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ACCOUNTING FOR GROUND MOTION DURATION IN THE NONLINEAR SOIL-PILE-STRUCTURE INTERACTION ANALYSIS OF EXTENDED PILE-SHAFT-SUPPORTED BRIDGES

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Abstract: *Bridge and viaduct structures supported on extended pile shafts are widely used for their economic and technical advantages. Meanwhile, the seismic response of structures supported on a monopile foundation is complex since the soil may experience large deformations, and hence exhibit strong nonlinear behaviour. Consequently, the pile-soil system would exhibit strong nonlinearity due to yielding of pile material and gapping at the pile-soil interface in addition to cyclic hardening/degradation of soil. This paper aims to evaluate the impact of the ground motion duration on the seismic performance of extended pile-shaft-supported bridges. A beam on nonlinear Winkler foundation model that is able to capture soil nonlinearities such as yielding, gapping, soil cave-in and cyclic hardening/degradation effects is used in the analysis. Incremental dynamic analyses are performed considering 4 real ground motion earthquake characterized by different durations to evaluate the effects of ground motion duration and soil nonlinearity on the performance of extended pile shafts. Various homogeneous soil profiles including saturated clay and sand in either dry or saturated state are considered. The results are presented in terms of incremental structural pseudo-acceleration curves and bending moments, and show that the ground motion duration may strongly affect the performance of bridges founded on soft degrading soils by increasing stresses within the extended pile-shaft because of a deeper point of virtual fixity than one obtained in non-degrading soils.*

Introduction

Single column bents supported on extended pile shafts are a type of bridges and viaduct structures characterized by a single circular pier column constructed by extending the pile foundation above the ground surface. Because of its efficient constructability, this system represents a cost-effective solution when compared to bridges on pile group foundations, and is widely employed for the support of highway and railway structures. Nevertheless, such system exhibits an important flexibility due to the foundation compliance that combined with its inherent minimal redundancy leads to a critical structural-geotechnical design. The ultimate flexural strength of the structure depends on the lateral restraint provided by the surrounding soil, hence, a dynamic soil-pile-structure interaction (SPSI) analysis is required to obtain a reliable design of single column bents supported on extended pile shaft. Several numerical soil-pile models with various levels of sophistication are available; an equivalent cantilever model concept was used by Chai (2002) to analyse the inelastic behaviour of the extended pile shaft, a classical beam on linear Winkler foundation model was used by Flores-Berrones and Whitman (1982) to analyse the seismic response of the supported mass on end-bearing piles, soil-pile models based on the Beam on Nonlinear Winkler Foundation (BNWF) such as in Boulanger et al., (1999), Allotey and El Naggar, (2008a), Gerolymos et al. (2009) have also been investigated. Additionally, 3D finite element models such as Trochanis et al., (1991), and Bentley and El Naggar (2000), have been devised to generate a continuous system.

Although the implications of soil-structure interaction effects on the onset of large lateral displacements are well recognized, there is still a need to evaluate the impact of the ground motion characteristics, such as the maximum amplitude, the frequency content and the duration, on the performance of the soil-pile system. In particular, the ground motion duration (GMD) has received more attention on its impact on the structural damage (see. e.g., Bommer et al., 2004, Chandramohan et al., 2016) than on the geotechnical failure (see e.g. Tombari et al., 2017,

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Khosravifar and Nasr, 2017, Song et al., 2018). In this paper, the performance of single column bents supported on extended pile shafts are evaluated through incremental dynamic analysis of ground motion events with various ground motion duration. Allotey and El Naggar's BNWF model (2008) is used to capture the dynamic nonlinear behaviour of hardening and degrading soils. Results demonstrating the effects of soil non-linearity and the duration of strong motion on the seismic behaviour are presented.

Numerical nonlinear soil-pile-structure model

The numerical soil-pile-structure model used to investigate the seismic response of extended pile shafts is depicted in Figure 1a. The lateral dynamic displacement, $v(t,z)$, of the whole system in which t is the time variable and z is the vertical spatial coordinate, is derived from the system of differential equations of a beam on nonlinear Winkler foundation subjected to the loading, $p(t,z)$, combination of static (e.g. lateral soil pressure) and dynamic load (e.g. seismic excitation) as follows:

$$\begin{cases} E_s I_s \dfrac{\partial^4 v}{\partial z^4} + c_s \dfrac{\partial v}{\partial t} + \mu_s \dfrac{\partial^2 v}{\partial t^2} = 0 \quad z > 0 \\ E_p I_p \dfrac{\partial^4 v}{\partial z^4} + \mu_p \dfrac{\partial^2 v}{\partial t^2} + c_p(v) \dfrac{\partial v}{\partial t} + k(v) v = p \quad z \leq 0 \end{cases} \tag{1}$$

where E_i is the Young's modulus, I_i is the second moment of area, μ_i is the mass per unit length, c_i is the damping for the extended pile-shaft, $i = s$, and for the pile $i = p$, respectively. Independent variables, t and z are omitted in Eq. (1) for sake of brevity. In this paper, the nonlinear soil-pile reaction modulus $k(v)$ as well as the tangent-stiffness proportional damping $c_p(v)$ are described through the Allotey and El Naggar (2008a) model. This dynamic BNWF model is a degrading polygonal hysteretic model that encompasses multilinear backbone curve with defined rules for loading, reloading and unloading in order to simulate synthesized generic cyclic normal force–displacement behaviour of sandy and clayey soil. Various parameters have to be set in order to govern the monotonic, cyclic and hardening/softening behaviour as described in the following sections.

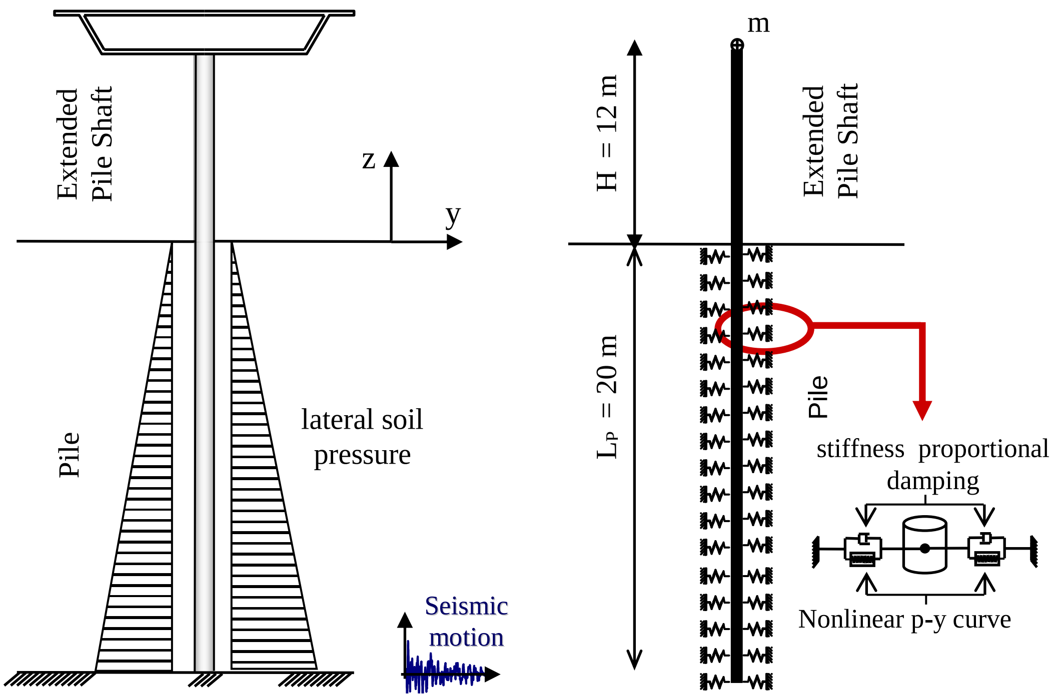

Figure 1. Extended Pile-Shaft supported bridge a) and b) numerical FE model.

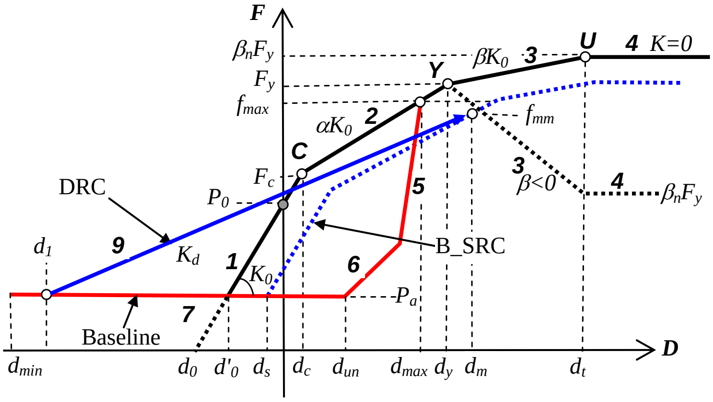

Figure 2. Generic cyclic normal force–displacement model (Allotey and El Naggar, 2008a)

Monotonic Behavior

The soil normal force-lateral displacement relation under monotonic static load, commonly referred to as p-y curve, is simulated by the multi-linear black-coloured curve (segments 1-4) depicted in Figure 2 through 6 parameters, i.e. the initial stiffness K_0, the normalized first turning point, F_c , the normalized ultimate force, β_n, ratio to the yield force, F_y, as well as the stiffness parameters, α and β, ratios of the stiffness of the second and third branch to the stiffness K_0, respectively. The initial stiffness K_0 is derived from the subgrade modulus, k_s, multiplied by the length of pile element, l_m. The subgrade modulus can be obtained by several methods (see e.g. Heidari and El Naggar, 2018). The other parameters, i.e. F_c, β_n, α and β, are determined through regression analysis techniques to best fit the four segment multi-linear curve to any given theoretical or experimental p-y curve.

Cyclic Behaviour

The cyclic behaviour is fully described by the definition of the unloading curve (segments 5-7 in Figure 2) and the reloading curve (segment 9 in Figure 2). Extended Masing rules are used to govern the shape of the unloading behaviour. If the unloading curve decreases the compressive lateral force to zero, the following reloading curve is termed as Direct Reload Curve (DRC), and it is modelled as a bilinear curve. The DRC is used to simulate the behaviour of the pile moving through a gap or a slack zone, namely, a weathered zone between the pile and the existing soil; it is governed by three parameters, the stiffness ratio of the first to second branch of the DRC, e_{p1}, the gap force parameter, p_1, the soil cave-in parameter, p_2. The stiffness ratio of the DRC, e_{p1}, governs the stiffness behaviour of the pile moving through the slack zone; therefore, it can control the shape of strain-hardening behaviour of the soil reaction to simulate the densification of the loose caved-in soil during the reloading. The stiffness ratio of the DRC, e_{p1} varies between $e_{p1} = 0$, i.e., pile moving into the soil gap, and, $e_{p1} = 1$, complete recovery of the mechanical characteristics during the reloading. The gap force parameter, p_1, simulates the recovery of the strength within the slack zone and it ranges from $p_1 = 0$, i.e. fully-unconfined behaviour, to $p_1 = 1$, fully-confined behaviour. Finally, the soil cave-in parameter, p_2, governs the soil cave-in phenomenon. It is used to estimate the starting displacement of the backbone curve; the parameter ranges as $0 \leq p_2 \leq \infty$, with $p_2 = 0$, meaning that no soil fall-in takes place (i.e. the backbone curve starts from the previous unload displacement point, while higher values lead to a full confined state with no-gap formation; usually, a numerical value of $p_2 = 5$ can be used to obtain a fully confined state which indicates the caved-in soil has the same mechanical properties of the surrounding soil and deterioration is not occurred.

Hardening/Softening Behavior

The cyclic curve previously defined can be progressively modified to consider the cyclic hardening/softening behavior. A generalized fatigue formulation (Allotey and El Naggar, 2008b) based on a phenomenological approach, characterized by uncoupling of the soil failure condition from the damage evolution function, is used. The failure condition curve aims to estimate the cumulative cyclic degradation/hardening at the beginning of each unload or reload; the degradation phenomenon is described by a conventional cyclic fatigue formulation through the Wöhler curve (or S-N curve) which describes the relation between the cyclic stress, S, and the number of current cycle, N, is required for evaluating the degradation/hardening response. The

S-N curve shows the number of cycles to reach failure ($N = N_f$) when a constant amplitude cyclic stress ratio is applied. The stress ratio is normalized in respect to the ultimate stress, S_u, to achieve failure ($S_u = S(N_f = 1)$). Cyclic triaxial tests or cyclic simple shear tests can be used to determine the S-N curve. In the Allotey and El Naggar (2008b) model, the failure condition curve is fully defined from the determination of the parameter k_s representing the slope of normalized S-N curve and by the parameter f_o indicating the soil stress corresponding to S_u, i.e. $f_o = S_u$. In this paper, f_o represents the peak strength of the monotonic behavior, usually defined as $f_o = F_u = \beta_n F_y$ for non-brittle soils. Moreover, the damage evolution function has to be determined. The damage evolution function describes the progressive degradation or hardening of stiffness and/or strength with the increase of the number of cycles under a constant stress ratio. Four parameters are required: p_s, and p_k, describe the strength and stiffness degradation factors, respectively, and e_s, and e_k, are strength and stiffness shape factors, respectively. It is worth mentioning that for values of p_s, and p_k smaller than one, cyclic degradation is accomplished, otherwise hardening is enforced. Furthermore, the strength and stiffness shape factors govern the shape of the damage evolution curve that may be concave ($e_{s,k} > 1$), linear ($e_{s,k} = 1$) or a convex ($e_{s,k} < 1$). Through these models, hardening and degradation behaviours can be simulated as shown in Figure 3a and Figure 3b, respectively. Further details are provided in Tombari et al. (2017).

Incremental Dynamic Analysis Methodology

Incremental Dynamic Analyses (IDAs) are carried out to investigate the effect of the soil nonlinearities and the impact of the Ground Motion Duration (GMD) on the seismic response of extended pile-shaft supported bridges considering soil-pile-structure interaction. A proposed three-step methodology consisting of i) iterative site response analysis, ii) soil-pile system calibration and iii) incremental soil-pile-structure interaction analysis is applied. Three types of soil, i.e. saturated clay and dry and saturated sand which characteristics are reported in Table 1, are investigated. The selection of these soil types aims to cover the largest possible range of soil-pile dynamic behaviour such as soil yielding, gapping, hysteretic loop shape and cyclic hardening/degradation. The proposed methodology is presented in the following sections.

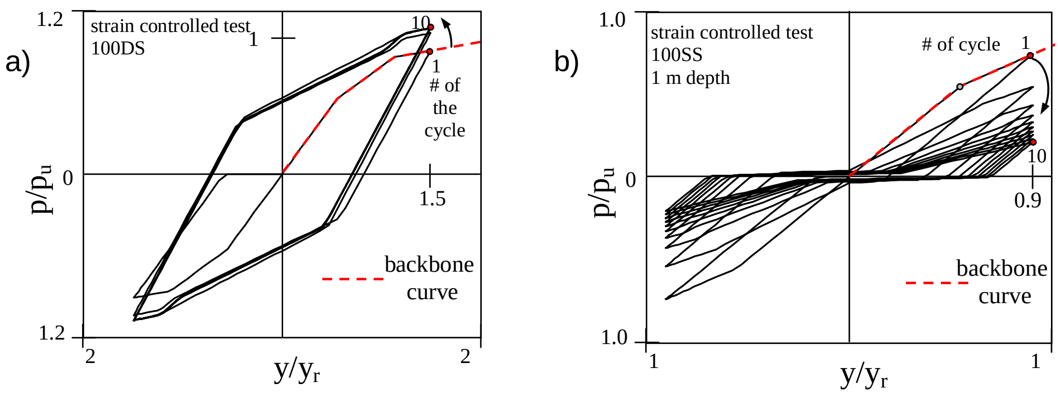

Figure 3. Strain-controlled unit cyclic test used in 100SS case for a) dry and b) saturated sand.

Soil	Soil type	Soil Consistency	D_r [%]	I_p	Vs [m/s]	γ [kN/m^3]	ν	φ [°]	Cu [kPa]
Toyoura sand	Dry sand	loose	35	/	100	14.22	0.3	30	/
Nevada Sand	Saturated sand	loose	42	/	100	19.65	0.3	33	/
Drammen Clay	Saturated clay	soft	/	27	100	15.5	0.45	/	30

Table 1 Soil profiles investigated in this paper

1st Step: Site Response Analysis

The 1st step of the proposed methodology consists of performing an iterative 1D linear-equivalent site response analysis in order to obtain the target Intensity Measure (IM), defined as the spectral pseudo-acceleration, Sa(T_{SSI}, 5%), at the fundamental period of the structure, T_{SSI}. Thus, scale factors are applied to the ground motion record defined at the bedrock outcropping and iteratively adjusted until the IM converges to the target value. This proposed method allows generating the same pseudo-acceleration and hence, shear stresses at the base of the bridge pier regardless of its structural and geotechnical characteristics if the system remains linear elastic; therefore, this approach allows to assess the onset of the nonlinear soil-structure interaction effects. Furthermore, this proposed method generates ground motion signals that are compatible with the soil deposit characteristics. Alternative approaches such as those based on post-scaling of the ground motion at the surface, are able to modify only the amplitude but not its frequency content. It is worth pointing out that with the onset of the soil-pile nonlinearities, the actual pseudo-acceleration experienced by the bridge will deviate from the target IM, differentiating between linear and nonlinear models.

2nd Step: Soil-Pile Parameters Calibration

In the 2nd step, the parameters of soil-pile model (Allotey and El Naggar, 2008a) are evaluated by using conventional geotechnical testing. Parameters that govern the monotonic, cyclic and hardening/softening behaviour have to be defined. The backbone curve or initial p-y curve, which simulates the monotonic behaviour, is obtained by fitting the reference monotonic curve recommended by American Petroleum Institute (API) for sand or clay. In order to simulate the cyclic behaviour, the following heuristic approach is adopted. In loose dry sand, the pile manifests a fully confined response due to a significant cave-in of the adjacent soil. An oval-shaped hysteresis loop is hence observed as depicted in Figure 3a. In order to simulate this hysteretic behaviour, the following parameters should be used in the soil-pile model: DRC stiffness ratio, $e_{p1} = 1$, the gap force parameter, $p_1 = 1$ and, the soil cave-in parameter, $p_2 = 1$. On the other hand, in saturated sand, the cyclic behaviour under undrained shearing manifests a degradation in the slack zone; therefore, on the upper part of the pile, a S-shaped hysteresis loop is expected due to combined effect of the low confining pressure with undrained shearing degradation as depicted in Figure 3b whereas a full oval-shaped hysteresis loop (e.g. see Figure 3a) is assumed to occur within the lower part of the pile, below five pile diameters from the top to the bottom of the pile, where the confining pressure is high. A linear transition between the two types of behaviour is simulated since the effect of the soil cave-in is expected to gradually enhance the soil-pile response with the increase of the depth. Same behaviour is simulated for soft clay soils. Finally the hardening/softening behaviour is calibrated from cyclic loading torsional shear tests or cyclic triaxial tests. The hardening/softening model is fully defined by the cyclic damage evolution curve that is the relation between the hardening/degradation parameter and the current number of cycles and by the cumulative damage curve, namely the S-N curve used in fatigue problems, to correlate the applied stress to the number of cycles leading to failure. An elliptical damage model as defined by Allotey and El Naggar, (2008b) is used. In dry sand, a cyclic hardening behaviour is expected. For instance, data elaborated from the results obtained by Lo Presti et al., (2000) on dry Toyoura sand show an increment of the stiffness value and ultimate strength of about 20%. Therefore, stiffness hardening factor of $p_k = 1.2$, and strength hardening parameter, $p_s = 1.2$, is applied along with stiffness curve shape $e_k = 2$, strength curve shape $e_s = 2$, and slope of normalized S-N curve, $k_s = 0.1$. For saturated soil, a softening behavior is expected. Results elaborated from centrifuge testing performed by Wilson (1998) with Nevada Sand show that for large number of cycles both strength and stiffness is reduced to 10% of the initial stiffness and strength, defined by the degradation factor, $p_k = 0.1$, and $p_s = 0.1$ for stiffness and strength, respectively. The cyclic damage evolution curve is slightly concave, hence there is a decreasing rate of damage, and it is defined by stiffness curve shape parameter $e_k = 0.9$ and strength curve shape parameter$e_s = 0.9$. For the soft clay, the cyclic stiffness degradation in cohesive soil is more marked than that of strength. Therefore, the stiffness degradation curve is convex (decreasing rate of damage) whereas the strength degradation curve is concave (increasing rate of damage). The strength degradation parameter is obtained from the tests performed by Yasuhara (1994) for Drammen Clay. The minimum amount of degradation is assumed as 70% of the initial stiffness, $p_k = 0.7$, and as 76% the initial strength, $p_s = 0.76$, since stabilization (shakedown) of the hysteretic behavior occurs.

3rd Step: Soil-Pile-Structure Interaction Analysis

This step involves the soil-pile-structure interaction (SPSI) analysis. A direct approach through finite element modelling the bridge structure and of the Allotey and El Naggar (2008a)'s soil-pile system as depicted in Figure 1b, is used. The bridge under consideration is a single column concrete bent bridge supported on large-diameter extended cast-in-place steel shells pile shaft, classified as the Type I shaft according to Caltrans (California Department of Transportation) with the above ground cross-section (i.e. the column) of the same 1.5-meter diameter of the below-ground cross-section (i.e. the pile). The concrete is characterized by a Young's modulus (E) of 2.25×10^7 kPa typical of cracked section and by a mass density (ρ) of 2.5Mg/m^3 and it is used for both the pile and the column. The length of the end-bearing pile ($\mathrm{L_p}$) is 20 meters and the pier height is 12m. The deck is simulated as a 115 Mg mass lumped at the top node, which neglects the transverse stiffness of the deck by considering that the structure is far from the abutment (Gerolymos et al., 2009). The fundamental period of the bridge considering linear soil compliance of the foundation is $\mathrm{T_{SSI}} = 1.2s$.The ground motion along the soil profile, obtained in the first step, is applied as accelerations for each lateral spring located at various pile depths. Dynamic time history analysis with implicit Hilber-Hughes-Taylor time integration scheme is used. A tangent-stiffness proportional damping with coefficient, $\beta = 0.038$, calibrated for obtaining 10% damping ratio at the fundamental period of the structure.

	Parameters		Dry Sand	Saturated Sand	Soft Clay
Backbone Curve	Initial stiffness K_0		18844	26039.76	22910.3
	1st turning point ratio F_c		0.64	0.64	0.417
	Ult. soil strength ratio β_n		1.16	1.16	1.39
	Stiffness ratio 2nd br. α		0.54	0.54	var.
	Stiffness ratio 3rd br. β		0.13	0.13	var.
Cyclic curve parameters	< L/3	soil cave-in p_2	5	0:lin:5	0:lin:5
	> L/3		5	5	5
	< L/3	DRC stiffness ratio e_{p1}	1	0:lin:1	0:lin:1
	> L/3		1	1	1
	gap force p_1		1	1	1
	unloading stiffness factor		1	1	1
Degradation parameters	stiffness degr p_k		1.2	0.1	0.7
	strength degr. p_s		1.2	0.1	0.76
	stiffness curve shape e_k		2	0.9	2.5
	strength curve shape e_s		2	0.9	0.7
	slope of the S-N curve		0.1	0.5	0.215
	cyc. stress rate at N=1 S		1	1	1

Table 2 Soil-pile model parameters for saturated clayey soil deposits

Event	Data	Station	Record	T [sec]	I_D
San Fernando	*09/02/1971 14:00*	994 Gormon-Oso Pump Plant	SFERN/OPP270	9.22	**4.4**
Loma Prieta	*18/10/1989 00:05*	57066 Agnews State Hospital	LOMPA/AGW000	39.95	**6.6**
Chi Chi	*20/09/1999*	CHYO50	CHICHI/CHY050-N	89.95	**14.3**
Imperial Valley	*15/10/1979 23:16*	6605 Delta	IMPVALL/H-DLT262	99.92	**24.2**

Table 3 Earthquakes data of the ground motion records adopted in the work

Results

IDA is performed to evaluate the effect of the nonlinearities at 4 levels of intensity varying between 0.1g to 0.6g. 4 ground motion events, reported in Table 3, are selected to cover different scenarios ranging from small to large duration, according to the integral parameter I_D, proposed by Cosenza and Manfredi (1997), which value increases with the increase of the GMD. The proposed three step methodology is applied with the aim to investigate the impact of the ground motion and of the soil nonlinearities on the performance of the bridge. Results in terms of incremental pseudo-acceleration curves, bending moment envelopes, as well as the lateral dynamic behaviour of the soil-pile interaction, here called dynamic P-Y curves, are presented.

Incremental Pseudo-Acceleration Curves

The pseudo-acceleration, a_p, is defined as the ratio of the maximum base shear force (V) at the base of the column to the effective mass of the system, m. Figure 4 shows the pseudo-acceleration curves obtained from the IDA as a function of intensity measure, IM, for every considered earthquake. The grey dash-dot line in Figure 4 represents the reference line for the correlation between the IM with the expected pseudo-acceleration of a linear fixed base (FB) model with period T_{SSI}. Because of the procedure proposed for the first step of site response analysis, any deviations from the reference line is attributed to the onset of soil nonlinearities, such as material yielding, gap formation and cyclic hardening/degrading behaviour. It is worth noting that in Figure 4, the pseudo-acceleration curves of the saturated and dry sand cases are nearly overlapping for the short scenarios represented by the events of San Fernando Earthquake and Loma Prieta Earthquake. Conversely, longer scenarios as those determined from the events of Chi Chi Earthquake and Imperial Valley Earthquake, the cases of saturated and dry sand behave differently also for low intensity measures. These observations demonstrate that the GMD strongly affects the overall structural response as it promotes different cyclic behaviour of dry sand (i.e. hardening) and saturated sand (i.e. softening/degradation). Cases related to clay soil, due to its higher linear threshold, manifest smaller soil deformations and the pseudo-acceleration curve is quite similar to the linear reference curve that slightly overestimates the actual response.

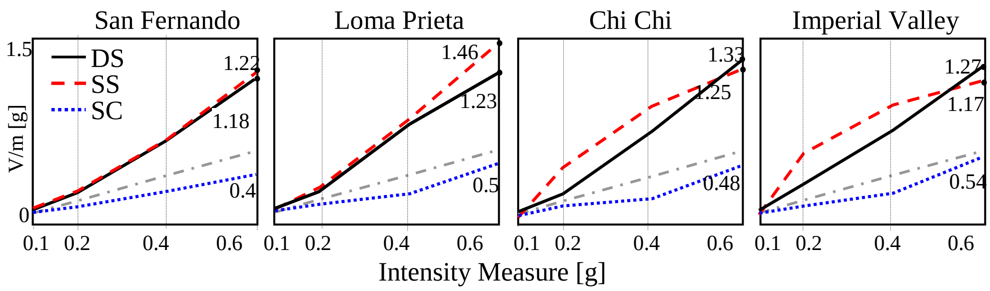

Figure 4. Pseudo-acceleration curves for the various duration scenarios and soil types: dry sand (DS), saturated sand (SS), and saturated clay (SC).

Dynamic P-Y Curves

The soil lateral confinement influence the overall dynamic behaviour of the bridge in terms of equivalent natural period, damping and transmission of seismic waves from the soil to the pile (e.g. see Tombari et al., 2017). The lateral confinement is represented by both static soil pressure acting on the pile and inertial and kinematic interaction between pile and soil. In Figure 5, the lateral confinement is evaluated in terms of hysteresis loops of the pile-soil system, namely dynamic P-Y curves for three different depths (z=-2.5m, z=-5m and z=-10m). At shallow depths, an oval-shaped curve is obtained until the ultimate soil-pile capacity is reached; at 10m-depth, the high confining pressure along with the smaller deformations of the pile maintained the linearity of the soil-pile system. In order to evaluate the GMD impact, Figure 6 shows the dynamic P-Y curves at 10m-depth for the case of saturated sand soil for two different GMDs. Remarkably, an almost linear behaviour is obtained for the short duration scenario of San Fernando earthquake, while an S-shaped hysteresis loop is produced during the long duration scenario of Imperial Valley earthquake. Therefore, for the same soil, two different behaviours are obtained.

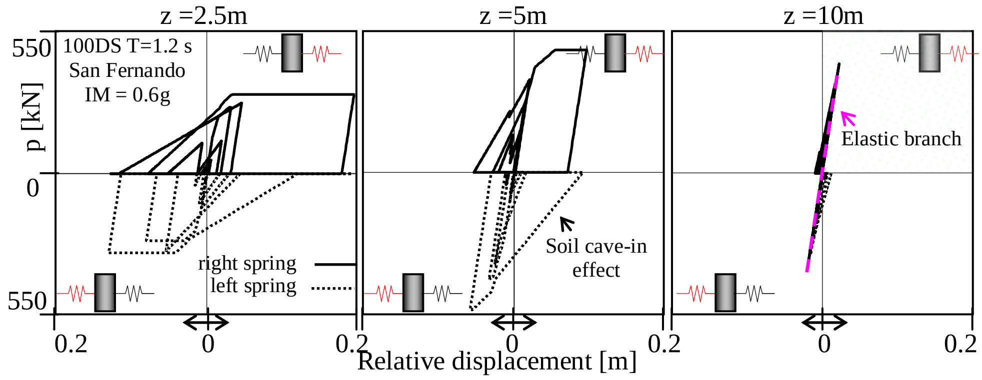

Figure 5. Dynamic P-Y curves of the soil-pile system at various depths.

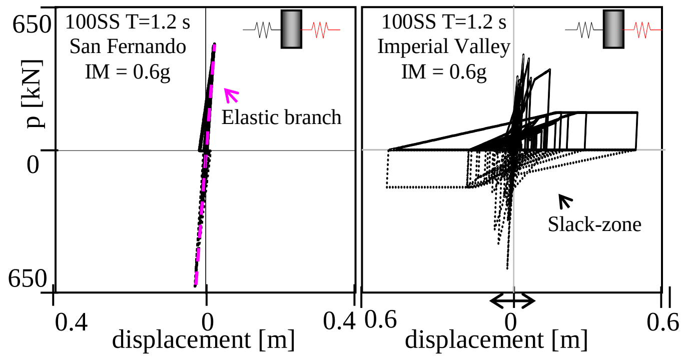

Figure 6. Dynamic P-Y curves at 10m-depth for a) short duration and b) long duration scenario

Incremental Bending Moment Envelopes

The bending moment envelopes obtained for every investigated case are depicted in Figure 7. For short scenarios, i.e. San Fernando earthquake and Loma Prieta earthquake, the envelopes of the global bending moment for saturated and dry sandy deposits are similar, especially for low-medium intensities. As observed earlier, for short GMD, the cyclic behavior of soil does not affect greatly the seismic response. On the other hand, soil yielding causes larger bending moments with a trend similar to that observed for the pseudo-acceleration curves; consequently, the soil-pile response in soft clay manifests smaller displacements than the one occurred in loose sand because of the nonlinear SSI. Conversely, for long GMD earthquakes, such as in the Chi-Chi earthquake and San Fernando earthquake event, a marked difference between soil-pile response in dry and saturated sand is observed, even for low intensity levels. It is worth emphasizing that because of the proposed procedure to generate the free field motion, the pseudo-acceleration, and hence, the shear strain, is similar, the maximum bending moment of piles in saturated sandy soils is higher than the ones in dry sands. This is due to the deeper point of fixity generated by the formation of the slack zone in the upper part of the pile exacerbated by the cyclic degrading behavior of the in saturated sandy soil. Therefore, for long GMD the cyclic degradation is an important phenomenon and cannot be neglected for a reliable assessment of the seismic performance of extended-pier shaft supported bridges. In clayey soils, although the free field deformations are smaller than the case of sandy deposits because of higher linear threshold at small strains, the ultimate soil strength of the soft clay is smaller than that of the sand. Therefore, for large strains caused by the soil-pile-structure interaction, the yielding of the clay is developed for larger depths along the shaft. The yielding causes a redistribution of strains in the plastic range of the deformation and the pile shear stress tends to be uniform within the pile.

Concluding Remarks

The effects on the soil nonlinearities with particular emphasis on the ground motion duration have been analysed on the seismic performance of single column bents supported on extended pile shafts. Incremental Dynamic Analyses have been performed considering 4 real ground motion earthquake with increasing duration.

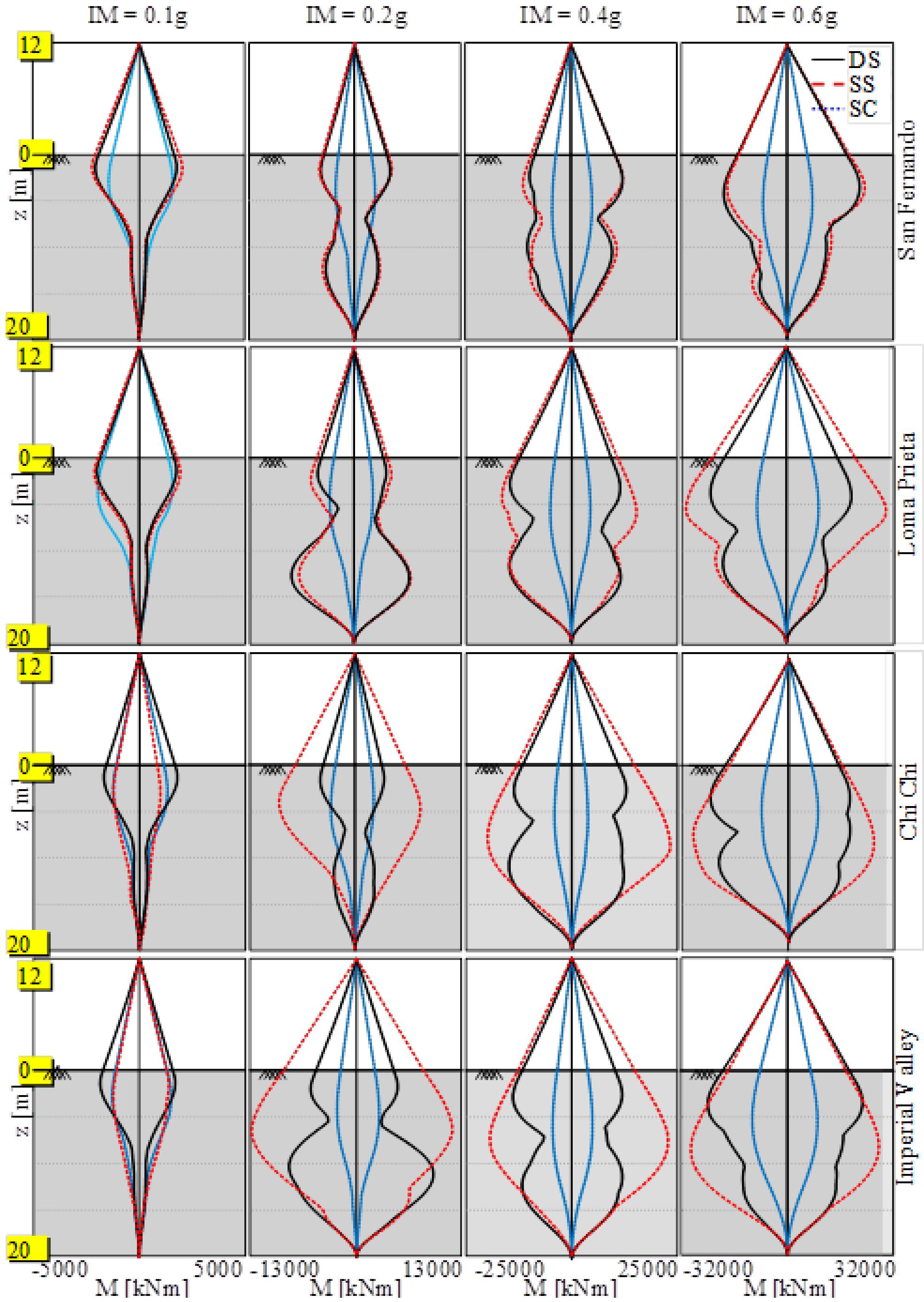

Figure 7. Envelopes of the bending moment for various earthquake events and soil types: dry sand (DS), saturated sand (SS), and saturated clay (SC).

.An iterative site response procedure is proposed to generate the same structural pseudo-acceleration regardless the soil type. Three types of soil, i.e. saturated clay and dry and saturated sand have been selected to cover the various soil-pile dynamic behaviours such as soil yielding, gapping, hysteretic loop shape and cyclic hardening/degradation, modelled through the Allotey and El Naggar's BNWF model (2008a). Based on the results of the analyses, the following conclusions can be drawn:

- Linear or equivalent linear soil pile models can be used in case of earthquakes with short duration scenarios.
- Long duration earthquakes strongly affect the cyclic hardening/degradation of the soil-pile system and soil nonlinearity must be considered for a reliable design.

- With degrading soils, the point of virtual fixity is deeper, leading to higher bending moments within the pile than in non-degrading soils.

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