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FREQUENCY DOMAIN ANALYSIS OF SITE AMPLIFICATION IN LOW SEISMICITY REGIONS

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ABSTRACT

The effect of site amplification can be modeled assuming linear, equivalent linear, or non-linear constitutive behavior of the soil. In low seismicity regions, the effect of the non-linear behavior of the soil may be expected to be small, which might be used as an argument to use a linear soil model. This paper presents a synthetic case study considering a site in an area where the reference peak ground acceleration is $0.1g$. The site response is analyzed using (1) a linear model, (2) an equivalent linear model, and (3) an equivalent linear model with frequency dependent properties. The different material models lead to considerably different results. It is therefore concluded that, even in low seismicity regions, linear site amplification models should be used with care.

Keywords: seismic hazard, site amplification, equivalent linear modeling, frequency domain analysis.

INTRODUCTION

Site amplification is an important issue in the assessment of the seismic hazard at sites where the top soil layers are particularly soft. In such cases, the seismic motion at the surface can be much higher than the outcrop motion due to resonance of the soft layers. This paper focuses on the modeling of site amplification at sites in low seismicity regions, such as Belgium.

Site amplification must be accounted for in the design of structures. Following Eurocode 8 (European Commitee for Standardization; 2004), the effect of site amplification can be taken into account through the use of the appropriate design response spectrum, dependent on the ground type. In two cases, however, Eurocode 8 specifies that a site response analysis must be performed: (1) for the design of important structures, and (2) for soils of type S_1. Important structures have an importance factor $\gamma_{\mathrm{I}} > 1$, these are buildings belonging to importance class III (buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.) or importance class IV (buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.). Soils of type S_1 consist of a layer with a thickness of at least $10\,\mathrm{m}$ of soft clays/silts with a high plasticity index and high water content. The average shear wave velocity $V_{\mathrm{s},30}$ in the upper $30\,\mathrm{m}$ of these soils is typically smaller than $100\,\mathrm{m/s}$.

While the soil is often modeled as a linear (visco-)elastic material, the constitutive behavior of soil is actually non-linear. In a site amplification analysis, it may be necessary to account for this non-linearity. Soil typically exhibits a softening non-linearity, or a decrease in modulus as strain increases. Increasing strains also cause progressively larger hysteresis in the stress-strain relation, leading to strain dependent wave attenuation.

Three strategies can be followed regarding the non-linear constitutive behavior of the soil: (1) the stress-strain relation is linearized, allowing for the use of a linear model, (2) the non-linearity is accounted for

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through the use of an equivalent linear model, where the layer properties are iteratively modified as a function of the effective strain level, and (3) a fully non-linear calculation is performed, using a time integration procedure.

In the equivalent linear approach, the equivalent soil properties are usually assumed to be frequency independent (this is the case in software such as SHAKE2000 (Ordóñez; 2008) and ProShake (EduPro Civil Systems; 2003)). However, this assumption often leads to an underestimation of the response in the higher frequency range. Alternative methodologies based on frequency dependent equivalent linear material models have therefore been presented by various authors, such as Sugito (1995), Kausel and Assimaki (2002), Assimaki and Kausel (2002), and Yoshida et al. (2002). The results obtained with these models agree well with the results from a fully non-linear calculation.

This paper presents a study of the usability of a linear model to perform a site response analysis in low seismicity regions, such as Belgium. The analysis is performed for a soil with a shear wave velocity profile that is typical for Belgium. The amplification of a vertically incident shear wave is considered. A seismogram compatible with the design response spectrum in Eurocode 8 is selected. The analysis is performed by means of (1) a linear, (2) an equivalent linear, and (3) a frequency dependent equivalent linear material model, using the ElastoDynamics Toolbox (EDT) for MATLAB (Schevenels et al.; 2009). The results obtained with the third method are considered as reference results. Both the acceleration at the surface and the corresponding response spectra are computed.

EXAMPLE PROBLEM

Soil profile

The site response analysis is performed for a soil profile presented in this subsection. The soil profile does not correspond to an actual site, but the values used are representative for many sites in Belgium.

A medium density sandy soil is assumed. The soil profile consists of four homogeneous layers on a homogeneous halfspace. For each layer, table 1 gives the thickness h, the small-strain shear modulus μ_0, the density ρ, the small-strain shear wave velocity $C_{s0} = \sqrt{\mu_0/\rho}$, and the small-strain material damping ratio D_{s0}. Material damping is taken into account through the use of a complex shear modulus defined as $\mu_0^* = \mu_0(1 + 2iD_{s0})$.

Table 1. Small-strain dynamic soil properties.

Layer	h [m]	μ_0 [MPa]	ρ [kg/m^3]	C_{s0} [m/s]	D_{s0} [−]
1	6	39.2	2000	140	0.005
2	9	64.8	2000	180	0.005
3	15	156.8	2000	280	0.005
4	18	320.0	2000	400	0.005
5	∞	1620.0	2000	900	0.005

The small-strain shear wave velocity C_{s0} and material damping ratio D_{s0} are visualized in figure 1. The shear wave velocity C_{s0} increases from $140\,\mathrm{m/s}$ at the surface to $900\,\mathrm{m/s}$ at a depth of $48\,\mathrm{m}$. The material damping ratio D_{s0} is constant and equals 0.005. This value is relatively low, but it is chosen in order to comply with the lower bound of the material damping curve for sandy soils used in the equivalent linear analysis (see below).

The average shear wave velocity $V_{s,30}$ in the upper $30\,\mathrm{m}$ equals $222\,\mathrm{m/s}$; the soil therefore belongs to ground type C.

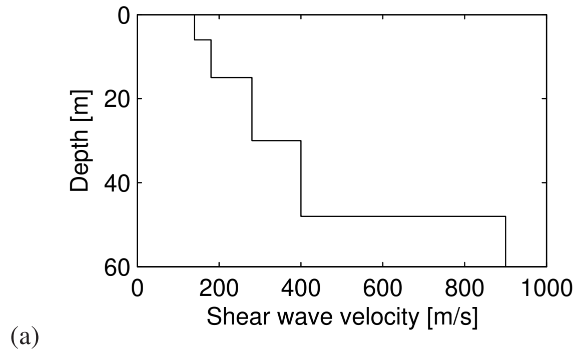


Figure 1. Small-strain (a) shear wave velocity C_{s0} and (b) material damping ratio D_{s0} versus depth.

Outcrop motion

The site is assumed to be located in a region in Belgium (figure 2) where the reference peak ground acceleration is $a_{\mathrm{gR}} = 0.1g = 0.981\,\mathrm{m/s^2}$. A building belonging to importance class III is considered; the importance factor is therefore $\gamma_{\mathrm{I}} = 1.2$. The corresponding design ground acceleration is given by $a_{\mathrm{g}} = \gamma_{\mathrm{I}} a_{\mathrm{gR}} = 1.18\,\mathrm{m/s^2}$.

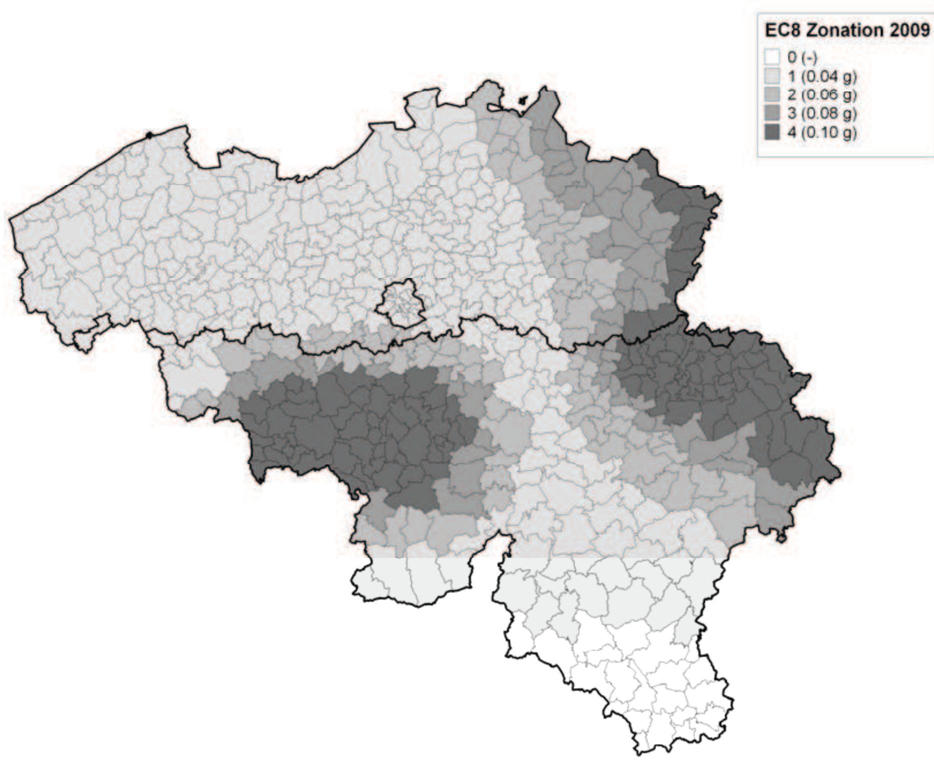


Figure 2. Seismic hazard map of Belgium according to the National Application Document for Eurocode 8 (Belgisch Instituut voor Normalisatie; 2009).

The outcrop motion represents the seismic hazard at bedrock level, which is characterized by the elastic response spectrum $S_{\mathrm{e}}^{\mathrm{A}}(T)$ from Eurocode 8 for ground class A. The elastic response spectrum $S_{\mathrm{e}}^{\mathrm{A}}(T)$ is shown in figure 3. Following the Belgian National Application Document (Belgisch Instituut voor Normalisatie; 2009), the elastic response spectrum of type 2 is used. The design ground acceleration is $a_{\mathrm{g}} = 1.18\,\mathrm{m/s^2}$ and the viscous damping ratio of the structure is assumed to be $5\,\%$.

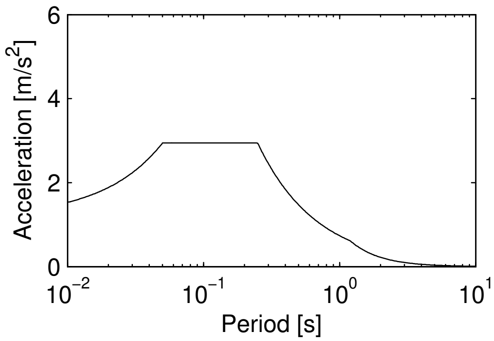


Figure 3. The type 2 elastic response spectrum $S_\mathrm{e}(T)$ from Eurocode 8 for ground type A, a reference peak ground acceleration $a_\mathrm{gR} = 0.1g$, and an importance factor $\gamma_\mathrm{I} = 1.2$.

Performing a site response analysis requires the selection of one or more strong motion records representing the outcrop motion. In this case, a single strong motion record is considered. An attempt is made to select a strong motion record for which the pseudo-acceleration response spectrum is similar to the elastic response spectrum $S_\mathrm{e}^\mathrm{A}(T)$ from Eurocode 8. A strong motion record from the Kocaeli (Turkey) earthquake hase thus been chosen, although Kocaeli is not located in Belgium.

The strong motion record is scaled in order to comply with the design ground acceleration a_g; the scaled strong motion record $a_\mathrm{o}(t)$ is shown in figure 4a. The corresponding pseudo-acceleration response spectrum $S_{a_\mathrm{o}}(T)$ is computed, assuming a viscous damping ratio of $5\,\%$ for the structure. Figure 4b shows both the response spectrum $S_{a_\mathrm{o}}(T)$ corresponding to the strong motion record $a_\mathrm{o}(t)$ and the elastic response spectrum $S_\mathrm{e}^\mathrm{A}(T)$ from Eurocode 8. This figure demonstrates that an appropriate strong motion record $a_\mathrm{o}(t)$ has been selected, as the corresponding response spectrum $S_{a_\mathrm{o}}(T)$ is relatively close to the elastic response spectrum $S_\mathrm{e}^\mathrm{A}(T)$ without (significantly) exceeding it.

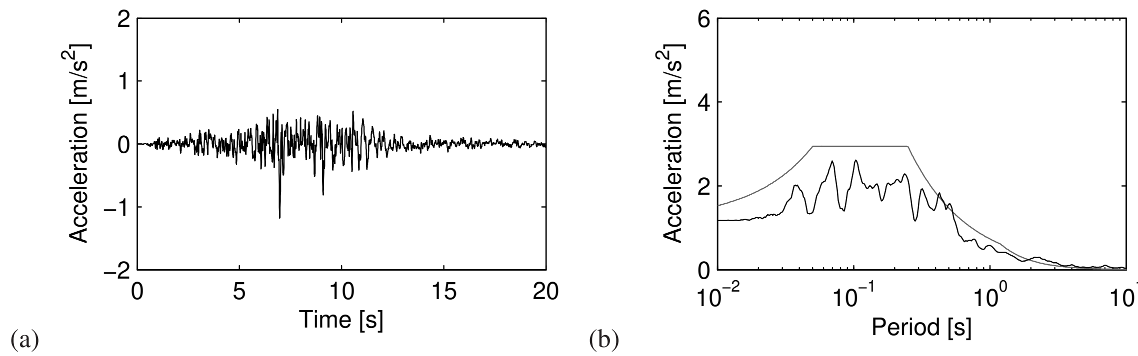


Figure 4. (a) Outcrop motion $a_\mathrm{o}(t)$ and (b) corresponding response spectrum $S_{a_\mathrm{o}}(T)$ (black line) compared with the elastic response spectrum $S_\mathrm{e}^\mathrm{A}(T)$ from Eurocode 8 for ground type A (gray line).

LINEAR ANALYSIS

In this section, a linear site response analysis is performed, using the soil profile and the outcrop motion defined in the previous section. The computations are performed by means of EDT, which is based on the direct stiffness method (Kausel and Roësset; 1981).

Figure 5 shows the time history $a(t)$ of the acceleration at the soil's surface and the corresponding response spectrum $S_a(T)$. Due to the presence of the soft layers, the acceleration $a(t)$ is considerably larger than the outcrop motion $a_{\mathrm{o}}(t)$ shown in figure 4a. Likewise, the response spectrum $S_a(T)$ exceeds the outcrop response spectrum $S_{a_{\mathrm{o}}}(T)$ shown in figure 4b, which means that the presence of the soft layers causes an increase of the response of structures due to an earthquake. For structures with a fundamental eigenperiod higher than $2\,\mathrm{s}$, the influence of the soft layers is less pronounced. The response of these structures mainly depends on the low frequency waves in the soil. The wavelength of the low frequency waves is much larger than the thickness of the soft layers, which explains the limited influence of the soft layers on these waves.

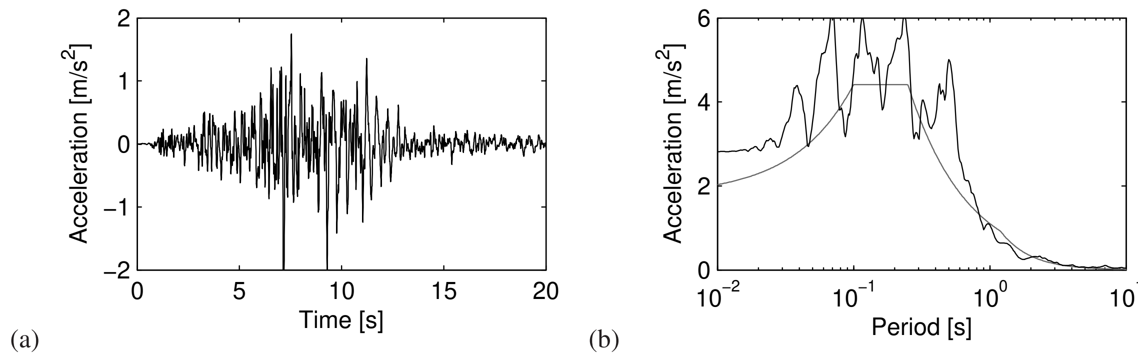


Figure 5. (a) Acceleration $a(t)$ at the surface obtained with a linear soil model and (b) corresponding response spectrum $S_a(T)$ (black line) compared with the elastic response spectrum $S_{\mathrm{e}}^{\mathrm{C}}(T)$ from Eurocode 8 for ground type C (gray line).

Although not allowed by Eurocode 8 (since we are considering a building belonging to importance class III), one could decide not to perform a site response analysis and simply use the elastic response spectrum corresponding to the appropriate ground type instead. The soil profile considered in this example belongs to ground type C. The type 2 elastic response spectrum $S_{\mathrm{e}}^{\mathrm{C}}(T)$ from Eurocode 8 for ground class C is therefore also computed, assuming a design ground acceleration $a_{\mathrm{g}} = 1.18\,\mathrm{m/s}^2$ and a structural viscous damping ratio of $5\,\%$. This spectrum $S_{\mathrm{e}}^{\mathrm{C}}(T)$ is compared with the response spectrum $S_a(T)$ obtained from the site response analysis in figure 5b. The response spectrum $S_a(T)$ obtained from the site response analysis clearly exceeds the response spectrum $S_{\mathrm{e}}^{\mathrm{C}}(T)$ for ground class C, suggesting that the use of the latter for the seismic design of a structure is not conservative. However, it will be shown in the following sections that the difference between the computed response spectrum $S_a(T)$ and the elastic response spectrum $S_{\mathrm{e}}^{\mathrm{C}}(T)$ is due to the use of a linear material model.

EQUIVALENT LINEAR ANALYSIS

Under strong earthquake motions, the constitutive behavior of soil can no longer be considered as linear. Soil typically exhibits a softening non-linearity, or a decrease in modulus as strain increases. Increasing strains also cause progressively larger hysteresis in the stress-strain relation, leading to strain dependent wave attenuation. However, the non-linear response of a layered soil due to an earthquake can still be modeled (in an approximate way) by means of a linear soil model, provided that the shear modulus and the material damping ratio of each layer are adjusted as a function of the strain level in the layer.

Various authors have investigated the variation of the shear modulus and the material damping ratio of soil with the strain level under cyclic loading. Seed et al. (1986) have presented the modulus reduction and material damping curves for sandy soils shown in figure 6. The modulus reduction curve, shown in figure 6a, represents the ratio μ/μ_0 of the (equivalent) shear modulus μ and the small-strain shear modulus μ_0 as a function of the shear strain γ. The material damping curve, shown in figure 6b, represents the material damping ratio D_{s} as a function of the shear strain γ.

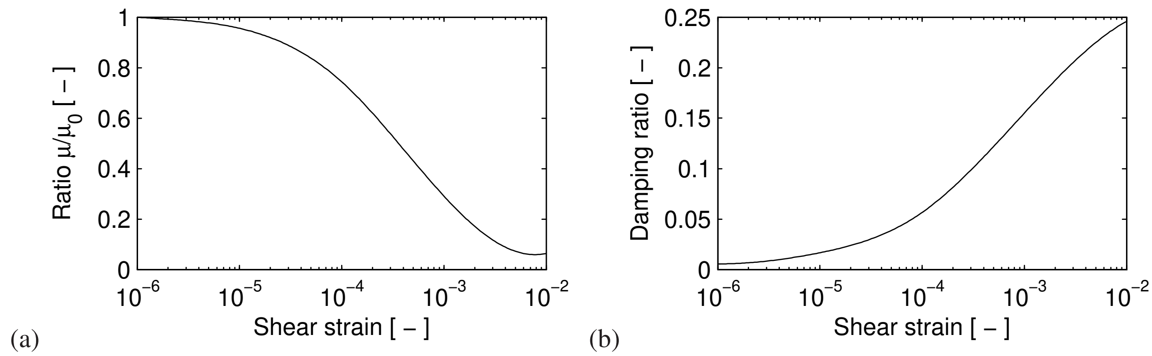


Figure 6. (a) Modulus reduction and (b) material damping curves for sandy soils.

The equivalent linear analysis proceeds as follows. The soil profile is subdivided in a sufficient number of sublayers to characterize properly the spatial variation of inelastic effects. In this case, 16 layers with a thickness of $3\,\mathrm{m}$ are used. A linear site response analysis is subsequently performed, using the small-strain shear modulus μ_0 and material damping ratio $D_{\mathrm{s}0}$ of the soil. The time history $\gamma(t)$ of the shear strain is computed in the center of each layer and used to obtain the peak shear strain $\gamma_{\max}$:

$$\gamma_{\max} = \max(\gamma(t)) \tag{1}$$

The peak shear strain $\gamma_{\max}$ can not be used to evaluate the modulus reduction and material damping curves; these curves are valid for soil under cyclic loading, whereas seismic loading is not cyclic. An effective shear strain level γ_{eff} is therefore determined for each layer. Following common practice in the equivalent linear modeling of site amplification (Ordóñez; 2008), the effective shear shear strain level γ_{eff} is defined as:

$$\gamma_{\mathrm{eff}} = 0.65\gamma_{\max} \tag{2}$$

The modulus reduction and material damping curves are evaluated at the effective shear strain level γ_{eff} to determine the equivalent shear modulus μ and material damping ratio D_{s} for each layer. The soil profile is modified accordingly, and a new linear site response analysis is performed. This procedure is repeated until convergence is reached.

The resulting equivalent shear wave velocity $C_{\mathrm{s}} = \sqrt{\mu/\rho}$ and material damping ratio D_{s} are shown in figure 7. A softening non-linearity is observed: the equivalent shear modulus μ has decreased compared to the small-strain value, while the equivalent material damping ratio D_{s} has increased considerably. The non-linearity is more pronounced in the softest layers, where the effective shear strain level γ_{eff} is large.

The time history $a(t)$ of the acceleration at the soil's surface and the corresponding response spectrum $S_a(T)$ are shown in figure 8. Compared to the results of the linear analysis (figure 5), the increase of the structural response is smaller. Especially for structures with a fundamental eigenperiod lower than $0.2\mathrm{s}$, the response appears to be limited. The response of these structures depends to a large extent on the high frequency waves in the soil, with a short wavelength. These waves are strongly attenuated in the equivalent linear model due to the use of a high equivalent material damping ratio. This effect is not realistic; the attenuation of high frequency waves is much weaker in reality (Kausel and Assimaki; 2002). An alternative equivalent linear approach with frequency dependent soil properties has therefore been developed by Kausel and Assimaki (2002). The frequency dependent approach is addressed in the next section.

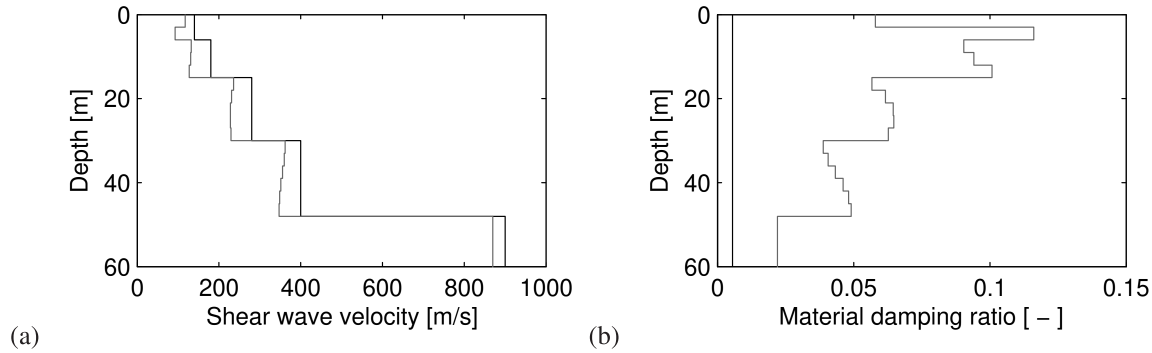


Figure 7. Equivalent (a) shear wave velocity C_s and (b) material damping ratio D_s (gray lines) compared with the small-strain soil properties (black lines).

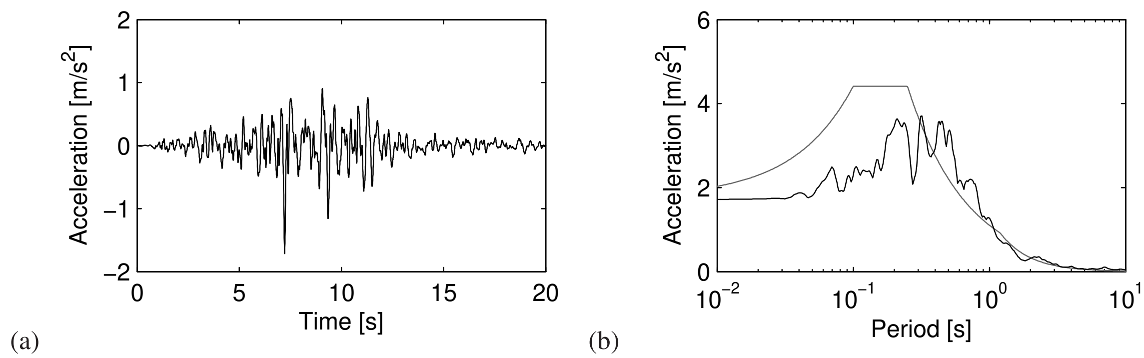


Figure 8. (a) Acceleration $a(t)$ at the surface obtained with an equivalent linear soil model and (b) corresponding response spectrum $S_a(T)$ (black line) compared with the elastic response spectrum $S_\mathrm{e}^\mathrm{C}(T)$ from Eurocode 8 for ground type C (gray line).

FREQUENCY DEPENDENT EQUIVALENT LINEAR ANALYSIS

This section focuses on the frequency dependent equivalent linear method developed by Kausel and Assimaki (2002). In this method, the frequency spectrum of the shear strain in each layer is considered to determine the equivalent dynamic soil properties. This implies that the modulus reduction and material damping curves are evaluated at a different strain level for each frequency, resulting in frequency dependent equivalent dynamic soil properties. Kausel and Assimaki (2002) demonstrate that this approach leads to more realistic results by means of a comparison with a fully non-linear calculation.

The modelling of site amplification with a frequency dependent equivalent linear material model proceeds along the same lines as in the frequency independent approach, but the modulus reduction and material damping curves are now evaluated for each frequency independently, using a scaled and smoothed shear strain spectrum $\hat{\gamma}_\mathrm{sm}(\omega)$ instead of an effective shear strain level γ_eff. The scaling operation is performed in order to ensure that the peak spectral amplitude equals the peak strain, so that the method remains valid in the case of cyclic loading. The use of a smoothed spectrum leads to a more robust and stable iterative algorithm Kausel and Assimaki (2002).

For each layer, the scaled and smoothed shear strain spectrum $\hat{\gamma}_\mathrm{sm}(\omega)$ is determined from the shear strain spectrum $\hat{\gamma}(\omega)$ as follows. First, the scaled shear strain spectrum $\hat{\gamma}_\mathrm{sc}(\omega)$ is computed:

$$\hat{\gamma}_\mathrm{sc}(\omega) = \frac{\gamma_\mathrm{max}}{\gamma_0}\hat{\gamma}(\omega) \tag{3}$$

where $\gamma_{\max}$ is the peak shear strain and γ_0 is the average value of the shear strain spectrum between zero and the mean frequency ω_0:

$$\gamma_0 = \frac{1}{\omega_0} \int_0^{\omega_0} |\hat{\gamma}(\omega)| \, d\omega \tag{4}$$

The mean frequency ω_0 is given by:

$$\omega_0 = \frac{\int_0^\infty \omega |\hat{\gamma}(\omega)| \, d\omega}{\int_0^\infty |\hat{\gamma}(\omega)| \, d\omega} \tag{5}$$

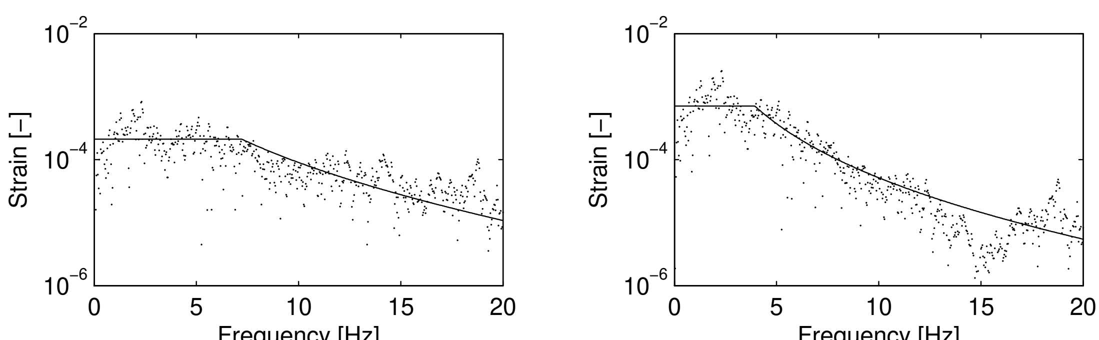
The smoothed shear strain spectrum $\hat{\gamma}_{\mathrm{sm}}(\omega)$ is subsequently obtained as:

$$\hat{\gamma}_{\mathrm{sm}}(\omega) = \begin{cases} \gamma_{\max} & \text{for } \omega \leq \omega_0 \\ \gamma_{\max} \dfrac{\exp\left(-\alpha\left(\frac{\omega}{\omega_0} - 1\right)\right)}{\left(\frac{\omega}{\omega_0}\right)^\beta} & \text{for } \omega > \omega_0 \end{cases} \tag{6}$$

The exponents α and β are determined by solving a linear least squares problem in order to minimize the misfit between the logarithm of the smoothed shear strain spectrum $\hat{\gamma}_{\mathrm{sm}}(\omega)$ and the scaled shear strain spectrum $\hat{\gamma}_{\mathrm{sc}}(\omega)$. The smoothed spectrum $\hat{\gamma}_{\mathrm{sm}}(\omega)$ is subsequently used to update the frequency dependent equivalent soil properties according to the modulus reduction and material damping curves. This procedure is repeated until convergence is reached.

It should be noted that equation (6) differs from the equation proposed in the original paper (Kausel and Assimaki; 2002); a term -1 is added here. This term ensures that the smooth shear strain spectrum is continuous at the mean frequency ω_0.

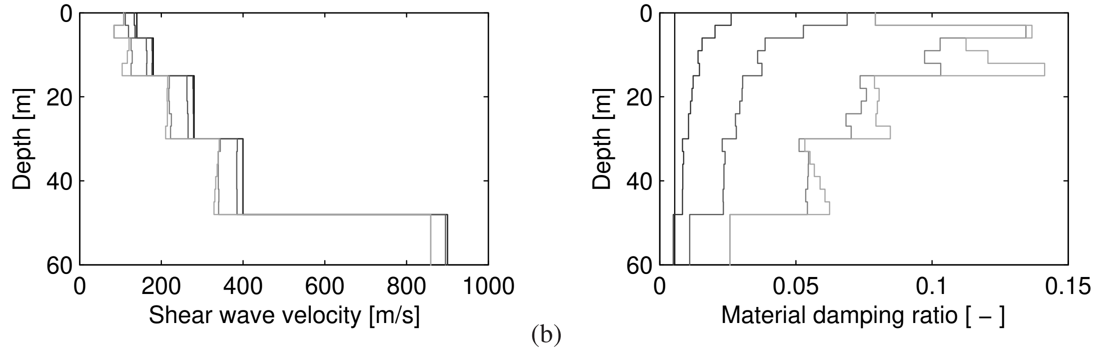
Figure 9 shows both the scaled shear strain spectrum $\hat{\gamma}_{\mathrm{sc}}(\omega)$ and the smoothed shear strain spectrum $\hat{\gamma}_{\mathrm{sm}}(\omega)$ for the top layer (i.e. at a depth of $1.5\,\mathrm{m}$) and the second layer (i.e. at a depth of $4.5\,\mathrm{m}$). It can be observed that the shear strain decreases with the frequency, explaining the overestimation of the high frequency wave attenuation in the frequency independent approach. The correspondence between the unsmoothed and the smoothed shear strain spectrum is acceptable. This confirms the statement by Kausel and Assimaki (2002) that the model proposed for the smoothed shear strain spectrum (equation (6)) is suitable for most earthquakes, even after the addition of the term -1 that ensures the continuity of the spectrum.



(a) (b)

Figure 9. Unsmoothed (dots) and smoothed (solid line) spectrum of the shear strain in the center of (a) the top layer and (b) the second layer.

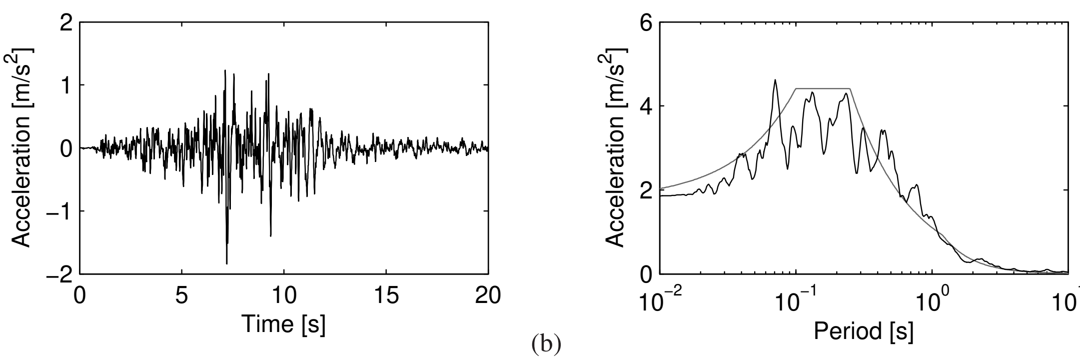
Figure 10 shows the equivalent shear modulus C_{s} and the equivalent material damping ratio D_{s}. Both properties are frequency dependent and are shown for four different frequencies. For comparison, the small-strain values are also shown. For low frequencies, the soil properties used in the frequency dependent approach are close to the values used in the frequency independent approach (figure 7). For high frequencies, the difference is larger, and a tendency towards the small-strain values is observed.



(a) (b)

Figure 10. Equivalent (a) shear modulus C_s and (b) material damping ratio D_s at 2 Hz (light gray lines), 4 Hz, 8 Hz, and 16 Hz (dark gray lines), compared with the small-strain soil properties (black lines).

Figure 11 shows the time history of the acceleration $a(t)$ at the soil's surface and the corresponding response spectrum $S_a(T)$. Compared to the results of the linear analysis (figure 5), a reduction of the structural response is observed, but not to the same extent as in the frequency independent approach (figure 8). This indicates that the overestimation of high frequency wave attenuation in an equivalent linear analysis can be avoided through the use of frequency dependent material properties.



(a) (b)

Figure 11. (a) Acceleration $a(t)$ at the surface obtained with a frequency dependent equivalent linear soil model and (b) corresponding response spectrum $S_a(T)$ (black line) compared with the elastic response spectrum $S_\mathrm{e}^\mathrm{C}(T)$ from Eurocode 8 for ground type C (gray line).

Figure 11b also shows the elastic response spectrum $S_\mathrm{e}^\mathrm{C}(T)$ from Eurocode 8 for ground type C. The response spectrum $S_a(T)$ obtained from the site response analysis does not deviate much from the elastic response spectrum $S_\mathrm{e}^\mathrm{C}(T)$. This is an indication that an equivalent linear site response analysis with frequency dependent material properties yields realistic results. It is not a strong proof that the results are correct, as the input spectrum is not completely identical to the elastic response spectrum for ground type A, and the elastic response spectrum for ground type C only accounts for site amplification in an implicit way, as it results from the averaging and smoothing of actual strong motion records obtained on type C sites Rey et al. (2002).

It is tempting to conclude that performing a site response analysis is superfluous in this case as simply using the elastic response spectrum for the appropriate ground type defined in Eurocode 8 yields almost the same results. However, it is unlikely that this conclusion can be generalized for all soil profiles and strong motion records.

COMPARISON

In order to facilitate the assessment of the three material models, the three response spectra $S_a(T)$ obtained are shown on the same graph in figure 12, together with the elastic response spectrum $S_e^C(T)$ from Eurocode 8 for ground type C. The response spectrum $S_a(T)$ obtained with the frequency dependent equivalent linear method is close to the elastic response spectrum $S_e^C(T)$ from Eurocode 8 for ground type C. The spectra $S_a(T)$ obtained with the two other methods deviate much more from the spectrum $S_e^C(T)$ from Eurocode 8.

For structures with a fundamental eigenperiod T higher than $2\,$s, the effect of site amplification is very small, and the response spectra $S_a(T)$ obtained with the three methods are almost identical. In the range of eigenperiods T lower than $2\,$s, the effect of site amplification is more important, and the three methods lead to considerably different results. Using the response spectrum $S_a(T)$ obtained with the frequency dependent equivalent linear method and the elastic response spectrum $S_e^C(T)$ for ground type C as a reference, the linear approach appears to be conservative, as it leads to a (large) overestimation of the response spectrum. The use of a frequency independent equivalent linear model is not safe, as it leads to a considerable underestimation of the response spectrum, especially for structures with a fundamental eigenperiod lower than $0.2\,$s.

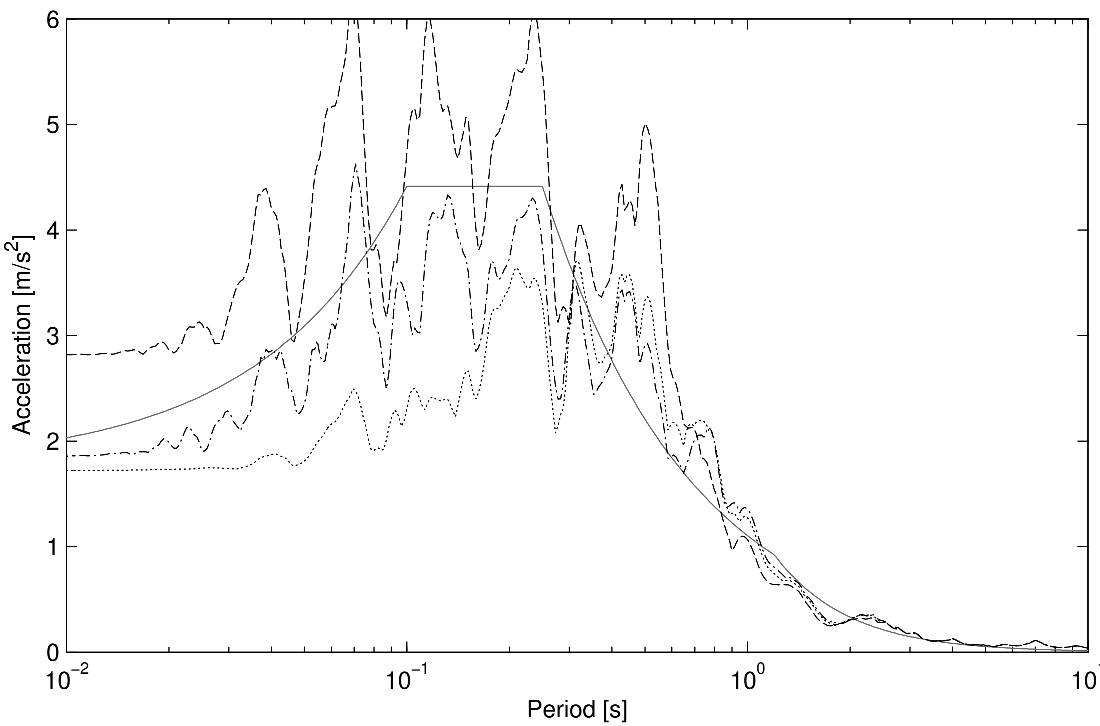


Figure 12. Response spectrum $S_a(T)$ for the acceleration at the surface obtained with a linear soil model (dashed line), an equivalent linear soil model (dotted line), and a frequency dependent equivalent linear soil model (dash-dotted line), compared with the elastic response spectrum $S_e^C(T)$ from Eurocode 8 for ground type C (gray line).

It can be concluded that, even in low seismicity regions such as Belgium, the non-linear behavior of the soil should be accounted for in a site response analysis, and frequency dependent soil properties should be used. In the present analysis, the use of a linear model or an equivalent linear model with frequency independent material properties yields worse results (in the sense that they deviate more from the reference results obtained with the frequency dependent method) than simply using the elastic response spectrum for the appropriate ground type from Eurocode 8.

CONCLUSION

The present paper focuses on the usability of a linear model to assess site amplification in low seismicity regions, such as Belgium. A site response analysis is performed for a soil with a shear wave velocity profile that is typical for Belgium. A strong motion record compatible with the design response spectrum in Eurocode 8 is considered. The analysis is performed by means of (1) a linear, (2) an equivalent linear, and (3) a frequency dependent equivalent linear material model. Both the acceleration at the surface and the corresponding response spectra are computed.

The different material models lead to considerably different results. It is therefore concluded that, even in low seismicity regions, it is advisable to assess site amplification by means of an equivalent linear material model. For structures with low eigenperiods, an equivalent linear soil model with frequency dependent properties should be used in order to avoid overestimation of high frequency wave attenuation.

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