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Centrifuge modelling to determine the influence of pile stiffness on pile capacity

Modélisation centrifuge pour déterminer l'influence de la rigidité des pieux sur la capacité des pieux

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ABSTRACT: The stiffness of piles relates to their ability to resist deformation in response to an applied force. The modulus of elasticity of an uncracked concrete piles typically varies between 30-40kN/m^2. Under axial loading these high stiffness piles transfer the load through the pile to the base. This results in low mobilisation of shaft friction as the stiff pile displaces uniformly and therefore the magnitude of skin friction along the entire length of the pile is small. In addition, base resistance of deep piles is mobilised at very high loads which may exceed the working load of the pile shaft. The adhesion factor, α, for bored piles in London Clay can range between 0.45 and 0.6 suggesting that a significant proportion of the soil strength cannot be mobilised. This low mobilisation of shaft resistance means that the ultimate bearing capacity is much reduced. The research investigated the behaviour of a low stiffness pile under axial load and compared this with a conventional high stiffness pile. The results demonstrated that the low stiffness pile exhibited marginally greater capacity at working load and a noticeably improved capacity at ultimate load.

RÉSUMÉ: La rigidité des pieux est liée à leur capacité à résister à la déformation sous l'effet d'une force appliquée. Le module d'élasticité des pieux en béton varie généralement entre 30 et 40 kN/m^2. Sous charge axiale, ces pieux à rigidité élevée transfèrent la charge à travers le pieu jusqu'à la base. Il en résulte une faible mobilisation du frottement de l'arbre car le pieu rigide se déplace uniformément et l'ampleur du frottement de la peau sur toute la longueur du pieu est donc faible. De plus, la résistance de base des pieux profonds est mobilisée à des charges très élevées qui peuvent dépasser la charge de travail du puits de pieux. Le facteur d'adhérence, α, pour les pieux forés dans l'argile londonienne peut varier entre 0.45 et 0.6, ce qui suggère qu'une proportion importante de la résistance du sol ne peut être mobilisée. Cette faible mobilisation de la résistance de l'arbre signifie que la capacité portante finale est très réduite. La recherche a étudié le comportement d'un pieu à faible rigidité sous charge axiale et l'a comparé à celui d'un pieu à rigidité élevée conventionnel. Les résultats ont démontré que le pieu à faible rigidité présentait une capacité légèrement supérieure à la charge de travail et une capacité sensiblement améliorée à la charge ultime.

Keywords: Pile stiffness; centrifuge modelling; pile capacity; deep foundations; pile design.

1 BACKGROUND

Conventional bored concrete piles are designed as very stiff inclusions which are designed to transfer loads from a structure into the ground. The performance of a pile is governed by its base and shaft resistance. Piles founded on sands and gravels generate the majority of their capacity from end bearing resistance. In clayey ground bored pile capacity is generally achieved through skin friction, with rough concrete piles achieving an adhesion factor, α, ranging between 0.45 – 0.6 in stiff soils, such as London Clay (Bell & Robinson, 2012). This α value is an indication of the proportion of soil strength that is mobilised by the pile; the higher the α value the greater the capacity of the pile for a given soil strength. In soft soils the α value is typically higher, however mobilising a larger proportion of the undrained shear strength of soil in stiff clays is challenging.

2 INTRODUCTION

In order for a concrete pile to generate capacity the pile is required to displace. The capacity of a pile at working load can be inferred as the axial load at approximately 1% settlement normalised against the pile diameter (Patel, 1992). It is generally accepted that a stiff pile will mobilise more load at smaller displacement as the load is shed at greater depth in stiffer soil. It is also acknowledged that greater displacements are necessary to mobilise end bearing.

Therefore an infinitely stiff concrete pile will mobilise a small proportion of shaft friction before reaching ultimate capacity. As the strain in the pile is negligble, the pile displaces uniformly and sheds the load at depth.

However, if piles were less stiff they would be subject to greater strain and may be able to transfer larger loads along the pile before base capacity is mobilised. This would consequently generate more shaft friction along the entire length of the pile, as opposed to transferring a large proportion of the load through to the base of the pile.

3 OBJECTIVES

The aim of this project was to compare the capacity of a conventional solid shafted stiff pile against a lower stiffness pile of equal dimensions.

One centrifuge model test was conducted at 50*g* at City, University of London. The purpose of the test was to understand the influence of pile stiffness on the overall behaviour and performance of piles and establish whether any benefits to capacity could be gained by reducing pile stiffness.

4 SOIL MODEL

The centrifuge test was conducted in a 415mm deep cylindrical steel tub and 420mm diameter in plan. It was necessary to create a soil sample that was flush with the top of the centrifuge tub, therefore a removable 300mm deep extension was bolted to the top of the tub. The internal faces of the tub and extension were lubricated with waterpump grease to reduce friction between the slurry and the tub. Speswhite kaolin clay was mixed with distilled water to produce a workable slurry with a water content of 120% which is approximately twice its liquid limit. Sheets of 3mm thick porous plastic and filter paper were placed at the bottom of the centrifuge tub over herringbone drainage channels. The clay slurry was carefully placed into the tub using a scoop and was agitated with a palette knife to avoid the entrapment of air bubbles.

The sample was transferred to a hydraulic press where a tightly fitting circular platen was attached to the loading ram. The platen was perforated to allow drainage from the top of the sample whilst the herringbone channels at the base of the sample diverted water to drainage taps and pipes leading to a bucket. The ends of the pipes remained submerged in water to prevent the entrapment of air in the sample. Drainage from the top and bottom of the sample halved the drainage path length which accelerated the rate of consolidation. The sample was consolidated

incrementally from 10kPa to 500kPa over a period of a week.

Once it was confirmed that the sample had consolidated it was swelled to 250kPa immediately prior to testing. A pore pressure transducer (PPT) was later installed at a depth of 200mm to the centre of the tub and was backfilled with slurry mixed to a water content of 120%.

5 APPARATUS

The testing apparatus used for this experiment was designed by Gorasia (2013) and comprised a loading frame to which a motor, lead screw actuator and loading beam were connected. 5kN miniature load cells were fixed to the loading beam and aligned with the model piles. The known speed of the motor permitted calculation of the rate of displacement of the piles.

The piles were machined from aluminium solid bar to produce 16mm diameter piles with an embedded length of 180mm. Boundary effects can be assumed to be negligible (Bousinesque, 1885; Ullah *et al*., 2016). The faces of the model piles were sandblasted to simulate the rough surface of a bored pile. Whilst the solid stiff pile was machined as a single piece, the lower stiffness pile was formed from a number of aluminium sections. The sections were adhered to 0.5mm thick rubber discs to produce a 180mm pile with an additional 20mm upstand. A photograph of the model piles is presented in Figure 1.

Figure 1. Model piles used in centrifuge test (a) low stiffness pile and (b) continuous high stiffness pile.

6 MODEL MAKING AND TESTING PROCEDURE

Prior to removing the sample from the consolidation press, all standing water was removed and the drainage taps closed. The extension was unbolted and the lifted off whilst care was taken to avoid disturbing the sample. A wire cutter was dragged across the top of the sample to remove excess clay and trim the sample flush with the top of the tub. The model was prepared under 1*g* conditions. A bespoke modelling frame was bolted to the top of the tub which established the locations of the piles, shown in Figure 2. Thin walled 16mmOD tubes were used to cut 180mm deep bores. 50mm high collars bolted to the frame ensured that the pile bore remained vertical and ensured that both bored piles were of equal length.

Two piles were arranged around two thirds of the central axis of the tub. The third location in the model was used for obtaining an undrained shear strength profile with depth. Once the clay had been bored using the guide and thin walled cutters the piles were carefully inserted.

PlastiDip, an aerosol applied synthetic rubber membrane, was sprayed across the surface of the sample to prevent moisture loss during in-flight consolidation. The loading frame, with miniature load cells pre-attached, was carefully lowered onto the model before being placed on the swing, shown in Figure 3. A standpipe was connected to the model to establish a water table 10mm below ground level. The model was accelerated to 50*g* and was typically consolidated for a minimum of 24 hours to allow excess pore pressures to dissipate, which was confirmed by the PPT in the sample. The apparatus and centrifuge model is illustrated schematically in Figure 4.

The piles were loaded at a rate of 1mm/minute whilst the load settlement response was observed. The test continued until the ultimate load was reached. The the motor operated at a constant rate which allowed The magnitude of settlement to be calculated. Figure 5 presents a photograph of the exhumed segmental pile post-test.

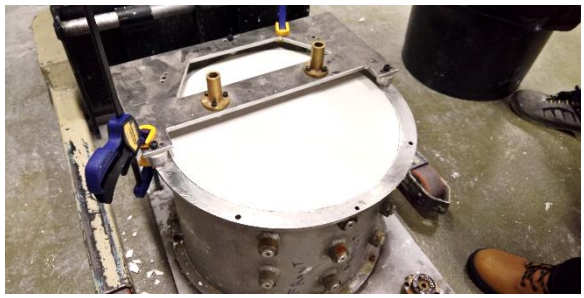

Figure 2. Image of modelling frame and collars used to locate the piles and form vertical bores of equal length.

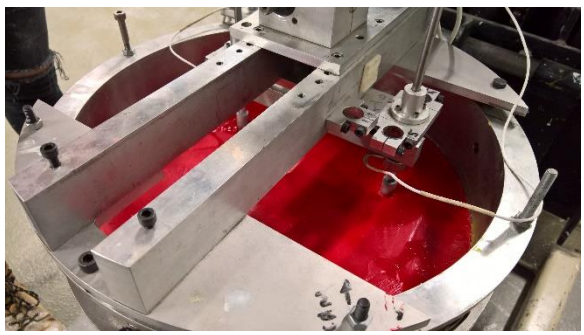

Figure 3. Model immediately prior to spin up.

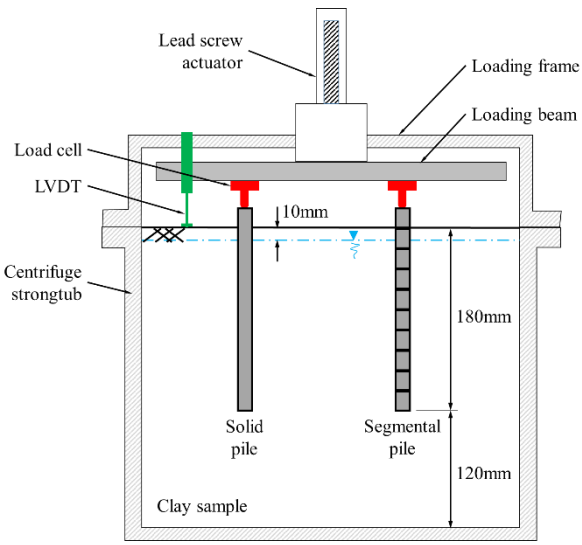

Figure 4. Centrifuge experiment set up.

Figure 5. Low stiffness pile exhumed from model following test.

7 TEST RESULTS

One preliminary test has been carried out to determine the comparative pile behaviours of a solid stiff pile and a segmented low stiffness pile.

Slight misalignment between the top of the solid pile and the load cell resulted in an observed bedding in load, illustrated in Figure 6. The loads measured in the test have been plotted against the pile displacement normalised with the pile diameter are presented in Figure 7. The bedding in load, shown as the light grey markers in Figure 7, suggested large pile displacements for little increase in load. The data was corrected to account for this in Figure 7.

The results demonstrated comparable stiffness behaviour between the high and low stiffness piles at working load, i.e. at 1% normalised settlement.

As the pile load continued to increase, the low stiffness pile demonstrated a noticeably stiffer geotechnical response than the high stiffness pile. At ultimate capacity, the low stiffness pile provided 7.5% greater capacity than the solid high stiffness pile.

Following the test the model was decelerated and the undrained shear strength profile was immediately obtained from shear vane readings. The measured results are plotted in Figure 8 alongside S_u estimates calculated using the Springman (1989) and Phillips (1987) empirical formulae; equations (1) and (2) respectively. The measured profile showed reasonable consistency with the empirical undrained shear strength profiles.

$$S_u = 0.19\sigma'_v\,(OCR)^{0.71} \quad (1)$$

$$S_u = 0.19\sigma'_v\,(OCR)^{0.67} \quad (2)$$

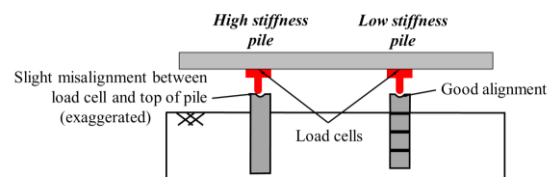

Figure 6. Bedding in load owing to slight misalignment of pile and load cell

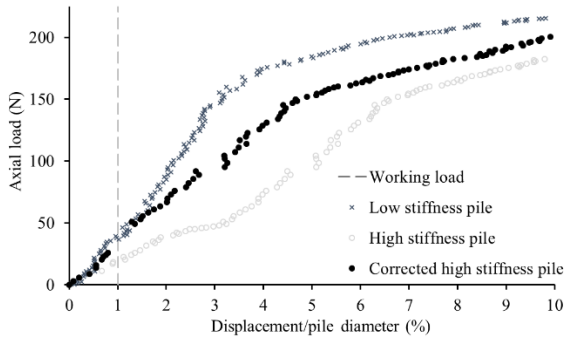

Figure 7. Results from centrifuge test demonstrating the load-settlement response of high and low stiffness piles.

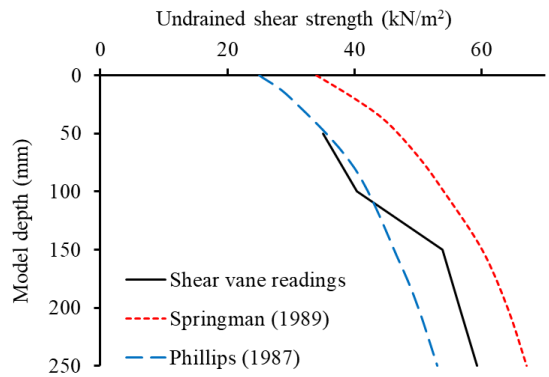

Figure 8. Model undrained shear strength profile measured using a shear vane.

8 RESULTS ANALYSIS

Following the principle of Terzaghi (1943), the total pile capacity, Q_f, is a summation of end bearing (Q_b) and shaft resistance (Q_s), as defined in equation 3.

$$Q_f = Q_b + Q_s \tag{3}$$

$$Q_b = A_{base}\,(N_c\,S_u + \gamma\,H) \tag{4}$$

$$Q_s = A_{shaft}\,\alpha\,S_u \tag{5}$$

Where A (m^2) is the area of the pile base or shaft respectively, N_c is the dimensionless bearing capacity factor, S_u (kN/m^2) is the undrained shear strength of soil, γ (kN/m^3) is the bulk unit weight of soil, H (m) is the embedded length of the pile and α is the dimensionless adhesion factor. These values are provided in Table 1.

Considering that the base area of both piles were identical, it is reasonable to suggest that the magnitude of end bearing capacity are equal. However, the concept of a low stiffness pile focuses on permitting additional straining along the length of the pile. In doing so, the low stiffness pile should in theory mobilise a higher proportion of the undrained shear strength. Consequently, the α value of the low stiffness pile would be expected to be greater than the high stiffness pile.

The capacities of both piles at working and ultimate states were used in the back analyses of α. Results of the analysis are presented in Table 2 and demonstrate that the lower stiffness pile was better able to mobilise the strength of the soil, resulting in 24% greater mobilisation of undrained shear strength at ultimate capacity. This supports the theory that a low stiffness pile sheds load along the length of the pile as it strains.

Table 1. Soil and model properties used in the back analysis of centrifuge test results

Property	Value	Units
γ	17.44	kN/m^3
$Su_{(ave)}$	46	kN/m^2
$Su_{(base)}$	56	kN/m^2
H	0.18	m
N_c ($^L/_B$ >> 4)	9	-
A_{base}	0.000201	m^2
A_{shaft}	0.009048	m^2
Q_b	0.133	kN

Table 2. Summary of alpha values

Normalised displacement	Load (kN)	Q_s (kN)	α
High stiffness solid pile			
1%	0.040	-	-
10%	0.200	0.067	0.161
Low stiffness rubber pile			
1%	0.040	-	-
10%	0.216	0.083	0.200

9 DISCUSSION

Low α values are often found during back analysis of centrifuge model tests owing to the very smooth bores that are created during model making. It is, however, necessary to form smooth bores to achieve consistency between tests.

Low stiffness piles were hypothesised to increase the capacity of the pile by allowing the pile to strain by a greater magnitude than conventional stiff piles. This subsequently increases the magnitude of shaft friction acting along the entire length of the pile and is schematically illustrated in Figure 10. Consequently, the relationship between the expected capacities of different piles can be approximated. For example, Figure 10 illustrates three cases; (a) a solid pile in clay (comparable with the profile observed in the centrifuge test), (b) a low stiffness material above a stiff material and (c) a low stiffness pile in clay soil. The typical load settlement response of a solid concrete pile in fine grained soil is illustrated in Figure 10(a). If, however, a compressible material was placed on top of a stiff section and loaded, then the load-settlement response could be expected to be akin to Figure 10(b). Based on this principle, a segmental pile comprising a series of rubber discs and solid aluminium sections and embedded in clay would exhibit the load-settlement profile shown in Figure 10(c).

Embedment of the segmental pile in clay would be affected by the undrained shear strength of the surrounding soil. The skin friction generated along each segment of the pile would result in an increase in capacity over and above the solid pile owing to the greater strain of the system. The locked in-situ stresses during spin up and consolidation were not quantified in this test.

In addition, the undrained shear strength profile of the ground measured in this centrifuge test was shown to steadily increase with depth up to approximately 150mm below ground level, as shown in Figure 8. Therefore, the additional resistance afforded to the pile would also increase with depth. Therefore, it would be reasonable to expect that the overall load settlement profile would continue to increase until the undrained shear strength in the overconsolidated sample plateaus.

Figure 11 focuses on the pile settlement response from the centrifuge test at low strain. Trendlines have been superimposed on the graph to provide an understanding of the behaviour of the pile up to working load. The high stiffness solid pile gave a relatively constant response to loading whilst the low stiffness pile was shown to have been affected by the rubber discs. Based on simple calculations of the measured undrained shear strength profile and segment areas failure loads would be expected to range from 30N near to the surface and 55N at depth. This is comparable with the results plotted in Figure 11.

This behaviour validates the hypothesis of this experiment as the low stiffness pile was shown to undergo cycles of large strain followed by a noticeable increase in the measured load. This response suggests that the compressible rubber disc deformed under the applied load by approximately 0.1% as it transferred some of the load into the surrounding soil and the segment immediately below.

The rubber disc pile was formed from ten aluminium segments and nine rubber discs; each disc was 0.5mm thick. Arguably, the aluminium segments were infinitely stiff in comparison to the rubber discs and the soil. Therefore, in order for the load to transfer through to the base of the pile, each of the nine rubber discs would need to be compressed.

The rubber material used in this test had a Shore hardness rating of 50A; which suggests a Young's Modulus of approximately 1.9MPa. At the pile working load the magnitude of displacement of the system was equal to 0.16mm. The measured load applied to the low stiffness pile at working load was 39N. Following Hooke's law, it would suggest that the rubber had strained 10%; however a solid aluminium section, with a Young's modulus of 69GPa would have strained the negligible amount of 0.0003%.

However, at 1% pile system strain the measured axial load was 39N which highlights the complex interaction between the differential stiffnesses of the pile system and soil in response to axial loading. Overall, a lower pile stiffness was shown to have resulted in an increase system stiffness.

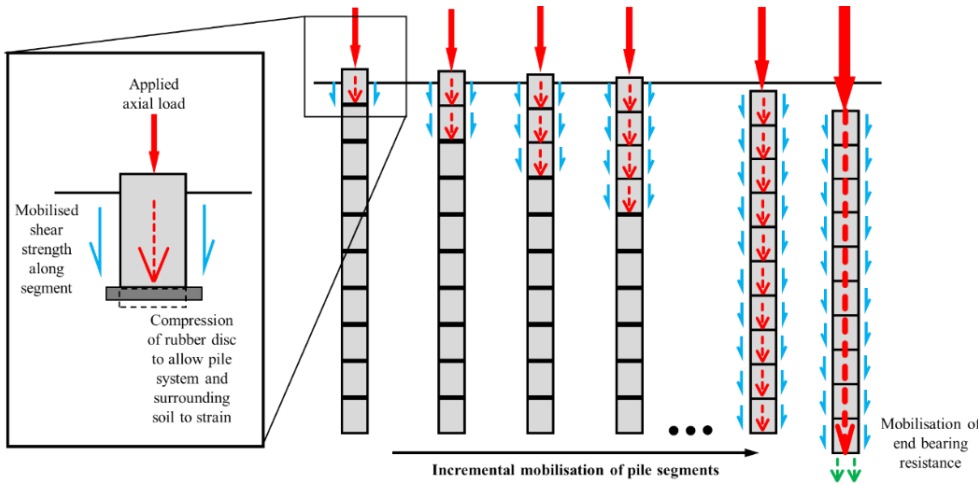

Figure 9. Effect of rubber discs on shaft friction of low stiffness pile.

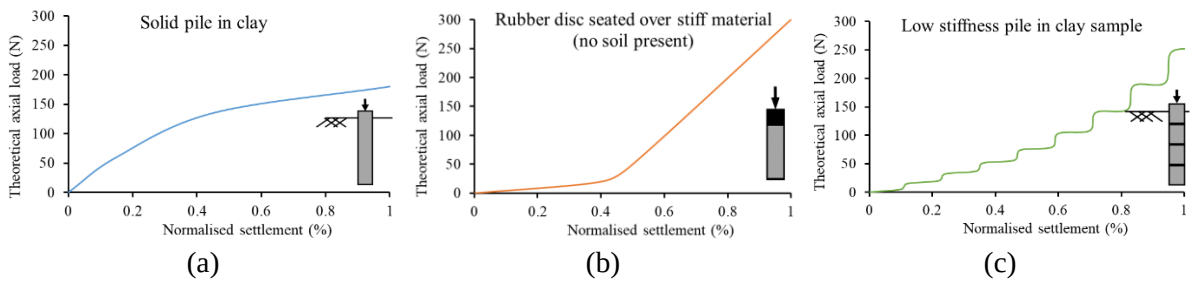

Figure 10. Simplified idealisation of the load-settlement characteristics for (a) a solid pile (b) a rubber disc seated on a stiff material and (c) a segmental rubber discs and aluminium pile in clay soil.

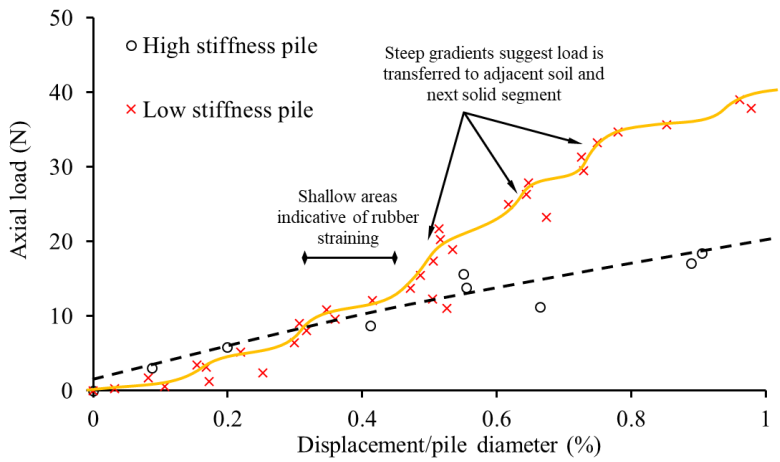

Figure 11. Illustration of the behaviour of different pile stiffnesses up to working load.

10 CONCLUSIONS

One preliminary centrifuge test at 50*g* was conducted to investigate the performance of a low stiffness pile compared with a conventional high stiffness solid pile.

It was found that although the low stiffness pile could displace by a greater magnitude than the solid pile, in doing so the low stiffness pile was able to mobilise a higher proportion of skin friction along each segment. This resulted in the low stiffness pile achieving greater capacity at both working and ultimate states.

The load-settlement behaviours of each pile were also analysed and clearly showed that the low stiffness pile transferred the applied axial load to the surrounding soil and solid aluminium segment directly below. The cyclic behaviour of load transfer is consistent with the theory that a lower pile stiffness allows the pile to strain which subsequently results in more interaction between the pile and soil.

11 RECOMMENDATIONS FOR FURTHER WORK

The results from this centrifuge test provide a basis for further research to determine an optimum stiffness for maximising the capacity of a pile.

Investigations regarding the actual soil-structure interaction should also be made in an attempt to quantify and define the load-displacement mechanism that arises as a result of loading low stiffness piles in fine grained soils.

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