Computers and Structures 155 (2015) 54–66

Contents lists available at ScienceDirect

Computers and Structures

journal homepage: www.elsevier.com/locate/compstruc

Rotational excitation of bridges supported on pile groups in soft or liquefiable soil deposits

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ARTICLE INFO

Article history:
Accepted 15 February 2015
Available online 10 April 2015

Keywords:
Rotational excitation
Kinematic interaction
Soil–structure interaction pile group foundation
Liquefaction

ABSTRACT

This paper investigates the seismic response of reinforced concrete bridges founded on pile groups in soft or liquefiable soil deposits under simultaneous translational and rotational earthquake excitation. The pile cap rotational excitation results from pile bending under vertically propagating seismic S-waves. The response of the pile cap (in terms of displacement and rotation) defines the Foundation Input Motion (F.I.M.) computed through kinematic interaction analysis. The resulting superstructure demand due to the coupled translational and rotational excitation is compared to the demand developed due to translational only F.I.M. The results confirm earlier observations regarding the impact of pile bending on rigid body rotations of bridges founded with cast-in-drilled hole single piles. It is also shown that the excitation mechanism of pile groups in soft soils is non-uniform along the pile length, thus exciting higher modes of vibration and inducing out-of-phase translational and rotational response of the pile cap.

1. Introduction

The dynamic impedance of a group of vertical piles and the energy radiated due to pile–soil–pile interaction are key factors that affect the seismic response of bridges, as they modify the dynamic characteristics of the system as a whole and introduce additional energy absorption mechanisms at the soil–foundation interface [1]. As this interaction is frequency dependent, the frequency content of the earthquake ground motion and the dynamic characteristics of the superstructure play a predominant role in the response of the soil-bridge system. This is a complex interplay since the natural frequencies of the bridge and the respective contribution of the individual modes, are evolutionary in time due to structural damage and nonlinearities at the soil–foundation interface. From all the above interdependent phenomena, however, the characteristics of earthquake loading have the highest level of aleatoric uncertainty, that is, inherent randomness which is directly propagated to the demand imposed on the soil-bridge system.

Along these lines, significant research effort over the last two decades has been put in defining schemes for selecting and scaling earthquake ground motions in the framework of performance-based design and assessment of buildings [2–4] and bridges [5]. Further research has also investigated the spatially variable nature of seismic waves [6–8] along the bridge length and its effect on the elastic and inelastic response of the coupled soil-bridge system [9]. In the vast majority of the research conducted for bridge engineering purposes, earthquake loading is solely associated with the translational (vertical and horizontal) degrees of freedom of the foundation ground motion; the contribution of the rotational component of earthquake ground motion generated by the bending of the piles within the soil medium under the incident seismic waves is neglected [10–12].

The implications of the rotational response of the pile head, which essentially constitutes an additional component of Foundation Input Motion to the superstructure, have only been studied for the case of bridges founded on cast-in-drilled hole (CIDH) piles [8,13]. It is therein observed that kinematically-induced rotational excitation may increase by an average factor of three, the transverse displacements of the bridge deck, especially in cases of tall piers, soft soil profiles and frequencies associated with the dynamic characteristics of the soil and the superstructure.

Given that the rotational stiffness of a pile group is generally significantly higher compared to CIDH piles, the assumption has been made that the rotational kinematic interaction factor (i.e., $I_\theta = \frac{\vartheta \cdot d}{U_{ff}}$) is close to zero [14], hence naturally, the potential effect of rotational excitation is negligible [15]. Recent research, however,

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http://dx.doi.org/10.1016/j.compstruc.2015.02.013
0045-7949/

has revealed cases [14] where the rotation of the pile group, might have a considerable effect on the seismic demand of the superstructure, particularly in cases of inhomogeneous soil formations.

The scope of this paper is to extend the aforementioned preliminary work and investigate in depth the coupling of two simultaneous, counteracting phenomena: the restraint provided by the pile group rigid cap that reduces the rotational input to the superstructure and the increased foundation rotation that results from significant relative deformations which often occur within soft soil formations. This investigation is made for different soil properties, pier heights and frequency excitation characteristics in the time, frequency and time–frequency domains. The methodological approach adopted, as well as the observations and interpretations made, are presented in the following.

2. Computational scheme

The computational procedure presented herein utilizes state-of-the-art earthquake engineering software as well as ad hoc developed computer codes tailored to serve the needs of the investigation. The rationale behind the deterministic analysis performed and the tools utilized and/or developed are summarized below, grouped in seven conceptual steps that remain identical in the application examples demonstrated in Sections 3 and 4.

2.1. Soil conditions

As the emphasis of this study is on the dynamic rotation of pile group foundations, two characteristic bounds of subsoil conditions are examined: the case of a uniform, moderate-to-soft soil stratum and the case of a highly non-uniform soil stratification consisting of potentially liquefiable loose sand layers. The above conditions essentially correspond to ground motions that are either uniform along the pile length, or vary significantly with depth, thus inducing negligible or considerable relative displacements, respectively, to the foundation piles.

2.2. Structural configuration and numerical modeling

Two different bridge configurations are selected to conform with the aforementioned distinct soil conditions; one extensively studied in the literature [16,17] and one typical overpass, already constructed along a major national highway in Greece. Both structures have been realistically re-designed to the standards of Eurocode 7 [18] and Eurococe 8, Part 5 [19] with a 2×2 and 4×4 pile group, respectively. Detailed description of the structural system is provided in Sections 3.2 and 4.2 together with the assumptions of the numerical analysis conducted with the finite element software SAP2000 [20] after extensive verification with analytical solutions, where appropriate.

2.3. Selection of earthquake ground motion

Two sets of ground motions have been employed in this study: (a) harmonic motions within the frequency range [0.5, 20 Hz] at a step of 1 Hz and amplitude 0.24 g, and (b) twelve recorded ground motions with peak ground acceleration ranging within 0.13–0.36 g. The latter set of recorded motions is scaled to 0.24 g, which is the level of the design acceleration in the zone of moderate seismicity in Greece. An effort was made to select motions with a breadth of characteristics in terms of magnitude, frequency content and source-to-site distances to cover both strong, far-field events and moderate intensity near-field excitations. Since the records were selected for the purposes of site response analysis initiating from bedrock level, only motions recorded on outcrop conditions or stiff soil formations (soil class A according to Eurocode 8) were selected. The small size of the earthquake record suite developed is due to the deterministic nature of the approach followed; clearly, a significantly larger number of earthquake records would be required in the framework of a probabilistic assessment. A summary of the characteristics of the ground motions used in this study is provided in Table 1.

2.4. Site response analysis

One-dimensional, linear and non-linear, site response analyses were performed for the two sites using the aforementioned sets of ground motions. For the first case of a linear elastic, uniform soil profile, the assumption can be made that the equation of motion of an SH wave which vertically propagates at a velocity V_s through a Kelvin-Voigt soil of viscosity η.

$$\frac{\partial^2 u}{\partial t^2} = V_s^2 \frac{\partial^2 u}{\partial z^2} + \frac{\eta}{\rho} \frac{\partial^3 u}{\partial t \partial z^2} \tag{1}$$

has a solution that can be written in the form of an upward (first term) and a downward (second term) travelling wave, with amplitudes A and B which depend on boundary conditions:

$$u(z,t) = Ae^{i(\omega t + k^* z)} + Be^{i(\omega t - k^* z)} \tag{2}$$

where $k^* = \omega\sqrt{\rho/(G + i\omega\eta)}$ is the complex wave number coordinate and z is the spatial coordinate which points downward. In such a case and for any given motion, the transfer functions between the surface of a multi-layer damped soil profile laying over an elastic bedrock, are derived using the 'reflectivity coefficient' algorithm in which all multiple reflections and conversions between wave types are retained in part of the soil structure [21].

For the case of soft, liquefiable soil, the analysis was conducted with the multi-yield-surface plasticity model developed at U C San Diego and incorporated into the computer program Cyclic-1D.The model is calibrated to the experimentally observed accumulation

Table 1
Selected earthquake ground motions.

ID	Earthquake event	Date	M_w	Station	Source-to-site distance (km)	PGA (g)
1	Loma Prieta	18/10/1989	7.1	1652 And. Dam (Downstream)	21.4	0.244
2	Northridge	17/1/1994	6.7	24389 LA-Century City CC North	25.7	0.222
3	Kern County	21/7/1952	7.7	1095 Taft Lincoln School	41	0.178
4	Cape Mendocino	25/4/1992	7.1	89509 Eureka–Myrtle & West	44.6	0.178
5	San Fernando	9/2/1971	6.6	126 Lake Hughes #4	24.2	0.192
6	Landers	6/28/1992	7.4	23559 Barstow	36.1	0.132
7	Imperial Valley	15/10/1979	6.9	6604 Cerro Prieto	26.5	0.169
8	Taiwan	11/14/1986	7.8	29 SMART1 M07	39.0	0.160
9	Superstitn Hills	11/24/1987	6.6	5052 Plaster City	21.0	0.121
10	Northridge	17/1/1994	6.7	24303 LA-Hollywood Stor FF	25.5	0.358
11	Loma Prieta	18/10/1989	7.1	1678 Golden Gate Bridge	85.1	0.233
12	Livermore	27/1/1980	5.5	57T02 Livermore–Morgan Terr Park	8.0	0.198