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Effect of numerical soil-foundation-structure modeling on the seismic response of a tall bridge pier via pushover analysis

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ABSTRACT

This paper examines the role of the numerical modeling of soil-foundation-structure (SFS) interaction on the seismic response of a tall, partially embedded, flared bridge pier. For this purpose, static, pushover, nonlinear, finite-element, stand-alone analyses are performed on nine different models of one of the two piers of the Mogollon Rim Viaduct, a long-span, reinforced-concrete bridge supported on pile foundations. Structural modeling considerations, such as selection of concrete constitutive models, material properties, and bond-slip and P-Δ effects, on the nonlinear response of this pier are investigated. *p-y*, *t-z* and *Q-z* nonlinear curves are applied to model the soil-pile interaction, and equivalent nonlinear springs are developed to reproduce the soil-pile cap interaction. In addition, the effects of the partial pier embedment and the slope of the ground surface on the lateral resistance of the pier and the total capacity of the SFS system are examined. The results illustrate how structural and geotechnical modeling approaches for the SFS interaction can affect the nonlinear response of tall bridges, and may lead to differences in the numerical prediction of local or global failure. For the case analyzed herein, the partial pier embedment and foundation flexibility can dramatically modify the structural response, and influence the bond-slip effect at the pier-pile cap connection.

1. Introduction

One of the challenging problems in earthquake engineering is the consideration of the soil-foundation-structure (SFS) interaction on the seismic behavior of structures. For single-support structures, the SFS interaction changes the dynamic properties of the system (dynamic effects) and modifies the foundation input motions (kinematic effects) [1,2]. For multiple-support structures, in addition to the two aforementioned effects, it may induce significant pseudo-static forces in the structure [3–5], and results in amplification of the contribution of the higher modes to the seismic response. As a consequence, the consideration of an appropriate finite-element model for the SFS system of long-span bridges, which are most sensitive to pseudo-static excitations [6], plays an important role in their reliable seismic assessment [4]. At the present time, this consideration is still an issue for the safe seismic design of bridges supported on pile foundations because: (1) the provisions of bridge design codes, such as AASHTO [7] or Caltrans [8], do not fully address the modeling of this type of foundation system, e.g. no specific recommendation is proposed for modeling the soil-pile cap interaction, and (2) limited studies have been conducted on the seismic response of pile-supported bridge piers [1,9–11]. The present study focuses on the model uncertainty in the numerical representation of bridge piers supported on pile foundations. For this purpose, the seismic behavior of a tall, partially-embedded, flared, reinforced-concrete pier supported on a pile foundation with a special configuration of its pile cap is parametrically investigated.

The SFS interaction can significantly affect the seismic response of bridges supported on soft soil. It may influence beneficially the seismic behavior of bridges by increasing their structural damping and decreasing the seismic forces [2]. On the other hand, it may be detrimental for bridges due to resonance, amplified P-Δ and pounding effects, that can cause overturning and unseating of the superstructure [12,13]. It is well known that the SFS interaction may shift the fundamental frequency of the bridge closer to (farther from) the prevailing frequency of the seismic excitation, which causes increase (decrease) of the structural response [2]. Regarding P-Δ and pounding effects, the rocking movement of the pile cap due to the SFS interaction is especially important for tall bridges. It should also be noted that the bridge superstructure may be subjected to considerable torsional moment due to the foundation rotation about its vertical axis during earthquakes caused by: (1) seismic loading conditions, as, e.g., bilateral seismic loading, torsional components of earthquake motions [14,15], and spatial variation of the ground motions [6,15]; (2) foundation

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conditions, as, e.g., non-uniform properties of the soil surrounding the foundation, geometrical configuration of the foundation, and partial damage or failure of the foundation [16]; (3) structural configuration, as, e.g., when the mass center and the center of rigidity of the structural system do not coincide [14] , and (4) collision of the bridge superstructure with abutments [17,18] or adjacent spans [19]. Because of deck rotation about the vertical axis, the induced transverse movement may also lead to pounding and unseating of the superstructure.

The pile foundations may resist seismic forces with a combination of four components: (1) pile-soil-pile interaction; (2) passive earth pressure on the sides of the pile cap; (3) frictional forces at the base and sides of the pile cap, and (4) vertical resistance at the base of the pile cap. Various methods have been proposed for numerical modeling of the pile foundation [20–23]. Commonly, nonlinear *p-y*, *t-z* and *Q-z* curves are used to approximate the resistance of single piles buried in soil [24], and pile group effects are considered by various methods such as the p-multiplier method [25]. Even though there are many studies on the seismic behavior of single piles and pile groups, e.g. [3,4] and [26–29], there are only few investigations related to the lateral and vertical behavior of the soil-pile cap system during seismic loading [30,31]. In earthquake engineering applications, it is generally assumed that either the pile cap behaves like an equivalent thick pile or its resistance is conservatively ignored in pile foundation modeling. This lack of consideration of the pile cap contribution to the response may be partly due to the fact that the pile caps are usually located on the ground surface or very close to the ground surface and surrounded with soft soil. However, recent experimental investigations indicated that heavily buried pile caps may considerably contribute to the lateral resistance of pile foundations [22,23].

There are six different aspects that may affect the numerical modeling of the SFS system in finite-element analyses. These include: (1) structural modeling, such as mesoscopic or macroscopic finite-element modeling approaches, and structural collapse criteria; (2) idealization of the mechanical behavior of structural materials, such as concrete and steel; (3) foundation modeling, such as direct or substructure (i.e., equivalent or Winkler springs) methods; (4) soil modeling and soil failure (or yield) criteria, such as, e.g., the Coulomb-Mohr criterion; (5) idealization of soil properties and soil profile, such as the variation of the cohesion, friction angle and lateral stress ratio of the soil with depth, and (6) boundary conditions, such as gaps between the ends of the bridge deck and the abutment, soil-foundation and soil-abutment interface, and absorbing boundary conditions. The appropriate consideration of each one of these factors in modeling the SFS interaction is a challenging problem due to the uncertainties associated with the numerical representation of the system. Simplified models cannot accurately reproduce the actual nonlinear interaction of a SFS system due to increased model uncertainty, and sophisticated ones may result in an unreliable response due to increased parameter uncertainty [32]. Therefore, the best numerical representation of a SFS system is one that optimizes the incorporation of essential geotechnical and structural components in the finite-element model taking also into account their corresponding parameter uncertainty.

In this paper, a detailed finite-element analysis on one of the piers of the Mogollon Rim Viaduct is performed by considering both geotechnical and structural modeling aspects of the SFS interaction to investigate the effect of the model uncertainty on the system response. The pier, a tall, partially embedded, flared, reinforced-concrete column, is modeled using three-dimensional, force-based, fiber-section, beam-column elements [33]. The bond slip at the pier-pile cap connection as well as P-Δ effects are included in the structural modeling. The influence of two well-known confined concrete models, i.e., the Mander [34] and the Kent-Scott-Park [35,36] concrete models, on the SFS system is studied. The soil-foundation interaction is modeled using *p-y*, *t-z* and *Q-z* nonlinear curves. The frictional resistance and passive earth pressure effects on the embedded part of the pier and the pile cap are considered. The significance of the natural slope of the ground surface on the lateral resistance of the pier and the capacity of the SFS system are investigated. The variations of the maximum moment and shear distribution along the pier and the pile foundation are examined. The effect of the loading patterns on the pushover response of the SFS system is also evaluated. Finally, the effect of the uncertainties in the specified vs. the actual compressive strength of the concrete on the pier response is investigated.

2. Model description

The Mogollon Rim Viaduct (built in 1991) is located on SR 260, which is the primary roadway between the rural towns of Payson and Heber in Central Arizona. It is a three-span bridge with total length of 277.36 m [910 ft] (with spans of 85.34 m [280 ft], 103.63 m [340 ft] and 85.34 m [280 ft]) and width of 18.59 m [61 ft] as shown in Fig. 1(a). Since the maximum span of the bridge is longer than 90 m, the bridge cannot be considered as “ordinary standard bridge” according to the Caltrans provisions [8]. The bridge superstructure is a precast, prestressed concrete, continuous girder, on a curve, with an uphill grade. The bridge has wing walls on the down-slope side of the two abutments, and extensive foundations for the two center piers, which are located on a natural slope.

The clear height of the bridge piers is 15.97 m [52.4 ft] and 20.79 m [68.2 ft] (first and second pier in Fig. 1(a), respectively). The cross section of the piers changes with increasing height and has a flared shape. The base dimensions of the piers are 2.74 m $\times$ 5.48 m [9 ft $\times$ 18 ft] and their top dimensions 2.74 m $\times$ 8.22 m [9 ft $\times$ 27 ft]. The first pier is rigidly connected to the superstructure, whereas the second, taller pier is seismically isolated from the deck with three elastomeric bearings (Fig. 1(a)). The elastomeric pads have the same properties, which are: thickness 5.08 cm (2 in.), cross-sectional area 9000 cm^2, shear modulus 930 kN/m^2, and lateral stiffness 167 kN/cm. Because the Mogollon Rim Viaduct is a long-span bridge, the tributary dead load corresponding to each pier (Fig. 1(a)) is large (33,361 kN). Hence, the total frictional resistance of these three elastomeric pads becomes large as well, and is evaluated by the product of the dynamic coefficient of friction between concrete and neoprene and the axial force of the pier due to the tributary dead load of the superstructure: $0.4 \times 33{,}361$ kN = 13,344 kN. This frictional force is much greater than the total yield resistance of the elastomeric bearings, which is 7635 kN. Therefore, in this design, the three elastomeric bearings are not dissipative friction devices, but are just used to partially release the rotational constraints at the top of the second pier of the bridge.

The effective height of the first pier, i.e. distance from the mass center of the superstructure to the base of the pier, of the Mogollon Rim Viaduct is approximately 4.57 m taller than its clear height (15.97 m) and is equal to 20.54 m. However, the effective and clear heights of the second pier of the Mogollon Rim Viaduct are the same and equal to 20.79 m. In addition, the top of the first pier can be practically assumed as rotationally free [8] in its strong direction. Therefore, its behavior becomes very close to the second pier, which is seismically isolated along this direction, because both piers have approximately the same cross section, effective height and boundary conditions. For the aforementioned reasons, the response of the bridge along its transverse direction is not significantly affected by its irregular configuration and becomes