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Contributions to the study of soil-structure interaction in bridge abutments

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Abstract. Bridge abutments are structures of varied characteristics that enable the transition between bridges and highways, with the function of supporting decks at the ends, in addition to retaining access embankments. A poor assessment of the soil-structure interaction (SSI) in these structures might compromise the bridge's in-service performance and structural safety. However, this topic has not yet been properly consolidated in international standards, and it is a common practice to design abutments through isolated and simplified models. This paper aims to investigate aspects that are not considered in the usual analysis, such as the three-dimensionality of the problem and a more realistic assessment of SSI. For that purpose, a parametric study is conducted through the numerical computational modeling of a conventional stub type concrete abutment, with deep foundations, the adjacent embankment and the foundation soil. Two- and three-dimensional finite element models are generated, considering variations in geometry, constructive phasing of the abutment and different constitutive models for soil representation: Winkler or continuum models. The comparative analysis of the results, in terms of stresses/strains of the soil and structural elements, should provide a broader understanding of the theme and contribute to the establishment of a modelling framework of SSI for bridge abutments.

Keywords: bridge, bridge abutment, soil-structure interaction, numerical modeling.

1 Introduction

In essence, the soil-structure interaction (SSI) corresponds to the static and dynamic phenomenon of contact and interaction between a material with a very deformable characteristic – the soil – and a relatively more rigid material – the structure [1]. However, the concept permeates a wide range of fields of action, so it is difficult to give a clear and unique definition for this phenomenon. In the context of structural and geotechnical designs, the term describes methods of analysis that consider, in addition to the equilibrium promoted by forces of interaction, the compatibility of displacements between the two parts (soil and structure). The first models that outlined this compatibility correspond to analytical solutions to simple problems, such as that of a semi-space punctually loaded by a force or of a rigid circular plate supported by an elastic medium. Modernly, the methods of analysis of SSI that have greater traditionality and application are the so-called Winkler models [2], in which the soil is taken as an elastic medium and characterized by a proportionality coefficient, and the continuum models, in that the soil is taken up by a continuous medium characterized by parameters such as the modulus of elasticity and the Poisson's ratio. These models may be seen in several scientific researches such as the study of Kim et. al [3].

This kind of analysis is also relevant in the context of structural bridge designs, mainly due to the magnitude of the horizontal efforts acting on bridge abutments or piers. These structures usually do not provide an efficient locking system for the foundation, so that the horizontal loads and bending moments must be fully resisted by the foundation soil. An inadequate assessment of SSI, among other factors, might compromise the structural safety of the bridge and generate functionality problems, such as the loss of granular material in the slope adjacent to the abutment [4]. Through a parametric study of a standard bridge, commonly observed on Brazilian highways, the present paper aims to examine the most important aspects of SSI modeling in the design of bridge abutments, with special focus on the effects promoted by horizontal loading on these structures.

2 Methodology

2.1 Parametric study

The study developed is based upon the modeling and structural analysis of a single span concrete beams bridge. The specific object of analysis in this paper is abutment A.1, a conventional stub type abutment placed in the left edge of the bridge. All the main features of the system are presented in the remarks below.

Remark 1: Geometry. The bridge has a representative length of 40 m, between center of joints, and deck width of 14.1 m (two traffic lanes of 3,50 m and two shoulders of 3,00 m, plus guard-rails and side plates). The geometrical features of the bridge's standard design are illustrated in Fig. 1. The deck structure is composed by 5 precast prestressed concrete beams, with 2,10 m of height and 3.24 m spacing between each other, and a concrete slab with approximate thickness of 0.20 m. There is also a 0.07 m thick asphalt pavement, two concrete guard-rails, with 0.38 m of width, and constructive precast concrete plates. The deck has a longitudinal slope of 0,5% and a transversal slope, from the center to the edges, of 2,0%. The beams connect to the abutments through elastomeric bearing devices, with dimensions of 0.041 x 0.25 x 0.45 m. The abutments consist on a low height transversal beam, with section dimensions of 2,25 x 1,40 m and length of 14.64 m, a 0.25 m thick and 1.84 m high curtain above the beam and two side wings, with 0.25 m of thickness. There is also an approach slab, partially supported by the curtain (connected through a Freyssinet hinge), with, 0.25 m of thickness, 4.00 m of length and width equivalent to the deck's dimensions. As seen in Fig. 1, the abutment is placed at the top of an embankment and is supported by 10 concrete excavated piles (5 lines of 2 piles), with a nominal diameter of 0.45 m. The piles develop through a length of 20 m, of which 15 m are embedded in the natural ground. The general characteristics listed above are those that give the structure the classification as a stub type abutment.

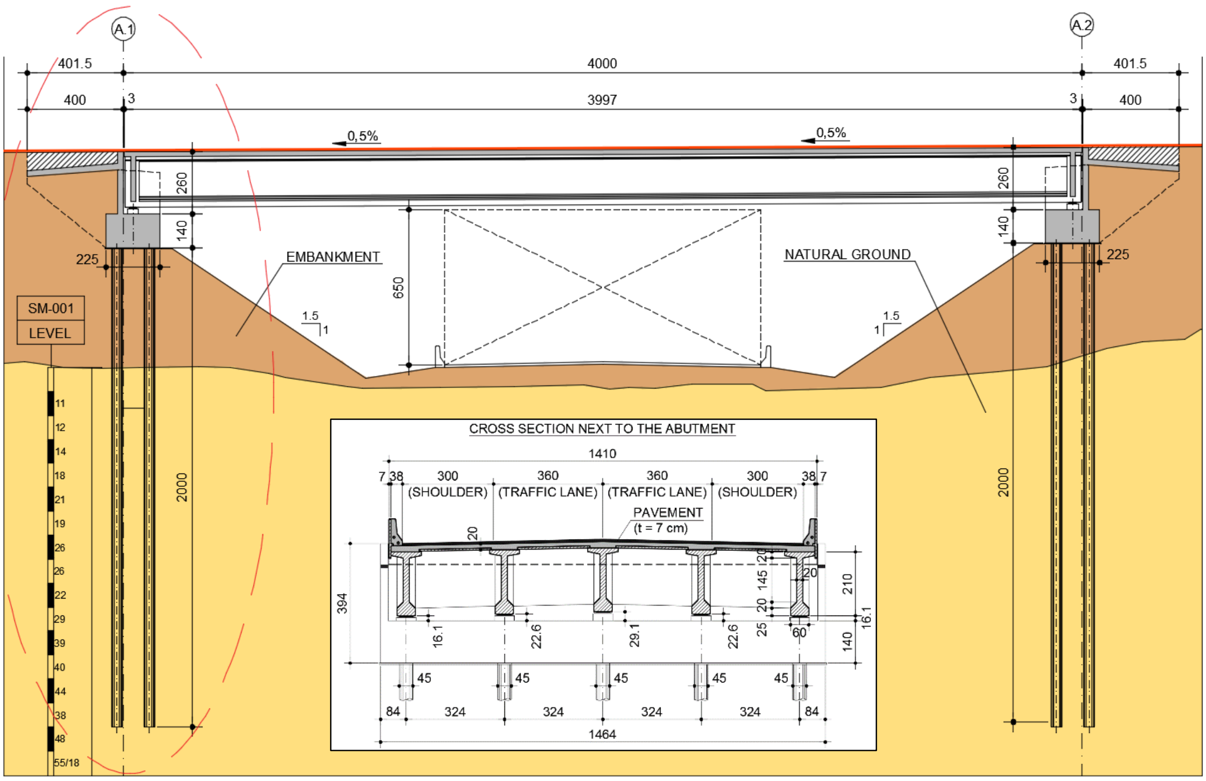

Figure 1. Geometrical features of the bridge's standard design (longitudinal and transversal sections)

Remark 2: Material properties. The structural components are made of concrete, with class of resistance C30. For the following analyzes, it is admitted that the concrete is an elastic material, with a modulus of elasticity of 26838.4 MPa. The unit weights for concrete and the asphalt pavement are 25 kN/m³ and 24 kN/m³, respectively.

Remark 3: Loads and combinations. Some of the main loads to consider in the structural design of bridges are self-weight, live loads (vehicles), acceleration and braking, wind, temperature, earth pressure, among others. The loads considered acting on abutment A.1, summarized in Tab. 1, are combined in appropriate directions to generate a characteristic combination of actions, with a global weighting factor of 1.00. This combination is traditionally used for foundation design and, in this case, will be used for the soil-structure interaction analysis. The deck loads of temperature and creep/shrinkage are defined through imposed deformations and, therefore, are dependent on the transverse rigidity of the bearing device, equivalent to 3879 kN/m (long-term loads). The axis of the bearing devices coincides with the axis of the foundations, so there are no additional moments generated due to the eccentricity of the vertical loads on the deck. The maximum axial load acting in the piles is equivalent to 969 kN, below the presumed pile's working load (1000 kN). The necessary length of the piles within the soil was estimated according to Aoki & Velloso's method [5].

Table 1. Loads acting in abutment A.1

Origin of load	Load	Parameter	Unit	Value	Direction
Deck derived loads	Deck's total weight (dead load)	G_{deck}	kN	4457	Vertical
	Live load above the deck (vehicle TB-450)	$Q_{deck,TB450}$	kN	360	Vertical
	Live load above the deck (distributed load)	$Q_{deck,dist}$	kN	1319	Vertical
	Acceleration and braking in the deck	$F_{deck,ab}$	kN	68	Long.
	Temperature loading	$F_{deck,t}$	kN	58	Long.
	Creep and shrinkage	$F_{deck,cs}$	kN	174	Long.
	Operational wind force	$F_{deck,w}$	kN	137	Transv.
Abutment loads	Abutment's weight (dead load)	G	kN	1482	Vertical
	Approach slab's weight (dead load)	G_{slab}	kN	544	Vertical
	Live load above the approach slab	$Q_{slab,dist}$	kN	132	Vertical
	Lateral pressure due to live load	E_{dist}	kN	92	Long.
	Lateral earth pressure	E_s	kN	649	Long.

Remark 4: Soil characterization. The foundation soil, in the analyzed plot, presents the following stratification: an upper layer of silty clay, an intermediate layer of clayey silt and a lower layer of sandy silt. A 5.0 m embankment (clayish soil based) is launched over the natural terrain until the base of the abutment, with an adjacent slope of 1,5H:1,0V. The main geotechnical properties admitted for the soil, per layer, are presented in Tab. 2: depth, specific weight (γ), friction angle (φ), cohesion (c) and the layer's average penetration resistance index (N_{SPT}) taken from Standard Penetration Tests (SPT). The secant Young's modulus (E_s), defined by empirical correlations with N_{SPT} values [6], and the Poisson's coefficient (ν), are also presented.

Table 2. Layers identification and soil parameters

Layer	Type	Depth (m)	γ (kN/m³)	φ (°)	c (kPa)	N_{SPT}	E_s (kPa)	ν
1	Embankment	0.0 – 5.0	18.0	30	15	-	10000	0.30
2	Silty Clay	5.0 – 11.0	18.0	27	25	16	22400	0.30
3	Clayey Silt	11.0 – 17.0	19.0	30	28	30	37500	0.35
4	Sandy Silt	17.0 – 20.0	21.0	35	35	43	58050	0.35

Remark 4: Parametric analysis. The bridge (deck and abutments) and soil described above represent the standard design established for this study. The parametric analysis corresponds to the variation of geometric parameters of the system above and below the values adopted in the standard design, in order to verify the impact on numerical model results. Three parametric analysis are carried out: (1) variation of the pile's diameter; (2) variation of the height of the embankment; (3) variation of the spacing between the edge of the abutment and the adjacent slope of the embankment. The variable parameters are illustrated in Fig. 2 and its variation ranges are given in Tab. 3. For each analysis, all the other features of the standard design are preserved, including the pile's total length.

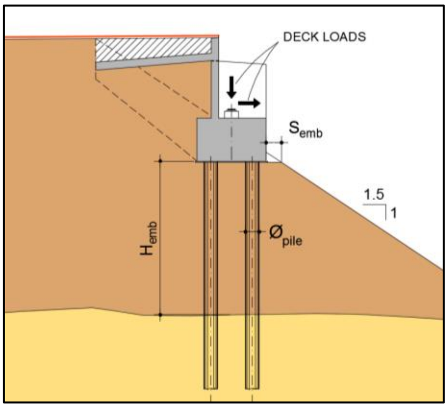

Figure 2. Variable parameters of the abutment, adopted for the parametric study

Table 3. Variation range applied to the chosen parameters

Parametric Analysis	Variable parameter	Unit	Variation range				
1	$Ø_{pile}$	m	0.35	0.40	0.45	0.50	0.55
2	H_{emb}	m	0.00	2.50	5.00	7.50	10.00
3	S_{emb}	m	0.00	0.25	0.50	0.75	1.00

The variations are promoted in several computational models, generated by two distinct numerical modelling methods: (1) Tridimensional Winkler Models and (2) Bidimensional Continuum Models. Thus, each value presented in Tab. 3 is simulated by 2 numerical models, one for each modeling method. The 2D models are representations of the central line of piles, so all the results extracted from the 3D models refer to that same line.

2.2 Method 1: Tridimensional Winkler Models

For the first modeling method, finite element models of frames and shell developed in three-dimensional space are generated, with the aid of a standard FEM software for structural analysis (SAP2000 v.21, by CSi). It is possible to develop a geometry close to reality for the abutment, as shown in Fig. 3, but the elements must be modeled according to their structural functioning and so that the progress of the loads in the structure is adequate. In short, the transverse beam and the piles are represented by frames, while the curtain and wing walls are represented by thin shells. The approach slab is substituted by loads applied to the top of the curtain and other load are applied at their relative position in relation to the abutment). Some elements are linked together by rigid frames, with infinite stiffness and null mass. Special attention must be given to the connection between the frames and shells, so that moments can be transferred between these elements.

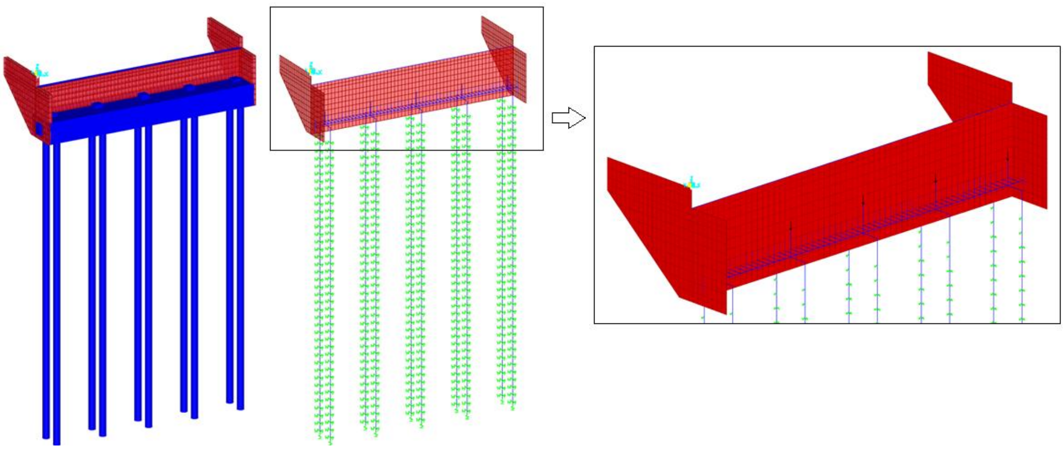

Figure 3. Tridimensional Winkler Models configuration (with and without extrusion)

The soil is represented through a Winkler model, with springs distributed over the length of the pile, in each 50 cm. The coefficient of horizontal subgrade reaction (k_h) is determined by Marche's theory [7], through correlations between the static penetration point resistance and N_{spt} values, given by Aoki & Velloso [5]. For the layers of embankment, silty clay, clayey silt and sandy silt, the values of k_h are, respectively, 10000, 35200, 69000 and 236500 kN/m². To consider the material non-linearity of the soil, the model undergoes an iterative process of comparison between the spring's reactions and the maximum admissible reactions, followed by the substitution of the springs for reaction loads when yielding occurs. The maximum reaction for each spring is defined by the equilibrium between passive and active earth pressures, through Rankine's theory [8].

2.3 Method 2: Bidimensional Continuum Models

The two-dimensional continuum models, generated with the software Plaxis 2D v.20, by Bentley, are illustrated in Fig. 4. The soil mass has dimensions of 40 x 30 m (displacements restricted in the boundaries) and is discretized in a mesh of 15 nodes triangular elements. Since the model is developed in plane strain, to transform the discrete 3D problem into an equivalent 2D continuous problem, it is necessary to condense the loads that act in the influence width of the pile line (by adding additional earth pressure to the back of the curtain, for instance) and then weigh the loads and elements inertia from the original resistant strip of soil (1.35 m considering a spreading factor of 3 times the diameter of the pile), to the width considered by the software, equal to 1.00 m.

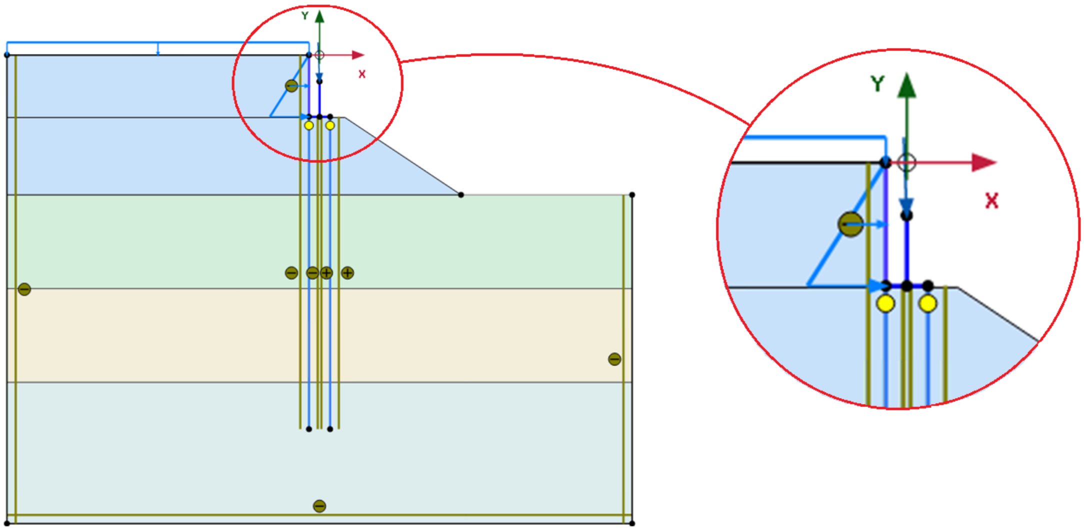

Figure 4. Bidimensional Continuum Models configuration

The soil is defined as an elastoplastic material, and the failure criteria selected for describing its strength is Mohr-Coulomb, which enables the occurrence of plastic points in the model. For the interaction between structure and soil the same Mohr-Coulomb criterion is adopted, but the strength is softened down by an interface strength parameter (R_{inter}) of 2/3, applied to the soil parameters defined in Tab. 2. In this method, it is also possible to simulate the effects of the full execution sequence of the bridge, as shown in Fig. 5.

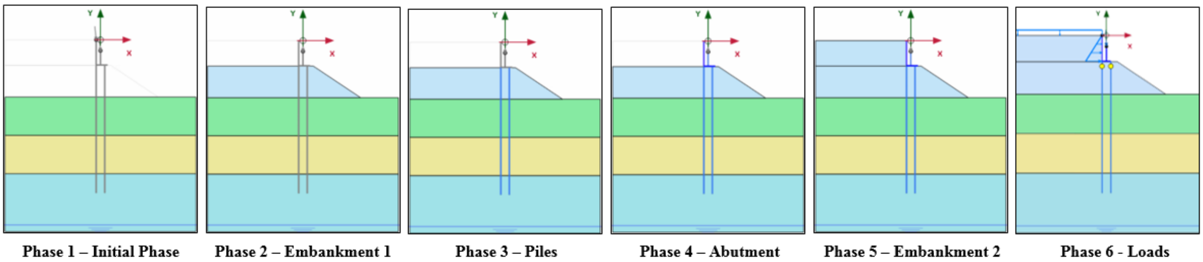

Figure 5. Modeling of the execution sequence for the Bidimensional Continuum Models

3 Results and Discussions

For each analysis series, results were taken in the form of graphics of correlation between the variable parameters and chosen analysis parameters, namely: (1) horizontal longitudinal displacement in the top of the internal pile; (2) maximum bending moment along the internal pile; (3) soil yielding height next to the pile-top.

In parametric analysis 1 (Fig. 6), the increase in diameter promotes expressive reductions in pile-top displacements (-67,1% / -40.4%) and in yielding height (-66,7% / -47.2%). Also, the bending moments nearly doubled in Method 2 models, partially due to the transfer of efforts from the external to the internal pile. This phenomenon is not observable in Method 1, which presents constant bending moments in all models.

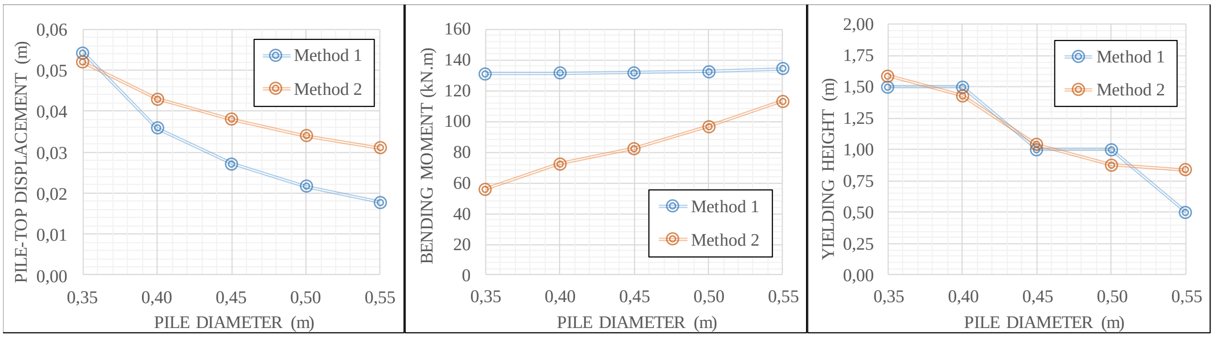

Figure 6. Results for the parametric analysis 1 – variation in pile diameter (m).

In the second analysis (Fig. 7), the increase in the embankment height promotes greater increase in displacements up to the third interval (+110,1%/+111,1%), than onwards (+1,8%/+18,4%). Both methods register a peak bending moment in the second interval (H_{at} = 2,50 m) of +6,5%/+22,0% in relation to the standard design.

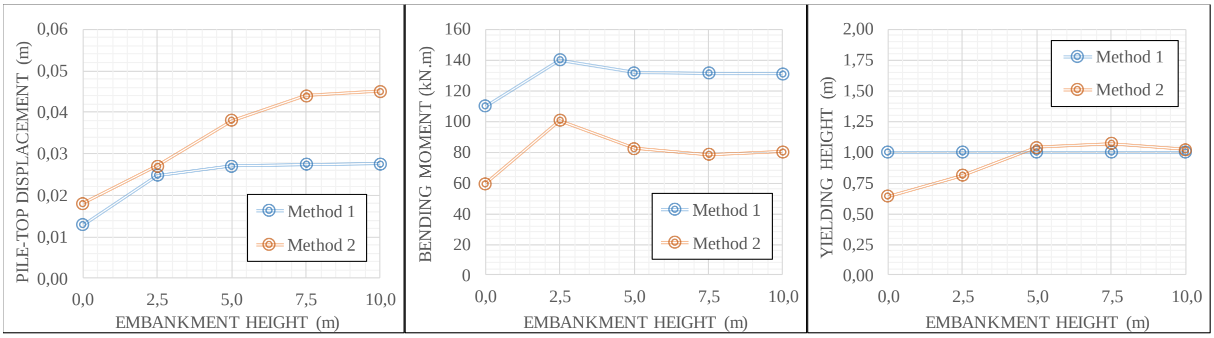

Figure 7. Results for the parametric analysis 2 – variation in embankment height (m).

In the third analysis (Fig. 8), all results for Method 1 models are equal, since no modification is made due to the variation of the spacing to the slope. Method 2, on the other hand, shows an almost linear variation for the 3 graphics, with a decrease of 10.0%, 14.1% and 30.4% for pile-displacement, maximum bending moment and yielding height, respectively.

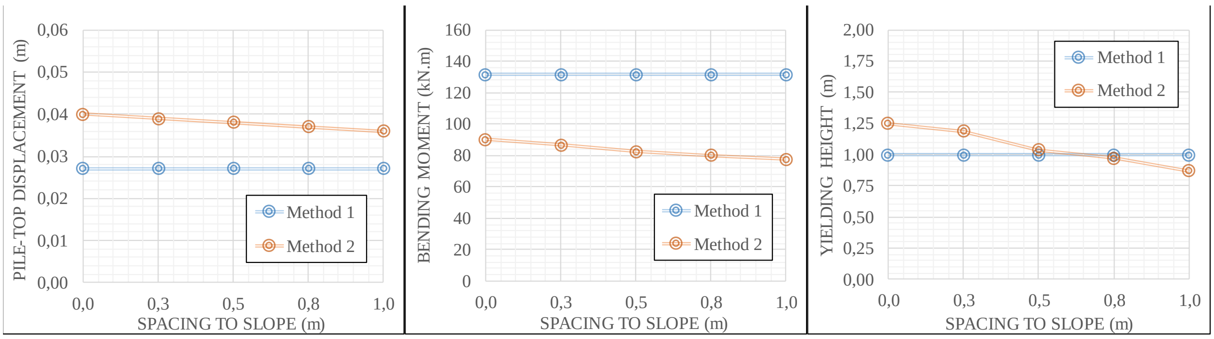

Figure 8. Results for the parametric analysis 3 – variation in spacing to embankment slope (m).

4 Conclusions

In general, there is a good convergence between the results obtained by the two modeling methods, in terms of absolute values of stresses/strains and trends for each analysis. Hereupon, some important conclusions can be drawn regarding the parametric study and the conveniences, difficulties and application of each modeling method.

In parametric analysis 1, the decrease in diameter promotes a fourth order reduction in pile flexural stiffness and so, as expected, both methods show a non-linear evolution of the pile displacements at the top, corresponding to a reduction in the stiffness of the pile-soil system. It is also possible to notice a lower concentration of tensions in the soil with increasing diameter, which is evidenced by the reduction in the yielding height of the soil.

In parametric analysis 2, the variation in embankment heights promote similar trends for both methods. In the first interval, the abutment is based directly on natural soil, so the displacements and bending moments are much lower than in the standard model. In the second point (2.5m high embankment), an interesting phenomenon occurs: the natural soil pulls more load towards it, bringing down reactions and producing a peak bending moment. After a certain depth, the variation of the soil profile is no longer relevant, which demonstrates the well-established idea that the initial meters of soil are the most important for the analysis of laterally loaded piles.

In parametric analysis 3, the variation in spacing between the abutment and beginning of the slope alters the confinement condition at the top of the pile, reflecting a greater displacement as the spacing is reduced. This can only be captured by the 2D Continuum Models (Method 2), since the adopted Winkler model does not have a specific criterion to evaluate this deconfinement effect. In structural design procedures, when foundations are places on sloping terrain, it is a common practice to despise the upper layers of soil (1~3 m), but this does isn't always applied to the SSI evaluation of abutments in conditions as those presented here.

The 3D Winkler Models method presents itself as a great alternative for modeling complex geometries, as it is the case of an abutment, offering much practicality to the calculation. It can produce good quality results in terms of deformations of the soil, as long as an adequate Winkler model is adopted for the soil, along with an adequate yielding criterion. The downside of this method is that there is no proper and precise way of considering deconfinement effects such as the one identified in the parametric study. On another note, bending moments were usually higher for this method, due to the fact that the upper spring is already positioned at a depth of 0.25 m below the abutment. It is advisable that springs be placed with a short spacing between them (generally ≤ 50 cm).

The 2D Continuum Models method allows a more reliable soil characterization (through several constitutive models), the verification of deconfinement effects and the establishment of an executive sequence for the abutment, since it makes a progressive non-linear stress/strain calculation. On the other hand, it doesn't allow an adequate consideration of the bridge's transversal loads (e.g. wind) and the necessity of planification of the 3D problem leads to the adoption of hypotheses that introduce uncertainties to the model, making it less reliable.

The study presented in this paper is part of an ongoing research on the topic of SSI in bridge abutments, so the conclusions obtained here are only partial and preliminary. Important analysis factors, such as the non-linearity of the concrete material and the variation in soil resistance and deformability parameters, are still to be studied.

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