Effect of prior loading cycles on vertical bearing capacity of clay

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The strength of a foundation on soft clay rises over time due to consolidation under the applied load. This effect can be advantageous in design. Previous studies have identified relationships between the level of monotonic vertical preload, as a proportion of the initial bearing capacity, and the subsequent consolidated bearing capacity. This study explores the influence of cycles of preloading by way of centrifuge model tests, and shows that repeated preloading leads to a further gain in bearing capacity. The gains are more significant if the preloading involves two-way cycles. For the cases explored in this study, the cyclic effect typically adds 50% to the gain in bearing capacity caused by the same preload applied monotonically.

Notation

A_p	projected area of penetrometer in a plane normal to shaft	v^*	normalised velocity
A_{plate}	circular plate area	z	depth
A_s	cross-sectional area of cylindrical shaft	α	net area ratio for ball penetrometer
A_{shaft}	area of cylindrical shaft	γ'	effective unit self-weight of soil
c_v	vertical coefficient of consolidation	Δq_{net}^c	increase in penetration resistance in compression
D	circular plate diameter	Δq_{net}^t	increase in penetration resistance in tension
$F_{buoyancy}$	soil buoyancy	Δq_{pl}	plastic stress component
f_{su}	strength factor	Δs_u	increase in undrained shear strength after consolidation
f_σ	stress factor	Δu	excess pore water pressure
g	acceleration due to gravity	σ_{v0}	in situ total vertical stress
h_{shaft}	length of cylindrical shaft penetrated into soil	σ'_{v0}	in situ effective vertical stress
k	shear strength gradient		
N_{ball}	resistance factor for ball penetrometer		
N_{cV}	vertical bearing capacity factor		
q_{ball}	net ball penetration resistance		
q_m	measured ball penetration resistance		
q_{net}^c	penetration resistance in compression		
q_{net}^t	penetration resistance in tension		
q_{net}^u	undrained net penetration resistance		
R	normally consolidated shear strength ratio		
s_u	undrained shear strength of clay		
s_{um}	undrained shear strength of clay at mudline		
T	dimensionless time factor		
T_{50}	dimensionless time factor for 50% consolidation to occur		
t	time		
t_{plate}	circular plate thickness		
t_{shaft}	shaft diameter		
t_{50}	time for 50% consolidation to occur		
u_0	hydrostatic pore pressure		
v	penetration rate		

1. Introduction

The undrained bearing capacity of a shallow foundation on soft clay varies with time due to consolidation caused by the loading condition. These changes in bearing capacity can be advantageous. In the offshore environment, the dead weight ‘preloading’ of the structure is sometimes relied on to enhance the strength and bearing capacity of the foundation (e.g. Watson and Humpheson, 2007). This strength enhancement is useful if the structure is enlarged or the design environmental load revised during the operating life. Onshore, it has become increasingly common to reuse existing foundations when urban sites are redeveloped (Butcher *et al.*, 2006; Chapman *et al.*, 2007). In these conditions, it may be beneficial to account for consolidation-induced enhancement of the foundation bearing capacity.

This study is concerned with soft, normally consolidated (NC) clay prevalent in deep water offshore, and sometimes

encountered onshore in estuarine environments. When a foundation is installed on soft clay and dead weight or preload is applied, excess pore pressure is generated in response to the increased total stress and potentially also from the shearing deformation. Previous research studies have identified the increase in undrained vertical bearing capacity from sustained monotonic preloading (Bransby, 2002; Gourvenec *et al.*, 2014; Zdravkovic *et al.*, 2003).

In practice, the load applied to a foundation during the working life may include a significant cyclic component. Examples include offshore structures that are exposed to hydrodynamic loading from wind and waves and changing self-weight from oil storage, or the foundations for tanks, stockpiles and subsea equipment that experience changing loads depending on the operating conditions. As these operating conditions change during the life of the facility, there may be opportunities to utilise the enhanced bearing capacity caused by consolidation.

Previous research on cyclic loading of coarse-grained soils (Andersen, 2009; Bjerrum, 1973) and fine-grained soils (Andersen, 2004; Hyde and Ward, 1986; Koutsoftas, 1978; Togrol and Güler, 1984; van Eekelen and Potts, 1978) has investigated the degradation of strength due to excess pore pressure generation, but the subsequent consolidation hardening effect has received only limited attention (France and Sangrey, 1977; Hodder *et al.*, 2013; White and Hodder, 2010).

The purpose of the present study is to extend previous work on monotonic preloading and to explore how cyclic preloading can enhance the bearing capacity of foundations on soft clay.

2. Methodology

2.1 Apparatus

The experimental programme was carried out in the beam centrifuge at the University of Western Australia (UWA). The physical characteristics of the equipment are described by Randolph *et al.* (1991). The swinging platform radius is 1.8 m and the strongbox measures 0.690 m by 0.390 m in plan with a depth of 0.325 m. The tests described in the current paper were carried out at an acceleration of 25g, on a model seabed consisting of UWA kaolin clay.

2.2 Model foundation and instrumentation

One shallow, circular foundation model was fabricated from aluminium for the tests; the dimensions of the model and of the resulting prototype are summarised in Table 1. The model consists of a 0.04 m diameter circular plate connected to a cylindrical shaft with diameter $t_{\text{shaft}} = 0.01$ m (Figures 1 and 2). Since the tests were carried out at 25g, the prototype circular foundation has a diameter of 1 m. The prototype dimensions analysed in this study are not intended to represent

	Model: m	Prototype: m
Plate diameter	0.04	1.0
Plate thickness	0.004	0.1
Shaft diameter	0.01	0.25

Note: Prototype dimensions for 25g.

Table 1. Foundation geometry

a specific offshore application, although foundation elements of 1 m in typical dimension are often used to support subsea structures.

The model is equipped with one pore pressure transducer (PPT) encased in the shaft and set flush with the underside of the circular plate. The purpose of the PPT is to monitor the excess pore pressure development during installation and subsequent dissipation underneath the circular plate.

Load- and displacement-controlled simulations were carried out by vertically attaching a 3 kN capacity load cell and a linear variable displacement transducer (LVDT) to the model through a rigid loading arm, which is connected to a one-dimensional actuator. Load- and displacement-controlled operations were performed using the package actuator control system (PACS), an in-house software developed at UWA (De

Figure 1. Model circular shallow foundation

Figure 2. Circular plate geometry

Catania *et al.*, 2010), and data acquisition was performed using the Digidaq software (Gaudin *et al.*, 2009).

2.3 Sample preparation

All the tests documented in the current study were conducted on kaolin clay, normally consolidated from slurry in the centrifuge. The mechanical properties of the kaolin used in the tests have been thoroughly investigated by previous researchers (Chen, 2005; Gourvenec *et al.*, 2009; Stewart, 1992) and are summarised in Table 2.

The sample was prepared by mixing dry kaolin powder with water to form a slurry with a moisture content double the liquid limit (LL). While mixing the slurry in a barrel mixer, vacuum was applied for de-aeration. Afterwards, the slurry was transferred to the strongbox overlying a 15 mm thick sand drainage layer. A head of surface water was maintained over the sample at all times. The sample was spun at 25g for 4 days (model scale) to achieve primary consolidation. Lastly, < 5 mm of the sample was scraped off the top in order to create a perfectly flat testing surface, which had the effect of creating a very slight strength intercept at the mudline.

Property	Symbol	Magnitude
Liquid limit: %	LL	61
Plastic limit: %	PL	27
Specific gravity	G_s	2.6
Angle of internal friction: degrees	ϕ	23
Virgin compression index	λ	0.205
Recompression index	κ	0.044
Coefficient of consolidation	c_v	2.6

Table 2. Mechanical properties of kaolin clay (Stewart, 1992)

All of the PPT results are presented as excess pore pressures, so the results are independent of the height of the head of surface water over the soil sample.

The shear strength profile of the soil sample was determined by means of piezoball testing (Chung and Randolph, 2004; Low *et al.*, 2007). The tests were carried out at a rate of 0.1 mm/s, ensuring undrained conditions (Randolph and Hope, 2004). The initial penetration resistance measured by the piezoball was corrected for unequal pore pressure and overburden pressure following the expression defined by Chung and Randolph (2004)

$$1. \quad q_{\text{ball}} = q_m - [\sigma_{v0} - u_0(1 - \alpha)] \frac{A_s}{A_p}$$

where q_m is the measured penetration resistance, σ_{v0} is the in situ total overburden stress, u_0 is the hydrostatic pressure, $\alpha = 0.85$ and $A_s/A_p = 0.1$ (Low *et al.*, 2010). The undrained shear strength, s_u , is back-calculated from the net penetration resistance, q_{ball}

$$2. \quad s_u = \frac{q_{\text{ball}}}{N_{\text{ball}}}$$

where $N_{\text{ball}} = 11.3$, which is a typical N -factor for piezoball penetration tests (Colreavy *et al.*, 2010; Low *et al.*, 2010) and in normally consolidated soil can be adopted as a constant value over the entire sample depth (Gourvenec *et al.*, 2009; Low *et al.*, 2010). The resulting shear strength profile is shown in Figure 3 and given by the following expression

Figure 3. Calculated undrained shear strength from cyclic piezoball test

$$3. \quad s_u = s_{um} + kz$$

where s_u represents the undrained shear strength at depth z , s_{um} is the undrained shear strength at the mudline and k is the strength increase per unit depth. The profile is fitted with Equation 3 by $k = 0.70$ kPa/m and $s_{um} = 0.15$ kPa, essentially modelling an NC soil, typical of offshore conditions.

A dissipation stage was included in the piezoball test in order to determine the coefficient of consolidation, c_v . Based on the work by Low *et al.* (2007), a representative dimensionless time factor is chosen, namely $T_{50} = 0.3$, which is used to determine the coefficient of consolidation for the current piezoball test. It results from $c_v = D^2 T_{50} / t_{50}$ that $c_v = 3.3$ m²/year, which is well within the expected values for UWA kaolin clay based on previous Rowe cell testing (Chen, 2005; Gourvenec *et al.*, 2009).

2.4 Experimental programme

The experimental programme consisted of six tests summarised in Table 3 and shown schematically in Figure 4. A base case test, PF1, was carried out in order to corroborate the initial undrained shear strength, s_u , and to provide a penetration resistance profile, q_{net}^u , against which all the other test results are compared. A programme of tests involving monotonic and cyclic preloading was devised. The variables investigated included the type of cyclic vertical preloading (periodic or continuous) and the range (one-way or two-way). After the specified preloading process, the foundation was failed in compression for all tests (PF2, PF3, PF4 and PF4e) except for test PF2e (Figure 4(e)) which was failed in tension.

All penetration and extraction stages were carried out at a rate of $v = 0.1$ mm/s to ensure undrained conditions within those movements. Taking into account that $c_v = 3.3$ m²/year, the resulting normalised velocity $v^* = vD/c_v$ is equal to 38, confirming undrained conditions (Finnie and Randolph, 1994) (where

$D = 0.04$ m represents the drainage path equal to the model plate diameter).

PF1 was performed solely under displacement control at a rate of 0.1 mm/s to determine the full undrained penetration response (Figure 4(a)). The rest of the tests were performed under displacement control to depth $z = 0.5$ m, after which the actuator was switched to load control. A specified load sequence was then imposed, during which consolidation of the surrounding soil occurred.

During test PF2, the vertical load was maintained constant and the consolidation step was considered finished when the excess pore pressure Δu reduced to < 0.6 kPa which was after 10 000 s (Figure 4(c)). This represents $> 95\%$ dissipation of the initial excess pore pressure, so a longer duration of dissipation would have negligible effect. For consistency, this period was then adopted for the consolidation step of all subsequent tests. Following the consolidation step of PF2, the actuator was switched back to displacement control and the foundation was failed under a controlled penetration rate of 0.1 mm/s. Test PF3 followed a similar process but with a regime of five periodic preload cycles with near-complete primary consolidation ($\Delta u < 0.6$ kPa) reached after each load and unload stage of the cycles (Figure 4(b)). Tests PF4 (Figure 4(d)) and PF4e (Figure 4(f)) underwent continuous one-way and two-way cyclic preloading during the consolidation stage, until a steady embedment was reached; the foundation was then failed in compression. During PF4 and PF4e all cycles were performed with a period of 10 s at model scale (Table 4). These cycles are sufficiently rapid to be effectively undrained within each cycle, as would be the case in prototype conditions, but not so fast that the behaviour is also influenced by inertial effects.

The time history for all tests are schematically represented in Figure 5, where q_{net}^c and q_{net}^t represent the gain in net penetration resistance in compression and tension, respectively.

Test name	Pre-loading regime			Failure mode
	Pre-load type	Cycle		
		Type	Range	
PF1	None	–	–	Compression
PF2	Monotonic	–	–	Compression
PF3	Cyclic	Periodic	One-way	Compression
PF4	Cyclic	Continuous	One-way	Compression
PF2e	Monotonic	–	–	Tension
PF4e	Cyclic	Continuous	Two-way	Compression

Table 3. Experimental programme

Figure 4. Schematic diagram of centrifuge testing programme; net resistance plotted against depth: (a) PF1; (b) PF3; (c) PF2; (d) PF4; (e) PF2e; (f) PF4e

3. Foundation results: monotonic cases

3.1 No preload

The undrained net penetration resistance, q_{net}^u , is given by

$$4. \quad q_{net}^u = q_m - \frac{F_{buoyancy}}{A_{plate}}$$

where q_m is the measured penetration resistance divided by the plate area, A_{plate} , and $F_{buoyancy}$ is the soil buoyancy term. This

	Model	Prototype
Period: s	10	6250
Frequency: Hz	0.1	0.00016

Table 4. Centrifuge testing period and frequency

is represented by the submerged weight of the kaolin soil displaced by the circular plate and the cylindrical shaft. The correction for soil buoyancy is given by

$$5. \quad F_{buoyancy} = \begin{cases} \gamma' z A_{plate}, & \text{if } z < t_{plate} \\ \gamma' [t_{plate} A_{plate} + (z - t_{plate}) A_{shaft}], & \text{if } z \geq t_{plate} \end{cases}$$

where A_{plate} and A_{shaft} represent the areas of the circular plate and cylindrical shaft respectively; t_{plate} is the plate thickness; γ' is the effective unit weight of soil.

Figure 6 presents the undrained vertical bearing capacity factor, N_{cV} , obtained from the base case test, PF1. The vertical bearing capacity factor resulting from the measured resistance is defined as

$$6. \quad N_{cV} = \frac{q_{net}^u}{s_u}$$

Bearing capacity factors determined from numerical studies involving large-deformation finite-element analysis – which captures the soil flow onto the plate – predict with reasonable accuracy the measured behaviour (Hu and Randolph, 1999). By taking into account the strain softening triggered by the soil backflow during the circular plate penetration, the bearing capacity factor is further reduced by 15–20% (Hossain and Randolph, 2009a, 2009b) (highlighted in grey in Figure 6), giving improved agreement with the measured response.

3.2 Monotonic preload

The effect of monotonic preload, as observed from PF2 and PF2e, is now explored. Figure 7 and Table 5 show the increase in net penetration resistance both in compression, Δq_{net}^c , and tension, Δq_{net}^t . The centrifuge tests show that, under monotonic preload and after primary consolidation, the net penetration resistance is increased twofold in compression and two-thirds in tension. Since the increase in penetration resistance is essentially proportional to the increase in the strength of the soil participating in the failure mechanism, it results that, after full primary consolidation, the kaolin beneath the footing is up to twice as strong compared to the undrained case.

Figure 8 illustrates the base case penetration resistance, q_{net}^u , compared to the enhanced penetration resistance due to consolidation, q_{net}^c and q_{net}^t . The results are also compared to the analytical approximation for increased bearing capacity after full primary consolidation derived by Gourvenec *et al.* (2014) through numerical analysis of modified Cam-Clay soil

$$7. \quad \frac{q_{net}^c}{q_{net}^u} = 1 + \frac{\Delta s_u}{s_u}$$

where Δs_u represents the change in soil strength due to consolidation under preload and is given by

Figure 5. Schematic diagram of centrifuge testing programme: net resistance plotted against time

$$8. \quad \Delta s_u = f_{su} R \Delta q'_{pl}$$

where

$$9. \quad \Delta q'_{pl} = f_{\sigma} (q_{net}^u + \gamma' h_{shaft})$$

where the introduced parameters are: f_{su} , a factor taking into account the non-uniform distribution of the increase in shear strength after consolidation; $R = s_u / \sigma'_{v0}$, the normally consolidated shear strength ratio (σ'_{v0} is the in situ vertical effective stress); h_{shaft} , length of shaft penetrated into the soil; $\Delta q'_{pl}$, the increase in consolidation stress following preloading, represented by the net penetration resistance coupled with the pressure of the soil layer above the foundation as a result of full soil backflow around the circular plate.

Figure 6. Bearing capacity factor results – comparison with theoretical solutions

The increase in penetration resistance due to monotonic preload as observed in the centrifuge is matched satisfactorily by the analytical approximation.

PF2 is further interpreted by comparing the pore pressure dissipation time history to a numerical study of a buried circular foundation on elastic soil (Gourvenec and Randolph, 2009). Good agreement is achieved by scaling the numerical result using a consolidation coefficient of $c_v = 3.3 \text{ m}^2/\text{year}$, derived from the piezoball (Figure 9).

4. Foundation results: cyclic cases

4.1 Periodic preload

The effect of periodic cyclic preload is presented in Figure 10. Each cycle consists of first applying a preload (equal to the

Figure 7. Net penetration resistance in compression and tension during monotonic preload

	Depth, z : m	$q_{\text{net},c}$: kPa/m ²	$q_{\text{net},u}$: kPa/m ²	$q_{\text{net},c}/q_{\text{net},u}$
PF2	0.62	9.08	4.74	1.9
PF2e	0.62	8.05	4.74	1.7
PF3	0.68	13.00	5.13	2.5
PF4	0.64	8.92	4.90	1.8
PF4e	1.10	17.35	7.81	2.2

Table 5. Net penetration resistance results

penetration resistance measured at $z = 0.5$ m), followed by full excess pore pressure dissipation, after which the loading is removed and the soil is allowed to swell. After five periodic cycles of preload (PF3), the net penetration resistance of the soil increased to 2.5 times the base case study, q_{net}^u . It can be seen from Figure 10 that the first cycle of preload followed by full primary consolidation results in the highest consolidation settlement. Each preload cycle increases the soil strength by generating excess pore pressure and causing a reduction in moisture content during the subsequent dissipation. All cycles create excess pore pressure, not only the first application of the preload. The changes in load – and the resulting shear strain – drive contraction of the soil, as well as the absolute level of applied load.

The excess pore pressure dissipation during the five cycles of periodic preload is summarised in Figure 11(a) and Figure 11(b). The dissipation is presented as a function of dimensionless time factor T and a suitable value of vertical coefficient of consolidation, c_v , is fitted to each dissipation stage, where

$$10. \quad c_v = \frac{TD^2}{t}$$

Figure 8. Net penetration resistance – comparison with theoretical solution (Gourvenec *et al.*, 2014)

with t representing the time since each change in preload level. A theoretical $T_{50} = 0.092$ is chosen from Gourvenec and Randolph (2009) based on their solutions for a buried circular foundation at $d/D = 0.5$ in an elastic material governed by Biot-type consolidation.

The back-calculated coefficient of consolidation, c_v , increases rapidly over the first few cycles before stabilising (Figure 11(b)). For the first stage, when the moisture content is highest, the low rate of pore water pressure dissipation (i.e. small c_v value) reflects the low stiffness of the soil. After each dissipation stage, the soil underneath the foundation is compressed and the stiffness rises. From the fourth periodic preload cycle onwards, the density and stiffness have converged close to an equilibrium state, with a c_v that is 10–20 times higher than in the initial state. This means that the gain in strength caused by the later cycles occurs much more rapidly than would be estimated from the in situ c_v .

4.2 Continuous cyclic preloading

Figure 12 shows the effect of one-way continuous cyclic preload (test PF4) on the net penetration resistance. The response is similar to the monotonic preload case (PF2): the net penetration resistance is doubled. Also, a similar pore water

Figure 9. Normalised dissipation curve during monotonic preload

Figure 10. Net penetration resistance in compression during periodic cyclic preload

pressure dissipation rate is achieved, with the PPT recording a similar underlying decay in excess pore pressure, with the response to the cyclic total stress superimposed (Figure 13). Owing to an error in the initial control sequence, the first ten cycles of preload did not reach the correct target loads. However, after the control was restarted at $T = 0.01$, the remaining 990 cycles followed the specified load pattern.

Although the mean value for the preload during one-way cycling is essentially equivalent to half of the preload subjected to the circular plate in PF2, similar gains in capacity are achieved. This suggests that the shearing during the unload–load cycles compensates for the lower mean applied load.

Under continuous two-way cyclic preload (PF4e), an even greater gain in capacity occurs. There is far greater settlement during pore pressure dissipation and the net penetration resistance is more than doubled compared to the base case study (Figure 14). The two-way nature of the preload cycles means that the mean applied load is zero. However, this is outweighed by the cyclic load component and the shearing effect, which cause a continuous rise in excess pore pressure beneath the foundation until $T = 0.3$, after which dissipation dominates over generation (Figure 15).

The total settlement under two-way cyclic preload is over six times greater than for monotonic preloading. The first cycles of continuous preload cause significant vertical settlement, and each cycle is clearly distinguishable in Figure 14. As the soil continues to be sheared and densified, the embedment stabilises. Also, the measured excess pore pressure reverts to simply a reflection of the applied total stress, as the elapsed time approaches $T \sim 1$.

Figure 11. Cyclic preloading: (a) normalised dissipation curve; (b) fitted coefficient of consolidation

The two continuous cyclic tests show that cyclic loads and the resulting shearing-induced excess pore pressures can be as significant as a static preload in contributing to consolidation-enhancement of bearing capacity. Comparison of tests PF2, PF4 and PF4e indicates that

- continuous one-way cyclic loading with a given maximum load can create the same consolidation hardening as monotonic loading at the same level, in the same time frame
- continuous two-way cyclic loading can create greater cyclic settlement and increased enhancement of bearing capacity compared to monotonic loading at the same level, in the same time frame.

The centrifuge test results mirror conventional pore pressure contour diagrams (i.e. S – N curves) derived from consolidated,

Figure 12. Net penetration resistance in compression during one-way cyclic preload

Figure 14. Net penetration resistance in compression during two-way cyclic preload

undrained, triaxial compression tests in that two-way loading creates higher excess pore pressure (Koutsoftas, 1978). The additional observation is that permitting pore pressure dissipation converts these increased pore pressures into an increase in the gain in soil undrained strength and bearing capacity.

The consolidation-induced increase in bearing capacity from the monotonic case in this study shows good agreement with a simple prediction method (Gourvenec *et al.*, 2014). A further challenge is to predict the influence of cycles of loading, and the resulting gain in bearing capacity. These model tests show that both continuous and periodic cycles create an additional significant enhancement of bearing capacity (see Figure 8).

Figure 13. Normalised dissipation curve during cyclic preload – comparison with monotonic preload

5. Conclusion

The effect of monotonic and cyclic vertical preload on the strength of a soft clay was investigated by means of centrifuge testing. The overall conclusion is that, although monotonic vertical preload is known to generate a significant increase in bearing capacity, this increase is enhanced further if the preloading is cyclic. It was found that the undrained strength of a foundation can double, after being subjected to drained cycles of vertical preload, with two-way cyclic loading generating a greater gain than one-way loading. The explanation for these trends is that, during the cyclic preloading, if sufficient time for drainage is permitted, the soil contracts and the undrained strength increases. Two-way loading is more effective at pore pressure generation and densification than one-way loading.

Figure 15. Consolidation settlement curves

The excess pore water pressure dissipates at similar rates for both monotonic and continuous cyclic preload. For periodic or drained cycles of preload, successive episodes of dissipation and strength gain become more rapid. The back-calculated consolidation coefficients indicate a 10–20 times increase in c_v after only five cycles.

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