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Comparison between predictions of the annual cyclic response of a semi-integral abutment using finite and discrete element methods

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Abstract. This work compared predictions of the annual cyclic response of a semi-integral abutment retaining a granular backfill using the Finite Element Method (FEM) and the Discrete Element Method (DEM). Finite element (FE) and discrete element (DE) models were developed based on the field data collected from an instrumented and monitored semi-integral abutment. A 15-node triangular FE mesh was used to discretize the backfill in the simulation with FEM while spherical particles were used to represent the backfill in the simulation with DEM. A ± 5-mm lateral displacement was imposed on the abutment to simulate the effects of expansion and contraction of the bridge superstructure due to annual temperature variations. The cyclic sequence of imposed lateral displacements was chosen to simulate the abutment lateral movements after the bridge construction completion in the summer season. Results showed that a good agreement was found between the numerical simulations using FEM and DEM. Lateral earth pressures on the abutment and vertical displacement of the backfill surface increased with annual cycles for both methods. Values of peak wall reaction ratio were similar in both methods while higher values of settlement were observed using DEM compared to FEM.

Keywords: semi-integral bridge, numerical simulation, finite element method, discrete element method.

1 Introduction

Integral and semi-integral bridges have become alternatives to conventional bridges, which have presented problems associated with expansion joints. However, the lack of expansion joints, characteristic of integral and semi-integral bridges, has led to a complex interaction between the backfill and the abutment of the bridge due to cycles of expansion and contraction of the superstructure induced by temperature variations, which requires in-depth assessment.

Numerical simulations using Finite Element Method (FEM) have been carried out to predict the cyclic response of the backfill-abutment system of integral and semi-integral bridges. Caristo et al. [1] simulated the response of a compact sand backfill upon cyclic lateral displacements of an integral abutment. The results showed that the lateral earth pressures built up quickly during the first cycles before reaching a tendency of stabilization. Moreover, settlement was observed behind the abutment with heave occurring at greater distances from the abutment. Similar results were also observed in numerical simulations performed by Silva et al. [2] involving a gravel backfill and a semi-integral abutment.

Despite being widely used to study the behavior of soils, Donzé et al. [3] affirm that the use of the FEM presents limited results when the soil undergoes large strains and fracture propagations. According to Cundall [4], an alternative is the use of the Discrete Element Method (DEM), since it can simulate the soil mass as particles. Zorzi et al. [5] used DEM to study the effects of cyclic loadings on a granular material retained by a wall. The results showed an increase of the horizontal force on the wall and an increase of the vertical displacements of the soil surface with the cycles. The authors associated the obtained results with the mechanism of material densification and microstructural changes. Similar results were also observed in numerical simulations performed by Ravjee et al. [6].

Understanding the potential of the DEM in predicting the soil behavior behind the bridge abutments under cyclic lateral displacements is essential to expand the possibilities of numerical methods applied for this purpose. Therefore, this work compared predictions of the annual cyclic response of a semi-integral abutment retaining a granular backfill using the Finite Element Method (FEM) and the Discrete Element Method (DEM). Finite element (FE) and discrete element (DE) models were developed based on the field data collected from an instrumented and monitored semi-integral abutment. Emphasis is placed on examining the lateral earth pressures on the abutment and the vertical displacements of the backfill surface throughout cycling.

2 Numerical modeling

2.1 FE model

A plain-strain FE model (Fig. 1) of the backfill-abutment system was developed by using the software Plaxis 2D version 2016 developed by Plaxis BV [7]. The model is composed of a 6.35-m thick silty sand layer over a 13.5-m thick sandy clay layer. The backfill in contact with the abutment is a free-draining material composed of industrially produced gravel particles from large pieces of crushed rock. The abutment consists of a 1.05-m high and 0.3-m thick reinforced concrete wall supported by 6.6-m long driven steel sheet piles connected to 0.83-m wide and 0.75-high reinforced concrete pile cap beam. The model boundaries extend to a length of 40 m in the horizontal direction and 20 m in the vertical direction. These dimensions were chosen based on the information of Knappett et al. [8] and were assumed to be enough to exclude boundary effects.

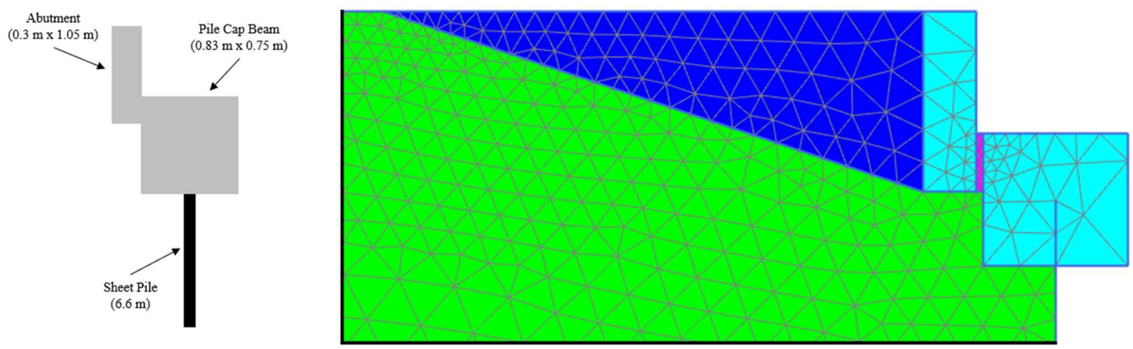


Figure 1. Semi-integral abutment (left) and finite element mesh (right).

The soil materials and the reinforced concrete were modeled by using a mesh with 4,410 15-node triangular solid elements. The lateral boundaries of the mesh are free to mode in the vertical direction and fixed in the horizontal direction while the base of the mesh is constrained from moving in both horizontal and vertical directions. The sheet piles were modeled by plate elements with the same axial and flexural stiffnesses of the sheet piles used in the abutment construction.

The lateral displacements of the abutment due to expansion and contraction of the bridge superstructure were simulated by imposing prescribed horizontal displacements at the top of the abutment. The imposed prescribed horizontal displacements (d) were calculated by Eq. (1), as suggested by Karalar and Dicleli [9]:

$$d = 0.5\alpha L \Delta T \tag{1}$$

where α is the coefficient of material thermal expansion of the bridge superstructure, L is the length of the bridge superstructure, and ΔT is the temperature variation of the bridge superstructure. A value of $\alpha = 10.8 \times 10^{-6}$ °C was assumed for the concrete, which is within the range recommended by AASHTO [10], and a value of L = 20.53 m was adopted based on the characteristics of the bridge. A value of ΔT = 45 °C was adopted to consider the annual temperature variation for the location of the bridge as recommended by AASHTO [10].

The stress-strain behavior of the soil materials was represented by the Hardening Soil hyperbolic constitutive model while the stress-strain behavior of the structural materials was represented by linear elastic constitutive model. Soil-structure interaction was taken into consideration in the model by using interface elements with strength reduction factors (R_{inter}) equal to 0.5 for the soil-steel interface and 0.7 for the soil-concrete interface. The R_{inter} values were chosen based on the suggestions made by Brinkgreve et al. [11].

The parameters of the clayey soil adopted to the numerical model were unsaturated unit weight (γ_{unsat}) equal to 19 kN/m³, saturated unit weight (γ_{sat}) equal to 22 kN/m³, secant stiffness modulus at 50% of the peak load (E_{50}) equal to 60 MPa and undrained shear strength (S_u) equal to 210 kPa. The parameters adopted for the sandy soil were γ_{unsat} equal to 17 kN/m³, γ_{sat} equal to 20 kN/m³, E_{50} equal to 40 MPa, effective cohesion (c') equal to 15 kPa and effective internal friction angle (ø') equal to 31.5°. The parameters adopted for the gravel were γ_{unsat} equal to 20 kN/m³, γ_{sat} equal to 23 kN/m³, E_{50} equal to 32 MPa, c' equal to 1 kPa and ø' equal to 40°. These soil parameters were estimated based on values for typical soil types proposed by Poulos and Davies [12], Stroud and Butler [13], Mesri [14], Kulhawy and Mayne [15], and Tomlinson [16]. The parameters for the concrete adopted in the numerical model were unit weight (γ) equal to 25 kN/m³, Young's modulus (E) equal to 30 GPa and Poisson's ratio (ν) equal to 0.2. Parameters used for the steel sheet piles were weight (w) equal to 1.182 kN/m/m, normal stiffness (EA) equal to 3.163 x 10^6 kN/m, flexural rigidity (EI) equal to 73.27 x 10^3 kN·m²/m and ν equal to 0.3. The structural parameters were estimated based on information from AASHTO [10] and Gerdau [17].

2.2 DE model

A 3D DE model (Fig. 2) of the backfill-abutment system was built by using the software Rocky DEM developed by ESSS [18]. The model boundaries extended to a length of 0.3 m, a width of 0.15 m and a height of 0.35 m. The model was subjected to a gravitational acceleration equal to 3 times the gravitational acceleration of the Earth so that the dimensions of the model match the dimensions of the FE model. The backfill soil was modeled by using spherical particles with diameter of 7 mm, and the abutment was modeled by using a plate with the same stiffness adopted for the abutment in the FE model. The lateral displacements of the abutment due to expansion and contraction of the bridge superstructure were simulated by imposing prescribed horizontal displacements at the top of the abutment with the same values applied to the FE model. A movement frequency of 0.5 Hz was adopted to avoid inertial effects, as suggested by Ravjee et al. [6].

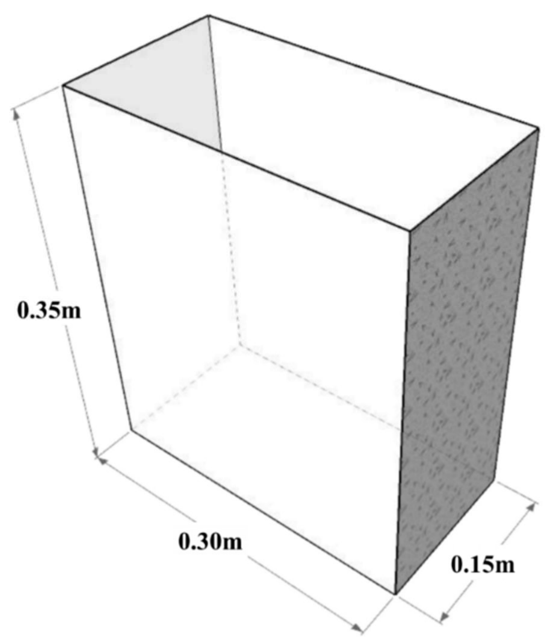


Figure 2. Dimensions of the DE model.

The parameters adopted to the gravel particles were E equal to 60 MPa and ν equal to 0.3, based on the study of Yoon et al. [19]. A coefficient of static friction of 0.6 and a coefficient of dynamic friction of 0.6 were adopted, based on the study of Coetzee [20], to represent the interaction between the model elements. A coefficient of restitution of 0.5 and a rolling resistance of 0.01 were assumed, following Ravjee et al. [6]. Constitutive models of linear hysteresis and Coulomb's linear spring-limit were used to determine the normal and tangential contact forces, respectively, as suggested by Tavares et al. [21] and Katinas et al. [22].

3 Results and discussions

Figure 3 shows the evolution of the peak wall reaction ratio (K_{wp}), obtained according to Eq. (2):

$$K_{wp} = \frac{P_{max}}{0.5\gamma h^2}, \tag{2}$$

where P_{max} is the maximum total soil lateral force acting on the abutment wall per unit length, γ is the unit weight of the backfill material, and h is the abutment height in contact with the backfill. It is possible to observe that, in general, the values of K_{wp} obtained with the FE model presented a good match with the values of K_{wp} obtained with the DE model. In both models, K_{wp} presented a nonlinear increase with a decreasing rate within the first cycles and then trended to a stabilization after 10th cycle. Similar results were observed in the studies of Caristo et al. [1], Silva et al. [2], Zorzi et al. [5] and Ravjee et al. [6].

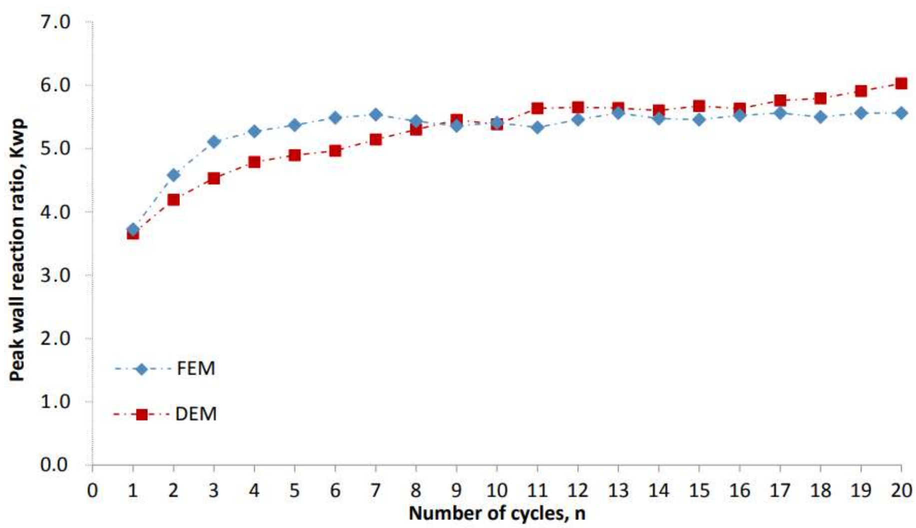


Figure 3. Peak wall reaction ratio on the abutment.

Figure 4 shows the evolution of the maximum settlement (ρ_{max}) of the backfill surface. It is noted a sharp increase in the ρ_{max} in the first 5 cycles, followed by a nearly linear increase in the next cycles, for the FE model. On the other hand, ρ_{max} increased with the cycles according to a nonlinear fashion with a slight decreasing rate for the DE model. In both models, no tendency of stabilization could expressly be identified with increasing cycles. Furthermore, the values of ρ_{max} obtained using the DE model were slightly lower that those obtained using the FE model in the first cycles and then became significantly higher from 6th cycle. Similar results were observed in the studies of Caristo et al. [1], Silva et al. [2], Zorzi et al. [5] and Ravjee et al. [6].

The results presented in Figs. 3 and 4 can be explained by the densification underwent by the backfill soil due to the accumulation of compression volumetric strains with cyclic loading. In a typical cycle of lateral displacement of the abutment, there are basically two movement directions: passive and active directions. In the active direction, the abutment displaces away from the backfill, and the soil slips downward toward the gap developed between the abutment and the backfill. On the other hand, in the passive direction, the abutment displaces against the backfill, and the soil compressed toward the backfill. The initial position of the backfill surface in each cycle is not significantly recovered during cyclic lateral displacements of the abutment due to the plastic behavior of the soil, according to Hovarth [23], and therefore the backfill surface settles as the cycles go by, as shown in Fig. 5.

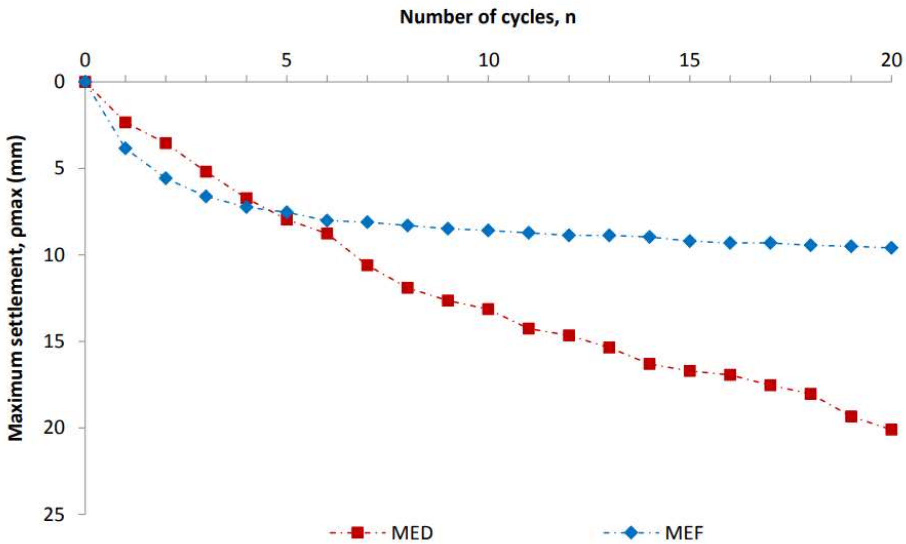
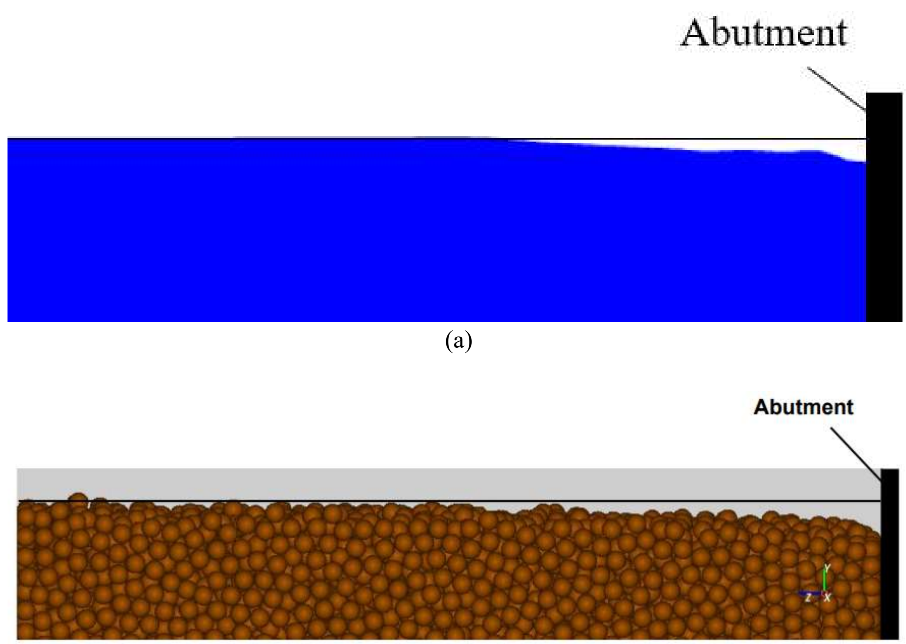


Figure 4. Maximum settlement of the backfill surface.



(b)

Figure 5. Vertical displacement profile of the backfill surface along the distance from the abutment after 20 cycles: (a) FEM; (b) DEM.

4 Conclusions

This present work compared predictions of the annual cyclic response of a semi-integral abutment retaining a granular backfill using the Finite Element Method (FEM) and the Discrete Element Method (DEM). Finite element (FE) and discrete element (DE) models were developed based on the field data collected from an instrumented and monitored semi-integral abutment. The main findings of this study are as follows:

- The peak wall reaction ratio obtained with the FE model presented a good match with the one obtained with the DE model. In both models, the peak wall reaction ratio presented a nonlinear increase with a decreasing rate within the first cycles and then trended to a stabilization after 10^{th} cycle.

➢ The maximum settlement sharply increased in the first 5 cycles, followed by a nearly linear increase in the next cycles for the FE model. On the other hand, the maximum settlement increased with the cycles according to a nonlinear fashion with a slight decreasing rate for the DE model. In both models, no tendency of stabilization could expressly be identified with increasing cycles. Furthermore, the values of the maximum settlement obtained using the DE model were slightly lower that those obtained using the FE model in the first cycles and then became significantly higher from 6^{th} cycle.

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