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Nonlinear Seismic Soil-Foundation-Structure Interaction for Analysis of Bridge Systems

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ABSTRACT

Bridge designers have adopted simple approximate methods to take into account soil-structure-interaction (SSI) in dynamic analysis of bridge systems. The most popular one is the substructuring method in which the response of the foundation soil and its interaction with the pile foundation and the abutment system are represented by a set of one-dimensional springs and dashpots. This paper aims to evaluate the substructuring method and to quantify the level of associated errors for the use in bridge engineering. The baseline data required for the evaluation process is provided by analyzing a fully-coupled validated continuum model of the instrumented two-span Meloland Road Overpass. The bridge system is also simulated using the substructuring method. Results of the two modeling methods are compared for the spectral responses of the bridge, pier deflections, shear forces and bending moments induced at the pier base, and longitudinal and transverse forces induced to the abutments. Comparing the obtained results illustrates several fundamental drawbacks in the substructuring method for predicting the seismic response of bridge systems.

Introduction

In past earthquakes, bridge structures have suffered severe damage mainly because of inadequate seismic design. The bridges could have survived those earthquakes if seismic demands (i.e., deflections and induced forces) were appropriately estimated. This indeed requires detailed numerical modeling of both geotechnical and structural components of the bridge system for which the nonlinear hysteretic response of soil and structure, and their seismic interaction are taken into account. Bridge designers have adopted simple approximate ways of including the effects of soil-structure-interaction (SSI) on the response of a bridge system subjected to earthquake shakings. The most common practical approach is the substructuring method which separates the bridge system into two subsystems: the bridge superstructure which typically includes the bridge deck and the piers, and the substructure which includes the soil-pile group and embankment-abutment systems. The dynamic stiffness of the substructure system is computed separately and is incorporated into the structural model of the superstructure to complete the analysis of the bridge.

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The substructuring method has been widely used in several research studies such as those of Zhang and Makris (2002), Tongaonkar and Jangid (2003), and Shamsabadi et al. (2010) to investigate the seismic performance of bridge systems. However, there has been limited validation of this method (e.g., Zhang and Makris, 2002 and Shamsabadi et al., 2013) where the results are compared with field measurements or those of full-scale analyses. This paper aims to comprehensively test how the substructuring method compares with more exact simulation approaches such as continuum modeling. The adequacy of the continuum modeling method in simulation of seismic SSI has been demonstrated in several studies such as Finn (2005), Kwon and Elnashai (2008), Jeremic et al. (2009), Thavaraj et al. (2010), and Rahmani et al. (2014).

For the work presented in this study, the baseline data required for evaluating the substructuring method is provided by analyzing the 'validated' full-scale continuum model of Meloland Road Overpass (MRO) in California under the two strongest ground motions recorded at the bridge site: the 1979 Imperial Valley and the 2010 El Mayor-Cucapah earthquakes. The bridge system is then simulated using the substructuring method based on the latest state of engineering practice in Caltrans (Shamsabadi, 2013). Both continuum and substructure models of MRO and the corresponding dynamic analyses are created and carried out in the finite element program OpenSees (McKenna and Fenves, 2001). In the following, the continuum and substructure models of the MRO are first described, and thereafter, the spectral responses, shear forces, bending moments and deflections obtained from the two numerical models are compared and discussed.

Description of the Meloland Road Overpass (MRO)

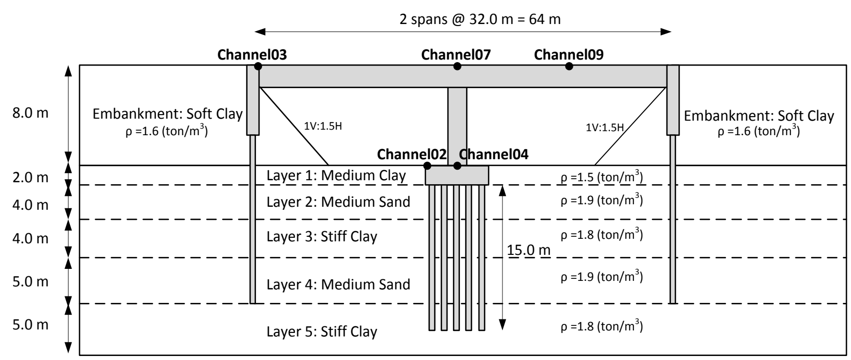


Figure 1: Schematic of the MRO and the channel locations at the site.

The MRO is a two-span integral abutment bridge built in 1971 near El Centro, California, US, as part of Highway 8. The bridge deck has a length of 64.0 m, width of 10.0 m, and thickness of 1.73 m. The box-girder deck has a section composed of four vertical webs, each with a thickness of 0.2 m. There are no expansion joints along the deck. The abutment is of integral type with no deck joints and bearings. The pier at the center of the deck is 5.0 m in height above the ground surface with a diameter of 1.52 m. The embankment soil material is composed of one layer of medium clay for which cohesion of 20.0 kPa and density of 1.6 ton/m^3. The underlying soil is composed of five layers of clays and silty sands. The clayey layers are located at depths of 0-2.7, 6.0-10.7, and more than 15.0 m below the ground surface with cohesions of 35.9, 76.6, and 86.2

kPa, and densities of 1.5, 1.8, and 1.8 ton/m^3, respectively. The in-between layers are silty sands with friction angle of 33°, and density of 1.9 ton/m^3. The bridge is instrumented with 29 accelerometers on the structure and 3 accelerometers at a free-field site. Figure 1 presents the schematic of the bridge and the location of five sensors on the bridge structure.

Continuum Model of the MRO

The finite element method is used to develop the continuum model of the MRO. This involves detailed modeling of the foundation soil, the abutment system, the bridge superstructure, the 5x5 pile group underneath the pier and the 7x1 pile group underneath the abutment. Solid eight-node brick elements are used to model the soil domain and the pile cap. Each node of the solid elements has three translational degrees of freedom. Four-node shell elements with three translational and three rotational degrees of freedom at each node are used to model the bridge deck. The equivalent thickness of 1.2 m is used for bridge deck in order to have similar moment of inertia to the original section. Nonlinear hysteretic response of the foundation soil and the bridge pier is taken into account in the analyses. The Pressure dependent multi-yield (PDMY) and pressure independent multi-yield (PIMY) constitutive models, developed by Yang et al. (2003), are used to simulate the nonlinear hysteretic response of sandy and clayey layers, respectively. The uniaxial Kent-Scott-Park model and a one-dimensional J2 plasticity model are used to account for the nonlinearity of the concrete and the reinforcing steels of the pier, respectively. Elastic behavior is assigned to the timber piles, the concrete pile cap, the deck, and the abutment walls because in seismic design of bridge systems these components are capacity-protected so that damage is not allowed. The continuum model includes a total of 41,177 nodes, 3996 beam-column elements, 1931 shell elements, and 31,844 solid elements representing a soil domain of 99.0 m long (in direction x), 50.0 m wide (in direction y), and 20.0 m deep (in direction z). Figure 2 presents the 3D continuum model of the MRO. The nodes at the base of the mesh are fixed in all directions, and the nodes with equal elevations at the four lateral boundaries are constrained to have equal displacements. The input ground motions (in the form of displacement) are applied to the nodes located at the base of model.

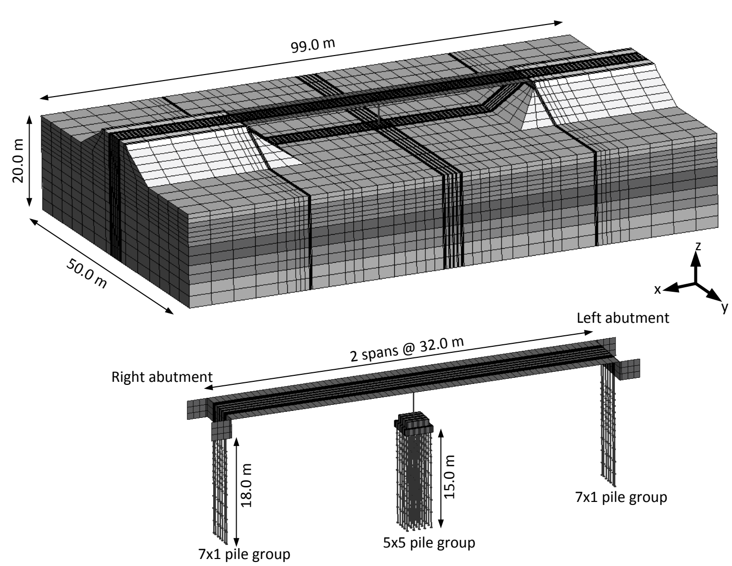


Figure 2: Three-dimensional continuum model of the MRO (Rahmani et al., 2014).

The continuum model has been validated in the study of Rahmani et al. (2014) by simulating the seismic responses of the bridge during the 1979 Imperial Valley and the 2010 El Mayor-Cucapah earthquakes. Further details about the continuum model of the bridge including input material parameters, input motions, and the validation have been given in Rahmani et al. (2014). The computational effort required to perform such a large scale nonlinear analysis has been discussed in detail in Babemzadeh et al. (2014).

Substructure Model of the MRO

Three-dimensional substructure model of the bridge is developed in five consecutive steps:

- In the first step, site response analysis is conducted in the free field for both the foundation soil and the embankment soil profiles in order to determine depth-varying time histories of displacement and the corresponding maximum displacements. To minimize the level of approximation in calculating the free-field motions, nonlinear time history analysis is conducted using advanced nonlinear constitutive models in the computer program OpenSees (McKenna and Fenves, 2001).
- In the second step, p-y, t-z, and Q-z nonlinear backbone curves are determined along the pile foundations following the guidelines of American Petroleum Institute (2007). The guidelines of AASHTO (2013) are used to determine the group reduction factors in order to account for group effects in the pile groups. The load-deflection backbone curves for an embedded pile cap are derived following the procedure presented by GEOSPECTRA (1997). Load-deflection backbone curves representing the interaction between the embankment and the abutment system are determined following the guidelines of Caltrans (2013). The lateral secant stiffnesses along the piles at the pile caps, and on top of the abutments are calculated using the previously determined force-deflection backbone curves and at the corresponding maximum lateral ground displacements in the free-field. The vertical secant stiffnesses are derived from the t-z and Q-z curves at a displacement equal to the settlement of the pile group under the tributary weight of the deck, the pier, and the pile cap. The settlement of the pile group is computed using the computer program GROUP v8 (2012).
- In the third step, the 6x6 stiffness and damping matrices that represent the flexibility of the pile group and the energy dissipation are computed. Both matrices are composed of 6 diagonal elements representing lateral, vertical, rocking, and torsional impedances. There are also four more off-diagonal elements, which represent the coupling effects between the lateral displacements and rocking of the foundation. To calculate the 6x6 stiffness matrix, the secant stiffnesses, obtained in the previous step, are used to create a 3D numerical model of the 5×5 pile group in the computer program, GROUP. Torsional stiffness of the system is beyond the scope of this research, and therefore, torsional stiffness of the piles are not taken into account in the analysis. The damping matrix is calculated as $C = 2\beta K/\omega$ where $\beta$is damping ratio, K is the elements of the 6×6 stiffness matrix of the pile group, and ω is the predominant angular frequency of the input motion. The damping ratio (β) is approximated to be 25% in both longitudinal and transverse directions following the study of Lee et al. (2011).

- In the fourth step, a massless finite element model of the 5×5 pile group supported on sets of linear springs (the secant stiffnesses) is created in OpenSees. The depth-varying time histories of displacement in the free-field are then applied to the ground nodes of the springs in the massless pile group.
- In the fifth step, the global model of the bridge is developed as shown in Figure 3. The model consists of the bridge deck and the pier, which are supported by the equivalent linear springs and dashpots at the base of the piers and top of the abutments. The elements and materials used to model the bridge deck and the piers are identical to those used in the continuum model of MRO. Lumped masses for the pile cap and the abutment system are assigned at the corresponding bridge supports.

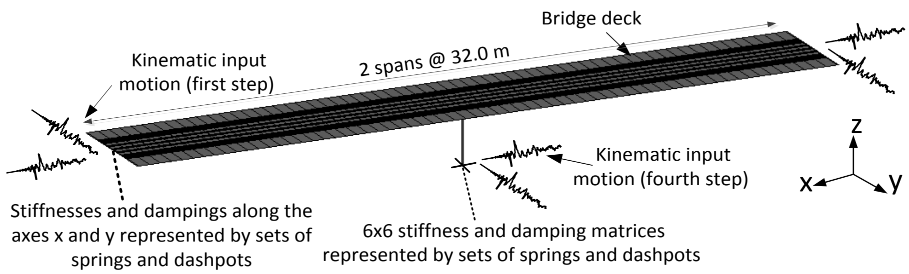
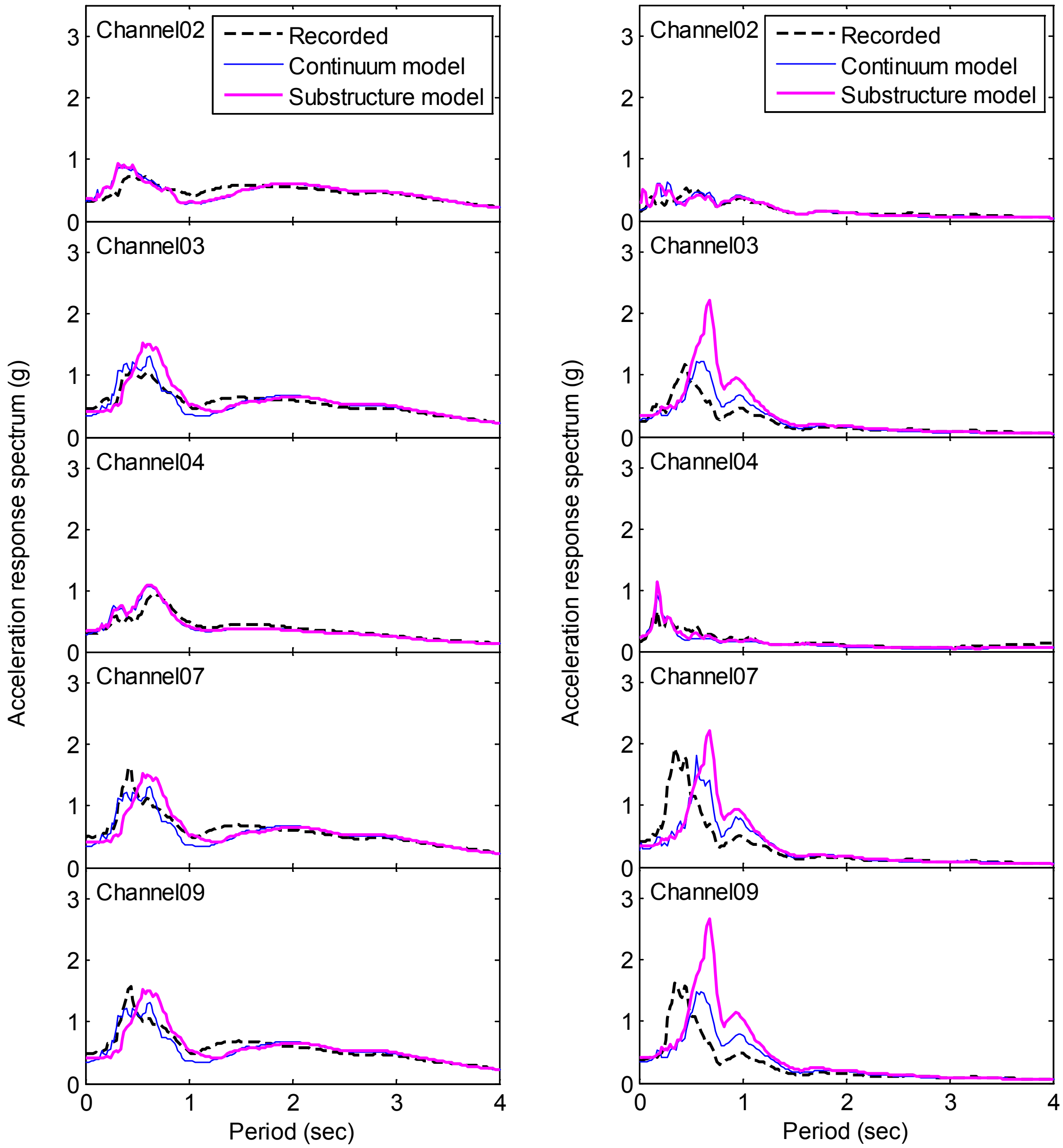


Figure 3: Three-dimensional substructure model of the MRO.

Results and Discussions

In Figure 4 acceleration response spectra for 5% damping obtained from the substructure model are compared to those obtained from the continuum model at the five recording locations of the bridge. The recorded spectral responses are also shown in this Figure. During the El Mayor-Cucapah earthquake the spectral response is poorly predicted especially at channels 3, 7, and 9. The spectral response at channel 9 is overestimated by a maximum factor of 2 for the periods in the range of 0.5 to 1.0 s. The level of difference during the Imperial Valley earthquake is negligible compared to that during the El Mayor-Cucapah earthquake. This is because the kinematic and inertial interactions between the bridge structure and the embankment are insignificant in the former event, which is not the case in the latter event.

The maximum displacement of the pier top with respect to the pier base, maximum shear force and bending moment at the pier base, and maximum force induced at the abutment are obtained from the continuum and substructure models of the MRO and listed in Table 1. The substructure model overestimates the pier top maximum displacements, shear forces, and bending moments at the pier base by factors in the range of 1.5 to 3.0 in the longitudinal and transverse directions, respectively. For example, maximum bending moments at the pier base was overestimated during the two events by factors of 2.0 and 3.0, respectively. The maximum force applied to the abutment in the longitudinal and transverse directions is underestimated by an average factor of 1.5 using the substructuring method.



(a) (b)

Figure 4: Acceleration response spectra (for 5% damping) of MRO for (a) the 1979 Imperial Valley earthquake, and (b) the 2010 El Mayer-Cucapah earthquake.

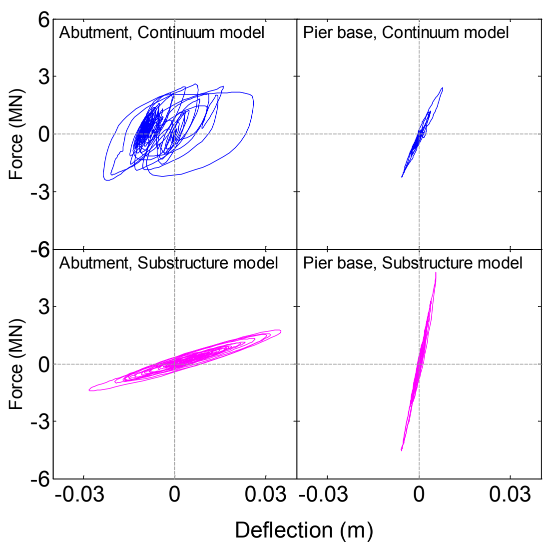
Figure 5 presents dynamic force-deflection responses at the right abutment and at the pier base in the transverse direction during both events. The dynamic stiffness is highly overestimated at the pile cap while it is highly underestimated at the abutments by the substructure method. The figure also highlights the significant deficiencies of the linear springs and dashpots in approximating the hysteretic force-deflection responses that includes loading-unloading-reloading. Similar observation is noted in the longitudinal direction (not presented in this paper).

Table 1: Peak responses of the MRO computed by the continuum and substructure models in the longitudinal (L) and transverse (T) directions.

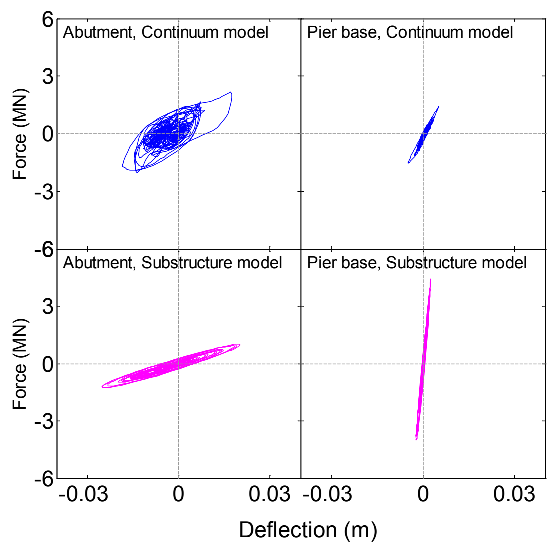
	Pier top disp.* (m)	Shear force** (MN)	Bending moment** (MN.m)	Force at abutment (MN)
Model	1979 Imperial Valley Earthquake			
Continuum	L:0.04, T:0.038	L:3.24, T:2.41	L:7.72, T:7.73	L:3.92, T:2.47
Substructure	L:0.08, T:0.13	L:5.1, T:4.8	L:15.0, T:15.0	L:2.67, T:1.76
	2010 El Mayor-Cucapah Earthquake			
Continuum	L:0.015, T:0.022	L:1.6, T:1.5	L:3.4, T:4.6	L:3.1, T:2.1
Substructure	L:0.023, T:0.044	L:3.2, T:4.5	L:10.3, T:15	L:2.1, T:1.3

*Relative displacement of the pier top with respect to the pier base.

**Shear forces and bending moments that are induced at the base of the bridge pier.



(a)



(b)

Figure 5: Dynamic force-deflection responses at the right abutment and at the pile cap in the transverse direction for (a) the 1979 Imperial Valley earthquake, and (b) the 2010 El Mayer-Cucapah earthquake.

The major causes for the observed differences in estimation of accelerations, deflections, forces, and bending moments stem from many simplifying assumptions in the formulation of the substructuring method. This is in line with the findings of Finn (2005) where he showed that the idealization of the soil domain with uncoupled springs and dashpots results in the inability of the approach to appropriately simulate the kinematic and inertial interactions. Furthermore, characterization of the spring models is always associated with considerable level of uncertainty. Although these guidelines are based on the results of static or slow cyclic loading tests, they are used in practice for seismic problems. This is in spite of the findings of several researchers such as Murchison and O'Neill (1984), Gazioglu and O'Neill (1984), Finn (2005), and most recently Choi et al. (2013) and Rahmani et al. (2012) who argued against the validity of these curves and reported significant levels of error in estimation of static or seismic responses of pile foundations when API springs are adopted in the analyses.

This study clearly shows that the formulation of the substructuring method lacks representing the nonlinear hysteretic response of soil. The method uses a constant dynamic stiffness matrix to represent the flexibility of the foundation system. In reality this stiffness varies at different levels of deformation. Seismic performance of a structure highly depends on its natural vibration periods, which in turn depend on the stiffness of the structure at its supports. Therefore, the modal shapes and the natural vibration periods of the bridge structure in the susbstructure model would be quite different from those in the continuum model.

Conclusions

This paper aimed to test how results of the substructuring method, which is a simple widely-used practical method, compare with those from a more exact method, i.e., continuum modeling. To this end, the seismic performance of the instrumented Meloland Road Overpass under two earthquake events was simulated using both methods. The results were compared, and it was shown that the substructure model overestimated the peak displacement of the pier top, the peak shear forces and bending moments at the pier base by factors in the range of 1.5 to 3.0 in the longitudinal and transverse directions. This study showed that idealization of the SSI by some simple equivalent linear springs and dashpots caused inadequate simulation of the foundation flexibility and energy dissipation.

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