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A CPT-based method for monotonic loading of large diameter monopiles in sand

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ABSTRACT: A joint academia-industry project, the Pile Soil Analysis (PISA) project, resulted in an empirical method for assessing the monotonic lateral loading response of large diameter monopiles. The method predicts four soil reactions, namely the distributed load and the distributed moment along the pile shaft, the pile base shear and the pile base moment. The method considers pile load test data and 3D numerical modelling. A 1D framework allows prediction of the four soil reactions. In this paper, a CPT-based approach is proposed to derive the four soil reaction components for use in a 1D model for conceptual design of monopiles in sand subject to monotonic lateral loading. The approach relies on results from 3D finite element analyses that were performed considering soil conditions for a sand site used in the PISA project (Dunkirk site). The results are compared to pile load test data from the PISA project, showing good agreement, particularly for load levels related to the serviceability limit state.

1 INTRODUCTION

Monopiles are commonly used as foundations for offshore wind turbine generators (WTGs). The current trend in the ever-growing offshore wind energy sector is for WTGs to becoming bigger which evidently leads to requirements for monopiles with large diameters up to 10 m to support the superstructure. It is expected that the ratio of embedded length to diameter, *L/D* (or slenderness ratio) of monopile foundations for the 10 MW+ next-generation wind turbines could be in the range between 2 and 6 (Panagoulias et al., 2018). Such structures are categorised as intermediate foundations according to ISO (2016).

An industry standard approach for assessing monopile lateral response was a *p-y* method for long slender piles, adjusted to large diameter monopiles. The *p-y* method is based on the Winkler assumption according to which the soil surrounding the pile is modelled as a set of uncoupled, non-linear, elastoplastic springs which define the lateral pressure (p) applied to the pile at a given depth, as a function of the lateral displacement (y). The method, however, does not capture the physics of the monopile behaviour accurately.

A joint academia-industry project, the Pile Soil Analysis (PISA) project, resulted in an empirical method for assessing the monotonic lateral loading response of large diameter monopiles. The method is based on conventional models for caisson design, predicting four soil reactions, namely the distributed load and the distributed moment along the pile shaft, the pile base shear and the pile base moment (Figure 1). The PISA schematisation excludes torsional foundation loading (Burd et al., 2020). The empirical method considers pile load test (PLT) data and 3D numerical modelling. A 1D framework allows prediction of the four soil reactions and requires, for sands, profiling of three soil parameters, namely the relative density, the vertical effective stress and the shear modulus at small strain.

In this paper, a CPT-based approach is proposed to derive the four soil reaction components for use in a 1D model for conceptual design of monopiles in sand subject to monotonic lateral loading. The approach relies on results from 3D finite element (FE) analyses that were performed considering soil conditions for a sand site used in the PISA project (Dunkirk site). The results are compared to PLT data from the

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PISA project, showing good agreement, particularly for load levels related to the serviceability limit state (SLS).

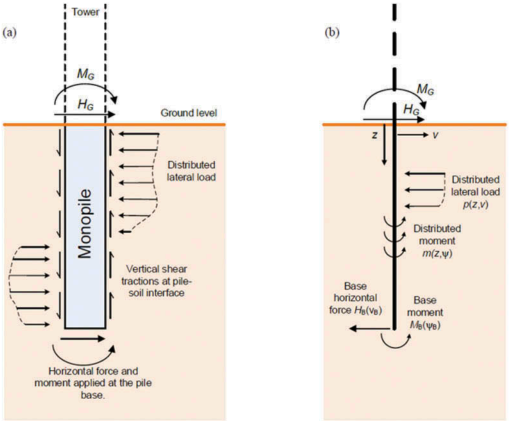

Figure 1. (a) Schematised soil reaction components acting on a laterally loaded monopile; (b) 1D design model. (after Burd et al., 2020).

2 DATABASE

Several piles driven into dense sand at the Dunkirk site were tested during the PISA project in order to investigate the effect of different design aspects such as pile geometry, load ratio, unloading/reloading behaviour and creep. In this paper, the results from three PLTs on medium diameter piles, D = 762 mm (i.e. DM3, DM7 and DM4; see Table 1) were compared to results from 3D FE analyses. This allowed, using the FE-derived resistance components, development of a CPT-based method.

Table 1. Geometry of PISA piles considered in this study (Taborda et al., 2020).

Pile	Diameter (m)	Length (m)	Slenderness ratio (-)	Wall Thickness (mm)
DM3	0.762	6.1	8.0	25
DM4	0.762	4.0	5.3	14
DM7	0.762	2.3	3.0	10

3 FINITE ELEMENT MODEL

3.1 *General*

The commercial software packages Plaxis 3D and Plaxis Monopile Design Tool, MoDeTo (Plaxis BV, 2018), were used to perform the FE analyses and extract the soil reaction curves. Through the latter, the monopile was modelled and then the FE analysis was performed in Plaxis 3D. Finally, each of the four soil reaction curves were extracted via MoDeTo at different load steps and pile depths.

3.2 *Soil model*

The Dunkirk test site was characterised using a range of in situ tests and advanced laboratory testing (Zdravković et al., 2020). Several CPTs were performed next to the test pile locations and other key locations. Figure 2 presents the average cone resistance at the site. The general soil stratigraphy is shown in Table 2. The water table is found approximately at 5.4 m below ground level.

The Hardening Soil small strain model (HSsmall) was used as soil constitutive model. The model was calibrated against available soil data from the Dunkirk site, including CPTs, seismic CPTs and laboratory tests such as triaxial tests with bender element measurements. The calibration process included study of several CPT-based and empirical parameter formulations from the literature (e.g. Robertson and Cabal, 2015; Brinkgreve et al., 2010), investigation of parameter interdependency and performance of single element test predictions.

The focus of the CPT-based approach was accurate representation of the SLS, according to which the allowance for the total permanent tower axis tilt rotation is 0.5° (DNVGL, 2016). By analysing the data obtained from the PISA project, this limit is reached at approximately 30% to 50% of the maximum horizontal load applied to the monopiles during pile load testing; hence only that portion of the horizontal load-deformation curve was considered for the HSsmall calibration process.

Table 3 shows an overview of the soil parameter values for the calibrated HSsmall soil model.

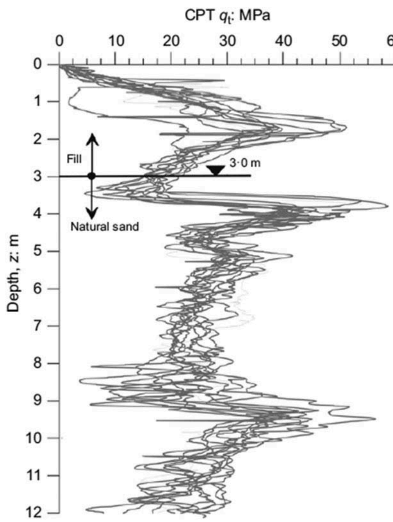

Figure 2. Cone resistance profile at the Dunkirk site (Zdravković et al., 2020).

Figure 3 illustrates the comparative results between the measured horizontal load-displacement responses from the PLTs and the predicted responses from the performed 3D FE analyses. A fairly good match is observed at the initial part of the curves, rendering the prediction of the stiffness response, which was of primary interest, satisfactory.

Additional (fictional) piles were considered in order to expand the database and check the influence of pile geometry on each of the four soil reaction components. Table 4 shows an overview of the additional piles considered for the sensitivity analyses.

4 SOIL REACTION CURVES

4.1 *Distributed lateral load (p-y)*

The relationship between p and y along the pile shaft has been widely studied. In recent years several formulations for *p-y* curves have been developed by taking into consideration the cone penetration test and considering the link between cone resistance (q_c) and in situ horizontal effective stress of the soil (Houlsby and Hitchman, 1988). An overview of some of those formulations together with their corresponding authors is shown below:

Table 2. Soil stratigraphy at the Dunkirk site (Zdravković et al., 2020).

Depth (m)	Material	Description
0 - 3	Hydraulic fill	Sand dredged from offshore Flandrian deposits
3 - 30	Flandrian sand	Marine sand deposited during three local transgressions
> 30	Ypresienne clay	Eocene marine clay located beneath the southern North Sea

Table 3. Summary of soil parameters for HSsmall model.

Depth [m]	0-3	3-5.4	5.4-9	9-12.2	12.2-15
γ' [kN/m^3]	19.1	20.8	11.0	11.8	9.8
$E_{50,ref}$ [MPa] (=$E_{oed,ref}$)	250	223	174	202	87
$E_{ur,ref}$ [MPa]	751	668	523	605	260
φ' [deg]	46	45	43	42	37
ψ[deg]	15	9	9	9	9
$\gamma_{0.7}$ [-]	1e-4	1.3e-4	1.3e-4	1.3e-4	1.3e-4
$G_{0,ref}$ [MPa]	321	285	223	259	111
R_f [-]	0.88	0.91	0.91	0.91	0.91
K_0 [-]	0.5	1.0	0.8	0.7	0.7

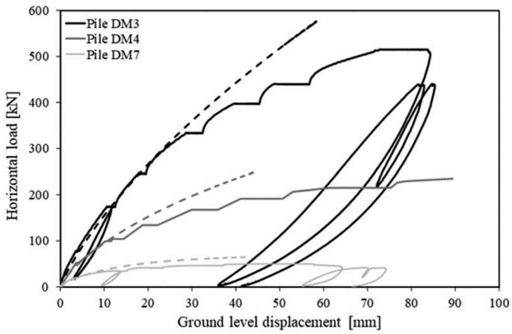

Figure 3. Comparison of ground level load-displacement for three piles tested during the PISA project (see Table 1 for details). Solid lines represent the results of the pile load tests (after Taborda et al., 2020), dashed lines represent the results of the 3D FE calculations.

Table 4. Geometry of additional (fictional) piles considered in the study.

Pile	Diameter (m)	Length (m)	Slenderness ratio (-)	Wall Thickness (mm)
DM3A	1.0	6.1	6.1	25
DM3B	1.2	6.1	5.1	25
DM3D	2.0	6.1	3.1	25
DM7B	0.762	3.0	3.9	10
DM7D	0.762	4.7	6.1	10
DM7E	0.762	3.8	5.0	10
PL1	0.762	15.0	19.7	20
PL2	0.5	15.0	30.0	25
PL3	0.6	21.0	35.0	30

$$p = \min\left(2D(\gamma' z)^{0.33} {q_c}^{0.67} \left(\frac{y}{D}\right)^{0.50}, Dq_c\right) \quad (1)$$

$$p = 2.84D(\gamma' D)\left(\frac{q_c}{\gamma' \cdot D}\right)^{0.72}\left(\frac{y}{D}\right)^{0.64} \quad (2)$$

$$p = 3.6D(\gamma' D)\left(\frac{q_c}{\gamma' \cdot D}\right)^{0.72}\left(\frac{y}{D}\right)^{0.66} \quad (3)$$

$$p = \begin{cases} 4.5G_{max}\, y, \; for \; \frac{y}{D} \leq 0.0001 \\ p_u f(y), \; for \; \frac{y}{D} > 0.01 \end{cases} \quad (4)$$

where Equation 1 is by Novello (1999), Equation 2 is by Dyson & Randolph (2001), Equation 3 is by Li et al. (2014), Equation 4 is by Suryasentana & Lehane (2016), D = pile diameter, γ' = effective unit weight of soil, z = depth, G_{max} = small strain shear modulus, p_u = ultimate lateral soil resistance (for more details refer to Suryasentana & Lehane, 2016) and *f(y)* = exponential function that depends on lateral displacement (for more details refer to Suryasentana & Lehane, 2016).

Equations 1 to 4 were used to derive *p-y* curves which were then inserted in a 1D Timoshenko beam model for modelling of the pile-soil lateral behaviour. Long slender (fictional) piles ($L/D \geq \sim 20$) were considered so that the influence of the other three soil reaction components (distributed moment, base shear and base moment) to the overall response is negligible (see Table 4; piles PL1, PL2 and PL3). The results obtained from the 1D model were thereafter compared with results from 3D FE analyses and it was found that Equation 2 (Dyson and Randolph, 2001) was providing the better match and was thus selected to define the *p-y* component for this study.

4.2 *Distributed moment (m-ψ)*

The distributed moment (m) is caused by the vertical shear stresses along the pile shaft due to pile rotation (ψ). It is considered that m is linked to p, which is acting as a normal force along the shaft, through consideration of the pile-soil interface friction angle (δ) and the pile geometry (L and D). A fitting parameter, $F_{m\psi}$, was adopted in order to investigate the relationship between the aforementioned parameters for the range of pile geometries considered.

$$F_{m\psi} = p \cdot D \cdot tan\delta \tag{5}$$

where δ = pile-soil interface friction angle taken as $2/3\varphi'$.

By considering the maximum value of the distributed moment at every slice along the pile shaft obtained from the 3D analysis, m_{max}, the influence of L/D on the ratio $m_{max}/F_{m\psi}$ was investigated (Figure 4) and a formulation for determination of m is proposed (Equation 6). The relatively low R^2 value is attributed to the small dataset and the fact that the proposed linear trend might be less suitable as L/D increases.

$$m = 0.07 \cdot p \cdot D \cdot \tan(\delta) \cdot \left(\frac{L}{D}\right)^{0.7} \tag{6}$$

The distributed load and distributed moment are soil reactions along the pile shaft, thus the pile was divided into slices and both soil reactions were computed per slice. By considering geometric continuity

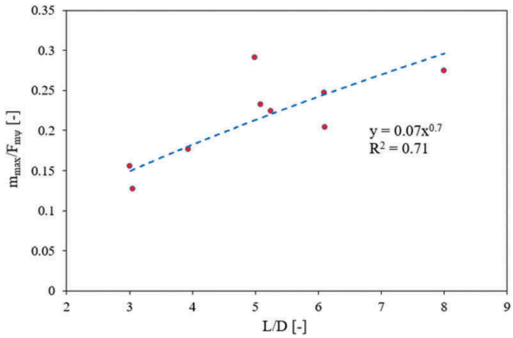

Figure 4. Distributed moment ratio as function of the slenderness ratio *L/D*.

of the rigid pile, the rotation can be obtained from the horizontal displacements. Figure 5 shows the distributed moment for various slices along the shaft of pile DM4, obtained both from the 3D FEA and the proposed CPT-based formulation (Equation 6).

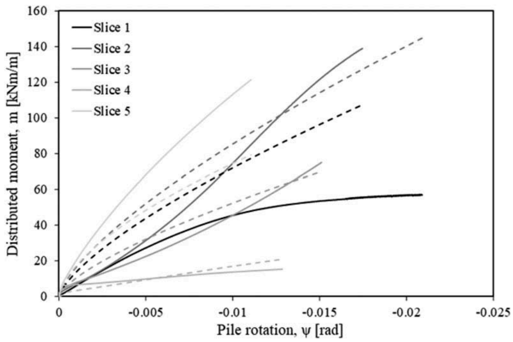

Figure 5. Pile DM4 distributed moment along each slice. Solid lines correspond to the results from 3D FE models and dashed lines correspond to the results from the proposed CPT-based formulation.

4.3 *Horizontal base force (H_B)*

Due to the applied force at the pile head, the base of the pile tends to move in the opposite direction, generating a horizontal base force (H_B). H_B was linked to the base displacement, v_b, via a fitting parameter, F_{HB}, which is a function of the q_c at the pile base and the pile geometry (Equation 7). Figure 6 shows the relationship between F_{HB} and the ultimate horizontal base force, $H_{B,ult}$, for all piles in the considered database.

$$F_{Hb} = \frac{q_c \cdot D^2}{(L/D)^{0.36}} \tag{7}$$

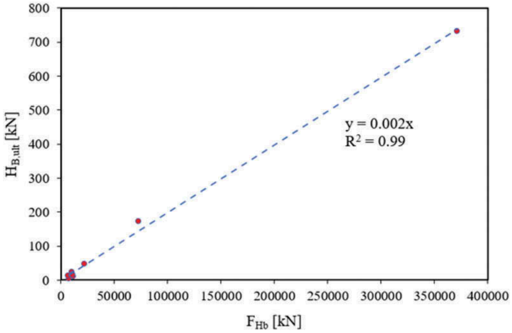

Figure 6. Fitting parameter F_{HB} versus the ultimate horizontal base force, $H_{B,ult}$.

Curve fitting with results from the Plaxis 3D models of the database resulted in the following bi-linear relationship:

$$H_B = \begin{cases} \frac{0.00235 q_c}{\left(\frac{L}{D}\right)^{0.36}} \left(\frac{\pi D^2}{4}\right) \left(\frac{v_b}{0.0005D}\right), \; for \frac{v_b}{D} \leq 0.0005 \\ \frac{0.00235 \mathrm{q_c}}{\left(\frac{\mathrm{L}}{\mathrm{D}}\right)^{0.36}} \left(\frac{\pi \mathrm{D}^2}{4}\right), \; for \frac{\mathrm{v_b}}{\mathrm{D}} > 0.0005 \end{cases} \tag{8}$$

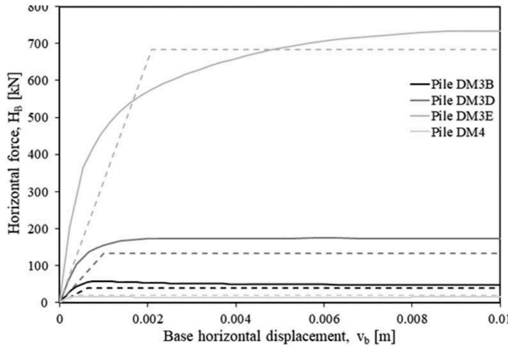

Figure 7. Pile base horizontal reactions. Solid lines correspond to the results from 3D FE models and dashed lines correspond to the results from the proposed CPT-based formulation.

Figure 7 shows the pile base horizontal reactions obtained from the 3D FEA in comparison to the reactions from the proposed CPT-based formulation (Equation 8) for a selection of piles from the database.

4.4 *Base moment* (M_B)

The base moment is caused by rotation of the pile toe. Similarly to the base horizontal force, the base moment relationship contains a first linear portion followed by a plateau. Curve fitting with results from the Plaxis 3D models of the database resulted in the following bi-linear relationship:

$$M_B = \begin{cases} \frac{0.00171 q_c D}{\left(\frac{L}{D}\right)^{0.52}} \left(\frac{\pi D^2}{4}\right) \left(\frac{\psi_b}{0.0007D}\right), \; for \frac{\psi_b}{D} \leq 0.0007 \left[\frac{rad}{m}\right] \\ \frac{0.00171 q_c D}{\left(\frac{L}{D}\right)^{0.52}} \left(\frac{\pi D^2}{4}\right), \; for \frac{\psi_b}{D} > 0.0007 \left[\frac{rad}{m}\right] \end{cases} \tag{9}$$

Figure 8 shows the pile base moment reactions obtained from the 3D FE models and the proposed CPT-based formulation (Equation 9) for a selection of piles from the database.

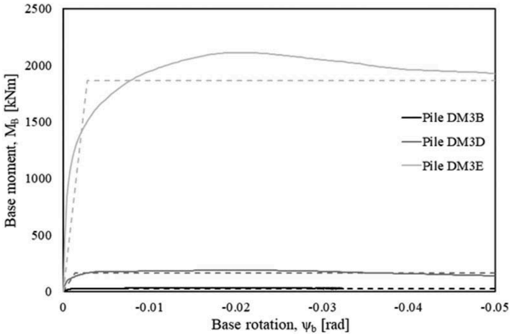

Figure 8. Pile base moment reactions. Solid lines correspond to the results from 3D FE models and dashed lines correspond to the results from the proposed CPT-based formulation.

5 PILE LATERAL RESPONSE

The four soil reaction components, as computed with the use of the proposed equations, were entered in a 1D Timoshenko beam model for modelling of the general monopile response under lateral loading. Results for the piles of Table 1 are shown in Figures 9 and 10. The predictions of the CPT-based method show in general good agreement with the PLTs and the 3D FE analyses for the initial part of the load-displacement curve, i.e. until approximately half the ultimate lateral load (Figure 9). The initial stiffness response of the monopiles is, therefore, fairly captured. Figure 10 depicts the pile deflections below ground level at different loads, all with magnitude lower than half of the ultimate lateral load. Again, a satisfactory agreement between the results of the CPT-based method, the 3D FE analyses and the PLTs is observed.

Figure 9 shows that after a certain level of ground level displacement, the response obtained from the proposed CPT-based method is stiffer than the response obtained from the 3D FE analyses and the PLTs. Therefore, a cut-off point needs to be defined beyond which the proposed

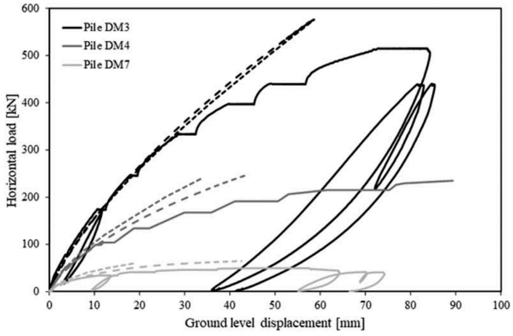

Figure 9. Comparison of ground level load-displacement for three piles tested during the PISA project. Solid lines represent the results of the pile load tests; thinly dashed lines represent the results of the 3D FE calculations; thickly dashed lines represent the results of the CPT-based method.

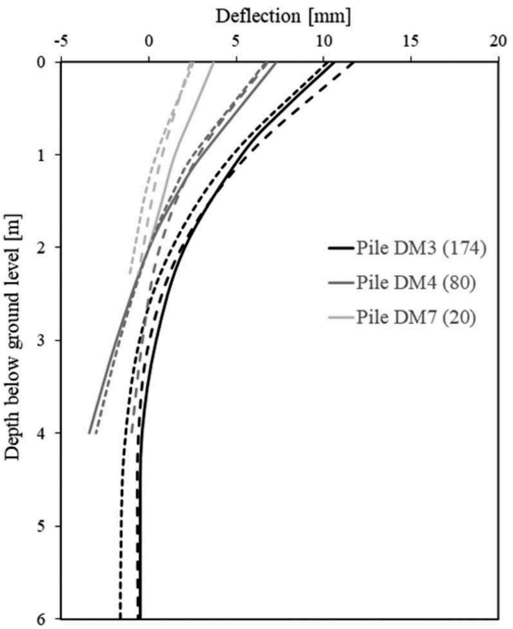

Figure 10. Comparison of deflection at and below ground level for three piles tested during the PISA project. Solid lines represent the results of the pile load tests; thinly dashed lines represent the results of the 3D FE calculations; thickly dashed lines represent the results of the CPT-based method. Values within brackets denote the applied load in kN.

method is less accurate. This point was defined by analysing, for all piles of the database, the difference in stiffness magnitude between the 3D FE analyses and the CPT-based method, at ground level and at all load levels. Consequently, it is recommended that the proposed CPT-based method is used for predictions of monopile lateral response in which the displacements at ground level are not larger than 2% to 3% of the pile outer diameter. This level of deformation generally corresponds with the serviceability limit state of monopiles used in the offshore wind industry.

6 DISCUSSION AND CONCLUSIONS

The paper presents a CPT-based method for predicting the response of laterally loaded monopiles in a sand setting. In order to verify application of the method to full-scale monopiles and until further experience is gained with the use of this method, a FE analysis, considering typical soil conditions of the investigated site and expected pile geometry, is recommended as a minimum.

The method allows for performing monotonic conceptual design calculations for monopile foundations supporting WTGs in a time-efficient manner, requiring only CPT data. Total computing time can be reduced by up to 90 % with respect to performing 3D FE analyses.

The proposed soil reaction formulations were calibrated against soil data from the PISA sand site in Dunkirk and consider a specific limit state, i.e. the SLS. Therefore, applicability of the method to marine sites with significantly different soil conditions (e.g. in terms of strength, stiffness, sand type) than the ones at Dunkirk and/ or for different limit states should be carefully checked. In these occasions, FE analyses are required prior to implementation of the approach shown in this paper to develop a site-specific CPT-based method. Alternatively, the PISA 'numerical-based method' can be employed (Byrne et al., 2017).

The curve fitting process considered the individual soil reactions from the 3D FE analyses and not the actual PLTs, since modelling of each individual soil reaction component based on measured PLT data has been shown to be problematic (Foursoff, 2018).

The proposed CPT-based method provides a representation of the global monopile response under monotonic lateral loading, although the individual soil reactions at a local level can differ considerably between the FE analyses and the CPT-based formulations. The latter can be attributed to factors such as imperfect curve fitting and inherent limitations of the 1D model which cannot accurately represent all mechanisms of soil-pile interaction at a local level.

The CPT-based method should be employed in its entirety, i.e., individual soil reaction components should not be excluded from the analysis or used independently.

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