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A calibration procedure for sand plasticity modeling in earthquake engineering: Application to TA-GER, UBCSAND and PM4SAND

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ABSTRACT: A generalized element-test based calibration methodology is developed. The calibration is based on best matching the response in terms of (a) the liquefaction resistance (*CRR*) curve as a function of the relative density of sand and the equivalent number of uniform cycles required to trigger liquefaction, and (b) experimentally derived shear modulus and damping ratio curves commonly used in practice. Regarding the first part of the calibration procedure, the correlation for the reference cyclic resistance ratio $CRR_{Mw=7.5,\ \sigma v0'=1atm}$ from SPT data by Idriss & Boulanger was combined with (a) the empirical formula of Seed & Idriss that relates the earthquake magnitude to the equivalent number of uniform cycles of the seismic motion, (b) the magnitude scaling factors (*MSFs*) proposed by Idriss that associates the reference cyclic resistance ratio with the actual one and (c) the correction factor for overburden stress K_σ suggested by the NCEER workshops. The developed calibration process is applied to three constitutive models available in the literature. Namely: (a) the TA-GER, (b) the PM4SAND and (c) the UBCSAND. For the last two models both the finite element code PLAXIS 2D and the finite deference one FLAC 2D were used in the calibration procedure. Despite the generally satisfactory performance of all models, re-adjustment of the recommended calibrated parameters may be required in boundary value problems.

1 INTRODUCTION

This paper presents a generalized calibration methodology of two well-known constitutive models for sands namely: UBCSAND, as was presented by Beaty & Byrne (2011) and PM4SAND by Ziotopoulou & Boulanger (2013) and a third plasticity model designated as TA-GER model which was recently developed by Tasiopoulou & Gerolymos (2016). The *p-q* version of the TA-GER model is utilized herein.

The accuracy of an analysis of an earthquake engineering application strongly depends on the versatility of the constitutive model used and the capabilities of the numerical platform in which it is implemented. Two commonly used platforms in geotechnical practice are deployed in the present work, the explicit finite difference program FLAC 2D and the finite element program PLAXIS 2D.

The calibration concerns both drained and undrained conditions of loading and it is based on best matching models response in terms of some well-established experimental curves and empirical correlations commonly used in practice. Single element tests were executed in order to obtain the response. Specifically, undrained stress-controlled D.S.S. tests and drained strain-controlled D.S.S. tests were conducted, considering isotropic consolidation conditions. Isotropic conditions were deliberately chosen, since it is considered that the constitutive models used in this study have been developed to mainly account for such conditions. In addition, the version of the TA-GER model used herein is expressed in the *p-q* stress space and not in the 3-dimensional one, so as to be compatible with anisotropic consolidation stress conditions and capable of reproducing the K_0 effects.

2 CALIBRATION METHODOLOGY

2.1 *Undrained response*

The clean – sand based liquefaction resistance *CRR* curves portrayed in Figure 1(a) compiled by Idriss & Boulanger (2008, 2010) apply only to magnitude 7.5 earthquakes. To adjust the aforementioned reference curves to magnitudes smaller or larger than 7.5, Seed & Idriss (1982) and several investigators afterwards introduced correction factors (Youd et al., 2001) termed "magnitude scaling factors" (*MSFs*). *MSFs* proposed by Idriss (1995) are considered as a lower-bound estimation of the liquefaction resistance according to the recommendations by the 1998 NCEER/NSF workshop participants and are formed the basis of the developed calibration procedure.

Although one could claim a sort of inconsistency between the *CRR* and *MSF* curves from different researchers, the ultimate goal of the proposed calibration methodology is to yield sets of model parameters for a wide variety of *CRR*-*MSF* combinations. In this way, not only the versatility of each used constitutive soil model in capturing diverse liquefaction behaviour is directly tested but the user is able to choose the appropriate combination that better match the measured (in the laboratory) response, avoiding recalibration of the model parameters. For the sake of brevity and due to space limitations, the results for a single *CRR*-*MSF* combination are only presented.

Adopting the Idriss & Boulanger (2008, 2010) correlation of the reference cyclic resistance ratio ($CRR_{Mw=7.5,\ \sigma'v0=1atm}$) with the corrected SPT number $(N_1)_{60cs}$ (Figure 1a) and assuming that $(N_1)_{60cs}$ is related to the relative density according to the Idriss & Boulanger (2008) expression:

$$(N_1)_{60\,cs} = 46\,D_r^{\,2} \tag{1}$$

the liquefaction resistance curve can thus be obtained as a function of the relative density. Then, by using the following curve fitting function (Seed & Idriss, 1982):

$$N = 0.0034\ M_w^{\,4.18} \tag{2}$$

that associates the earthquake magnitude M_w with the equivalent number of uniform cycles of the seismic motion and multiplying the reference cyclic resistance ratio

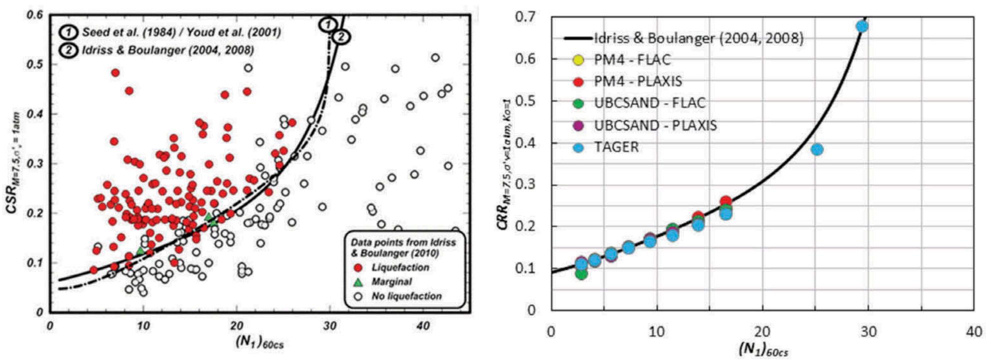

Figure 1. (left) Correlations for cyclic resistance ratio (*CRR*) from SPT data [after Idriss & Boulanger (2010), Seed et al. (1984), Youd et al. (2001), Ziotopoulou & Boulanger (2013)], (right) Computed cyclic resistance ratio against the corrected SPT number of the calibrated models, versus suggested by Idriss & Boulanger (2004, 2008) reference $CRR_{Mw=7.5,\ \sigma'v0=1atm}$, combined with Eq. 3 to account for isotropic consolidation conditions. The numerical results were obtained by simulating undrained cyclic DSS tests with σ'_{v0}=100 KPa and K_0=1.

($CRR_{Mw=7.5,\ \sigma'v0=1atm}$) by the magnitude scaling factor (*MSF*), the liquefaction resistance curve can be explicitly determined as a function of both the relative density and the number of uniform cycles. Since it is arbitrarily assumed that the "observed" $CRR_{Mw=7.5,\sigma'v0=1atm}$ refers to anisotropic consolidation conditions with K_0 = 0.5, it is necessary to convert the anisotropic *CRR* values to equivalent isotropic (K_0 = 1) ones. The adopted transformation expression:

$$CRR_{K_0\neq 1} = \left(\frac{1+2K_0}{3}\right) CRR_{K_0=1} \tag{3}$$

has been frequently applied in the literature (e.g. Idriss & Boulanger, 2010) and is also used for the purposes of this calibration study. To account for the nonlinearity between *CRR* and effective overburden pressure, the correction factor K_σ is introduced for overburden pressures over 1 atm, as follows:

$$CRR_{\sigma'_{vo}\neq 100} = \mathrm{K}_\sigma CRR_{\sigma'_{vo}=100} \tag{4}$$

The expression for K_σ factor by the NCEER (1996, 1998) workshops is:

$$\mathrm{K}_\sigma = \left(\frac{\sigma'_{v0}}{p_a}\right)^{(f-1)} \tag{5}$$

in which σ'_{v0} is the effective overburden pressure, p_a is the atmospheric pressure and f is an exponent that is a function of the relative density *Dr* of sand:

$$f = 1 - \frac{D_r}{2} \leq 0.8 \tag{6}$$

The liquefaction resistance curves derived via the combination of the aforementioned relationships, are reproduced by the calibrated models. The performance of the models in terms of the reference cyclic resistance ratio $CRR_{Mw=7.5,\ \sigma'v0=1atm,\ K0=1}$ as a function of the corrected SPT value is depicted in Figure 1(b) in comparison with the design curve by Idriss & Boulanger (2004, 2008) with the help of Eq. 3. The comparison between computed and suggested values for the cyclic resistance ratio vs number of uniform cycles to cause liquefaction is given in Figure 2 for four selected values of the relative density *Dr* = 30%, 40%, 50% and 60%. The results refer to undrained cyclic direct simple shear tests with uniform stress-controlled loading, under an initial vertical effective stress of σ'_{v0}=100 KPa and K_0 =1. The effect of the overburden stress on the predictions of the calibrated models is illustrated in Figure 3 for the same values of *Dr*. It is noted that the *CRR* value is determined as the cyclic stress ratio required to cause liquefaction in 15 uniform loading cycles. For the TA-GER and the UBCSand models, the onset of liquefaction and associated *CRR* value is deliberately assumed when the excess pore water pressure ratio r_u exceeds 0.97. For the PM4Sand model it was found that liquefaction triggering is better reproduced when the single-amplitude shear strain equals 3% and the excess pore water pressure ratio exceeds 0.97.

It is stated that, while the effect of multidirectional seismic loading on the liquefaction resistance has been imprinted on the red data points on which the calibration procedure has been based (Figure 1b), the conducted numerical load tests are unidirectional. Although the aforementioned effect can be indirectly taken into account by applying a reduction factor on the *CRR* curve (Kramer 1996), treating the multidirectional shaking effects in an equivalently unidirectional way, the authors believe that the proposed unidirectional-based procedure suits better the purposes of a 2D plain strain analysis where the multidirectional nature of the seismic shaking (with respect to the horizontal components of motion) cannot be explicitly considered. Evidently, it should be used with caution in a fully 3D boundary value problem.

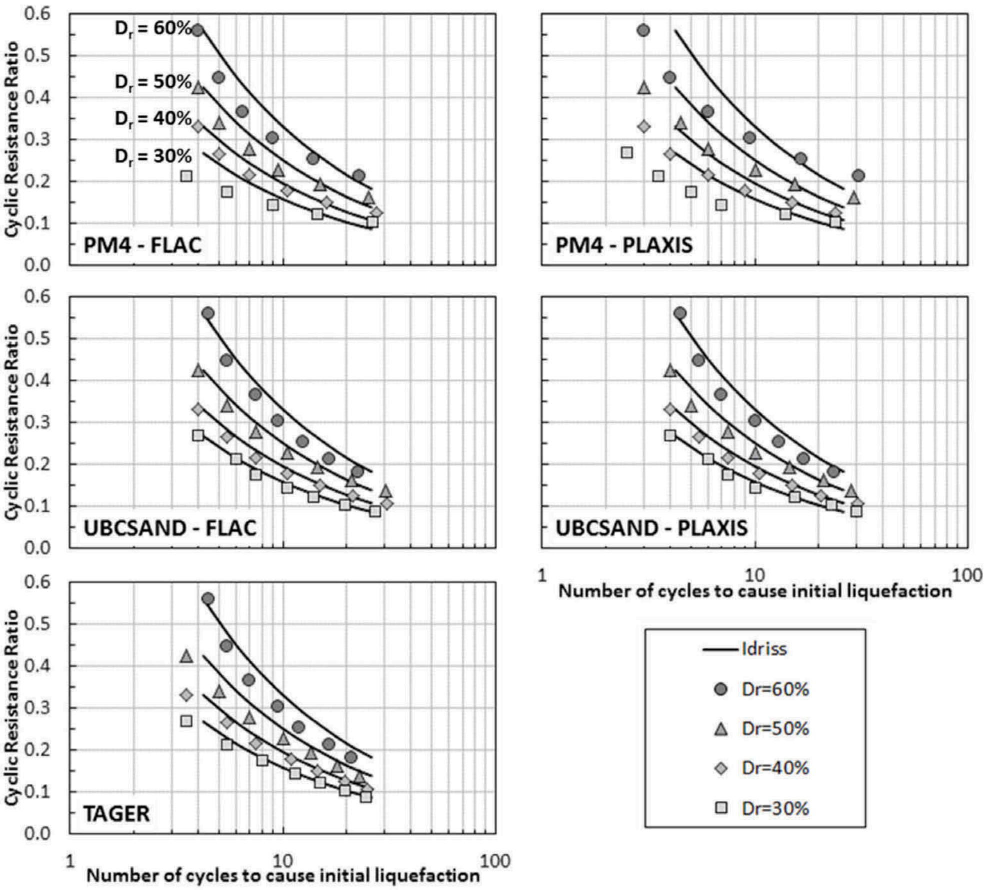

Figure 2. Cyclic resistance ratio as a function of the equivalent number of uniform cycles in order to trigger liquefaction for selected values of the relative density (*Dr*= 60%, 50%, 40%, 30%). Comparison is given between the predictions of the calibrated models and the corresponding curves derived by the Idriss (1995) magnitude scaling factor curve, the Idriss & Boulanger (2004, 2008) reference $CRR_{Mw=7.5,\sigma'v0=1atm}$ and Eq. 3 to correct the reference *CRR* for isotropic consolidation conditions. The numerical results were obtained by simulating undrained cyclic DSS tests with σ'_{v0}=100 KPa and K_0=1.

2.2 *Drained response*

The shear modulus reduction (G/G_{max}) and the equivalent damping ratio values (ξ) versus cyclic shear strain amplitude (γ) for *Dr* =60%, 50%, 40% and 30% are depicted in Figure 4, and compare with a set of pressure independent curves for sand recommended by Seed & Idriss (1970). Cyclic direct simple shear tests were performed, maintaining the same input parameters that were used to calculate the undrained response. Drained conditions were assumed with initial values of σ'_{v0}=100 KPa and K_0=1. The cyclic loading was stain-controlled and the average secant modulus and damping ratio were determined from the 8th cycle of each applied harmonic strain history. While in most of the cases the constitutive models achieve reasonable agreement with the suggested values, the UBCSAND model incorporated in the FD program FLAC predicts damping values excessively higher than those experimentally observed for strain amplitudes greater than 1%.

2.3 *Calibrated parameters*

The results of the calibration procedure regarding the UBCSand and the PM4Sand models are summarized in Table 1. In all cases, the input parameter determined by the user is the

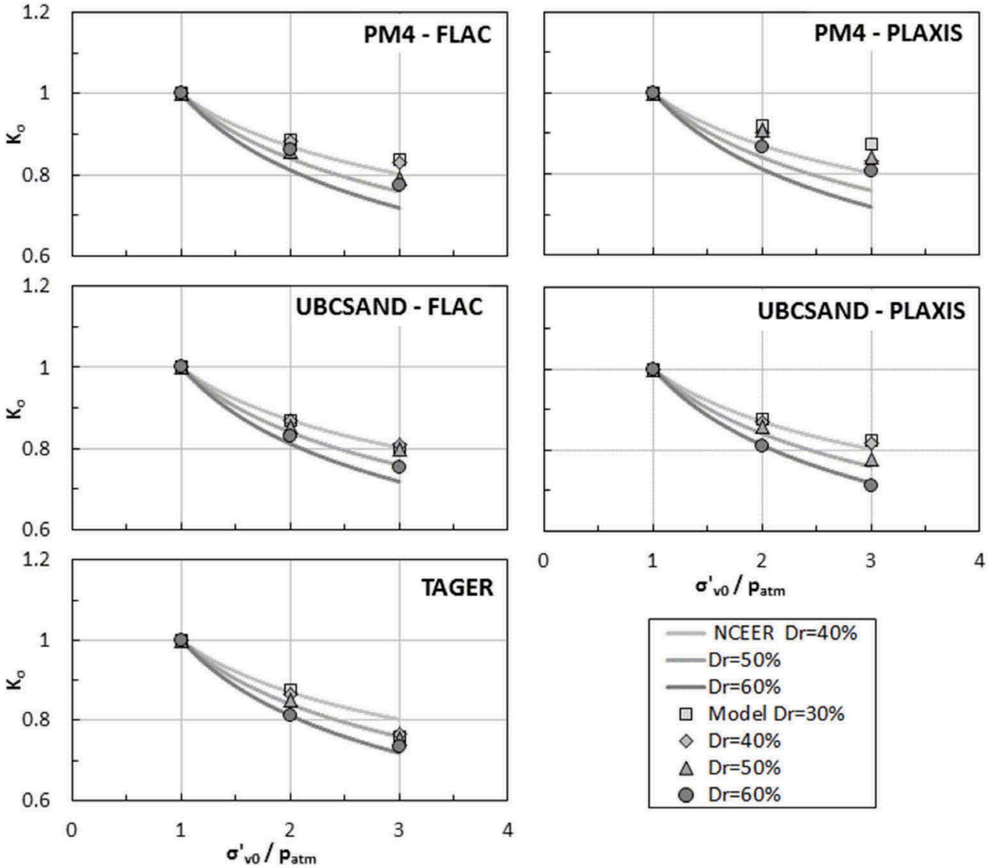

Figure 3. Comparison of K_σ factors derived from the calibrated models for selected values of the relative density (*Dr*= 60%, 50%, 40%, 30%) for liquefaction triggering in 15 uniform loading cycles. Comparison is given with the expressions recommended by the 1996 NCEER/1998 NCEER/NSF workshops. The numerical results were obtained by simulating undrained cyclic DSS tests with σ'_{v0}=100 KPa, σ'_{v0}=200 KPa and σ'_{v0}=300 KPa, and K_0=1

initial relative density of sand Dr_0. Additionally, in the UBCSand model the expression of the PLAXIS parameter fac_{hard} and the FLAC parameter $hfac_1$ as functions of the initial vertical effective stress σ'_{v0} was required to better approximate the effect of the overburden pressure. In a boundary value problem this implies that the soil medium should be discretized into zones with different $hfac_1$ and fac_{hard} values. This limitation was avoided for the PM4Sand model at the cost, however, of a less accurate performance in terms of the K_σ effect. The results regarding the calibration procedure for the TA-GER model has been published in a previous work by Gerolymos et al. (2018). For the sake of brevity, they are not presented herein.

The parameters presented in Table 1 correspond to the best model performance in terms of soil behaviour at the soil element level (Cyclic direct simple shear test). They should be used with caution when analyzing a boundary value problem, as additional adjustments may be required. For more information on this subject, the reader is referred to a companion paper by Gerolymos et al. 2019.

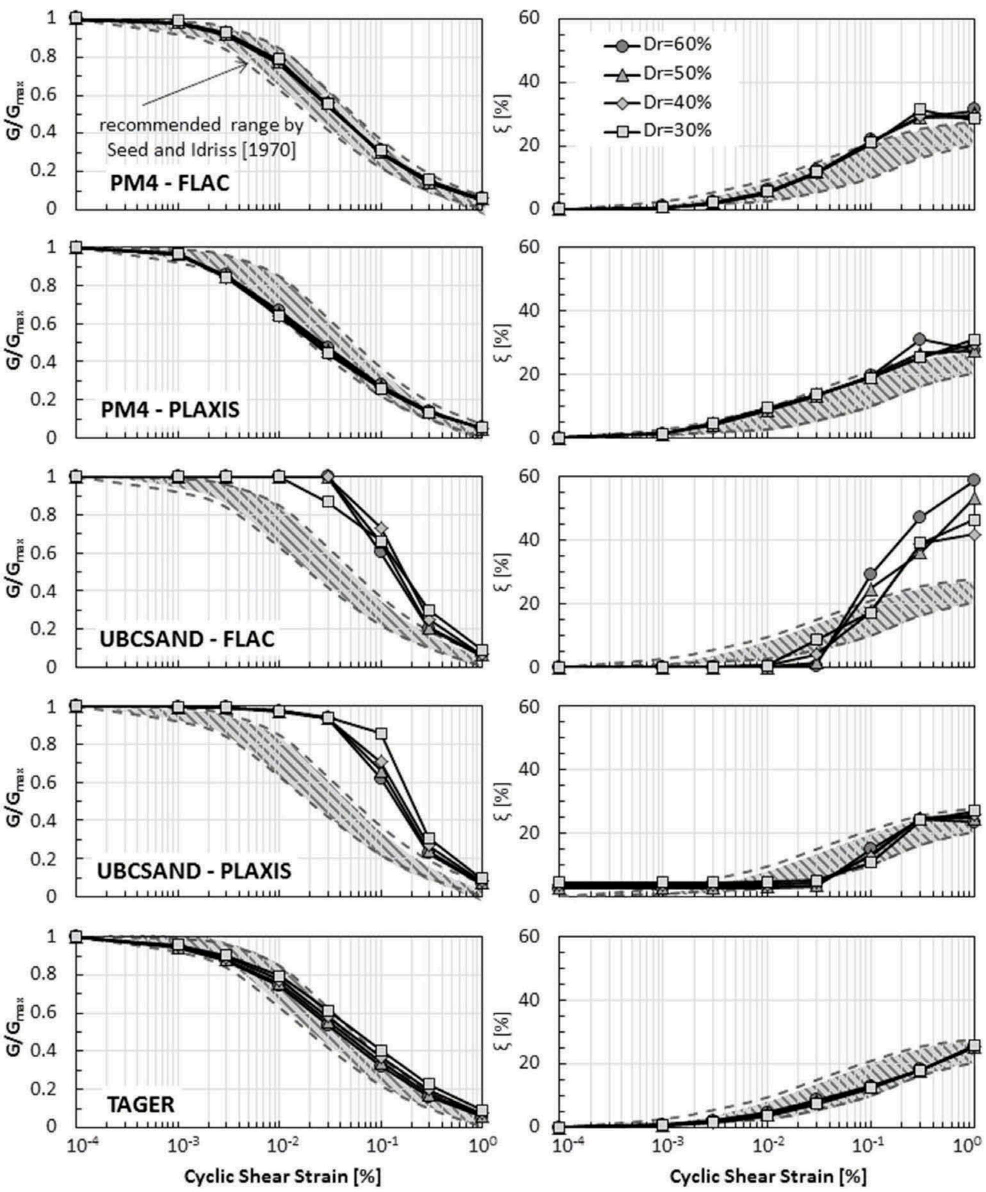

Figure 4. Shear modulus reduction and damping ratio curves for selected values of the relative density (*Dr* = 60%, 50%, 40%, 30%). Comparison is given between results by the calibrated models and suggested by Seed & Idriss (1970) range of sand behaviour. The numerical results were obtained by simulating drained cyclic DSS tests, with σ'_{v0}=100 KPa and K_0=1.

3 CONCLUSIONS

A methodology was presented for calibrating three constitutive models for sand: the PM4Sand and the UBCSand available in the material libraries of the geotechnical engineering softwares PLAXIS 2D and FLAC 2D, and the *p-q* version of the TA-GER model implemented in MATLAB. The calibration was based on matching the response in terms of liquefaction resistance and shear modulus/damping ratio experimental curves. Isotropic consolidation conditions were considered in all cases. It was shown that the calibrated models

Table 1. Recommended model parameters derived from the applied calibration procedure. They are associated with the best performance at the soil element level (Cyclic Direct Simple Shear Test)

UBCSAND - PLAXIS					
Dr	input	K_B^e	K_G^e	σ_t	0
φ_{cv}	36	m_e	0.5	a	$52.93\ Dr^3 - 38.3\ Dr^2 + 5.47\ Dr + 1.915$
φ_p	$0.5626\ e^{3.9°33Dr} + \varphi_{cv}$	n_e	0.5	b	$-5.67\ Dr^3 + 9.9\ Dr^2 - 4.683\ Dr + 0.872$
c	0.5	n_p	0.5	fac_{hard}	$a\ \sigma'_{v0}{}^{-b}$
K_G^e	$1229.2\ Dr + 374.56$	R_f	0.15	$(N_1)_{60}$	$46\ Dr^2$
K_G^p	$372.84\ e^{2.°113\ Dr}$	p_a	100	fac_{post}	0.01
PM4SAND - PLAXIS					
Dr_0	input	e_{min}	0.5	Q	10
G_0	$1450\ Dr_0^{°.3}$	n_b	$0.26\ Dr_0^{-1.°8}$	R	1.5
hp_0	0.75	n_d	0.1	$post_{Shake}$	0
p_a	100	φ_{cv}	33		
e_{max}	0.8	n_u	0.3		
PM4SAND - FLAC					
Dr_0	input	hp_0	0.21	h_0	1
G_0	$1530\ Dr_0^{°.3}$	n_b	$0.3152\ Dr_0^{-1.°8}$		
UBCSAND - FLAC					
Dr_0	input	$hfac_1$	$a\_N\ (\sigma'_{v0}/p_a)^{b\_N}$	n_e	0.5
$(N_1)_{60cs}$	$46\ Dr_0^2$	$hfac_2$	0.06	n_p	0.4
φ_{cv}	33	$hfac_3$	1	R_f	$1.1\ (N_1)_{60cs}^{-.o15} < 0.99$
φ_f	$\varphi_{cv} + (N_1)_{60cs}/10 + \max(0, ((N_1)_{60cs} - 15)/5)$	K_G^e	$434\ (N_1)_{60cs}^{°.333}$	$anisofac$	1
a_N	$-0.01468\ (N_1)_{60cs}^3 + 0.5359\ (N_1)_{60cs}^2 - 5.808\ (N_1)_{60cs} + 28.9$	K_B	$0.75\ K_G^e$	$static$	1
b_N	-0.17	K_G^p	$0.0693\ e^{0.0973\ (N1)_{60cs}}\ K_G^e + 100$		
p_a	100	m_e	0.5		

reproduced the recommended design curves (behaviour at the soil element level) with sufficient engineering accuracy. However, the proposed values should be adopted with caution when simulating a boundary value problem in view of the fact that reproducing the observed or the desired soil behaviour in the mesoscale does not guarantee a successful prediction of the response in the macroscale.

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