

# Full scale rapid uplift tests on transmission tower footings

## Tests grandeur nature d'arrachement rapide sur les fondations d'une tour relais

Levy F.M., Richards D.J.

*Geomechanics Research Group, Faculty of Engineering and the Environment, University of Southampton, UK*

**ABSTRACT:** This paper describes results from a series of full scale tests on transmission tower footing at a London clay site in Kent. The testing series investigated the effects of rapid loading at field scale with footings founded directly onto clay or coarse granular material, with a clay or coarse granular material backfill. It was shown that footings founded on London clay mobilised significantly greater uplift capacities at smaller displacements than those with a breakaway condition. In a suite of centrifuge tests this enhanced capacity was shown to originate from suctions that formed on the base of model footings (Lehane et al. 2008). Back analysis of the field tests revealed that inferred normalised suctions were similar to those generated in the centrifuge tests. However the enhanced uplift capacity was not sufficient to cause the uplift resistance to reach design capacity by the displacement serviceability limit state of 10mm.

**RÉSUMÉ :** Cet article présente les résultats issus d'une série de tests grandeur nature sur les fondations d'une tour relais basées sur un terrain en argile de Londres (London clay), dans le Kent, Angleterre. Ces séries de tests ont été menées pour étudier les effets d'un chargement rapide en conditions réelles avec une tour remblayée et fondée dans un sol argileux ou un matériau granuleux. Il a déjà été prouvé que, pour de petits déplacements, des fondations basées sur de l'argile de Londres présentaient une résistance à l'arrachement plus importante que des fondations placées sur une base en gravier. Des essais en centrifugeuse ont montré que cette résistance à l'arrachement accrue provenait de suctions sur la base de la maquette (Lehane et al. 2008). De nouvelles analyses sur le terrain ont révélé que les forces de succion calculées étaient similaires à celles générées durant les tests de centrifugeuses. Cependant cette meilleure résistance à l'arrachement n'a pas été suffisante pour satisfaire la limite de résistance imposée par le cahier des charges pour un déplacement fonctionnel de 10mm.

**KEYWORDS:** clays; footings; foundations.

## 1 INTRODUCTION

In the UK there are 22,000 high voltage transmission tower pylons supported by a pyramid (or pad) and chimney type footing under each tower leg. The majority of these towers have cable bundles (conductors) that are reaching the end of their design life (Clark et al. 2006) and due to demand increases and changing power generation patterns the majority of these cable bundles require upgrading to transmit higher voltages. This will require larger cable bundles increasing the loads transmitted to the tower support foundations.

Recent studies undertaken by Southampton University for National Grid UK have shown that the design basis for transmission tower foundations may not be reliable. The uplift capacity derived from the conventional UK design practice is higher than that predicted by other models (e.g. American IEEE design methods (IEEE 2001)). The ultimate reliability of National Grid foundation systems in terms of their uplift capacity is therefore uncertain, particularly as the imposed loads transmitted to the foundations are likely to increase in both magnitude and frequency as climate change produces more extreme storm/loading events.

However, the failure of tower foundations/footing systems in service is extremely rare suggesting that although the design methodologies may be unsound as they are not based on actual failure mechanisms there are additional factors not considered in the simplified design methods used that are providing increased resistance to uplift, particularly under rapidly applied loading conditions. To examine these issues a series of full scale rapid uplift tests were carried out at the Building Research Establishment's London clay test site at Chattenden, Kent.

### 1.1 UK design and construction practice

The uplift capacity of transmission tower footings in the UK is calculated using the frustum method. This method assumes that there is no transfer of tensile resistance from the soil on the founding plane to the base of the footing and that a breakaway condition exists.

The design uplift resistance of a footing is derived from the footing mass ( $W_f$ ) and the soil mass ( $W_s$ ), contained within an inverted frustum extending to the surface from the base of the footing. The geometry of the frustum is governed by the in-situ soil properties. In accordance with TS 3.4.15 Issue 2 (National Grid 2004) in 'strong' soils ( $SPTN > 20$  or  $s_u > 50$  kN/m<sup>2</sup>) the angle of the failure plane to the vertical (frustum angle) is 25°. In all other cases the frustum angle is set to 15°.

The serviceability limit state (SLS) displacement criterion of shallow foundations is considered to be approximately 10% of footing width ( $B$ ). However the serviceability displacement criterion of individual tower footings is very low to prevent buckling failure of the tower support structure members. Both the UK (National Grid 2004) and the United States (IEEE 2001) design codes set minimal SLS vertical footing displacements ( $w$ ) regardless of footing size – 10mm and 13mm (0.5") respectively. These values are based on the assumption that the lattice tower has enough flexibility to redistribute load as a result of these maximal differential movements but will weaken considerably thereafter.

A truncated pyramid base with an inclined chimney is the most common footing type for lattice transmission towers in the UK. The chimney is constructed from steel reinforced concrete with the reinforcement extending into the base of the pyramid;

the pyramid is constructed of mass concrete. The footing is cast inside a large excavation which is typically backfilled using the excavated or imported material. In each case, the embedment (H) to width (B) ratio is typically between 1 and 1.5.

## 1.2 Uplift resistance of footings

It is conventional to express the uplift capacity ( $Q_u$ ) of a footing as:

$$Q = W_f + q_0 B^2 \quad (1)$$

where

$$q_0 = N_{uc} s_u + \gamma H N_{us} \quad (2)$$

The contributions from any base tension and backfill are represented in Eq. 2 in the form of reverse bearing capacity factors  $N_{uc}$  and  $N_{us}$ , respectively. The value of  $N_{us}$  may be quantified by a number of back analysed failure surfaces (e.g. Murray and Geddes 1987) or through the use of design charts derived from parametric numerical analysis (e.g. Merifield and Sloan 2006).

Early physical testing at quasi static uplift rates investigated the variation  $N_{uc}$  with embedment ratio (H/B). There are therefore many solutions available (e.g. Rao and Datta 2001). However more recent centrifuge studies have shown that  $N_{uc}$  is also dependent on uplift rate ( $v_f$ ) (Lehane et al. 2008). Under rapid loading ( $v_f = 30\text{mm/s}$ ) it was found that a single footing founded on kaolin clay generated more than twice the capacity in comparison to a slow uplift rate ( $v_f = 0.3\text{mm/s}$ ). The difference in uplift capacities between  $v_f = 0.1\text{mm/s}$  and  $v_f = 30\text{mm/s}$  was proportional to the reduction of pore water pressure below the footing base (Lehane et al. 2008). It was proposed that the slow uplift rate allowed suction relief to occur due to the gradual base/soil separation during uplift. This is sufficient to relieve suctions and at approximately  $w/B \geq 6\%$  residual capacity was equivalent to a full breakaway condition.

At very fast uplift rates ( $v_f > 30\text{mm/s}$ ) base separation does not occur due to the full development of suctions that eventually cause a reverse bearing failure to occur in the clay. This type of failure results in a clay wedge remaining adhered to the footing base post-pullout and capacity is determined by the undrained shear strength of the clay (fully bonded).

## 2 FIELD TESTS

The aim of the field tests was to reduce the uncertainty surrounding the in situ performance of transmission tower footings. Reduced scale physical model tests conducted in a geotechnical centrifuge demonstrated that during continuous pullout at increasing velocities that uplift capacity may be significantly enhanced due to the development of suctions occurring across the footing base. It was shown that uplift capacity had a log linear relationship with the uplift velocity (Lehane et al. 2008). The source of this contribution was the formation of negative pore water pressures on the footing base. However it is only at field scale that these effects can be examined and quantified in the context of realistic in situ soil conditions and construction variabilities associated with full scale footings.

To examine these issues a series of full scale tests were commissioned at the Building Research Establishment's London Clay test site at Chattenden, Kent (OS ref: TQ 75521 73987). The field tests aimed to bridge understanding of the load-displacement, load-rate and suction behaviour of soils from small scale and numerical modelling to field scale. By using different construction backfill materials to replicate as-built construction practices, uplift rate and base interfaces across five L4M tower type footings (Footings 1-5) that different uplift mechanisms at full scale could be revealed.

## 2.1 Site layout

Five L4M footings were constructed at the Chattenden site in August 2010. The footing geometry used is shown in Figure 1. The footings were designed and constructed to TS 3.4.15 Issue 2 (National Grid 2004). The design uplift resistance ( $Q_{des}$ ) of the footings using TS 3.4.15 was 420kN based on a 25° frustum angle as  $s_u > 50\text{ kN/m}^2$  on the founding plane (Butcher et al. 2009).

Each footing was installed with different base contact conditions and backfill material (see Figure 1.). These variables allowed the contribution of each resistance mechanism to be isolated. Footings 1 and 2 were backfilled with tumbled and compacted London clay – representing field/early construction practices. Footings 3-4 used DoT Type 2 backfill - a coarse granular material from recycled aggregate (Depart of Transport 2009). All footings were directly cast on the underlying clay apart from Footing 3, which had a Type 2 free drainage layer.

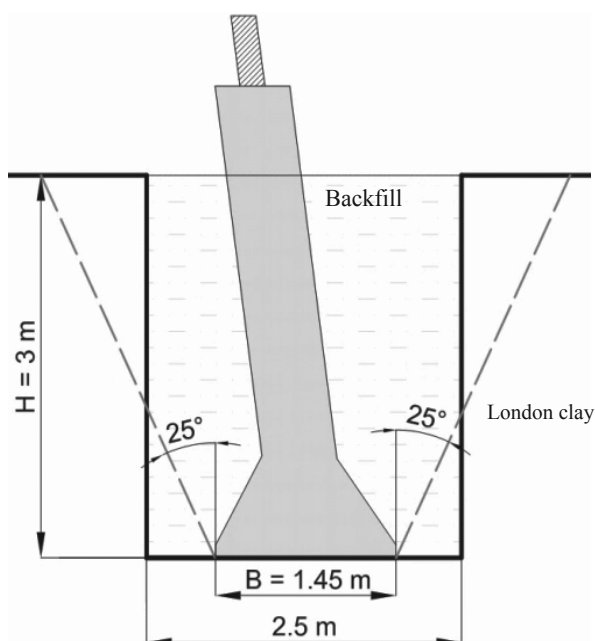


Figure 1. L4M footing with a 25° frustum

Table 1. Footing specifications

Footing	Backfill	Base
1	Loose London clay	London clay
2	Dense London clay	London clay
3	Type 2	Type 2
4	Type 2	London clay
5	Type 2	London clay

## 2.2 Ground conditions

The Chattenden site has been used extensively for foundation testing due to the presence of the deep and uniform London clay strata (e.g. Butcher et al. 2009). The depth of the London clay strata is ~30m and it was evident that during the construction of the footings that the top 3m was heavily weathered and fissured. The foundation tests were conducted over a two week period in July 2012. The extremely wet summer of 2012, particularly in the weeks prior to the field tests resulted in the top layer of weathered clay becoming soft ( $\sim s_u = 10\text{kPa}$ ). It was also evident that the excavations backfilled

with granular material were fully saturated leading to extreme softening (swelling) of the clay in the base of the excavations.

A total of five ~10m deep CPTs were used to characterise the site and backfill. Using a  $N_{kt} = 20$  (Butcher et al. 2009) the variation of  $s_u$  and density ( $\gamma$ ) of the London clay below 3m (the founding plane) corroborates with previous observations. A profile from Footing 5 is shown in Figure 2.

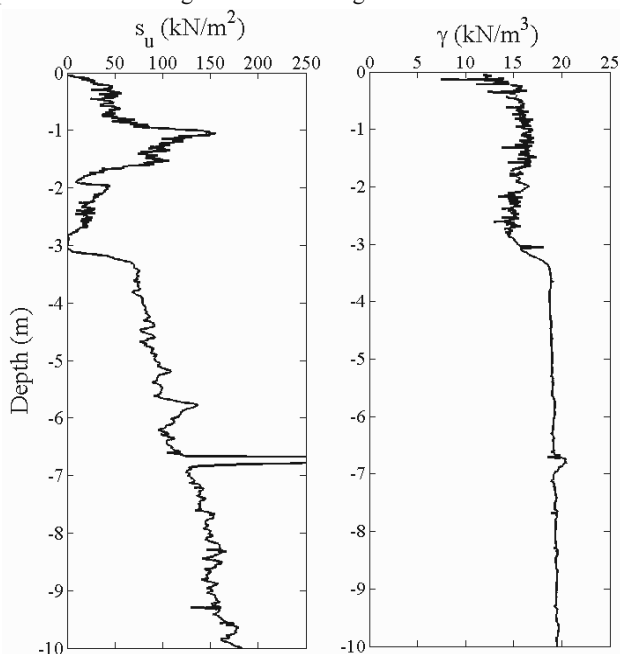


Figure 2. CPT profile of Footing 5

### 2.3 Load schedule

The footings tests were carried out over a period of six working days (18<sup>th</sup> - 25<sup>th</sup> July 2012). Table 3.2 shows the order with which the tests were carried out. The small displacement tests (denoted 'A') of Footings 2 and 3 examined the rate effect at small displacements in the fully bonded and breakaway conditions. The first load test on Footing 4 was a design test to BS EN61773:1997 (BSI 1997).

The load was applied to the footing stubs using a hydraulic jack system. The setup of the reaction beams and jack is shown in Figure 3. The reaction beams at the base were orientated parallel to the line of the excavations and outside the failure zone of a 30° frustum. The cross beams with the hydraulic jack were inclined so that the footings could be pulled up in line with the footing chimney. Wedges were placed under the cross beam to achieve the required rake angles. The hydraulic ram had a total stroke length of 150mm ( $w/B=10\%$ ).

The resistance of the footing during uplift was measured using a load cell mounted above the hydraulic jack. The displacement of the footing was measured by mounting LVDTs on a reference beam. The LVDTs were vertically orientated and recorded the movement from the head of the chimney. The data from the instrumentation was acquired using a Campbell CR5000 data logger sampling at 10-100s/s.

## 3 FIELD TEST RESULTS

### 3.1 Load-displacement behavior

The load-displacement profiles of the tests conducted with a London clay base displayed an extremely stiff response (see Test 1-A and 5-A).

The loading rates during the first few millimetres of movement was in the range 276 kN/mm (Test 2-A) – 333 kN/mm (Test 1-A). The peak capacities of the footings were achieved between  $w/B = 2.1 - 4.7\%$  versus the secondary test which were generally in excess of 7%. Although the Tests 1-A, 2-A and 5-A were conducted at different uplift velocities this

did not affect uplift capacity. The measured uplift capacities were similar for both tests (<10% difference) during the first 10mm of uplift and at peak there is a difference of 50 kN versus Test 1-A and 5-A.

Test 3-A was used to examine the difference that base contact conditions at small displacements would have. Although the test did not reach its peak the stiffness response is comparable with Footing 2-A. This implies that the base condition contribution was similar to the footings tested on London clay and that the blinding did not fulfill its purpose of excluding suctions from uplift i.e. acted a free draining material.

Tests 3-B and 4-B indicated that the performance of Type 2 granular fill is extremely poor and could only mobilise ~50% of  $Q_{des}$  at  $w=10\text{mm}$ . Large displacements were required for the granular fill to mobilise sufficient strength to produce uplift capacities equivalent to  $Q_{des}$  ( $w = 60\text{mm}$  Test 3-B and  $w = 80\text{mm}$  Test 4-B). It should be noted on Test 5-A reached  $Q_{des}$  within a 10mm displacement.



Figure 3. Field test arrangement

Table 2. Load schedule

Footing	Test	$w$ (mm)	$v_f$ (mm/s)
1	1-A	150	35
2	2-A	15	5
2	2-B	150	15
3	3-A	15	15
3	3-B	150	10
4	4-A	BS EN61773:1997	
4	4-B	150	15
5	5-A	150	35

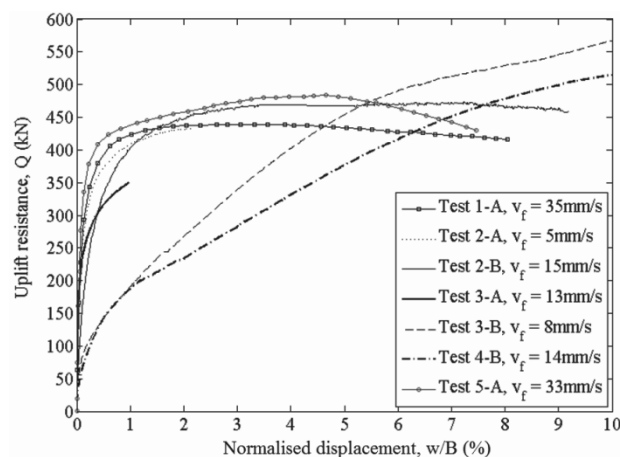


Figure 4. Load-displacement results for rapid uplift footings

### 3.2 Suction

In previous studies (e.g. Lehane et al. 2008) the degree of uplift capacity enhancement may be estimated as the difference between the suction and breakaway uplift resistances (Eq. 3). When the uplift capacities are evaluated using the slope method (BSI 1997) the average enhancement due to suction is 325 kN. This is equivalent to a pore water pressure drop of 155 kPa from hydrostatic. The magnitude of pore water pressure drop is similar to the values of pore water pressure measurements observed during undrained uplift of model footings (Lehane et al. 2008).

$$N_{uc} = (Q - Q_{br})/s_u B^2 \quad (3)$$

The effects of suction may be presented using a normalised velocity ( $V = v_f B/c_v$ ) where  $c_v$  is the coefficient of consolidation of the underlying London clay (0.24, Skempton and Henkel 1957). The results from Lehane et al. (2008) are presented in this manner against the undrained bearing coefficient  $N_{uc}$ .  $Q_{br}$  was defined as the average uplift capacity values for second tests on Footing 3 and 4 using the slope method with  $Q$  the uplift resistance of the remainder.

Figure 5. shows the similarity between the results of the model tests at  $v_f = 30$  mm/s and the field tests results on London clay. The range of  $N_{uc}$  of the tests on London clay is between 3.3 - 3.9 compared to 3.7 - 4.1 for  $v_f = 30$  mm/s. The data point at  $N_{uc} = 1.9$  corresponds to Test 3-A, which did not reach peak but evidently mobilised a degree of suction.

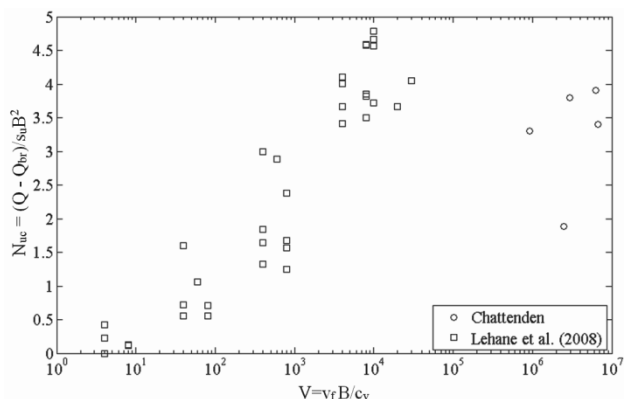


Figure 5.  $N_{uc}$  from tests on transmission tower footings

### 4 CONCLUSIONS

The series of field tests on a number of full scale L4M footings has confirmed that base suction may contribute significantly to footing performance. Preliminary analysis has shown that the magnitude of suction developed is similar to that observed in physical model tests conducted in a centrifuge.

The results have also shown that the design uplift performance is not reached (in general) before the ultimate limit state displacement criterion set by UK design guidance. This includes the performance of footings where suction developed. In the case where suctions did not develop, the uplift performance of the footings was extremely poor. Such a poor performance will require a re-evaluation of the use coarse granular material, specifically Type 2, when used in excavations bounded by London clay.

### 5 ACKNOWLEDGMENTS

The authors wish to thank National Grid UK for the financial support required to conduct the field tests. The authors gratefully acknowledge the work of Grid Line Foundations Ltd, Andrew Hewitt of Lankelma and PMC.

### 6 REFERENCES

BSI 1997. *Overhead lines - Testing of foundations for structures*, BS EN 61773:1997, London.

Butcher, A.P. et al. 2009. Comparison of behaviour of CFA piles in London clay as determined by static, dynamic and rapid testing methods. In *5th International Symposium on Deep Foundations on Bored and Auger Piles*. London, 205-212.

Department of Transport, 2009. *MCHW Volume 1 SHW: Roads pavements - Unbound cement and other hydraulically bound mixtures*. London

IEEE, 2001 *Guide for transmission structures: Foundation design and testing, IEEE Std 691-2001*, New York.

Lehane, B.M., Gaudine C., Richards, D.J. and Rattley, M.J. 2008. Rate effects on the vertical uplift capacity of footings founded in clay. *Géotechnique* 58(1), 13-21.

Merifield, R.S. and Sloan, S.W. 2006. The ultimate pullout capacity of anchors in frictional soil. *Canadian Geotechnical Journal* 43(8), 852-868.

Murray, E.J. and Geddes, J.D. 1987. Uplift of anchor plates in sand. *Journal Of Geotechnical Engineering* 113(3), 202-215.

National Grid 2004. *Overhead line support foundations, TS 3.4.15 Issue 2*. Warwick, UK.

Rao, N.S. and Datta, M. 2001. A comparison of uplift and bearing behaviour of plate anchors in soft clay. In *Eleventh International Symposium Offshore and Polar Engineering Conference*. Stavanger. 560-565.

Skempton, A.W. and Henkel, D.J. 1957. Tests on London clay from deep borings at Paddington, Victoria and the South Bank. In *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering*. London, 100-106.