TECHNICAL NOTE

Consolidated undrained capacity of shallow foundations subjected to self-weight and horizontal in-service loading

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The effect of subsea infrastructure self-weight and pipeline in-service horizontal loading on the consolidated undrained capacity of deep-water shallow foundations was investigated by means of fully coupled three-dimensional finite-element analyses. The results show that the undrained uniaxial and combined capacity increase under horizontal preloading and over realistic in-field loading time regimes. The gain in undrained capacity is quantified through simple equations incorporating a critical state soil mechanics framework and is defined as a function of magnitude and duration of vertical and horizontal preload, foundation embedment ratio and soil–skirt interface roughness.

KEYWORDS: bearing capacity; clays; consolidation; finite-element modelling; footings/foundations; offshore engineering

INTRODUCTION

Deep-water subsea infrastructure is designed to resist pipeline expansion/contraction exerted during start-up and shut-down operations, in other words, in-service loading. The duration of these operations may last months at a time. Upon application of foundation self-weight or in-service loading, excess pore water pressure is generated around the soil in the vicinity of the subsea foundation. Over time, the soil consolidates and its undrained shear strength increases, leading to an increase in bearing capacity.

The self-weight of deep-water infrastructures is low, commonly mobilising less than 50% of the available unconsolidated undrained capacity. The deep sea architecture is generally constructed well in advance of commencement of hydrocarbon reserves extraction operations, leaving sufficient time for the excess pore water pressure generated under the low self-weight of the structure to dissipate. The effect of self-weight (vertical) preloading and consolidation on the undrained capacity of shallow foundations has been previously investigated (Bransby, 2002; Zdravkovic *et al.*, 2003; Chatterjee *et al.*, 2014; Gourvenec *et al.*, 2014; Feng & Gourvenec, 2015; Fu *et al.*, 2015; Vulpe *et al.*, 2016a, 2016b) but the effect of in-service horizontal loading on the undrained response of shallow foundations has not been previously considered. The pipeline expansion/contraction loading is cyclic and the soil consolidates following each cyclic loading event until a critical state is reached. Nonetheless, the first in-service loading scenario is expected to generate the largest improvement in undrained shear strength. The effect of the initial in-service horizontal load on the undrained shear strength of soil is the object of the current study.

The present study proposes a method for predicting the consolidated undrained capacity of skirted circular foundations as a function of relative magnitude and duration of

both foundation self-weight and in-service preload, foundation embedment ratio and soil–skirt interface roughness.

FINITE-ELEMENT MODEL

A program of over 2000 three-dimensional small-strain finite-element analyses was performed using Abaqus commercial finite-element computer software (Dassault Systèmes, 2012). The meshes of skirted circular foundations with embedment depth, d , to foundation diameter, D , ratios of $d/D = 0, 0.10, 0.25$ were modelled. Figure 1 illustrates a typical finite-element mesh used in the analyses. The free surface of the mesh, unoccupied by the foundation, was prescribed as a drainage boundary; the other mesh boundaries and the foundation were modelled as impermeable.

The ‘wished-in-place’ skirted foundations were represented as rigid bodies with a single reference point (RP) located at skirt tip level along the centreline of the foundation. The skirt internal wall–soil interface was prescribed as fully bonded (accounting for undrained response to uplift); the skirt outer wall–soil interface was prescribed as either fully bonded (rough in shear and no separation allowed) or fully smooth (frictionless).

The modified Cam Clay (MCC) critical state model (Roscoe & Burland, 1968) was used to represent the coupled elasto-plastic pore fluid–stress behaviour of a typical kaolin clay investigated in the current study. The kaolin clay MCC parameters are listed in Table 1 (Stewart, 1992). A surcharge equivalent to 1 m of soil overburden was imposed on the soil mass at the free surface in order to avoid zero shear strength at the mudline. The soil is considered to be one-dimensionally consolidated (K_0) and the in situ effective stresses vary according to the prescribed soil unit weight (Table 1). The relationship between the undrained shear strength of the soil, s_u , which linearly increases with depth, and the in situ effective vertical stress, σ'_v , is given by Potts & Zdravkovic (1999). Details of the adopted MCC model used in the current study are presented by Gourvenec *et al.* (2014) and Vulpe *et al.* (2016a).

Limitations of the MCC model

The MCC critical state model approach models a linear elastic soil response inside the Von Mises circular yield

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Fig. 1. Example of finite-element mesh – cross-sectional view ($d/D = 0.25$)

Table 1. Soil properties used in finite-element analyses

Parameter input for finite-element analyses	Magnitude
Index and engineering parameters	
Saturated bulk unit weight: kN/m^3	17.18
Permeability: m/s	1.3×10^{-10}
Elastic parameters (as a porous elastic material)	
Recompression index (κ)	0.044
Poisson's ratio (ν')	0.25
Tensile limit	0
Clay plasticity parameters	
Virgin compression index (λ)	0.205
Stress ratio at critical state (M_{cs})	0.89
Wet yield surface size*	1
Flow stress ratio†	1
Intercept (e_1 , at $p' = 1$ kPa on critical state line)	2.14

*The wet yield surface size is a parameter defining the size of the yield surface on the 'wet' side of critical state, β . ($\beta = 1$ means that the yield surface is a symmetric ellipse.)

†The flow stress ratio represents the ratio of flow stress in triaxial tension to the flow stress in triaxial compression.

surface, defined in Abaqus by setting the flow stress ratio $K = 1$. Changes in pore pressure result only from changes in mean effective stress. Thus, the constitutive model may not exactly replicate the actual stress path during consolidation in over-consolidated deposits at stress levels less than the pre-consolidation pressure.

The undrained shear strength of clay is dependent on the direction of the major principal stress with respect to the axis of consolidation (Duncan & Seed, 1966). That is, under isotropic and anisotropic consolidation, the soil reaches different ultimate limit states. For clay, the anisotropy of the undrained shear strength is the result of stress-induced anisotropy (Ohta & Nishihara, 1985). This anisotropy occurs when the initial stress state acting during K_0 -consolidation is changed by increasing shear stress up to failure under undrained conditions, along with principal stress rotation.

The undrained shear strength under vertical, horizontal or moment loading is represented by the combination of soil failed in triaxial compression, triaxial extension and plane strain conditions. The effect of induced anisotropy on the undrained shear strength of soil failed in triaxial compression and plane strain conditions is relatively negligible (Mayne, 1985; Toyota *et al.*, 2014). A more pronounced

effect is observed for triaxial extension tests on clay (Mayne, 1985); the difference in the undrained shear strength of soil following isotropic and anisotropic consolidation is found to be dependent on the governing shearing mechanism. The effect of stress-induced anisotropy on the undrained shear strength of the clay cannot be handled by the MCC model. The effect is more pronounced for overconsolidated soils. Nonetheless, Zdravkovic *et al.* (2003) predicted that the undrained shear strength of overconsolidated soils following consolidation is likely to be conservative following finite-element analyses with MCC. The MCC method is best suited to modelling the response of normally consolidated and lightly overconsolidated soils. The current study considered only normally consolidated clay.

METHODOLOGY

Initially, analyses were carried out to determine the unconsolidated undrained vertical (V_{uu}) and horizontal (H_{uu}) capacities for each foundation condition. Then, analyses were performed to determine the uniaxial and combined capacity for each foundation as a function of relative magnitude and duration of both vertical and horizontal preload. A vertical preload (V_p) was imposed, in undrained conditions, as a fraction of the V_{uu} relative to the foundation system, taking values of $V_p/V_{uu} = [0.1, 0.7]$ at intervals of 0.2 followed by full primary consolidation. Consolidation was prescribed by allowing the excess pore water pressure generated by the applied preload to dissipate through the imposed drainage boundary. The foundation was then subjected to horizontal preloading (H_p), applied at the RP, as a fraction of the relative H_{uu} . The horizontal preload takes values of $H_p/H_{uu} = [0.3, 0.9]$ at intervals of 0.2. Periods of consolidation corresponding to partial (20 and 50% of the full primary consolidation) and full primary consolidation were considered. Last, the soil was brought to failure in undrained conditions by imposing displacement-controlled probe tests at the RP level. A list of notations for unconsolidated undrained and consolidated undrained capacities is summarised in Table 2.

Critical state soil mechanics framework

Gourvenec *et al.* (2014) proposed a simple theoretical framework based on critical state soil mechanics (CSSM) to

Table 2. Definition of notation

Direction of loading	Vertical	Horizontal	Rotational
Load	V_p (preload)	H_p (preload)	M
Uniaxial (unconsolidated) undrained capacity	V_{uu}	H_{uu}	M_{uu}
Consolidated undrained pure uniaxial capacity following vertical preloading	V_{cu}^V	H_{cu}^V	M_{cu}^V
Consolidated undrained pure uniaxial capacity following vertical and horizontal preloading	$V_{cu}^{V,H}$	$H_{cu}^{V,H}$	$M_{cu}^{V,H}$

quantify the increase in consolidated undrained uniaxial vertical capacity of surface foundations following vertical preloading and consolidation (equation (1)). The framework was successfully applied to predict gains in consolidated undrained vertical, horizontal and moment uniaxial capacity of skirted circular foundations (Vulpe *et al.*, 2016a, 2016b)

$$\frac{H_{cu}^V}{H_{uu}}, \frac{M_{cu}^V}{M_{uu}} = 1 + f_{\alpha} f_{su} R \alpha \left(\frac{V_p}{V_{uu}} \right)^{\beta} N_{cV} \quad (1)$$

where the scaling factor $f_{\alpha} f_{su}$, which is independent of foundation size, MCC soil properties and applied soil overburden stress (Vulpe *et al.*, 2016a), is summarised in Table 3. Scaling factor $f_{\alpha} f_{su}$ accounts for the non-uniform distribution of the stress in the affected zone of soil and scales the gain in strength of the ‘operative’ soil element to that mobilised during subsequent failure. Coefficients α and β account for the non-linear effect of the embedment on the gain in capacity. Coefficients α and β are defined for both rough and smooth skirted circular foundations as a function of embedment ratio for each uniaxial capacity through polynomial functions.

$$\alpha = 1 + \alpha_1 \left(\frac{d}{D} \right) + \alpha_2 \left(\frac{d}{D} \right)^2 \quad (2)$$

$$\beta = \beta_1 \left(\frac{d}{D} \right) + \beta_2 \left(\frac{d}{D} \right)^2$$

Values of coefficients α and β are given in Table 4. R is the normally consolidated strength ratio of the soil and N_{cV} is the vertical bearing capacity factor for the given foundation geometry and normally consolidated soil conditions.

Table 3. Stress and strength factor $f_{\alpha} f_{su}$ for critical state interpretation, equation (1)

Loading direction	$f_{\alpha} f_{su}$
V	0.43
H	0.88
M	0.57

Table 4. Coefficients α and β for critical state interpretation, equation (2)

Soil–skirt interface roughness	Loading direction	α_1	α_2	β_1	β_2
Rough	V	−1.32	1.1	1.34	−0.44
	H	−2.77	2.99	0.73	−0.38
	M	0.4	−1.79	1.42	−1.18
Smooth	V	−0.71	0.53	1.56	−2.23
	H	−3.11	3.75	0.12	−0.04
	M	1.17	−3.12	0.43	0.21

RESULTS

Excess pore water pressure is generated in the soil around the foundation upon application of self-weight preloading and then allowed to fully dissipate through the drainage boundary. Excess pore water pressure is again generated during in-service preloading and dissipation is prescribed through the same drainage boundary. The pore pressure dissipation time history, where Δu is the excess pore pressure and Δu_i is the initial excess pore pressure during H_p/H_{uu} preloading, measured at the centre of the foundation base plate at mudline level, is exemplified in Fig. 2 for both rough and smooth skirted foundations with $d/D = 0.1$. The time is expressed through the non-dimensional time factor T as

$$T = \frac{c_{v0} t}{D^2} \quad (3)$$

where c_{v0} is the initial coefficient of consolidation at skirt tip level and t is the actual time passed following application of horizontal preload. All excess pore water pressure–time histories display the characteristic Mandel–Cryer effect (Mandel, 1950; Cryer, 1963).

Effect of foundation embedment and soil–skirt interface roughness on the gain in capacity

The effect of both foundation embedment ratio and soil–skirt interface roughness on the relative gain in uniaxial capacity following horizontal preloading is summarised in Fig. 3 for a particular case of $H_p/H_{uu} = 0.3$. The relative gain in uniaxial horizontal capacity decreases with increasing embedment ratio for both rough and smooth foundations, with the smooth skirted foundations consistently showing lower relative gains compared to the rough foundation

Fig. 2. Normalised excess pore time histories for rough and smooth skirted circular foundations with $d/D = 0.1$ during discrete levels of in-service (H_p/H_{uu}) preloading

Fig. 3. Effect of embedment and soil-skirt interface roughness on the relative gain in uniaxial horizontal capacity after vertical and horizontal preloading and full primary consolidation

counterpart. This trend is consistent with that observed for skirted circular foundations preloaded under self-weight conditions only (Vulpe *et al.*, 2016b).

Full primary consolidation following horizontal preloading

The relative gain in uniaxial capacity following vertical and horizontal preloading and full primary consolidation is compared to the relative gain in uniaxial capacity following vertical preloading only and full primary consolidation for surface foundations (equation (1)) in Fig. 4. The effect of relative in-service loading on the relative gain in capacity is manifested through an additional relative increase in both horizontal and moment capacity for all preload levels. As a result of horizontal preloading and consolidation, the soil densifies laterally around the foundation skirts, close to the mudline. The gain in capacity is represented by the intersection between the zone of increase in soil shear strength and the kinematic mechanism at failure. For both ultimate horizontal and moment loading, the shearing mechanism cuts through the relatively shallow soil layer, thus intersecting the zone of increase in shear strength.

In order to evaluate the relative increase in uniaxial horizontal and moment capacity following vertical and horizontal preloading, the results from Fig. 4 are reinterpreted by adopting a link between gains in capacity following sole vertical preloading and combined vertical and horizontal preloading, respectively. Thus, Fig. 5 illustrates the evolution of the proportion of maximum potential gain in undrained horizontal, G_H , and moment, G_M , capacity following vertical and horizontal preloading and full primary consolidation as a function of the consolidated undrained uniaxial capacity following vertical preloading and full primary consolidation

$$G_H = \frac{H_{cu}^{V,H} - H_{uu}}{H_{cu}^V - H_{uu}} = a_H \left(\frac{V_p}{V_{uu}} \right)^{b_H} \quad (4)$$

$$G_M = \frac{M_{cu}^{V,H} - M_{uu}}{M_{cu}^V - M_{uu}} = a_M \left(\frac{V_p}{V_{uu}} \right)^{b_M}$$

where H_{cu}^V and M_{cu}^V are determined from the CSSM framework (equation (1)) and H_{uu} and M_{uu} from numerical solutions (Vulpe, 2015). Non-dimensional fitting coefficients a_H and a_M are given in Table 5 as a function of d/D and soil-skirt interface roughness. Fitting coefficient b

Fig. 4. Normalised gain in undrained uniaxial capacity after vertical and horizontal preloading and full primary consolidation for surface foundation with rough interface: (a) horizontal capacity; (b) moment capacity

takes the form

$$b_{H,rough} = - \left[\left(0.92 \frac{d}{D} + 0.51 \right) \frac{H_p}{H_{uu}} + 0.16 \frac{d}{D} + 0.08 \right]$$

$$b_{H,smooth} = - \left[\left(0.72 \frac{d}{D} + 0.51 \right) \frac{H_p}{H_{uu}} + 0.04 \frac{d}{D} + 0.08 \right]$$

$$b_{M,rough} = - \left[\left(0.91 \frac{d}{D} + 0.31 \right) \frac{H_p}{H_{uu}} + 0.77 \frac{d}{D} + 0.25 \right]$$

$$b_{M,smooth} = - \left\{ \left[-15.33 \left(\frac{d}{D} \right)^2 + 4.03 \left(\frac{d}{D} \right) + 0.33 \right] \times \frac{H_p}{H_{uu}} + 16.27 \left(\frac{d}{D} \right)^2 - 4.03 \left(\frac{d}{D} \right) + 0.26 \right\} \quad (5)$$

and incorporates the effect of applied relative horizontal preload, foundation embedment ratio and soil-skirt interface roughness. Fig. 5 and Fig. 6 illustrate good agreement between the finite-element analyses and equation (4) irrespective of foundation embedment ratio and soil-skirt interface roughness. They also show that the potential for increase in capacity decreases with increasing relative vertical preload. The soil densification increases as a result of increasing vertical preload and consolidation, resulting in less opportunity for further reduction in void ratio in the next consolidation step.

Fig. 5. Proportion of the potential maximum gain in uniaxial horizontal and moment capacity following vertical and horizontal preloading and full primary consolidation (equation (4)), G_H and G_M , for surface foundations ($d/D=0$): (a) horizontal capacity, G_H ; (b) moment capacity, G_M

Table 5. Fitting coefficient a_H and a_M for determining the gain in capacity following full primary consolidation for rough and smooth skirted circular foundations

d/D	Rough interface			Smooth interface	
	0	0.1	0.25	0.1	0.25
a_H	0.9	0.8	0.8	0.77	0.77
a_M	0.9	0.75	0.75	0.8	0.8

Partial consolidation following horizontal preloading

Finite-element analyses incorporating partial consolidation following horizontal preloading were carried out in order to simulate realistic in-field in-service loading conditions. Fig. 7 illustrates the progression of the potential gain in undrained horizontal and moment capacity as a function of consolidation time for surface circular foundations. A relationship between the consolidation time, represented by time factor T , and the proportion of maximum potential gain following vertical and horizontal preloading and full primary consolidation is proposed

$$G_{H,p} = \frac{H_{cu,p}^{V,H} - H_{cu}^V}{H_{cu}^{V,H} - H_{cu}^V} = \frac{1}{1 + [(V_p/V_{uu})/0.7][m_H(T/T_{50})^n]}$$

$$G_{M,p} = \frac{M_{cu,p}^{V,H} - M_{cu}^V}{M_{cu}^{V,H} - M_{cu}^V} = \frac{1}{1 + [(V_p/V_{uu})/0.7][m_M(T/T_{50})^n]} \quad (6)$$

Fig. 6. Proportion of the potential maximum gain in uniaxial moment capacity following vertical and horizontal preloading and full primary consolidation (equation (4)), G_M for rough and smooth skirted foundations with $d/D=0.25$: (a) rough soil-skirt interface; (b) smooth soil-skirt interface

from which predictions of relative gains in undrained uniaxial capacity following partial consolidation, $H_{cu,p}^{V,H}$ and $M_{cu,p}^{V,H}$, may be quantified. The non-dimensional time factor, T_{50} , representing the time required for 50% of the full primary consolidation to occur under $V_p/V_{uu}=0.7$, is given in Table 6 for each embedment ratio and soil-skirt interface roughness. Fitting coefficients m_H and m_M are given in Table 7. Fitting coefficient n is -1.20 irrespective of foundation type. Fig. 7 and Fig. 8 indicate good agreement between the finite-element analysis results and the relative gains in undrained uniaxial capacity derived from equation (6), irrespective of d/D and soil-skirt interface roughness.

Consolidated undrained VHM capacity

Failure envelopes in normalised horizontal and moment load space (HM) for rough skirted foundations with $d/D=0.25$ under unconsolidated undrained (UU) (Vulpe, 2015) and consolidated undrained consolidations (Vulpe *et al.*, 2016b and current study) are compared in Fig. 9. Notations T_{20} and T_{50} represent the non-dimensional time required for 20 and 50% of the full primary consolidation to occur. Full primary consolidation is denoted by T_{99} . The positive effect of in-service preloading and consolidation is evident even in early consolidation stages (T_{20}). A simple algebraic equation accounting for the effect of vertical and horizontal preloading on the HM failure envelope could not be formulated. This is as a result of the convoluted path

Fig. 7. Proportion of the potential maximum gain in uniaxial capacity following vertical and horizontal preloading and partial consolidation (equation (6)) for surface foundations: (a) horizontal capacity, $G_{H,p}$; (b) moment capacity, $G_{M,p}$

Table 6. Non-dimensional time factor for 50% partial consolidation, T_{50} , for $V_p/V_{uu} = 0.7$

d/D	Rough interface	Smooth interface
0	0.18	—
0.1	0.22	0.18
0.25	0.26	0.22

Table 7. Fitting coefficients m_H and m_M for determining the gain in capacity following partial consolidation for rough and smooth skirted circular foundations

	Rough interface			Smooth interface	
d/D	0	0.1	0.25	0.1	0.25
m_H	0.2	0.74	0.4	0.95	1.6
m_M	0.2	0.5	0.25	0.65	1.7

‘a point’ from the UU failure envelope travels during both vertical preloading and consolidation followed by horizontal preloading and consolidation towards the consolidated undrained (CU) failure envelope.

Fig. 8. Proportion of the potential maximum gain in uniaxial horizontal capacity following vertical and horizontal preloading and partial consolidation (equation (6)), $G_{H,p}$, for rough and smooth skirted foundations with $d/D = 0.25$: (a) rough soil–skirt interface; (b) smooth soil–skirt interface

Fig. 9. Normalised HM failure envelopes of rough skirted circular foundations with $d/D = 0.25$ for $V_p/V_{uu} = 0.3$ under various dissipation scenarios

CONCLUDING REMARKS

Three-dimensional small-strain finite-element analyses have quantified the effect of relative magnitude and duration of self-weight and in-service horizontal preloading, embedment ratio and soil–skirt interface roughness on the

multi-directional undrained capacity of skirted circular foundations. The results show that the undrained uniaxial and combined capacity increase under horizontal preloading and over realistic in-field loading time regimes. Approximating expressions incorporating a CSSM framework for prediction of consolidated undrained uniaxial capacity of smooth and rough skirted circular foundations have been proposed.

This study has highlighted the potential conservatism in foundation design by not accounting for the enhanced soil undrained shear strength following foundation self-weight and in-service loading and consolidation in determining ultimate limit states.

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NOTATION

$a_H, a_M, b_{H,rough}, b_{H,smooth}, b_{M,rough}, b_{M,smooth}$	non-dimensional fitting coefficients
c_{v0}	in situ coefficient of consolidation
D	foundation diameter
d	skirt length
f_{su}	scaling factor
G_H, G_M	proportional maximum gain in bearing capacity following vertical and horizontal preloading and full primary consolidation
$G_{H,p}, G_{M,p}$	proportional maximum gain in bearing capacity following vertical and horizontal preloading and partial consolidation
H_p	horizontal preload
H_{uu}	unconsolidated undrained horizontal capacity
K_0	in situ earth pressure coefficient of normally consolidated deposit
m_H, m_M, n	fitting coefficients
$N_{cV}, N_{cH}, N_{cM}, N_{cV}^{smooth}, N_{cH}^{smooth}, N_{cM}^{smooth}$	non-dimensional bearing capacity factors
R	normally consolidated undrained strength ratio
s_u	undrained shear strength
T	non-dimensional time factor
T_{50}	time for 50% consolidation to occur
t	time
V_p	vertical preload
V_{uu}	unconsolidated undrained vertical capacity
$V_{cu}^V, H_{cu}^V, M_{cu}^V$	consolidated undrained pure uniaxial capacity following vertical preloading
$V_{cu}^{V,H}, V_{cu}^{V,H}, V_{cu}^{V,H}$	consolidated undrained pure uniaxial capacity following vertical and horizontal preloading
α, β	fitting coefficients
Δu	excess pore water pressure
Δu_i	initial excess pore water pressure
σ_v	in situ vertical effective stress

REFERENCES

- Bransby, M. F. (2002). The undrained inclined load capacity of shallow foundations after consolidation under vertical loads. *Proceedings of the 8th international symposium on numerical models in geomechanics (NUMOG)*, Rome, Italy, pp. 431–437.
- Chatterjee, S., Gourvenec, S. & White, D. J. (2014). Assessment of the consolidated breakout response of partially embedded subsea pipelines. *Géotechnique* **64**, No. 5, 391–399, <http://dx.doi.org/10.1680/geot.13.P215>.
- Cryer, C. W. (1963). A comparison of three dimensional consolidation theories of Biot and Terzaghi. *Q. J. Mech. Appl. Math.* **16**, No. 4, 401–412.
- Dassault Systèmes (2012). *Abaqus analysis user's manual*. Providence, RI, USA: Simulia Corp.
- Duncan, M. & Seed, H. B. (1966). Anisotropy and stress orientation in clay. *J. Soil Mech. Found. Div.* **92**, No. 5, 21–50.
- Feng, X. & Gourvenec, S. (2015). Consolidated undrained load-carrying capacity of subsea mudmats under combined loading in six degrees-of-freedom. *Géotechnique* **65**, No. 7, 563–575, <http://dx.doi.org/10.1680/geot.14.P090>.
- Fu, D., Gaudin, C., Tian, C., Bienen, B. & Cassidy, M. J. (2015). Effects of preloading with consolidation on undrained bearing capacity of skirted circular footings. *Géotechnique* **65**, No. 3, 231–246, <http://dx.doi.org/10.1680/geot.14.P2120>.
- Gourvenec, S. M., Vulpe, C. & Murthy, T. G. (2014). A method for predicting the consolidated undrained bearing capacity of shallow foundations. *Géotechnique* **64**, No. 3, 215–225, <http://dx.doi.org/10.1680/geot.13.P101>.
- Mandel, J. (1950). Etude mathématique de la consolidation des sols. *Actes du colloque international de mécanique*, Poitiers, France, vol. 4, pp. 9–19 (in French).
- Mayne, P. W. (1985). Stress anisotropy effects on clay strength. *J. Geotech. Engng* **111**, No. 3, 356–366.
- Ohta, H. & Nishihara, A. (1985). Anisotropy of undrained shear strength of clays under axis-symmetric loading conditions. *Soils Found.* **25**, No. 2, 78–86.
- Potts, D. M. & Zdravkovic, L. (1999). *Finite element analysis in geotechnical engineering – theory*. London, UK: Thomas Telford.
- Roscoe, K. H. & Burland, J. B. (1968). On the generalised stress–strain behavior of ‘wet clay’. In *Engineering plasticity* (eds J. Heyman and F. A. Leckie), pp. 535–609. Cambridge, UK: Cambridge University Press.
- Stewart, D. P. (1992). *Lateral loading of pile bridge abutments due to embankment construction*. PhD thesis, University of Western Australia, Perth, WA, Australia.
- Toyota, H., Susami, A. & Takada, S. (2014). Anisotropy of undrained shear strength induced by K_0 consolidation and swelling in cohesive soils. *J. Geotech. Engng* **14**, No. 4, 0401409.
- Vulpe, C. (2015). Design method for the undrained capacity of skirted circular foundations under combined loading: effect of deformable soil plug. *Géotechnique* **65**, No. 8, 669–683, <http://dx.doi.org/10.1680/geot.14.P200>.
- Vulpe, C., Gourvenec, S., Leman, B. & Fung, K. N. (2016a). A failure envelope approach for consolidated undrained capacity of shallow foundations. *J. Geotech. Geoenviron. Engng* **142**, No. 8, 0401603, [http://dx.doi.org/10.1061/\(ASCE\)GT.1943-5606.0001498](http://dx.doi.org/10.1061/(ASCE)GT.1943-5606.0001498).
- Vulpe, C., Gourvenec, S. & Cornelius, A. F. (2016b). Effect of embedment on consolidated undrained capacity of skirted circular foundations in soft clay under planar loading. *Can. Geotech. J.*, in press.
- Zdravkovic, L., Potts, D. M. & Jackson, C. (2003). Numerical study of the effect of preloading on undrained bearing capacity. *Int. J. Geomech., ASCE* **3**, No. 1, 1–10.