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INFLUENCE OF NON LINEAR SOIL STRUCTURE INTERACTION ON THE SEISMIC DEMAND IN BRIDGES

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ABSTRACT: Examination of the seismic behavior of bridge foundations during earthquake points out that even at low shaking level permanent displacements, i.e. nonlinear soil structure interaction, takes place. Nonlinear soil structure interaction is usually not considered in design although it may have a beneficial effect as illustrated in this paper. Results of incremental dynamic analyses (IDA) of a simple structural bridge pier for a fixed base system or the same system with consideration of linear and nonlinear soil structure interaction, including uplift and soil plasticity, are presented. The results highlight the beneficial role of foundation nonlinearities in decreasing the ductility demand in the superstructure but point out the need to carefully assess the variability of the response when non linearity is allowed at the foundation design.

KEY WORDS: Soil structure interaction, nonlinearity, ductility demand, incremental dynamic analyses.

1 INTRODUCTION

The topic of soil structure interaction (SSI) has long been recognized as a major factor controlling the design of the structure. During an earthquake, the soil deforms under the influence of incident seismic waves and imposes its motions to the foundation and to the supported structure. In turn, the induced motion of the foundation creates inertial forces in the superstructure that are transmitted back to the foundation and to the underlying soil. Therefore, the induced deformations create additional waves that emanate from the soil-foundation interface. Both phenomena occur simultaneously and therefore are closely linked and dependent on one another. SSI increases in significance as the supporting soil becomes softer. Although recognized by recent building codes, like Eurocode 8, [1], which requires to take into consideration SSI for massive structures founded on soft deposits, most building codes ignore the effect of SSI: "For the majority of usual building structures, the effects of SSI tend to be beneficial, since they reduce the bending moments and shear forces acting in the various members of the super-structure", [1]. As pointed in [2] this statement may hold for a large class of structures but may be misleading for others and mainly relies on the smooth shape of normalized code spectra. To the best, SSI is considered in the dynamic analysis assuming a linear behavior of the soil foundation interface. Even though, the results obtained by various authors on the beneficial or detrimental effect of SSI are controversial. Within the framework of performance based design, the question becomes essential to know how nonlinear soil structure interaction may affect the seismic demand in the superstructure. With the advance of efficient numerical tools to model the nonlinear behavior of foundations, it becomes possible to investigate this effect through the concept of incremental dynamic analysis (IDA) as defined by Vamvatsikos & Cornell, [3].

The paper presents one example of observed nonlinear soil structure interaction on the foundations of the Rion Antirion Bridge during a moderate earthquake, highlighting the fact that, although such a phenomenon is not usually considered in design, it may exist. Furthermore, preliminary results obtained through IDA show that this effect may be beneficial for protecting the superstructure by reducing the ductility demand in the pier and suggest alternatives for the seismic design of foundations. For that purpose, the studied structure is a reinforced concrete bridge pylon founded on the surface of a homogeneous cohesive soil by means of a circular footing.

2 ILLUSTRATION OF NONLINEAR SOIL STRUCTURE INTERACTION

The Rion Antirion bridge, a five-span cable stay bridge (286m+560m+560m+560m+286m), has been designed to withstand earthquakes with p.g.a. of 0.48g and tectonic movements up to 2m between consecutive pylons. The soil profile consists of deep alluvial deposits with poor mechanical characteristics. The piers are founded on shallow foundations resting on reinforced soil, [4], and were designed taking into account nonlinear soils structure interaction for the extreme (2000 year return period) design earthquake; this implies that uplift, sliding and even partial soil yielding would occur for that event. However, for moderate earthquakes, with return periods typically of the order of 100 to 500 years, linear soil structure interaction was considered.

The bridge was hit by the Achaia-Ilia earthquake that took place on June 08, 2008. The epicenter of this earthquake with moment magnitude Mw = 6.5 was located at a distance of approximately 36km SW from the bridge and its focal depth was estimated to around 30km (Figure 1). Examination of available seismological data recorded during the main shock and the aftershocks indicated that the earthquake occurred on a dextral strike slip fault. The peak ground acceleration recorded on site (Rion shore) was 0.127g, significantly smaller that the design earthquake. This was the first major earthquake event experienced by the bridge initiating full scale inspection in order to identify potential damages of the structure. Given that tectonic movements might take place at this site, a geometrical survey was conducted to monitor permanent movements due to the event.

Figure 1. Epicenter and bridge site

Interpretation of the measured values of the ground motion parameters (p.g.a, Arias Intensity and Cumulative Absolute Velocity) leads to the conclusion that the return period of the earthquake is in-between 80 to 135 years. This observation complies with the calculated spectra at the banks and the feet of the bridge pylons that were shown to be in the range of or slightly smaller than the 120 year period design earthquake. For such a small event nonlinear soil structure interaction would never been considered in design. Nevertheless, monitoring of the foundations permanent movements after the earthquake clearly show that permanent movement (settlements) of the foundation took place, an evidence of nonlinear foundation behaviour.

After the earthquake a complete geometrical monitoring was conducted in order to check if tectonic movements or settlements had occurred during the earthquake, [5]. It is important to mention that just before the earthquake the scheduled geometrical monitoring campaign had been completed for planimetric and leveling measurements on Rion and Antirion shores. The comparison of measurements before and after the earthquake does not show significant movements. Relative altitudes on both shores remain very consistent with their pre- earthquake values. The maximum earthquake induced settlement was measured at M1 pier and is 21 mm. At the other piers the settlements range between 0mm and 16mm. In plane no important displacements that can be attributed to the earthquake have been observed. The settlements, although small and representing less than 10% of the total settlements experienced since the beginning of construction, are however large enough to draw attention on the nonlinear foundation behaviour.

The lesson drawn from these observations is that even if engineers would attempt to prevent nonlinear foundation behavior and request, following usual practice, that they remain essentially elastic during earthquake, nonlinearities will nevertheless occur. The obvious question, to which the remaining of the paper will attempt to bring partial answers, is to know whether they are beneficial or detrimental.

3 SOIL STRUCTURE INTERACTION

Despite the fact that SSI is not very often considered in building codes, it has a long history which started back in 1936 with the work of Reissner. Since then, several improvements have been achieved and the present state of the art is well developed and understood.

3.1 Linear soil structure interaction

Several modeling techniques are available to account for SSI in the dynamic analysis. The most sophisticated ones are based on finite element analyses in which the supporting medium is explicitly modeled as a continuum. This technique is very demanding, both in computer time and manpower, and is not very efficient at early design stages of a project. Therefore, a substructure approach is often preferred in which all the degrees of freedom of the supporting medium are lumped at the soil-foundation interface in the form of dynamic impedances that can be viewed as frequency-dependent springs and dashpots, [6]. If those impedances are assumed frequency-independent or if simple rheological models are used the analysis is rendered very attractive and efficient. Such simple models can therefore be implemented to analyze the impact of linear soil structure interaction on the seismic response of structures.

A recent very comprehensive study, [7], has investigated the effects of soil-shallow foundation-structure interaction on the seismic response of structures using Monte Carlo simulations. The structure was modeled as a nonlinear one degree of freedom system and SSI was taken into account with conventional springs and dashpots; in other words, SSI was treated as a linear phenomenon. The authors concluded that SSI effects on the median response of a structure exhibiting a nonlinear behavior is relatively small; however there is a 30-50% probability for an increase in the total structural displacement of more than 10% due to SSI. Therefore, based on these results, SSI does not seem to be a major issue, at least in terms of median response. However, one may wonder how much these conclusions are influenced by the initial modeling assumptions regarding SSI.

3.2 Nonlinear soil structure interaction

More than 30 years ago the earthquake engineering community realized that the increase of strength of a structural system does not necessarily enhance its safety. This recognition has led to the development of new design principles, aiming at rationally controlling seismic damage and rendering the structure "fail-safe". This concept is embedded in the capacity design philosophy which is widely implemented in structural design, but is given less attention in geotechnical engineering. Even when foundation compliance is taken into account, little care is given to the nonlinearity of soil and foundation. Such an approach may lead to non-conservative oversimplifications, especially in the case of strong geometric nonlinearities, such as foundation uplifting and sliding. Most importantly, neglecting such phenomena prohibits the exploitation of strongly non-linear energy dissipating mechanisms in case of occurrence of ground motions larger than design. Today, a growing body of evidence suggests that soil-foundation plastic yielding under seismic excitation is not only unavoidable, but may even be beneficial, [8] to [13]. Such evidences has even led some authors to make the proposal of totally reversing the foundation design philosophy by allowing significant yielding in the foundation to protect the structure, [8].

However, implementation of a design philosophy in which, even partial, yielding is allowed at the foundation level requires that efficient and reliable tools be available for design. Nonlinear structural analyses are very sensitive to small changes in the structural properties and in the input motion. Obviously, the situation is even worse in foundation engineering where the properties of the soil are never known with a great accuracy. A safe design will therefore require a large amount of analyses to be run and this can hardly be efficiently achieved with heavy, although rigorous, numerical models such as finite element models. The concept of dynamic macroelements, developed over the last decade, offers a unique opportunity to evaluate the effect of nonlinear soil structure interaction on the response of a yielding structure.

Advantage of macroelement modelling is used in this paper, to examine the effect of nonlinear soil structure interaction on the response of a yielding structure. This is performed with a series of Incremental Dynamic Analyses (IDA). The results are further compared to analyses with linear SSI and without SSI (fixed-base structure) to highlight the changes in behaviour of the structure when SSI is accounted for either with a linear assumption or with a nonlinear one.

4 ILLUSTRATIVE EXAMPLE

4.1 Description of physical model

The studied structure is depicted in Figure 2; it represents a typical highway bridge pier under seismic excitation. The deck of mass md is monolithically connected to the reinforced concrete circular column of diameter d and height h. The pier is founded on a relatively stiff homogeneous clay stratum by means of a shallow circular foundation of height h_f and diameter D. Separation (uplift) and no sliding are allowed along the soil-footing interface. The system is subjected to seismic loading only along the transverse (with respect to the bridge axis) horizontal direction.

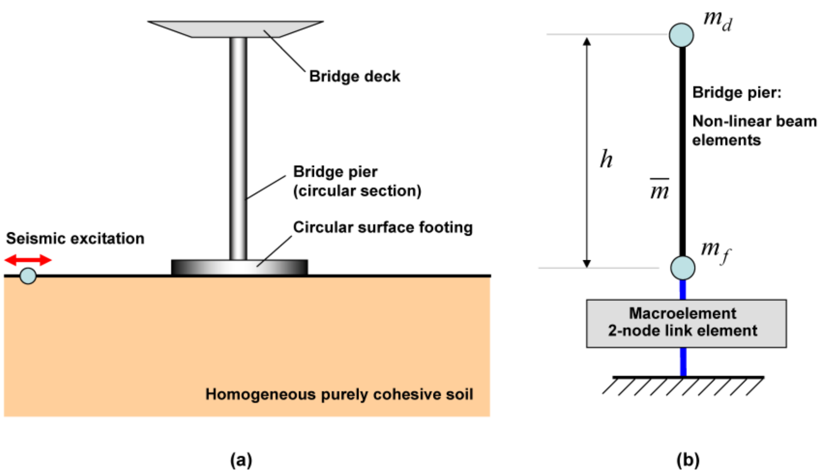

Figure 2. Soil-foundation-structure system (a) physical, (b) model

A direct displacement-based design procedure (DDBD), [14], appropriately modified to take into account soil-structure interaction effects, has been implemented for the pier design. The procedure is detailed in [15]. The design of the bridge pylon has been performed considering a seismic input represented by the Eurocode 8 design spectrum, Type 1, with firm soil conditions and a peak ground acceleration $a_g = 0.5g$. The following design performance criteria have been defined:

– System drift limit $\Delta_d = 0.03h$.
– Maximum foundation rotation $\theta_{lim} = 0.01$.
– Maximum structure ductility demand $\mu_{lim} = 3.2$.

The bridge pier is modeled with non-linear beam elements. The foundation and the soil are replaced by one unique 2-node link element, which is the non-linear dynamic macroelement for shallow foundations as developed in [16] and [17]. The first node of the macroelement is attached to the superstructure. The mass of the foundation is lumped at this node; the input motion is applied at the second node. The constitutive behavior of the macroelement reproduces the non-linear phenomena arising at the soil-footing interface: elastoplastic soil behavior leading to irreversible foundation displacements, possibility for the

footing to get detached from the soil (foundation uplift). Additionally, the macroelement is coupled with a viscous dashpot reproducing radiation damping.
The numerical parameters defining the problem are given in Table 1.

Table 1. Properties of the soil-structure system

Physical quantity	**Symbol**	**Unit**	**Value**
Mass of deck	m_d	kt	0.973
Column height	h	m	20.
Column diameter	d	m	2.5
Column mass	m_c	kt	0.245
Concrete compression strength	f_c	MPa	30.
Steel yield strength	f_y	MPa	400.
Number of longitudinal rebar	n	-	100
Diameter of longitudinal rebar	d_{long}	mm	26
Diameter of transverse rebar	d_{trans}	mm	12
Spacing of transverse rebar	s	mm	70
Foundation diameter	D	m	7.5
Foundation height	h_f	m	2.0
Foundation mass	m_f	kt	0.221
Total weight of structure	W_{tot}	MN	14.12
Soil undrained shear strength	c_u	MPa	0.15
Soil shear modulus	G_s	MPa	104.
Soil shear wave velocity	V_s	m/s	255.
Static bearing capacity factor	FS	-	2.84
Fixed base period of structure	T_0	s	1.379
Period for structure with SSI	T_{SSI}	s	1.650

4.2 Summary description of the macroelement

Several macroelement models have been developed during the last decade to account for nonlinear soil structure interaction. A comprehensive review of the existing models is presented in [18]. The model used in the present study is detailed in [16] and [17]. Only the main features are recalled hereafter.

The model comprises three non-linear mechanisms: a) a mechanism of sliding at the soil-footing interface, b) a mechanism of soil yielding in the vicinity of the footing and c) a mechanism of uplift as the footing may get detached from the soil. The first two are irreversible and dissipative and are combined within a multi-mechanism plastic formulation, [19]. The third mechanism is reversible and non-dissipative. It is reproduced with a phenomenological non-linear elastic model. Each non-linear mechanism participating in the global response of the system is modeled independently and the surface of ultimate loads is retrieved as the combined result of all active mechanisms. This allows formulating each mechanism by respecting its particular characteristics and offers the possibility of activating, modifying or deactivating each mechanism, an option that will be used in this paper.

4.3 Model parameters

The model parameters are listed in Table 2. Derivation of those parameters is briefly commented below.

4.3.1 Viscoelastic parameters

They are determined using the classical impedance functions for a circular footing on a halfspace, [20].

4.3.2 Bounding surface parameters

The ultimate vertical force for a centred load is given by the ultimate bearing capacity of a circular footing on a cohesive soil $N_{\max} = 6.05c_uA$ where c_u is the soil undrained shear strength and A the footing area. The parameters ψ and ξ are given by $\psi = V_{\max}/N_{\max}$ and $\xi = M_{\max}/DN_{\max}$. The ultimate shear force and overturning moment for a perfectly bonded footing are given by $V_{\max} = c_uA$ and $M_{\max} = 0.67c_uAD$.

4.3.3 Plasticity model parameters

These are the only parameters (h_0, p, p_g) that require a calibration from a 3D static finite element model. The numerical parameters h_0, p are chosen to reproduce the soil hardening behavior in diagrams of vertical force versus vertical displacement and diagrams of horizontal force versus horizontal displacement. The numerical parameter p_g is calibrated to fit the accumulated vertical settlement during the loading phase under the horizontal force.

Table 2. Model parameters

Parameter description	Symbol	Unit	Value
Footing diameter	D	m	7.50
Ultimate vertical force	$N_{\max}$	MN	40.09
Ultimate horizontal force	$V_{\max}$	MN	6.63
Ultimate moment	$M_{\max}$	MN.m	33.30
Bounding surface parameter	ψ	-	0.17
Bounding surface parameter	ξ	-	0.11
Vertical elastic stiffness	K_{NN}	MN/m	2225
Horizontal elastic stiffness	K_{VV}	MN/m	1833
Rocking elastic stiffness	K_{MM}	MN.m	20862
Vertical dashpot coefficient	C_{NN}	MN.s/m	27.8
Horizontal dashpot coefficient	C_{VV}	MN.s/m	18.0

Rocking dashpot coefficient	C_{NN}	MN.m.s	4.8
Plastic parameter (initial loading)	h_0/K_{NN}	-	4.0
Plastic parameter (reloading)	p	-	0.5
Non-associative parameter	p_g	-	5.0
Uplift initiation parameter	α	-	6.0
Uplift parameter	γ	-	2.0
Uplift parameter	δ	-	0.5
Uplift parameter	ε	-	0.2
Uplift plasticity coupling parameter	ζ	-	1.5

4.4 Superstructure model

The bridge pier is modeled using small-displacement/small-rotation Timoshenko beam elements with an elastoplastic constitutive law. For simplicity, an elastoplastic bilinear model is adopted in the analyses. The moment-curvature diagram for the examined concrete column has been calculated in [15]. The parameters used for the definition of the bilinear moment-curvature diagram are the yield moment M_y and the post yield stiffness of the beam in pure tension. The elastic stiffness of the beam elements is calculated from the geometric characteristics of the cross section and the elastic properties of reinforced concrete. The numerical parameters used for the definition of the bilinear model for the beam elements are presented in Table 3.

Table 3. Structural model parameters

Parameter	**Symbol**	**Unit**	**Value**
Yield moment for elastoplastic beam element	M_y	MN.m	37.5
Yield curvature	κ_y	-	$6.52\ 10^{-4}$
Post-yield stiffness for beam elements under traction	K_{post}	MPa	5.0

5 INCREMENTAL DYNAMIC ANALYSES

An incremental dynamic analysis (IDA) consists in performing a series of non-linear time-history analyses, using as input motion the same acceleration record scaled to increasing amplitudes, and keeping track of some characteristic quantities of the response of the structure. Using the terminology introduced in [3], we refer to *intensity measures* (IMs), characterizing the severity of the input motion and to *damage measures* (DMs), characterizing the response of the structure. The output of an IDA is an *IDA curve*, *i.e.* a plot of a selected IM versus a selected DM. Similarly, an *IDA curve set* is a collection of IDA curves of the same structural model under different records that have been parameterized on the same IM and DM.

Different options are available for the IM to be used in the IDA curves (PGA, CAV, spectral acceleration at some specific frequency…). In the following, we choose the cumulative absolute velocity (CAV), guided by its cumulative character which may be a better proxy for the residual response parameters of the structure.

Several quantities may also be considered for the DMs: drift, residual displacements (rotations), etc… We choose the maximum structural ductility demand in the concrete column, μ_d related to the maximum curvature κ and yield curvature (tab. 3) through:

$$\mu_d = \frac{\max\{\kappa\}}{\kappa_y} \tag{1}$$

A set of 30 acceleration records has been chosen for the incremental dynamic analyses. The compilation of the suite of records has been given in [3]. The selected acceleration records are from relatively large-magnitude earthquakes (M = 6.5-6.9) with moderate distances and exhibiting no marks of directivity. Additionally, they have all been recorded on firm soil conditions. They represent a realistic earthquake scenario for the examined soil-structure system. Figure 3 presents the 5% damped response spectra of the suite of records.

The incremental dynamic analyses have been performed for every possible combination of the IMs and DMs. Each IDA analysis is made of 30 curves corresponding to the 30 time histories. A common feature to all IDA analyses is that some curves exhibit instabilities, [3], possibly followed by regain at higher levels, while other do not show any sign of instability, at least up to the highest tested IM. These kinds of curves are instructive because they clearly evidence the variability of the response as a function of the individual records, although all records are deemed to represent an almost unique earthquake scenario. The content of an IDA set is more compact and meaningful if, instead of individual curves, the median and some fractiles, for instance 16% and 84% fractiles, are presented, which can be further incorporated in a PBEE framework, [3]. IDA curves have been constructed for the three cases involving (or not) soil-structure interaction, namely:

- Nonlinear fixed-base structure
- Nonlinear structure with linear soil-structure interaction
- Nonlinear structure with nonlinear soil-structure interaction

As mentioned previously, it is anticipated that IMs that reflect the cumulative damaging effect of the earthquake are good proxies for correlation with a DM related to residual states (permanent settlement, permanent foundation rotation). Therefore ductility demand and permanent settlements have been related to CAV. On the other hand, DMs related to peak responses, like the maximum deck displacement, should be better correlated to the spectral acceleration at the fundamental period of the system.

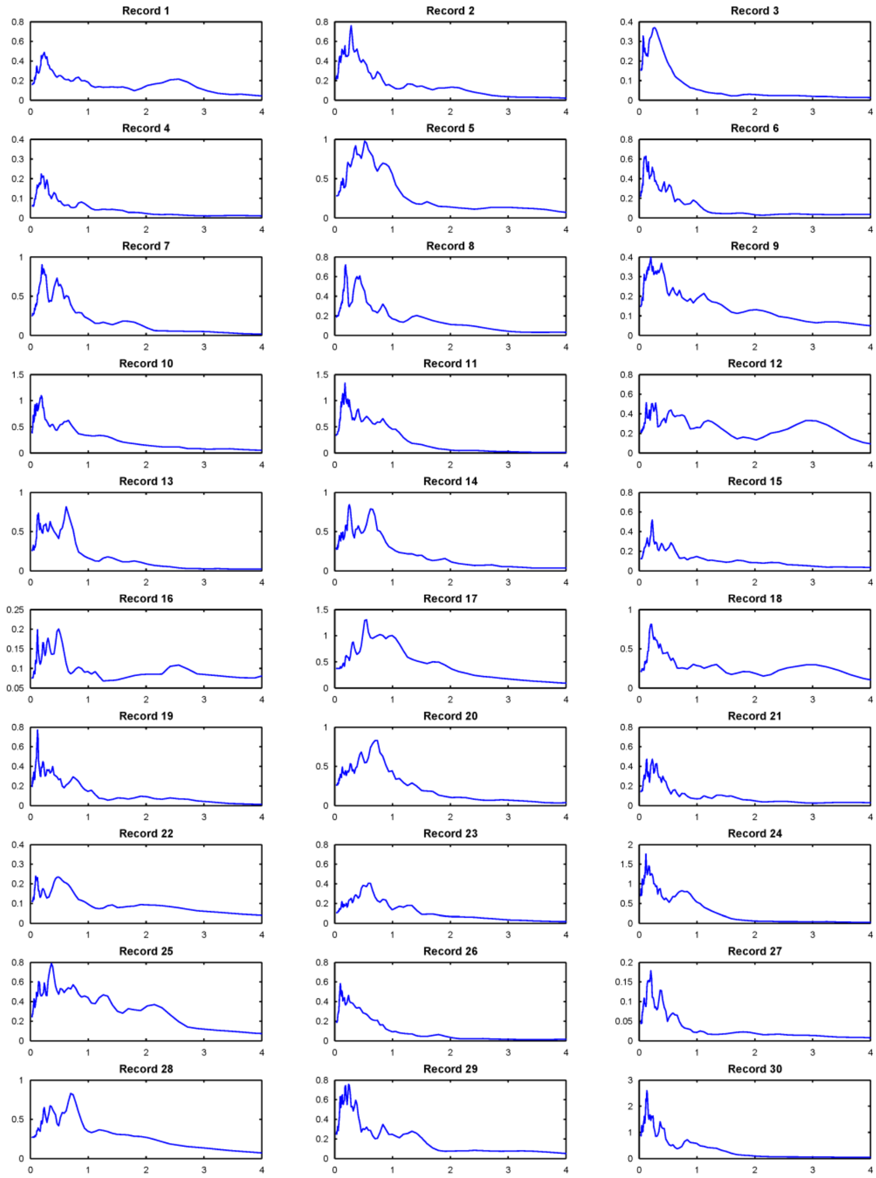

Figure 3. Acceleration response spectra of the suite of unscaled records

5.1 Typical results of an IDA analysis

A typical dynamic response produced with the macroelement is depicted in Figure 4 showing the variation during excitation of typical quantities: pier curvature versus bending moment, horizontal displacement and deck drift versus time, foundation rotation versus rocking moment and foundation settlement versus rotation.

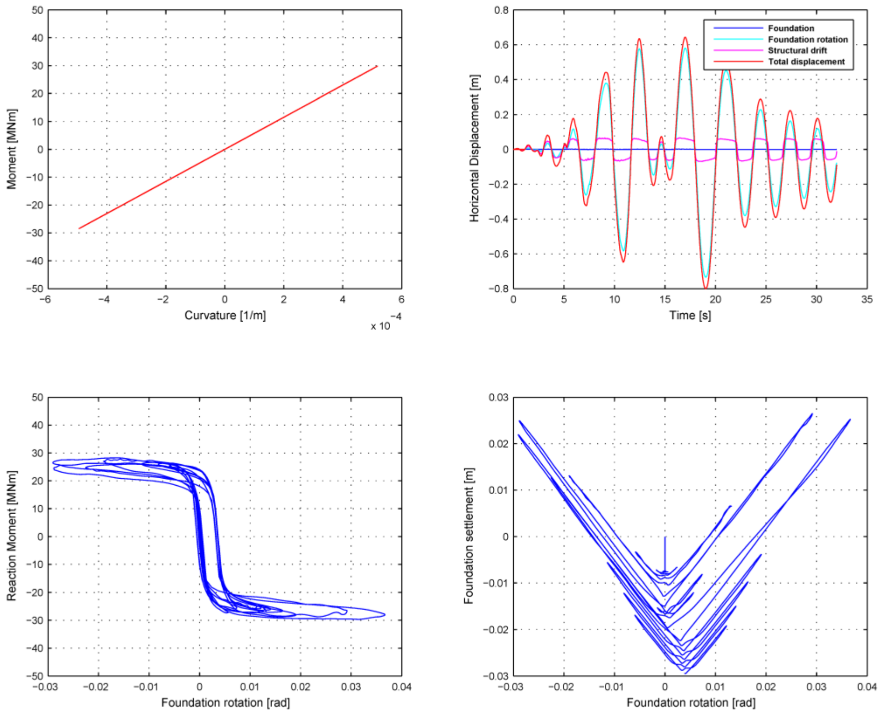

Figure 4. Example of a dynamic analysis with the macroelement; non-linear foundation, record 1, PGA=0.5g

This figure illustrates the capability of the macroelement to produce permanent settlement under horizontal excitation and uplift of the foundation, evidenced by the S-shaped of the moment-rotation diagram. The deck develops significant horizontal displacements almost entirely due to foundation rotation. However, the structure remains elastic as revealed by the moment-structural curvature diagram: it is clear that uplift acts as an isolation mechanism for the superstructure.

5.2 Statistical results

For each of the IDA set of curves, similar to those of Figure 4, statistical values corresponding to the median and to the 16% and 84% fractiles are computed. Comparisons are made in terms of structural behavior for the three possible assumptions for the foundation behavior: fixed-base structure, elastic linear foundation (linear SSI) and nonlinear behavior. All results are obtained with the macroelement model, with the proper options activated, and the nonlinear structural model for the structure. Due to space limitations, only few significant results are presented. They correspond to the ductility demand in the bridge pier, the permanent foundation settlement (for the nonlinear foundation). As mentioned previously the ductility demand and the residual settlements are related to the CAV.

Figures 5 to 8 present the statistical curves for the ductility demand for the three cases of foundation behavior. The overall behavior is not so different between the fixed-base structure and the linear elastic foundation: beyond a CAV of the order of 20, the ductility demand increases at a very rapid rate, denoting the onset of instability. It is interesting to note that up to a CAV of 10m/s, both systems produce the same median curve; for larger CAV, the ductility demand is slightly larger, for a given IM value, for the linear elastic SSI system indicating that SSI may not be favorable. This result is in line with the more extensive study of [7]. For the non linear foundation system, the behavior is strikingly totally different: up to a CAV of 35m/s, the ductility demand remains limited, of the order of 1.0, with no evidence so far of instability in the bridge pier.

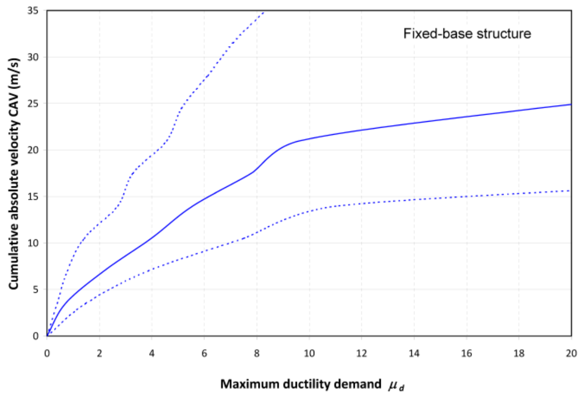

Figure 5. IDA curves for ductility demand versus CAV for the fixed-base structure. Thick curve: median, dotted curves: 16% and 84% fractiles

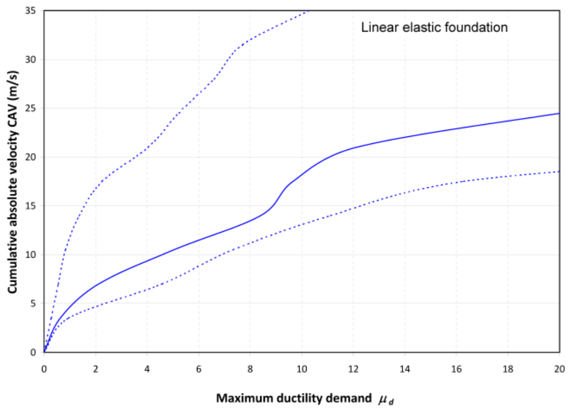

Figure 6. IDA curves for linear elastic foundation

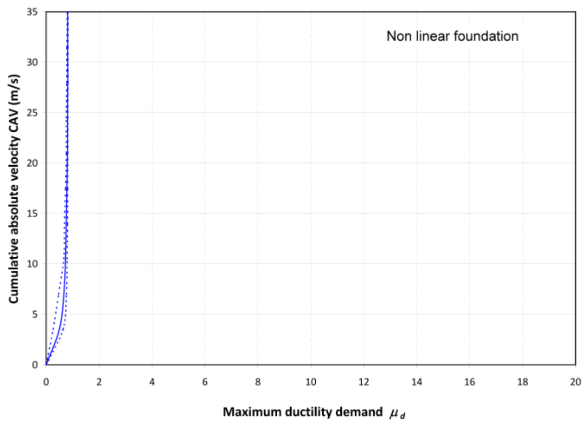

Figure 7. IDA curves for nonlinear foundation

The explanation for such a different behavior lies in the yielding of the foundation that protects the structure, as pointed out in [8]; the structure is prevented from yielding but permanent settlement and rotation are developed at the foundation. This is evidenced in Figure 8 showing the residual foundation displacement; obviously for the fixed-base structure and the linear SSI system no such values exist. The median maximum displacement remains limited but the variability increases drastically as the CAV increases; at the maximum CAV value the 84% fractile is 2.5 times the median. Therefore, foundation settlement may become highly unpredictable and can easily go from an acceptable quantity to an unacceptable one depending on the probability of exceedance the designer is ready to accept. This factor requires in depth consideration before accepting significant foundation yielding.

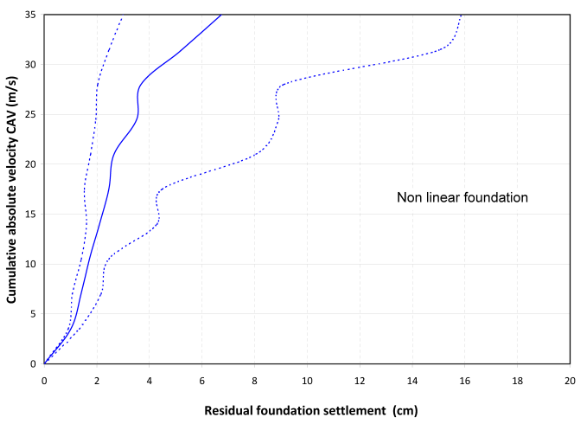

Figure 8. Residual foundation settlements versus CAV for non-linear foundation

Other results not shown herein exhibit the same trend: yielding of the foundation “protects” the structure but the price to pay is an increase of the maximum or permanent displacements and rotations of the foundation; more importantly, the variability in the computed response becomes large for the nonlinear SSI system when the IM is approaching the value for which the fixed-base structure or the linear SSI system show instabilities.

6 CONCLUSIONS

The development of a dynamic macroelement renders possible the use of extensive time history analyses to analyze the effect of foundation compliance and nonlinearity on the structural response of a nonlinear structure. The approach followed in this paper is based on the concept of incremental dynamic analyses which allows the derivation of statistical properties of the response. A simple bridge pier modeled either as a fixed-base structure, or founded on a foundation, for which linear or nonlinear soil structure interaction is considered, has served to illustrate the most salient features of the response. On a whole, consideration of nonlinear soil structure interaction appears beneficial to drastically reduce the ductility demand in the structure; however, this positive effect is counterbalanced by larger displacements and rotations at the foundation which may become unacceptable. Furthermore, it has been noticed that the variability in the response becomes large as more demand is placed on the foundation. Therefore, care must be exercised before accepting to transfer the ductility demand from the structure to the foundation. This implies a careful definition of acceptable criteria for the foundation displacement and rotation, and a thorough investigation of the variability of the response. As demonstrated in the paper the variability is conveniently handled with incremental dynamic analyses, which can be further incorporated in a performance based design approach. Nevertheless, this concept of allowing nonlinearities to develop in the foundation shows some promise as already pointed out in [2]. A final interesting finding of this study is that, as already shown in [7], consideration of linear soil structure interaction may not be always as beneficial as considered in practice.

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