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EFFECTS OF SOIL NON-LINEARITY ON BENDING MOMENTS IN PILES DUE TO SEISMIC KINEMATIC INTERACTION

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ABSTRACT

This paper presents the results of different types of dynamic numerical analysis performed to evaluate kinematic bending moments developed during earthquakes in a single pile embedded in a two layer subsoil profile. A quasi 3D finite element computer program has been used for the analyses performed in the time domain. The pile has been considered as an elastic beam, while the soils have been modelled using the equivalent linear constitutive model with strain-dependent modulus and damping, using typical relationships among these parameters and shear strain. The results of these analyses have been compared with those obtained with linear analyses. The study shows the relevance of soil non linearity on the bending moments induced by seismic kinematic interaction. The comparison between results shows that the distribution of bending moments is also affected by the seismic input.

Keywords: kinematic interaction, single pile, finite elements, non linear seismic response

INTRODUCTION

Dynamic response of pile foundations is a very complex process involving a number of factors, such as inertial interaction between superstructure and foundation, kinematic interaction between piles and soil, seismically induced pore-water pressures and the non linear soil response to strong earthquakes. On the contrary in engineering practice simple pseudostatic analyses are used, neglecting most of the factors that strongly affect pile behaviour. Much of the reported research in the field of dynamic analysis of pile foundations assumes linear behaviour of the soil media: Flores-Berrones and Whitman (1982), Kaynia and Kausel (1982), Gazetas (1984), Dobry and Gazetas (1988), Makris and Gazetas (1992), Kavvadas and Gazetas (1993), and others have carried out linear analyses of single piles and pile groups in the frequency domain. Under strong excitation, however, the nonlinear behaviour of the soil media at the soil-pile interface has a strong influence on the response of the pile foundation. Therefore the focus in recent years has shifted to incorporate the non-linear behaviour of soil media using time domain analyses. Wu and Finn (1997a) presented a quasi 3D finite element method for nonlinear dynamic analysis; Bentley and El Naggar (2000) investigated the kinematic response of single piles to account for soil plasticity using the Drucker-Prager soil model and gapping at the soil-piles interface; Cai et al. (2000) included work hardening plasticity of soil in a finite element analysis in the time domain; Mahesshwari et al. (2003; 2005) used a hierarchical single surface soil model to study the the free-field and kinematic response of single piles. The aim of the large part of these studies has been restricted to investigate the influence of soil non linearities on the pile head stiffness and motion. The associate pile bending has not been adequately explored.

In this paper the effect of material nonlinearity of the soil on kinematic bending moments in single piles in layered soil is investigated. A quasi 3D finite element computer program VERSAT-P3D (Wu,

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2004) is used for a parametric study. The analysis is performed in the time domain using the equivalent linear constitutive model with strain-dependent modulus and damping using typical relationships among the shear modulus, damping ratio, and effective shear strain (Seed and Idriss, 1970; Seed et al., 1986). Simplified subsoil conditions are considered, a two layered profile, with different values of the stiffness contrast between the two soil layers, in terms of their respective S-waves velocities V_{s2}/V_{s1}. Italian real acceleration time histories are considered.

NUMERICAL MODEL FOR NONLINEAR DYNAMIC ANALYSIS

The numerical model for non linear analysis is an extension of the method for elastic analysis presented by Wu and Finn (1997b). Under vertically propagating shear waves (Fig. 1) the soil undergoes primarily shearing deformations in *XOY* plane, except in the area near the pile where extensive compressional deformations develop in the direction of shaking. The compressional deformations also generate shearing deformations in *YOZ* plane. Therefore, assumptions are made that dynamic response is governed by the shear waves in the *XOY* and *YOZ* planes, and the compressional waves in the direction of shaking, *Y*. Deformations in the vertical direction and normal to the direction of shaking are neglected. Comparisons with full three-dimensional (3D) elastic solutions confirm that these deformations are relatively unimportant for horizontal shaking (Wu and Finn 1997b).

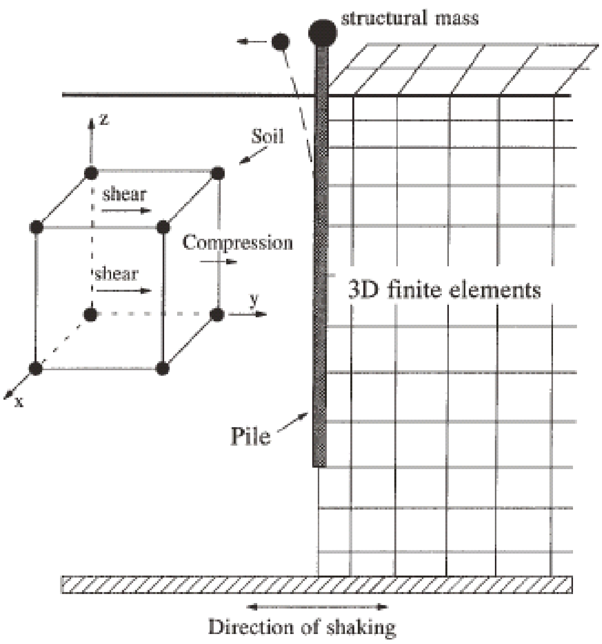

Figure 1. The principle of the quasi-3D dynamic analysis of the pile-soil-structure interaction (after Wu and Finn, 1997)

Piles are modelled using the ordinary Eulerian beam theory. Bending of the piles occurs only in the direction of shaking. Dynamic soil pile interaction is maintained by enforcing displacement compatibility between the pile and soils. A quasi-3D finite element program VERSAT-P3D (Wu 2004) has been developed for the analysis of dynamic soil-pile-structure interaction. An eight-node brick element is used to represent soil, and a two-node beam element is used to simulate the piles. The global dynamic equilibrium equations are written in matrix form as

$$[M]\{\ddot{v}\}+[C]\{\dot{v}\}+[K]\{v\}=-[M]\{I\}\ddot{v}_0(t) \tag{1}$$

in which $v_0(t)$ is the base acceleration, $\{I\}$ is a unit column vector, and $\{v\}$, $\{v\}$, and $\{v\}$ are the relative nodal acceleration, velocity, and displacement, respectively. $[M]$, $[C]$, and $[K]$ are the mass, damping, and stiffness matrices, respectively. Direct step-by-step integration using the Wilson θ method is employed in VERSAT-P3D to solve the equations of motion in eq. (1). The nonlinear hysteretic behaviour of the soil is modelled by using a variation of the equivalent linear method in the SHAKE program (Schnabel et al. 1972). An equivalent linear method is employed in VERSAT-P3D to model the nonlinear hysteretic behaviour of soil. The basis of this method is the assumption that the hysteretic behaviour of soil can be approximated by a set of secant shear moduli and viscous damping ratios that are compatible with current levels of shear strain. This method has been widely accepted in engineering practice: it has been incorporated in the computer code SHAKE (Schnabel et al. 1972) for one-dimensional ground motion analyses and in QUAD-4 (Idriss et al. 1973) for two-dimensional plane strain analyses. To approximate better the nonlinear behaviour of soil under strong shaking, in VERSAT-P3D, compatibility among the secant shear modulus, damping ratio, and shear strain may be enforced at each time step during the integration of equations of motions. This ensures that the time histories of moduli and damping ratios in each soil element are followed during the analysis, in contrast with the equivalent linear approach described earlier in which a single effective value is used to represent the entire time history. In a practical VERSAT-P3D analysis, the shear moduli and damping ratios are updated at specified time intervals ranging from each time step for integration to intervals that balance accuracy and computational time. For the analyses presented herein, it was sufficient to update the soil properties every *0.5 s* based on the peak strain levels from the previous time interval. This value was selected on the basis of preliminary analyses using different time intervals.The hysteretic damping ratio λ of soil is included by using equivalent viscous damping. A procedure for estimating viscous damping coefficients for each individual element proposed by Idriss et al. (1974) is employed in VERSAT-P3D. The main advantage of this procedure is that a different degree of damping can be applied in each finite element according to its shear strain level. The damping is essentially of the Rayleigh type, which is both mass and stiffness dependent. The damping matrix $[C]_{elem}$ for a soil element is given by

$$[C]_{elem} = \lambda_{elem}\left(\omega_1 [M]_{elem} + \frac{[K]_{elem}}{\omega_1}\right) \tag{2}$$

where ω_1 is the fundamental frequency of the pile-soil-system and is applied to each element. The frequency ω_1 is obtained by solving the corresponding eigenvalue problem. The hysteretic damping ratio, λ_{elem}, is prescribed as a function of element shear strain (Seed et al. 1986).

REFERENCE SUBSOIL MODELS

The analytical simulations have been performed on simplified subsoil conditions, a two layered profile, with a total thickness of 30 m, overlying a soft rock half-space as a bedrock (fig.2). The two layers, a soft clay and a medium density gravel, have the same thickness of 15 m and constant values of S-waves velocity with depth. Two different values of S-waves velocities have been considered for the first layer (100 and 150 m/s), while the second layer has always a S-wave velocity of 400 m/s. The two resulting profiles can be classified respectively as subsoil type D and subsoil type C, according to EN-1998-1 (2003). It is worth mentioning that Eurocode 8-5 (2003) recognises the importance of kinematic interaction for important structures in regions of moderate to high seismicity, when the ground profile contains consecutive layers of sharply differing stiffness. The well known relation between the shear wave velocity and the small strain shear modulus G_0 is:

$$V_s(z) = \sqrt{\frac{G_0(z)}{\rho}} \tag{3}$$

where ρ is the soil density.

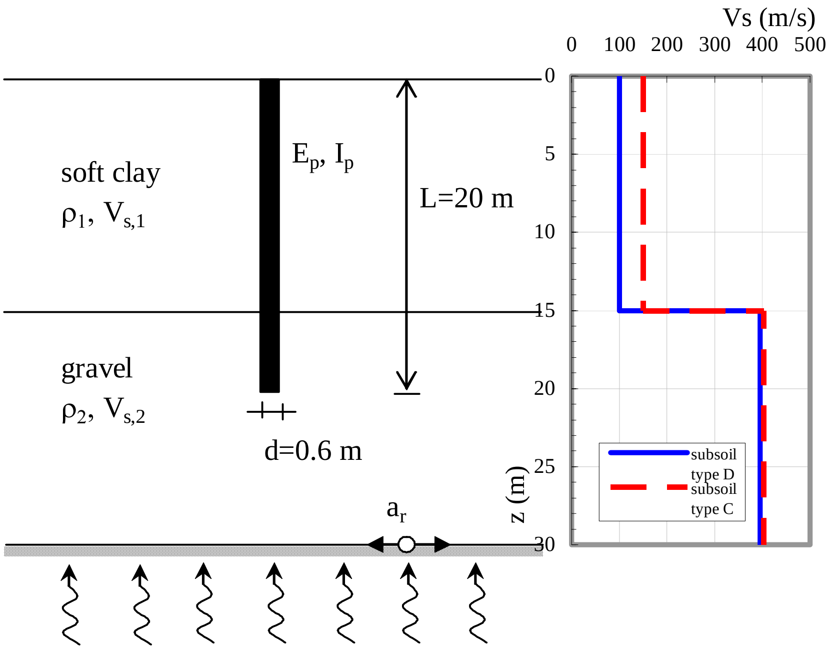

Figure 2. Reference soil models

Table 1 summarizes the geotechnical parameters and the corresponding equivalent velocity defined by Eurocode 8-1:

$$V_{s,30} = \frac{30}{\sum_{i=1,n} \frac{h_i}{V_{s,i}}} \tag{4}$$

Table 1. Geotechnical parameters of the soils

	G_{01} (kPa)	G_{02} (kPa)	V_{s1} (m/s)	V_{s2} (m/s)	ν_1	ν_2	γ_1 (kN/m^3)	γ_2 (kN/m^3)	V_{s2}/V_{s1}	$V_{s,30}$ (m/s)
soil type D	19000	304000	100	400	0.4	0.4	19	19	4	160
soil type C	42750	304000	150	400	0.4	0.4	19	19	2.667	218

Nonlinear analyses are performed to account for the changes in shear modulus G and damping ratios D due to dynamic shear strains. The shear-strain dependent shear modulus $G(\gamma)/G_0$ and damping ratios $D(\gamma)$ used in the analysis are shown in Fig. 3. The pile has the following characteristics: diameter d = 0,60 m, length L = 20 m and Young modulus E_p = 30.000 MPa; it passes through the soft clay layer and it is embedded in the gravel layer. The pile head is fixed against rotation.

ANALYSIS RESULTS

In order to investigate the effects of material nonlinearity of soil on the dynamic behaviour of a single pile, three different types of analysis have been performed:

- linear analyses with constant small strain shear modulus and damping (L analysis);
- non linear analyses with shear-strain dependent shear modulus $G(\gamma)/G_0$ and damping ratios $D(\gamma)$ (NL analysis);
- “equivalent linear” analyses, with constant shear modulus and damping deduced from previous non linear free-field response analyses (LE analysis).

The finite element mesh used for the analysis is shown in Fig. 4: it consists of 6500 nodes and 5400 elements. Due to symmetry, analyses have been conducted on a half mesh.

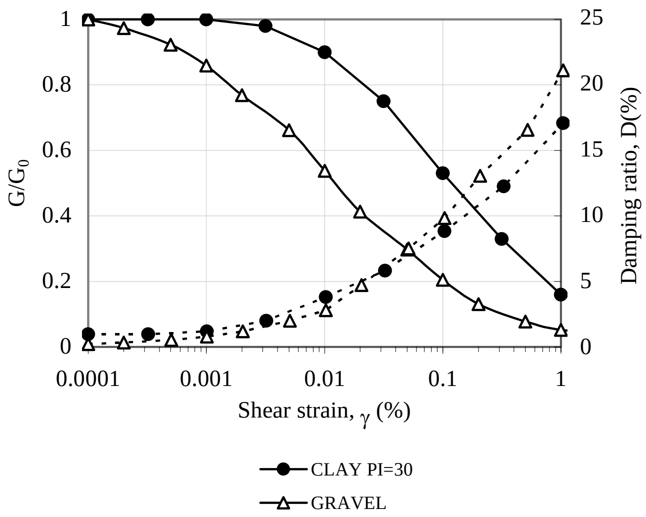

Figure 3. Variation of normalized stiffness and damping

The sizes of the elements varies in the mesh with smallest sizes close to the pile and to the soil surface in order to allow more detailed modelling of the stress and strain field where lateral soil-pile interaction is strongest. The pile is modelled using 20 beam elements.

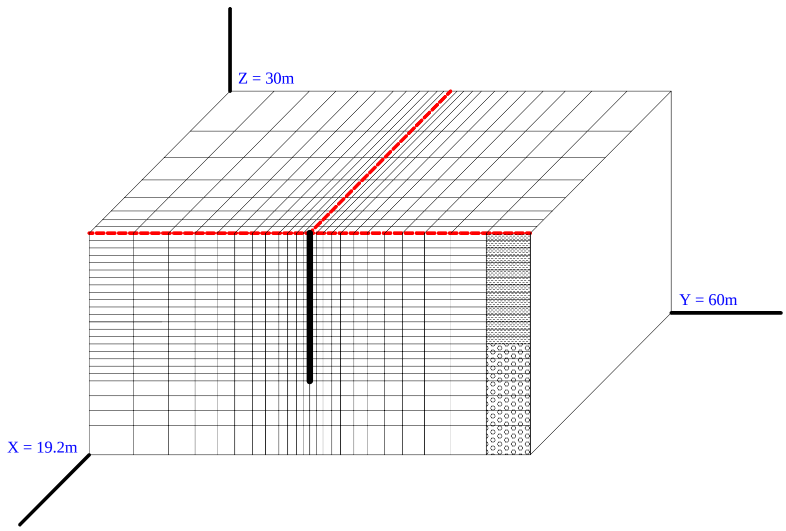

Figure 4. Finite element pile model

The analyses have been performed in the time domain; the input acceleration time histories have been selected from a database of records of Italian seismic events, assembled in the framework of the ReLUIS research programme (Lanzo, 2006). The signals have been scaled to values of a_r equal to 0.35g (according to the seismic zonation specified by OPCM 3274, 2003), and have been applied to the base of the subsoil models. Table 2 summarizes the main data (seismic event, magnitude, peak ground acceleration, location of the recording station, distance from the epicentre) of the acceleration time histories used in the analyses.

Table 2. Acceleration time-histories used in the analyses

Name	Event	Station	M_w	Distance (km)	PGA (g)
A-TMZ270	Friuli, 1976	Tolmezzo	6.5	23	0.357
A-STU270	Irpinia, 1980	Sturno	6.9	32	0.320
E-NCB090	Umbria-Marche, 1997	Norcia Umbra	5.5	10	0.382

Linear analysis

The results of linear analyses in terms of kinematic bending moments along pile depth are reported in fig. 5. The analyses were performed considering constant values of shear modulus for each layer and equal to the small strains values deduced from S-wave velocity by means of equation (3); two different values of damping were considered (10% and zero) constant with depth. In all the cases the bending moments envelope due to kinematic interaction presents a relative maximum at the pile head (for fixed head piles) and at the interface between the two layers. The value of the maximum moment at the interface increases with increasing stiffness contrast and is strongly influenced by damping ratio. If no damping is considered the maximum moments are very high, often much bigger than the range of possible yielding moments of the pile section. Similar results have been obtained by Maiorano and Aversa (2006) and Aversa et al. (2005).

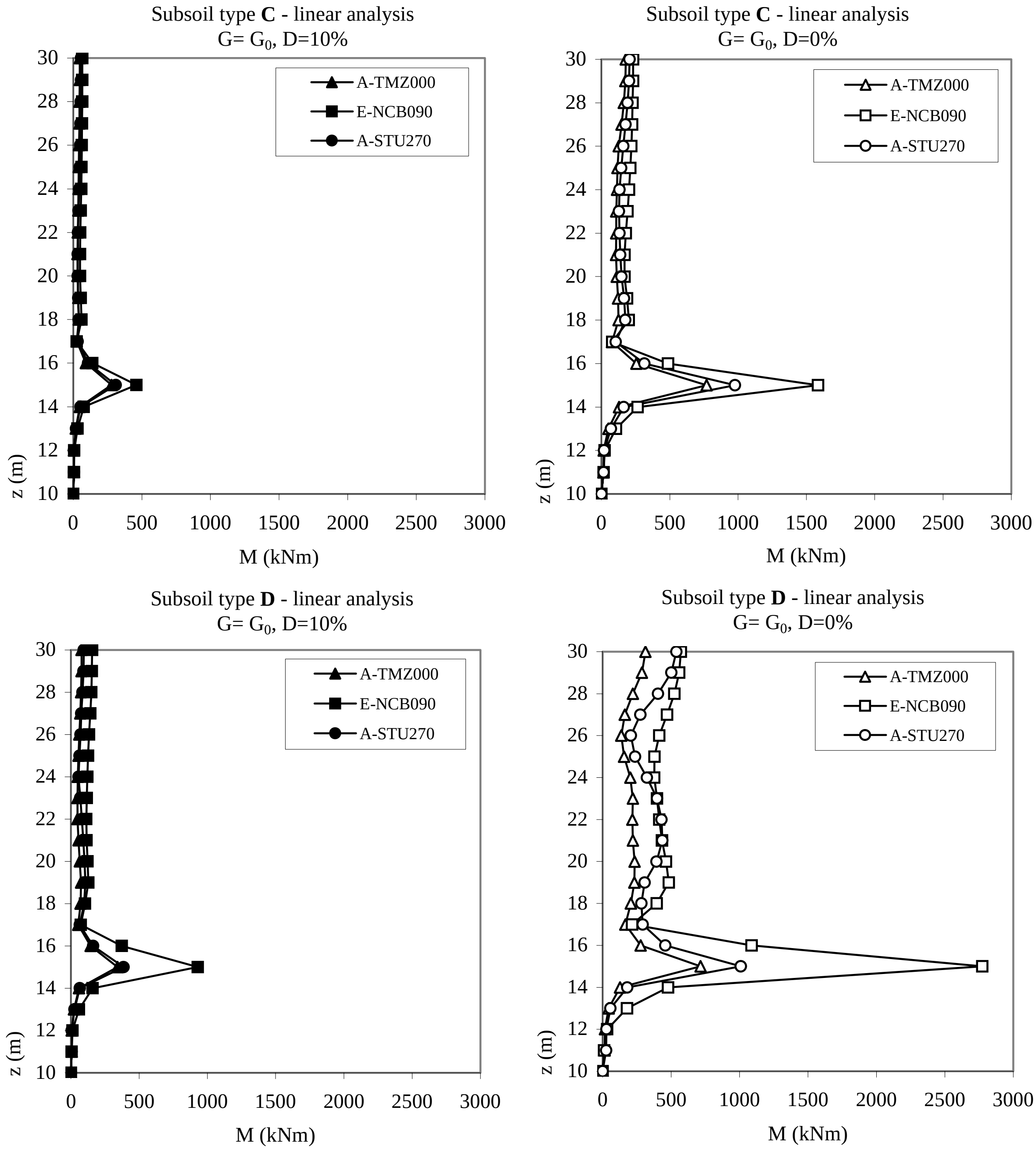

Figure 5. Kinematic bending moments envelope using elastic analysis

Nonlinear analysis

Nonlinear analysis have been performed to account for the changes in shear modulus G and damping ratios D due to dynamic shear strains. The shear-strain dependent shear modulus $G(\gamma)/G_0$ and damping ratios $D(\gamma)$ used in the analysis are shown in Fig. 3. The results in terms of kinematic bending moments along pile depth are reported in fig. 6 for the two subsoil types. The distributions of bending moments are different from those obtained with linear analysis and presents two relative maximum values: at middle height of the pile and at the interface between the two layers.

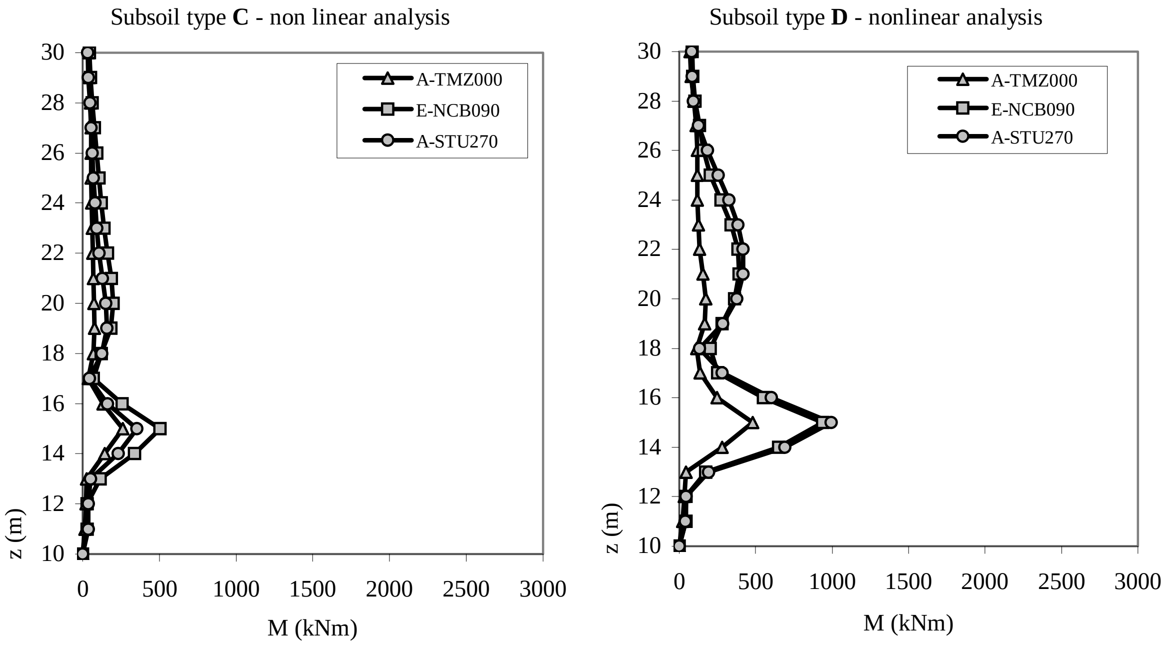

Figure 6. Kinematic bending moments derived from non linear analysis

Equivalent linear analysis

A so called "equivalent linear analysis" was also performed using constant values of shear modulus and damping deduced from a previous non linear free-field response analysis in the two types of subsoils. The analyses have been performed by means of the computer code EERA (Bardet et al., 2000). Ground conditions and soil behaviour have been modelled according to Figs. 2 and 3. In fig. 7 the profiles of shear modulus and damping obtained from site response analyses are reported for the two subsoils. For subsoil type D the mean values of G/G_0 varies between 0.7 and 0.5 for the first layer and between 0.9 and 0.7 for the second layer, while damping ratio varies between 9 and 15 for the clay and between 3 and 5% for the gravel. For subsoil type C the range of mean values are 0.6-0.8 for G/G_0 and 6-10% for D, for the first layer, 0.6-0-9 forG/G_0 and 3-5% for D, for the second layer. In tab. 3 and 4 the value of G and D used in the "equivalent linear" analyses are reported.

Table 3. Subsoil type C – Mean values of G and D form EERA nonlinear analyses

Layer	A-STU270		A-TMZ000		E-NCB090	
	G (kPa)	D (%)	G (kPa)	D (%)	G (kPa)	D (%)
1	25650	10	29925	9	34200	6
2	182400	5	212800	4	243200	3

Table 4. Subsoil type D – Mean values of G and D form EERA nonlinear analyses

Layer	A-STU270		A-TMZ000		E-NCB090	
	G (kPa)	D (%)	G (kPa)	D (%)	G (kPa)	D (%)
1	9500	15	11400	10	13300	9
2	212800	5	212800	4	258400	3

Using these values of shear modulus and damping, linear analyses (called “equivalent linear analyses”) have been performed. In fig. 8 comparisons between the results of linear, “equivalent linear” and non linear analyses in terms of bending moments profiles are reported for the subsoil type C and the input motion A-TMZ000.

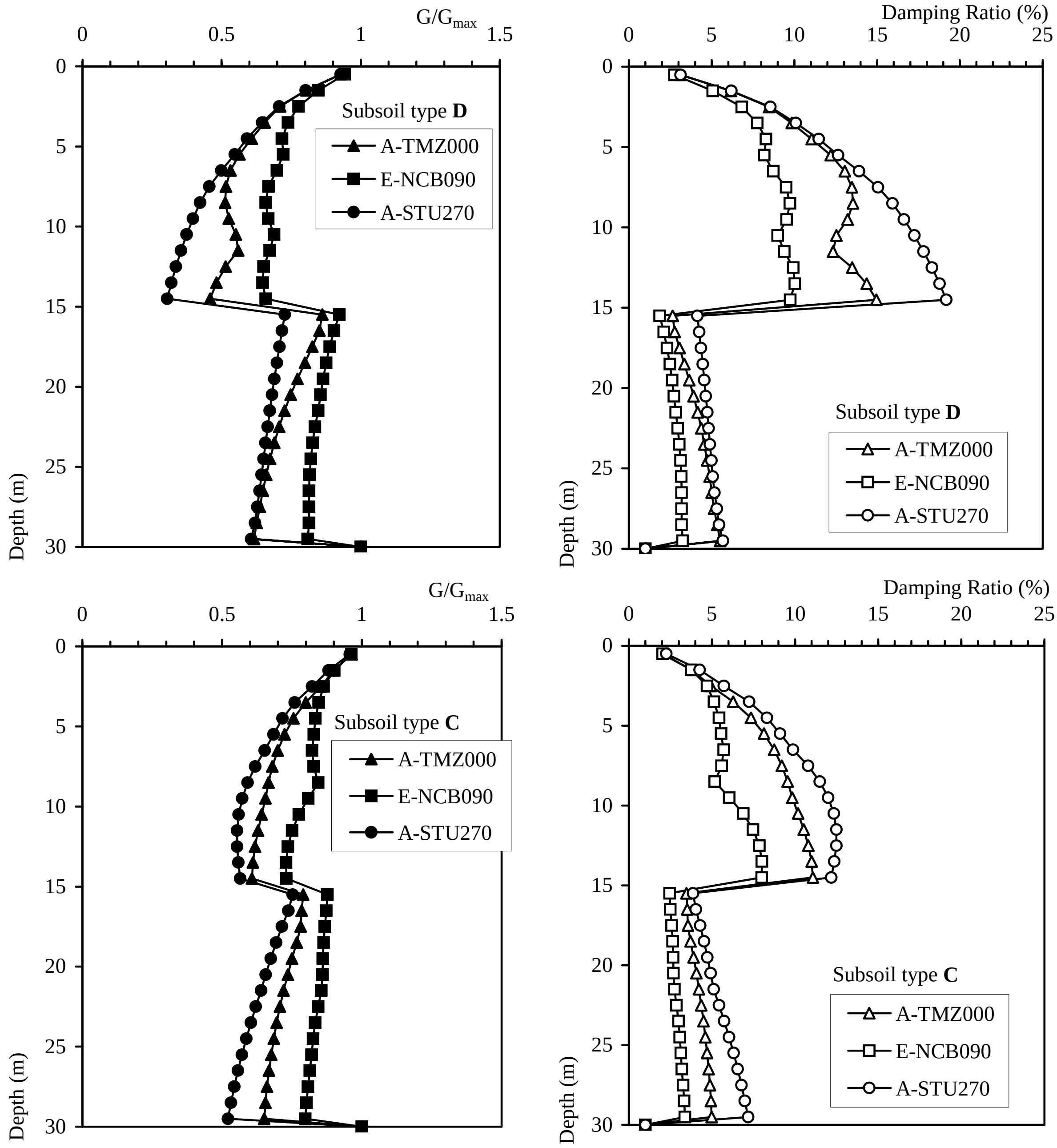

Figure 7. Distribution of shear modulus and damping from EERA

In tab. 5 and 6 the moments at the pile head (M_{cap}) and at the interface between the two layers (M_{int}) for all the analyses and for the two subsoils are reported. For the two type of subsoil the kinematic bending moments derived from NL and LE analyses are in good agreement, especially at the interface between the two layers. At the pile head the moments obtained with the LE analyses are always higher than those derived from NL analyses; this is due to the fact that the distribution of shear modulus and damping in LE analyses was assumed constant in each layer. The linear analysis with constant shear modulus and damping obtained from a previous nonlinear free-field analysis is therefore able to give a good estimation of the maximum kinematic moments.

Table 5. Subsoil type C - kinematic bending moments

analysis	A-STU270		A-TMZ000		E-NCB090	
	M_{int} (kNm)	M_{cap} (kNm)	M_{int} (kNm)	M_{cap} (kNm)	M_{int} (kNm)	M_{cap} (kNm)
L (D=0%)	977.99	205.60	769.92	179.01	1585.64	232.01
L (D=10%)	309.30	57.29	279.43	48.20	459.93	66.33
LE	307.11	74.28	373.57	76.59	538.24	88.75
NL	352.50	34.75	262.04	40.10	503.62	76.59

Table 6. Subsoil type D – kinematic bending moments

analysis	A-STU270		A-TMZ000		E-NCB090	
	M_{int} (kNm)	M_{cap} (kNm)	M_{int} (kNm)	M_{cap} (kNm)	M_{int} (kNm)	M_{cap} (kNm)
L (D=0%)	1009.64	538.09	715.13	312.59	2774.11	571.17
L (D=10%)	384.87	94.53	341.12	77.76	929.26	155.19
LE	980	118	463.59	168.35	974.25	217.75
NL	994.03	81.07	481.71	69.63	939.94	84.48

On the contrary L analyses with $G=G_0$ and D=10% give rise to kinematic moments at the interface between the two layers very close to the ones obtained with NL analyses only for subsoil type C.For subsoil type D the moments obtained from NL analysis are higher than those obtained from L analysis with D=10%; only in one case the moments are equal to the moment obtained from a linear analysis with zero damping. These results show the importance of the choise of the damping values to introduce in a linear analysis.

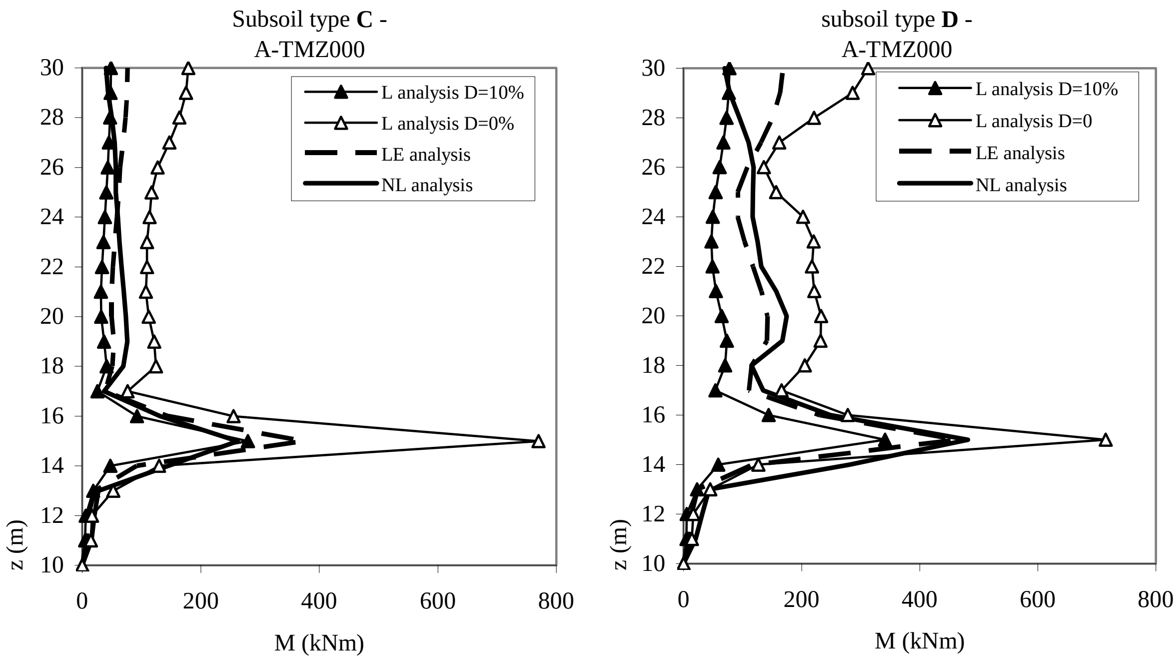

Figure 8 . Comparisons between linear, equivalent linear and non linear analyses

COMPARISONS WITH SIMPLIFIED APPROACHES

A number of simplified methods to calculate kinematic bending moments are available in the literature. Margason and Holloway (1977) method and NEHRP (1997) seismic provisions assume that the pile follow the free-field soil motion and derive the kinematic bending moments from the peak curvature, neglecting the interaction between pile and soil and several important parameters such as the pile-soil relative stiffness, pile slenderness, radiation damping. These methods are also inapplicable to inhomogeneous soils.

Dobry & O'Rourke (1983) developed a simple model for determining kinematic pile bending moments at the interface of two layers, modelling the pile as a beam on Winkler foundation and assuming that: (i) the soil in each layer is homogeneous, isotropic, and linearly elastic; (ii) both layers are thick enough so boundary effects outside the layers do not influence the response at the interface; (iii) the pile is long, vertical, and linearly elastic; (iv) perfect contacts exist between pile and soil; (v) the soil is subjected to a uniform static stress field, τ, which generates constant shear strain ($\gamma_1 = \tau_1/G_1$, $\gamma_2 = \tau_2/G_2$) within each layer; (vi) displacements are small. The explicit expression for the pile bending moment at the interface developed by the authors is:

$$M \simeq 1.86\left(E_p I_p\right)^{3/4} \cdot (G_1)^{1/4} \cdot \gamma_1 \cdot F \tag{5}$$

where G_1 is the soil shear modulus in the first layer, γ_1 is the soil shear strain at the interface that can be computed from a free-field response analysis or alternatively, if the maximum acceleration $a_{max,s}$ is specified at the soil surface, from the approximate expression of Seed and Idriss (1982):

$$\gamma_1 = \frac{r_d \rho_1 H_1 a_{max,s}}{G_1} \tag{6}$$

where ρ_1 is the density and H_1 the thickness of the upper soil layer and r_d=r_d (z) is the well-known depth factor, that for preliminary design purposes can be assumed:

$$r_d \simeq 1 - 0.015 \cdot z \tag{7}$$

in which z is the depth from the ground surface measured in meters. The parameter F of equation (5) is a dimensionless function of the ratio of the shear moduli in the two layers:

$$F = \frac{\left(1-c^{-4}\right)\left(1+c^3\right)}{\left(1+c\right)\left(c^{-1}+1+c+c^2\right)} \tag{8}$$

where:

$$c = \left(\frac{G_2}{G_1}\right)^{1/4} \tag{9}$$

Nikolaou and Gazetas (1997) and Nikolaou et al. (2001) derived two simplified expressions for kinematic pile bending moments at the interface of two soil layers modelling the pile as a beam on dynamic Winkler foundation and assuming that the soil in each layer is homogeous, isotropic, and linearly elastic, with constant soil damping ratio. The above mentioned expressions were derived from a comprehensive parametric study in layered soil profile subjected to harmonic steady-state excitation. The first expression is:

$$M_{\max} = 0.042 \cdot \tau_{\text{int}} d^3 \left(\frac{L}{d}\right)^{0.30} \left(\frac{E_p}{E_1}\right)^{0.65} \left(\frac{V_2}{V_1}\right)^{0.50} \tag{10}$$

where τ_{int} is the shear stress at the interface that can be expressed as a function of the free-field acceleration of the surface, $a_{max,s}$:

$$\tau_{\text{int}} = a_{\max,s} \rho_1 H_1 \tag{11}$$

The second expression is:

$$M_{\max} = \frac{2.7}{10^7} E_p d^3 \left(\frac{a_r}{g}\right) \cdot \left(\frac{L}{d}\right)^{1.30} \cdot \left(\frac{E_p}{E_1}\right)^{0.7} \cdot \left(\frac{V_2}{V_1}\right)^{0.30} \cdot \left(\frac{H_1}{L}\right)^{1.25} \tag{12}$$

where a_r is the maximum bedrock acceleration. Under transient seismic excitation the peak values of the bending moments, usually smaller than the steady-state amplitudes, can be computed using a reduction factor η which, according to the Authors, ranges between 0.15 and 0.50. Even if the authors recognizes the importance of soil damping, all the analyses were performed using a prefixed value of D=10% and no sensitive study was performed.
Another simplified method for predicting the kinematic bending moment at the interface between two layers was developed by Milonakis (2001). The assumptions are the same of the Dobry & O'Rourke model: the soil profile is constituted by two layers of homogeneous linear elastic soils, both layers are assumed to be thick. The improvements with reference to the Dobry & O'Rourke model are: (i) the seismic excitation is a harmonic horizontal displacement imposed at the bedrock; (ii) both radiaton and material damping are accounted for. The maximum bending moment can be compactly expressed as:

$$M = \frac{\left(E_p I_p\right) \cdot \left(\varepsilon_p / \gamma_1\right) \cdot \gamma 1}{r} \tag{13}$$

where γ_1 is the shear strain at the interface between two layers computed by the eq. (6) and ε_p/γ_1 is the strain trasmissibility function obtained from a complex procedure not described in the paper. For more detail of the procedure it is possible to refer to the paper of Milonakis (2001).

All the above mentioned simplified procedures do not consider the non linear behaviour of the soil. The methods of Dobry & O'Rourke (1983) and Milonakis (2001) can be directly used in the time domain performing a free-field response analysis to derive the valye of $a_{max,s}$. The results of the present study have been compared with the kinematic moments deduced using these two methods, with the maximum free-field accelerations obtained by linear and non-linear EERA analyses. The comparison among the kinematic bending moments obtained by the Dobry & O'Rourke and Milonakis methods and those deduced by linear and nonlinear VERSAT-P3D analyses are reported in tab.7 and fig.9.

Table 7. Comparisons with the simplified solutions of Dobry e O'Rourke and Milonakis (2001)

soil type	event	L analysis D=10%				NL analysis			
		$a_{max,s}/g$ EERA	M_{max} (kNm) Dobry & O'Rourke	M_{max} (kNm) Milonakis 2001	M_{max} (kNm) VERSAT-P3D	$a_{max,s}/g$ EERA	M_{max} (kNm) Dobry & O'Rourke	M_{max} (kNm) Milonakis 2001	M_{max} (kNm) VERSAT-P3D
D	A-TMZ000	0.48	361.13	424.89	341.12	0.49	370.22	435.59	481.71
D	A-STU270	0.61	458.51	539.47	384.87	0.46	350.67	412.59	994.03
D	E-NCB090	0.97	738.40	868.78	929.26	0.52	395.23	465.02	939.94
C	A-TMZ000	0.68	219.71	264.23	279.43	0.45	145.37	174.82	262.04
C	A-STU270	0.79	256.68	308.69	309.30	0.55	178.89	215.14	352.50
C	E-NCB090	0.96	310.66	373.61	459.93	0.46	148.52	178.61	503.62

The moments computed with the Milonakis procedure are always larger than those obtained by the Dobry & O'Rourke method. The comparison among the maximum moments calculated with VERSAT-P3D analysis and the corresponding moments calculated with the simplified approaches is quite satisfactory for linear analysis. For non linear analysis, the simplified methods leed to maximum kinematic bending moments much lower than the ones calculated with nonlinear VERSAT-P3d analysis and almost indipendent of the time-history acceleration input. This is due to the fact that the amplification of the free-field acceleration ($a_{max,s}/a_r$) in a nonlinear EERA analysis is smaller than that obtained by linear EERA analysis, while bending moments at the interface calculated by VERSAT-P3D nonlinear analysis are sometimes larger than those obtained with linear analysis.

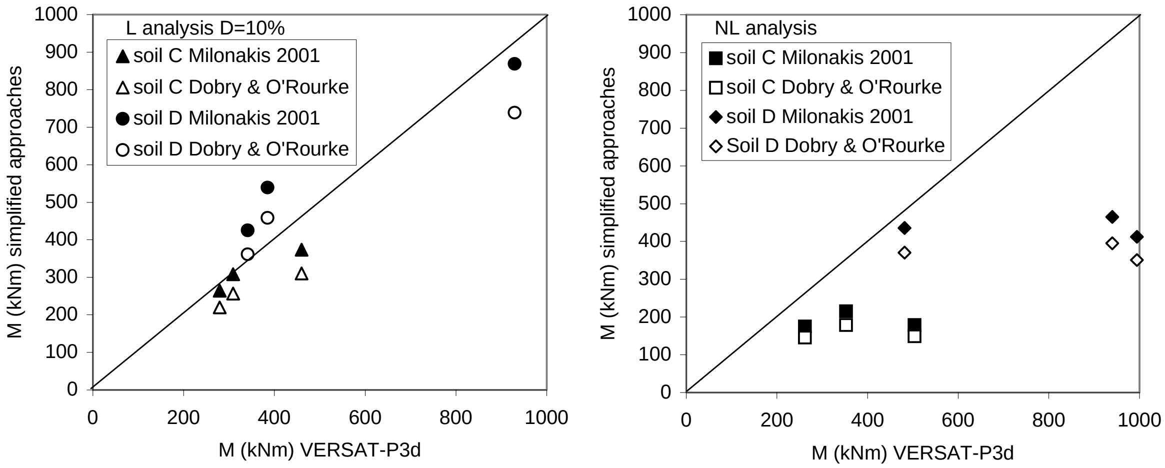

Figure 8 . Comparisons with Dobry & O'Rourke (1983) and Milonakis (2001) methods

CONCLUSIONS

In this paper, the results of a dynamic analysis of the kinematic interaction between a single pile and a two-layer subsoil due to real seismic input has been presented. The reinforced concrete pile (D=0.60 m and L= 20 m) has been always modelled as an elastic beam. Three different types of analysis have been performed:
- linear analyses with constant small strain shear modulus and damping of the soil (L analysis);
- non linear analyses with soil shear-strain dependent shear modulus $G(\gamma)/G_0$ and damping ratios $D(\gamma)$ (NL analysis);
- "equivalent linear" analyses, with constant soil shear modulus and damping deduced from previous non linear free-field response analyses (LE analysis).

The analyses have been performed in the time domain with three acceleration time histories selected from a database of records of Italian seismic events (Lanzo, 2006). The records have been scaled to values of bedrock acceleration a_r equal to 0.35g. The maximum kinematic bending moments at the interface of the two layers are compared with some simplified solutions.
The main results obtained are summarised in the following points.
- Bending moments are strongly influenced by the type of analyses.
- The seismic input has a significant influence on the values of bending moments.
- The linear analysis with no damping gives rise to very high bending moments, often higher than the range of possible yielding moments of the pile.
- The damping in linear analyses strongly affects the distribution and the maximum value of kinematic bending moments.
- Non linear analyses give rise to maximum bending moments sometimes higher and sometimes lower than those obtained with linear analyses performed with G= G_0 and D=10%.
- The bending moments obtained by the linear analysis with constant soil shear modulus and damping deduced from previous non linear free-field response analyses (LE analysis) is a good approximation of non linear analysis.
- The Dobry & O'Rourke (1983) and Milonakis (2001) simplified methods are able to give a good estimation of the maximum kinematic moments at the interface between two layers only for linear analysis.

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