SIMPLIFIED P-y RELATIONSHIPS FOR MODELING EMBANKMENT-ABUTMENT SYSTEMS OF TYPICAL CALIFORNIA BRIDGES

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ABSTRACT:

The flexibility of the embankment-abutment stiffness has been found to drastically effect the seismic response of the entire bridge, typically depending on the intensity of earthquake ground motion, but also on the geometrical and mechanical properties of embankment-abutment system. Consequently, significant effort has been made to incorporate the nonlinear behavior of the lateral support of the deck primarily through inelastic static (pushover) analysis. However, the difficulties in modeling the nonlinear behavior of soil as well as the computationally demanding three-dimensional geometry often lead to the disregard of some sources of nonlinearity such as those associated with the foundation soil and the boundary conditions at bridge abutments. Along these lines, the scope of this paper is to model and study the nonlinear response of six typical Californian abutment-embankment systems to derive specific force-deflection (i.e., P-y) relationships as a function of foundation soil properties, abutment type, and embankment geometry within the framework of an alternative simplified method proposed by Kappos et al. (2007). Comparative assessment of the current Caltrans guidelines is also performed and design recommendations are made.

KEYWORDS: abutment–embankment system, soil-structure interaction, pushover analysis, P-y relationships

1. INTRODUCTION

Nonlinear static (pushover) analysis has become a popular tool for the seismic assessment of reinforced concrete (R/C) bridges primarily as a means to identify the hierarchy of failure at a low computational cost. Nevertheless, the nonlinearity expected in bridges during strong ground motions, cannot be attributed solely to yielding of R/C sections, although these are the elements that are often purposely designed to exhibit inelastic behavior. On the contrary, additional material nonlinearity mechanisms (of the foundation, approach embankment, and/or backfill soil) and geometrical nonlinearity mechanisms (activation of control components such as bearings, 'stoppers', or seismic joints) can also play a significant role in the overall system response. Typically, both sources of nonlinearity effect the seismic response of a bridge; however, hysteretic response of nonlinear materials may have higher levels of uncertainty compared to the (pre-defined) presence of gaps and joints. Despite the importance of modeling such complex bridge lateral boundary conditions and the existence of specific guidelines in the US (Caltrans, ATC, MCEER) and in Europe (Eurocode 8-2) for the design of pile foundations and abutments, only minor guidance is provided for numerical modeling or for the practical consideration and assessment (even statically) of the nonlinear soil-foundation-pier-deck (Kappos & Sextos, 2001), and soil-abutment-deck system interaction (Goel and Chopra, 1997, Siddharthan et al., 1997, Mackie and Stojadinovic, 2002, Shamsabadi et al., 2007, Kotsoglou and Pantazopoulou, 2007). Moreover, huge discrepancies between the research studies conducted so far are reported (Zhang and Makris, 2002) especially when the problem is studied dynamically. Since the 3D effects of soil nonlinearity are commonly too complex, too uncertain, and computationally expensive for a practicing engineer to implement for every project, engineers resort to the simplified relationships prescribed by Caltrans for the estimation of the abutment-backfill soil capacity and stiffness. Another option is a simplified alternative that was proposed recently (Kappos et al, 2007). In this method, separate pushover analyses can be performed for the abutment and foundation systems that provide lateral support to the bridge. Both soil and concrete nonlinear behavior are included as a means to provide case-specific force-deflection (i.e. P-y) relationships that can, in turn, be used as nonlinear spring boundary constitutive models in the pushover analysis of the overall bridge structure.

The scope of this paper therefore, is to extend the above concept by performing a set of 3-dimensional nonlinear finite element analyses on typical California overpass abutment-embankment systems (Mackie and Stojadinovic, 2002). Simplified P-y relationships of the lateral supporting system are provided as a function of abutment type, foundation-embankment-backfill geometry, and soil properties that can be potentially used in cases where more accurate data are not available. The descriptions of the abutment-embankment systems as well as the P-y relationships derived are presented in the following.

2. OVERVIEW OF THE CASES STUDIED

Six typical reinforced concrete California bridges (namely Route 14, LADWP, W180, MGR, Adobe, and La Veta) consisting of box-girder superstructures, seat-type abutments, and shallow/pile foundations were adopted in the framework of the particular study. Given the short spans and relatively high deck stiffness of the particular structures, the embankment mobilization and the inelastic behavior of the soil material under high shear deformation levels is anticipated to have a significant effect on the response of the bridge under seismic loading. The geometry of the cases studied is illustrated in Figure 1 and is also summarized in Table 2.1. In general, the total abutment height H varies from 4.0m to 5.30m, the total length L varies from 12.50m to 23.20m, the width of pile cap l lies between 2.50m to 3.50m, the wing wall length L_{ww} extends from 4.80m to 6.0m while the width of stem wall b is in order of 1.0m. The width of the parapet for all the six abutment types is equal to 30cm while the wing wall height is equal to 90cm. The abutments of the Route 14 and the LADWP bridges are founded on spread footings while all other bridges are supported on pile groups having the configuration pattern depicted in Figure 1. The specified compressive strength for unconfined concrete is 25 MPa for both piles and abutments in all cases studied, corresponding to an elastic modulus E of 30.5 MPa. The foundation soil properties were initially based on available data (Table 2.1), and the slope of all the embankment cross sections was taken equal to 2:1 (Figure 1).

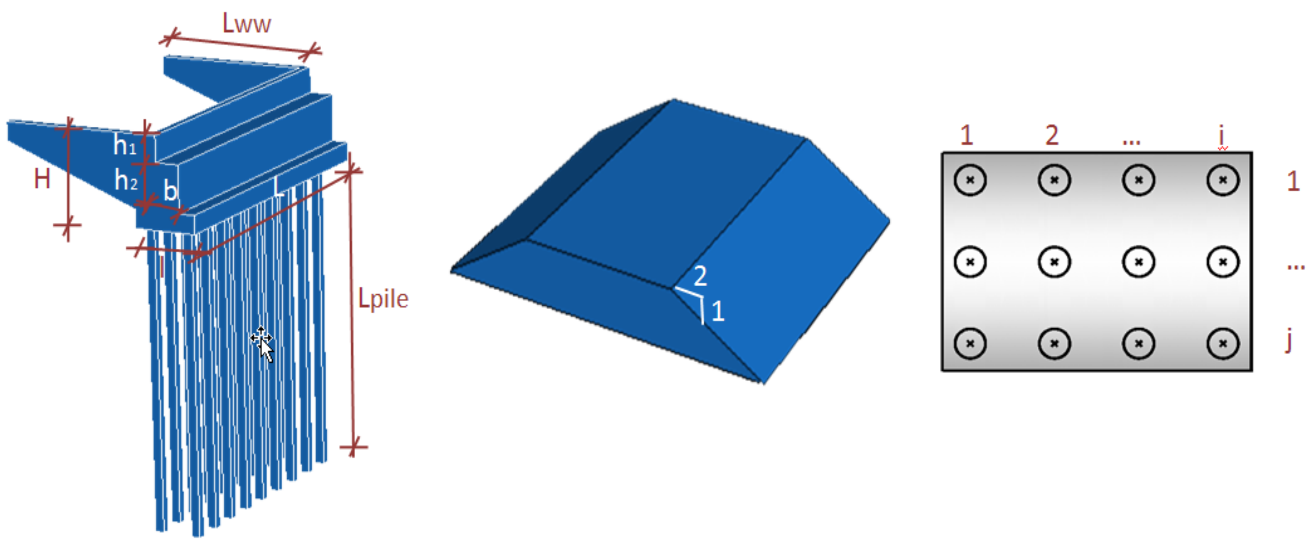

Figure 1 Configuration of a typical seat type abutment-embankment system studied

Table 2.1 Geometrical characteristics of the six abutment types

	H (m)	h_1 (m)	h_2 (m)	b (m)	l (m)	L (m)	L_{ww} (m)	L_{pile} (m)	D_{pile} (m)	pile group	Soil (blow count in parenthesis)
Route 14	4.70	1.75	2.25	1.00	3.40	16.50	5.35	-	-	-	
LADWP	4.00	1.30	2.10	1.00	2.90	13.50	4.80	-	-	-	
W180	5.30	2.35	2.25	1.20	3.50	13.00	5.70	13.00	0.40	3×10	3m silty sand (15), 6m med.fine sand (27), 6m med./fine sand (40)
MGR	5.00	1.90	2.35	1.10	3.50	14.00	5.30	15.00	0.75	5×1/6×1	5m rock, 5m sandy gravel
Adobe	4.40	1.25	2.25	1.20	2.45	13.00	6.00	15.00	0.40	2×10	2m sandy silt (13), 12m clayey silt (15-20), 3m medium sand (45)
La Veta	4.15	1.90	1.60	1.15	3.00	23.50	5.25	17.00	0.60	2×11	5m fine/medium sand, 4m dense sand

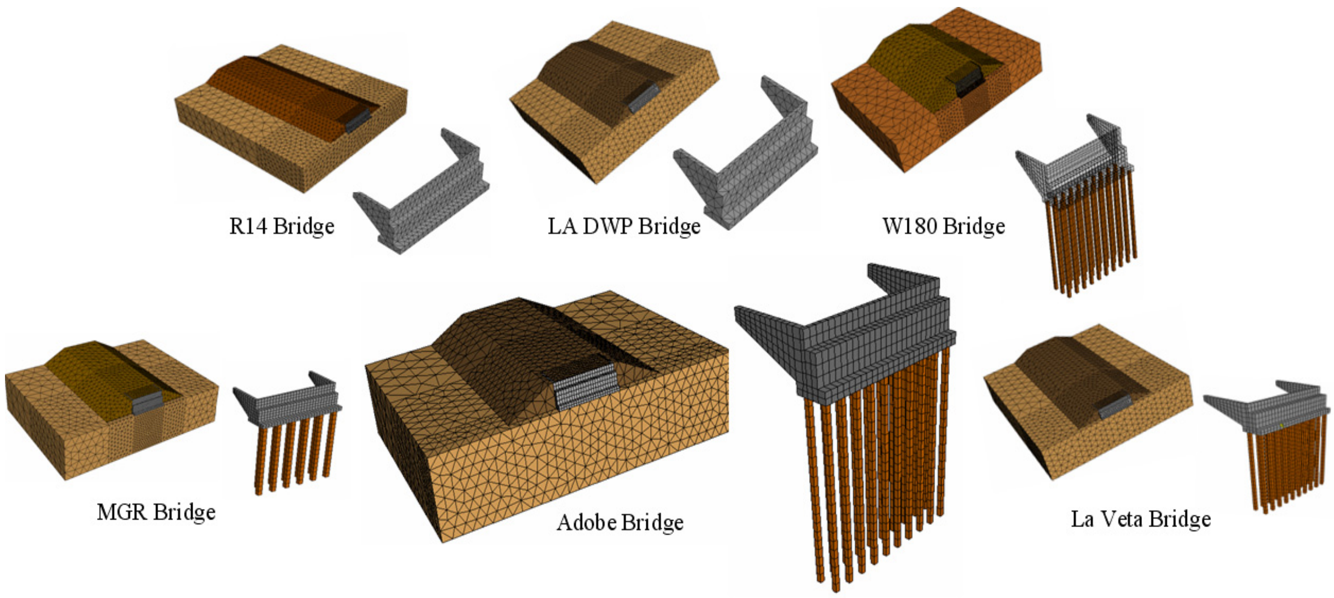

Figure 2 Finite element models of the six abutment-embankment systems

3. MODELING OF THE ABUTMENT-FOUNDATION-BACKFILL-EMBANKMENT SYSTEMS

The abutment-embankment systems were modeled in 3-dimensional space using the finite element program ABAQUS. Tetrahedral solid elements (C3D4 ABAQUS type) were used for the backfill, the embankment, and the foundation soil discretization while brick elements (C3D8 ABAQUS type) were used for the abutments and the pile groups. A dense finite element grid was adopted for the areas of stress concentration and/or abrupt geometry change, that is in the vicinity of the abutment, backfill, and pile groups. The approach embankment was modeled along a distance that in all cases exceeded the critical embankment length l_c = 50m while the foundation soil volume was considered equal of $5L$ x $2L$ x l_c, where L the abutment total length (Figure 2). It was deemed a realistic assumption to consider the soil (backfill, embankment and foundation) as the nonlinear material mechanism while the R/C (abutment and pile) sections remain linear elastic with cracked section properties reduced by a factor of 2/3 compared to their gross section stiffness. Backfill soil was considered as well compacted granular material according to the Caltrans guidelines despite the fact that in practice different soil types may exist. For modeling purposes, a Young's modulus equal to 60MPa and a friction angle of 39 degrees were adopted for the backfill soil type while, for simplicity, the same properties were assumed for the embankment as well, even though backfill material is usually stiffer and stronger than the embankment material. The foundation soil was based initially on profiles of boring logs near the abutments for comparison purposes but was also parametrically modified to correspond to competent soil (A) and poor soil (B) according to Caltrans SDC (2006). Characteristic values of modulus of elasticity E=15MPa, friction angle $\varphi=22^o$ and cohesion c=100kPa for Soil A and E=5MPa, $\varphi=5^o$, cohesion c=50kPa for Soil B were also assumed for this study while poisson's ratio (ν) as well as unit weight (γ) were taken equal to 0.3 and 20kN/m^3 respectively. It is noted herein, that in the framework of the static (pushover) analysis performed, compatible (Das, 1994) static soil stiffness was adopted since the implementation of shear wave velocity typically overestimates the static stiffness of the finite element volume. The Mohr-Coulomb constitutive model implemented in ABAQUS was used to simulate the nonlinear soil behavior while a gradually increasing pressure was applied for the pushover analysis as distributed normal and shear force on the abutment, along the longitudinal and transverse directions, respectively. Finally the corresponding nonlinear P-y relationships were derived for the six bridges studied and for both soil types and excitation directions. It is noted herein that the particular approach is not affected by the potential presence of bearings, gaps, stoppers, and joints that have to be modeled independently as part of the main bridge structure, but rather represent *solely* the abutment-embankment system *once and if* it is activated.

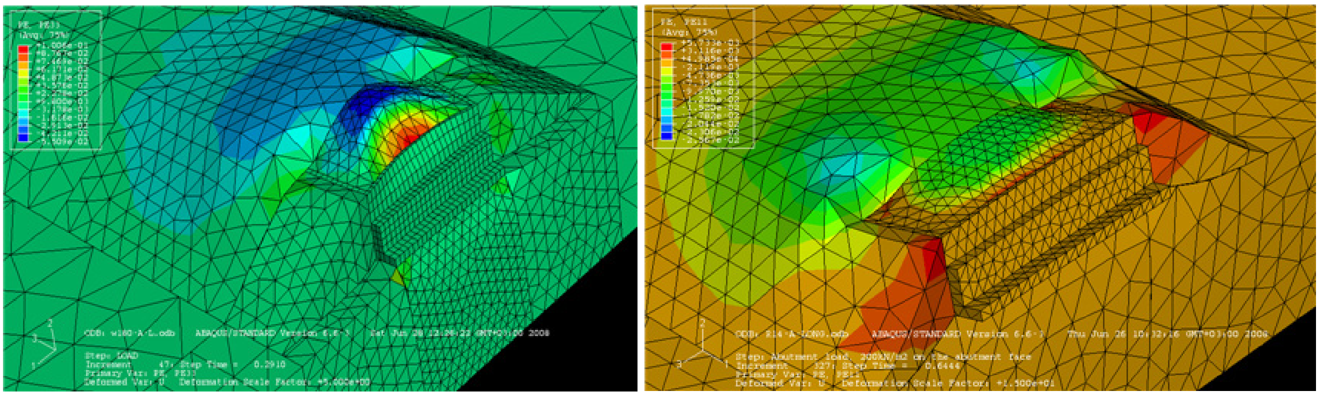

Figure 3 Plastic strains developed at the backfill the approach embankment of the W180 bridge (left) and Route 14 bridge (right) due to deck-induced longitudinal pressure

4. MODEL VERIFICATION

In order to verify the numerical accuracy of the 3-dimensional ABAQUS abutment-embankment models, the aforementioned approach was also used for the case of a three-span overpass that is part of the new 680km EGNATIA highway in northern Greece. The behavior of the particular structure had already been studied by Kappos et al. (2007) using a separate set of pushover analyses of the abutment-foundation system. The compliance of the embankment and the foundation soil were taken into account through nonlinear springs that were implemented on both the abutment surface and the along the pile lengths. The pushover curves of the two systems studied for the longitudinal and the transverse direction are presented in Figure 4. It is noted that in the longitudinal direction the response of the spring supported abutment is bi-linear due to pile failure in shear (modeled as elastic-perfectly plastic in the pile elements). It is observed that given the uncertainties related to the calibration between the nonlinear force-displacement curves of the springs used in the Winkler model and the stress-based yield criterion adopted for the solid of the 3D-FE model, the agreement between the two approaches is satisfactory especially in the transverse direction (where the overall stiffness is controlled by the pile foundation). As a result, the assumptions made regarding the simulation of the nonlinear soil response of the six bridges studied using ABAQUS can be deemed as verified.

5. ANALYSIS RESULTS

Figures 5 to 10 illustrate the pushover curves for the longitudinal (left) and the transverse (right) direction of the six abutment-embankment systems studied for the two Caltrans soil types (A and B). Each curve is idealized with a bilinear relationship for potential use as a force-deflection (P-y) curve of a nonlinear boundary soil spring for similar bridge structures. The elastic and inelastic stiffness values of all the examined systems as well as the corresponding yield displacements δ_y are summarized in Table 5.1, normalized by the width of the abutment so the resulting unit are [force]/[length]^2. Moreover, the initial stiffness and the ultimate capacity of the abutments

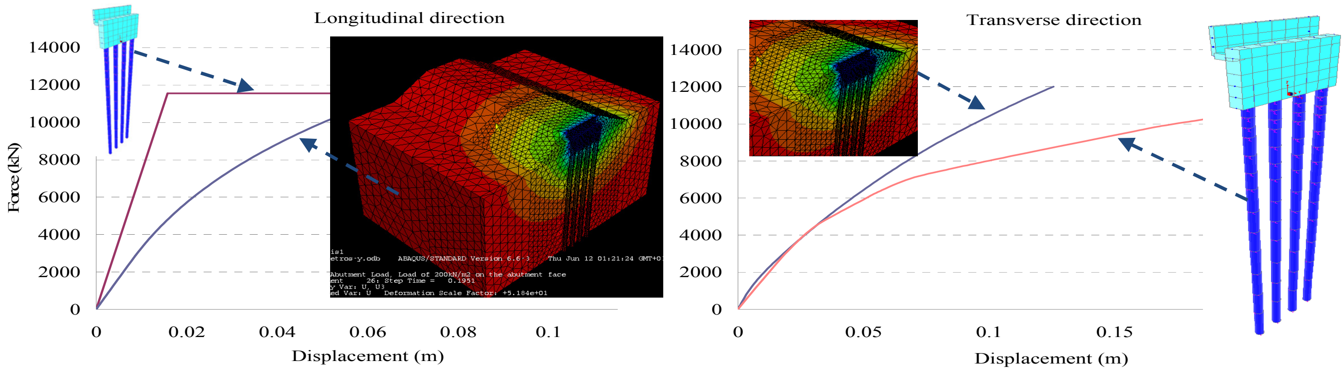

Figure 4 Comparison of the pushover curves derived for the spring supported abutment and 3-D model of an Egnatia highway bridge along the longitudinal (left) and transverse direction (right).

according to Caltrans method (2006) are also plotted for comparison on each Figure 5-10. Based on passive earth pressure tests and the force deflection results from large-scale abutment testing at UC Davis, a value for the initial embankment fill stiffness equal to 11.5kN/mm/m width of the wall is recommended by Caltrans guidelines. The initial stiffness can be adjusted proportionally to the backwall height as documented in Eqn. 5.1:

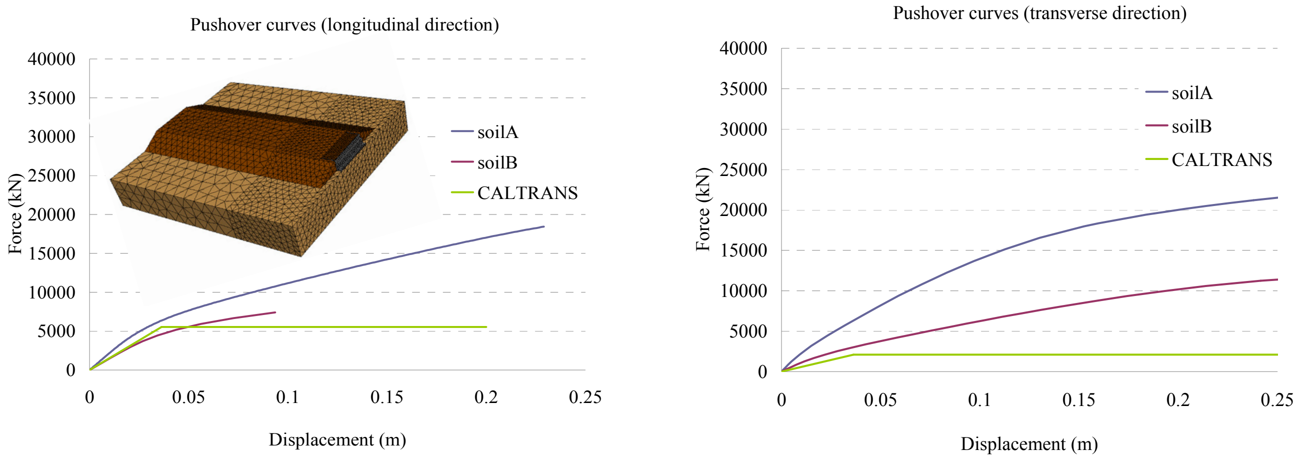

Figure 5 Pushover curves of the Route 14 bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

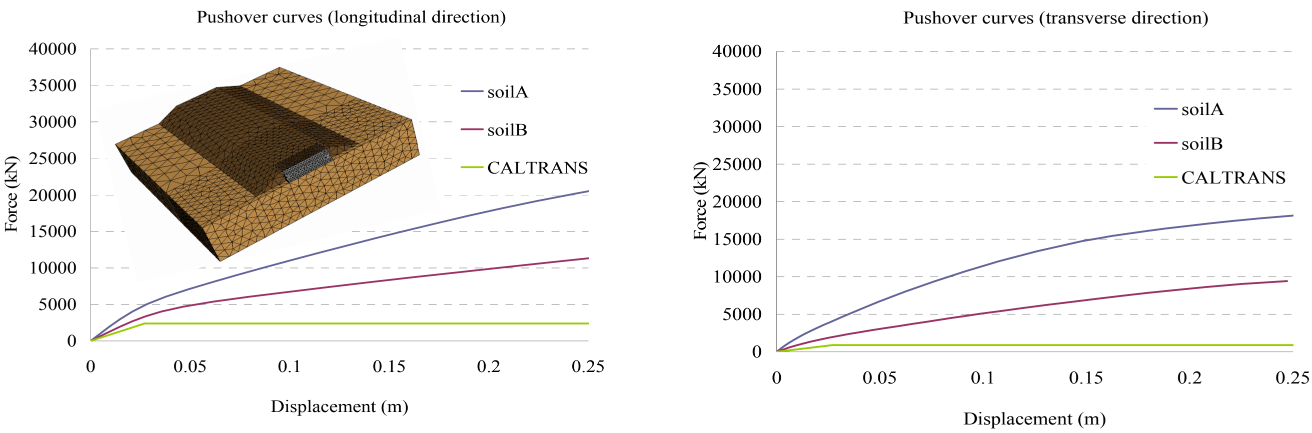

Figure 6 Pushover curves of the LADWP bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

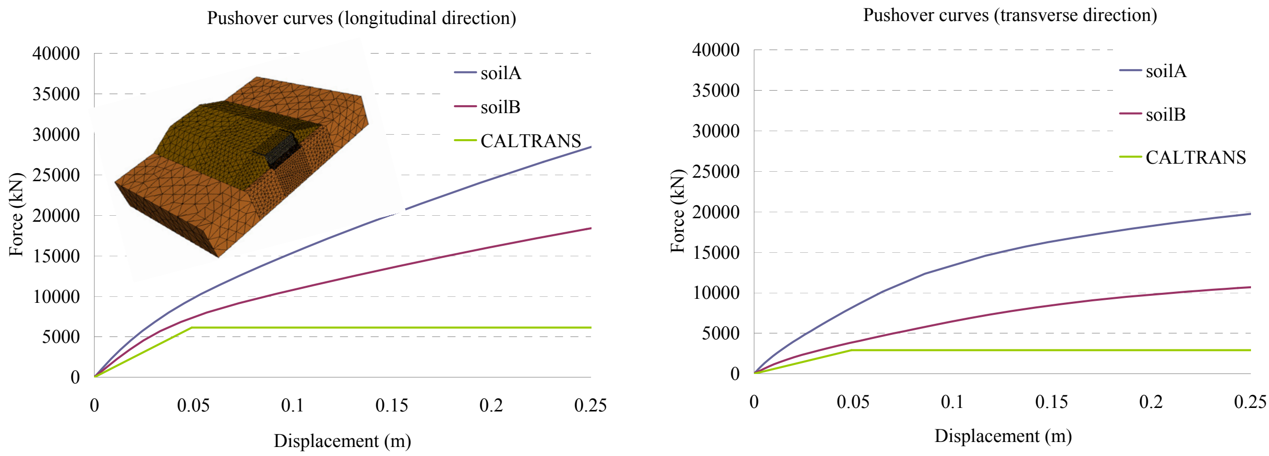

Figure 7 Pushover curves of the W180 bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

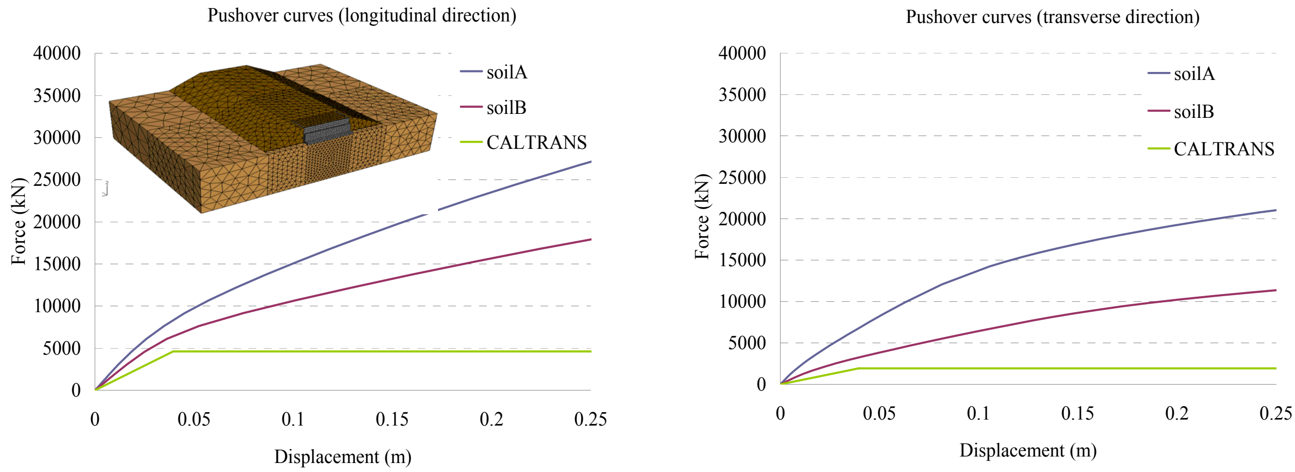

Figure 8 Pushover curves of the MGR bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

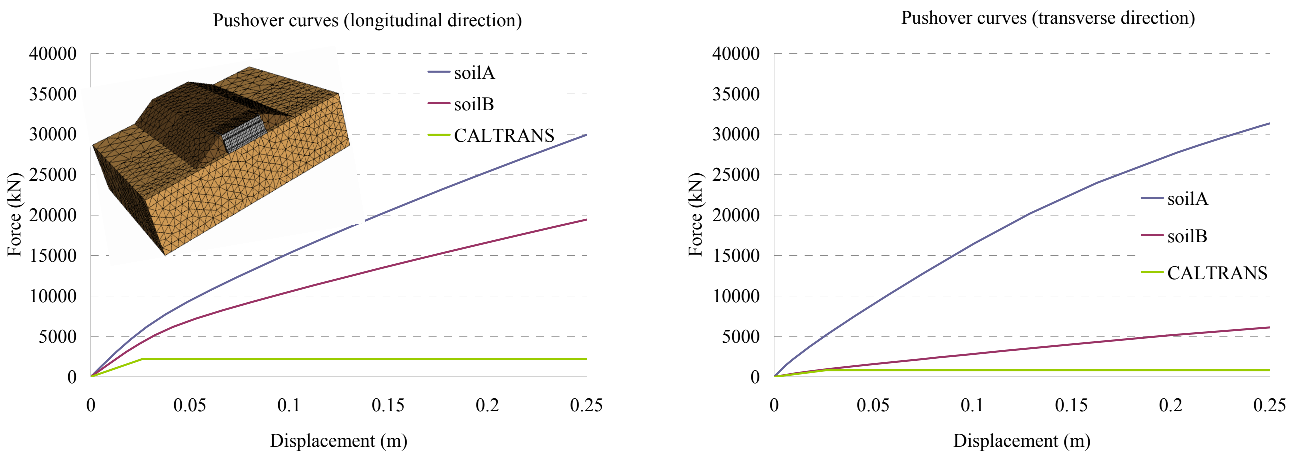

Figure 9 Pushover curves of the Adobe bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

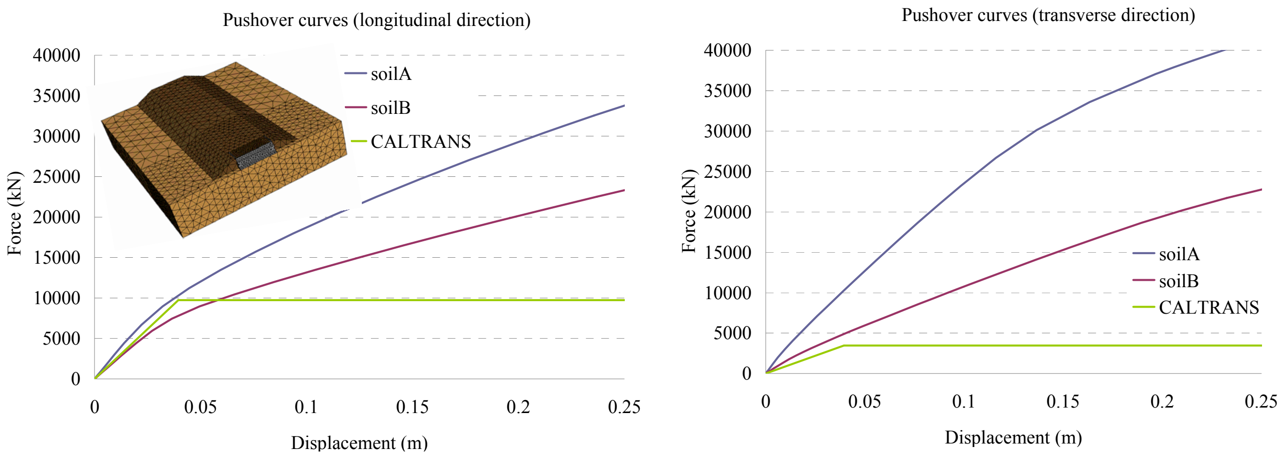

Figure 10 Pushover curves of the La Veta bridge abutment-embankment system along the longitudinal (left) and transverse direction (right)

Table 5.1 Bilinear idealization of the pushover curves derived for the longitudinal (left) and the transverse direction (right) as normalized by the abutment width

LONG. direction (MN/m^2)	*Soil A*		*Soil B*		*Soil A*		*Soil B*		*TRANS. direction (MN/m^2)*
	$K_{initial}$ / K_{inel}	$\delta_{yA}(m)$	$K_{initial}$ / K_{inel}	$\delta_{yB}(m)$	$K_{initial}$ / K_{inel}	$\delta_{yA}(m)$	$K_{initial}$ / K_{inel}	$\delta_{yB}(m)$	
Route 14	15.7	0.037	11.2	0.033	12.3	0.037	5.4	0.033	*Route 14*
	4.4		3.4		2.8		1.9		
LADWP	18.2	0.041	12.6	0.038	12.9	0.041	6.1	0.038	*LADWP*
	6.1		3.0		3.3		2.6		
W180	28.7	0.05	22.8	0.046	21.1	0.05	9.3	0.046	*W180*
	10.8		6.3		4.7		2.6		
MGR	24.6	0.052	19.9	0.043	18.0	0.052	8.0	0.043	*MGR*
	8.4		5.3		4.6		2.9		
Adobe	21.8	0.047	16.5	0.043	17.1	0.047	3.2	0.043	*Adobe*
	9.6		5.9		8.1		2.1		
La Veta	14.8	0.048	11.5	0.042	12.7	0.048	5.7	0.042	*La Veta*
	5.1		3.5		4.7		3.4		

$$K_{abut}=K_i \times w \times (h/1.7) \tag{5.1}$$

where *w* is the width of the backwall and (*h*/1.7) is the proportionality factor based on the 1.7m height of UC Davis abutment specimen (Maroney, 1994a and 1994b). On the other hand, the ultimate abutment load is assumed to be limited by a maximum static passive pressure of 239kPa resisting movement at the abutment. In the transverse direction, the abutment stiffness and strength obtained for the longitudinal direction were modified using factors corresponding to a wing wall effectiveness and participation coefficients of 2/3 and 4/3, respectively (Maroney and Chai, 1994b). Based on the values summarized on Table 5.1, as anticipated, the stiffness values (both initial and post-elastic) along the longitudinal direction are in general larger than those along the transverse direction. Moreover, the initial stiffness values obtained from ABAQUS models are generally higher than those derived using the Caltrans procedure with the exception of the longitudinal stiffness of the Route 14 and the La Veta abutment-embankment systems. The reason may be attributed to the fact that Caltrans procedure does not account for factors such as abutment dimensions, embankment geometry, soil properties and foundation system stiffness. The Caltrans procedure is based on a single large-scale abutment test whereas the ABAQUS models allow site- and abutment-specific modeling. In addition, some of the difference in the procedures is inherent to comparison between a simplified hand calculation and a 3-D finite element model, the latter equally related to a number of modeling uncertainties.

6. CONCLUSIONS

This paper is an effort to contribute to the determination of static stiffness of abutment-embankment systems, modeled in detail using the finite element program ABAQUS. Models are generated for six typical California bridges with known properties. Through pushover analyses along the longitudinal and the transverse directions, idealized force-deflection (P-y) relationships are derived for two main soil types. The results of this study indicate that the initial stiffness values of the refined numerical models are generally higher compared to those derived by applying the widely used Caltrans procedure. It is deemed therefore that, in case of a lack of more accurate data and/or analysis results, the P-y relationships presented herein can potentially be used together with the typically applied Caltrans equation, in the framework of a parametric analysis. Further study is also required not only for additional abutment-embankment systems but also towards the identification of the complex and multi-parametric dynamic effect soil-abutment-embankment interaction.

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