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## **HOLDING CAPACITY OF SUCTION ANCHORS WITH TRENCH - CENTRIFUGE TEST RESULTS AND INTERPRETATION**

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### **ABSTRACT**

As part of the research initiated by SBMO on the chain trenching phenomena, a series for centrifuge tests (mainly at 75g) were performed on model anchors embedded in reconsolidated and intact Gulf of Guinea clay. Vertical and inclined pull-out tests were performed both with and without a trench. Anchor movement and pore pressures (suction) at different positions in the soil below the anchor were monitored during loading to assess reverse end bearing mobilisation. The results demonstrate that the trench does not significantly affect reverse end bearing but the holding capacity for typical inclined loading is reduced by approximately 20% for this specific anchor and trench geometry. 2D and 3D total and effective stress finite element analyses were performed as part of the interpretation work and some results are briefly presented in this paper.

**Keywords:** holding capacity, anchor, trench, centrifuge modelling, interpretation

### **INTRODUCTION**

In-service oscillations of the anchor chain in semi-taut mooring systems have been observed to create a trench in front of the anchor in some soils (Bhattacharjee et al., 2014; Arslan et al., 2015; Colliat et al., 2018; Sassi et al., 2018). This potentially affects the ultimate holding capacity of the anchor.

As part of the re-assessment of the holding capacity of anchor piles with a trench, SBM Offshore (SBMO) in collaboration with Total commissioned a series of centrifuge tests at the University Gustave Eiffel (Nantes Campus). SBMO commissioned Cathie Associates (Cathie) to provide advice on the program and provide interpretative reports following the tests.

The scope of the centrifuge testing included a series of vertical and inclined holding capacity tests on pile anchors without a trench (to provide reference test cases) and with a trench. A maximum trench depth equal to the padeye depth (9.5m below mudline or about 2/3 of the pile penetration) was considered. Centrifuge tests included measurement of pore pressures at different levels below the anchor tip and measurements of pile head settlements and rotations to provide insights into the failure mechanism and mobilisation of reverse end bearing (REB). In total, 10 series of centrifuge tests were performed in 10 containers with different types of sample reconstitution (CT1 – CT10). These included vertical pull-out test series (CT2, CT7, CT8) and inclined load test series (CT6, CT9 and CT10) which are the subject of this paper.

Finite element (FE) simulations of the tests were performed but only selected results are provided in this paper. Further details of the FE work are available in Sassi et al (2018).

## CENTRIFUGE MODELLING

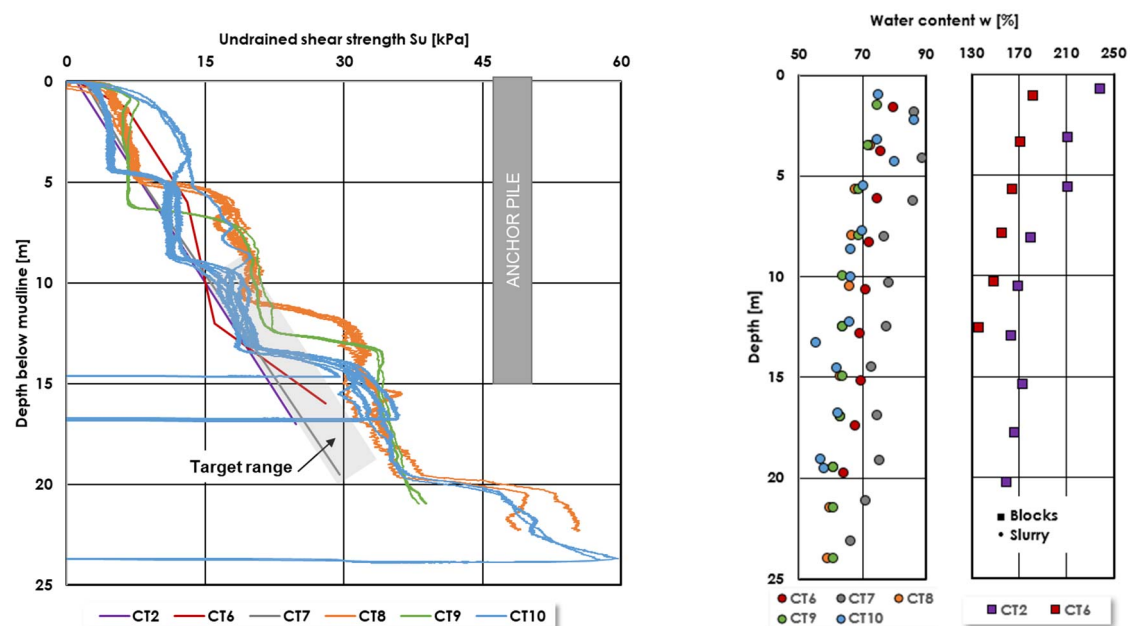
### General

All reported centrifuge tests were performed in the Univ. Gustave Eiffel (formerly Ifsttar) fixed beam centrifuge operating at 75 or 80g. Further details are provided in Sassi et al. (2018). Two slightly different model piles were used so that the prototype anchor pile was 4.5m OD and 15m penetration. For series CT2, the pile was made of aluminium with a machined surface and 56mm wall thickness. For all other tests, the piles were of stainless steel, sandblasted with a wall thickness of 37.5mm. The padeye was attached at 9.5m below seabed level. In the first tests, installation was by push-in. In subsequent tests, a combination of self-weight penetration and suction installation was used.

### Soil model

Gulf of Guinea (GoG) clay was used for these centrifuge tests, either in reconsolidated slurry form or as intact blocks cut directly from undisturbed large diameter samples. Intact blocks were used over the whole model for CT2. All other tests reported here used reconsolidated GoG slurry to form the main soil body, in order to mobilize the REB, which was not developed in blocky reconstituted soil mass (probably due to preferential drainage at the blocks' interfaces). A zone was then cut out (at 1g) in front of the pile and replaced with undisturbed blocks into which a trench was cut. These model soils are referred to as the "hybrid" model. The undisturbed soil allowed a realistic and stable trench to be formed. However, the use of blocks creates interfaces between blocks which were approximately normally consolidated after the final centrifuge consolidation stage. As the particle size of GoG clay is very small in comparison to the size of the suction caisson model or of the T-bar, no soil material scale effect needs to be considered (e.g. Garnier et al., 2007).

Water content profiles were taken at the end of most of the tests and several miniature T-bar tests (5mm OD) were performed at different stages of testing in both reconstituted clay and in blocks (at 3mm/sec to ensure undrained conditions). The results are interpreted using  $N_T=10.5$  (e.g. Gaudin et al. 2008) in order to estimate the undrained shear strength (Figure 1).



Note: Only simplified strength profiles from T-bar tests are shown for CT2, 6 and 7 for legibility

**Figure 1: Static T-bar results and water content profiles**

The differences in shear strength profile are a result of slight changes between the test series affecting consolidation procedures, consolidation stresses, layering or incomplete consolidation at 1g due to scheduling limits. Field target strength profiles are enveloped by the range achieved. The lower water content and thus higher unit weight of the reconstituted clay is clearly apparent in Figure 1 (right-hand panel). Cyclic T-bar tests were also performed to characterize the sensitivity. A simplified set of undrained strength profiles (Table 1) are considered representative of the test conditions.

**Table 1: Simplified soil profiles**

Parameter	Unit	CT2	CT6	CT7	CT8	CT9	CT10
Submerged unit weight	kN/m <sup>3</sup>	3.5	6	6	6	6	6
$s_{u0}$	kPa	1	1	2	3	2	2
k	kPa/m	1.4	1.4	1.35	1.9	2.1	1.8
$s_u$ at pile tip	kPa	22	22	22	32	34	29

Notes:  $s_u$  based on T-bar test with  $N_T=10.5$

Higher unit weight in CT6-10 corresponds to reconstituted slurry with lower water content than natural Angola clay

## Trench

Tests CT2 – CT9 were performed with a trench geometry which had a width of 2D at the surface reducing to 1D at a depth of 1D, and then constant width of 1D down to the padeye level. This was selected at the start of the program before actual field data from surveys was available (Sassi et al., 2018; Colliat et al., 2018). For the final test set (CT10), a realistic geometry with a side slope of approximately 60° was adopted (based on observed stable slopes in the field).

## Loading

Tests reported herein were performed for vertical and inclined loading (between 30° and 40° representative of extreme conditions for semi-taut deepwater mooring systems). Table 2 summarises the loading and trench conditions for the tests presented in this paper. All test series except CT2 were performed at a loading rate of 25mm/min. This may be compared to the T-bar penetration rate of 180mm/min (3mm/sec). No rate correction has been applied in this report to the inclined load tests since all were performed at the same rate. However, a rate correction in the form of a strength reduction factor is applicable to matching FE modelling based on the T-bar measured shear strength with centrifuge model results.

## VERTICAL LOADING TEST RESULTS (CT2, CT7, CT8)

Test series CT2 was performed in the blocky clay model. The net pull-out load-deflection response of the CT2 series of tests are presented in Figure 2, where the net load is the measured pull-out load less the anchor weight.

The presence of the trench has negligible influence on the ultimate vertical pull-out capacity when full REB is available (tests C2P1 and C2P2) as the trench does not affect the mobilized end bearing resistance. A slight reduction in stiffness is identified which may be due to the reduced volume of soil around the anchor due to the trench.

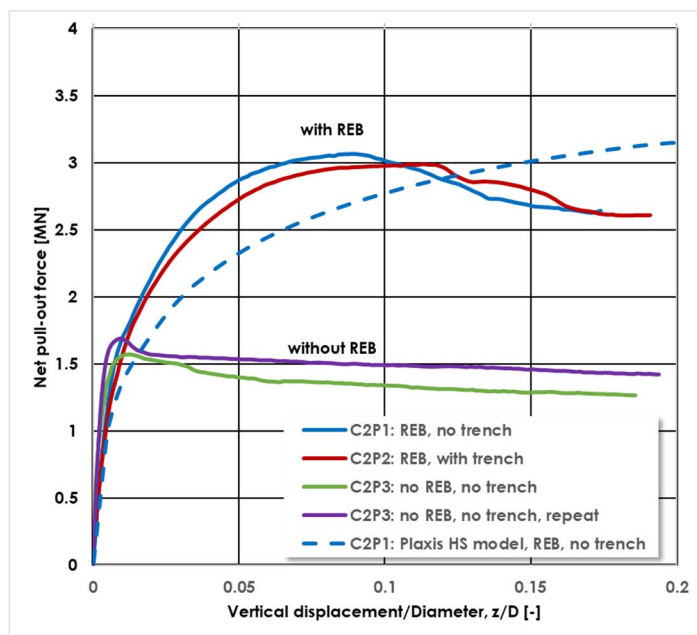
For the two tests C2P3 without REB (the anchor tip was in contact with a sand layer at the bottom of the container which was drained), the consistency is very good. Furthermore, a markedly different behaviour is obtained for these tests without REB, showing that the REB

**Table 2: Summary of centrifuge tests**

Test series	CT2			CT6				CT7		CT8		CT9		CT10	
Test name	C2P1	C2P2	C2P3	C6P1	C6P2	C6P2	C6P4	C7P1	C7P2	C8P1	C8P2	C9P1	C9P2	C10P1	C10P2
<b>Feature</b>															
Soil preparation method [-]	Undisturbed blocks (70x70mm model scale)			Hybrid 3 layers				Hybrid 3 layers		Hybrid 4 layers		Hybrid 4 layers		Hybrid 5 layers	
Penetration [m]	15			15				15   15.8		15		15		14.3   14.8	
Load direction [°] (*)	90			30   0   40   0				90		90		30   40		35	
Trench [-]	no	yes	no	no   yes				no	yes	no	yes	no	yes	no	yes
Model load rate [mm/min]	6			25				25		25		25		25	

**NOTES**

- (\*) load direction is referred to horizontal axis, *counterclock direction*
- C2P3 was reinstalled twice and tested at different pull out rates in the same location
- C6P2 and C6P4 not interpreted
- C7P1 installation by suction caused soil disturbance



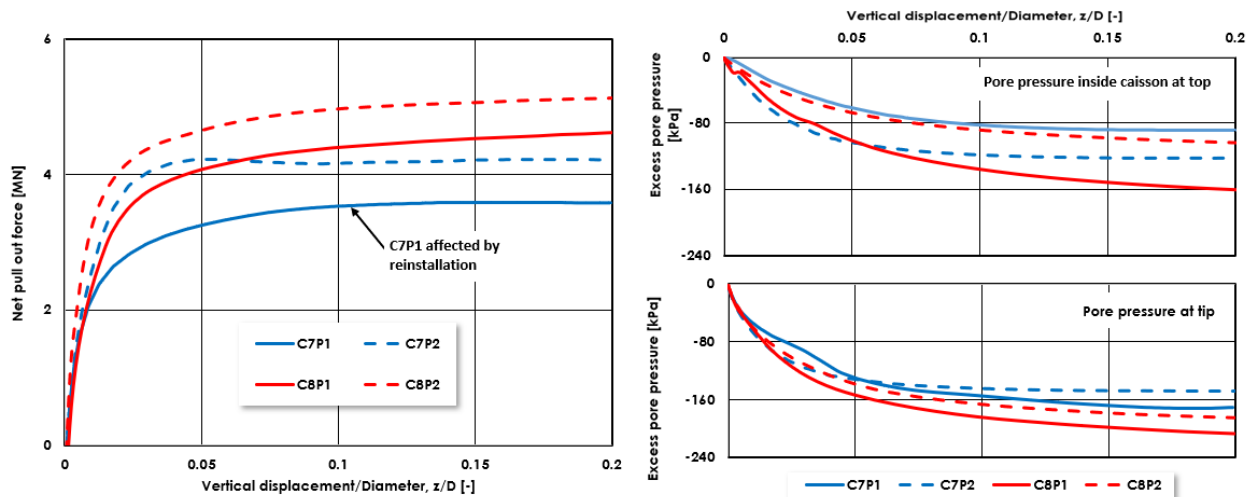
**Figure 2: Vertical pull-out test series CT2 using intact clay blocks showing influence of REB and trench**

capacity represents about 50% of the ultimate pull-out capacity, as noted by Andersen et al., (2005). Peak-residual response is apparent for shaft resistance only, as observed also for pile shaft resistance in sensitive soft clays. The REB is mobilized at higher displacements (10%) than shaft resistance (1.4%). This is in good agreement with previously published centrifuge tests results, e.g. Raines et al. (2005). An average interface factor  $\alpha$  of 0.34 can be back calculated for the inside and outside shaft resistance from tests C2P3.

Test series CT7 was designed to replicate CT2, but with the model soil primarily consolidated from clay slurry and including measurement of the pore pressures. A thicker model soil was used to reduce the base boundary effects, and the anchor was penetrated by suction. In the area of the trench, the consolidated slurry was replaced by intact clay blocks (“hybrid” model

soil). Results for the series CT7 and CT8 are presented in Figure 3. In the reference test C8P1, the pile had to be removed and re-installed which probably explains the lower pull-out capacity. A larger set of experiments (more than 2) is ideally required to confirm these results.

A summary of the results of all vertical pull-out tests is provided in Table 3. Comparing CT7 and CT8, the uplift resistance in both reference cases is lower than the resistance with a trench. Considering the installation issues with both tests for the reference pile, it is not prudent to draw conclusions from these results except that the pull-out capacity does not appear to be significantly affected by the trench. This is probably due to the large contribution of the REB as revealed by the suction pressures.



Note: P1 – reference tests without trench, P2 - tests with trench

**Figure 3: Vertical pull-out tests CT7 and CT8**

Suction pore water pressures at the pile tip are slightly lower for the trench case in both tests although the effect of disturbance in the reference tests on these pressures is unknown. This may imply a reduction in REB due to the trench but if there is any reduction it is very limited. Pore pressure dissipation was negligible based on a reasonable estimate of the coefficient of consolidation and drainage path length between pile tip and the trench surface.

The lower uplift capacities in series CT2 (Figure 2) when compared to CT7 and 8 (Figure 3) may be due to slower pull-out rate, partial dissipation of suction pressures on block interfaces, and the proximity of the bottom sand boundary at 1.3D below the anchor tip. The machined aluminium pile material used for CT2, may also have contributed to the lower uplift resistance compared to the CT7 and 8 tests.

The effective REB bearing capacity factor  $N_c$  (defined here as  $\Delta u/s_{u\_tip}$ , where  $\Delta u$  is the suction pressure at the pile tip) is shown in Table 3. Values of  $N_c$  between 6 and 8 agree with the findings of Jostad et al. (2015) but slightly lower than Jeanjean et al. (2006) at peak resistance ( $N_c = 9$ ). A rate correction (0.84) on  $s_u$  based on T-bar rate of 180mm/min and loading rate of 25mm/min – Colliat et al, 2010) would reduce  $s_u$  and increase  $N_c$ .

**Table 3: Results of vertical pull-out test series - CT2, CT7 and CT8**

Test	Installation method	Trench	Peak pull-out resistance [MN]	$\Delta u$ peak [kPa]	$\delta/D$ at peak resistance [%]	REB $N_c \Delta u/s_u$ (3)
C2P1	Push in	No	3.1	-	9	
C2P2	Push in	Yes	3.0	-	12	
C2P3 (1)	Push in	No	1.6	-	1.4	
C7P1	Self-weight/suction	No	3.6 <sup>(2)</sup>	-172	12	7.8
C7P2	Self-weight/suction	Yes	4.2	-148	5	6.7
C8P1	Self-weight/suction	No	4.6 <sup>(2)</sup>	-206	20	6.4
C8P2	Self-weight/suction	Yes	5.1	-185	20	5.9

Notes: (1) Coring and vented; (2) soil disturbed due to repeat installation probably reduced uplift resistance; (3) ignoring potential rate correction which would reduce  $s_u$  and increase  $N_c$ .

### INCLINED LOADING TEST RESULTS (CT6, CT9, CT10)

Results for the inclined loading tests are summarised in Table 4 and the load-deflection curves in Figure 4 (continuous line for reference tests, dotted lines for with trench tests).

**Table 4: Comparison of inclined pull-out tests CT6, CT9 and CT10**

Test	Installation method	Trench	Load Inclination [°]	Maximum pull-out resistance [MN]	$\Delta u^1$ [kPa]	$\delta/D^2$ [%]
C6P1	Push-in	No	30	6.7	-80	22
C6P2	Push-in	Yes	40	5.4	-76	16
C9P1	Self-weight/suction	No	30	8.0	-97	19
C9P2	Self-weight/suction	Yes	40	6.3	-	16
C10P1	Self-weight/suction	No	35	8.2	-95	14
C10P2	Self-weight/suction	Yes	35	7.0	-91	14

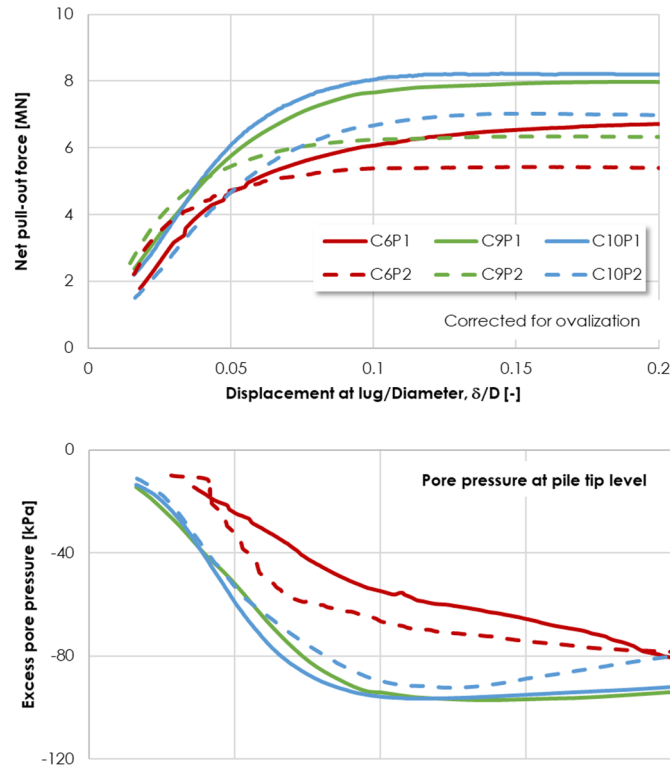
$\delta$  = anchor displacement at load application point, D = pile diameter,  $\Delta u$  = tip pore pressure

<sup>1</sup> at peak pullout resistance

<sup>2</sup> at peak pullout resistance – displacements corrected to compensate for ovalization (plastic deformation) of the pile

C9P1 and C10P1 reference cases indicate very similar load/deflection behaviour and ultimate capacity (~8MN). The lower capacity exhibited in C6P1 is probably due to the lower shear strength profile (on the order of 70% - see Figure 1). The presence of the trench causes a reduction in holding capacity of between 15 – 21% although the effect of the different loading angles may also play a part. In the case of CT10, in which the loading angle was the same, the capacity reduction was 15%.

In all the tests the maximum capacity is mobilized between 14% and 22%D; at these ratios, the suction pressure measurements at the tip were typically in the range 80 to 100kPa which is much less than the fully mobilised suction pressure observed in the vertical loading tests (150 – 200kPa, see Table 3). This limited mobilisation of suction pressure is due to the translational and rotational failure mechanism of the pile.



**Figure 4: Comparison of results of inclined pull-out tests CT6, CT9, CT10**

Measured pore pressures broadly follow the trend of the force-displacement response suggesting a direct relationship.

Finite element (FE) modelling of the tests was performed; further details are provided in Sassi et al (2018). The ability of a simple Mohr Coulomb/Tresca model, and an effective stress model to predict the holding capacity of the anchor with and without trench is summarised in Table 5. Soil parameters for the modelling were based on best estimates from the in-situ and laboratory testing. No attempt has been made to calibrate the finite element model to fit the centrifuge results.

Without a strain rate correction, the FE results at 0.1D tend to overshoot the measured capacities. If the strain rate correction is applied (0.84 – see above) the FE results for CT6 and CT9 match quite well but undershoot significantly for CT10.

The failure mechanism of piles with and without a trench for the inclined load tests is by backwards rotation and forward translation. The failure plane is visible from a section which was excavated after the test (Figure 5). This figure also reveals full attachment of the soil and pile in the passive zone. No gap developed despite quite large rotations and translation. Concentric shear failure planes were observed in the soil behind the pile after large deformations (0.5D). These are also consistent with undrained shear failure rather than tensile cracking.



**Table 5: Comparison of measured and predicted holding capacity for inclined tests**

Test	Trench	Load Inclination [°]	Measured holding capacity, $R_m$ [MN]	Predicted holding capacity from FEA <sup>(1)</sup> , [MN]	Predicted capacity corrected for strain rate effect <sup>(2)</sup> , $R_p$ [MN]	Ratio $R_m/R_p$
C6P1	No	30	6.7	7.7	6.5	1.04
C6P2	Yes	40	5.4	6.2	5.2	1.04
C9P1	No	30	8.0	9.2	7.7	1.04
C9P2	Yes	40	6.3	6.9 <sup>(3)</sup>	5.8	1.09
C10P1	No	35	8.2	6.6	5.5	1.48
C10P2	Yes	35	7.0	6.9	5.8	1.21

Notes: (1) FE holding capacity taken at 0.1D; (2) strain rate correction 0.84; (3) Effective stress soil model



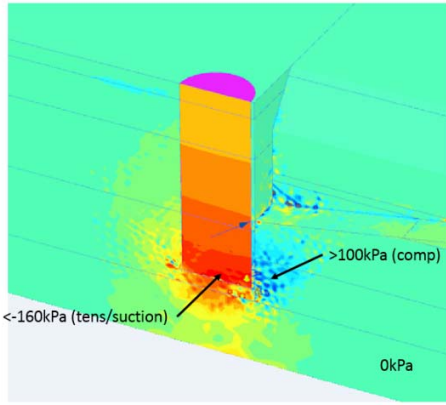
**Figure 5: Backward rotation and failure plane (C9P2)**

The failure mechanism of a caisson anchor in soft clay is dominated by the suction pressures at the rear (which do not allow development of a gap) and below the caisson (i.e. REB). Suction pressures are the undrained response of the soil to the applied total stress changes and can be seen in the FE simulation shown in Figure 6.

**Comparison with other published results**

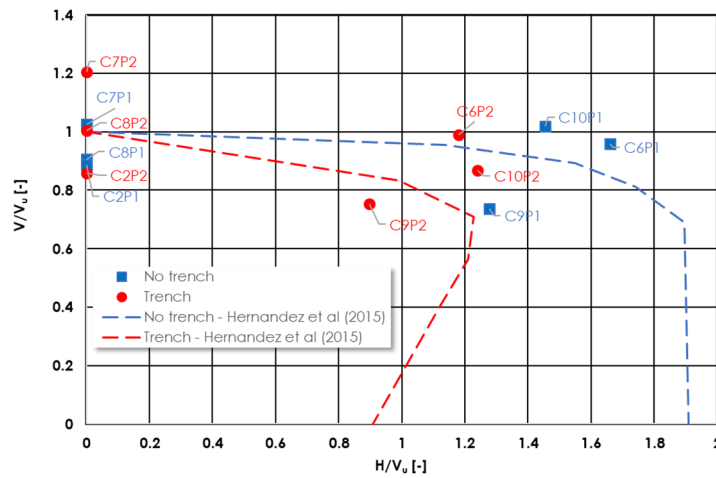
Failure envelopes for similar pile dimensions and in similar soil conditions with and without a trench have been published by Arslan et al., (2015) and Hernandez-Martinez et al., (2015) based on finite element analyses. For comparison with the centrifuge data presented in this report, the calculated envelopes were normalised with their respective values of  $V_u (N_c * S_u)$  where  $S_u$  is the undrained strength at tip level and  $N_c = 10$  based on Martin, 2001) as shown in Figure 7.

The present centrifuge results are broadly consistent with the Hernandez findings although the centrifuge tests suggest a slightly higher holding capacity for the inclined load tests in which 2 of the 3 tests lie outside the respective envelopes.



Analysis performed in with elastic-perfectly plastic Mohr-Coulomb/Tresca model (Plaxis undrained model B simulation in which pore pressure response is based on changes in mean stress only and not from shear-induced tendency to dilate or contract)

**Figure 6: Excess pore water pressures**



**Figure 7: Comparison of centrifuge test results with other published calculated envelopes (normalised  $V/V_u$ ,  $H/V_u$ )**

## CONCLUSIONS

The centrifuge model testing on caisson anchors in clay with a trench has led to the following conclusions:

1. For the anchor and trench geometry studied in this work, the trench in front of the anchor pile leads to a reduction in holding capacity of 15 – 21% for load inclinations between 30° and 40°. Passive suction behind the anchor pile and reverse end bearing (provided by suction) ensures that the loss of capacity is limited.
2. Under vertical load, the trench has very little impact on the holding capacity because the main component of pull-out resistance is the REB. This was confirmed by the suction pressure measurements below the base of the anchor.
3. The failure mechanism for the pile tested was backward rotation combined with forward translation. This mechanism helps to ensure that the suction pressures at the pile top are limited.
4. Detachment was not observed at small and medium pull-out displacements (less than 50% of the diameter). At this deformation, concentric shear failure planes were observed in the soil behind the pile which is consistent with undrained shear failure rather than tensile cracking.

5. Although not presented in detail in this paper, finite element modelling using the undrained shear strength measured in the model soil, with a strain rate correction applied can be used to predict the holding capacity of a pile anchor with a trench. Mohr Coulomb/Tresca criteria with  $\phi=0$  or an effective stress model with undrained conditions and  $c / \phi'$  derived to match the average shear strength profile with depth may be used

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