Resistance factors for high-strain dynamic testing regarding German application of Eurocode 7 and correlation of dynamic and static pile tests

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ABSTRACT: In EC7 dynamic pile load tests are accepted if calibrated by a static load test. The German national application document of EC7 defines safety factors (correlation factors) that reflect the source of the static test results – same site, similar site or experience. It is tacitly assumed that there is a common understanding of calibration as a straight forward procedure where the static ultimate resistance by dynamic load test is transferred (at best by multiplication by a factor) into an ultimate resistance as would be determined by a static test. However in actual pile testing practice the conditions for this calibration procedure are not given in many cases.

The results from several case histories of comparison of results from dynamic to static load tests will be discussed. On the basis of selected examples the difficulties of this calibration procedure will be shown and requirements for calibration will be defined. Important boundary conditions such as test sequence, tests of same or different piles, soil and loading conditions need to be considered and are discussed.

1 INTRODUCTION

In 2011 the German Institute of Standards (DIN) published the 'Handbook Eurocode 7 - Geotechnical design – Part 1: General rules' which combines DIN EN 1997-1:2009, DIN EN 1997-1/ NA:2010 (national appendix) and DIN 1054:2010 (National Application Document to DIN EN 1997-1,) to one work of German rules in geotechnical design (Handbook EC 7-1, 2011). The national German recommendations ("EA-Pfähle", EAPfaehle 2012), which have the state of a code of practice, also refer to the determination of ultimate capacities by dynamic load tests.

As many other codes the EC7 incorporates the loads and resistance factor design (LRFD). As a general model a load factor multiplies the variable load by γ_Q = 1,5 and the and permanent loads by γ_G = 1,35 to define a factored action. The resistance is divided by the resistance factor.

For piles the resistance factor is a combination of a partial safety factor for resistance γ_R and a correlation factor ξ_i. The resistance γ_R has to be chosen with respect to the method of determination of pile resistances, i.e. experience, static pile load tests or dynamic pile load tests. The correlation factor ξ_i is defined for load tests, different for static and dynamic and is chosen according to the number of tests executed.

Dynamic pile load tests are allowed for the verification of pile capacities in several codes around the world. In EC7 it is required that the dynamic load tests are calibrated against static load tests. However in EC7 it is not clearly defined how a calibration is carried out as can be understood from the respective formulations.

EC7

7.5.3 Dynamic load tests

(1) Dynamic load tests may be used to estimate the compressive resistance provided an adequate site investigation has been carried out and the method has been calibrated against static load tests on the same type of pile, of similar length and cross-section, and in comparable soil conditions, (see 7.6.2.4 to 7.6.2.6).

7.6.2.4 Ultimate compressive resistance from dynamic impact tests

(1)P Where a dynamic impact (hammer blow) pile test [measurement of strain and acceleration versus time during the impact event (see 7.5.3(1))] is used to assess the resistance of individual compression piles, the validity of the result shall have been demonstrated by previous evidence of acceptable performance in static load tests on the same pile type of similar length and cross-section and in similar ground conditions.

For dynamic load tests the evaluation method (driving formulas, wave equation based formulas, full modeling) has an influence on the accuracy of the determined resistance and is taken account of by the multiplier η_D , called model factor.

In the German Application Document to EC7 (DIN 1054:2010 – National Application Document) correlation factors have to be increased by an added penalty with respect to the availability of static tests:

$$\xi_i = (\xi_{o,i} + \Delta\xi) \cdot \eta_D$$

where
ξ_i is the combined correlation factor
$\xi_{o,i}$ is the correlation factor according to EC7 to be chosen with respect to the number of piles tested
$\Delta\xi$ is the calibration penalty according to the German application document
η_D is the model factor according to EC7

The penalty $\Delta\xi$ is defined as follows:

– $\Delta\xi = 0$: is applied if the static load tests have been executed on the same site;
– $\Delta\xi = 0{,}10$: is applied if the static load tests have been executed on comparable piles at comparable soil conditions;
– $\Delta\xi = 0{,}40$: is applied if the equivalence of dynamic and static load tests is validated from experience [cf. EAPFaehle 2012]. In this case only signal matching procedures are allowed. Wave equation approaches like CASE-formula are not accepted.

Equivalent global safety factors for piles under compression can thus vary from 1,6 to 2,6 depending on the number of piles tested, the testing and the evaluation method, and the availability of static test results for calibration.

Because there is no explanation with respect to the procedure of calibration it is concluded that the requirement for calibration is stated because the community of foundation engineers has a deep rooted scepticism against dynamic pile testing in general.

As the development of EC7 was started around 1990 it can be assumed that the attitude against dynamic load test is based on the uncertainties that are connected with the application of the CASE method and the use of empirical damping factors (see e.g. Goble e.al. 1980).

With the development of the processing speed of electronic calculation in the past 20 years the signal matching (or full modeling or systems identification) procedure developed as the acknowledged evaluation standard.

The signal matching procedure is capable of finding a model of the pile in the soil that allows the reliable determination of pile behavior under static load application. Therefore the equivalence of dynamic and static pile tests is not to be verified because of the use of empirical data but only because of the different behavior of piles in the different soils.

However this status is not generally known. Thus it must be assumed that the need for calibration is stated in EC7 because:

- static capacities derived from dynamic tests are taken as not being equally reliable as from static tests,
- static capacities from dynamic tests are assumed non-conservative, i.e. too large,
- settlements from dynamic load tests are assumed non-conservative, i.e. too small.

Whereas Likins et al. (1996, 2004), Rausche et al. (2008) and other writers have presented several studies on the correlation of static and dynamic piles tests and demonstrated the conditions under which the equivalence can be verified in general overviews here the problem of calibration is discussed by a set of examples of load settlement curves.

2 CALIBRATION EXAMPLES

2.1 Cohesive soils

The requirement for calibration can be recognized by the comparison of load settlement curves of static and dynamic load tests in cohesive soils.

In Figure 1 load settlement curves of a set of small diameter CFA piles have been installed in soft clay are shown. VBS1 with length 15,10 m has been tested under static maintained load, piles VBS2 and VBS3 have been tested dynamically by means of a 10 t drop weight with small stroke. By fig.1 it can be seen that the settlements increase under static load with time for a maintained load at loading level 650 kN and 750 kN where extensive creep indicates that the ultimate capacity was reached. If the time dependent settlements are removed (red line) it can be seen that the load settlement behavior is similar to the load settlement determined from the dynamic load test.

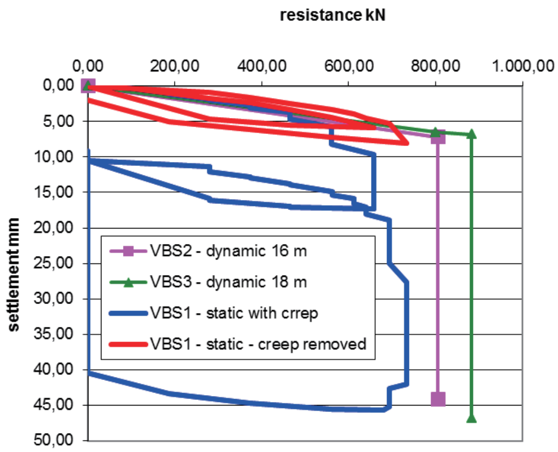

Figure 1. Cast in place piles in soft clays (Seeton).

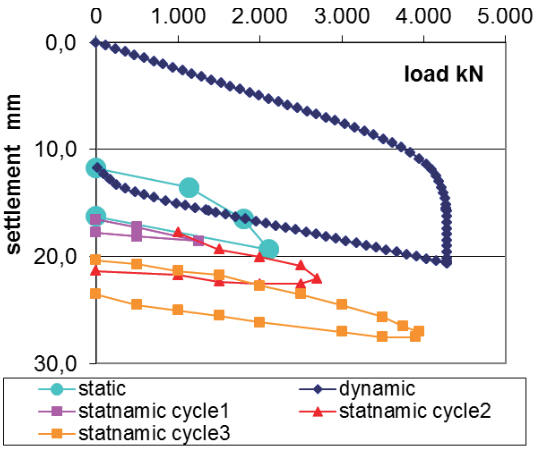

Figure 2. Cast in place piles in till (Mergel).

In Figure 2 a load settlement curve for a pile with diameter 62 cm in a stiff clay (till, or Mergel) is given. The dynamic load test by a drop mass of 7 t and a drop height of 2 m produced a resistance larger than 4 MN at a settlement of 10 mm. After the dynamic load test a static load test was executed to twice the working load, i.e. 2 MN. As can be seen for this load level nearly 10 mm of settlement occurred. In a subsequent rapid load test shooting up a 20 t mass applying 3 load cycles the pile behavior was found similar to the dynamic impact test.

In both cases the calibration cannot be done directly because in the case of fig. 1 the piles are not of the same length and in the case of fig. 2 the static test did not reach the ultimate resistance but was stopped. Maybe a static test to a settlement of 10% of the diameter (62 mm) according to German definition of the ultimate capacity would result in a larger capacity.

2.2 Non-cohesive soils

The load-settlement-curves of fig. 3 (precast concrete) and fig. 4 (closed steel pipes) refer to dynamic and static load tests in medium to dense sand. In both cases the static tests have been executed several weeks after the dynamic tests. In this additional waiting time the piles could freeze in. Therefore the load settlements curves from static tests show a stiffer behavior than from dynamic tests.

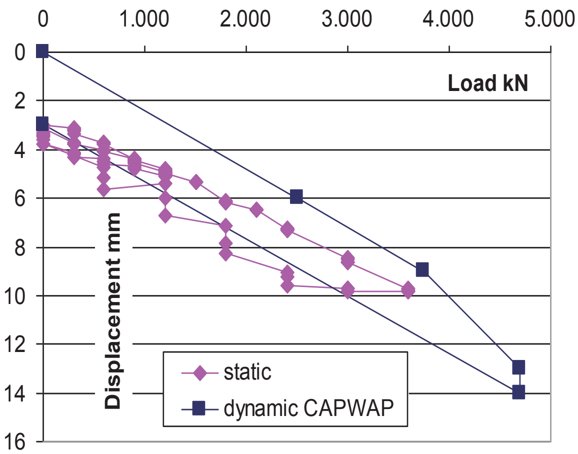

Figure 3. Precast concrete pile in sand.

For the steel piles (Fig.4) the 6 test piles have been tested at end of initial driving (EOI denoted e.g. dyn-a0-p1- “1”) and at begin of rediving (BOR e.g. denoted dyn-a0-p1- “2”). It can be seen that most of the static resistances from dynamic tests are smaller than the resistance from static test. The forces of the jacks for the static tests are defined by the test load to be applied whereas the activated static resistance from dynamic tests depends on the energy applied and the dynamic forces. If a driving rig is designed for achieving the predefined depth against the resistance during driving it may be not strong enough to activate the ultimate resistance in restrike after a certain waiting time.

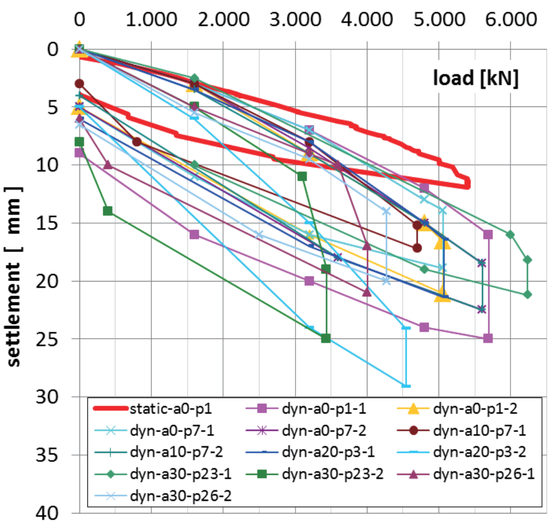

Figure 4. Closed steel pipes in sand.

This insufficiency of driving rigs is even more obvious in example Fig. 5 for a sheet pile in weathered sandstone and Fig. 6 for a cast in place concrete pile Ø 1.200 mm length L = 15,5 m in stiff clay.

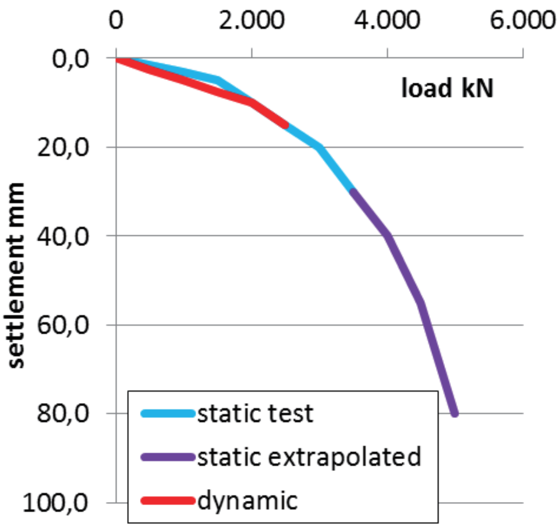

Figure 5. Sheet pile in wheathered sandstone.

The sheet pile of length 14 m was designed for vertical axial load transfer and a static test was carried out for 3.500 kN. For the verification of the limit state of ultimate capacity the load settlement curve was extrapolated to 80 mm set and 5.000 kN by hyperbolic approximation. As the sheet piles have been driven by a Diesel hammer with a ram of 2,5 t mass dynamic load tests have been executed.

However at a blowcount of 100 to 150 blows per meter in redriving it was not possible to activate the resistance as determined in the static test.

This was also the case in the testing of the cast-in-place pile. A 12 t drop weight was used and was not able to activate higher static resistance than shown.

In these cases the comparison shows an equivalence of settlements at working load level – a calibration by scaling for the verification of ultimate capacities does not make sense.

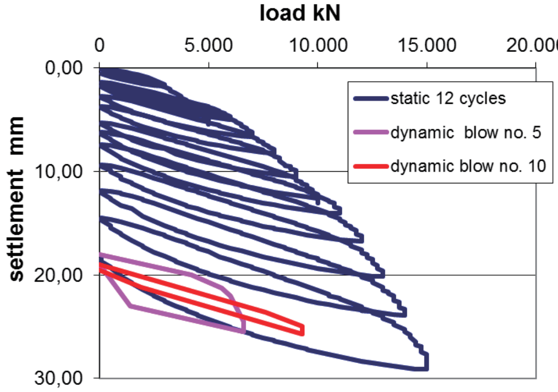

Figure 6. Cast-in-place pile in stiff clayey sand.

2.3 Open ended tubes in cohesive soils

Open ended tubes D = 900 mm have been driven into a soil described as clayey sand. At the end of initial driving (EOI) a resistance of around 2 MN was determined (see light blue line in fig 7). In a redriving test 4 days later a resistance of 4,5 MN could be activated in the beginning of the redriving BOR, yellow line in Fig. 7). After the application of about 50 blows the resistance was reduced to 3 MN (red line in Fig. 7).

In a static load test 2 weeks later a resistance of 2.700 kN has been determined (see Fig. 8). As can be seen by the load settlement curve after a limit resistance was achieved the pile is plunging down. Obviously the open ended pile was cutting into the clayey sand and settlement increased with time. If time dependent settlements are removed this load settlement curve is comparable to dynamic resistance at end of driving (see green line in Fig. 7).

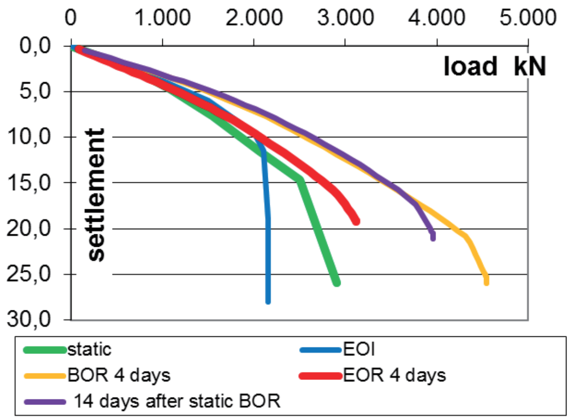

Figure 7. Comparison of load settlement curves in clayey sand - static load settlement without creep.

At a second redrive 14 days after the static test a resistance of 4 MN was confirmed in the first blows.

An indication of this different behavior of piles in cohesive soils under static and dynamic loading may be recognized if the development of resistances with depth is considered. The CASE-formula (see Goble e.a. 1980) can be evaluated in microseconds and therefore the CASE resistance can be shown for every blow in consecutive driving.

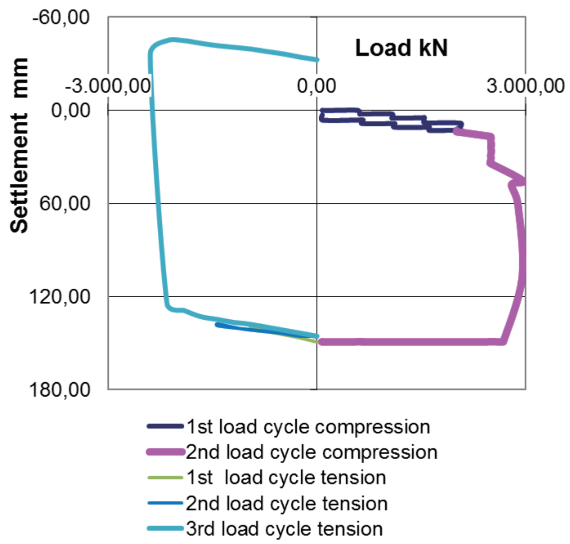

Figure 8. Static load settlement with creep.

The evaluation by the CASE-formula showed that the resistance was reduced in initial driving (see fig. 9). Also in the first redriving after 4 days the resistance was reduced very fast. However at the end of the second redriving after a longer setup time a resistance of near 4 MN was determined (see fig. 7) which is more than given for by the CASE formula for high damping. This difference in resistance in driving and redriving is an indication that the cohesive elements of the soil have an important influence on the resistance and time dependent settlements might reduce the static resistance.

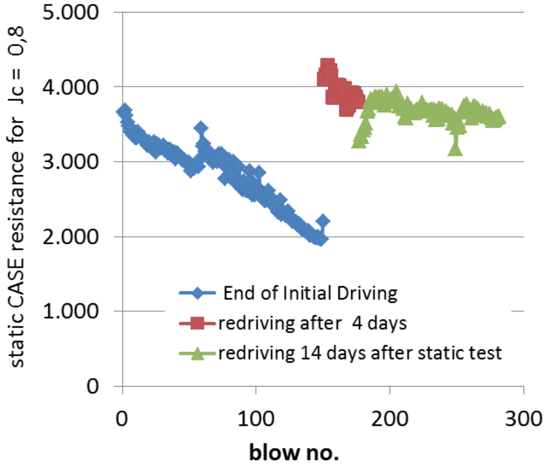

Figure 9. Development of resistances during driving.

2.4 *Open ended tubes in non-cohesive soils*

A neighbouring pile to the case described in the previous paragraph was driven deeper into a competent sand.

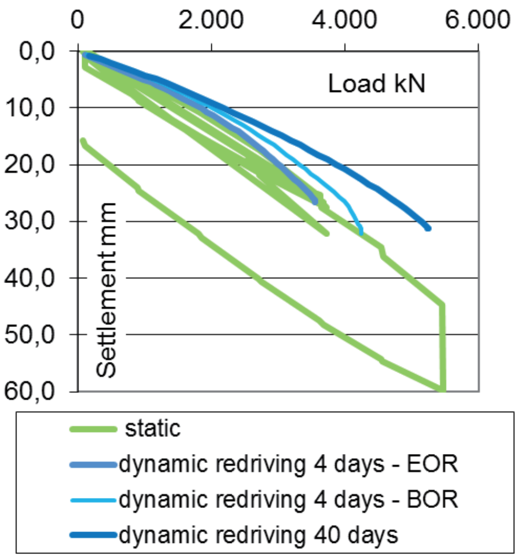

Figure 10. Comparison of load settlement curves in sand - static load settlement without creep.

The hydraulic driving rig with a 10-t-ram was not able to activate the required ultimate resistance of 5.700 kN as was determined by a static load test (see Fig. 10).

The comparison of load settlement curves shows a satisfactory equivalence of static and dynamic load settlements (Fig. 10). The development of resistances during driving shows that resistances continuously increase in driving and also in redriving (see Fig.11).

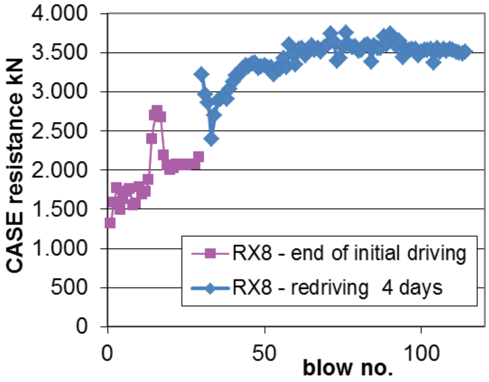

Figure 11. Development of resistances during driving.

The comparison of Figs. 9 and 11 shows the different behavior and different development of resistance to dirving of the piles driven into clayey sand (Fig. 9) and sand (Fig. 11). This reflects also how the difference of soil behavior will be seen in the load settlement curves.

2.5 *Cast in place piles in Marl*

In this project cast in place concrete piles of D=1200 mm for a working load of 2,5 MN have been installed 20 m deep into stiff marl (Mergel).

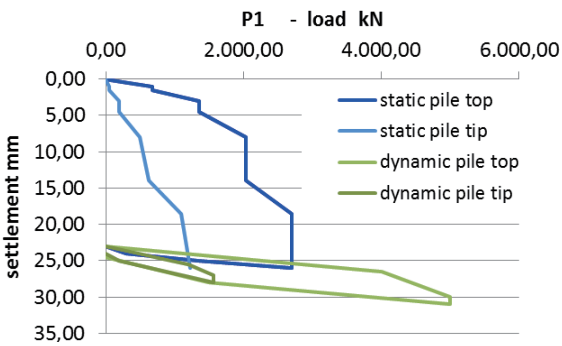

Figure 12. Comparison of load settlement curves of cast-in-place pile.

The piles have been installed in a cohesive soil of varying layers with various consistencies. The capacity verification included a static load test up to 2,5-times the working load of 2,7 MN for pile P1. The second pile should be loaded to the working load of 2,7 MN in compression and to the tension working load of 2,3 MN. For the third test only tension at working load level should be applied.

Two weeks after the static tests dynamic load tests have been executed.

In the first static test at 2,7 MN the creep settlements did not stop (see fig.12).

However the dynamic load test 4 weeks later by means of a 6,5 t drop weight with stroke 2 m demonstrated a stiff pile with limited settlements of 8 mm at 5 MN (see Fig. 12).

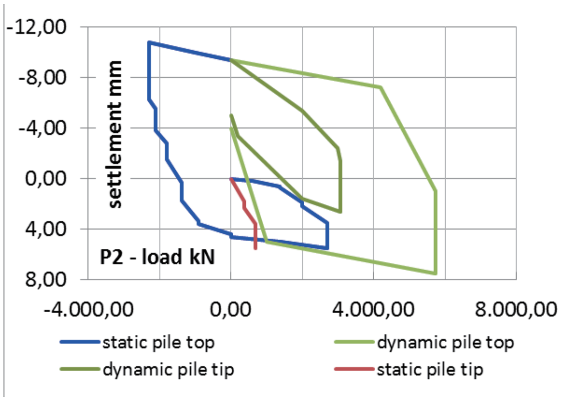

Figure 13. Comparison of load settlement curves of cast-in-place pile.

In a second static test (see Fig. 13) on pile P2 the compression was only applied to working load level of 2,7 MN and was followed by a tension load of 2,3 MN with a heave of 16 mm. The subsequent dynamic test activated a resistance of 5,7 MN but after unloading the full heave of was not equalized.

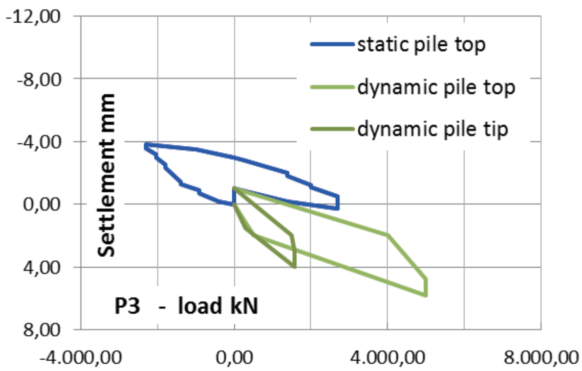

Figure 14. Comparison of load settlement curves of cast-in-place pile.

In a third static test on pile P3 the tension load of 2,3 MN was applied first and the compression to working load of 2,7 MN was applied afterwards (see Fig. 14). This load settlement curve showed a different and much stiffer behavior than the other two piles and only very limited time dependent settlement.

In a last test set only dynamic tests were executed using a 10-t-drop mass instead of the 6 t drop mass. An ultimate resistance of 7 MN could be activated (see Fig. 15).

By these tests it was concluded that the ultimate resistance of 2,5 times of a working load of 2,7 MN, i.e. 7,75 MN cannot be verified. A new design resistance had to be defined for the foundation.

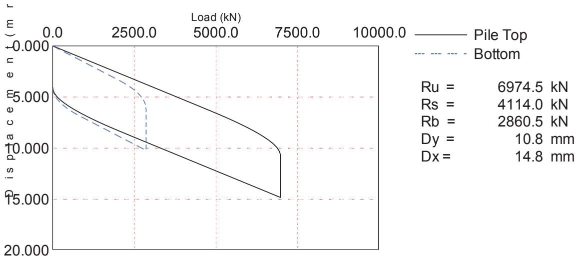

Figure 15. Load settlement curve after 4 weeks of waiting time.

From this example it can be learned that especially in cohesive soils the calibration of dynamic tests by comparison to static tests needs a very careful test design. If as in this project the soil is varying and also the designed pile tip levels it is better to prepare for more static tests to the ultimate resistance than for only one.

From the one test here even doubts are raised towards the quality of the first test pile. Also it is necessary to push the pile back to the original position after a tension load was applied with non-neglectable heave.

2.6 *Cast in place piles in sand*

A test series was carried out for concrete cast in place piles, dia. 750 mm and length 14 m. First a static compression load of twice the working load of 1,5 MN, i.e. 3 MN, was applied in three cycles. After this test a tension load was applied to 3 times the tension working load of 700 kN, i.e. 2,1 MN (see Fig. 16).

After unloading of the compression load there was a final set of 10 mm and after unloading of tension there was a final heave of 40 mm plus 10 mm.

In a subsequent dynamic load test using a 6,5 t drop weight the load was applied 4 times before the pile equalized the heave of the tension test.

CAWPAP load settlement curves showed a consecutive softening because the skin friction was released and the stiffness of tip resistance was dominant.

Compared to the previous example of load settlement curves in a cohesive soil here it can be recognized that the behavior determined by dynamic load tests is sufficiently similar to the behavior as determined by static load tests.

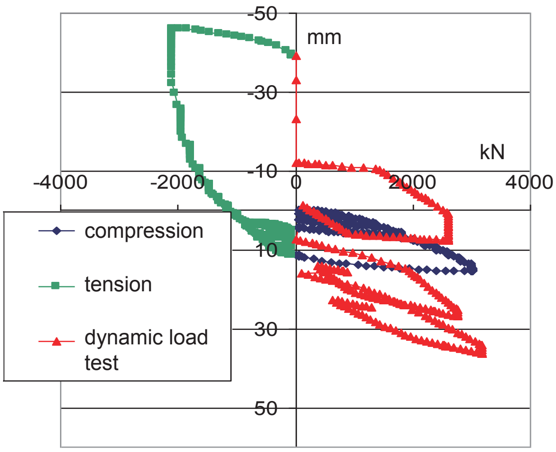

Figure 16. Comparison of load settlement curves of cast-in-place pile in sand.

On the other hand it must be taken in mind that static load tests have not been designed for a possible ultimate resistance and also dynamic tests by a 6,5-t-drop weight could not activate and ultimate resistance.

2.7 *Pre-cast driven piles in sand*

Precast piles have been driven into a dense sand by a 10-t-hydraulic hammer. The evaluation of the impact loading showed the load settlement curve of Fig. 17. A static load test four weeks later was designed for twice the working load of 1,8 MN, i.e. 3,6 MN. As can be recognized the behavior in static and dynamic is predominantly elastic and with 14 mm final set it must be concluded that the activated resistance is well below the ultimate resistance.

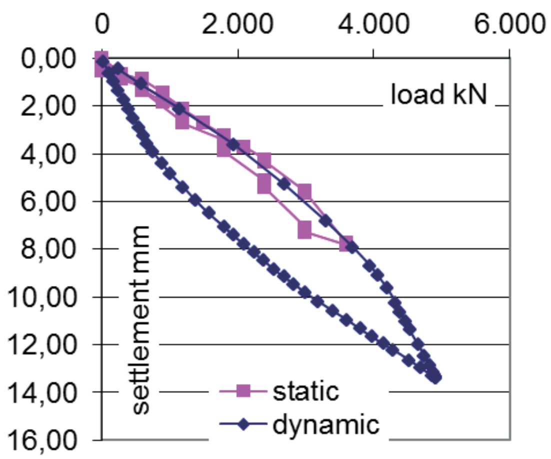

Figure 17. Comparison of load settlement curves of precast driven concrete piles in dense sand.

3 THE CONCEPT OF CALIBRATION

As in the codes calibration is not defined it is assumed that calibration is understood to be a scaling procedure that transfers static resistances of dynamic tests to the correct values. As was illustrated by the examples the calibration, i.e. the derivation of pile capacities for static loading from by dynamic load tests must be understood as a variable procedure.

Calibration by scaling is only possible if the dynamic resistance and the static resistance are definitely ultimate resistances as defined by the same criterion (settlement of 0,1 D like in Germany or e.g. Davissons criterion see Likins et al. 2004). This condition is only fulfilled if the static test load is at least twice the working load as defined by a precalculation. For the dynamic test this demands in medium soils (cohesive or noncohesive) that the ram mass is at least 2% of the test load in t.

In the 11 cases described this condition was only given in the first case. In all other cases either the drop mass for the dynamic load test was too small or the static test load was only taken as the working load. In the cases in cohesive soil the static test revealed time dependent settlements (creep) as a limiting factor for the ultimate resistance.

In these cases a calibration as understood by the German National Application Document to EC7 (see Introduction) is not possible and the full penalty for the correlation factor should be applied.

As can be learned by the different cases there is a clear difference between dynamic load tests in cohesive or noncohesive soils.

Compared to cohesive soils the tests in medium to dense sand always verified a good correlation of static and dynamic tests. Also as shown in figures 3, 5, 6, 10, 16 and 17 in the tests either static or dynamic tests did not determine the ultimate resistance.

If this situation is given it is concluded that resistances as determined by dynamic load tests are conservative and an increased resistance (or correlation) factor as is foreseen by the EC7 for dynamic tests is not reasonable.

As was shown for driven piles the equivalence of static and dynamic resistances has to bear in mind the sequence of the testing. Also the development of resistances as can be shown by a driving record including CASE-resistances has to be observed. If there is a very large difference between End of Initial Driving to Begin of Redriving resistances care should be taken to accept the static resistance from the redriving.

As was shown in figures 13, 14 and 16 the sequence of the static tests has to be taken account of. If the tension test load was applied before the dynamic test the heave has to be compensated before the dynamic load test can be expected to produce a comparable resistance to the static compression test.

4 CONCLUSIONS

It was shown that “calibration” of static resistances from dynamic load tests by static resistances from static load tests is a procedure that demands homogeneous data of ultimate resistances as have been used for establishing the correlation of the two load tests (see Rausche e.a. 2008).

In many cases however there are static load tests that allow an assessment of static resistances from dynamic tests even if no comparable ultimate resistances are available. In these cases calibration must be understood as a qualitative procedure where the resistances from dynamic tests are carefully defined by engineering judgment using all available information. In the definition of allowable pile loads this form of calibration could find safety factors different (smaller) than defined.

Especially for piles driven into a medium to dense sand the comparison of load settlement curves show a nearly complete equivalence. As in these cases the load settlement curves show that the ultimate resistance was not activated and the determined resistances are conservative it may be justified to consider this additional safety for the determination of the allowable action.

As a conclusion “calibration” should be understood as a set of variable procedures depending on pile and soil type:

- Calibration by scaling:
  This approach demands that in static and in dynamic load test an ultimate load has been defined.
- Driven piles and cast in place piles in sand:
  Calibration is understood as a comparison of load settlement curves. If only elastic behavior was determined capacities of either test can be accepted as conservative.
- Piles in cohesive soils:
  To compare the static and dynamic load settlement curves the time dependent settlements should be removed from the load settlement curve of the static test. The influence of time must be assessed separately if there is no influence on ultimate resistance.
- Driven piles in cohesive soils or sand with high fine content:
  The development of CASE-resistances during driving should be carefully observed. The sequence of tests, waiting times between initial driving, static tests and redriving is very important for the evaluation of the resistance to static loading.

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