Modelling Nonlinear Static Analysis for Soil-Structure Interaction Problems

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Abstract. Nonlinear static analysis is a well-established seismic design method for structural systems. However, soil-structure interaction can significantly affect the earthquake-induced effects in the structural members. This is particularly evident for bridges with integral abutments, that imply the full transmission of the inertial effects developing in the soil to the entire structural system. In this view, this paper proposes an extension of the capacity spectrum method to soil-bridge systems as a simplified numerical procedure directly applicable in design. In the proposed method, the capacity of the system is evaluated through two different distributions of equivalent inertial forces applied to a computationally manageable soil-bridge domain, reproducing the deformation patterns associated with the dominant vibration modes. These modes are mainly controlled by the soil response and can be reasonably determined by a modal analysis of the soil deposit. The entire procedure is implemented in OpenSees and validated against the results of several dynamic analyses carried out on a reference case study.

Keywords: Soil-structure interaction · Seismic design · Nonlinear static analysis · Modal analysis · Parallel computing · OpenSees

1 Introduction to Nonlinear Static Analysis in Geotechnical Engineering

As an efficient means for limiting the complexity and computational demand of non-linear dynamic analyses, the nonlinear static analysis is commonly employed in the seismic design of structures, such as buildings and bridges (i.e., by using the Capacity Spectrum Method [1, 2] or the N2 method [3, 4]). Nevertheless, in many problems the effects associated with the dynamic interaction between a structure with the surrounding soil can significantly affect the dynamic response of the system, hence the existing methodologies in this field need to be extended for incorporating these effects in the assessment of structural performance.

Laguardia et al. [5] proposed an extension of the nonlinear static analysis approach to dynamic soil-structure interaction problems: this method was inspired by the Capacity Spectrum Method [1] and is based on the combined use of the conventional pseudo static approach, in which the seismic action is simulated through equivalent

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static forces, with a direct estimate of the capacity curve. In the latter study, the method was mainly applied to the case of retaining structures.

Integral abutment bridges (IABs) are characterised by an integral connection between deck and abutments, implying a full transmission of forces and moments between them. This solution was originally conceived to reduce construction and maintenance costs. Notwithstanding, the integral connection can induce relevant internal forces in the deck-abutment system due to thermal deformations in the deck. To limit these effects, integral abutments are typically designed to exhibit a large deformability in the horizontal direction, for instance using a single row of foundation piles [6]. Although the numerous studies available in the literature on the behaviour of IABs, there is the lack of standardized design guidelines around the world and especially with regard to the seismic performance of this structural typology.

In this view, the present study extends the nonlinear static analysis approach proposed by Laguardia et al. [5] to the case of IABs. The method includes salient dynamic features of the entire soil-bridge system, analysed through a detailed investigation of its modal properties. The numerical procedure is fully implemented in the analysis framework OpenSees [7] and is validated against the results of nonlinear dynamic analyses of a real case study.

2 Numerical Model

2.1 Reference Case Study

This study considers a reference case inspired by a well-documented single-span integral overpass recently built in Italy along the A14 Adriatic highway, shown schematically in Fig. 1a: the bridge deck is 50 m long and its cross section consists of a steel-concrete composite structure. The reinforced concrete front walls, having height and width of 8.0 m and 2.2 m, are supported by seven reinforced concrete piles, placed in a single row, with length and diameter of 20.0 m and 1.2 m, respectively.

The idealised soil domain is constituted by two dry layers of gravelly sand with increasing stiffness and by a sandy embankment. Figure 1b shows the profile of the small-strain shear modulus with depth where the dashed line in the figure considers the increase in the effective stresses produced by the embankment.

2.2 Implementation in OpenSees

A three-dimensional finite element model (3D model) of the reference soil-bridge system was developed in OpenSees, illustrated in Fig. 2a. The main features of the 3D model are illustrated in the following, while the reader can refer to Gallese [8] for a more comprehensive description of the implementation.

The soil domain of Fig. 2a includes about 51000 eight-node brick elements with physically stabilized single-point integration (SSPbrick-class brick elements [9]). For simplicity, an equivalent rectangular cross section was assumed for the embankments, and equal transverse displacements were enforced to the nodes along the lateral sides. This modelling technique aims at simulating a reinforced earth embankment.

The structure, formed by the piles, the abutments and the deck, was modelled through about 2400 two-noded displacement-based beam-column elements exhibiting a linearly elastic behaviour. A grillage modelling approach was adopted for the abutment walls and the deck, where the properties of each beam were taken proportional to its area of influence.

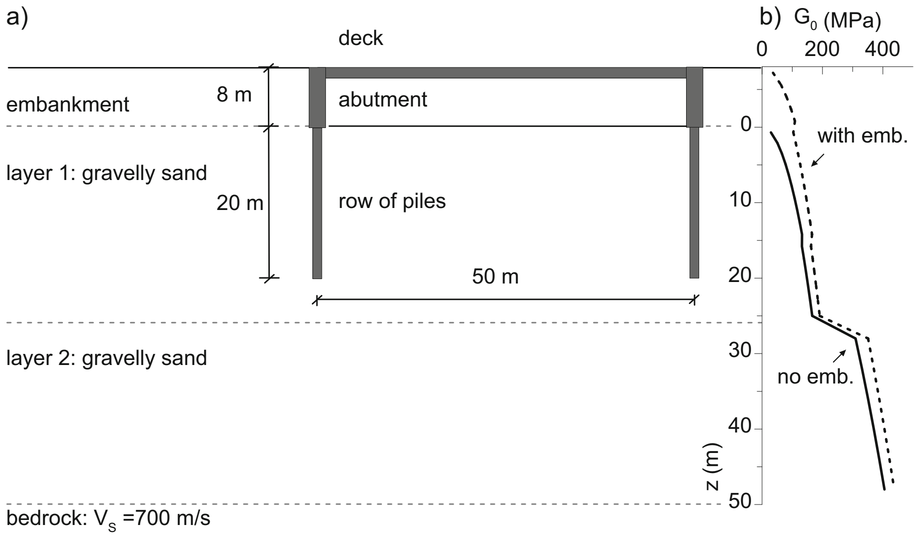

Fig. 1. (a) Schematic longitudinal section of the case study; (b) profile of the small strain shear modulus G_0 with depth.

To connect the piles to the surrounding soil, the solid elements in the region occupied by the piles were removed, and at each elevation the pile nodes were connected to the corresponding soil nodes using rigid links. In addition, thin layers of solid elements with reduced strength properties (soil-wall friction $\delta = 2/3\phi'$) were added along the pile shaft to model the soil-pile interface. The same procedure was applied to connect the abutment elements to the corresponding embankment elements.

The mechanical behaviour of the soil was simulated using the pressure-dependent multi-yield model (PDMY) developed by Yang et al. [10], having kinematic hardening and non-associated plastic flow, whose calibration for the case at hand was exhaustively discussed in Gallese [8].

The plan extension of the model was chosen on the basis of a parametric study aimed at minimising the effect of spurious reflected waves along the boundaries, while keeping the overall size of the model within manageable limits. The focus is on the longitudinal response of the bridge as the one more significantly affected by dynamic soil-structure interaction [11]. Therefore, in order to devise a manageable model for a prompt use in dynamic computations, an additional two-dimensional model (2D model) was developed, depicted in Fig. 2b, where appropriate boundary conditions were assigned to reproduce a plane strain deformation for the soil and plane stress conditions for the structural elements. This 2D model considers a length of 1.8 m in the transverse direction, corresponding to the pile spacing, and incorporates a single pile, the abutments and the deck. Preliminary nonlinear dynamic analyses, whose results are here omitted for brevity, demonstrated that this 2D representation provides a reasonable estimate of the bridge performance assessed through the full 3D model.

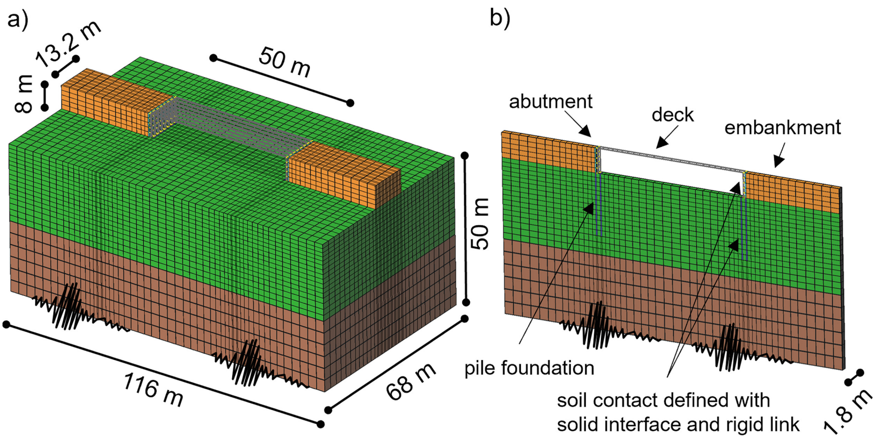

Fig. 2. (a) 3D soil-bridge model used in the analysis and (b) equivalent 2D model developed in OpenSees.

2.3 Analysis Procedure

In both 3D and 2D models, all construction stages of the bridge were accurately simulated (see Gallese [8]). After that, time-domain dynamic analyses were carried out by applying the seismic motion to the base nodes through viscous dampers, that simulate the presence of a compliant bedrock [12]. It is worth noting that most of the energy dissipation in the present model relates to the plastic strains produced by the constitutive assumptions. Nevertheless, a small viscous damping ratio of 2% was applied to the full domain using the Rayleigh formulation in order to attenuate the effect of spurious high frequencies.

The OpenSees parallel computing, performed by using the application OpenSeesSP [13], was employed to get reasonable computation times. Part of the analyses were carried out through the DesignSafe facility [14].

3 Modal Analysis

To have information about the deformation modes that control the seismic performance of the bridge, a modal analysis was carried out on the models of Fig. 2 and on a soil column (1D model) representing the soil domain under free field conditions (foundation soil and embankment). In this analysis, operational values of the elastic stiffness were assigned to the soil elements, corresponding to the stress state at the end of construction of the bridge. This procedure was implemented in OpenSees retrieving the stress tensor for each gauss point at the end of the static construction and computing the corresponding small strain stiffness.

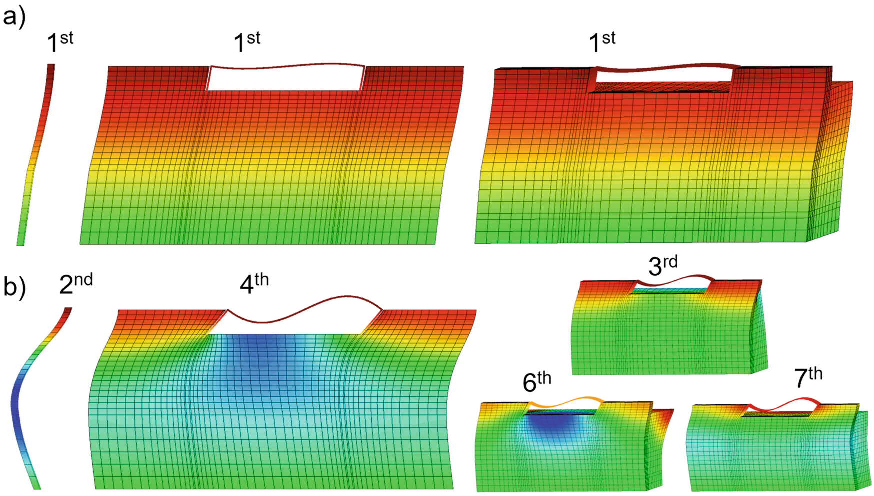

Fig. 3. Comparison between the a) first and b) second modal shapes of the 1D soil column with embankment and the corresponding modes of the 2D and 3D soil-bridge model.

Figure 3 shows the most significant longitudinal vibration modes in terms of participating mass of the soil-bridge domain and particularly of the embankment, whose values are reported in Table 1. The similarity of the first and second vibration modes (Figs. 3a, b) of the 1D soil column with the significant modes of the full soil-bridge model demonstrates that the dynamic response of the bridge is mainly dominated by the surrounding soil. This can be ascribed to a double effect: the high mass participation of the soil compared to the structural elements and the structural continuity between the abutments imposed by the deck. In fact, as reported in Table 1, the participating mass of the 1D soil column involved in each mode is close to the one of the full 2D and 3D model, in the order of 72–79% and 10–15%, respectively. A clear correspondence is also provided in terms of vibration periods, where the first one is governed by the fundamental mode of the deposit while the second one is essentially correlated to the presence of the embankment.

Note that, although the fundamental vibration mode is nearly identical among the considered models, the second modal shape of the sole soil deposit splits into the combination of three modes in the 3D soil-bridge model, mainly due to the finite transverse extension of the embankment altering the free-field response. Nevertheless, in the 3D model the sum of the participating masses of the embankment (about 56%) is close to the one of the 1D soil column (62%) and of the 2D model (57.4%), proving that the dynamics of the embankment can be described sufficiently well by the two first modes of the reduced-order models (1D and 2D).

Overall, the modal analysis demonstrated that the modal shapes are influenced by the dimension of the full model, with formation of multiple modes for each mode of the reduced-order representations due to the irregular, three-dimensional geometry of the domain. Therefore, for a clear identification of the dominant vibration modes a design strategy should refer to the essential features of the geometry of the system. To this end, for such structural typology, the simple 1D soil column appears as a promising candidate, but the modal response of the 2D model still needs to be considered as a reference for the development and validation of a seismic design method for IABs, as recounted in the following sections.

Table 1. Results of the modal analysis in terms of period and participating mass of the full soil-bridge system and embankment for the 1D soil column, 2D and 3D soil-structure models.

1D soil column				2D				3D			
Mode	T (s)	m_{tot} (%)	m_{emb} (%)	Mode	T (s)	m_{tot} (%)	m_{emb} (%)	Mode	T (s)	m_{tot} (%)	m_{emb} (%)
1	0.63	72.9	31.4	1	0.62	74.6	29.1	1	0.57	78.7	15.0
2	0.24	15.3	30.4	4	0.27	10.2	28.3	2	0.31	2.2	14.8
								6	0.24	0.9	13.7
								7	0.23	8.2	12.7
tot		88.2	61.8			84.8	57.4			90.0	56.2

4 Proposed Design Method for IABs

4.1 Methodology

Nonlinear dynamic analysis cannot be regarded as a routinary approach at the design level because of the high computational effort. A simplified design procedure is therefore proposed for a direct use in the assessment of the seismic performance of IABs in the longitudinal direction, devised on the basis of the salient dynamic features of the soil-bridge system pointed out in Sect. 3. In the proposed method, the Capacity Spectrum Method (CSM) [1, 2] is used in conjunction with a nonlinear static analysis (NLSA) of the soil-bridge system. The method requires two basic ingredients, namely: the capacity curve of the system and the seismic demand, the latter in the form of the acceleration-displacement elastic response spectrum (A-D response spectrum).

The capacity curve of the system is obtained by applying equivalent forces to all nodes of the model. These forces are proportional to a seismic coefficient k_h, representing the ratio of the horizontal body forces to the unit weight of the soil. In OpenSees, the body forces were retrieved through the implementation of an automatized calculation of the tributary mass for each node. Then, the equivalent forces were applied to all nodes as a succession of static analyses using the LoadControl integrator and the Newton-Raphson algorithm to solve the nonlinear residual equation. The amplitude of k_h increases progressively during the analysis until the attainment of the ultimate capacity of the soil-bridge system. The spatial distribution of k_h follows the two trends in Fig. 4, that reproduce the deformation patterns associated with the dominant vibration modes of the soil-bridge system. The resulting capacity curves for the system at hand are shown in Figs. 6b,e, in which k_h and the longitudinal displacement u refer to the top of the abutment.

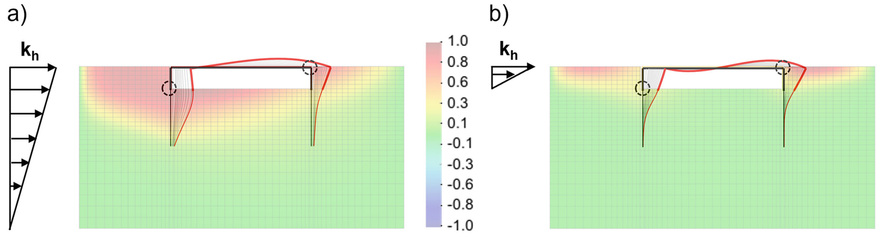

Fig. 4. Representation of the assumed profiles of the longitudinal seismic coefficient k_h and of the resulting deformed shapes of the bridge, associated with the a) first and b) second dominant mode.

The seismic demand is characterized by the A-D spectrum of the seismic motion of the embankment top, the latter deriving from a one-dimensional site response analysis including the presence of the embankment. The equivalent damping ratio ξ of the A-D spectrum is found by an iterative procedure, i.e., by evaluating the damping ratio spectrum (using the Masing unloading-reloading rule) at the intersection of the capacity curve, called *performance point*, and re-plotting the A-D spectrum accordingly with the calculated damping until the computation of ξ stabilises.

4.2 Seismic Input

As a limited validation of the proposed design method, two compatible seismic records with the seismo-tectonic setting at the bridge location, referred in the following to as the Yamakoshi and Parkfield records, were selected from the PEER ground motion database (https://ngawest2.berkeley.edu/). These records represent the seismic input for the models in Fig. 2 and present a very different frequency content, chosen to involve alternately the dynamic contribution of the dominant vibration modes of the considered system (modes no. 1 and 2 of the 1D model in Fig. 3). The main parameters of the records reported in Table 2 are listed in the following: peak ground acceleration of the records, *PGA*, the Arias intensity I_A, the significant duration T_s (between 5% and 95% of the Arias intensity), the mean quadratic period T_m, the moment magnitude M_w of the earthquake, and the Joyner-Boore distance R_{JB} of the recording station.

Figures 5.a,b show the initial and the final (at convergence) A-D spectra obtained at the top of the embankment through a ground response analysis on the 1D model, for both the imposed deformed shapes (Fig. 4) and input motions. Once convergence is attained, the current performance point, shown in Figs. 5.b,e, provides the maximum acceleration and displacement of the abutment top. The ordinate of the performance point can be also used to determine the maximum, seismic-induced bending moment in the structural elements of interest, recorded in the pushover analysis (Figs. 5.c,f), that are the deck abutment node and the pile head.

Table 2. Main properties of the selected input motions

No	*Record*	*Name event*	*PGA* (g)	I_A (m/s)	*Ts* (s)	T_m (s)	*Vs 30* *(m/s)*	M_w (-)	R_{jB} (km)
1	RSN 4868	Yamakoshi scaled 50%	0.20	1.03	13.6	0.66	655	6.8	22.2
2	RSN 4064	Parkfield	0.35	0.78	4.9	0.23	657	6.0	4.3

4.3 Validation

The proposed design method (NLSA) is validated against the results of nonlinear dynamic analyses (DYN) carried out on the 2D soil-bridge model for the selected seismic scenarios, while the reader can refer to [8, 15, 16] for a comprehensive application of the method. The maximum, seismic-induced values of i-ii) the bending moment at the deck-abutment node and piles top, and iii) of the displacement of the deck obtained with the NLSA and DYN approaches are compared in Fig. 6. For both seismic scenarios, the maximum bending moments at the deck-abutment node given by DYN are bounded by the two NLSA solutions (first and second k_h distribution), demonstrating the important role played by the dynamic response of the embankment. The NLSA solution based on the first mode appears instead useful for evaluating the internal forces in the piles. This is because the latter are embedded in the foundation soils and therefore the effects of the first modal shape of the system (i.e., fundamental mode of the soil deposit) tend to dominate. Finally, the maximum displacement of the abutment top is quite well captured by both k_h distributions.

The results discussed above show promising predictive capabilities of the simplified method, that involves a much lower computational effort if compared with a full dynamic analysis: the proposed NLSA implies indeed a limited effort in the implementation of the numerical analysis and a low computational demand (two static analyses instead of a series of dynamic analyses, no selection of seismic records, no necessity of additional damping sources).

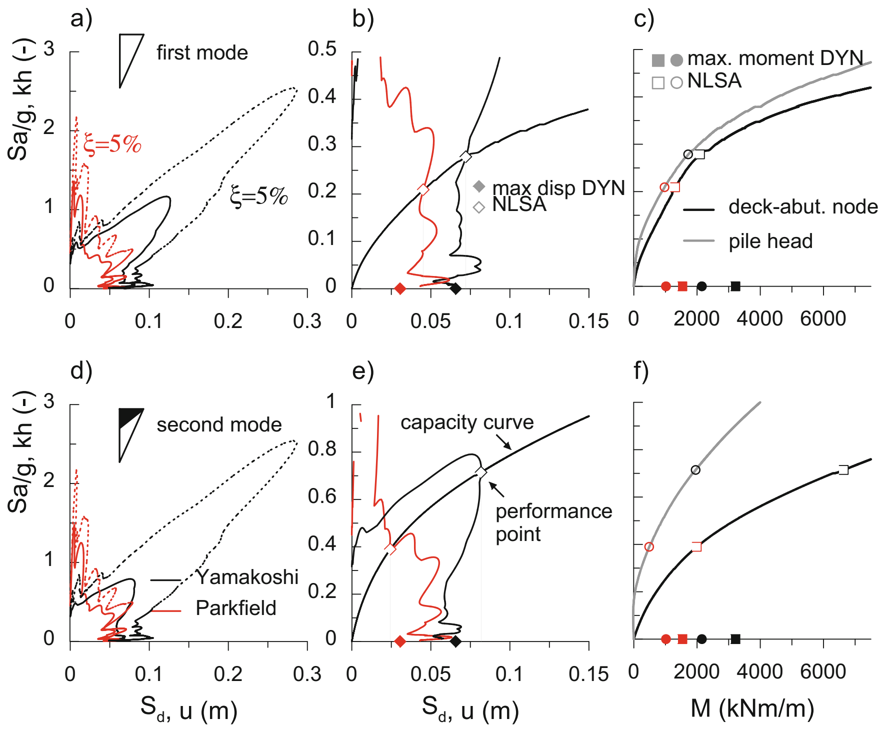

Fig. 5. Application of the NLSA: a,b) AD spectra before and after the iterations, b,e) evaluation of the performance point; c,f) moment-seismic coefficient (k_h) curves and evaluation of the maximum forces in the structural elements

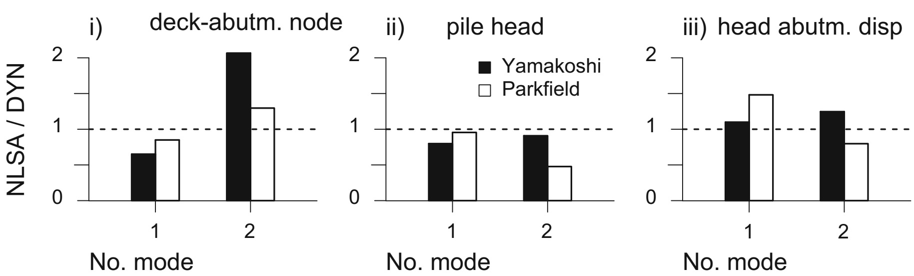

Fig. 6. Ratios of the maximum, seismic-induced bending moment at the deck-abutment node (i), pile head (ii), and of the maximum displacement of the deck (iii) evaluated with the NLSA to the ones obtained with the dynamic analyses (DYN).

5 Conclusion

The results above show that the proposed design method can be efficiently used for a direct assessment of the seismic performance of an integral abutment bridge, as a structural typology whose dynamic response is mainly controlled by the modal features of the soil. More in general, by virtue of its robust conceptual framework, the presented methodology may be straightforwardly generalized to different soil-structure systems. On the one hand, the imposed force pattern associated with the first mode is seen to be poorly affected by the structural typology and, in some cases such as embedded retaining walls and excavations [5, 17], can be taken as constant in all the domain. On the other hand, the higher order vibration modes are strictly dependent on the case study, whose contribution can be promptly evaluated by using a modal analysis on a 1D or 2D numerical representation of the system.

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