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KINEMATIC BENDING OF PILES: ANALYSIS VS. CODE PROVISIONS

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ABSTRACT

Recent seismic codes prescribe that piles have to be designed for "*soil deformations arising from the passage of seismic waves which impose curvatures and thereby lateral strains on the piles along their whole length"*(EC-8). Accepting these lines, recent seismic Italian normatives (OPCM 3274/2003 and more recent ones) specify that kinematic effects should be taken into account when seismicity of the area is moderate or high, subsoil type is C or worse, and consecutive soil layers with sharply different shear moduli are present. The final aim of the study is to evaluate, from a practical viewpoint, to what extent kinematic pile bending is worth considering in typical subsoil conditions, namely soil types C and D according to EC8. The analysis method is that proposed by Mylonakis et al. (1997) based on a numerical Beam-on-Dynamic-Winkler-Foundation (BDWF) model. In this paper analysis results for two-layered soil profiles are illustrated. Italian acceleration recordings are used as input motions. Results indicate that in the high seismicity zones of the Italian zonation (OPCM 3274/2003 and OPCM 3519/2006) and for subsoil types D and C, kinematic bending moments may be very high and well above the yielding level of typical RC piles, the latter being dependent on the magnitude of the normal load acting in the pile.

Keywords: piles, earthquakes, kinematic interaction, seismic code

INTRODUCTION

Despite extensive research on seismic response of piles and some case histories confirming the significant role of kinematic pile bending, this effect is rarely taken into account in design. Instead, piles are essentially dimensioned to withstand only loads imposed at the pile head, generated from the weight and inertia of the superstructure. Increasing field evidence from earthquakes all over the world has demonstrated the role of kinematic interaction in the development of pile damage. This has been confirmed in several recent events, including Mexico City (1985), Kobe (1995) and Chi Chi (1999).
Modern seismic codes have acknowledged this aspect and demand that piles must be designed for soil deformations arising from the passage of seismic waves, which impose curvatures and thereby lateral strain on the piles, even in depth. For instance, EC8 suggests that kinematic effects should be taken into account when all the following conditions simultaneously exist: (1) seismicity of the area is moderate (specifying that moderate or high seismicity areas are characterized by a peak ground acceleration $a_g S$>0.1g, where a_g is the design ground acceleration on type A ground and S is the soil factor); (2) subsoil type is D or worse, characterized by sharply different shear moduli between consecutive layers, (3) the superstructure is of importance III or IV (e.g., schools, hospitals, fire stations, power plants, etc).

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Recent seismic Italian normatives (OPCM 3274/2003 and subsequent revisions) acknowledge these demands, with the exception of considering ground profiles corresponding to the subsoil type C and disregarding the importance class of the superstructure.
The topic of the paper is to evaluate, from a practical viewpoint, to what extent kinematic interaction of piles is worth considering in typical subsoil conditions, namely those corresponding to types C and D of EC8. To this end, a two-layer soil stratigraphy, with a single floating fixed-head pile penetrating both layers has been adopted. The shear wave velocity ratio between the first and the second layer varies from 2 to 4, corresponding to a shear modulus contrast varying between 4 and 16.
Kinematic pile interaction has been studied by means of the analysis method proposed by Mylonakis et al. (1997) based on a Beam-on-Dynamic-Winkler-Foundation (BDWF) model. In this approach, the pile is connected to the soil through continuously-distributed springs and dashpots (Flores-Berrones & Whitman, 1982; Kavvadas & Gazetas, 1993).
Italian accelerograms have been used as input motion in the analyses. For each seismic zone, only accelerograms with the original peak ground acceleration close to the reference maximum peak acceleration on soil type A, a_{gR}, have been selected. In this study the values of a_{gR} have been obtained from the Italian seismic zonation contained in the OPCM 3274/2003, which defines four seismic zones as shown in Table 1. At present, this zonation has been further refined by introducing seismic subzones characterized by a_{gR} in-between the range limits provided in Table 1 and with acceleration increments not less than 0.025g (OPCM 3519/2006).

Table 1. Peak ground accelerations on ground type A for the four Italian seismic zones

Zone	a_{gR} with probability of exceedance 10% in 50 years	maximum a_{gR}
1	$0.25g<a_{gR}<0.35g$	0.35 g
2	$0.15g<a_{gR}<0.25g$	0.25 g
3	$0.05g<a_{gR}<0.15g$	0.15 g
4	$a_{gR}\leq 0.05g$	0.05 g

REVIEW OF THE ANALYSIS METHODS FOR KINEMATIC PILE BENDING

Seismically-induced kinematic interaction of piles has been studied by various researchers including Margason (1975), Kagawa & Kraft (1980), Flores-Berrones & Whitman (1982), Sheppard (1983), Dobry & O'Rourke (1983), Tazoh et al. (1987), Mineiro (1990), Kavvadas & Gazetas (1993), Kaynia (1996), Poulos & Tabesh (1996), Mylonakis (2001), Saitoh (2005) and others. Reviews of the subject have been presented by Novak (1991), Pender (1993) and Gazetas & Mylonakis (1998). Closed-form expressions have been derived for computing the maximum steady-state bending moment at the interface between two consecutive soil layers (Margason, 1975; Dobry & O'Rourke, 1983; Nikolaou et al. 2001, Mylonakis 2001).

Available analytical approaches can be roughly classified in the following groups:

A. <u>Simplified methods</u> which assume that pile deformation follows the free-field soil response. This approach is suggested by Margason (1975) and was adopted in NEHRP-97. These methods are inapplicable to soil layer interfaces, since soil curvature becomes infinite at such locations. Their use is, therefore, limited to homogeneous subsoils, although experience has shown that kinematic pile bending is of minor importance in homogeneous soil (except perhaps at the top of fixed-head piles);

B. <u>Winkler models</u> in which the pile is connected to the soil through continuously-distributed springs and dashpots (Novak 1974; Flores-Berrones & Whitman, 1982; Kavvadas & Gazetas, 1993). In these methods, the springs represent soil stiffness while the dashpots damping due to radiation and hysteretic energy dissipation in the medium. The wave-induced motion in the soil represents the input excitation of the pile-soil system.

A simple method for determining kinematic bending moments at the interface separating two soil layers of different stiffness has been proposed by Dobry & O'Rourke (1983). It is based on two main assumptions: (i) both soil layers are thick so that boundary effects outside the two layers (e.g., pile head and tip) do not influence the response at the interface; (ii) each layer is subjected to a uniform stress field, τ, which generates constant shear strain ($\gamma = \tau/G$) in the soil. Based on these assumptions and modelling the pile as a beam on Winkler foundation, Dobry & O'Rourke (1983) derived an explicit solution for pile bending at the interface. This solution is written as:

$$M = 1.86(E_p I_p)^{3/4}(G_1)^{1/4} \gamma_1 F \tag{1}$$

$$F = \frac{(1-c^{-4})(1+c^3)}{(1+c)(c^{-1}+1+c+c^2)} \tag{2}$$

where E_p and Ip are the pile stiffness and cross section inertia respectively; G_1 the shear modulus of the first layer; γ_1 denotes the maximum soil shear strain at the interface; $c = (G_2 / G_1)^{1/4}$.
An alternative formulation has been proposed by Nikolaou et al. (2001). It refers to harmonic steady-state bending moments developing in a cylindrical pile at the interface between two layers of different stiffness, under resonant conditions. The formula of Nikolaou et al. (2001) has been derived by fitting results obtained by a numerical BDWF formulation, and is based on a characteristic shear stress τ_c that is likely to develop at the interface, as function of the free-field surface acceleration:

$$\tau_c \approx a_s \rho_1 h_1 \tag{3}$$

$$M_{\max}(\omega) = 0.042\tau_c d^3 (L/d)^{0.3}(E_p/E_1)^{0.65}(V_2/V_1)^{0.5} \tag{4}$$

where a_s represents the surface peak acceleration; ρ_1 and h_1 the density and thickness of the first layer respectively; d the pile diameter; L is the pile length; E_p the pile stiffness; E_1 is the Young modulus of the first layer; V_1 and V_2 the shear wave velocity of the first and second layer respectively. The above equation indicates that bending moments tend to increase with increasing pile diameter and length, pile soil stiffness contrast, and layer stiffness contrast. Under transient seismic excitation, Nikolaou et al. (2001) showed that the above trends are still valid, but with one exception: the peak values of the transient bending moments are smaller than the steady-state amplitudes by a factor of about 3 to 5 depending on the development of resonance or not in the profile.

Mylonakis (2001) developed a model analogous to the Dobry & O'Rourke (1983), which, however, fully incorporates the geometric characteristics (thickness) of the soil layers and the dynamic nature of excitation. Pile bending is quantified by dimensionless pile bending strain. The maximum pile bending strain at the outer fiber of the pile cross section is estimated as:

$$\varepsilon_p = \frac{Mr}{E_p I_p} \tag{5}$$

where r denotes the distance from neutral axis to the farthest fiber in the cross section. Bending strain has certain advantages over bending moment: (i) it is dimensionless; (ii) it is directly measurable experimentally; (iii) it can be used to quantify damage. Mylonakis (2001) derived the following equation for estimating ε_p/γ_1 under low-frequency seismic excitation:

$$\frac{\varepsilon_p}{\gamma_1} = \frac{1}{2c^4}\left(c^2 - c + 1\right)\left(\frac{h_1}{d}\right)^{-1}\left\{\left[3\left(\frac{k_1}{E_p}\right)^{1/4}\left(\frac{h_1}{d}\right) - 1\right]c(c-1) - 1\right\} \tag{6}$$

The ratio, ε_p/γ_1, is regarded by Mylonakis (2001) as a "strain transmissibility" function depending on the layer stiffness contrast G_2/G_1, pile-soil relative stiffness E_p/E_1, and embedment ratio h_1/d (Figure 1). The effect of frequency on strain transmissibility can be incorporated through an amplification factor Φ that multiplies the strain transmissibility ε_p/γ_1. Parameter analyses showed that Φ tends to increase with frequency and may exceed the value of 2, especially for stiff piles and deep interfaces. More discussion can be found in the above publication.

C. Numerical methods based on a continuum approach where the soil, pile and superstructure are modelled as a whole. The soil-pile geometry typically is modelled in 3-D and discretized by F.E.M. or B.E.M. techniques (Kaynia 1996, Kimura e Zhang, 2000; Zhang et al. 2000; Zhang e Kimura 2002; Wu e Finn, 1997; Bentley & El Naggar, 2000; Finn e Fujita, 2002). Soil is usually modelled by elastoplastic models. A few works in literature adopt models of the so-called Advanced Plasticity to properly reproduce soil response under cyclic loading conditions, and some work describes the possibility to simulate gap at the pile-soil interface under strong motion events (Maheshwari et al., 2004). Worth mentioning is the simplified continuum approach developed by Wu & Finn (1997) in which vertically propagating shear waves are modelled disregarding seismic induced deformations in the vertical direction and normal to the direction of shaking (quasi 3-D approach). In such a way bending of the pile occurs only in the direction of shaking. Soil non linearity is modelled by an equivalent linear approach.

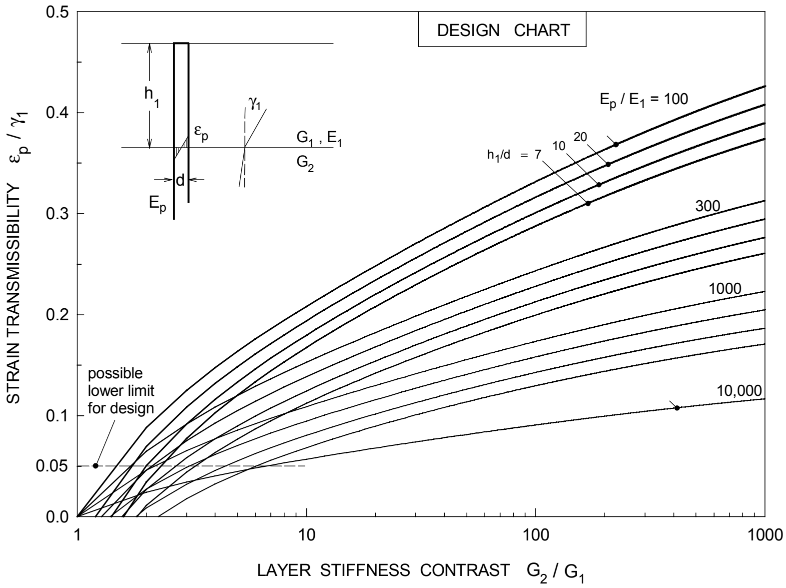

Figure 1. Strain transmissibility ε_p/γ_1, at a soil layer interface as function of layer stiffness contrast G_2/G_1, pile-soil relative stiffness E_p/E_1, and embedment ratio h_1/d (Eqn. 6 - after Mylonakis, 2001).

PARAMETRIC STUDY

With reference to soil profiles belonging to types C and D of EC8, a comprehensive parameter study has been performed considering a single floating and fixed-head pile in a two-layer soil deposit subjected to vertically propagating seismic waves.

The analysis method is that proposed by Mylonakis et al. (1997) using a BDWF approach. As the analytical aspects of the method are well known (Kavvadas & Gazetas 1993, Mylonakis et al., 1997; Nikolaou et al., 2001) only results will discussed here. It is worth pointing out just that the analyses are performed in the frequency domain, and the results are transformed in the time domain through standard Fourier transformations (DFT), as suggested by Veletsos & Ventura (1984).
The soil profiles consist of a soft surface soil layer, of thickness H_1 and shear wave velocity V_{s1}, underlain by a stiffer stratum, of thickness H_2 and shear wave velocity V_{s2}. An elastic bedrock has been considered at the bottom of the second layer (Figure 2).

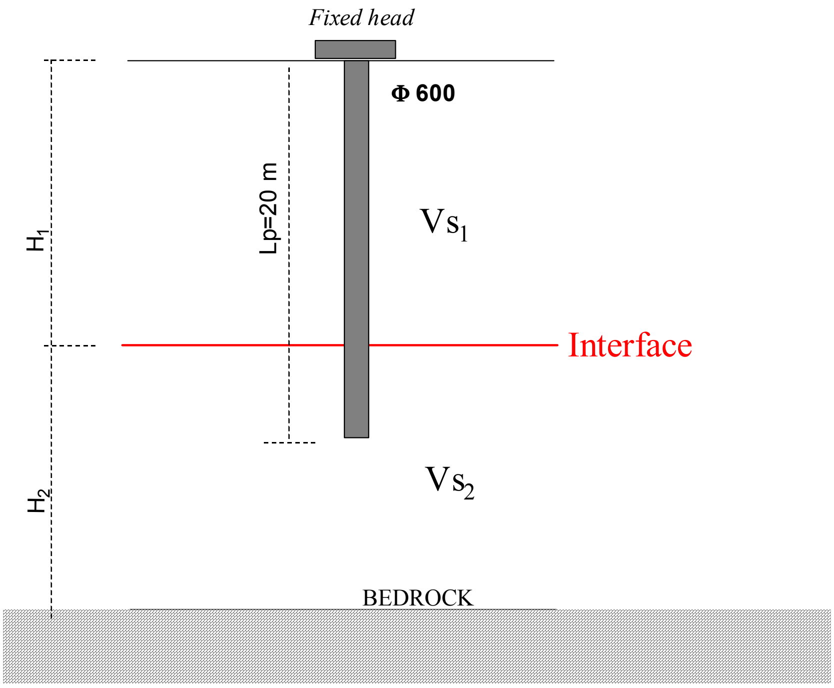

Figure 2. Reference scheme for the herein-reported parameter study

In all analyses the following parameters remain invariable: thickness of first layer (H_1=15 m), pile length (L=20 m) and diameter (d=0.60 m); Soil Poisson's ratio ($\nu_1 = \nu_2 = 0.4$); soil density ρ = 1.9 (Mg/m^3) and soil material damping ($\beta_1 = \beta_2 = 10\%$). The shear wave velocity of the elastic bedrock has been assumed equal to 1200 m/s. Preliminary free-field analyses indicated that the adopted value of soil damping is roughly consistent with the strain level induced by most of the accelerograms selected for this analysis set. To conform with most of the analytical results presented in the literature (Nikolaou et al., 2001; Mylonakis, 2001), the pile has been modelled as a solid elastic cylinder of Young modulus E_p=2.5 10^7KPa, moment of inertia $I_p=\pi d^4/64$ and density ρ=2.5 Mg/m^3.

As stated above, Italian recordings have been used as input motions in the numerical analyses. The accelerograms have been chosen in such a way that their original peak ground accelerations are as close as possible to the reference maximum peak acceleration on soil type A, a_{gR} , of the seismic zone in hand. All results shown in this paper refer to the Zone 1 of the actual Italian seismic zonation, which prescribes a_{gR}=0.35g on soil type A (Table 1). The selected accelerograms have been scaled in amplitude to provide a peak acceleration equal to 0.35g, linearly de-convoluted at bedrock level and then propagated upward in the soil to provide the excitation motion of the pile-soil system.

All parameter cases analysed in this study have been listed in Table 2. The analyses have been divided in three groups corresponding to three different geometrical schemes (S_1, S_2, and S_3) with varying thickness H_2 of the second layer. For each group, 6 cases have been investigated, corresponding to different shear wave velocity contrasts between the first and second layer (V_2/V_1 varies from 2 to 4). The shear wave velocities of the first and second layers were selected in such a way that subsoil profiles correspond to class C or D of the EC8 classification, on the base of the equivalent shear wave velocity $V_{s,30}$ in the upper 30 m of the soil profile. In Table 2 the fundamental frequency f_1 of the soil profile is also shown.

Table 2. Parameter cases analysed in this study

Scheme	H_1 (m)	H_2 (m)	V_1 (m/s)	V_2 (m/s)	f_1 (Hz)	V_2/V_1	$V_{s,30}$ (m/s)	Soil type EC8
S_1	15	15	100	200	1.34	2.0	133	D
			100	300	1.54	3.0	150	
			100	400	1.61	4.0	160	
			150	300	2.00	2.0	200	C
			150	400	2.22	2.7	218	
			150	600	2.40	4.0	240	
S_2	15	30	100	200	1.05	2.0	133	D
			100	300	1.34	3.0	150	
			100	400	1.51	4.0	160	
			150	300	1.56	2.0	200	C
			150	400	1.90	2.7	218	
			150	600	2.29	4.0	240	
S_3	15	6	100	200	1.54	2.0	160	D
			100	300	1.64	3.0	169	
			100	400	1.68	4.0	174	
			150	300	2.29	2.0	235	C
			150	400	2.39	2.7	245	
			150	600	2.47	4.0	255	

RESULTS

Selected results are shown in Figures 3 to 8. They represent envelopes of maximum kinematic bending moments computed (in time domain) along the pile for each of the 18 accelerograms chosen for the Zone 1 of the Italian seismic zonation. In particular, Figures 3, 5 and 7 correspond to EC8 class D soil profiles while Figures 4, 6 and 8 to EC8 class C profiles.
In all figures three grey zones have been added corresponding to the range of reinforced concrete pile yielding moments for typical reinforcements of the cross section (8ϕ16, 24ϕ12 and 12ϕ30) and magnitude of the normal load acting within the pile (the pile M-N domain was computed assuming a concrete of class C20/25 with Rck=25 N/mm^2 and fck=20N/mm^2 and steel rebars with fyk=375 N/mm^2 and ftk=450 N/mm^2, corresponding to the Italian steel class FeB38K). For each reinforcement the lower limit of the grey zone represent the cross section yielding moment corresponding to zero normal load while the higher one to a normal load equal to 1200 KN. From the above results several observations may be drawn:

- at the soil layer interface the kinematic bending moment increases dramatically when the shear wave velocity contrast between the second and first layer V_2/V_1 increases from 2 to 4. This happens for both for subsoil types D (Figure 3, 5, 7) and C (Figure 4, 6, 8); in the latter case, however, the peak bending moments at the soil layer interface and pile head are much smaller that those computed for subsoil type D.
- the maximum kinematic bending moment is always found in correspondence to the soil layer interface; for the geometrical schemes studied in this study and stiffness values adopted for concrete and soil, the active pile length l_a[1] results less than the first layer thickness H_1 ($l_a \leq$ 4.2m) and the pile behaves as “long”;
- three accelerograms, labelled A-STU000, A-STU270 and A-TMZ270, systematically induce the higher kinematic bending moments along the pile. This could be attributed to the long

[1] Randolph (1981) provides a well-known formula to compute the pile active length: $l_a = 1.75\, d\, (Ep/E_1)^{0.25}$, where *Ep* and E_1 represent the concrete and first layer normal stiffness, respectively; *d* is the pile diameter.

duration of strong motion even if the dominant frequencies (2.63, 5 and 1.56 Hz, respectively) are not close to the resonant ones (Table 2);

- for subsoil profiles corresponding to class D the computed kinematic bending moments may be well above the yielding moments of the pile cross section computed for typical concrete reinforcements and normal load acting along the pile. As the analyses have been performed assuming linear elastic behaviour for the pile, the computed kinematic bending moments can be assumed valid until they exceed the pile yielding moment.
- for subsoil profiles corresponding to class C and shear wave velocity contrasts $V_2/V_1 > 2$ the computed kinematic bending moments may also be worth of consideration, specially if the concrete pile has low reinforcement.
- as found by Nikolaou et al. (2001) by performing frequency domain analyses by a BDWF model, the maximum pile bending moment at the soil layer interface results slightly affected by soil depth below the pile tip. This statement is confirmed by results shown in Figures from 3 to 8, obtained in the time domain. For a fixed value of shear wave velocity contrast and subsoil class, the maximum kinematic bending moment at the soil layer interface, in fact, results almost unaltered by soil depth below the pile tip, i.e. by the different geometrical scheme S_1, S_2 and S_3 here investigated.

CONCLUSIONS AND PERSPECTIVES

The paper evaluates, from a practical viewpoint, to what extent pile kinematic bending moments are worth considering in typical subsoil conditions, namely soil types C and D according to EC8.
A comprehensive parameter study has been performed considering a single floating and fixed-head pile in two-layered soil deposits subjected to vertically propagating seismic waves. The Mylonakis et al. (1997) analysis method, based on a Beam on Dynamic Winkler Foundation model, has been adopted for computing the bending moments. Italian acceleration recordings have been used as input motion for the numerical analyses. Results have been discussed in terms of maximum bending moment envelopes computed along the pile for each of the 18 natural accelerograms selected for the Zone 1 of the Italian seismic zonation (OPCM 3274/2003 and OPCM 3519/2006).
Analysis results indicate that in the high seismicity zones of the Italian zonation and for subsoil types D and C, kinematic bending moments may be very high depending on the shear wave velocity contrast between the second and first layer. As expected, the most dangerous conditions verify for subsoils of class D. Anyway the paper evidences that also for subsoil profiles corresponding to class C of EC8 the computed kinematic bending moments may be worth of consideration specially for shear wave velocity contrast $V_2/V_1 > 2$.
To assess safety against yielding of a reinforced concrete pile (assuming that the concrete cross section first yielding moment is almost equal to the failure one), computed kinematic bending moments have been compared to the pile yielding moments for typical cross section reinforcements and magnitude of the normal load acting in the pile. It has been observed that in most of the class D profiles investigated in the paper the kinematic bending moments at the soil layer interface may be well above the yielding moment of the pile; for class C profiles the above conclusion is still valid but only for subsoil deposits with shear wave velocity contrasts $V_2/V_1 > 2$ and piles with lower percentage of steel reinforcement.
At present, further numerical analyses on different geometrical and mechanical schemes are in progress, by using both BDWF approaches and 3D continuum ones, the latter expected to provide a deeper insight into the seismic response of piles as soil and pile non linearity may be better modelled.

This research work has been developed in the framework of the *RELUIS Project* promoted and funded by the Department of Civil Protection (DPC) of the Italian Government and coordinated by the AGI (Italian Geotechnical Association). A theme of the project (Task 6.4 - Deep Foundation) is aimed at individuating elements to be introduced in the Italian technical code, regarding the seismic design of deep foundations, with particular attention to the effects of kinematic interaction.

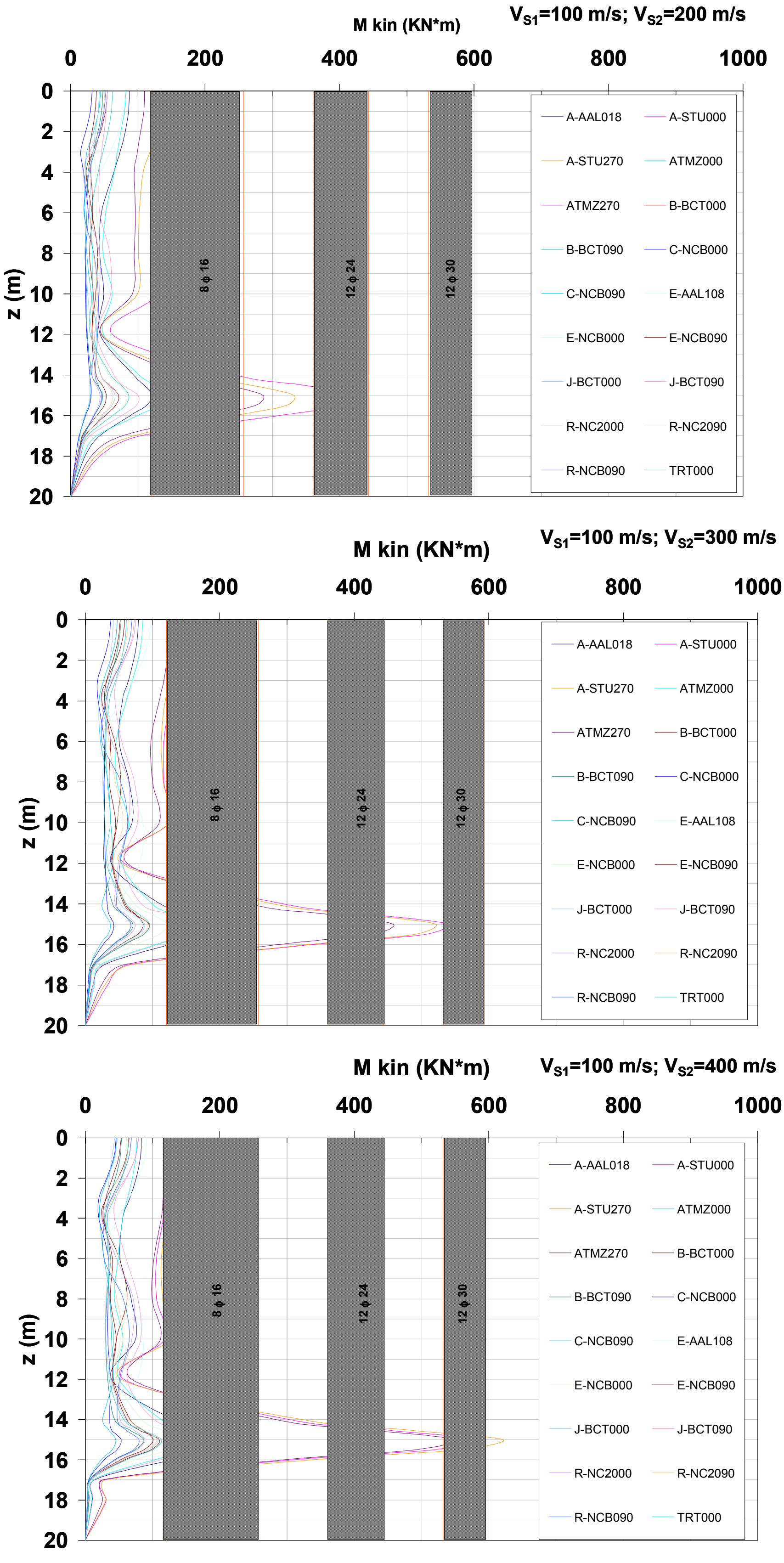

Figure 3. Kinematic bending moments for the two-layer soil profile S_1 (H_1=15 and H_2=15m) and soil type D of EC8. From top to bottom: V_2/V_1=2, 3 and 4

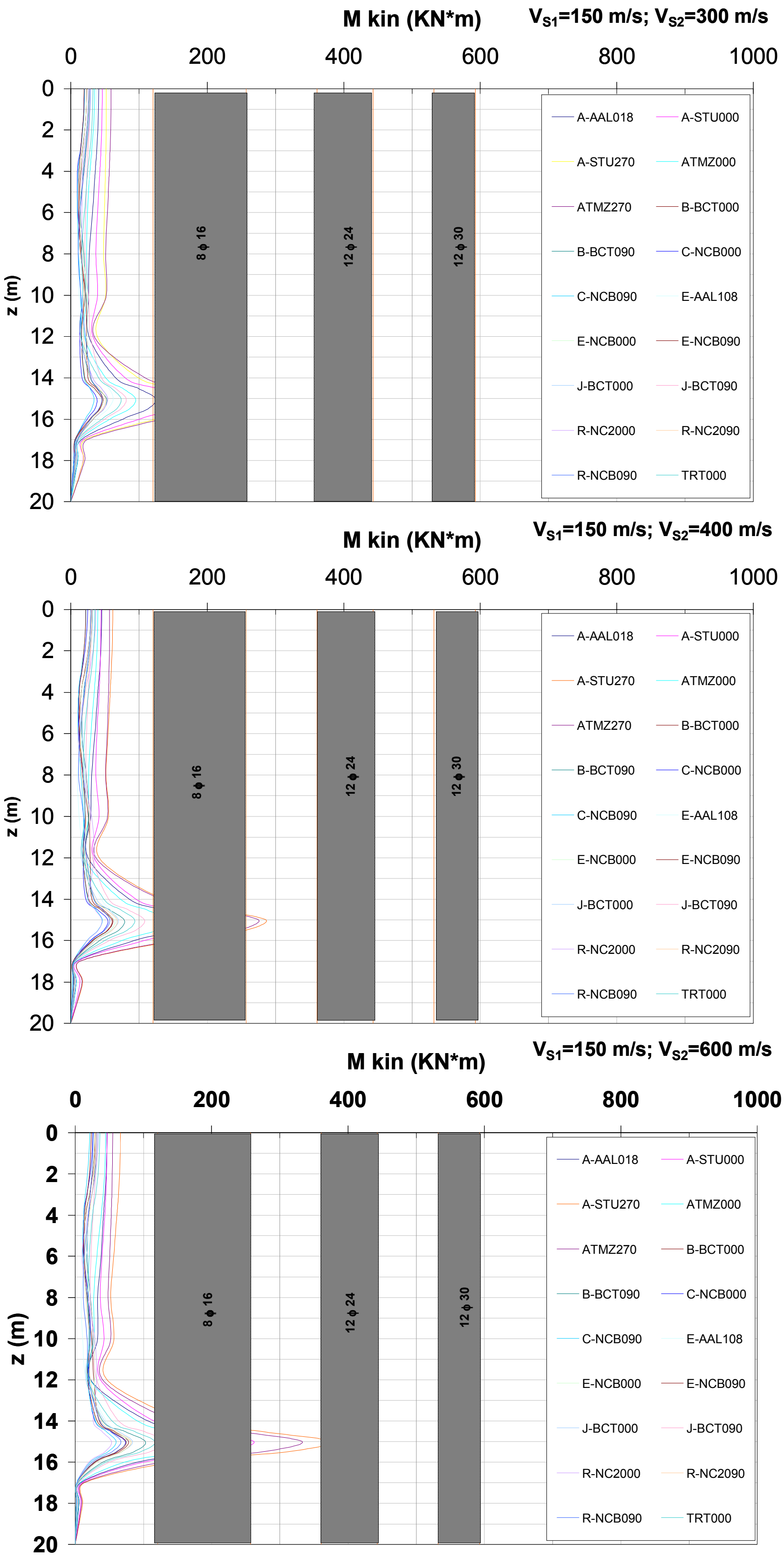

Figure 4. Kinematic bending moments for the two-layer soil profile S_1 (H_1=15 and H_2=15m) and soil type C of EC8. From top to bottom: V_2/V_1=2, 2.7 and 4

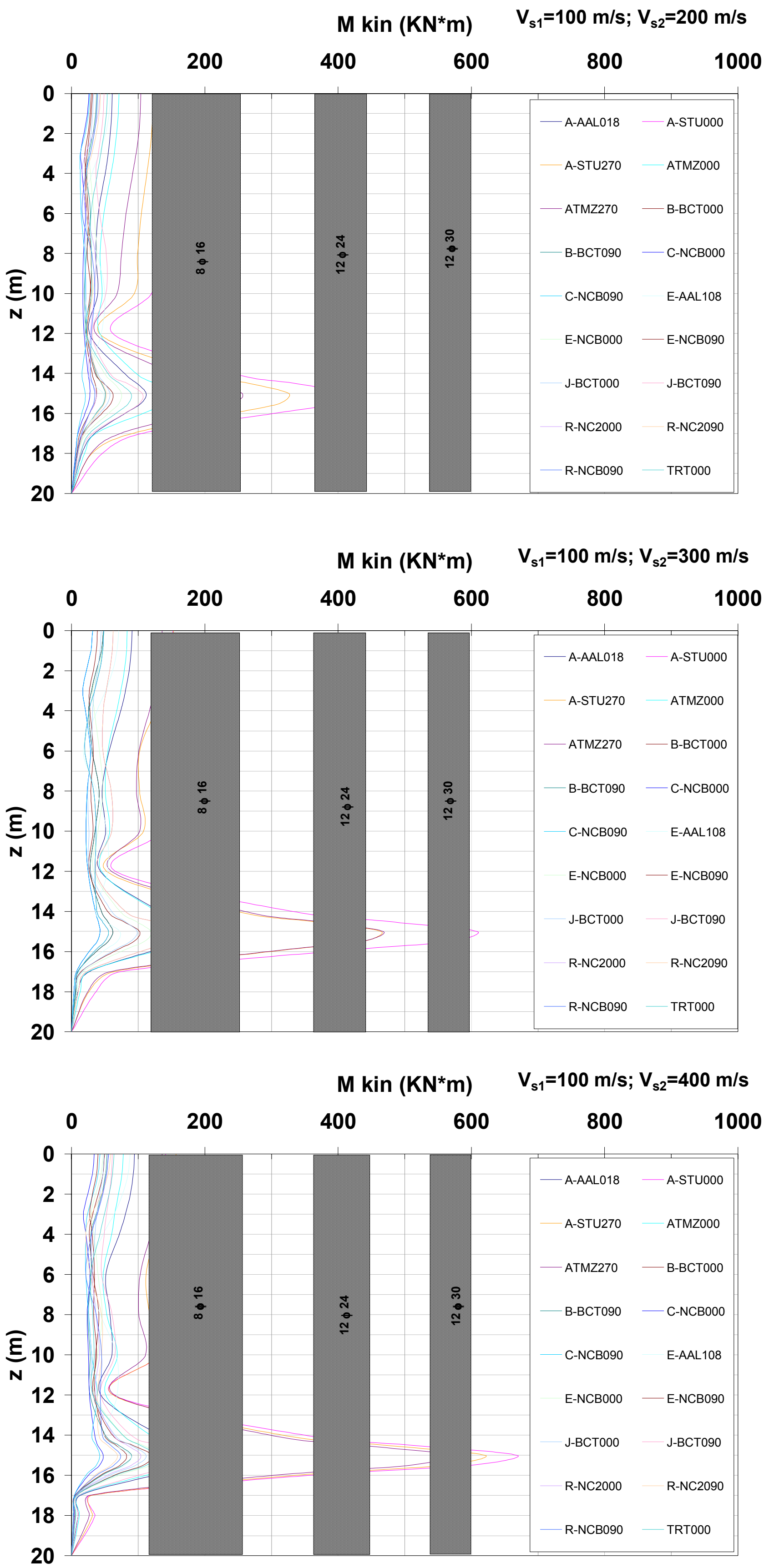

Figure 5. Kinematic bending moments for the two-layer soil profile S_2 (H_1=15 and H_2=30m) and soil type D of EC8. From top to bottom: V_2/V_1=2, 3 and 4

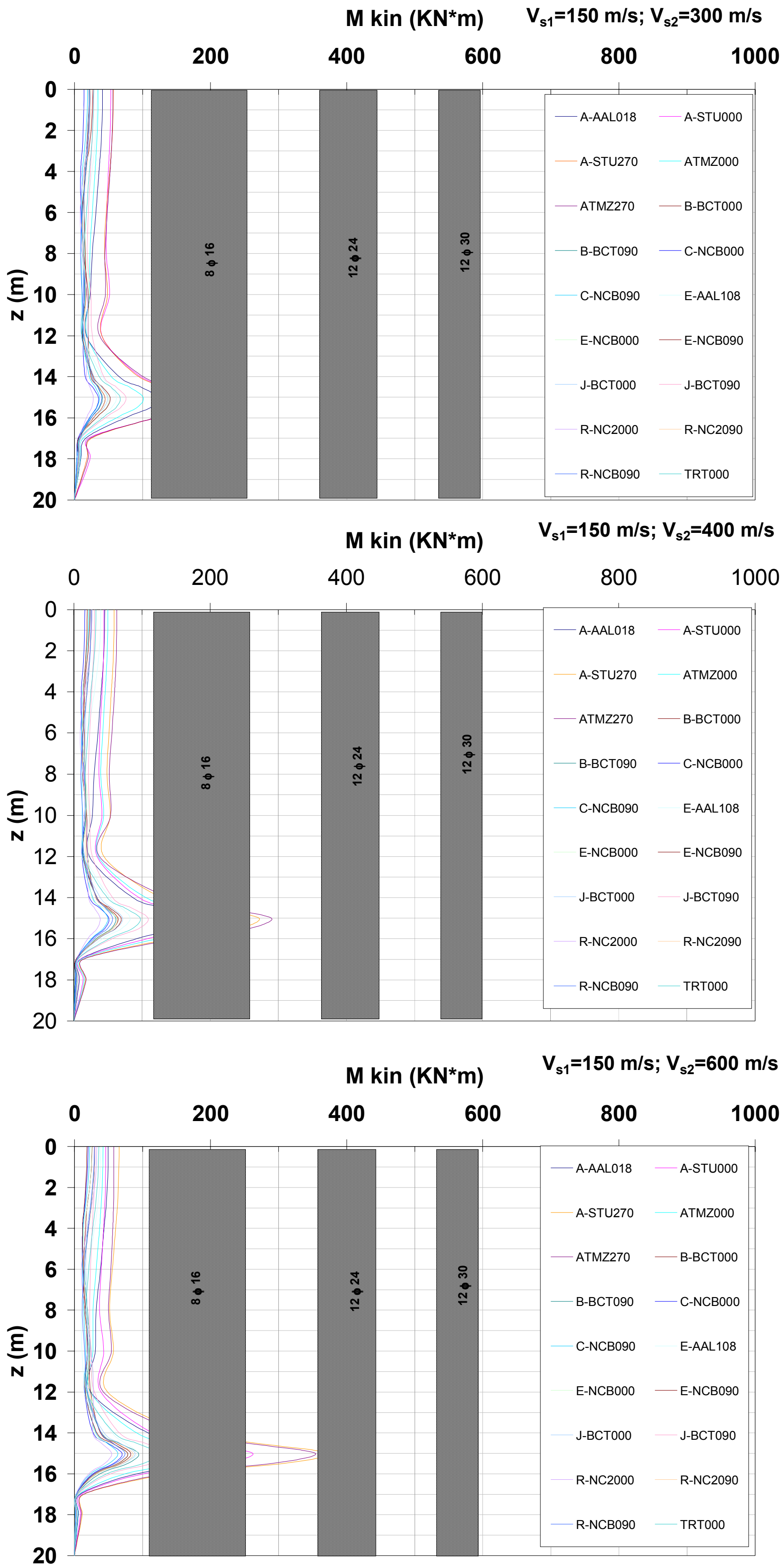

Figure 6. Kinematic bending moments for the two-layer soil profile S_2 (H_1=15 and H_2=30m) and soil type C of EC8. From top to bottom: V_2/V_1=2, 2.7 and 4

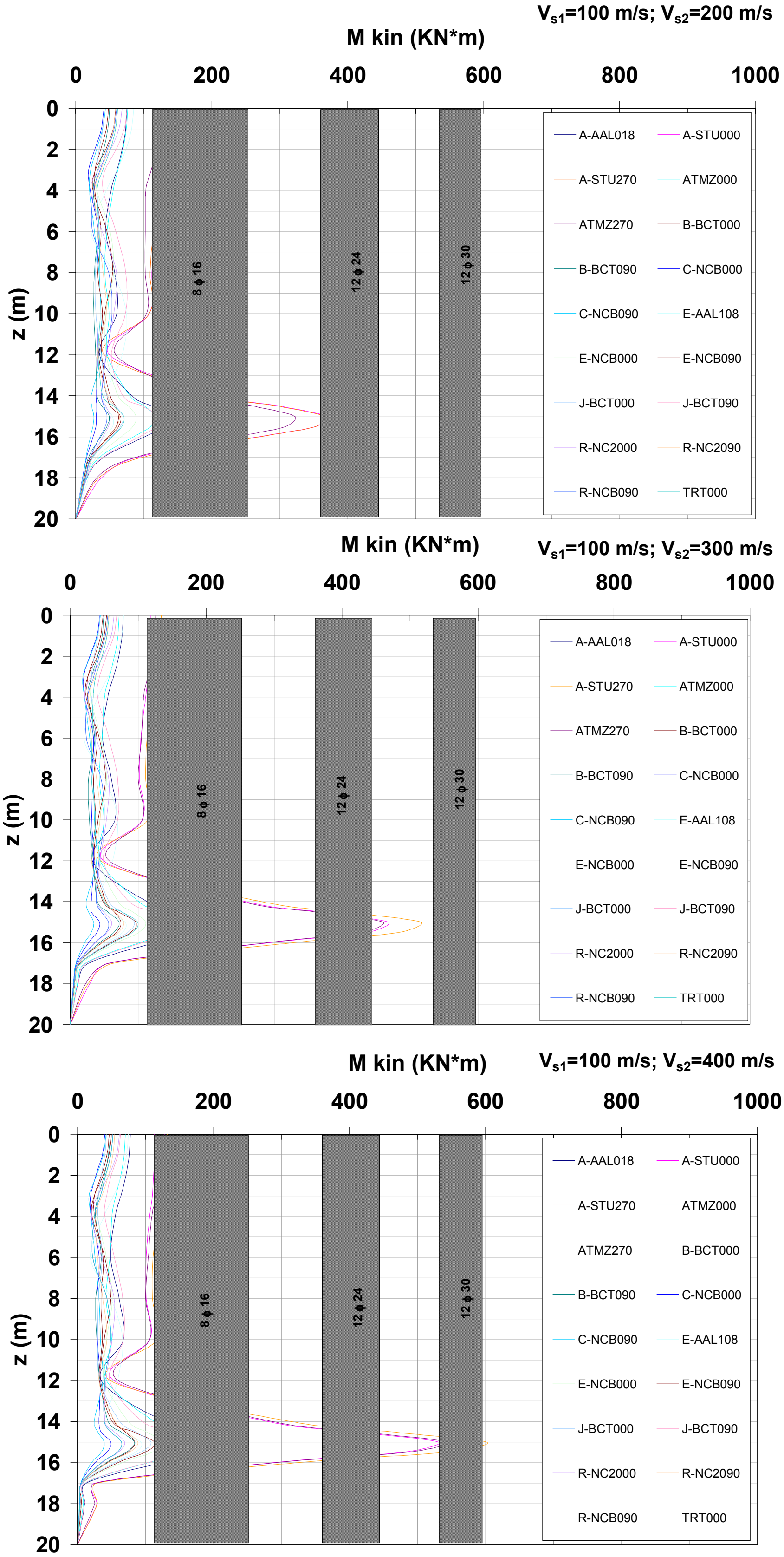

Figure 7. Kinematic bending moments for the two-layer soil profile S_3 (H_1=15 and H_2= 6m) and soil type D of EC8. From top to bottom: V_2/V_1=2, 3 and 4

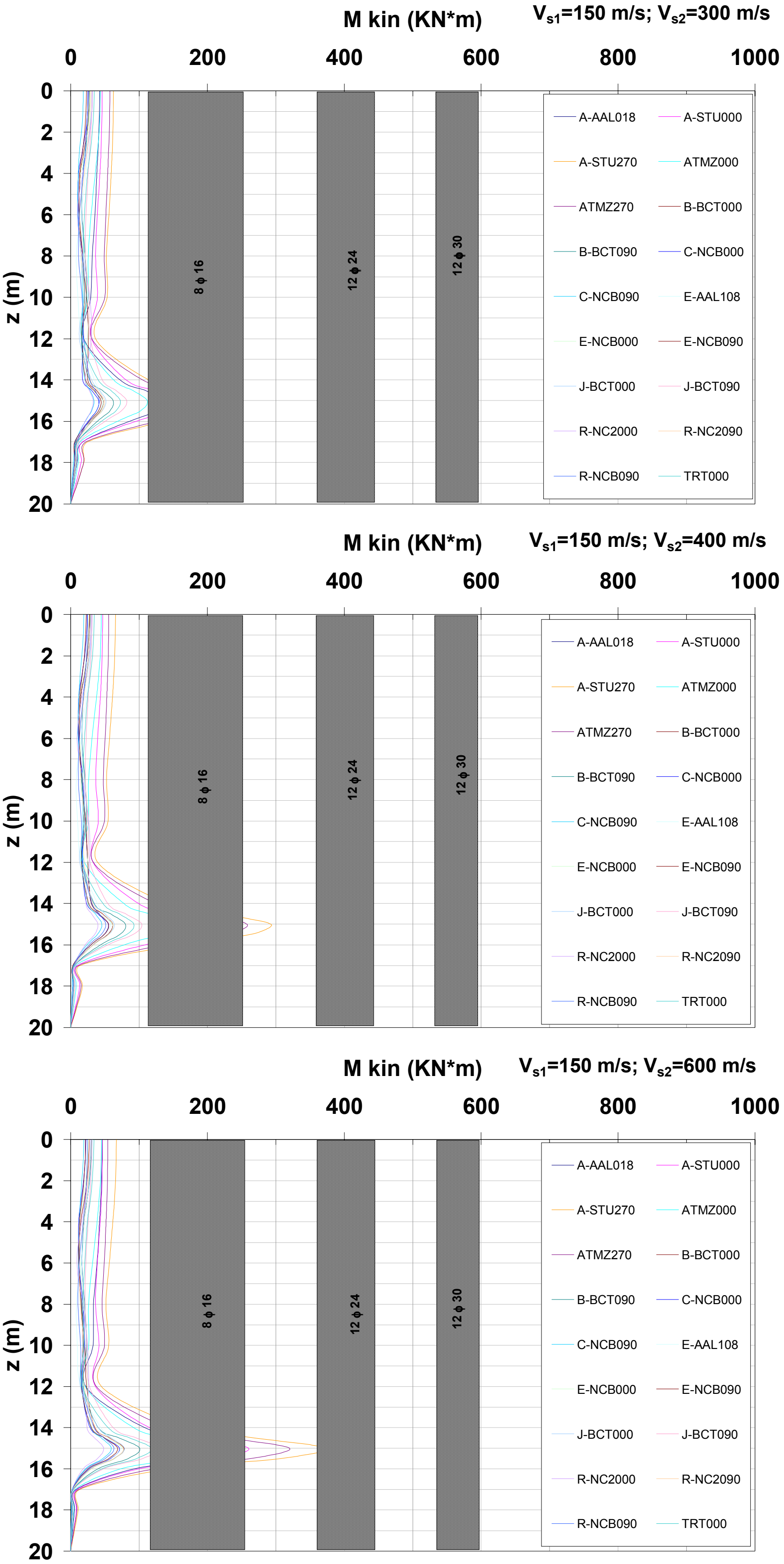

Figure 8. Kinematic bending moments for the two-layer soil profile S_3 (H_1=15 and H_2=6m) and soil type C of EC8. From top to bottom: V_2/V_1=2, 2.7 and 4

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