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# Pile aging in cohesive soils

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## **Abstract:**

This technical note presents the results of a field investigation into pile aging in soft clay which was conducted over a period of ten years. Static load tests were conducted after the excess pore pressure generated by the installation of 6 m long driven concrete piles were fully equalised. These tests allowed the time-capacity aging profile to be established. A normalised capacity-time trend established for the case history is seen to be consistent with the response observed from a wider database of pile tests in clay compiled from the literature. A simple reliability based design example is provided to highlight the positive impact that pile aging could have for industrial practice.

## 1. Introduction

The tendency for the axial capacity of piles installed in clay to increase with time has been observed for over a century. Wendel (1900) described timber piles which exhibited strength gains for two to three weeks after driving. Set-up is commonly observed during pile installation where a delay in driving can lead to substantially higher blow counts when driving recommences (e.g. Fox et al, 1976). Consolidation theory suggests that dissipation of pore water pressure set-up during pile installation would lead to increased effective stress, and hence higher pile resistance. It is common practice to allow sufficient time between pile installation and load testing to allow dissipation of these excess pore pressures. However, a number of case histories reported in the literature suggest increases in pile capacity in both sand and clay soils after pore pressure equalisation has occurred (See Huang 1988, Pestana et al. 2002, Chow et al. 1998, Axelsson 2000, Bullock et al 2005 and others).

Skov & Denver (1988) describe a case history of the installation of 250 mm square pre-cast concrete piles, driven 22 m into a primarily cohesive deposit at Aalborg, Denmark. They measured the variation of the pile load resistance by performing periodic dynamic and one static pile load test at intervals after driving. They found that the increase in pile capacity followed a logarithmic trend with time and suggested an empirical correlation to describe aging:

$$[1] \quad \frac{Q_t}{Q_0} = 1 + A \log \frac{t}{t_0}$$

Where:  $Q_t$  is the capacity at time  $t$  after driving,  $Q_0$  is the reference capacity at time  $t_0$ , and  $A$  is a constant which varies with soil type. The authors suggested values for  $t_0$  and  $A$  of 1 day and 0.6 respectively. Since pore pressure equalisation was unlikely to have been complete one day after the installation of their piles, the increase in pile capacity described using these suggested values of  $t_0$  and  $A$  include both the effects of consolidation and aging. A number of workers have proposed similar correlations (See Guang-Yu 1998, Huang 1988 and others). However, due to a combination of factors which include combining set-up effects and aging (i.e. choosing a  $t_0$  value which is lower than the time taken for consolidation to occur), using data from both static and dynamic tests (which are not directly comparable, particularly when performed at different times) and including data from sites with variable ground conditions, the measured data tend to exhibit significant scatter.

A series of static load tests performed over a ten year period on piles installed in soft clay located in Belfast are presented in this note. The test piles of identical geometry were installed in a well characterised geotechnical test bed facility and statically load tested after full equalisation of pore water pressure. A database of case histories for pile installed in clay was assembled and the trends for aging observed from the Belfast tests were compared to piles installed in soils with a range of

strengths. In the final section of this technical note, a reliability based framework to consider aging is proposed.

## **2. Case History – Belfast Field Tests**

### **2.1 Test Site**

The pile tests described in this note were conducted at the Kinnegar soft clay research site in Belfast harbour. The soil stratification at the site comprises three distinct soil layers, a layer of fill typically 1m thick, which overlies approximately 7.5m of soft silty clay, which is underlain by medium dense sand at approximately 8.5 m depth. The silty clay deposit is known locally as *sleech*. The geotechnical properties of *sleech* have been described in detail by Doherty (2010). A brief summary of the typical geotechnical properties are given in Table 1.

### **2.2 Full Scale Load Tests:**

An experimental program designed to assess the behaviour of axially loaded single piles and pile groups in Belfast *sleech* was initiated in 1997, See McCabe and Lehane (2006). The 250 mm square, 6 m concrete piles were installed using light driving to a final penetration depth of 6m. The results of load tests on three single piles known as piles S1 to S3 (See Table 2) and one pile installed at the centre of a pile group, G1 (which was loaded as a single pile) are described in this note. The minimum distance between piles in the pile group was 0.7m between pile centres. Vibrating wire piezometers installed in the *sleech* deposit at radial distances of 300mm to 3m from the shaft of S1, allowed the dissipation of excess pore pressure to be measured after pile installation of the 250 mm square test piles, (See figure 1). After a period of 41 days, the average excess pore pressure from from all piezometers had reduced by 80%. Measurements made using pore pressure sensors on the shaft of model 73 mm and 168 mm diameter piles installed at the Belfast test site (See Gavin et al. 2010 and Doherty and Gavin 2012) allowed the development of a normalised pore pressure dissipation curves for soil at the pile-soil interface. This suggested that full pore pressure equalisation of the concrete test piles occurred between 60 and 74 days after installation, based on the measured coefficient of consolidation in the horizontal direction,  $c_h$ , that ranged from 7 m<sup>2</sup>/year to 12 m<sup>2</sup>/year.

This case history considers the load-displacement response of piles at various times after installation. The load tests were performed using a 20 tonne Cone Penetration Test (CPT) rig for reaction. The tests were carried out in tension with loading increments ranging from 5-10 % of the expected ultimate load. Each increment was maintained until the creep rate had reduced to negligible levels (<0.004 mm/min). The load tests were conducted until either ultimate conditions (defined as no

increase of load for increase of displacement) or a pile head displacement of 28mm displacement (equivalent pile diameter) had been achieved.

The reload tests which were performed ten years after pile installation were conducted on one single pile and the centre pile of a pile group (after the pile cap had been removed allowing loading as a single pile). Complete details of the pile installation and load test sequence for all piles can be found in Doherty (2010) and McCabe (2002). The effect of pile ageing is clearly evident in Figure 2, which compares the load-displacement response of a fully equalised pile (S4) tested 142 days after installation, with a re-load test performed on the same pile 3086 days after installation. Whilst the initial stiffness response of the two piles is similar, the aged pile exhibited a higher capacity, mobilising an ultimate resistance of 95 kN. This represented a 48% increase on the axial load response measured after 142 days. A second load test performed on the centre pile of a group (SG1) yielded an ultimate capacity of 91 kN which was 40% higher than the axial resistance of a virgin pile tested after 104 days.

The ultimate shaft capacity developed for all the concrete test piles tested at the Belfast site are shown in Figure 3. A clear trend for the shaft capacity to increase with time is evident. The data include tests performed on both compression and tension piles (see Table 2), which were seen (at a given time) to have equal resistance in tension and compression when the effects of self-weight were corrected. A log-linear regression provides a reasonable fit to the measured time-capacity data. The shaft capacity is seen to increase by 16.5 % per logarithmic increase in time (in days), over the range of times considered in this study. Adopting a reference time ( $t_0$ ) of 100 days to indicate a representative time for complete equalization of full-scale displacement piles results in a set-up coefficient,  $A$  of 0.26, as per Equation 1.

### **Database Study**

A database of nine case histories which recorded pile ageing in clay was assembled, (see Table 2). The pile diameters considered ranged from 100 mm up to 762 mm, and the pile lengths from 5 m to 70 m. The load tests in the database were all undertaken at sufficient times after installation to allow complete dissipation of the excess pore pressures, as checked by instrumentation or site permeability data. The studies largely reported multiple load tests performed on a single pile and therefore most had experienced complex loading histories. A notable exception was the pile tests performed in Haga clay reported by Karlsrud and Haugen (1986), where both virgin piles and multiple load tests on previously failed piles were undertaken. The piles in Haga clay demonstrated a pronounced dependence on the shearing history, with presheared piles undergoing a much higher capacity increase with time. There is insufficient evidence available in the literature to assess whether the impact of preshearing is as pronounced at other sites and therefore only the first time load tests from Haga have

been considered in this study. Additional details regarding the loading history of these piles and the database in general can be found in Doherty (2010).

### **Statistical Analysis**

The variation with time in the total capacity of the database piles is shown in Figure 4. A regression analysis at each site suggested that a reference time of 100 days should be used to establish the reference capacity,  $Q_0$ . The normalised capacity is observed to increase steadily with time and exhibits a mean relationship given by equation 2.

$$Q/Q_0 = 1 + 0.25 \log(t/100) \quad \text{Eqn 2}$$

Figure 4 also shows the variation about the mean, as lines corresponding to +/- one standard deviation. The standard deviation,  $\sigma$ , of 0.085 suggests a reasonable fit between the mean trend and the measured data. The Belfast pile tests are illustrated as a separate series in Figure 4, and are clearly representative of the global database trend adding confidence to the general applicability of these findings. However, the limited extents of the database should be considered before adopting this trend directly into routine design practice.

The measured capacities are compared to the normal probability distribution in Figure 5, which shows a reasonable fit to the data.

The Kolmogorov-Smirnov test was used to assess the validity of a normal distribution using the cumulative frequencies shown in Figure 6. The maximum difference between the theoretical normal distribution and the experimental cumulative frequency plot is 0.07. This is significantly less than the required critical value of 0.16 for this sample size at the 5% significance level, thus confirming the validity of the normal distribution to describe the variation between the calculated to measured set-up capacities. These statistical results compare favourably to previous database studies that yielded large degrees of scatter in the fitted time-capacity relationship. The statistics can be used within a reliability based design framework, as illustrated by the following simple example.

### **Reliability Based Design Example**

In this example, the equation proposed to quantify the effects of pile ageing is incorporated into a simple reliability based framework to quantify the positive impact of ageing on foundation safety, as captured using a reliability index. The first step in the design process is to estimate the fully equalised

capacity using for example a static design formula. The Imperial College method (ICP-05) which is described in detail by Jardine et al (2005) is ideal, as it was developed using equalised radial effective stresses measured on instrumented piles installed at a number of well characterised clay sites. The next step is to use radial consolidation solutions (eg. Randolph, 2003) to estimate the equalisation time, which takes into account the pile diameter, soil stiffness and soil permeability. A rational initial time,  $t_0$  and the corresponding capacity  $Q_0$ , can thus be calculated from the ICP-05 method. Equation 2 can then be applied to the initial capacity to determine the aged pile capacity at any time following equalisation. This is of particular importance where (i) a structure will be subject to higher loads in the future or (ii) where a structure may require a higher degree of safety in the future.

The uncertainty in the capacity-time trend can be incorporated into the analysis using a simplified reliability based approach, such as a First Order Second Moment (FOSM) method (eg Baecher and Christain, 2003). This approach allows both the uncertainty in the original pile design model and the uncertainty in the pile aging process to be considered. The predictive accuracy of the Imperial College method was assessed using a load test database and found to have a reasonable predictive performance with a mean calculated to measured capacity of 1.01 and a coefficient of variation (COV) of 0.2 (Jardine et al, 2005). Incorporating these statistics with the COV of 8.5% established for the capacity-time trend and assuming a 10% COV for the applied load, the effect of pile aging on the reliability index,  $\beta$  can be calculated for a range of assumed factors of safety (FOS):

$$\beta = \frac{E[FOS]-1}{\sigma[FOS]} \quad \text{Eqn} \quad 3$$

Where  $E(FOS)$  and  $\sigma[FOS]$  are the mean value and standard deviation of the FOS.

The calculated reliability index shown in Figure 7 was found to increase dramatically with time, with the most significant improvements in capacity occurring for low initial factors of safety. The rate of increase in the reliability index was observed to be most dramatic immediately after the end of consolidation, with the relative gain in safety reducing with time. An analysis similar to the one described could offer significant competitive advantages for projects where the pile foundations can be installed well in advance of the loading phase or in cases where existing piles can be reused. The inherent positive bias offered by the capacity-time relationship could act as a buffer against future loading conditions and suggests that existing design methods may be conservative or at least not designed for optimum efficiency. However, the authors would like to highlight the limited amount of data available to determine the proposed time-capacity trend and would suggest that additional long-term pile load test information is needed to confirm this trend before it is incorporated into routine

design practice. An additional caveat with the available database is that insufficient load-test information is available to directly assess the impact of cyclic loading on the aging response.

## **Conclusions**

This note investigated the aged reload response of piles in clay. A case study of piles installed in soft clay indicated that the fully equalised shaft resistance of the pile increased by at least 40% over a ten year period. Similar trends were observed from a database study of piles installed in a range of clay strengths. A normalized capacity-time relationship was established to quantify the aging process. The positive effect of set-up was incorporated into a reliability based framework, which highlighted the potential benefit of considering aging in the design process.

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**Table 1: Soil Properties of Belfast Sleaford**

Bulk Unit Weight (kN/m <sup>3</sup> )	16.1±0.3
Clay Fraction (%)	20±10
Silt Fraction (%)	70±10
Moisture Content (%)	60±10
Plastic Limit (%)	30±5
Liquid Limit (%)	65±10
Organic Content (%)	11±1
Vane Shear Strength (kPa)	22±2
Yield Stress Ratio	1.6-1.0
Triaxial Peak Friction angle (°)	33±1
Coefficient of Consolidation, c <sub>v</sub> (m <sup>2</sup> /year)	0.5-3
Coefficient of Permeability(x10 <sup>-10</sup> ), k (m/s)	1.5-5
G <sub>max</sub> (MPa)	11±1

**Table 2: Full Scale Load Test Details**

Test #	Days since Installation	Additional details
S1/T	99	Virgin pile
S2/T	104	Virgin pile

S2/TT	251	Reload test
S3/C	82	Vibrating wire Strain gauges allow separation of base and shaft loads
S4/cyT	142	Static Test conducted immediately following unidirectional tension cyclic loading
S4/cyTT	3086	10 year reload test
G1/C	98	Group Pile load test not used in assessing time characteristics
G1cr/CT	3683	10 year reload test on Centre Pile of group
T=Tension, C=Compression, cy=cyclic, G=Group,		
cr=centre pile		

**Table 3: Summary of Database Details**

Site	D (mm)	Tip Depth (m)	No. of load Tests	Test Times	Reference
Belfast	250	6	7	82-3683	Present Study
Cowden	457	9.14	3	30-9125	Gallagher & St John (1980) Powell et al (2003)
Houston	273	13.1	3	18-108	O'Neill et al (1982a, 1982b)
Drammen	250	15.5	3	31-799	Eide et al (1961)
Canons Park	168	6.5-6.65	15	74-6200	Powell et al. (2003), Bond and Jardine (1991, 1995), Wardle et al. (1992)
Bothkennar	101	6	2	4-32	Lehane (1992)
Nitsund	180	11.7	10	32-1043	Flaate (1972)
Ska-Edeby	100	14.5-15.3	25	30-1116	Bergdahl&Hult (1981)
Haga	153	5	3	7-36	Karlsruud and Haugen (1986)

**Figure Captions:**

Figure 1: Dissipation of average excess pore pressure measured by a piezometer in the soil adjacent to the test pile

Figure 2: Reload Test on Single Pile compared to previous static load test

Figure 3: Fully Equalized Time-Capacity Characteristics of Concrete Piles in Belfast

Figure 4: Normalised Capacity-Time Relationship for database of pile tests

Figure 5: Comparison between experimental histogram and theoretical normal distribution

Figure 6: Cumulative Frequencies used in Komogorov-Smirnov Test

Figure 7: Reliability analysis considering long term variation in pile resistance