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Nonlinear and Equivalent Linear Site response analysis for the Groningen area

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De ondiepe bodemopbouw in het Groningen gebied, gekenmerkt door relatief slappe grondlagen als klei en veen en stijvere zandlagen, is dusdanig dat er een significante invloed van de ondiepe ondergrond op de grondbeweging tijdens aardbevingen verwacht wordt. Deze invloed van de ondiepe ondergrond op de grondbeweging wordt de 'site response' genoemd. Voor het bepalen van de ontwerp-aardbevingsbelasting en de evaluatie van de seismische hazard (seismische dreiging) in het Groningen gebied is het belangrijk om rekening te houden met het grondgedrag bij dynamische belastingen door aardbevingen.

Het gedrag van de grond tijdens een dynamische aardbevingsbelasting kan niet-lineair zijn: Zowel de (schuif)stijfheid van de grond als de demping zijn afhankelijk van de hoeveelheid (schuif)deformatie. Bij het berekenen van de site response is het van belang om dit niet-lineaire grondgedrag in rekening te brengen. Het effect van het niet-lineaire gedrag van de grond op de site response en de resulterende grondbeweging aan het oppervlak kan op verschillende manieren bepaald worden. In dit rapport worden twee methoden vergeleken om het niet-lineaire gedrag van de grond en de site response tijdens een aardbeving te bepalen, nl. :

1) De equivalent lineaire methode.
2) De niet-lineaire methode.

De eerste, equivalent lineaire methode is gebruikt bij de bepaling van de '*Ground Motion Prediction Equations*' (GMPE, versie 2 [Bommer et al 2015]) voor Groningen, als onderdeel van de seismische hazard en risico-analyse voor het Groningen gasveld. De tweede, niet-lineaire, methode is gebruikt bij de totstandkoming van de Nationale Praktijkrichtlijn voor aardbevingsbestendig bouwen ([NPR 9998)] [Arup 2015]. In beide gevallen worden de methoden gebruikt om het effect van het (niet-lineaire) gedrag van de ondiepe ondergrond op de site response in het Groningen-gebied te verdisconteren; het is dus van belang om inzicht te krijgen in de verschillen tussen de twee rekenmethoden.

In deze studie zijn met beide methoden simulaties uitgevoerd en zijn de resultaten, in termen van piek grondversnelling aan het maaiveld (PGA) en spectrale versnellingen[1] met elkaar vergeleken. De site-response berekeningen zijn uitgevoerd voor een representatief grondprofiel in het Loppersum gebied, gekenmerkt door een relatief slappe bodem en voor een stijver, zandig grondprofiel in het Slochteren gebied. Het Loppersum profiel heeft een gemiddelde schuifgolfsnelheid (vs30) van 156 m/s en wordt gekenmerkt door het voorkomen

[1] Spectrale versnellingen als functie van de trillingsperiode worden gebruikt om de ontwerp-aardbevingsbelasting voor constructies te bepalen

van een 8 m dikke slappe kleilaag aan de top van het profiel (met schuifgolfsnelheden tussen 100-112 m/s). Het Slochteren profiel heeft een vs30 van 245 m/s. Schuifgolfsnelheden in de bovenste 5 m van dit zandige profiel lopen op van 143 tot 200 m/s, terwijl de schuifgolfsnelheden op grotere diepte boven de 200 m/s liggen. In beide gevallen is de site response in de bovenste 30 meter van de ondergrond berekend. De bodemparameters en materiaalmodellen voor de grondlagen in beide berekeningen zijn identiek, alleen de oplosmethode voor het bepalen van de site response in de niet-lineaire en equivalent lineaire methode verschilt.

Uit vergelijking van de resultaten van beide rekenmethoden kan het volgende geconcludeerd worden:

- Voor de kortere spectrale perioden zijn de verschillen tussen beide methoden significant, in het bijzonder voor de spectrale periode van 0.01 s. De spectrale versnelling bij deze periode kan beschouwd worden als de piek grondversnelling (PGA). De niet-lineaire methode geeft lagere resultaten voor de PGA dan de equivalent lineaire methode. De verschillen tussen beide methoden nemen toe met een toename van de input-versnelling, die op 30m diepte aan de basis van het ondiepe grondprofiel als (*outcrop motion*) wordt opgelegd. Voor het Loppersum profiel zijn verschillen klein voor input-versnellingen lager dan 0.05 g. Voor het meer zandige Slochteren profiel zijn de verschillen klein tot ongeveer 0.1 g input versnelling. Voor toenemende input-versnellingen worden met de niet-lineaire methode voor het Loppersum en Slochteren profiel tot een factor 2 lagere PGA-waarden berekend.

- Voor de spectrale periode van 1 seconde zijn de verschillen tussen beide methoden voor het relatief slappe Loppersum profiel klein voor input versnellingen lager dan 0.2 g. Voor het zandige Slochteren profiel zijn voor de spectrale periode van 1 s verschillen tussen beide methoden klein voor input versnellingen lager dan 0.5 g. Voor hogere input versnellingen ligt de equivalent lineaire respons boven de niet-lineaire respons.

- Voor spectrale perioden van 2 seconden en langere perioden zijn de verschillen tussen de equivalent lineaire methode en niet-lineaire methode klein voor input versnellingen tot 1 g.

De equivalent lineaire methode is een benadering van de volledig niet-lineaire site response berekeningen. De equivalent lineaire methode geeft voor de bestudeerde profielen hogere spectrale versnellingen dan de niet-lineaire methode en is daarmee conservatief. Er zijn uitzonderingen op dit algemene beeld, nl. voor spectrale periodes tussen ongeveer 0.05 s en 0.08 s, waar de niet-lineaire respons soms iets boven de equivalent lineaire respons ligt.

Naast de equivalent lineaire en niet-lineaire berekeningen zijn additionele berekeningen met zuiver lineaire eigenschappen uitgevoerd, waarbij is uitgegaan van een constante stijfheid van de grond, onafhankelijk van de schuifdeformatie. De gebruikte stijfheid is de stijfheid bij kleine rek. Uit vergelijking met de eerdere berekeningen blijkt dat het niet-lineaire grondgedrag voor de PGA al een rol speelt vanaf 0.02 g voor het slappere profiel en vanaf 0.05 g voor het stijvere profiel. Vanaf deze versnellingen liggen de PGA-waarden uit de lineaire berekeningen boven de niet-lineaire resultaten. De niet-lineaire berekeningen laten bij hogere input versnellingen een verzwakking van de PGA zien in vergelijking met de lineaire analyses door de toename van de demping ten gevolge van het niet-lineaire hysteresegedrag van de grond. Voor huidige geregistreerde aardbevingen zijn deze niveaus op sommige meetlocaties al bereikt en het is aan te bevelen om (toekomstige) seismische registraties uit boorgaten te gebruiken voor validatie van de lineaire en niet-lineaire rekenmodellen.

Summary

For the shallow soil conditions in the Groningen area, characterised by relatively soft clays and peats and stiffer sand layers, a significant influence of the shallow soils on the seismic ground motion is expected, the so-called site response. For determination of design earthquake loads and evaluation of the seismic hazard in the Groningen area these site response effects have to be taken into account.

During seismic loading, nonlinear dynamic soil behaviour can occur, i.e. both soil shear stiffness and damping characteristics depend on the amount of shear deformation. This nonlinear soil behaviour has to be incorporated in site response analysis. In this study, two different methods for incorporating the nonlinear soil effects into site response are compared:

1) The equivalent linear method.
2) The nonlinear method.

The equivalent linear method is used in the development of the v2 '*Ground Motion Prediction Equations*' (GMPE) as part of the seismic hazard and risk analysis for the Groningen gas field [Bommer et al 2015]. The nonlinear method is used in analyses carried out for the Groningen area in the NPR 9998, the Dutch Annex for the Eurocode 8 for seismic design of buildings [Arup 2015]. As in both cases the methods are used to incorporate the effects of nonlinear soil response on the ground motions in the Groningen area, it is important to gain insight into the differences between the methods.

In the present study, simulations with the equivalent linear and nonlinear method were performed on two typical soil profiles in the Groningen area. Results of the two methods are compared in terms of peak ground accelerations (PGA) and spectral accelerations[2]. The first soil profile studied represents soil conditions in the Loppersum area. The soil profile consists of relatively soft soil, characterized by an average shear wave velocity in the upper 30 m (vs30) of 156 m/s. The top layer consists of an 8 m thick soft clay, with shear wave velocities increasing from 100 to about 112 m/s. The second profile represents soil conditions in the Slochteren area and consists of more sandy, stiffer soils with a vs30 of 245 m/s. Shear wave velocities in the top 5 m increase from 143 to 200 m/s, whereas shear wave velocities at larger depths are above 200 m/s. For both profiles, the site response in the upper 30m of the subsurface has been calculated. Both nonlinear and equivalent linear approaches use the same material models for the soil, and similar methods of parameter identification. The main difference lies in the solution method used for obtaining the soil surface response.

From comparison of site response results obtained for both methods it is concluded:

- Differences in results obtained by the two methods are significant for short spectral periods, in particular for the spectral period of 0.01 s. The nonlinear method gives lower values of PGA (equivalent to the spectral acceleration at a

[2] Seismic loads on constructions are commonly evaluated using spectral accelerations from a response spectrum

spectral period of 0.01 s) compared to the equivalent linear method. The differences increase with increasing levels of input acceleration, applied as outcrop motion at the base of the soil model (30 m depth). For the Loppersum profile, differences in computed PGA are significant for input acceleration levels of 0.05 g and higher. For the stiffer Slochteren profile deviations between both methods are significant for input acceleration levels of 0.1 g and higher. For increasing levels of input acceleration, the nonlinear method results in up to a factor 2 lower PGA values than the equivalent linear method.

- At larger spectral periods of around 1 s and 2 s the differences between the equivalent linear and nonlinear methods are smaller than for the shorter spectral periods. At a spectral period of 2 s differences are small up to at least 1 g input acceleration levels. At a spectral period of 1 s differences are small up to 0.2 g input level for the Loppersum profile and 0.5 g input level for the Slochteren soil profile.

For the two profiles, covering a reasonable stiffness range of non-organic soil profiles found in the Groningen area, the equivalent linear site response method is an approximation of the fully nonlinear method. The equivalent linear method leads to conservative estimates of spectral accelerations, as it computes higher spectral accelerations for most spectral periods. There are a few exceptions for periods between 0.05 and 0.08 s, in which case the nonlinear response is slightly higher than the equivalent linear response.

In addition to the equivalent linear and nonlinear computations, additional simulations with fully linear elastic soil behavior were performed, for which shear stiffness of the soils were assumed constant and strain-independent. These calculations show that nonlinear soil response already affects PGA-values for the Loppersum profile for input acceleration levels of 0.02 g and higher. PGA-values for the stiffer Slochteren profile are affected by nonlinear soil response for input acceleration levels of 0.05 g and above. For these higher levels of input acceleration, PGA values resulting from fully linear elastic simulations exceed nonlinear results. In the nonlinear calculations, at higher input acceleration levels, ground motions are more damped due to the nonlinear hysteretic soil response. For the current registered events these acceleration levels have already been reached at several locations in the Groningen Field. It is recommended to use (future) seismic registrations in deep boreholes to validate the numerical models.

Contents

1 Introduction

Site response is the effect of shallow soils on the seismic ground motion. Previously, Probabilistic Seismic Hazard Analyses (PSHA) for the Groningen area [KNMI 2015] only took into account linear site response effects, where the soil is assumed to behave linear elastic and both shear stiffness of the soils and damping are assumed to be constant values. However, for the Groningen area the soil types are such that at design earthquake acceleration levels nonlinear site effects (in terms of a strain-dependent shear stiffness and damping) are expected to be significant.

Two main studies incorporating nonlinear site effects for the Groningen area have been carried out:

1) [Arup 2015] Groningen Earthquakes Structural Upgrading Site Response Analysis. This is part of the study for the Nederlandse Praktijk Richtlijn [NPR 9998], the Dutch Annex for the Eurocode 8 for seismic design of buildings.
2) [Bommer et al 2015] Development of Version 2 Ground Motion Prediction Equations (GMPEs) for Induced Earthquakes in the Groningen Field. The GMPEs translate earthquake magnitudes to surface ground motion and are used in Probabilistic Seismic Hazard Analyses.

In both studies, surface acceleration response spectra are defined, taking into account nonlinear site effects, hence it is important to understand the differences and similarities between the two studies. The first study by [Arup 2015] uses the nonlinear site response method, whereas the study by [Bommer et al 2015] uses the equivalent linear method. The material models used in [Arup 2015] for the nonlinear approach and in [Bommer et al 2015] for the equivalent linear approach are the same. The methods of parameter identification are similar as well in both studies. The main difference between both methods is the solution method. The equivalent linear method uses linear elastic properties of the soil layers and uses an iterative procedure to update the elastic properties, depending on the amount of computed shear deformation. The final iteration is still a fully linear elastic simulation with “equivalent” elastic parameters for the layer that approximate the nonlinear response of the soil layers. In the nonlinear method the dynamic wave equation with nonlinear stress-strain relationship is solved directly.

In this report nonlinear and equivalent linear site response simulations are compared. In addition, simulations based on linear elastic small strain parameters are performed to find the input acceleration levels at which nonlinear site effects become significant. Site response simulations are made for two different soil profiles, i.e. a profile representing soft clayey soil conditions in the Loppersum area and a stiffer, more sandy profile in the Slochteren area. Profiles with peats are not considered because of the limited amount of data available and the large variation in correlations for soil parameters for peats used by [Bommer et al 2015] and [Arup 2015]. The implications of this could not be studied within the time frame of this project.

This report is structured as follows: Chapter 2 gives an overview of site response and the assumptions commonly used in analyses as well as the conceptual differences between the nonlinear and equivalent linear site response methods. Chapter 3 summarizes the soil input parameters needed for site response analysis. Chapter 4 discusses in detail the (differences between) simulation results with the nonlinear and equivalent linear site response methods for the two profiles. Chapter 5 summarizes the conclusions and recommendations.

2 Overview of site response and equivalent linear vs nonlinear methods

In this chapter the assumptions commonly used in site response analyses are discussed, together with site response boundary conditions and the conceptual differences between the equivalent linear and fully nonlinear site-response methods.

2.1 Site response overview

Site response is the effect of shallow soil layers on the seismic surface ground motion. In most site response analyses, only horizontal ground motion is considered, as it is the dominant motion component responsible for structural damage. Figure 2.1 gives a graphical representation of site response, with upward travelling shear waves causing horizontal ground motion. In general soils tend to be stiffer at larger depths and softer closer to the surface. Under linear elastic conditions (i.e. assuming strain-independent soil properties and a linear site response) this leads to amplification of the motion amplitude towards the surface. However, the deformation behavior of soils is nonlinear and both the shear stiffness and damping of soils depend on the amount of shear deformation, which causes the site response to be nonlinear. In a nonlinear site response analysis, the strain dependence of stiffness and damping of the soil is incorporated.

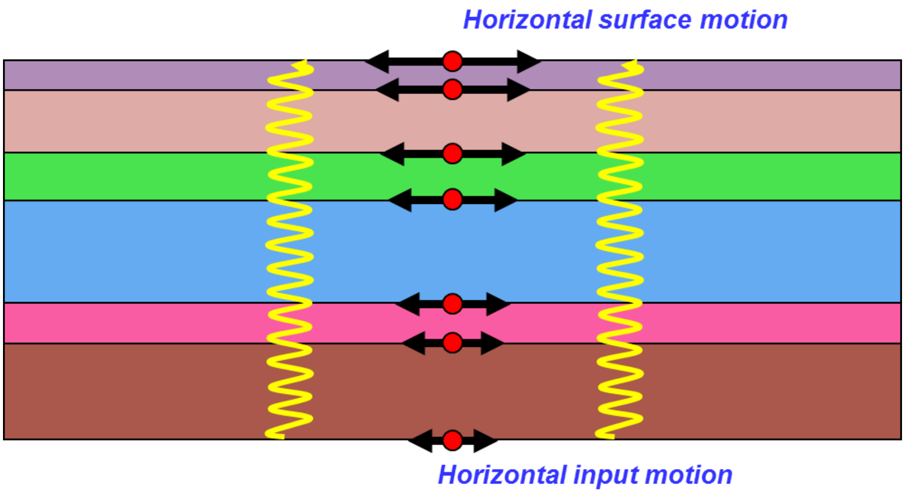

Figure 2.1 Graphical representation of site response for a horizontal input motion.

Figure 2.1 is a very simple depiction of site response, as the actual situation is much more complex with waves reflecting at each layer boundary as well as at the surface of the soil.

2.2 Site response analyses assumptions

To perform site response analyses, several simplifications are commonly made. It is assumed that earthquake incident waves are vertical, which means the ray-path or direction of wave propagation is perpendicular to the surface. This assumption enables one-dimensional site response simulations in horizontal soil layers. In site response software (e.g. SHAKE, STRATA, DEEPSOIL, SIREN) earthquake motion is applied in only one horizontal direction and it is implicitly assumed that both horizontal directions are independent. This is the case if soil behaviour is linear which for the Groningen field turns out to be in the range up to 0.02 - 0.05 g, as shown later in this report. For nonlinear situations one-dimensional site response methods are conservative compared to bi-directional site response methods [Motamed et al 2015]. Furthermore, compressional or P-wave input motions are not simulated by standard site response software.

The only relevant stress and strain component in standard site response software is the shear stress τ and shear strain γ. If for all soil layers in the site profile the constitutive relation between shear stress τ and shear strain γ is known for arbitrary (cyclic) loading, together with the soil density needed to define acceleration, the soil behaviour is fully determined.

In site response analyses, the complete nonlinear soil behavior, relating τ and γ is usually stated in terms of:

1) A small strain shear modulus G_0, sometimes denoted as G_{max}. For non-organic soils this is derived from shear wave velocity v_s and the soil density ρ, with the relation $G_0 = \rho v_s^2$.
2) A 'backbone curve', describing the reduction in (secant) shear modulus G with increasing shear strain γ, usually made dimensionless as $G(\gamma)/G_0$.
3) Unloading-reloading rules. Most nonlinear site response software, including finite element packages, use the so-called Masing rules for unloading an reloading of the soils [Phillips & Hashash 2009].

According to the Masing unloading/reloading rules, the stress-strain curve during initial loading follows the backbone curve. The reloading curve of any cycle starts with a shape that is identical to the shape of the positive initial loading backbone curve, enlarged by a factor of two. Extensions to these rules are used for special unloading/reloading cases (including crossing curves of previous cycles) that are implemented in most nonlinear site response software.

In a fully nonlinear site response analysis, the above 3 components are incorporated. In the equivalent linear method, components 1) and 2) are also used, but instead of the unloading-reloading rules a damping curve is used. This damping curve is generally calculated from the Masing unloading/reloading rules, with a possible modification for larger strains [Phillips & Hashash 2009].

2.3 Conceptual difference of equivalent linear and nonlinear site-response methods

The fully nonlinear method uses a time integration of the wave equation taking into account the correct loading, unloading or reloading stiffness according to the Masing or modified Masing rules in each time step.

The equivalent linear method solves the wave equation for a linear elastic soil with constant shear modulus and damping per layer. A procedure is used to determine this constant shear modulus and damping in such way as to approximate the nonlinear response. An iterative process is used to determine the elastic shear modulus and damping ratio ξ from the effective strain γ_{eff} reached in each layer. This effective strain γ_{eff} is defined as a fraction of the maximum strain γ_{max} reached in a layer $\gamma_{eff} = \alpha\gamma_{max}$. This fraction α is typically in the range from 0.5 to 0.7 [Kramer 1996]. In site response software applications the default value for this fraction is often $\alpha = 0.65$. [Kramer 1996] and the DEEPSOIL manuals give a correlation with earthquake magnitude M of $\alpha = \frac{M-1}{10}$. It is unclear to what extent this correlation holds for the Groningen area.

The iterative process used in the equivalent linear approach is shown in Figure 2.2. An elastic simulation is performed using initial guesses for shear modulus and damping ratio $G^{(1)}$ and $\xi^{(1)}$ corresponding to a strain zero. After the simulation in each layer an effective non-zero strain $\gamma_{eff}^{(1)}$ is computed, which corresponds to different values of shear modulus $G^{(2)}$ and damping ratio $\xi^{(2)}$. Updated values of shear modulus and damping are then used in the following iteration, resulting in an updated effective strain in the layer $\gamma_{eff}^{(2)}$, corresponding to new values $G^{(3)}$ and $\xi^{(3)}$. This process is continued until a certain accuracy is reached.

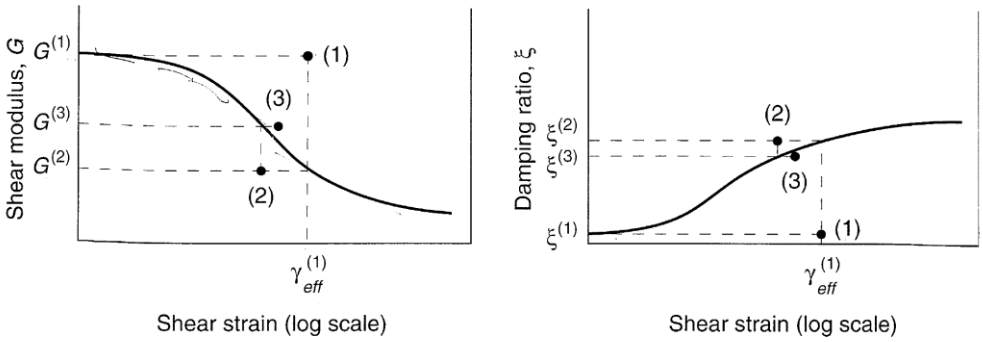

Figure 2.2 iteration procedure towards a strain compatible shear modulus and damping ratio from [Kramer 1996].

It is clear from the above that the equivalent linear method is an approximation to the actual nonlinear material behaviour. In contrast with the nonlinear method, in the final iteration damping and shear modulus are still a constant per layer over the entire input earthquake signal.

In practice it is found that for longer spectral periods, lower peak ground accelerations, or lower maximum shear strain (because of stiff soil), the two methods give similar results. Figure 2.3 from [Kaklamos et al. 2013] gives an example from literature, suggesting that for peak ground accelerations above 0.1 g, and maximum shear strains above 0.1%, nonlinear methods need to be applied for shorter spectral periods. This will be checked for two Groningen profiles in the next chapter.

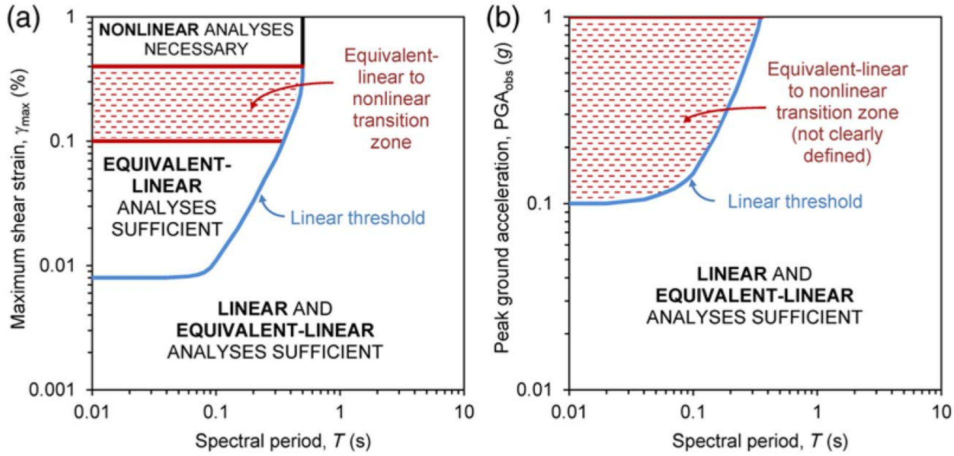

Figure 2.3 Approximate ranges of applicability of linear, equivalent linear and nonlinear site-response analysis [Kaklamos et al. 2013].

2.4 Site response boundary conditions

In site response simulations the input acceleration signals can be applied at the bottom boundary of the soil profile in two ways: as 'within motion' or as 'outcrop motion'.

Within motion is easiest to understand. Suppose that for a site the soil profile is modelled up to a certain depth and the actual ground motion during an event is measured at that same depth. In this case, this ground motion can be applied directly as an acceleration time history at the bottom of the soil model. This condition is approximately equivalent to the presence of a rigid base rock underlying the soil model. In case of a base rock much more rigid than the overlying soil, the motion of the base rock is undisturbed by reflecting waves from the overlying soil. Within motion can be used either when measurements are carried out at depth or when the base rock can be considered rigid compared to the overlying soil.

Figure 2.4 explains the definition of outcrop motion. On the right side of the figure the rock outcrops and the incoming wave travels in a homogeneous (bedrock) material. In case of outcrop motion the incoming wave is such that the rock outcropping motion is the requested input acceleration time signal. The site response software generates the incoming wave that would result in the requested input acceleration time signal if the bedrock was outcropping as in the right of the figure and apply that incoming wave to the situation on the left of the figure where soil is overlying the bedrock for actual site response. For site response software packages to generate the incoming wave properly, the properties of the bedrock must be specified.

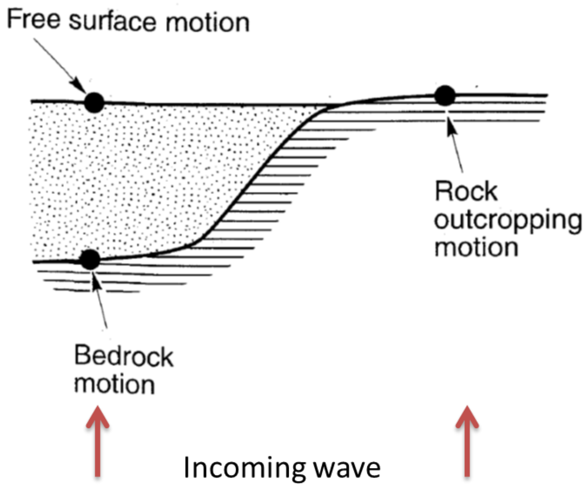

Figure 2.4 Ground response terminology for soil overlying bedrock, b) from [Kramer 1996].

In equivalent linear site response programs that operate in the frequency domain (e.g. SHAKE, STRATA) this incoming wave can be achieved directly as the software solves using upward and downward travelling wave coefficients.

In (nonlinear) time domain site response analyses generating the incoming wave is less trivial and the method described in [Joyner & Chen 1975] is used where the signal is applied at the end of a dashpot at the bottom of the model having properties of the bedrock. In this way reflected waves from the layers above can be (partly) transmitted back into the bedrock, while keeping the upward wave amplitude as required.

In [Arup 2015], as in this study, the input signals have been specified as outcrop motion at 30 m depth. As no bedrock is encountered at this depth, soil layers below 30 m depth characterised by shear wave velocity of 300 m/s or higher function as bedrock in the simulations.

3 Soil parameters

This chapter summarizes the material parameters used in the site response approaches by [Arup 2015] and [Bommer et al. 2015]. The parameters used to define the backbone curve are identical in the two studies and differences only exist in the damping definition, for which [Bommer et al. 2015] use two additional parameters.

The correlations used in the two studies to determine the parameters from experimental data are given below. It is marked here that some differences in correlations used in the two studies exist. As the focus of the present study is on the comparison of the equivalent linear and nonlinear site response methods, these differences in correlations have not been studied.

3.1 Soil density

To describe soil inertia the soil density must be known.

Parameter	**Description**
ρ	Soil density

Table 3.1 Parameters for site response: soil density.

3.2 Shear modulus degradation curve parameters

For the non-organic soil materials, both [Arup 2015] and [Bommer et al. 2015] use the hyperbolic model from [Darendeli 2001] :

$$\frac{G}{G_{max}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_r}\right)^a}$$

Here, G is the shear modulus, G_{max} is the maximum shear modulus (at small strains), γ is the shear strain and γ_r and a are model parameters. The ratio $\frac{G}{G_{max}}$ versus the shear strain γ is commonly referred to as the normalized shear modulus degradation curve, as the shear modulus G decreases for increasing strain. In some literature the term reduction is used instead of degradation. In the $\frac{G}{G_{max}}$ versus strain curve the ratio $\frac{G}{G_{max}} = 1$ at a strain zero. Site response packages need also the G_{max} parameter to be able to calculate the shear modulus degradation curve is $G(\gamma)$.

This hyperbolic model in some cases leads to an incorrect shear strength of the soil. To remedy this, the hyperbolic model is extended to incorporate the correct shear strength with the method of [Yee et al. 2013]. This introduces two extra parameters, the shear strength τ_{ff} and a crossover strain γ_I. For strains up to γ_I the above hyperbolic model is used and for larger strains this model is gradually adapted to reach the required shear strength. For sands the shear strength τ_{ff} and crossover strain γ_I parameters are not used in [Arup 2015] and [Bommer et al 2015], see also the discussion below Figure B1 in appendix B. The full parameter set used to define the shear modulus degradation curve $G(\gamma)$ is given in Table 3.2, together with the correlations used by [Arup 2015] and [Bommer et al 2015].

Parameter	**Description**	**Value/correlation used**	
		Arup 2015	Bommer et al 2015
G_{max}	Small strain shear modulus	ρv_s^2	ρv_s^2
a	Power of the hyperbolic model	0.919	0.919
γ_r	Reference strain	$\left(0.0352 + 0.001 I_p OCR^{0.3246}\right)\left(\frac{\sigma'}{p_a}\right)^{0.3483}$	
τ_{ff}	Shear strength	$0.25\sigma_v OCR^{0.8}$	$\frac{q_t - \sigma_v}{11.5}$
γ_I	Crossover strain for shear strength branch	0.1%	0.3%

Table 3.2 Parameters for the shear modulus reduction curve used by [Arup 2015] and [Bommer et al. 2015].

In the table, ρ is soil density, v_s the shear wave velocity, I_p the plasticity index, OCR the soil overconsolidation ratio, σ' the mean effective stress, p_a the atmospheric pressure, σ_v the total vertical stress and q_t the cone resistance obtained from cone penetration tests. The reader is referred to [Darendeli 2001] and [Yee et al. 2013] for a more extensive description of the parameters used to define the hyperbolic shear modulus degradation curve.

For the damping curve [Arup 2015] uses the Masing damping rules, while [Bommer et al. 2015] use Masing damping corrected with a damping reduction factor F(ϒ) from [Darendeli 2001]:

$$F(\gamma) = p_1\left(\frac{G_{sec}}{G_{max}}(\gamma)\right)^{p_2}$$

Table 3.3 lists the parameters used to define the damping reduction factor.

Parameter	**Value [Arup 2015]**	**Value used by [Bommer et al 2015]**
p_1	Not used	$0.6329 - 0.0057\ln(10)$
p_2	Not used	0.1

Table 3.3 Parameters for the modified Masing damping curve used by [Bommer et al. 2015].

The Masing damping underestimates damping at small strains. It is common to use an additional small strain damping ξ_{min} to correct this.

Parameter	**Value [Arup 2015]**	**Value [Bommer et al 2015]**
ξ_{min}	0.5%	$\left(0.8005 + 0.01291 I_p OCR^{-0.1069}\right)\left(\frac{\sigma'}{p_a}\right)^{-0.2889}$

Table 3.4 Parameter for small strain damping used by [Arup 2015] and [Bommer et al. 2015].

4 Equivalent linear and nonlinear site-response results for two representative soil profiles in the Groningen area

In this chapter, site response simulation results for a Loppersum soil profile with a soft clay top layer and a Slochteren soil profile with a more stiff sandy soil are presented. The simulations were made with the program DEEPSOIL for both the equivalent linear and nonlinear site response method using the same parameters, input motions and boundary conditions for both methods.

The comparison presented here does not cover all differences in approach used by [Bommer et al 2015] and in the NPR [Arup 2015]. In the NPR, as in the simulations performed here, acceleration signals are used that are applied at 30 m depth as outcrop signals (see Section 2.4). In [Bommer et al 2015] signals have been applied at larger depth of 350 m, at the base of the Upper North Sea Formation (NU_B) on the basis of a different input spectrum. As input, the Random Vibration Theory (RVT) method is used rather than using time signals. Some variation in soil parameters still exist as discussed in the previous chapter. As the present study focusses on the comparison of equivalent linear and nonlinear site response methods, the above differences in input parameters are not addressed in this chapter.

4.1 Simulations for a Loppersum soil profile with clayey soil

A soil profile representative of Loppersum soft soil conditions is studied first. This is profile 60533 from [Arup 2015], with an average shear wave velocity in the upper 30m (vs30) of 156 m/s. The top of the soil profile consists of a soft clay layer of 8 m thickness, with shear wave velocity increasing from 100 to about 112 m/s. Table A1 shows the shear wave velocities with depth for this profile. Figure A1 in Appendix A, taken from [Arup 2015], shows the cone resistance, cone friction and shear wave velocity of this profile. In addition, the correlations are plotted as derived by [Arup 2015] for the NPR study. For this profile, seven soil layers are defined, and figures A2 and A3 show the shear modulus degradation and damping curves. Although only seven soil layers are defined, shear wave velocities (and therefore G_{max}) are not constant over these layers. Sublayers are modelled having the same G/G_{max} curve but different G_{max} values.

For comparison purposes, during the NPR study a STRATA (an open source equivalent linear site response software) input file of this profile has been provided by Arup containing all the layers with shear wave velocities, backbone and damping curves. The STRATA input has been converted to Siren (the nonlinear site response software by Arup) and to DEEPSOIL (free nonlinear site response software by [Hashash et al 2010]). The mean spectral response obtained from STRATA, Siren and DEEPSOIL for a set of 0.42 g outcrop input signals is compared in figure 4.1 and 4.2. These figures show that nonlinear results in DEEPSOIL and SIREN are in close agreement, both in terms of the response spectrum giving spectral accelerations and in terms of the maximum strain reached. The equivalent linear results are similar for longer periods (T>1s) but deviate for shorter periods. At 0.4 s, the spectral acceleration is close to 1 g for the equivalent linear method whereas for the nonlinear methods 0.7 g is obtained. For periods approaching zero, equivalent to the PGA value, the equivalent linear method gives a mean PGA of 0.36 g, while the nonlinear method results in a lower mean PGA of 0.18 g. Figure 4.3 shows that when plotted in the figure by [Kaklamos et al. 2013], the results for the 60533 profile are within the transition zone from equivalent linear to nonlinear, with the left graph indicating the simulations being partly plot within the "nonlinear analyses necessary" region.

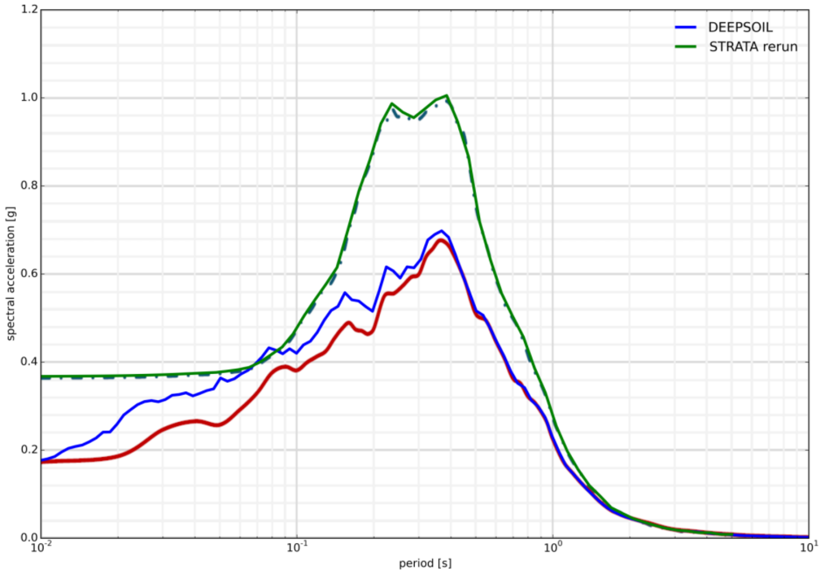

Figure 4.1 Mean spectral response for Loppersum profile 60533 of a set of 0.42 g input signals. Green line: equivalent linear result using STRATA, red line: nonlinear simulation with Siren, blue line: nonlinear result with DEEPSOIL.

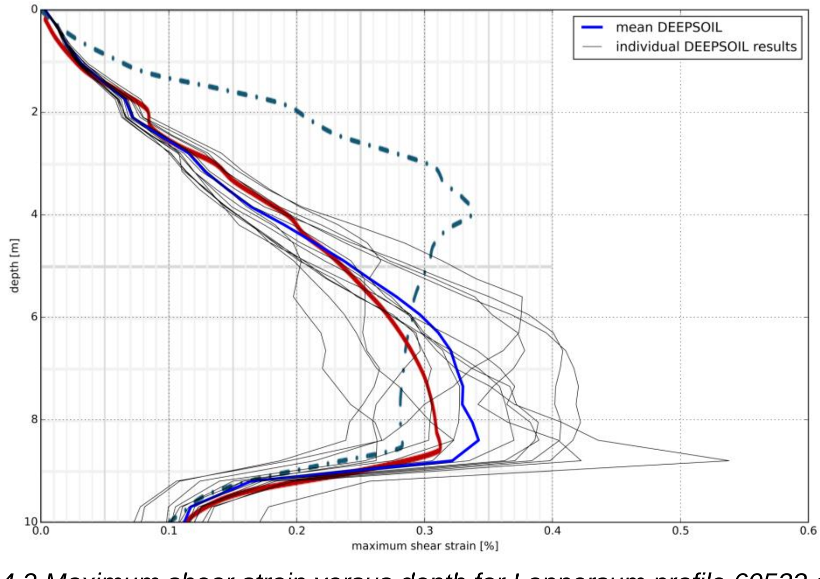

Figure 4.2 Maximum shear strain versus depth for Loppersum profile 60533 of a set of 0.42 g input signals. dotted line: equivalent linear (mean) result using STRATA, red line: nonlinear simulation (mean) result with Siren, blue line: nonlinear (mean) result with DEEPSOIL. Thin lines: individual simulations with DEEPSOIL.

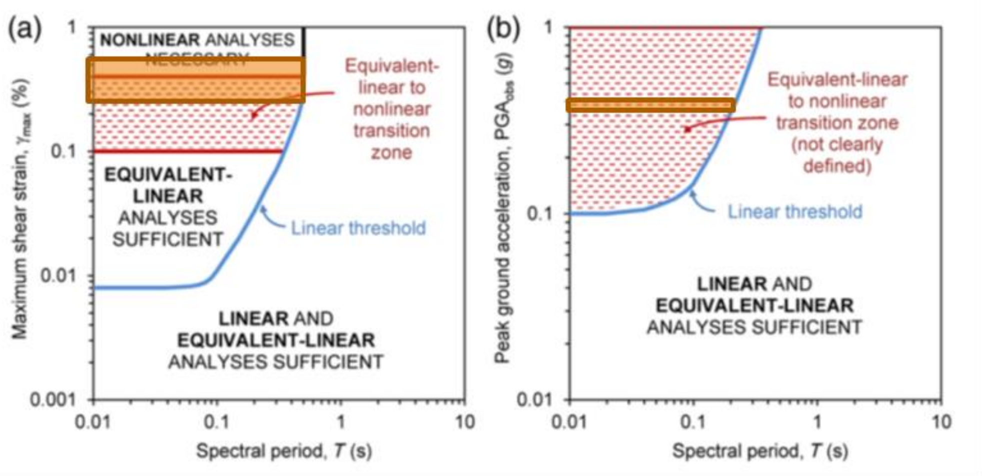

Figure 4.3 Approximate ranges of applicability of linear, equivalent linear and nonlinear site-response analysis [Kaklamos et al. 2013]. Shaded orange zone represents the 60533 site response result.

To investigate for the Loppersum profile at which input acceleration levels and maximum reached strain levels in the layers differences between equivalent linear and nonlinear site response methods become noticeable, signals are applied at 21 input acceleration levels (or g-levels) ranging from 0.01 g to 1 g as input in DEEPSOIL. The signals used are the February 2016 NPR signals downloaded from the NEN website [NPR 2016]. The DEEPSOIL software has the ability to generate equivalent linear results at the same time as the nonlinear results and is also able to generate fully linear results based on the small strain stiffness G_{max} and small strain damping ξ_{min}. In the NPR 11 signals in 2 directions are defined, which add up to 22 individual signals. Applied at 21 g-levels this results in 22*21=462 nonlinear simulations, 462 equivalent linear simulations and 462 linear simulations. The results are summarized in Figure 4.4 and 4.5, showing a spectral response graph for four spectral periods (0.01 s, 0.5 s, 1.0 s and 2.0 s) versus outcrop input acceleration level. The points at the 21 different g-levels are connected for the same input signal, resulting in 22 lines for the nonlinear case in each graph, as well as 22 lines for each of the other cases.

The linear site response shows straight lines as expected. The PGA (spectral acceleration at a period of 0.01 s) is about a factor 2 higher than the input outcrop acceleration, regardless of the input g-level. The equivalent linear and nonlinear results show a PGA saturation, with no significant increase of the PGA observed at input levels above 0.4 g for the nonlinear simulation and above 0.6 g for the equivalent linear case.

Figure 4.5 shows that at 0.1 g outcrop acceleration levels the PGA values are about 0.1 g for the nonlinear case and 0.2 g for the linear case. This means that at acceleration levels of 0.1 g behaviour of soils, representative for the Loppersum area, is already likely to be nonlinear. Accelerations approaching this level have been recorded in some events and locations in the Groningen field. Below 0.02 g the nonlinear, linear and equivalent linear results for the PGA are similar. Below 0.05 g the nonlinear and equivalent linear results for PGA are similar.

At longer spectral periods, the differences between results from the three methods become smaller. In particular at a periods of 1 s the linear, nonlinear and equivalent linear simulations are similar for input acceleration levels up to 0.2 g. For higher input outcrop acceleration levels, the nonlinear results are lower than the equivalent linear results. At a period of 2 s, results are similar for all three methods up to approximately 0.3 g input outcrop acceleration level, whereas linear and equivalent linear results are similar up to 1 g.

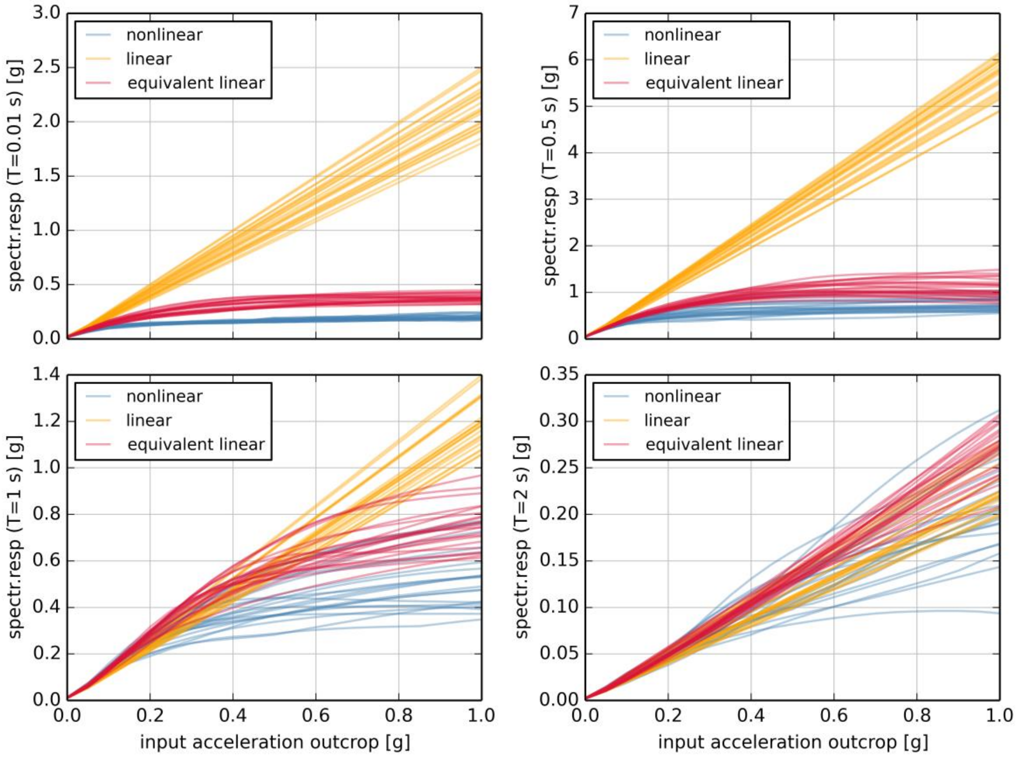

Figure 4.4 Comparison of nonlinear, equivalent linear and linear site response at 4 spectral periods for the typical Loppersum soil profile.

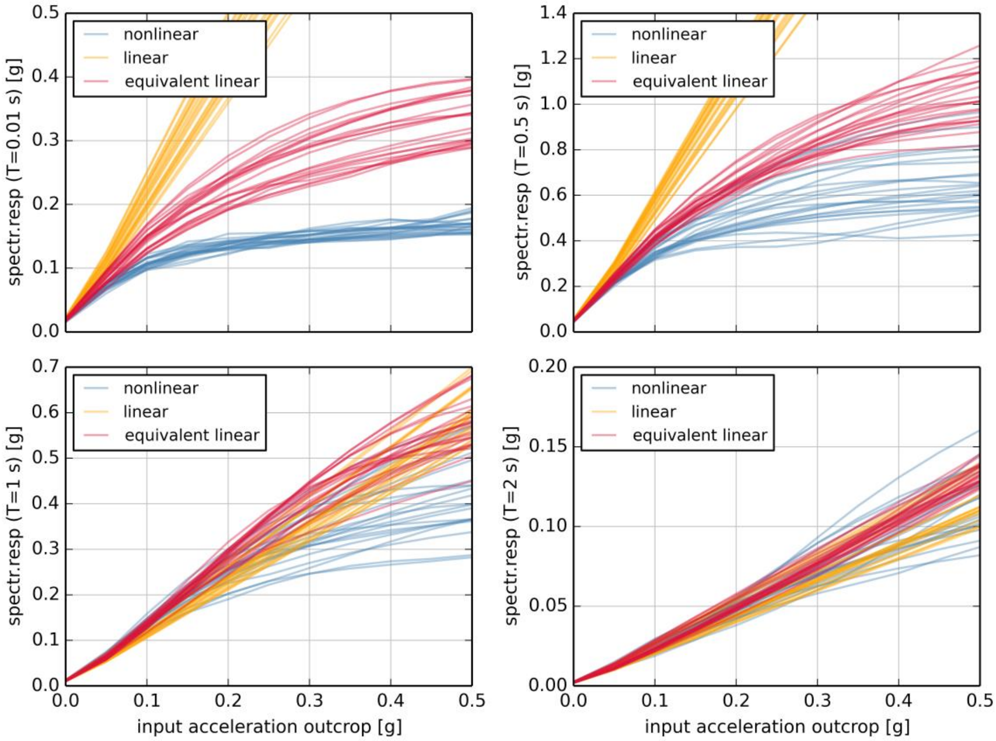

Figure 4.5 Zoomed in version of figure 5.6. Comparison of nonlinear, equivalent linear and linear site response at 4 spectral periods for the typical Loppersum soil profile.

Figure 4.6 below shows the shear strains versus depth for 0.01 g, 0.1 g, 0.2 g and 0.4 g outcrop acceleration input levels. At 0.01 g the results of the three methods are similar. Results for individual methods diverge with increasing input acceleration levels. Figure 4.6 shows that at input outcrop acceleration levels of 0.1 g and above, equivalent linear and nonlinear methods give much higher strain levels than the linear method. In general highest strains are computed for the nonlinear simulations.

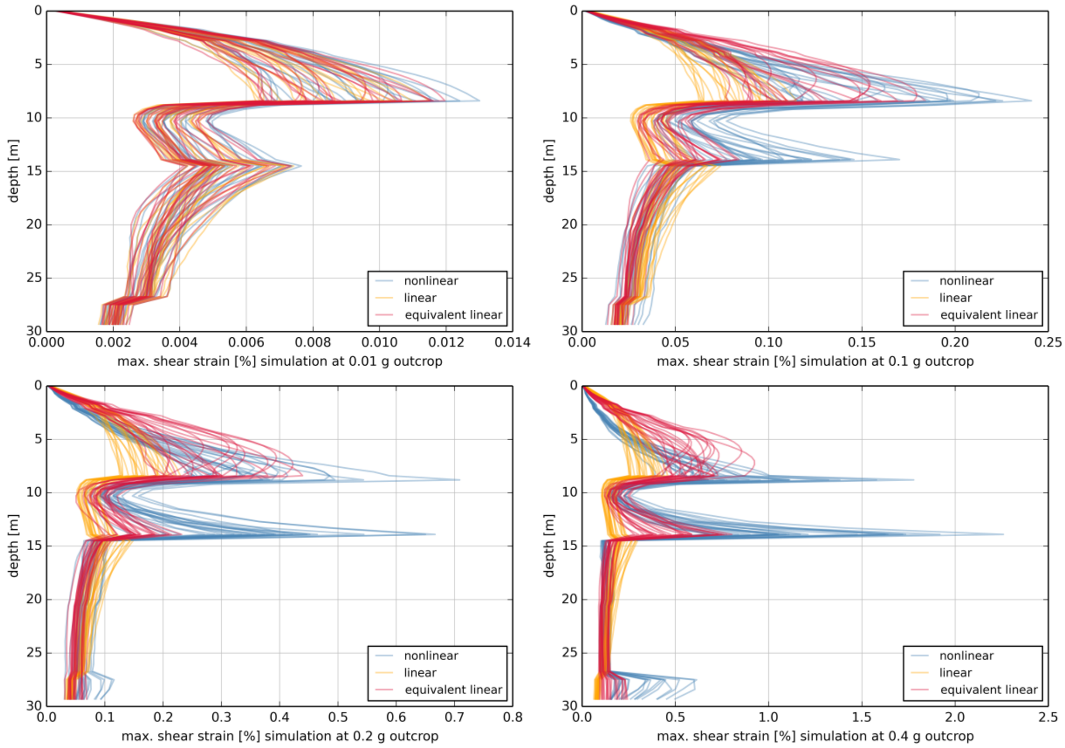

Figure 4.6 Maximum shear strain versus depth for soft clayey Loppersum profile at 4 outcrop input g-levels.

In figure 4.6, at 0.01 g, the strain is mostly below 0.012%. From figure 4.5 it can be observed that for this input level the spectral response at all periods of linear, equivalent linear and nonlinear methods is almost similar. This is in agreement with Figure 4.3 [Kaklamos et al. 2013], which shows that at strains below 0.008 % linear and equivalent linear analyses can be used.

Strains at input accelerations of 0.1 g in Figure 4.6 exceed 0.2% for some of the nonlinear simulations. Figure 4.3 [Kaklamos et al. 2013], shows that for short periods, strains of 0.2% plot within the transition zone between equivalent linear and nonlinear methods. Figure 4.5 shows that for short periods the equivalent linear results start to deviate from the nonlinear results.

Strains at input accelerations of 0.2 g and 0.4 g in Figure 4.6 exceed 0.4%, for which Figure 4.3 [Kaklamos et al. 2013] points out that a nonlinear analysis is necessary at short periods. At 0.4 g, the limit of 0.4% is exceeded in all nonlinear analyses. At 0.2 g in a part of the analyses the 0.4% strain limit is exceeded. Figure 4.5 shows that at these g-levels the differences between equivalent linear and nonlinear analyses are significant. This is in agreement with the plot from [Kaklamos et al 2013].

4.2 Simulations for a sandy profile

A second profile is considered, representing soil conditions in the Slochteren region, which are in general more sandy and stiffer than soil conditions in the Loppersum section. The soil profile has an average shear wave velocity of vs30=245 m/s. The shear wave velocity in the top 5 m increases from 143 to 200 m/s. Table B1 in

Appendix B shows the shear wave profile used and figures B1 and B2 show the shear modulus degradation and damping curves. The shear modulus degradation curves are only slightly different for the layers. This is because the plasticity index of sand being zero, in combination with the [Darendeli 2001] correlation used for determining the shear modulus degradation curve parameters, leads to small differences in reference strain, as discussed in Appendix B. The same signals and outcrop g-levels are used in the simulations as for the Loppersum profile. Figures 4.7 to 4.9 show the simulation results.

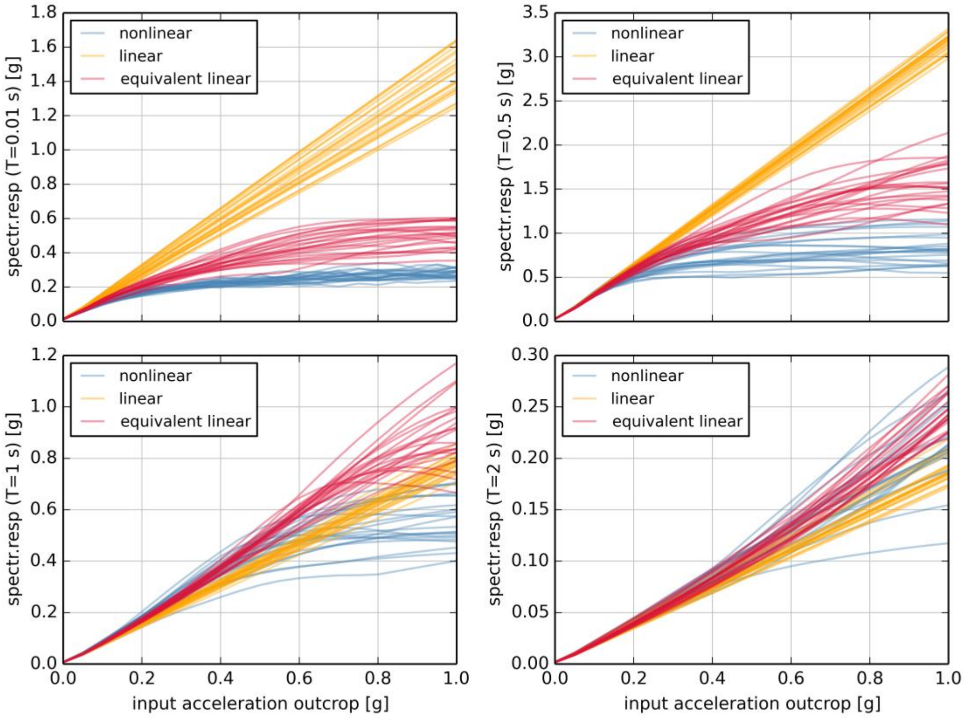

Figure 4.7 Comparison of nonlinear, equivalent linear and linear site response at 4 spectral periods for the Slochteren sandy soil profile.

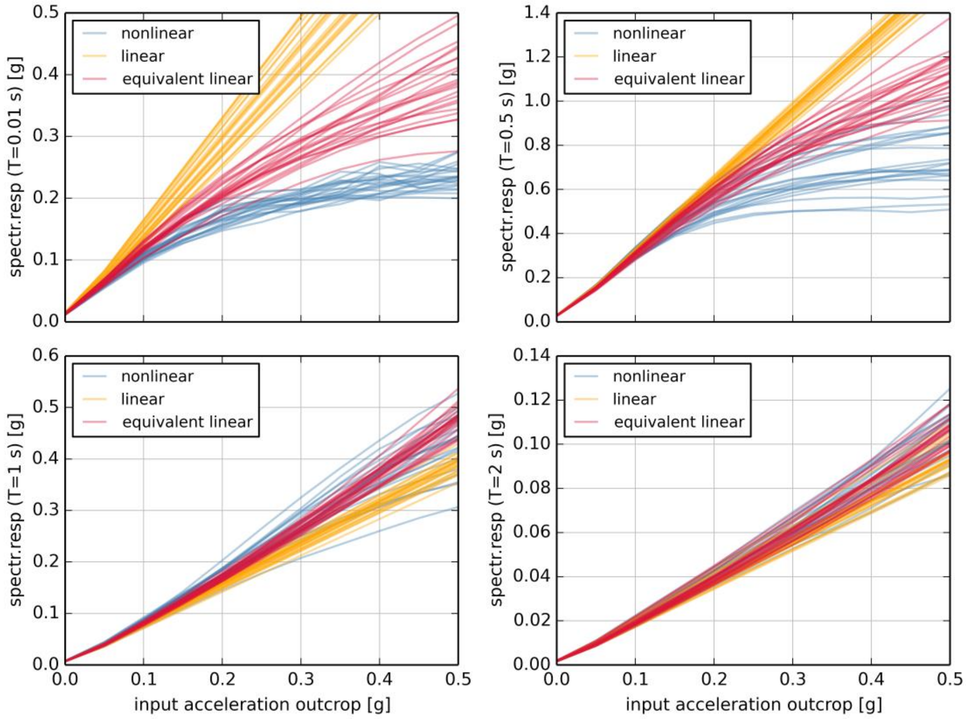

Figure 4.8 Zoomed in version of figure 5.6. Comparison of nonlinear, equivalent linear and linear site response at 4 spectral periods for the typical Slochteren sandy soil profile.

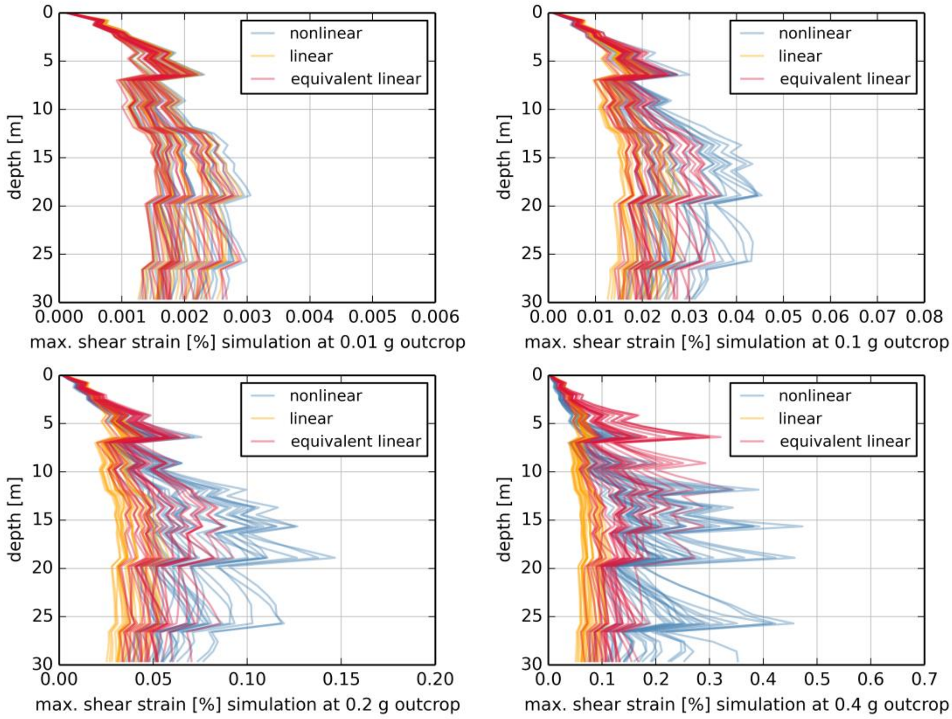

Figure 4.9 Maximum shear strain versus depth for the typical Slochteren sandy soil profile at 4 outcrop input g-levels.

Figure 4.7 shows the spectral response curves from the linear method are straight lines as expected. The PGA (at 0.01s) is about a factor 1.4 higher than the input outcrop acceleration, regardless of the input g-level. This is lower than the factor 2 found for the Loppersum profile, which can be explained by the higher soil stiffness. The equivalent linear and nonlinear results show a saturation of PGA, which shows no significant increase for input outcrop acceleration levels above 0.4 g for the nonlinear simulation and above 0.6 g for the equivalent linear case.

Figure 4.8 shows that below input acceleration levels of 0.07 g the nonlinear, linear and equivalent linear results for the PGA are similar. Below 0.1 g the nonlinear and equivalent linear results for PGA are similar. Computed PGA values are higher than for the Loppersum profile.

Comparing the strains versus depth for the Slochteren profile (Figure 4.9) and the Loppersum profile (Figure 4.6) shows a factor 4 to 5 lower maximum strain for the Slochteren profile, again due to the higher stiffness of the profile. For input acceleration levels of 0.01 and 0.1 g the strains of the linear, equivalent linear and nonlinear methods are similar. For 0.2 g and 0.4 g, computed nonlinear strains are clearly larger.

For the Loppersum case, in some simulations strains computed for 0.1 g input acceleration exceed 0.2 %. Strains of 0.2% at shorter periods plot within the transition zone from equivalent linear to nonlinear in Figure 4.3 [Kaklamos et al. 2013]. For the Slochteren profile, strains computed for 0.1 g input accelerations are below 0.045%, for which equivalent linear analyses are sufficient according to [Kaklamos et al 2013]. Indeed, this is confirmed by Figure 4.8, where similar results are found for nonlinear and equivalent linear simulations.

At input acceleration levels of 0.2 g, computed strains are higher than 0.1% for some simulations. For this g-level, Figure 4.8 shows a divergence of equivalent linear results and nonlinear results for the PGA. This is in agreement with Figure 4.3 from [Kaklamos et al 2013], in which 0.1% strain is the boundary of the transition zone from equivalent linear to nonlinear analysis.

It is noted here that the maximum strain calculated in equivalent linear simulations for input accelerations of 0.2 g is lower than 0.1%. In both Figure 4.9 and Figure 4.6, at higher outcrop input g-levels, the maximum strains are highest for the nonlinear case. Whereas lower strains resulting from equivalent linear analyses, using Figure 4.3, may lead to the conclusion that equivalent linear analyses are sufficient for these cases, nonlinear simulation show that the transition zone is already reached. Judging whether or not equivalent linear analyses are sufficient based on Figure 4.3 [Kaklamos et al 2013] is more accurately done using nonlinear analyses, especially if a case is near the edge of the transition zone.

At longer spectral periods, the differences between outcomes of the three calculation methods become smaller, as was also observed for the Loppersum profile.

5 Conclusions and recommendations

In this study, site response results obtained with the equivalent linear and nonlinear site response methods have been compared for two different soil profiles, which are representative of soil conditions in the Groningen field. Fully linear simulations have been performed as well to observe the acceleration levels at which nonlinear effects become significant.

The first soil profile studied is a profile which represents soil conditions in the Loppersum area. The soil profile consists of relatively soft clayey soil, characterized by an average shear wave velocity in the upper 30m (vs30) of 156 m/s. The top layer consists of an 8 m thick soft clay, with shear wave velocities increasing from 100 to about 112 m/s. The second profile represents soil conditions in the Slochteren area and consists of more sandy, stiffer soils with a vs30 of 245 m/s. Shear wave velocities in the top 5 m increase from 143 to 200 m/s, whereas shear wave velocities at larger depths are above 200 m/s.

Identical material models and parameters have been used in the equivalent linear and nonlinear methods and the same input signals developed in the NPR have been used in both methods. Input signals are applied at 30 m depth assuming a bedrock shear wave velocity of 350 m/s below 30 m. Input signals have been scaled from 0.01 to 1 g to study the effects of increasing acceleration.

5.1 Conclusions

The nonlinear method leads to significantly lower PGA levels (up to about a factor 2 for increasing input acceleration levels) compared to the equivalent linear method. The differences increase with increasing input acceleration level. For the Loppersum profile, differences in computed PGA become significant for input acceleration levels of 0.05 g and higher. For the stiffer Slochteren profile deviations between both methods are significant for input acceleration levels of 0.1 g and higher.

At larger spectral periods of around 1 s and 2 s the differences between the equivalent linear and nonlinear methods are smaller than for the shorter spectral periods. At 2 s differences are small up to at least 1 g input acceleration levels. At 1 s differences are small up to 0.2 g input level for the Loppersum profile and 0.5 g input level for the Slochteren soil profile.

For the two profiles, covering a reasonable stiffness range of non-organic soil profiles found in the Groningen area, it can be concluded that the equivalent linear site response method is an approximation of the fully nonlinear method. The equivalent linear method leads to conservative estimates of the spectral accelerations, as it computes higher spectral accelerations for most spectral periods. There are a few exceptions for periods between 0.05 and 0.08 s, in which case the nonlinear response is slightly higher than the equivalent linear response

Maximum strains in the nonlinear method are larger than maximum strains in the equivalent linear method. Strain based criteria [Kaklamos et al 2013] to determine the application range of equivalent linear methods can lead to a too wide application range when strains based on equivalent linear methods are used to test against these criteria.

Additional simulations with fully linear elastic behaviour show that nonlinear soil response already affects PGA-values for the Loppersum profile for input acceleration levels from 0.02 g. PGA-values for the stiffer Slochteren profile are affected by nonlinear soil response for input acceleration levels of 0.05 g and above.

5.2 Recommendations

This study specifically addresses the differences between the equivalent linear and nonlinear site response methods which are used resp. by [Bommer et al 2015] and the NPR [Arup 2015]. It is noted here that not all differences are covered. In this report (as in the NPR study) signals are applied at 30 m depth, while in [Bommer et al 2015] signals have been applied at larger depth of 350 m at the base of the Upper North Sea Formation (NU_B). It is recommended to investigate the effect of a deeper base for site response methods.

Soil profiles with peats are not considered in this report because of limited available data and large variation in correlations for soil parameters for peats used by [Bommer et al 2015] and [Arup 2015]. The implications of this could not be evaluated within the time frame of this project. If a comparison between nonlinear and equivalent linear site response methods is to be made for soil profiles containing peats, it is recommended to take into account the multitude of modelling possibilities for peats and validate these with experimental and field data.

The nonlinear site response calculations depend on correlations from experiments performed on non-Dutch sands and clays [Darendeli 2001]. The simulations here show that for input acceleration levels higher than 0.02 – 0.05 g, nonlinear effects are noticeable. Current registered surface accelerations due to earthquakes of up to 0.08 g have been measured. It is expected that a reduction in surface acceleration due to the nonlinear effect (compared to the fully linear case) is already present in the field. It is recommended to use the KNMI borehole sensor arrays to validate the site response models. Sensors at surface 50 m, 100 m, 150 m and 200 m are available, which can be used to study nonlinear soil effects. In addition, measured accelerations at depth could serve as input signal using the ‘within’ boundary condition for site response. Events with lower registered surface accelerations can be used to calibrate linear site parameters, while events with higher surface accelerations exceeding 0.05 g can then be used to study the nonlinear effect and to validate the site response models.

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7 Signature

Delft, 25 March 2016

TNO

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A Data from soil profile 60533 (Loppersum)

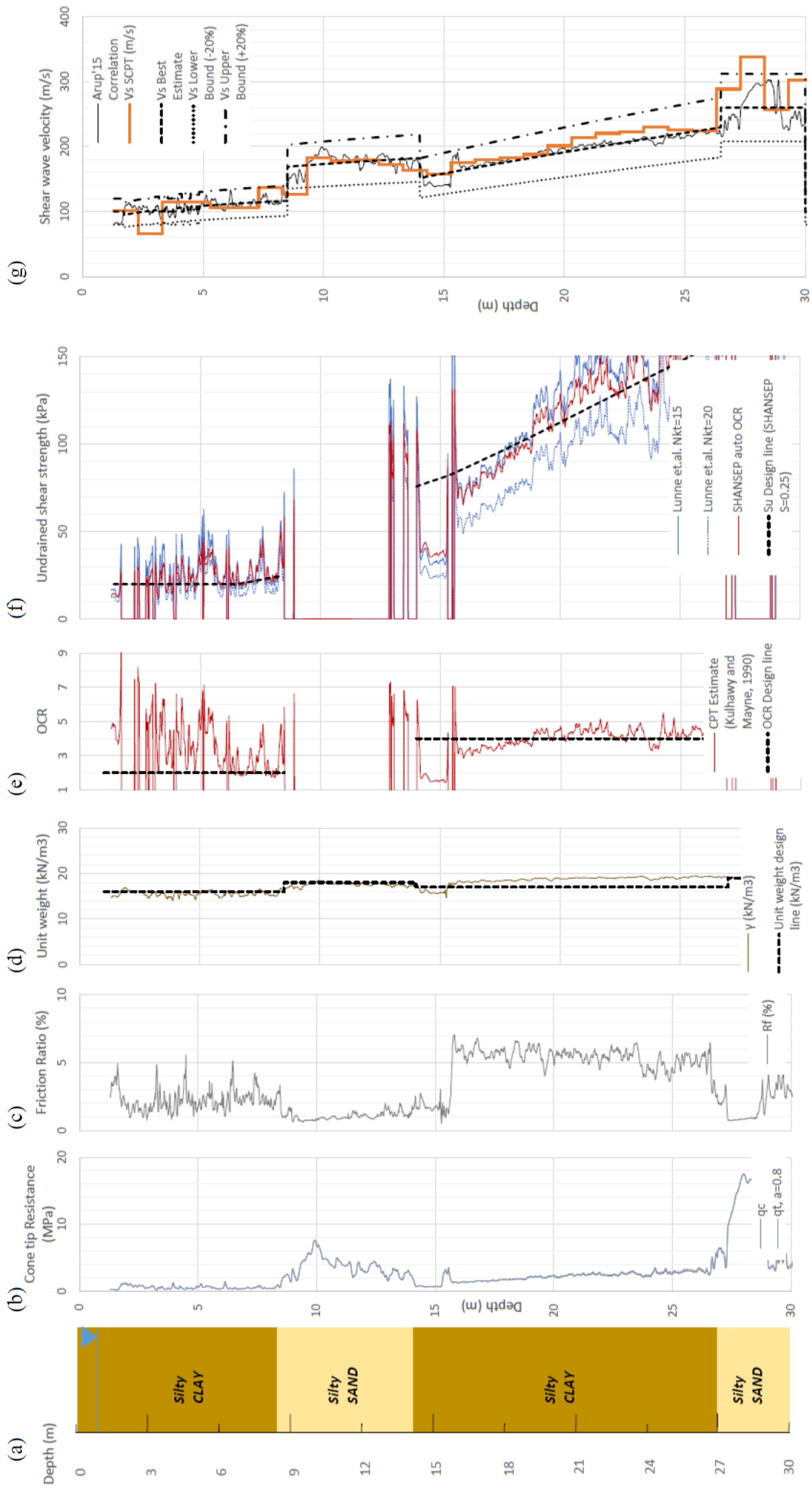

Figure A1 Typical Loppersum profile no 60533 with correlations from [Arup 2015].

Table A1 Loppersum profile 60533 shear wave velocities [Arup 2015].

depth [m]	thickness [m]	soil type	unit weight [kN/m^3	Vs [m/s]
0	0.35	Clay 0-2m	16	100
0.35	0.35	Clay 0-2m	16	100
0.7	0.35	Clay 0-2m	16	100
1.05	0.35	Clay 0-2m	16	100
1.4	0.35	Clay 0-2m	16	100
1.75	0.35	Clay 0-2m	16	100
2.1	0.35	Clay 2-4m	16	100
2.45	0.35	Clay 2-4m	16	100
2.8	0.35	Clay 2-4m	16	100
3.15	0.35	Clay 2-4m	16	102.67
3.5	0.35	Clay 2-4m	16	104.07
3.85	0.35	Clay 2-4m	16	105.36
4.2	0.35	Clay 4-8.5m	16	106.55
4.55	0.35	Clay 4-8.5m	16	107.65
4.9	0.35	Clay 4-8.5m	16	108.68
5.25	0.35	Clay 4-8.5m	16	109.65
5.6	0.35	Clay 4-8.5m	16	110.57
5.95	0.35	Clay 4-8.5m	16	111.44
6.3	0.35	Clay 4-8.5m	16	112.26
6.65	0.35	Clay 4-8.5m	16	113.05
7	0.35	Clay 4-8.5m	16	113.79
7.35	0.35	Clay 4-8.5m	16	114.51
7.7	0.35	Clay 4-8.5m	16	115.2
8.05	0.35	Clay 4-8.5m	16	115.86
8.4	0.4	Clay 4-8.5m	16	116.5
8.8	0.4	Sand 8.5-14m	17.5	154.34
9.2	0.5	Sand 8.5-14m	17.5	167.73
9.7	0.6	Sand 8.5-14m	17.5	178.57
10.3	0.6	Sand 8.5-14m	17.5	184.55
10.9	0.6	Sand 8.5-14m	17.5	184.88
11.5	0.6	Sand 8.5-14m	17.5	182.95
12.1	0.6	Sand 8.5-14m	17.5	181.04
12.7	0.6	Sand 8.5-14m	17.5	179.13
13.3	0.6	Sand 8.5-14m	17.5	177.23
13.9	0.6	Sand 8.5-14m	17.5	175.32
14.5	0.5	Clay 14-20m	19	151.57
15	0.5	Clay 14-20m	19	156.25
15.5	0.5	Clay 14-20m	19	160.78
16	0.5	Clay 14-20m	19	165.16

16.5	0.5	Clay 14-20m	19	169.39
17	0.6	Clay 14-20m	19	173.49
17.6	0.6	Clay 14-20m	19	178.2
18.2	0.6	Clay 14-20m	19	182.71
18.8	0.6	Clay 14-20m	19	187.01
19.4	0.6	Clay 14-20m	19	191.1
20	0.6	Clay 14-20m	19	194.98
20.6	0.6	Clay 20-27m	19	198.66
21.2	0.6	Clay 20-27m	19	202.12
21.8	0.7	Clay 20-27m	19	205.37
22.5	0.7	Clay 20-27m	19	208.91
23.2	0.7	Clay 20-27m	19	212.15
23.9	0.7	Clay 20-27m	19	215.12
24.6	0.7	Clay 20-27m	19	217.79
25.3	0.7	Clay 20-27m	19	220.19
26	0.7	Clay 20-27m	19	222.3
26.7	0.8	Clay 20-27m	19	224.12
27.5	0.9	Sand 27-30m	20	259.65
28.4	0.9	Sand 27-30m	20	262.05
29.3	0.7	Sand 27-30m	20	264.4
30	Half-Space	Bedrock	21	350

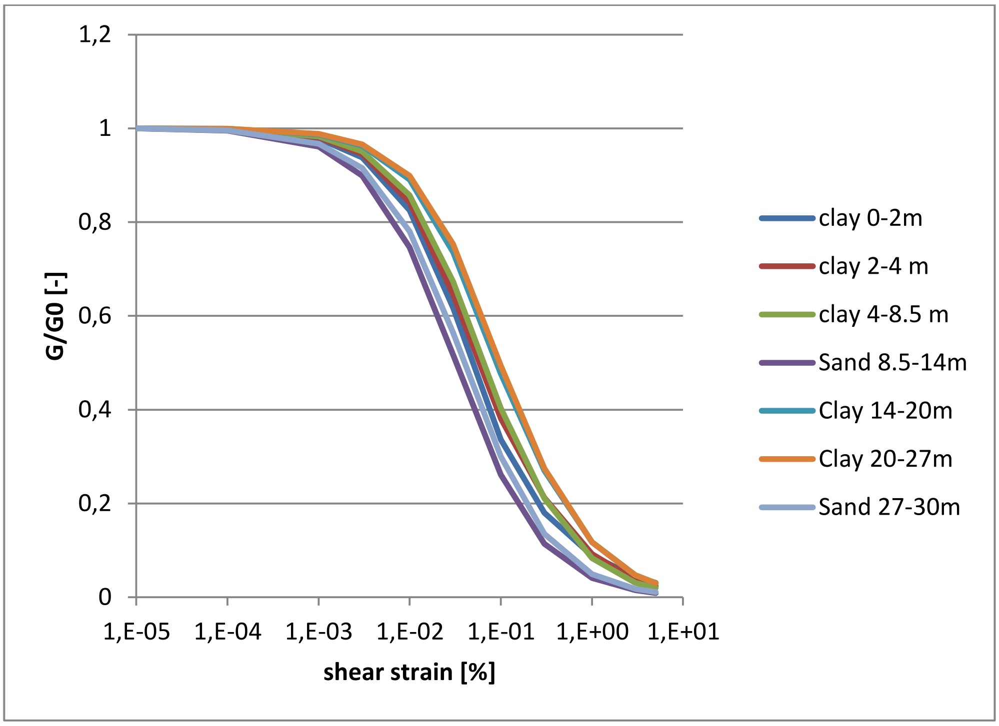

Figure A2 shear modulus degradation curve for the loppersum profile.

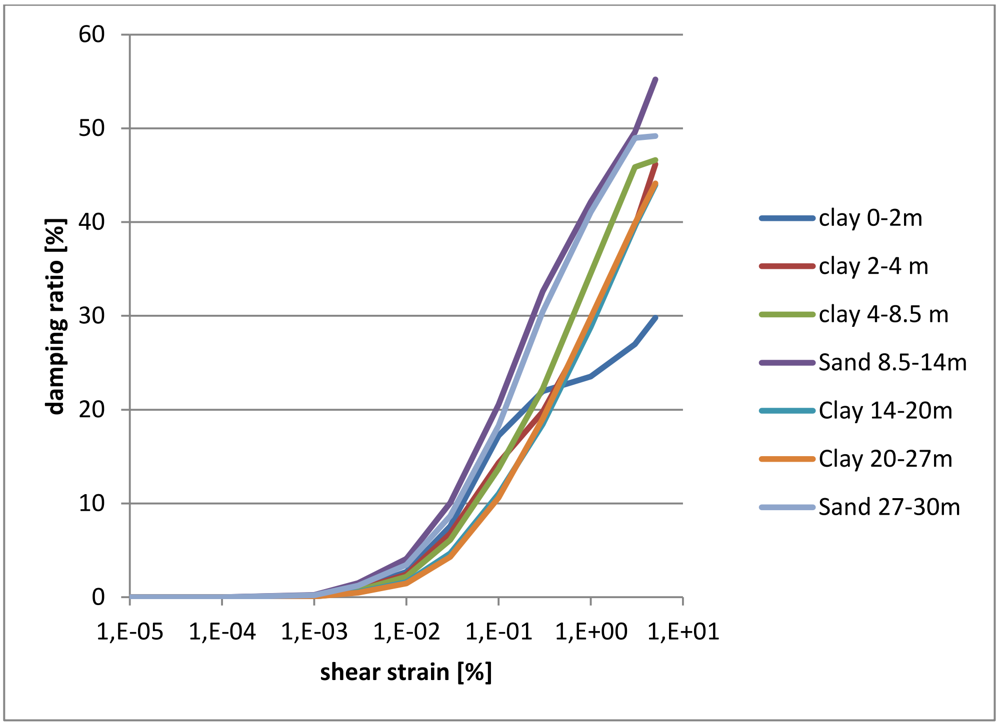

Figure A3 damping ratio curve for the loppersum profile (small strain damping not included).

B Slochteren sandy soil profile

Table B1 Slochteren shear wave velocities.

depth [m]	thickness [m]	soil type	unit weight [kN/m^3	Vs [m/s]
0	1.25	Sand SLO1	19.4	143
1.25	1.25	Sand SLO1	19.4	176
2.5	2.25	Sand SLO2	19.4	186
4.75	2.25	Sand SLO2	19.4	203
7	2.75	Sand SLO3	20.6	248
9.75	2.75	Sand SLO3	20.6	265
12.5	1.95	Sand SLO4	20	255
14.45	1.95	Sand SLO4	20	263
16.4	3.4	Sand SLO5	20	267
19.8	6.8	Sand SLO5	20	290
26.6	3.4	Sand SLO5	20	309
30	Half-Space	Bedrock	16	350

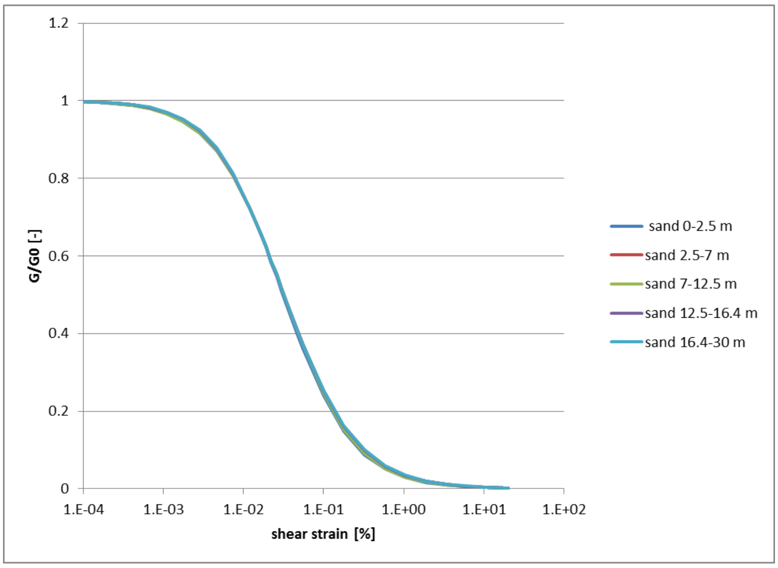

Figure B1 shear modulus degradation curve for the Slochteren profile.

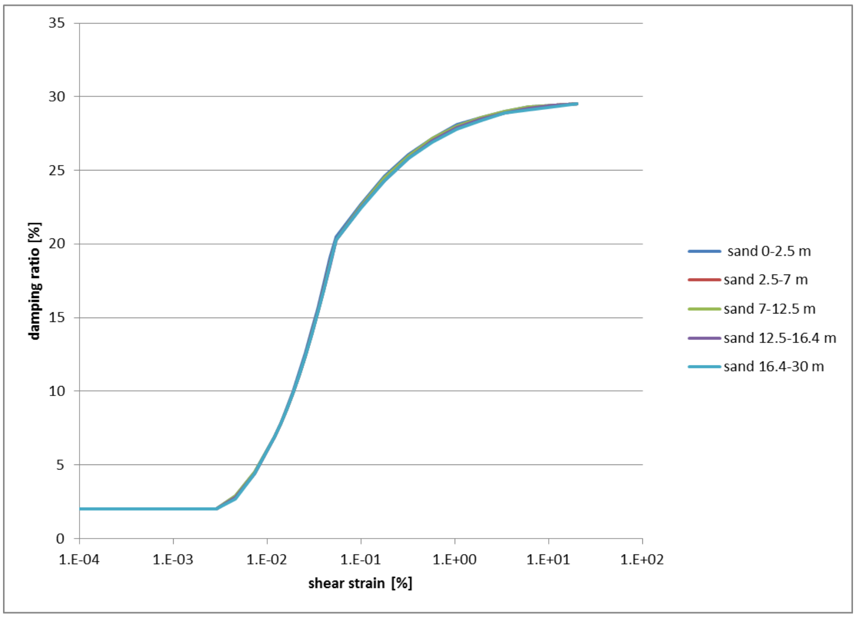

Figure B2 damping ratio curve for the Slochteren profile (small strain damping included).

The shear modulus degradation and damping curves for all the layers in the Slochteren profile are nearly the same. This is because for sands the shear strength τ_{ff} and crossover strain γ_I parameters are not used. For sands, differences in the shear modulus degradation curve are only in the reference strain γ_r, which now only depends on the mean effective stress σ' because the plasticity index $I_p = 0$.