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NONLINEAR SEISMIC RESPONSE OF A BRIDGE SITE SUBJECT TO SPATIALLY VARYING GROUND MOTION

Zhaohui Yang [1], Associate Member ASCE
Liangcai He[1]
Jacobo Bielak[2], Member ASCE
Yuyi Zhang[1]
Ahmed Elgamal[1], Member ASCE
Joel Conte[1], Member ASCE

ABSTRACT

This paper studies the effects of spatially varying input excitation at an existing bridge site. A nonlinear finite element model of this bridge site, including the bridge structure, pile groups, and the supporting foundation soil, is developed under 2D plane-strain conditions. The computational model is analyzed using OpenSees, a software platform developed at the Pacific Earthquake Engineering Research Center. Carefully calibrated nonlinear stress-strain models are employed for both bridge and soil materials, in order to realistically reproduce actual site conditions. Seismic input motions are defined as equivalent excitation forces using the Domain Reduction Method. In this study, the input excitation is prescribed at different angles of incidence to examine the significance of spatially varying excitation on the overall response of the bridge-foundation system.

Keywords: Bridge, earthquake response analysis, soil-pile-structure interaction, nonlinear finite element analysis, soil plasticity

INTRODUCTION

Model-based experimental and numerical simulations of seismically induced site effects commonly assume a uniform base excitation (either incident or total). This assumption may not be realistic for structures whose horizontal dimensions are comparable to the wavelength of foundation ground motions. In this paper, we present some preliminary results of a computational study on the effects of spatially varying input excitation at an existing bridge site. The bridge, Humboldt Bay Middle Channel Bridge, is located near Eureka in northern California. This bridge

[1] Department of Structural Engineering, University of California at San Diego, La Jolla, CA92093. E-mails: zhyang@ucsd.edu, lhe@ucsd.edu, y8zhang@ucsd.edu, elgamal@ucsd.edu, jpconte@ucsd.edu

[2] Department of Civil and Environmental Engineering, Carnegie Mellon University, Pittsburgh, PA 15213. E-mail: bielak@cs.cmu.edu

site was selected by the Pacific Earthquake Engineering Research (PEER) Center as one of the testbeds for synthesizing the next-generation Performance-Based Earthquake Engineering (PBEE) methodologies being developed at the PEER center.

In the following, we first briefly describe the structural and geotechnical conditions of the bridge site, the finite element model, and definition of input motion. Thereafter, the computational results are presented and discussed.

SITE DESCRIPTION

The Middle Channel bridge was designed in 1968 and built in 1971, and has been the object of two Caltrans (California Department of Transportation) seismic retrofit efforts. The bridge (Fig. 1) is a 330 meters long, 9-span composite structure with precast-prestressed concrete I-girders and cast-in-place concrete slabs and piers to provide continuity. It is supported on eight pile groups, each of which consists of 5 to 16 prestressed concrete piles (Fig. 1). The foundation soil (Fig. 1) is composed of mainly medium-to-dense sand (SP/SM), organic silt (OL), and stiff clay layers. In addition, loose silty sand (OL/SM) layers are present near the ground surface. The bedrock is located at 220 m depth. Available logging data shows the site shear wave velocity profile varying from about 150 m/s at ground surface to 600 m/s at 220 m depth, and exceeding 900 m/s in the underlying bedrock.

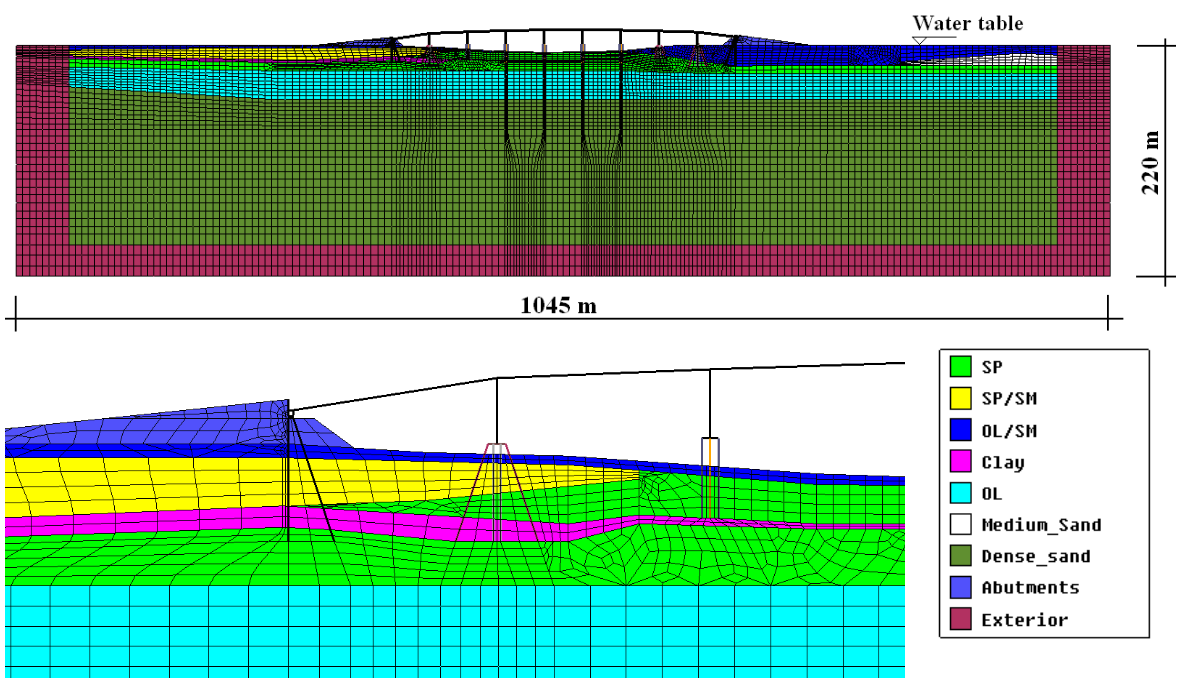

FIG. 1. Finite element model of the Humboldt Bay, Middle Channel bridge site and close view of the bridge structure.

FINITE ELEMENT MODEL

The computational model was constructed using the new software framework OpenSees (McKenna and Fenves, 2001) developed at the PEER center. OpenSees is an open-source finite element computational platform that combines advanced structural and geotechnical seismic response simulation capabilities. In the current model (Fig. 1), the bridge piers are modeled using 2D nonlinear material, fiber beam-column elements. The superstructure and pile groups are modeled as linear elastic beam-column elements. A more detailed description about modeling of the bridge structural components can be found in Conte *et al*. (2002) and Zhang *et al*. (2003).

The soil domain was analyzed under plane strain conditions. It was divided into the interior and exterior regions (Fig. 1), which were discretized spatially using four-noded, bilinear, isoparametric finite elements. In the interior region, the various layers of the supporting soil medium were modeled using an effective-stress, cyclic-plasticity constitutive model (Elgamal *et al*. 2002, Yang *et al*. 2003). This soil stress-strain model has been extensively validated and calibrated for both drained (or dry) and undrained conditions through numerous sources including laboratory tests (Arulmoli *et a*l., 1992; Kammerer et al., 2000), centrifuge experiments (Dobry and Taboada, 1995), and downhole-array seismic records (Elgamal *et al*., 2001). Soils below the water table (Fig. 1) are modeled as undrained materials, and soils above as dry materials. Different sets of material constitutive parameters are used for the various soil types in the supporting soil medium.

In the exterior region, the material was specified as linear elastic, with a shear wave velocity of 450 m/s (average velocity from ground surface to 220 m depth) and a compressional wave velocity of 1500 m/s (typical value for pore water). All the boundary nodes (along the lateral sides and the base of the model) were connected to Lysmer-type dashpots for absorbing outgoing waves. In addition, constant forces were specified at the boundary nodes to balance the model self weight.

SPECIFICATION OF INPUT EXCITATION

To specify input motions at an arbitrary angle of incidence, the Domain Reduction Method (DRM) developed by Bielak *et al*. (2003, see also Yoshimura *et al*. 2003) was implemented in OpenSees. Using this method, the input excitation was defined as equivalent nodal forces along the boundary of the interior and exterior regions. The equivalent forces may be computed independently of the finite element computations. In this study, these forces were obtained analytically assuming an incident plane SV wave in a homogenous, linear elastic half space, at two different incident angles: 0 degree (vertical incidence) and 30 degrees, respectively. A Ricker wavelet (Fig. 2) was used as the incident wave, with a maximum acceleration of 2 m/s/s and a peak frequency of 2 Hz.

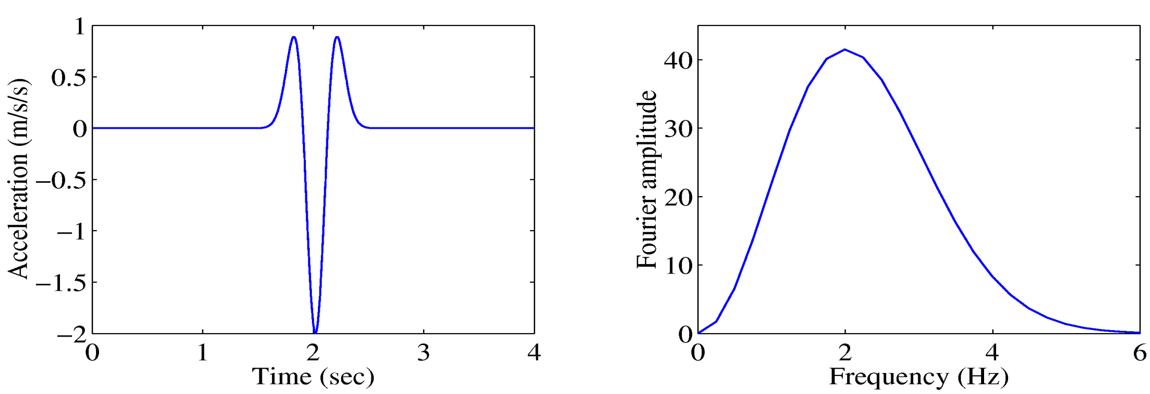

FIG. 2. Incident motion time history and its Fourier amplitude.

RESULTS AND DISCUSSION

Fig. 3 shows a snapshot of deformed mesh in the case of vertical incidence, at the instant of 2.33 seconds. This figure indicates that: 1) all bridge piers deformed in phase, and 2) minor surface waves were generated at the interior-exterior interface due to the material non-uniformity across the boundary. Fig. 4 shows the deformed mesh at the instant of 1.75 seconds, in the case of 30-degree incidence. This figure indicates that: 1) motions of the bridge piers were out of phase, and 2) significant spurious surface waves were generated at the interior-exterior interface. Note that the critical angle for this site is very small, as the shear wave velocity near the ground surface (150 m/s) is only a small fraction of the compressional wave velocity (1500 m/s). This gives rise to strong surface waves within the interior region.

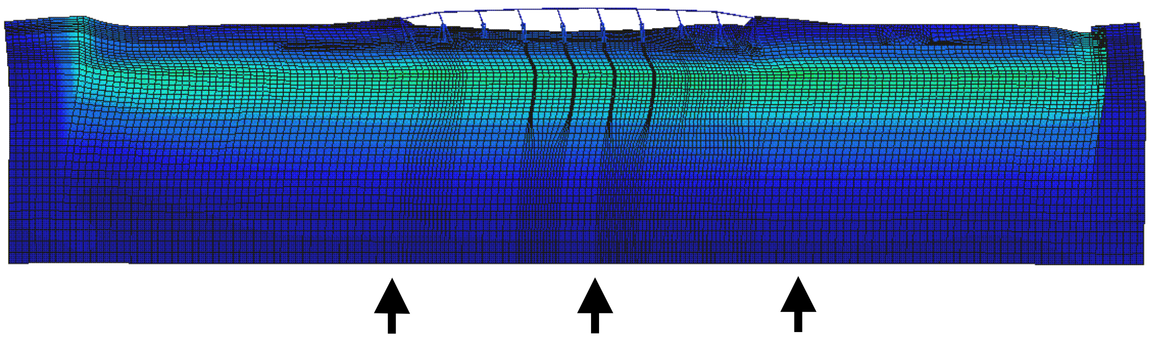

FIG. 3. Snapshot of deformed mesh at 2.33 seconds, in the case of vertical incidence (deformations were amplified by 500 times for clarity).

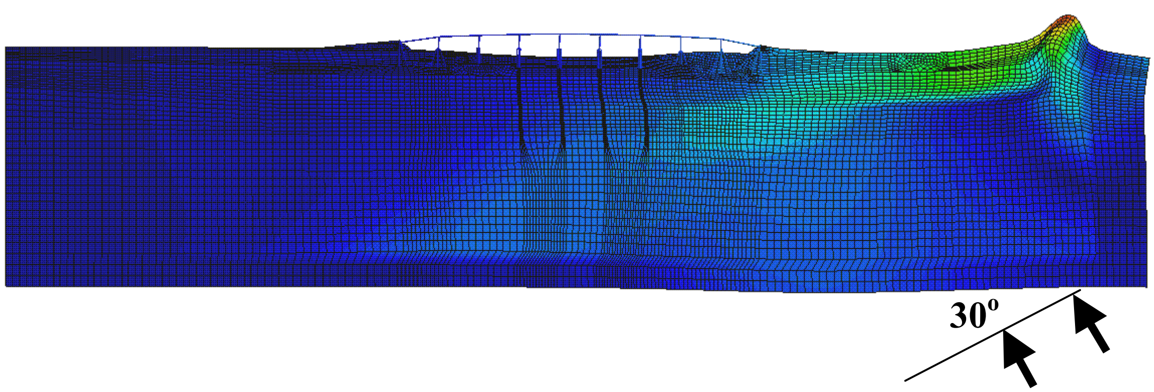

FIG. 4. Snapshot of deformed mesh at 1.75 seconds, in the case of 30-degree incidence (deformations were amplified by 500 times for clarity).

Fig. 5 depicts moment-curvature response at the bottom of bridge piers for both cases. In both cases, all the piers experienced minor nonlinear response. However, in the case of vertical incidence, the moment-curvature curves were very similar; whereas in the case of inclined incidence, the moment-curvature curves showed noticeable deviations (due to greater spatial variation of excitation). In this figure, the superposed dots indicate the instantaneous moment-curvature response in the piers when the rightmost pier was experiencing the largest moment. In the case of vertical incidence, all piers reached maximum moment simultaneously; whereas in the case of inclined incidence, a phase delay is clearly seen among the piers. Again, the phase differences were due to the significant spatial variation in excitation.

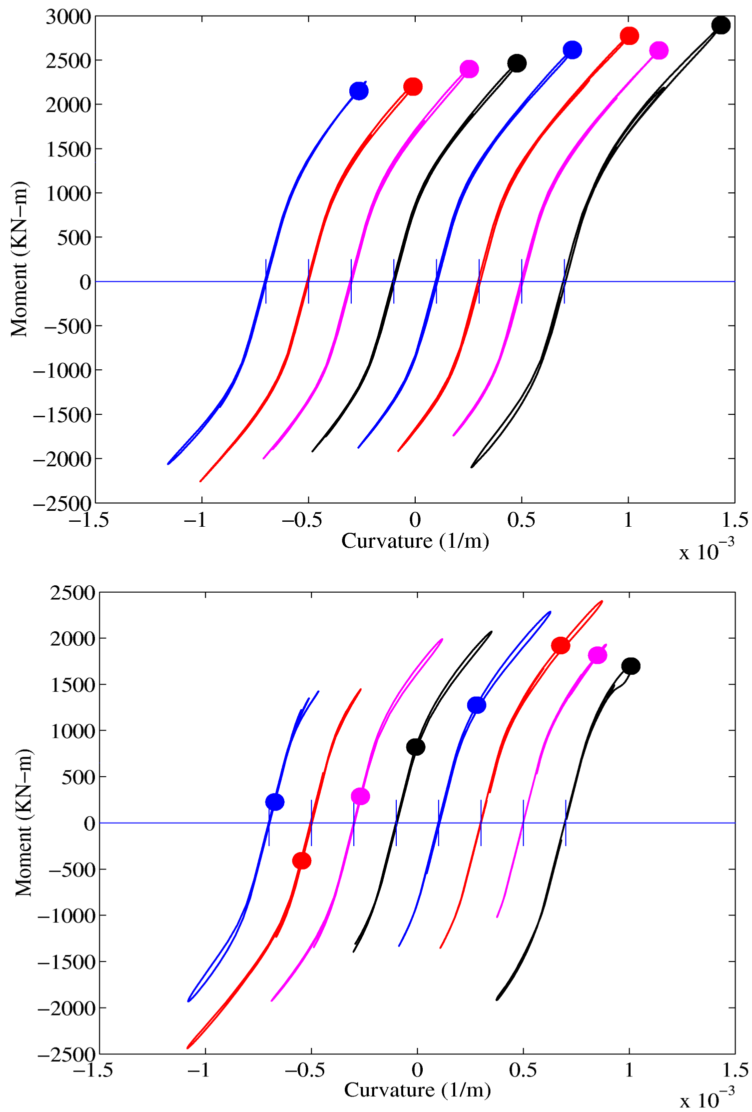

FIG. 5. Moment-curvature response at the bottom of bridge piers (the abscissas are shifted for clarity).

Fig. 6 shows soil shear stress-strain response at various depths along the model centerline. The extent of nonlinear response was seen to be dependent on both depth (or confinement) and soil type. In general, for the same soil type (e.g., dense sand in Fig. 6), more significant nonlinear response occurred at shallower depths. Moreover, silt material exhibited severer nonlinearity than the dense sand.

Discussion

The assumption of a uniform exterior region is not quite consistent with the vertically varying soil properties of the interior region. A more realistic approach is to represent the exterior domain as a layered half space with properties consistent with the interior region. This is a research effort currently underway.

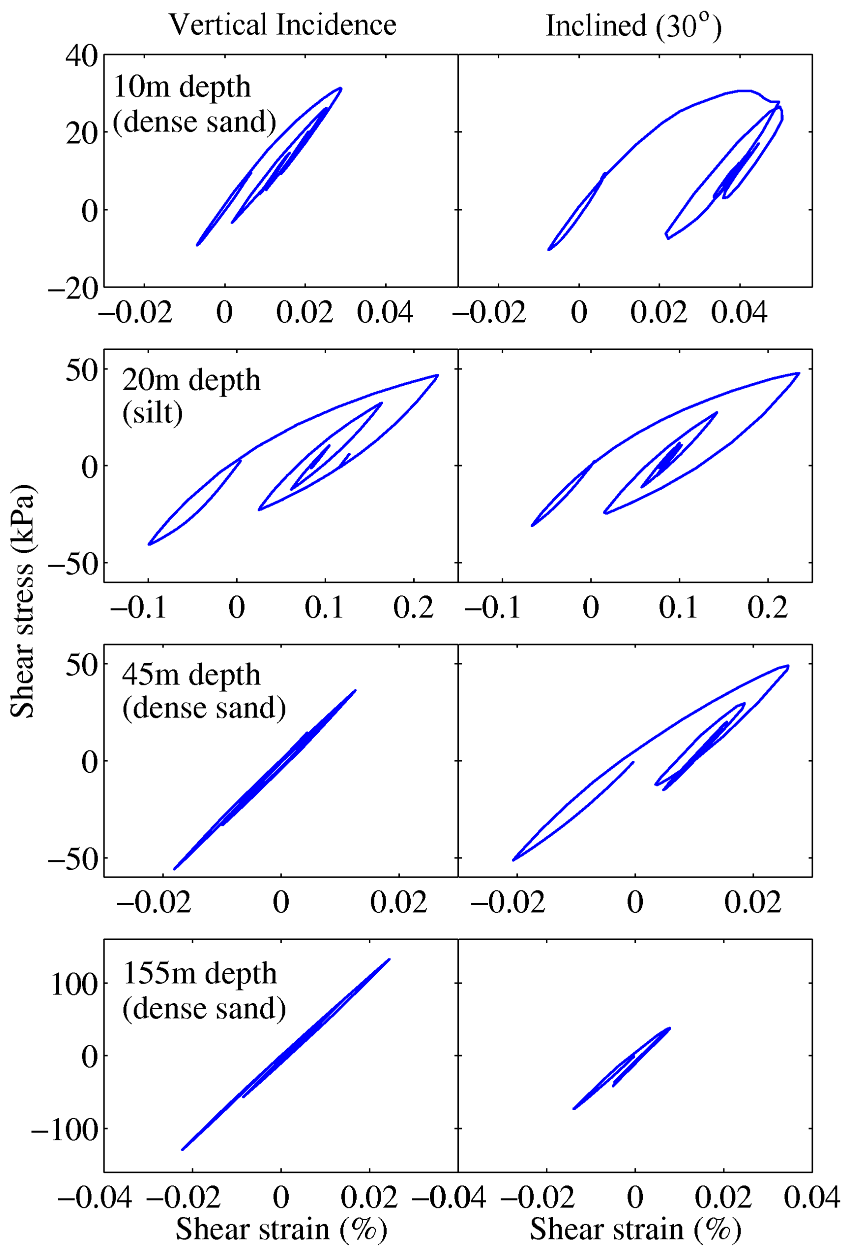

FIG. 6. Soil shear stress-strain response along the model centerline.

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