The role of non-linear dynamic soil-foundation interaction on the seismic response of structures

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1 Introduction

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 non-linearity of soil and foundation. Such an approach may lead to non-conservative oversimplifications, especially in the case of strong geometric non-linearities as the ones corresponding to foundation uplifting and sliding. More importantly, neglecting such phenomena prohibits the exploitation of strongly non-linear energy dissipating mechanisms in case of occurrence of ground motions larger than design.

As a matter of fact, the capability of the foundation to dissipate energy during seismic loading is generally neglected in seismic design and in most cases explicitly avoided by the requirement that the foundation should resist the action effects resulting from the response of the superstructure, without substantial permanent deformations (CEN 2004a). The main practical reason behind this requirement is that the foundation cannot be easily inspected and repaired after an earthquake. However, this conservative approach may have several important drawbacks, mainly related to the following considerations:

(i) spectral ordinates of recorded accelerograms during moderate-to-strong earthquakes are often in excess of design spectra and development of significant permanent settlements and rotations of the soil-foundation system may be unavoidable;
(ii) the elastic requirement for seismic design may lead to uneconomic oversized foundations;
(iii) foundation retrofitting of existing or damaged structures is practically impossible to be accomplished to comply with the requirement of elastic behavior.

In the past, at least up to the recent research work that is overviewed in this paper, the "no-damage" approach to seismic foundation design was also supported by the lack of practical approaches and experimental results providing a reliable quantification of permanent displacements and rotations of foundations under realistic seismic loading. Such limitation is becoming more and more obsolete, as a relevant number of experimental and numerical results are clearly showing that sharing the yielding capacity between foundation and superstructure is an effective means to reduce the ductility demand on the superstructure, by providing a complementary, economic and very effective source of energy dissipation. The price to pay is obviously the degradation of the foundation system. However, such degradation, expressed in terms of permanent foundation settlements and/or rotations, turns out to be in most cases limited and well below the serviceability limit states, as the recent experimental and numerical results have shown.

Furthermore, the improved reliability of numerical tools, calibrated by the increasing number of carefully controlled experimental results, is making feasible the practical application of dynamic non-linear soil-foundation-superstructure concepts to seismic design, as shown in this paper in the framework of displacement-based seismic design approaches.

All these developments confirm that a rational and integrated approach to seismic design of foundations and structures must involve a controlled share of ductility demand between the superstructure and the foundation. Therefore, this paper intends to support such concept, by providing:

– an overview of recent experimental results of seismically loaded structures on non-linear shallow foundations;
– a summary of theoretical advancements based on improved macro-element modeling of the soil-foundation system;
– a proposal for taking into account non-linear soil-foundation-superstructure interaction (SFSI) effects in the framework of displacement-based seismic design of bridge piers;
– a summary of numerical results of incremental dynamic analyses (IDAs) of bridge systems with non-linear macro-elements at the base.

2 Experimental results of seismically loaded structures on non-linear shallow foundations

The large-scale experiments on non-linear dynamic soil-structure interaction (SSI) carried out at the JRC Ispra, Italy (Negro et al. 2000; Faccioli et al. 2001) and at CEA, Saclay, France (Combescure and Chaudat 2000), under the EC funded projects TRISEE and ICONS, respectively, and the later series of cyclic and shaking table tests at PWRI, Tsukuba, Japan (Shirato et al. 2008), are some of the few pioneering experiments in such field. More recently, there has been a further important research effort to clarify the role of foundation non-linear response on the amount of energy dissipation of different structural systems such as frames, shear walls and bridge piers.

Cyclic tests still are fundamental to calibrate experimental results and check the dependence on the amplitude, frequency and number of cycles. Nevertheless, dynamic experiments, where the base excitation is given in terms of imposed accelerograms, allow for a better understanding of the system behavior under realistic seismic motions. Taking advantage of improved experimental devices, such as the earthquake simulators for centrifuge seismic testing (e.g., Heron et al. 2012), a significant amount of recent experimental work was carried out on relatively simple soil-foundation-structure systems, either on shaking table or on centrifuge. An overview of such works is presented in Table 1, where it is worth noting that all referenced experiments deal with shallow foundations.

A synthesis of relevant information from this experimental work is reported in Table 2, where results are summarized in terms of: (i) the static safety factor of the foundation (N_{max}/N), (ii) the horizontal seismic coefficient (V_{max}/N), (iii) the eccentricity ratio (M_{max}/NB), (iv, v) the peak and residual foundation rotation (θ_{max} and θ_{res}, respectively), (vi) the residual normalized settlement (δ/B). Considering the recent dates of most of the experiments, partly not published yet, it is to be expected that such wealth of results may support a large amount of research in this field in the near future.

Although it is difficult to derive general conclusions, because of the different experimental setups, soil conditions, structural types, and input motions, some common features are worth to be highlighted. First, as illustrated in Table 2, in spite of the high intensity of the earthquake excitation, the normalized residual settlements seldom exceed about 1 %, which only occurs either under "extreme" loading conditions or in the case of liquefiable soils (Allmond et al. 2011).

This means, considering typical foundation widths of few meters, that maximum residual settlements are of the order of few centimeters, typically within the serviceability limits under static loads for isolated footings of conventional structures (e.g., CEN 2004b). In what refers to residual foundation rotations, in spite of the very severe loading conditions of practically all the experiments, which have all reached or exceeded the seismic bearing capacity, and excluding the case with liquefiable soil, more than 50 % of the records do not

Table 1 Some references of experimental activity on dynamic non-linear structure-foundation interaction under seismic excitation

Seismic excitation	Structure	Structural response	Soil	Laboratory	References
Shaking table	CAMUS-IV RC frame	Non-linear	Dry sand Dr 71 %	CEA, Saclay, France	Combescure and Chaudat (2000)
Centrifuge	2-storey shear wall + RC frame	Non-linear	Dry sand Dr 80 %	UC Davis, USA	Chang et al. (2007)
Centrifuge	Bridge pier	Linear-elastic	Dry sand Dr 80 %	UC Davis, USA	Ugalde et al. (2007)
Shaking table	Steel rack	Rigid	Dry sand Dr 80 %	PWRI, Tsukuba, Japan	Shirato et al. (2008)
Shaking table	Steel plates	Rigid	Dry sand Dr 50 %	U Bristol, UK	Massimino and Maugeri (2013)
Centrifuge	SDOF column	Linear-elastic	Dry sand Dr 38 %	UC Davis, USA	Deng et al. (2012)
Centrifuge	2-story frame building with structural fuses	Non-linear	Overcon-solidated clay	UC Davis, USA	Liu et al. (2012)
Centrifuge	2-column bent bridge	Non-linear	Dry sand Dr 77 %	UC Davis, USA	Deng et al. (2012)
Shaking table	Bridge pier	Linear-elastic	Dry sand Dr 85 %	NTUA, Athens, Greece	Anastasopoulos et al. (2013)
Shaking table	Bridge pier	Linear-elastic	Dry sand Dr 85 %	NTUA, Athens, Greece	Drosos et al. (2012)
Centrifuge	SDOF column	Linear-elastic	Submerged dense sand or coarse gravel on liquefiable sand layer Dr 68 %	UC Davis, USA	Allmond et al. (2011)
Centrifuge	SDOF column	Linear-elastic	Overcon-solidated clay	UC Davis, USA	Hakhamaneshi et al. (2012)
Centrifuge	SDOF structures	Linear-elastic	Dry sand Dr 51 and 85 %	U Cambridge, UK/IFSTTAR, Nantes, France	Heron et al. (2012)

exceed 2 mrad, often considered as a serviceability limit state under static loads (CEN 2004b). Further investigations are worth to be undertaken in order to clarify the conditions, typically related to unsymmetry in loading and/or configuration, that lead to large permanent rotation values (>10 mrad).

As shown in Table 1, one-third of the reported experiments involved the interaction of the foundation with a superstructure allowed to respond in the non-linear range. The pioneering CAMUS IV test (Combescure and Chaudat 2000) was carried out in the seismic laboratory of the French Commissariat à l'Energie Atomique (CEA), in Saclay, France, as the fourth

Table 2 Some relevant information and results of the experimental tests on shallow foundation-structure interaction under seismic excitation

Laboratory	Test case (see ref.)	N_{max}/N	V_{max}/N	M_{max}/NB	θ_{max} (mrad)	θ_{res} (mrad)	δ/B (%)
CEA Combescure and Chaudat (2000)	Nice 0.05 g	4.2	0.10	0.17	0.6	0	~0
	Nice 0.33 g	4.2	0.23	0.35	4.2	0.4	0.10
	Nice 0.52 g	4.2	0.27	0.42	9.5	0.8	0.14
PWRI Shirato et al. (2008), Paolucci et al. (2008)	1–2 1993 Hokkaido Nansei Oki 0.60 g	28.6	0.43	0.48	36	23	2.34
	1–4 Kobe 0.71 g	28.6	0.46	0.44	228	163	4.04
	2–2 Kobe 0.55 g	28.6	0.44	0.37	47	4	0.49
JRC Faccioli et al. (2001)	Phase II	5	0.22	0.19	3.2	0.4	0.27
UC Davis Deng et al. (2012)	SF	11	0.27	0.49	10	1	0.18
	ISF (footing on concrete pads)	n.a.	0.32	0.56	13	~0	0.14
UC Davis/UC San Diego Liu et al. (2012)	Balanced design model (B = 3.2 m) Gazli—0.84 g	9.1	0.35	0.48	20	~2	0.31
	Structural-hinging dominated (SHD) model (B = 6.1 m) Gazli—0.84 g	14.7	0.59	0.54	9	~0	0.14
	Foundation-rocking dominated (FRD) model (B = 2.3 m) Gazli—0.84 g	6.9	0.23	0.42	23	~5	0.33
NTUA Anastasopoulos et al. (2013)	Aegion 0.37 g (B = 11 m)	6.9	0.28	0.34	3.9	0.2	0.24
	Aegion 0.37 g (B = 7 m)	3.3	0.14	0.26	6.1	0.2	0.41
	Gilroy 0.32 g (B = 11 m)	6.9	0.40	0.48	3.2	0.5	0.09
	Gilroy 0.32 g (B = 7 m)	3.3	0.20	0.38	6.9	1.3	0.46
	Lefkada 0.43 g (B = 11 m)	6.9	0.40	0.49	5.0	0.2	0.75
	Lefkada 0.43 g (B = 7 m)	3.3	0.17	0.33	6.9	1.3	0.46
	Rinaldi0.82 g (B = 11 m)	6.9	0.47	0.56	27.2	7.4	0.53
	Rinaldi 0.82 g (B = 7 m)	3.3	0.20	0.37	26.0	13.7	2.17
	Takatori 0.61 g (B = 11 m)	6.9	0.43	0.51	25.6	0.9	0.36
	Takatori 0.61 g (B = 7 m)	3.3	0.26	0.49	88.0	75.0	1.57
UC Davis Allmond et al. (2011)	SC (B=7.35 m) Kobe—0.57 g	–	0.31	0.47	36	14	5.80

Table 2 continued

Laboratory	Test case (see ref.)	N_{max}/N	V_{max}/N	M_{max}/NB	θ_{max} (mrad)	θ_{res} (mrad)	δ/B (%)
	SF (B = 7.35 m) Kobe—0.57 g	–	0.37	0.56	41	12	5.93
	EC (B = 7.35 m) Kobe—0.57 g	–	0.35	0.52	29	~0	5.22
	EF (B = 7.35 m) Kobe—0.57 g	–	0.40	0.60	45	39	3.97
UC Davis Hakhamaneshi et al. (2012)	SD Structure Gazli—0.64 g	3.2	0.18	0.34	37.7	1.2	0.85
	LD Structure Gazli—0.64 g	6.0	0.23	0.45	37.6	0.4	~0
UC Davis Deng et al. (2012)	North Bridge, North Footing (rocking dominated, B = 5.04 m) Gazli—0.62 g	29	~0.37	0.64	12.9	3.5	–
	North Bridge, South Footing (rocking dominated, B = 5.04 m) Gazli—0.62 g	29	~0.31	0.51	12.2	2.8	–
	South Bridge, North Footing (stuctural-hinging dominated, B = 7.56 m) Gazli—0.62 g	53	~0.39	0.42	~4.0	~0.6	–
	South Bridge, South Footing (stuctural-hinging dominated, B = 7.56 m) Gazli—0.62 g	53	~0.42	0.43	3.4	0.6	–

B = foundation width; N_{max} = static bearing capacity, N = vertical load; V_{max} = max. horizontal load transmitted by the superstructure during the excitation; M_{max} = max. overturning moment transmitted by the superstructure during the excitation; θ_{max} = max. foundation rotation; θ_{res} = residual foundation rotation; δ = residual foundation vertical settlement

of a series of shaking table tests on building specimens consisting of two parallel 5-floor reinforced concrete (RC) walls without openings. For the CAMUS IV experiments, a 40 cm deep sand layer was placed in a 4 m × 4 m box fixed to the shaking table. Two strip foundations were designed at the base of the building, to achieve a realistic value of vertical stress under static loading (126 kPa). In Fig. 1 (left side) a sketch of the experimental setup is shown, while in the right side we illustrate the base shear versus top displacement plots obtained by the CAMUS III (approximately the same structure but with fixed base) and by the CAMUS IV tests. On the bottom right side, the base moment versus rotation plot for the latter test is also shown. Although the comparison of results is not straightforward, since the base excitation of the CAMUS III and IV tests was scaled to 0.66 and 0.52 g, respectively, and the steel reinforcement of the two specimens was not the same, some clear indications are highlighted, namely:

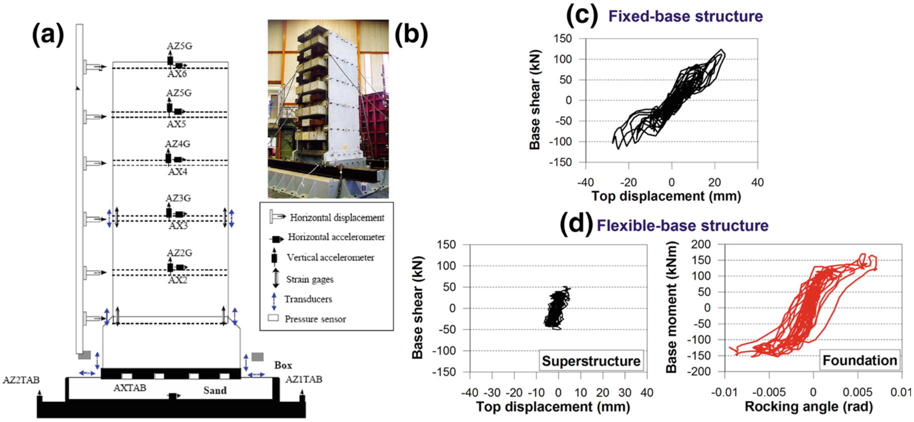

Fig. 1 **a** and **b**: experimental setup of the CAMUS IV shaking table test at the Saclay seismic laboratory of CEA (Combescure and Chaudat 2000). **c** Base shear versus top displacement of CAMUS III fixed-base specimen, under the Nice accelerogram scaled to 0.66 g; **d** Base shear versus top displacement of CAMUS IV specimen on D_r 71 % dense sand, under the Nice accelerogram scaled to 0.52 g, together with the corresponding plot of base moment versus foundation rotation angle. Records available at the web site: http://www.tamaris.cea.fr

– a relevant fraction of the input energy is dissipated at the foundation level;
– the seismic demand on the superstructure is highly reduced with respect to the fixed-base case, resulting in a nearly elastic behavior of the structure with limited onset of damage and no reinforcement yielding.

Subsequent and more recent experiments reported in Tables 1 and 2, either with linear elastic or non-linear superstructures, essentially confirmed the previous findings, giving rise to preliminary indications on how to design the whole superstructure-foundation system in order to achieve a "balanced design" configuration, where the footing and structural system are intended to yield at approximately the same strength, and share a similar amount of seismic ductility demand (Liu et al. 2012).

Finally, it is worth noting that in all reported cases with strong foundation systems where structural collapse was experimentally observed, it was avoided when non-linear foundation behavior was considered.

3 Numerical approaches by non-linear soil-foundation macro-elements

Implementation of a design philosophy in which yielding at the foundation level is allowed for, albeit partially, requires that efficient and reliable tools are available for design. On the other hand, non-linear structural analyses are very sensitive to small changes in the soil and structural properties and in the ground motion. Moreover, there is a significant uncertainty in the definition of this input data. A safe design will therefore require a large number of analyses to be run and this can hardly be achieved efficiently with computationally expensive numerical approaches such as solid finite element models.

The concept of dynamic macro-elements, developed over the last decade, represents an incomparably more efficient tool to evaluate the effect of non-linear SSI on the response of a yielding structure. Such approach, described herein, corresponds to an attempt to move a step forward in performance-based seismic design of structures. Within the macro-element framework, the behavior of the entire foundation and supporting soil system is condensed into a fully coupled non-linear joint element connecting the free-field motion to the base of the structure. It thus consists of a simplified approach which reduces the size of the problem significantly, while preserving the essential features of the dynamic response of the system. The concept of macro-element can be better understood by introducing the following scales describing the examined soil-foundation-superstructure system:

(i) the local scale, which is the one of the constituent materials of the soil, the foundation and the superstructure. Elements at this scale are described by conventional constitutive laws for soil, concrete etc;
(ii) the global scale, which is the one of the system in its entirety;
(iii) the meso-scale, which can be viewed as an intermediate scale between the local and the global scales. This is, for instance, the scale of some structural elements or parts of the superstructure such as beams, columns, footings, etc.

Within this context, the macro-element approach can be viewed as a change in scale within the global model, where one passes from the local scale of the constituent materials of a specific part of the global model, say a particular structural element, to the meso-scale of this structural element viewed in its entirety. In making such a change in scale, what was originally described by a large number of elements in the local scale now constitutes a single macro-element in the meso-scale. This reduces the complexity of the global model significantly, which can then be dealt with in a much more inexpensive and efficient way.

The macro-element, viewed simply as a part of the global model, must be described by a constitutive law compatible with the rest of the global model elements. This constitutive law must be selected in such a way to ensure that the response of the system, examined at the meso-scale (i.e., with the macro-element) correctly reproduces the features of the actual response of the model (i.e., at the local scale) that were retained in making the passage from the local to the meso-scale. This is an essential remark, since the passage from the local scale to the meso-scale wipes out all characteristics of the local scale (e.g., stresses and displacements at any point in the soil domain near the footing) except for those that are deemed essential for the overall behavior of the global model. The features of the system to be retained at the meso-scale model are usually represented through a number of "generalized stress" variables and by the corresponding "generalized strain" variables according to the type of examined problem.

Although the concept of macro-element, as described above, has been extensively used in structural engineering (by the construction of "generalized media" for beams, membranes, plates, shells, etc.), its use in geotechnical engineering is up to now rather restricted. The term "macro-element" was initially introduced by Nova and Montrasio (1991) in their study of the settlements of shallow foundations on sand. Their model was developed for quasi-static monotonic loading, which was later extended by Pedretti (1998) to describe quasi-static loading-unloading cycles more efficiently. In parallel, Paolucci (1997) proposed a numerical tool based on the model by Nova and Montrasio (1991) permitting the study of the response of simple structures subject to dynamic (seismic) loading and taking into account the coupling between the non-linear response of the soil-foundation system and the response of the superstructure. The macro-element was further extended by Cremer et al. (2001, 2002) in order to consider all the material and geometrical non-linearities at the soil-footing interface, the coupling between them and their coupling with the response of the superstructure. Such macro-element was developed for strip footings, while the latest generations of macro-elements for shallow foundations (Chatzigogos et al. 2009, 2011; Figini et al. 2012) are

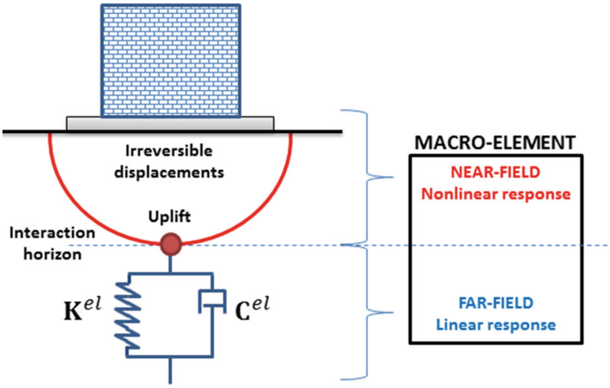

Fig. 2 Structure of the macro-element

also applicable to circular footings. They are all based on the same concepts derived from the work by Cremer et al. (2001, 2002) with simpler and easier-to-use constitutive models. Other macro-elements models were developed for cyclic and dynamic response of shallow foundations such as by di Prisco et al. (2006) and by Grange et al. (2008). The various examples of application have proven that this modeling approach is a very cost-effective solution in terms of the balance between physical behavior, simulation accuracy and computational cost (see, e.g., Muir Wood 2007, for a review). Recently, the macro-element approach was extended to the case of pile-supported bridges (Correia et al. 2012), opening a further interesting domain of application.

The structure of any of the macro-elements listed above is depicted in Fig. 2. The far field is governed by the propagation of seismic waves and easily modeled with the concept of dynamic impedances (usually represented by frequency-independent springs and dashpots); the near field takes into account all the non-linearities occurring in the system: geometrical non-linearities like foundation sliding or uplift for shallow foundations, gapping for pile foundations, and material non-linearities due to soil yielding under the foundation or along the pile shaft, with possible plastic hinging on the pile.

Establishing the constitutive relationship is equivalent to define the relationship between the increments of the generalized forces $\underline{\dot{Q}}$ and of the generalized associated displacements $\underline{\dot{q}}$ of the foundation:

$$\underline{\dot{Q}} = f\left(\underline{\dot{q}}\right) \tag{1}$$

The displacement increment $\underline{\dot{q}}$ is initially decomposed into an elastic, $\underline{\dot{q}}^{\mathrm{el}}$ and an inelastic component, $\underline{\dot{q}}^{\mathrm{in}}$. The elastic component incorporates the linear response of the foundation $\underline{\dot{q}}^{\mathrm{el-lin}}$ and also its response due to any non-linear mechanism that is reversible and non-dissipative $\underline{\dot{q}}^{\mathrm{el-nlin}}$. For shallow foundations, the latter component typically corresponds to the mechanism describing foundation uplift. For deep foundations, it corresponds to the mechanism of gap formation between the pile and the surrounding soil.

Similarly, the component of inelastic displacement $\underline{\dot{q}}^{\mathrm{in}}$ incorporates all mechanisms that induce energy dissipation in the system such as soil yielding (occurring both for shallow and deep foundations), sliding along the soil foundation-interface, etc. Each of these dissipative mechanisms can be described with an appropriately chosen plasticity model. The total displacement increment can thus be decomposed as follows:

$$\underline{\dot{q}} = \underbrace{\underline{\dot{q}}^{\text{el-lin}} + \underline{\dot{q}}^{\text{el-nlin}}}_{\underline{\dot{q}}^{\text{el}}} + \overbrace{\underline{\dot{q}}^{\text{pl},1} + \underline{\dot{q}}^{\text{pl},2} + \cdots}^{\underline{\dot{q}}^{\text{in}}} \tag{2}$$

For each of the components $\underline{\dot{q}}^{i}$ presented in (2), a generic tangent stiffness matrix $\mathbf{K}_i$ is defined, which allows to retrieve the corresponding applied force increment $\underline{\dot{Q}}$ in the system:

$$\underline{\dot{Q}} = \mathbf{K}_i \underline{\dot{q}}^{i} \tag{3}$$

Having established the above decomposition for the incremental total displacements, we give a brief review of the basic ingredients for establishing a complete constitutive relationship for a foundation macro-element:

– Concerning the linear component $\underline{\dot{q}}^{\text{el-lin}}$ this is by definition linked to the force increment $\underline{\dot{Q}}$ through a stiffness matrix $\mathbf{k}_{\text{el-lin}}$ which does not depend on the level of displacement but only on the geometric characteristics and on the elastic properties of the soil-foundation system. This matrix is identified with the impedance matrix of the foundation; for quasi-static loading conditions it is a real-valued matrix, while for dynamic conditions it is complex and frequency dependent. However, since macro-elements are typically used for non-linear analyses in the time domain, it is a standard practice to use frequency-independent impedance terms which can be calibrated with respect to a characteristic frequency of excitation of the system.
– The non-linear elastic components $\underline{\dot{q}}^{\text{el-nlin}}$ representing either the uplift or the gap formation mechanism, can be generically linked to the force increment $\underline{\dot{Q}}$ by a stiffness matrix $\mathbf{k}_{\text{el-nlin}}$ which, at variance with the previous case, is now function of the level of displacements in the system. Such dependence, defined by appropriate analytical expressions, can be expressed either directly on the reversible components of total displacements, as it has been done in the formulation presented by Chatzigogos et al. (2009, 2011) (general uplift model for shallow foundations of arbitrary shape) or indirectly, by means of auxiliary geometric parameters that evolve according to the loading history. Examples of such auxiliary quantities, are the percentage of uplifted width in the strip footing macro-element model presented by Cremer et al. (2002) and the current gap depth parameter introduced in the pile macro-element presented by Correia et al. (2012). It is worth noting that analytical expressions for uplift and gap formation models are typically obtained with curve fitting using results obtained with sophisticated numerical models of foundations under cyclic loading, in which the soil is modeled as an elastic medium and in which interfaces allow for uplift or gap formation (respectively for shallow and deep foundations).
– Regarding the components related to dissipative mechanisms $\underline{\dot{q}}^{\text{in}}$ a distinction needs to be made on mechanisms due to soil yielding and due to sliding along the foundation interface. In what refers to soil yielding, an essential feature of the response is the fact that, due to the extremely non-linear nature of soil in the local scale, a continuous plastic behavior is obtained from the very beginning of the loading path. This is true for both shallow and deep foundations. A convenient formulation to satisfy this condition has been proposed by Chatzigogos et al. (2009, 2011), was further developed in the works of Figini (2010); Correia (2011) and Correia et al. (2012), and is based on the formalism of bounding surface plasticity theory (Dafalias and Hermann 1982). The basic ingredients of this

model are: (i) a domain in the space of force parameters representing the combination of forces that can be supported by the foundation, and (ii) a mapping rule, which associates every point at the interior of this domain with an image point on the contour of the domain. The contour of this domain is called bounding surface and is typically identified with the surface of ultimate loads of the foundation, which has been defined using the yield design theory and kinematically admissible virtual velocity fields (Salençon 1990). The extent of plastic response for virgin loading and reloading can then be obtained as a function of the distance between the actual force state and its image point on the bounding surface. The direction of plastic displacements is also defined at the image point using an appropriately chosen plastic potential.

– Plasticity mechanisms activated along the foundation interface, such as sliding, do not require sophisticated hardening laws and can be formulated using perfect-plasticity models. Nonetheless, when more than one plasticity model are present in the macro-element (e.g., soil yielding and sliding), it is probable that the domains of admissible forces of each overlap with one another. In this case, the global admissible domain is defined as the intersection of all domains and the total plastic response needs to be obtained using the formalism of multi-mechanism plasticity (Mandel 1965).
– The above points highlight modeling principles for the basic components of the global response, examined independently from one another. From this point on, it is essential to introduce the elements of coupling between all these mechanisms. Generally speaking, coupling is obtained because Eq. (3) need to be solved simultaneously, for i indexing all active non-linear mechanisms. In these equations, the matrices $\mathbf{k}_i$ depend in general on the current force and displacement levels, thus resulting on all equations to be coupled with one another. This can be qualified as a first-order coupling. In addition to it, one can recognize a second type of coupling which is inherent to cyclic loading and refers to the gradual reduction of the foundation-soil contact area, relevant both for piles due to gapping formation and for shallow foundations due to uplift in the presence of soil yielding. One consequence of this effect is the cumulative reduction of the impedance terms of the foundation. Paolucci et al. (2008) have proposed to treat this type of coupling using a formulation inspired in damage mechanics: first, a relevant plastic kinematic parameter is selected as damage indicator; afterwards, the reduced geometrical characteristics of the foundation are related to such damage parameter through an ad hoc analytical expression, obtained from either experimental evidence or numerical analyses; finally, the impedance matrix is updated in each time increment according to the reduced geometry of the foundation.

Using the above modeling principles it has been possible to obtain macro-element formulations for a large variety of foundation configurations, covering shallow foundations of different shapes, pile foundations, cohesive and frictional soils, quasi-static and dynamic loading conditions, etc. Moreover, all these developments have often been validated with experimental tests, numerical analyses and post-earthquake evidence, as shown in Fig. 3, regarding the successful calibration on the CAMUS IV test results (Figini et al. 2012), which shows a remarkable agreement both on the rocking angle and on the settlement time-histories, the latter one presenting subsequent uplift phases that are captured nicely by the model.

In conclusion, macro-element models have by now reached a very satisfactory level of sophistication and the required maturity to undertake a systematic work of parameter calibration. Furthermore, in order for them to be of use for the practicing engineer, such calibration

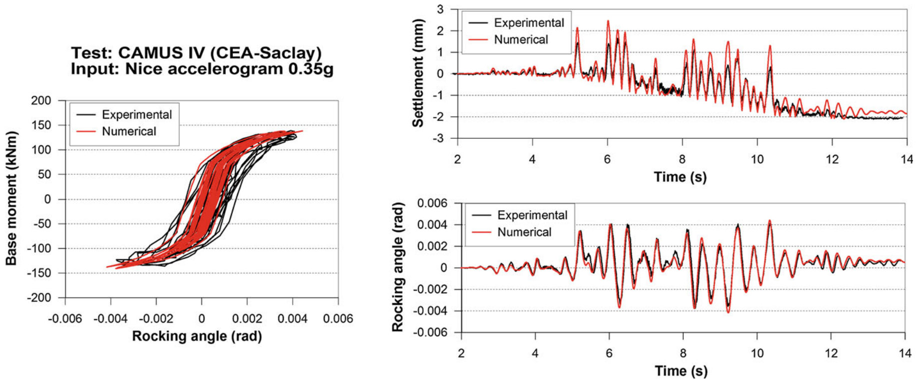

Fig. 3 Calibration of the non-linear foundation macro-element proposed by Figini et al. (2012), based on the CAMUS IV specimen tested at the CEA shaking table laboratories of Saclay, under the input Nice accelerogram scaled at 0.35 g

effort should eventually lead to a database associating specific macro-element models and sets of parameters to specific foundation configurations and soil conditions.

4 Introduction of an integrated structure-foundation design approach

A performance-based oriented seismic design approach of both structure and foundation should consider that each part of the system participates in the energy dissipation process during earthquake loading. The goal of such approach would thus be to design both parts of the system (structure and foundation), in order to achieve a prescribed performance in terms of allowable displacements and rotations. A direct displacement-based design (DDBD) approach is naturally suited for this purpose, because it explicitly sets the system performance in terms of displacements of both the foundation and of the superstructure systems (Priestley et al. 2007; Calvi et al. 2012).

An iterative design procedure was developed by Paolucci et al. (2013) to this purpose, which modifies the fixed-base formulation of DDBD, or the one with linear-elastic soil-foundation system, by introducing non-linear SSI effects. The procedure is based on the use of empirical curves to evaluate the rotational stiffness degradation (K_F/K_{F0}) and the increase of damping ratio ξ_F as a function of foundation rotation (θ). Iterations are performed to ensure that admissible values of foundation rotations are complied, in addition to the standard checks on structural displacements and drifts.

In Fig. 4, a set of such curves is presented for shallow foundations on dry sands, as a result of a parametric study, illustrated in more detail by Paolucci et al. (2009). Empirical equations for both $K_F/K_{F0}(\theta)$ and $\xi_F(\theta)$ as a function of the static safety factor (N_{max}/N) are introduced, for dense ($D_R = 90\,\%$) and medium dense ($D_R = 60\,\%$) soil conditions. An extended set of such curves is under development, based on the availability of new macro-element models, applicable both to cohesive and granular materials and to deep foundations as well (Correia 2011).

From Fig. 4, it is interesting to note that for light foundations, i.e., when large values of the static safety factor are used, as it is typically the case in seismic design of shallow foundations when displacement-controlled criteria are applied, the decay of rotational stiffness is much faster than for heavy foundations. The following empirical equations were calibrated by curve-fitting numerical results:

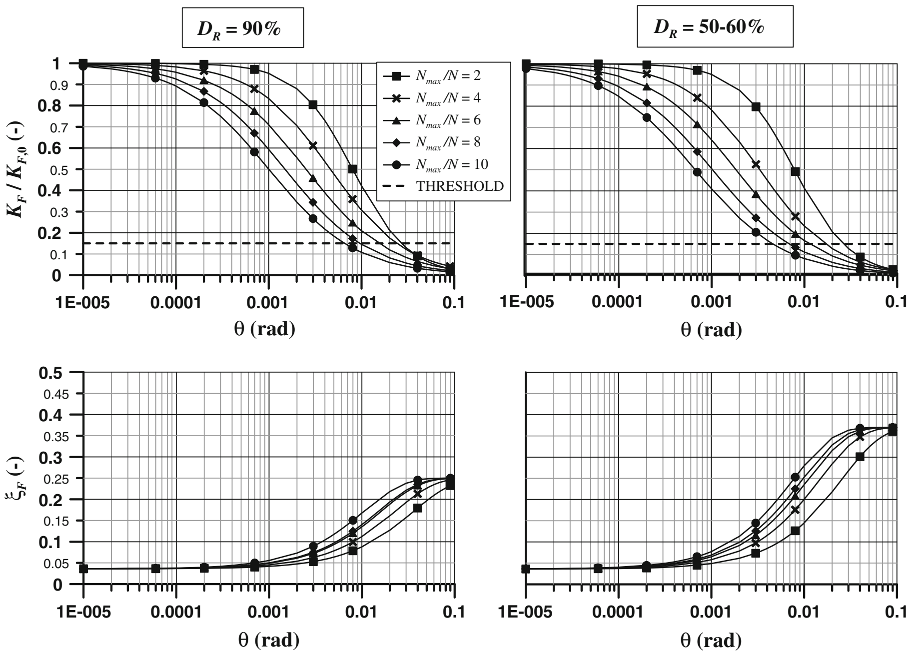

Fig. 4 Influence of the loading path on the rotational secant stiffness degradation (*top*) and damping increase (*bottom*), for dense (*left*) and medium dense (*right*) sand. Adapted from Paolucci et al. (2009)

$$\frac{K_F}{K_{F,0}} = \frac{1}{1 + a\theta^m} \tag{4}$$

$$\xi_F = \xi_{F,min} + \; \xi_{F,max} - \xi_{F,min} \; [1 - exp\,(-b\theta)] \tag{5}$$

where $\xi_{F,min}$, $\xi_{F,max}$, a, m and b are non-dimensional parameters and vary as a function of relative density D_R and of N_{max}/N. A saturation value, i.e., $\xi_{F,max} = 0.25$ for dense sands and $\xi_{F,max} = 0.37$ for medium sands, was introduced in Eq. (5), based on the available laboratory test results. A minimum value of the damping ratio was also introduced, $\xi_{F,min}$, set equal to 0.036 regardless of sand relative density, which represents radiation damping. The values of a, m and b are summarized in Table 3, both for dense and medium dense sands and for different values of the static safety factor.

The previous curves are one of the main inputs of an iterative procedure that was developed to be introduced in the framework of DDBD approaches. Its basic steps are summarized as follows, and sketched in Fig. 5:

1. Definition of the desired performance in terms of a target design value Δ_d, depending on the limit state to be considered, which may be expressed either in terms of the total lateral displacement Δ_{TOT} (including the foundation rotation contribution Δ_F) or of the structural displacement alone Δ_S;
2. Tentative initial design of the superstructure (e.g., in terms of the pier diameter D and of the reinforcement content);
3. Definition of the initial parameters of the foundation design (width B, bearing capacity, stiffness);

Table 3 Values of a, m and b in Eqs. (4) and (5)

N_{max}/N	Dense sand			Medium dense sand		
	a	m	b	a	m	b
2	458	1.30	27.7	686	1.30	39.4
3	282	1.11	32.8	386	1.11	47.6
4.5	263	1.00	43.9	339	0.98	67.8
6	293	0.94	62.2	352	0.92	90.6
7.5	325	0.91	67.0	398	0.89	104.5
9	378	0.89	85.1	433	0.86	119.2
10	415	0.88	95.6	452	0.84	130.8
15	575	0.83	164.4	653	0.79	210.4
20	1, 011	0.95	233.7	1,219	0.83	285.2

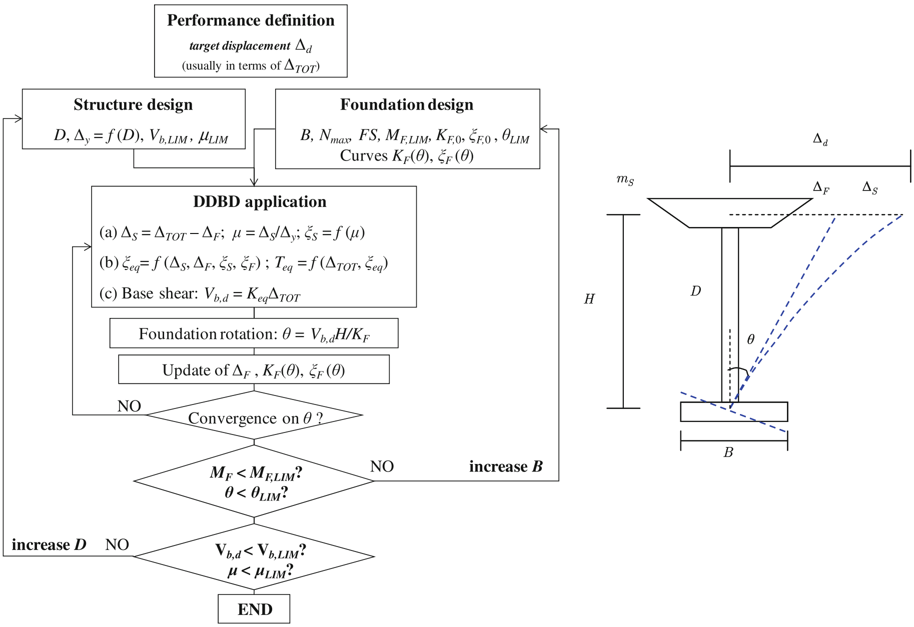

Fig. 5 Flow chart summarizing the DDBD procedure to account for the effects of non-linear SFSI. Details on the procedure are reported by Paolucci et al. (2013)

4. Characterization of the SDOF substitute structure (in the first iteration a fixed base is considered), whose effective natural period and damping ratio are modified to account for the non-linear interaction with the foundation;
5. Computation of base shear and overturning moment for the prescribed design displacement spectrum;
6. Determination of the corresponding foundation rotation θ;
7. Evaluation of the contribution of the foundation rotation $\Delta_F = \theta H$ to the total structural displacement;
8. Update K_F and ξ_F based on the expressions (4) and (5);
9. Test of convergence on foundation rotation (if not, return to step 4);

10. Check of the foundation seismic bearing capacity and admissible values of foundation rotation (if check is not satisfied, return to step 3 with an increased value of foundation width B);
11. Check of the structural base shear and ductility capacities (if check is not satisfied, return to step 2 with an increased value of pier diameter D).

This design approach was applied by Paolucci et al. (2013) to the seismic design of two bridge piers, its results compared with a standard DDBD procedure (considering the structure founded on a rigid base), and subsequently validated by time-history analyses of a non-linear SDOF oscillator supported by the 3DOF non-linear macro-element introduced by Figini et al. (2012). In both cases, it was possible to achieve a rational design solution with a reduction of the ductility demand on the superstructure by reducing the foundation width and allowing for controlled permanent damage on the foundation itself, both in terms of settlements and rotations.

5 Parametric non-linear time-history analyses

As previously discussed, the probabilistic set of performance-based design, together with the uncertainty in seismic input definition, requires a large number of non-linear time-history analyses to be carried out. Therefore, when SSI effects are significant, one of the main advantages of using foundation macro-element models for seismic design is their cost-effectiveness, expressed both in terms of accuracy of results and computational time required. In fact, such models have rendered feasible the study of the non-linear response of a structure under seismic loading of increasing intensity. One methodology that scrutinizes this progressive behavior, and arguably the most efficient and versatile one, is the technique of incremental dynamic analyses (IDAs) (Vamvatsikos and Cornell 2002).

An IDA is a parametric analysis method, suitable to provide a complete characterization of the non-linear behavior of a structure from the initial occurrence of non-linear effects up to complete collapse. Its methodology consists of computing the structural seismic response to one or more ground motions, each scaled to multiple levels of intensity. The result of this incremental analysis, for each record, is a curve providing the evolution of each engineering demand parameter (EDP) versus an appropriately chosen intensity measure (IM). The probabilistic nature of the seismic input is dealt with by using a set of records compatible with a particular earthquake scenario. Consequently, the evolution of non-linear structural response is probed by as many results as records considered, for each intensity level. A statistical treatment of the results can then be accomplished in order to provide meaningful information on the overall response of the structure. Taking into account the number of records, intensity levels, and a possible parameterization of the structural characteristics, several thousands of time-history analyses are typically carried out in such studies. It is thus seen that an essential requirement for performing IDAs is the existence of efficient non-linear models which, however, remain sophisticated enough in order to adequately capture the intrinsic characteristics of non-linear structural response. Due to these stringent requirements, the majority of IDAs' results presented so far have been mainly concerned with structural models considering either fixed supports or a linear behavior of the foundation substructure.

One of the first applications of non-linear foundation macro-elements in the context of IDAs was presented by Pecker and Chatzigogos (2010), who have studied the effect of shallow foundation non-linearities (soil plasticity and foundation uplift) on the ductility demand

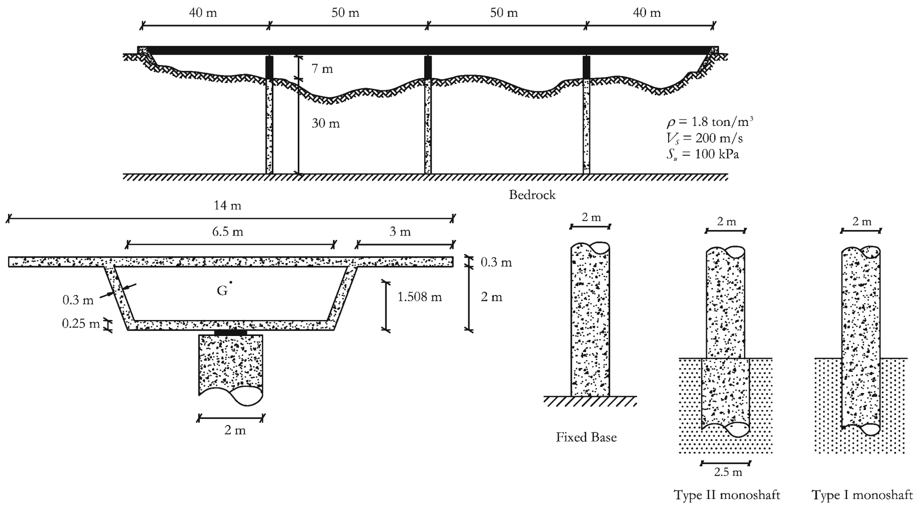

Fig. 6 Bridge geometry, soil deposit characteristics and alternative design solutions. After Correia et al. 2012

of a typical RC bridge pylon. They have shown that the compromise between analysis speed and model sophistication required in IDAs can clearly be accommodated by the use of such macro-elements. These replace both the foundation and the surrounding soil domain, and their use can actually be seen as equivalent to performing a degree-of-freedom condensation which reduces drastically the size of the numerical problem. Consequently, non-linear foundation macro-elements allow for non-linear SSI effects to be accurately considered within an IDA framework, thus reproducing their impact on the global non-linear response of the structure.

Further applications of foundation macro-elements within the context of IDAs are presented in two companion papers (Godoy et al. 2012; Correia et al. 2012). The first one focuses on assessing the structural safety of bridge pylons resting on footings. A structural model of a real bridge support using a shallow foundation non-linear macro-element is developed. It allows for safety margins in the bridge pylon to be determined, due to the actual non-linear response of the foundation in contrast to the conventional assumption of linear foundation response. In the second paper, a novel macro-element for pile foundations is proposed. This macro-element is then used in IDAs for assessing the seismic response of extended pile-shaft-supported bridges. Both studies were conducted after implementing the shallow and deep foundation macro-elements in the advanced seismic analysis software SeismoStruct (Seismosoft 2011).

Correia et al. (2012) study the response of a bridge structure with three different design solutions for the supports, as represented in Fig. 6. The corresponding maximum and residual displacements at the top of the middle column are depicted in Fig. 7 for the IDAs performed considering the seismic action in the transverse direction. As expected, the increased flexibility and influence of second-order ($P - \Delta$) effects when the pile-head macro-element is used is evident.

The interested reader can find, in the aforementioned papers, detailed information on these analyses and, more generally, on performing IDAs with non-linear SSI effects. A summary of a few important issues related to such task is presented and discussed in the following points:

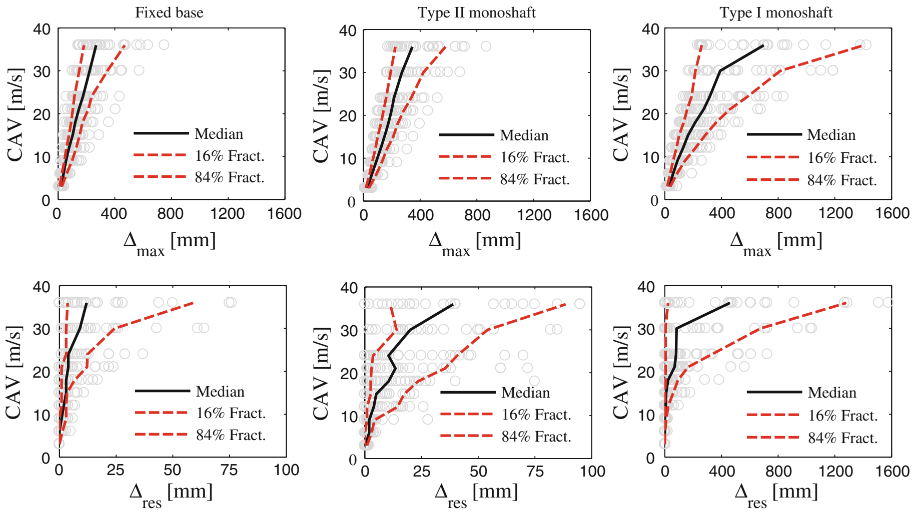

Fig. 7 IDAs' results: maximum and residual displacements at the top of the middle column. After Correia et al. 2012

- Probing non-linear foundation response is typically performed by selecting either maximum or residual kinematic quantities. Referring to shallow foundations, a relevant selection of EDPs would include the residual foundation settlement, the maximum and residual foundation rotation, and the maximum and residual foundation horizontal displacement. Considering deep foundations instead, one can additionally examine the residual gap opening, the maximum ductility demand in the piles and so on. Moreover, each of these quantities may be better correlated to a different IM. Intuitively, one may think that an EDP corresponding to a maximum instantaneous response quantity may be better correlated with an IM of the same sort (e.g., PGA). On the contrary, an EDP expressing a residual quantity may be better correlated with a cumulative IM instead such as the cumulative absolute velocity (CAV), the Arias intensity, etc. Quantitative evidence of this intuitive remark has been presented by Chatzigogos et al. (2009). A careful choice of a particular IM to be used in the IDAs is thus essential for the record selection and scaling procedures and to reduce the scatter in the seismic input and structural response to a minimum.
- The consideration of non-linear response at the foundation level (uplift in shallow foundations and gap formation in deep foundations; inelasticity in both foundation solutions) implies that significant rotations may occur, which in turn may lead to significant drifts for tall structures. It is thus essential for the analyses to take into account $P-\Delta$ effects. In fact, neglecting them may lead to erroneous results, especially in what refers to residual rotations, since $P-\Delta$ effects reveal the significance of the unstable nature of self-weight on a rocking structure.
- Particular emphasis must be placed on treating the analyses in which loss of numerical convergence occurs. Although theoretically unacceptable, this is a case that may inevitably appear as seismic intensity reaches higher levels. In general, a time-history analysis in which the calculation stops at some point in time due to loss of numerical convergence is considered as a case of structural instability or of progressive collapse. In both cases, the large magnitude of resulting displacements may reach the limits assumed in the numerical tools for the material models to be valid. When a complex system with

different possible failure mechanisms is treated, it is essential to identify which is the instability mechanism that is instigated by the excitation. It should be mentioned, however, that the typical lognormal distribution of the statistical description of EDPs is not significantly altered by this phenomenon, except when it affects a large number of sample values for a given seismic intensity.

– An additional remark for systems with multiple failure mechanisms, and in particular, with mechanisms at the foundation and the superstructure level, is that an IDA can and actually is meant to be able to identify any isolation effect offered by the activation of a non-linear mechanism in some part of the structure. This can be understood by examining two elastic-perfectly plastic sliders placed in series: the slider with the smaller sliding resistance is always the one that yields while the other remains in all cases elastic. It is thus clear that, if interaction is to be obtained between several instability mechanisms working in series, it is essential that they are designed so that their hardening behavior allows yielding in several locations of the foundation-structure system.
– Finally, it should be pointed out that there are cases where an increase in seismic demand corresponds to a decrease in the EDPs' values. Such peculiar behavior is, nevertheless, simply an expression of the evolving dynamic behavior of the inelastic system and the time-history evolution of a given mechanism. In fact, while one mechanism may be more meaningful to the structural response for a seismic demand level, an increased seismic demand may excite preferentially another one, due to the different spreading of inelastic deformations throughout the corresponding time-histories.

The above comments are useful hints for developing IDAs with nonlinear SSI effects using foundation macro-elements. This analysis procedure is deemed to offer a practical tool for verifying the requirements imposed by performance-based design. In addition, it opens yet another perspective in the same course of development, which consists in exploiting these analyses techniques for producing fragility curves of the examined systems. These are computed by relating the probabilistic seismic demand on the EDPs to the corresponding damage measures and possibly to the reparation cost, thus completing the steps within a complete performance-based earthquake engineering framework. Such evolution of geotechnical seismic design is now possible to be accomplished with reasonable effort by using the proposed macro-element models. Hence, the latter pave the way towards a correct implementation of probabilistic concepts for soil-structure nonlinear systems.

6 Conclusions

The growing awareness of the possible role of the foundation system within performance-based design approaches is presently supported, on one side, by the dramatic increase of experimental results of foundation response and non-linear soil-structure interaction under strong seismic loading, and, on the other side, by the calibration of improved numerical tools, boosted by the introduction of non-linear macro-element models and by their implementation into finite element codes for the seismic response of structures.

This paper has provided an overview of such experimental and numerical progress, aiming at supporting the concept that it is now possible to introduce more rational and integrated displacement-based seismic design approaches, to achieve a controlled share of ductility demand between the superstructure and the foundation.

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