. With this assumption, $V_P$ should be calculated as follows:

\[
B = \rho \left( V_{Pc}^2 - \frac{4}{3} V_{Sc}^2 \right)
\]  

(3)
where \( \rho \) is the soil mass density.

Calculation of the strain compatible S–wave velocity, \( V_s \), from the site response analysis:

Calculation of the strain compatible P–wave velocity according to:

\[
V_P = \sqrt{\frac{B}{\rho} + \frac{4}{3} \frac{V_s^2}{\rho}}
\]  \hspace{1cm} (4)

Note that use of eq.(4) may lead to high Poisson's ratio which may create numerical issues; in that case, it is recommended to limit the value to a maximum value compatible with the numerical software used for the SSI analysis; typically it ranges from 0.45 to 0.49. In the example provided in the annex it was limited to 0.48.

With respect to the damping ratio associated with \( V_P \) there is no better assumption than assuming the same value as for \( V_s \).

With these soil properties three embedments for the structure are analysed: surface foundation (no embedment), 14m and 36m and two different types of analyses are run: one with a massless structure and one with real structure. The former analyses provide the kinematic interaction motion (see section 7.3.2) and the latter one the global response including kinematic and inertial interaction. The most salient features of the response of each model are illustrated in Figures 8–18 to Figure 8–20.

![Graph](image)

**Figure 8-18**: 5% damped response spectra at roof elevation for 3 embedments

Figure 8-18 compares the 5% damped response spectra on top of the structure (roof elevation) for the global response; the freefield ground surface response spectrum is also shown. As expected, for
the surface structure and the shallow embedded structure, the roof spectra show a marked amplification which corresponds to the fundamental SSI frequency; as the embedment increases from 0 to 14 m, the peak is shifted towards higher frequencies because the stiffness of the foundation is increased. As opposed to the two previous cases, the deeply embedded structure does not show any marked amplification at a given frequency; furthermore, the spectrum is not very different from the surface motion. The structure motion is imposed by the soil displacements rather than the inertia of the structure. This behaviour is typical of underground structures.

Figure 8-19: Kinematic translational response spectra

Figure 8-19 and Figure 8-20 presents the foundation input motions (base of the structure) due to pure kinematic interaction (massless structure).

As pointed out in section 7.3.2, when the structure is embedded in the ground, the foundation input motion can no longer be taken equal to the freefield motion. This phenomenon is referred to as kinematic interaction; the calculated motion at the base of the structure are the foundation input motions that should be used in the substructure approach (figure 7–4). It has also been pointed out (section 7.3.2) that embedment creates a rotational component of motion at the foundation, although the incoming motion consists of plane, vertically propagating, shear waves and the soil profile is uniform in the horizontal direction.

As expected, for the case of the surface foundation, kinematic interaction is totally negligible and the rotational component of motion is nil. Be careful that only holds under the assumptions made in the
analyses: horizontally layered soil profile subjected to vertically propagating body waves. The increase in the structure embedment has two effects:

- The foundation response spectrum decreases, at all frequencies in this case, when the embedment increases;
- The rotational component of motion increases, at all frequencies in this case, when the embedment increases.

Figure 0.20: Kinematic rotational response spectra

8.4.2.4 Deeply embedded foundations (SMR)

As opposed to shallow or deep foundations, modelling and analysis of deeply embedded structures, like SMRs, are more easily achieved in a global direct time domain or frequency domain analysis. Unless the whole SSI analysis is run within a single software (like SASSI or CLASSI)\(^1\), the conventional substructure approach is not well adapted, although still theoretically possible under the assumption of linear behaviour, because of the large embedment. The embedment creates a strong kinematic interaction between the soil

\(^1\) It is recalled that SASSI and CLASSI use a substructure approach, but the same software and model are used for the analysis of the soil-foundation substructure and of the structure. Conventional substructure approaches calculate the impedance matrix, simplify it with frequency independent springs and dashpots to be connected to the structural model, which is analyzed with a different software.
and the structure which significantly alters the freefield motion and develops pressures on the lateral walls. Calculation of these two effects in a conventional substructure approach is complicated and tedious:

- Kinematic interaction motion, i.e. the effective foundation input motion, needs to be calculated from a model reflecting the embedment and variation of the freefield motion with depth; furthermore, the true effective input motion contains a rocking component which is not easily applied to the structural model;

- There is no simple means for evaluating the earth pressures on the outside walls; classical solutions, like the Mononobe and Okabe solution, are not valid for deeply embedded retaining structures that cannot develop an active pressure condition; furthermore, earth pressures and inertia force are likely to be out-of-phase, without any simple solution to easily define the phase shift between both.

Rigorous consideration of these two factors, requires a global finite element model of the embedded part of the structure, and the additional amount of effort to include the structural model is then minimal.

8.4.3 Analyses methods

8.4.3.1 Dynamic analyses

8.4.3.1.1 Substructure methods
Substructure methods are only valid provided a linear elastic behaviour of all components can be assumed. Therefore, the first task before choosing the analysis method, between a direct method and a substructure method, is to assess this importance of this aspect. However, slight nonlinearities in the soil behaviour can be accepted in the substructure approach and considered, at least in an approximate manner: as indicated in several instances in the TECDOC, reduced soil characteristics can be used in the model; these reduced characteristics represent the strain compatible properties and reflect the soil nonlinearities in the freefield. They are usually calculated from a (1D or 2D) site response analysis (section 8.3.2 and 8.3.3). It is further assumed in the substructure approach that additional nonlinearities that develop due to the interaction between the structure and the soil have a second order effect on the overall response; however, they may impact the local response, like for the soil pressures developing along a pile shaft.

The substructure approach has been described in section 7.3 and the successive steps in the approach are illustrated in the flow chart of Figure 8.21; the flowchart, with reference to the boxes numbers in brackets, is detailed below.
Two examples in the appendices illustrate some of the steps ([5], [6], [7], [10], [12], [13], [14], [15]) listed in the flowchart: one example is for an embedded structure and the second one for a piles foundation. They both refer to the conventional substructure approach in which impedances are calculated in a first step and introduced in a structural model.

One example on a deeply embedded structure is developed along the lines of the substructure approach but with all the steps of the SSI analysis run with the same software (SASSI, see note in section 8.4.2.3); therefore, some of the simplifying assumptions of the conventional substructure approach are overcome in this example.

The first step of the analysis starts with the site response analysis to calculate the strain compatible soil characteristics and ground surface response spectra. Site response analyses have been detailed in section 6.3 and 8.3. The input data for this step are:

- the geotechnical data ([1]) from which a design profile and a constitutive model are chosen for the site (section 3);
- the seismological data ([2]) from which the rock spectra ([3]) are established either from a probabilistic, or a deterministic, seismic hazard assessment (section 6.4). Time histories representing the rock motion need to be defined following one of the procedures described in section 6.5.

With these data, site response analyses provide the ground surface motion and the strain compatible soil characteristics ([6]). Usually, they are run assuming an equivalent linear constitutive model as illustrated in

**Figure B.2112: Flowchart for the implementation of the substructure approach**
the examples on embedded foundation and piles foundation in the appendix. Although nonlinear analyses are also possible, the choice of the strain compatible soil properties is less straightforward in this case and requires some amount of judgment.

The second step corresponds to the top right boxes of the flowchart: it establishes from the formwork drawings ([3]) the structural model ([8]) and the foundation model ([9]). As noted in section 8.4.2, the foundation model for the shallow embedded foundation is assumed to correspond to a stiff foundation; the one for the piles foundation of the appendix gathers the piles and the surrounding soil, modelled as continuum media.

With the foundation model and the strain compatible soil characteristics, an impedance matrix is calculated ([10]) and introduced in the structural model with frequency independent values. Section 8.4.2.1 presented two possible alternatives to define the frequency independent impedance matrix. This step produces the SSI model ([11]).

The same foundation model and the surface ground motion are used to calculate the kinematic response of the foundation ([9]); this kinematic response is composed of the foundation input motion ([13]) and of the kinematic forces developed in the foundation ([12]).

The foundation input motion serves as the input to the structural model from which the inertial components of the response are retrieved ([14]).

Finally, the results from the inertial response and from the kinematic response are combined ([15]) to yield the structural design quantities: forces, accelerations, displacements, Floor Response Spectra. If the kinematic response quantities and the inertial response quantities are obtained from time history analyses (in time or frequency domains) there is no difficulty in combining, at each time step, their contributions. The total response quantity at any time is given by:

$$R_t(t) = \pm R_I(t) \pm R_K(t)$$

However, in most cases the response quantities are not known as a function of time, and only the maximum inertial response quantities are retrieved from the SSI analyses (for instance when a modal spectral analysis is used). To combine both components, each of them should be alternatively considered as the main action and weighted with a factor 1.0, while the other one is the accompanying action and weighted with a factor \(\lambda\):

$$R_t = \pm \max (R_I) \pm \lambda \max (R_K) \quad \text{or}$$

$$R_t = \pm \lambda \max (R_I) \pm \max (R_K)$$

The coefficient \(\lambda\) depends on how close to each other are the main frequencies leading to the maximum kinematic response quantity and the main frequency leading to the maximum inertial response quantity. The first one is controlled by the SSI mode and the second one by the soil response (fundamental frequency of the soil column). If these two frequencies are well separated, let’s say by 20%, both maxima are uncorrelated in time and their maximum values can be added with the SRSS rule.

$$R_t = \sqrt{\left[\max (R_I(t))\right]^2 + \left[\max (R_K(t))\right]^2}$$

If both frequencies are within 20% of each other, it is reasonable to assume that both phenomena are correlated, and the kinematic and inertial response quantities should be added algebraically:

$$R_t = \max (R_I(t)) + \max (R_K(t))$$
\[ R_t = \max_i R_i(t) + \max_i R_k(t) \]  

8.5 Incoherent motions (Jim)
- Freefield
- SSI

8.6 Uncertainties and sensitivity studies

8.6.1 Ground motion (Jim)
- Frequency characteristics
- Time history representation of ground motion
- Spatial variation

8.6.2 Soil
It has been pointed in several instances throughout the document that great uncertainties prevail in the soil characteristics due to the difficulty to test soils, to the inherent randomness and spatial variability of soil deposits; uncertainty in soil characteristics is the second, after the ground motion, largest source of uncertainty in SSI analyses. Spatial variability is characterized by correlation distances of the order of a meter in the vertical direction and of some meters in the horizontal one; such small distances preclude a thorough characterization of the deposit. Nevertheless, when enough investigation points are available, stochastic models have been proposed to characterize the spatial variability used in seismic analyses (Popescu 1995, Popescu et al. 1995, Assimaki et al. 2003). These models remain however seldom used in practice and soil uncertainties are usually handled through sensitivity analyses.

As the constitutive model becomes more complex, the effects of these uncertainties become more and more significant. For the elastic characteristics, it has been recommended to consider at least three velocity profiles corresponding to the best estimate characteristics and to those characteristics divided or multiplied by \((1 + \text{COV})\); typically, the coefficient of variation (COV) on the elastic shear wave velocity should not be taken less than 0.25. However, the uncertainty on the elastic properties is not the single parameter that needs to be considered: large uncertainties exist in the determination of the nonlinear shear stress–shear strain curves (or equivalently \(G/G_{\text{max}}\) and damping ratio curves used to define the equivalent linear model); this uncertainty stems from the difficulty to recover undisturbed samples from the ground and to test them in the laboratory; it is therefore essential to compare any measurement to published data to assess its representativeness.

With the use of nonlinear models the number of soil parameters to define increases and therefore so does the uncertainty in the prediction of the soil response. Furthermore, there is a large variety of nonlinear models in the technical literature and none of them can be considered as the best model; the choice of the constitutive model, and the control and ability of the analyst, therefore contributes to the overall uncertainty. To cover this aspect, the use of preferably 2 nonlinear constitutive models, run by 2 different analysts has been recommended.

8.6.3 Structure (Jim)
- Stiffness of elements (cracked inertia)
- Eccentricity
8.6.4 Soil-foundation-structure models (Boris)

- Modelling of partial embedment
- Modelling of sliding, uplift, gaps (piles)
- Modelling of incompressibility
- Structure-soil-structure interaction

Developed simulations are used to illustrate differences in elastic and inelastic SSI for a Nuclear Power Plant (NPP) and deeply embedded Small Modular Reactor (SMR). Inelastic response comprises dissipation of seismic energy in soil, contacts and structure. Dissipating energy in structure can lead to material degradation and damage. It is desired to dissipate most of the energy in soil and contacts with acceptable level of deformations in structure. Proper modeling of energy dissipation (Yang et al., 2017) can be used to optimize soil structure systems for safety and economy. Presented modeling addresses differences between linear elastic and inelastic behavior of NPPs and SMRs (Sinha et al. (2017), Wang et al. (2017)).

8.6.4.1 Nuclear Power Plant Model Development and Simulation Details

The Nuclear Power Plant (NPP) modeled here is a symmetric structure with shallow foundation. Figure 1a shows a slice view of the finite element model. Solid brick elements were used to model soil and foundation. The NPP structure was modelled by elastic shell elements, while contact elements are used at the interface of foundation and soil.

Structural Model

The NPP structural model consists of auxiliary building, containment building and shallow foundation as shown in Figure 8.22b. The auxiliary building consists of 4 floors that are 0.6m thickness, ceiling floor of 1m thickness, exterior wall of 1.6m thickness and interior walls of 0.4m thickness. The exterior and interior walls are embedded down to the depth of the foundation. The containment building is a cylinder of diameter 20m and height 40m with wall thickness of 1.6m. There is a gap of 0.2m between the containment and auxiliary building. Top of the containment building is covered by semi-spherical dome of radius 20m. The foundation is square shallow footing of size 100m and thickness 3.5m. The containment building and the auxiliary building were modelled as shell elements and foundation as linear brick elements.
Soil Model

The depth of the soil modelled below the foundation is 120 m, representing also the depth of DRM layer. It is assumed that within this range the soil can behave inelastically. The soil is assumed to be a stiff saturated-clay with undrained behavior with shear velocity of 500 m/s, unit weight of 21.4 kN/m^3 and Poisson’s ratio of 0.25. To represent the travelling wave accurately for a given frequency, ten 8 node brick or three 27 node brick elements are required (Watanabe et al., 2016). Seismic waves are analyzed up to \( f_{\text{max}} = 10\) Hz. The smallest wavelength \( \lambda_{\text{min}} \) to be captured thus, can be estimated as

\[
\lambda_{\text{min}} = \frac{v}{f_{\text{max}}}
\]

where, \( v \) is the smallest shear wave velocity of interest. For \( v = 500\) m/s and \( f_{\text{max}} = 10\) Hz the minimum wavelength \( \lambda_{\text{min}} \) would be \( (500\text{m/s})(10/\text{s}) = 50\) m. Choosing ten nodes/elements per wavelength the element size would be 5m. (Jeremic’ et al. (2009), Watanabe et al. (2016)) state that even by choosing mesh size \( \Delta h = \lambda_{\text{min}}/10 \), smallest wavelength that can be captured with confidence is \( \lambda = 2\Delta h \) i.e. a frequency corresponding to 5\( f_{\text{max}} \). Based on the above analysis, soil was modeled as linear 8-node brick elements with grid spacing of \( \Delta h = 5\) m.

Dickenson (1994) proposed the following relationship between shear wave velocity \( V_s \) and undrained strength \( S_u \) for cohesive soils in San Francisco Bay Area.

\[
V_s(\text{m/s}) = 23(S_u[\text{kN/m}])^{0.475}
\]

Thus, for \( V_s = 500\) m/s, the undrained strength \( S_u \) would be 650kPa. Here, two scenarios of soil properties were considered in analysis. One linear elastic and the other as von-Mises with non-linear kinematic hardening of Armstrong – Frederick type. For \( S_u = 650\) kPa and \( E = 1.3\) Gpa, the non-linear inelastic model was calibrated for yield strength achieved at 0.01% shear strain with linear kinematic hardening rate \( h_2 = 30\) MP a and non-linear hardening rate \( c_f = 25\). The stress-strain response for the non-linear material model is shown in Figure 2a

Contact Modeling

Node-to-node penalty based soft contact (interface) element (Sinha & Jeremic’, 2017) was used to model the interaction between foundation and soil. Figure 8-23b shows the axial stiffness curve.
Contact elements were applied all around the foundation connecting to the soil as shown in Figure 8.22a in red color zone. The Coulomb friction coefficient between the soil and the foundation was chosen as $\mu = 0.25$. Viscous damping of 100Ns/m in normal and tangential damping was provided to model viscous damping arising from pumping action when contact gaping is activated.

**Seismic Motions**

3D seismic motions were developed by Rodgers (2017) using SW4 (Serpentine Wave Propagation of 4th order) (Petersson & Sjögreen, 2017) for an earthquake of magnitude $(M_w = 5.5$ modelled on a fault of dimension $5.5 \times 5.6km$ with up-dip rupture slip model. The ESSI box to capture the free-field motion was located on the foot-wall of the reverse thrust fault.

Acceleration and displacement time-series of the motion at the center of ESSI box is shown in Figure 3. The peak ground acceleration (PGA) in x and y direction is about 0.5g. Significant amount of vertical motions PGA of 0.2g can be observed. Since the fault is located at foot-wall side of reverse thrust fault, there is permanent subsidence of about 50mm in z-direction at the end of shaking event. Fourier transform and response spectrum of the motions are shown in Figure 4. The frequency range of the motion is within 20Hz. Response spectrum plot shows amplification for natural frequency greater than 2Hz.

**Domain Reduction Method**

Domain Reduction Method (DRM) Bielak et al. (2003) was used to apply 3D seismic motions generated from SW4 all around the model as shown in Figure 8.22a. DRM was described in some detail in Chapter 7.

**Staged Simulation**
The analysis was simulated with two loading stages. First stage was static self-weight, in order to initialize stress state of the soil and contact elements. In second stage, seismic motion is applied using DRM method. For each stage, equilibrium was achieved using full Newton-Raphson method with a small tolerance of $1 \times 10^{-4}$ N on second-norm of unbalanced force. For dynamic analysis, Newmark integration method with numerical damping $\gamma = 0.7$ was used. Rayleigh damping of 2% in structure and 30% in soil was applied. The time step considered here was 0.02 seconds with simulation running in total for 40 seconds. Figure 8-24 and Figure 8-25 show acceleration and displacement results at the soil surface for the free field analysis.

Figure 8-24: Acceleration and displacement time series
The analysis was run in parallel using the **MS ESSI (Real ESSI) Simulator** (Jeremic et al. (2018)) on eight CPUs. The model consisted of about 300k degrees of freedoms (dofs). Four scenarios (a) elastic no contact (b) elastic with contact (c) elastic-plastic no contact and (d) elastic-plastic with contact were performed. In this particular case, elastic means elastic without contact and inelastic means elastic-plastic with contact.

**Simulation Results**

 Locations selected for showing results are shown in Figure 5. Since the containment building is more flexible than auxiliary building, location (D) in Figure 8-26 located on the top of the containment building is the point of interest. Three locations (A), (B) and (C) located on foundation level are also selected to study the slip at interface during shaking.

Due to location of the site the tectonic foot wall, the whole structure along with surrounding soil moves down by about 50mm in elastic (tectonic movement) and 100mm in inelastic case (tectonic in addition to inelastic settlement). Overall, if self-weight stage is also included, the soil settles by 150mm in elastic and 350mm in inelastic case as shown in Figure 8-26b.

Figure 8-276 shows the acceleration and its Fourier amplitude for the location (D) for both elastic and inelastic cases. It is important to observe that the elastic-plastic behavior reduces high frequency excitations in the structure.

Figure 8-282 shows differences in acceleration response for elastic and inelastic/nonlinear analysis, along the depth of soil and along the height of the NPP. Significant reduction of accelerations is observed.
Figure 8.265: Locations selected to study non-linear effects and plot of total displacement at center of model Elastic (elastic with contact) and Inelastic (Elastic-Plastic with contact).
Figure 8-276. Elastic and Inelastic response on top of the containment building

Figure 8-282. Difference in seismic motions with depth for elastic (left) and inelastic (right) models.
Model described below illustrates inelastic modeling of a deeply embedded small modular reactor (SMR) (Wang et al. (2017)).

Deeply embedded structures do have a number of particular features that need to be carefully addressed:

- **Ground Motion**: Deeply embedded structures are exposed to both body waves (P, SV and SH) as well as surface waves (Rayleigh, Love, etc.). For surface structures, it is common to use historical earthquake records and 1D seismic wave propagation models. Vertical ground motion are usually neglected. However, Oprsul and Fah (2007) have emphasized the necessity to use 3D ground motion by showing the big difference between 1D and 3D computation result. Deconvolution is used to develop both horizontal and vertical motions (Elgamal et al., 2008). It is important to note, however, that vertical motions at the surface are mostly a result of surface wave, rather than a vertically propagating compressional, primary (P) wave. In addition, surface waves do disappear with depth so deeply embedded structures will be exposed to different seismic motions at different depth and at the surface.

- **Seismic Input into SSI Model**: Input of seismic motions into the finite element models for sil structure systems can also play an important role in modeling uncertainty.

- **Inelastic/Nonlinear ESSI Effects**: Inelastic behavior of material can significantly influence response of soil structure systems. There are three sources of nonlinearity in an ESSI system: (a) Inelastic (elastic-plastic) behavior of soil, (b) inelastic (elastic-plastic) behavior of the contact zone, and (c) inelastic (elastic-damage-plastic) behavior of the structure. Early works found that structural response can be quite different when elastoplasticity of surrounding soil is considered (Bielak (1978), Iguchi and Luco (1981)). In addition to that Jeremic et al. (2004) reported that ESSI behavior can have either beneficial and detrimental effects on structural behavior. The nonlinear contact (interface) was analyzed by Hu and Pu (2004) and it was shown that its accurate modeling is a key part to realistic modeling of ESSI systems.

Model presented here illustrates an approach to modeling deeply embedded structures using both linear elastic modeling (using SASSI and Real ESSI Simulator) as well as a using inelastic modeling (using Real ESSI Simulator).

For the inelastic approach, in this example, energy propagation through the model is also modeled (Yang, et al., 2017, and Yang et al. 2018). Energy dissipation is a widely used indicator of material damage in elastic plastic materials.

**Seismic Input**

Input of seismic motions into finite element model is done using the Domain Reduction Method (Bielak et al., 2003), (DRM), as described in Chapter 7.
Free Field Motions

Development of free field motions was done using a fourth order finite difference program SW4 (Petersson and Sjogreen 2017) developed at LLNL. Modeled was propagation of fault rupture in a model with dimensions 9km $\times$ 6km $\times$ 20km. The magnitude of simulated earthquake is 5.5. The shear wave velocity of soils in surface layer (500 meters thick) is $V_s = 500$m/s. Motions were recorded in a box with dimensions 300m $\times$ 300m $\times$ 200m.

The characteristic ground motions recorded by ESSI nodes are plotted in Figure 8-29. The peak ground acceleration (PGA) in x and y direction is about 1g. Apart from that, significant amount of vertical motions with PGA $0.5g$ is also observed. The peak ground displacement (PGD) is about 0.1m in horizontal direction. Since ESSI box is located in the foot wall of the reverse fault, the permanent ground subsidence of about 6cm is recorded. Fourier transformation and response spectrum of the motions are shown in figure 8-30. The frequency range of the motion is within 15Hz. The dominant frequency of the motion is around 5 Hz. In response spectrum, a significant resonance effect is observed for structure with fundamental frequency of 5 Hz.

Figure 8-29: Acceleration and Displacement Time Series of Motion (X, Y and Z, vertical directions)
Figure 8.302: Strong Motion Fourier Transform and Response Spectrum

Development of free field motions is described in some detail in Chapter 7.
Model Description

Figure 8.31: FEM model of SMR

Model, shown in Figure 8.31, is described in some detail by Wang et al. (2017).

The size of FEM model is 72m×72m×56m. There are 177,806 nodes, 20172 27-node brick elements, 3,177 contact elements, modeling the interface between soil and embedded structure, with a total of over 533 thousand degrees of freedom (DoFs). The average mesh size is 3 meters. Newmark time integration method is used in this study with parameters γ = 0.7 and β=0.36. In order to capture the wave propagation in FEM model, mesh size is controlled so that there is no artificial filtering to motions above certain frequency (Watanabe et al., 2016). Ten linear interpolation finite elements and three quadratic interpolation elements are needed per wavelength. Since second order 27 node brick elements (quadratic displacement interpolation) are used here, the minimum wave length captured is 6 meters (f_{max} = v_s/\lambda_{min}). Considering shear wave velocity v_s = 500m/s, the maximum modeled frequency is 83 Hz. Even when material plastifies (becomes softer), model is still propagating high frequencies of up to required f_{max} ≤ 15Hz.

Embedded Nuclear Installation

Small Modular Reactor (SMR) analyzed here is a 4 story high (deep) reinforced concrete structure with total height 50 meters with 36 meters embedded in the ground. The length and width of the structure is 30 meters. The whole structure is modeled using 27-node solid brick element with linear elastic material. The Young’s
modulus is selected as \( E = 30 \text{GPa} \) and Poisson’s ratio \( v = 0.2 \). Single layer of 27 node bricks can accurately model (beam and plate) bending, and is hence chosen for structural model.

**Soil Model**

The depth of the model surrounding the structure is 45m. It is assumed that the soil is elastic-plastic, fully saturated, with undrained behavior during the earthquake. Elastic plastic von Mises material model with linear kinematic hardening rule is used here. The material parameters are presented in table 1.

<table>
<thead>
<tr>
<th>Table 1: Modeling parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear wave velocity [m/s]</td>
</tr>
<tr>
<td>Young’s modulus [GPa]</td>
</tr>
<tr>
<td>Poisson ratio</td>
</tr>
<tr>
<td>von Mises radius [kPa]</td>
</tr>
<tr>
<td>kinematic hardening rate [MPa]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Contact parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial normal stiffness [N/m]</td>
</tr>
<tr>
<td>hardening rate [m]</td>
</tr>
<tr>
<td>maximum normal stiffness [N/m]</td>
</tr>
<tr>
<td>tangential stiffness [N/m]</td>
</tr>
<tr>
<td>normal damping [N/(m/s)]</td>
</tr>
<tr>
<td>tangential damping [N/(m/s)]</td>
</tr>
<tr>
<td>friction ratio</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Damping parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>structure</td>
</tr>
<tr>
<td>surrounding soil</td>
</tr>
<tr>
<td>DRM layer</td>
</tr>
<tr>
<td>outside layer 1</td>
</tr>
<tr>
<td>outside layer 2</td>
</tr>
<tr>
<td>outside layer 3</td>
</tr>
</tbody>
</table>
Soft Contact Element

Model for contact (axial contact, gap opening and closing and slip behavior) of the interface between structure and surrounding soil, relies on a node-to-node soft contact (interface) element (Sinha and Jeremic (2017)). In soft contact, the normal stiffness exponentially grows as the relative displacement between two contact nodes decreases. Contact parameters are shown in Table 1.

Simulation System and Approach

The nonlinear ESSI analysis was conducted using MS ESSI (Real ESSI) Simulator (Jeremic et al., 2018) (http://ms-real-essi.info/). Two SMR simulation models were simulated. First model uses linear elastic soil without contact element and second model uses inelastic/nonlinear soil with inelastic/nonlinear contact. In both cases, two loading stages were modeled: First loading stage is a selfweight, developed by adding a uniform gravity field. This is a necessary stage for inelastic analysis in order to develop initial stress state of structure and surrounding soil before earthquake loading. Selfweight was also applied to the elastic model, just so that we have comparable displacement results. Second loading stage is an earthquake load. Simulations were performed on a local parallel computer with parallel version of the Real ESSI Simulator using 10 CPUs. It is noted that Real ESSI Simulator is also available on Amazon Web Services cloud parallel computers as well as on large national parallel computers at the LBNL (EDISON and CORI).

In addition to the use of the Real ESSI Simulator, SASSI program (Lymser, 1988, Lysmer et al., 1988, Ostadan, 2007a,b) was also used. Linear elastic analysis results using SASSI were then compared with linear elastic results using Real ESSI.

Simulation Results

Result of Elastic and Inelastic Analysis using MS ESSI (Real ESSI)

Figure 8-32 shows time series acceleration response, while Figure 8-33 shows response in frequency domain, at the top center of SMR.
It is noticeable that the inelastic response significantly decreases accelerations with structure. Figures 8.324 and 8.335 show such decrease.

Figure 8.335: Acceleration Response in frequency domain

Figure 8.346 shows difference in elastic and inelastic response, with noticeable reduction in seismic motions (accelerations) for inelastic response.
Energy Dissipation

According to the thermodynamics framework presented by Yang, Sinha, Feng, McCallen and Jeremić (2017), the energy energy balance of a SSI system is given by:

\[ W_{\text{Input}} = E_{\text{Stored}} + E_{\text{Dissipated}} = KE + SE + P_F + P_D \]  

(2)

where \( W_{\text{Input}} \) is the input work due to external loading, \( KE \) is the kinetic energy, \( SE \) is the elastic strain energy, \( P_F \) is the plastic free energy, and \( P_D \) is the energy dissipation due to material plasticity. Note that in Equation 2, it is assumed that no other forms of energy dissipation exists in the system.

Figure 8-352 shows a snapshot of energy dissipation. Note that close to surface, contact zone dissipates significant energy, while at depth, soil volume takes over and dissipates more seismic energy.
Result of Elastic Analysis using SASSI and MS ESSI (Real ESSI)

1D free field motion is used for this comparison. The response of five points at different elevations ($z = 14.0m, 7m, -8.5m, -22m and -33m$) is shown in Figure 8-36.

Figure 8.36: Configuration of recording points
For illustration purposes, comparison is done for the top (z = 14m) and bottom (z = −33) of the structure, as shown in Figures 8.37, 8.38, 8.39 and 8.40, 8.10, 8.11.

Figure 8.37: Acceleration response comparison of location Z=14m

Figure 8.38: Displacement response comparison of location Z=14m

Figure 8.39: Acceleration response comparison of location Z=−33m
References


Peterson, N. A. and Sjogreen, B. (2017). 'High order accurate finite difference modeling of seismic...


Inelastic analysis of earthquake soil structure interaction (ESSI) behavior requires expertise in a number of technical areas. For example, knowledge of soil and rock mechanics is necessary for proper modeling of dry, partially or fully saturated soil and rock domain under a structure. Modeling interface between structural foundations and the soil/rock beneath requires knowledge of soft contact elements, that can also be dry or saturated. Modeling of structure, made of concrete and/or steel, requires knowledge of inelastic structural mechanics. Modeling of systems and components (SCs) within structure can also be done within the same large scale model, although such SCs can also be modeled separately, de-coupled as they might not contribute in any significant way to ESSI of the system. Earthquake motion development for ESSI analysis, requires knowledge seismology, geophysics and soil and rock mechanics.

Equally important is the numerical simulation approach used to develop numerical results using above developed models. Different finite elements with a variety of mass, damping and stiffness matrices, different numerical algorithms for constitutive and global solution advancement, and different finite element meshes will influence results.

Presented here briefly is a step by step, hierarchical approach to modeling and simulation of earthquake soil structure interaction (ESSI) for a generic concrete building. Approach is based on reliance on numerical modeling and simulation expertise as well as on sound engineering judgment.

Input files for all the examples from this section are available online at this LINK. All the examples can run directly at the Amazon Web Services (AWS), using MS ESSI Real (Real ESSI) Simulator, through MS Real ESSI image that is available on AWS.
The Model

Chosen for this exercise is a simple concrete box structure with 3 levels and 2 bays. Structure is rather tall, (3 × 20m, including roof level) with two bays (2 × 15m) and is made of walls/plates that are 2m thick. Structure is founded on slab foundation that is 5m thick. Concrete foundation is embedded 5m so that top level of the foundation is flush with soil surface.

Beneath the structure is a uniform layer of soil. Interface of concrete foundation and the soil beneath can slip in friction. Axial, normal contact between concrete and the soil beneath can open a gap.

Figure 8.41 shows dimensions of the model. This figure also shows extent of the soil model,
Figure 8.41: Soil structure model to be analyzed. Full 3D motions, inelastic soil, inelastic contact, inelastic structure.

Together with a layer of finite elements that is used to input earthquake motions into the finite element model using Domain Reduction Method (DRM) (Bielak et al., 2003, Yoshimura et al., 2003) that is described in Chapter 7 in some more detail.

Figure 8.42 shows a full finite element mesh of the complete systems. It is often tempting to start using this complete model at once, with all the inelastic/nonlinear components and with full 3D seismic motions. This temptation should be resisted and countered by a good engineering approach where the system is broken into components. Components are properly modeled and tested against known solutions (verification) and comparison with programs that are verified, that can model those components of this sophisticated model. Modeling results thus produced should be evaluated using sound engineering judgement.

Free Field 1D

Analysis begins with a 1D wave propagation analysis. Figure 8.43 shows a 1D wave propagation model. The model is made up with 3D brick finite elements, that are constrained
with boundary conditions so that only 1D shear waves, polarized in vertical plane, hence SV waves, can propagate. A presence of the DRM layer in the model shown in Figure 48.43.

as well as two layers of finite elements outside of the DRM layer, that are used to support the model.

Seismic motions used for this 1D analysis are obtained from one component of a full 3D seismic motions at the surface. Surface motions are then deconvoluted to the DRM layer, and are used to develop DRM forces.

Elastic Material. Analysis of 1D wave propagation using elastic material is first performed. Results obtained with linear elastic material using Real ESI Simulator can be compared using analytic 1D wave propagation solution for elastic material. Such analytic solutions are available in books (Kramer, 1996, Semblat & Pecker, 2009, Kausel, 2006, 2017), and are also implemented in a number of available programs, such as SHAKE (Idriss & Sun, 1992).

Elastoplastic Material. Elastic plastic analysis of 1D wave propagation can be accomplished on the very same 1D model, as described in a section above.
Deformed shape of a 1D wave propagating through the model, at particular time instance, is shown in Figure 8-447. It is noted that a DRM layer, single finite element in this case, is significantly deformed, as that layer is used to input seismic motions into the model.

Figure 8-447: 1D wave propagation model, deformation shape at particular time instance.

Free Field 3D

The very same seismic wave field that is used the previous 1D example, is used for input in the full 3D finite element model, shown in Figure 8-458.

Figure 8-458: 3D simulation model for free field wave propagation modeling.
Although finite element model is full 3D, since the seismic wave field input is 1D, only 1D wave is expected to propagate, and results should be very similar, in fact the same, as for the 1D model results.

It is important to emphasize again that material models for 1D and 3D cases are exactly the same. The difference between 1D and 3D models is in model geometry and boundary conditions.
Deformed shape at a particular time step for a 3D model subjected to 1D motions is shown Fig. 8.461.

Soil-Foundation Interaction 3D

After 1D and 3D free field motions are developed, using first linear elastic and then inelastic material models, next step is to add a foundation. The idea is that by adding just a foundation and not a complete foundation-structure system, model is not significantly changed from previous free field model. Hence, response of the soil foundation system should be similar to the free field response.

Elastic Material. Initial analysis is using linear elastic model, for soil, for foundation, for the contact zone, as well as for the structure. Analysis begins with very small motions, in 1D and then extends to 3D motions, both of which are described later.

Inelastic Material Model, von-Mises Armstrong-Frederick or von-Mises G/Gmax Material or Drucker-Prager Armstrong-Frederick or Drucker-Prager G/Gmax Material. Analysis using in-elastic, nonlinear soil material begins with very small motions. It is expected that such very small motions, will produce essentially linear elastic response that should be comparable and very close to linear elastic response.

It should be noted that depending on complexity of soil behavior and on available test data, different material models can be used for modeling soil, as discussed in Chapter 3. For pressure insensitive material behavior (total stress analysis for example, von Mises based material models are used. When mean stress (mean confining pressure/stress) is important, pressure sensitive models need to be used, that are based on versions of Drucker Prager yield surface.

Contact Elements. In addition to inelastic behavior of soil, modeled using elastic-plastic material models for soil solids, contact zone, between foundations and adjacent soil/rock significantly contributes to the inelastic/nonlinear behavior of the ESSI system.

Addition of contact elements to the ESSI model requires further model verification, as was done for addition of inelastic/elastic-plastic models. It is recommended that contact elements be initially added to a model where all other components are linear elastic. ESSI model with contact elements is initially
tested with using very small motions so that contact is not expected to open a gap or slip. Response with no slip can achieved by prescribing large friction angle that is used only to prevent slop. No gap condition cannot be insured, even if the first loading stage if selfweight, since some gaps might open during selfweight application, however large or small friction angle is used. Another option for initial testing is to apply sticky condition to contact elements, where plastic slip or gap opening is prevented by contact model implementation.

Once contact elements are verified for very small forces and motions, more realistic material parameters should be used. For frictional behavior, elastic – perfectly plastic, or elastic – hardening plastic or elastic – hardening-softening contact constitutive law should be chosen based on contact behavior test data. Axial contact behavior for concrete and soil/rock is best modeled using soft contact, where axial stress-strain response is a nonlinear function, as described in chapter 7.
Inelastic soil material model and inelastic contact are used next. Once both inelastic, elastic-plastic modeling of soil (solids) and contact is verified separately, inelastic models for soil and contact can be introduced into the finite element model. Again, very small motions or forces should be used initially, with expectation of an elastic. Increase in demand (forces or motions) should result in yielding of elastic-plastic soil and contacts. Inelastic response results in reduction of frequencies and dynamic motions, with a potential increase in permanent deformation after shaking.
Results of the simulation with inelastic soil, inelastic contact and the foundation are shown in Fig. 8.4713.

![Soil foundation interaction results](image)

Figure 8.4713: Soil foundation interaction results.

Analysis of a Base Supported Structure Alone

In order to verify structural model, it is useful to separate structural model from the soil and analyze fixed based model, as shown in Figure 8.48.

Eigen Analysis

Eigen analysis of a fixed base structural model should provide a good check of the structural model, natural (eigen) frequencies, and natural (eigen) modes.

![Structure on a fixed based simulation model](image)

Figure 8.4814: Structure on a fixed based simulation model.

For this particular example, eigen modes and frequencies are given in Figures 8.4915 and 8.5016.
Figure 8.49.5: Eigen frequencies: \( f_1 = 3.47 \text{Hz} f_2 = 3.47 \text{Hz} f_3 = 6.88 \text{Hz} \) (eigen mode 1 to 3 from left to right).
Figure 8.5016: Eigen frequencies: \( f_4 = 11.50 \text{Hz} \), \( f_5 = 11.50 \text{Hz} \), \( f_6 = 12.13 \text{Hz} \) (eigen modes 4 to 6 from left to right).

Imposed Motion

In addition to eigen analysis, fixed base structural model is used to test response of a fixed base structure. This is important as it provides an opportunity to compare results between different finite element programs, some of which can only model dynamics of fixed base structures.

Fixed base structural model deformation during imposed motions at, at one particular time instance, is shown in Figure 8.5148.

Figure 8.5148: Fixed base structural model, deformation shape during imposed motions at the base.
Full 3D ESSI System

Much like in the case of soil and the foundation alone, full earthquake soil structure interaction (ESSI) system, with complete model for soil, contacts, foundation and the structure, analysis begins, with very small forcing and small motions, expecting essentially linear elastic response.

Elastic Material. Elastic material applies to all components of the system including the soil, contacts (no slip or gapping), foundation and the structure.

Inelastic Soil and Inelastic Contact Model. Inelastic soil modeling uses any of the models that are appropriate, as discussed in section on soil modeling in chapters 4 and 7 as well as in this chapter.

It is important to note that initially, due to small shakes and/or forcing, elastic response is expected. With the increase in amplitude of forcing and/or shaking, development of nonlinear response is expected, in both soil and the contact zone.

Figure 8-5219 for example shows a deformed pattern of full 3D ESSI model at a particular time instance. Each component of this model was verified and prediction results using such verified models should provide analyst with confidence to use those results in decision making for design and assessment.

Figure 8-5219: Displacement at one time instance for a complete 3D ESSI model.

Summary:
Step by step, hierarchical approach is advocated. Expertise in a number of fields is necessary, including soil mechanics, structural mechanics, seismology and earthquake motions as well as numerical analysis and the finite element method. In addition, sound engineering judgement is needed for proper model development.
References


8.7 Structural response quantities

8.7.1 Deterministic analyses

8.7.1.1 Design forces, displacements and stresses

The structural response quantities (displacements, stresses, strains, bending moments, shear forces...) should be defined in accordance with the type of analysis used to compute them. Aside from the uncertainties in the soil and structural input data, which may be accounted for by sensitivity analyses, it must be realized that direct step by step analyses, as opposed to modal spectral analyses, introduce another cause of uncertainty in the response. This is due to the variability of the acceleration time histories derived from response spectra (see section 6.5). This variability is further enhanced when nonlinear step by step analyses are implemented. Some regulations (e.g. ASN Guide 2/01) recognize these possible sources of variability by specifying design quantities related to the type of analyses.

If \( R_{i,j} \) represents the maximum value of any response quantity for a given input motion, \( i \) (response spectrum or time history), and for one model \( k \) amongst the \( N \) models used for the sensitivity analyses, \( R_{w,i} \) is defined as a weighted average of these \( R_{i,j} \):

\[
R_{w,i} = \sum_{k=1}^{N} w_k R_{i,j} \quad \text{with} \quad \sum_{k=1}^{N} w_k = 1
\]  

(92)

Note that the previous equation allows the designer to introduce any degree of conservatism in his design. The maximum value will be obtained, if model \( q \) gives the maximum response quantity, by setting \( w_q = 1 \) and \( w_p = 0 \) for all \( p \neq q \). An average value, over all sensitivity analyses, will be obtained by setting \( w_k = 1/N \) for all \( k \).

With this definition, it is suggested that the design structural response quantity, \( R_0 \), be taken equal to:

- For modal spectral analyses to

\[
R_0 = R_{w,1}
\]  

(108)

- For step by step linear time history analyses (with \( i=1, P \) time histories) to the mean value \( \bar{m} \) of the \( R_{w,i} \), provided \( P \geq 3 \);

\[
R_0 = \bar{m}(R_{w,i}) = \frac{1}{P} \sum_{i=1}^{P} R_{w,i} = \frac{1}{P} \sum_{i=1}^{P} \sum_{k=1}^{N} w_i R_{i,j}
\]  

(119)

- For step by step nonlinear linear time history analyses (\( i=1, K \)) to the mean value \( \bar{m}(R_{w,i}) \) plus some fraction \( \lambda \) of the standard deviation \( s(R_{w,i}) \), provided \( K > 5 \); the fraction \( \lambda(K) \) depends on the number of simulations (time histories used for the analyses) and is based on the Student–Fisher test for a confidence interval of 95%. These values are provided in the following table.
In summary, any design quantity is given by

$$ R_{ij} = \frac{1}{K} \sum_{i=1}^{K} R_{ij} + \lambda(K) \sigma(R_{ij}) $$

For modal spectral analyses, $i=1$, and for linear step by step analyses with $i \geq 3$, $\lambda(K)=0$; for nonlinear step by step analyses with $i \geq 5$, $\lambda(K)$ is given in the previous table.

The rules detailed above are valid for design of new structures in deterministic analyses. For assessment of existing structures, $R_{iw}$ represents the value calculated for the best estimate properties ($k=1$, no sensitivity analyses) and the same rules apply.

### 8.7.1.2 Seismic input to sub-systems

The seismic input to sub–systems is represented by the Floor Response Spectra (FRS). FRS are preferably calculated from the time histories of the response at the required location; however, methods used for direct generation of in-structure response spectra are acceptable when the system remains linear. They shall be computed in accordance with USNRC Regulatory Guide 1.122 and USNRC NUREG-800, 3.7.2, Rev. 3. Consideration should be given in the analysis to the effects on floor response spectra (e.g., peak, width) of expected variations of structural properties, damping values, soil properties, and SSI. In addition, for concrete structures, the effect of potential concrete cracking on the structural stiffness should be specifically addressed. To account for these uncertainties in the structural frequencies the computed floor response spectra from the floor time-history motions should be smoothed, and peaks associated with each of the structural frequencies should be broadened.

Amongst these parameters, the influence of the soil characteristics and time histories of the design earthquake are the most important. When multiple sets of time histories, derived from actual earthquake records, are used as the input motion to the supporting structure, the multiple sets of in-structure response spectra already account for some of the uncertainty (NUREG-800, 3.7.2) and there is no need to further broaden the peaks of the calculated FRS.

To account for the variability of the soil characteristics, at least three sets of velocity profiles should be used (section 8.6.2) for the analyses. In addition, the ASN2/01 guide recommends broadening by at least 15%, on either side, the peaks of the FRS associated with the best estimate soil properties.
8.7.2 Probabilistic analyses (Jim)

- Internal forces, moments, displacement, stresses
- Seismic input to sub-systems

Appendices: Examples

- Site response analyses:
  - Develop approaches 1, 2, 3
- SSI analyses (coherence)

REFERENCES


Petersson, N. A. and Sjogreen, B. (2017), ‘High order accurate finite difference modeling of seismo-
acoustic wave propagation in a moving atmosphere and a heterogeneous earth model coupled across a realistic topography’, *Journal of Scientific Computing* pp. 1–34,