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TWO CASE HISTORIES OF ANALYSIS OF PILE RESPONSE USING THE UNIPILE SOFTWARE

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Two Case Histories of Analysis of Pile Response

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ABSTRACT Analyses using basic soil parameters were applied to the results from static pile loading test on a strain-gage instrumented, 406 mm diameter, 45 m long pile driven in soft clay. The analyses employed effective stress analysis, simulation of the pile head load-movements from t-z and q-z functions, and delineation of residual load. The t-z and q-z functions were derived from (calibrated by) the measured values of load vs. movement at the gage locations. The analyses were made with the UniPile software which employs basic soil parameters, such as soil stress, and correlates pile resistances to effective stress (beta-analysis) or total stress (alpha analysis). The results showed that the fitting of results to analysis can be achieved without resorting to sophisticated numerical methods.

INTRODUCTION

All analyses of results from loading tests on piles rely on basic soil parameters, such as total and effective stress distribution, unit soil strength whether by total stress (undrained shear strength), or by effective stress (correlation to force by the coefficient of proportionality to effective stress, the beta-coefficient). The parameters and correlations are these days usually employed in sophisticated numerical methods software, e.g., employing finite-element methods. The purpose of this paper is to show that, while computer software is necessary in order to save time and to obtain the full benefit of testing results and analysis, no more complicated numerical treatment is required than software that relies on principles similar to an informed hand-calculation.

Head-down Static Loading Test on a Driven Strain-gage Instrumented, Concrete Pile

A static loading test was performed on a strain-gage instrumented 406-mm diameter, concrete-filled steel pipe pile driven to a depth of 45 m through a 9 m thick surficial sand layer into thick deposit of slightly preconsolidated, soft clay in Sandpoint, Idaho. Figure 1 shows the results of a CPTu sounding pushed close to the test pile location. Details of the soil profile and the pile, as well as driving information, etc., were published by Fellenius et al. (2003).

Figure 2 shows the load-movement curves from the static loading test of the test pile. The test was performed 48 days after the driving by the quick maintained-load method with equal increments of load applied every ten minutes. The measured pile-head load-movement curve was fitted to theoretical curves by the Chin-Kondner hyperbolic method, the Hansen 80-% method, the Ratio method, and the Exponential method described by Fellenius (2012). As indicated in Figure 3, the 80-% method agreed very well with the test data.

A total of eight strain-gage levels were arranged in the pile to facilitate determining the distribution of axial load in the pile. The uppermost gage, SG8, was placed about 1 m below the pile head and level with the ground surface. The other gage levels were spaced out at approximately even distances in the pile with the lowest gage, SG1, placed 1.0 m above the pile toe.

Figure 4 shows the measured distribution of axial loads in the pile during the test for all the loads applied, as converted from the strain-gage readings, taking all records as zero loads at the start of the test. The details of the conversion from strain to axial load is described by Fellenius et al. (2003). The figure also includes the loads after all load had been removed from the pile.

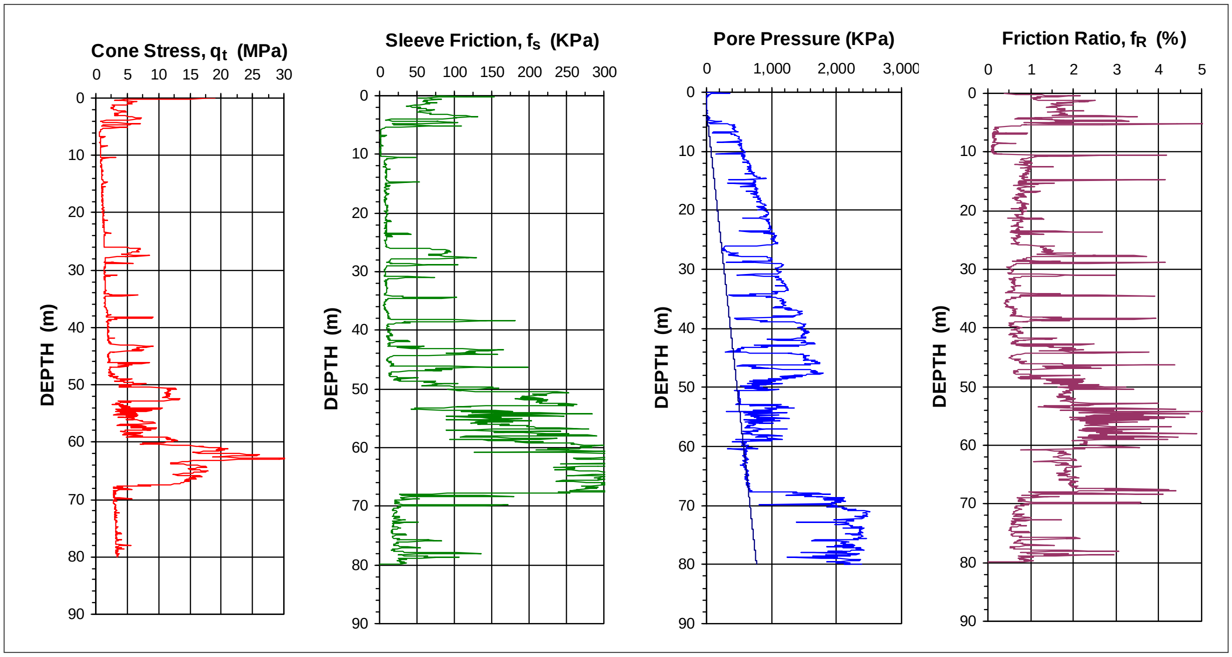

Fig.1 Results of a CPTu Sounding Close to the Test Pile Location

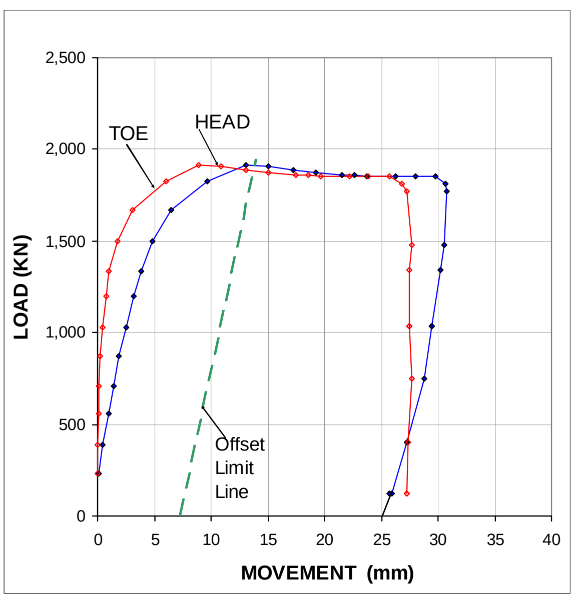

Fig. 2 Pile-Head and Pile-Toe Load-Movement Curves Measured in the Static Loading Test

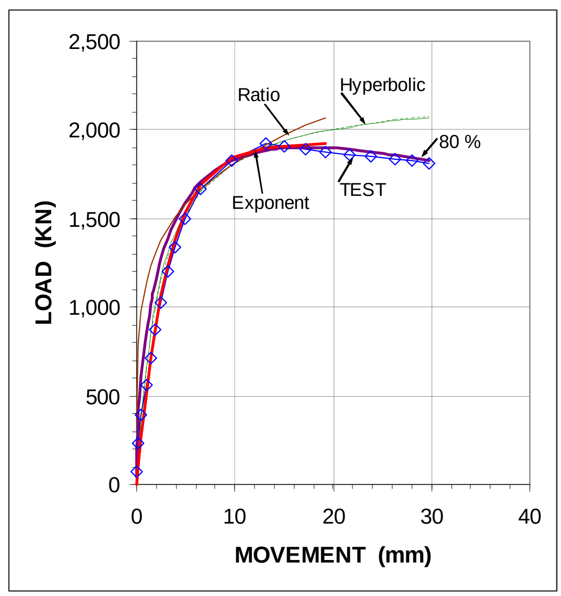

Fig. 3 Load-Movement Curve with Four Methods of Curve Fitting

The pile is affected by a significant amount of residual load. For example, below about 30 m depth, the measured load distribution does not indicate presence of any shaft resistance. This is a false impression, however, because the residual load is here caused by fully mobilized positive shaft resistance and no more than that can be mobilized by the test. The amount of residual load was determined manually by the method proposed by Fellenius (1988; 2012) and Figure 5 shows the resulting distributions of residual load and "true" load. The curve labeled "After unloading" is not corrected for residual load. The latter curve indicates that some of the residual

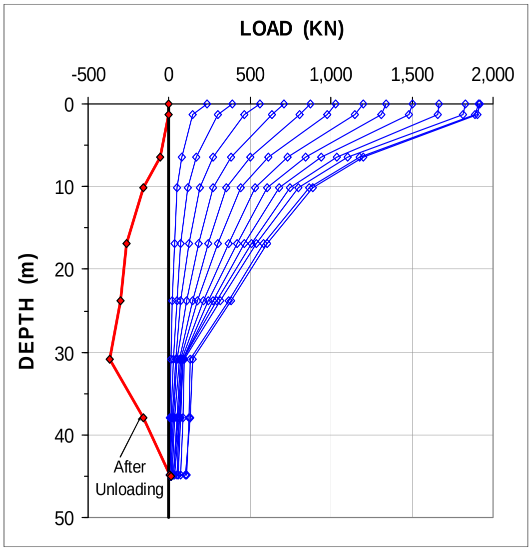

Fig. 4 Load Distributions during the Test

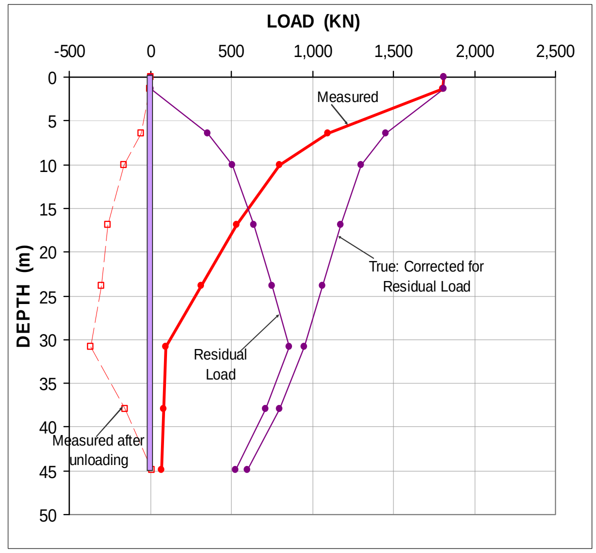

Fig. 5 Measured Load, Residual Load, and Corrected ("True") Distributions

load was released by the static loading test. The evaluated "true" pile toe resistance of about 650 KN correlates to a pile toe stress of 5 MPa, which does seem to be a bit large for soft clay.

The method for determining the distribution of the residual load is based on the assumption that the residual load is from fully mobilized negative skin friction from the pile head down to a transition zone below which the distribution changes to fully mobilized positive shaft resistance plus toe resistance. In the upper part, i.e., above the transition to positive resistance, the measured reduction of the applied load with depth, "the load distribution", consists in equal part of residual load and positive shaft resistance. The so calculated distribution of resistance, called "true" resistance, is used to back-calculate the shaft resistance parameters, most conveniently in an effective stress analysis resulting in applicable beta-coefficients. Below that depth, no similar direct evaluation is possible. However, if it is assumed that the beta-coefficients in the upper portion also apply to the remaining length of the pile, a resistance distribution can easily be calculated for the full length of the pile. Figure A shows the results of these calculations. The analyses were carried out using the loads determined from the strain-gage records. For the two strain-gage measured values from below 30 m depth, it was assumed that the same effective stress coefficient used above 30 m depth applied also below 30 m depth.

The analysis of the "true" resistance distribution for the case history presented is straightforward and a couple of iterations in a spread sheet—a "hand calculation"—will provide the distributions of residual load and "true" load. However for more complex cases and where what-if studies are desired, a computer software, such as UniPile by Goudreault and Fellenius (2012), is necessary. There is little difference between a simple load distribution produced by means of a hand-calculation and that produced using UniPile—other than about two hours of work for the hand calculation.

Figure 6B includes the load distribution calculated using the Eslami-Fellenius CPTu-method (1997; 2012). As shown, down to a depth of about 20 m, the distribution calculated by the CPTU-method agrees quite well with the effective stress calculations—the plotted dots. Below 20 m depth, there is quite a difference, however. This is not surprising because pile resistance distributions determined from CPTU-methods are often very different from actual distributions. Nevertheless, if the CPTu-determined distribution now would be taken to be correct, UniPile can easily fit the

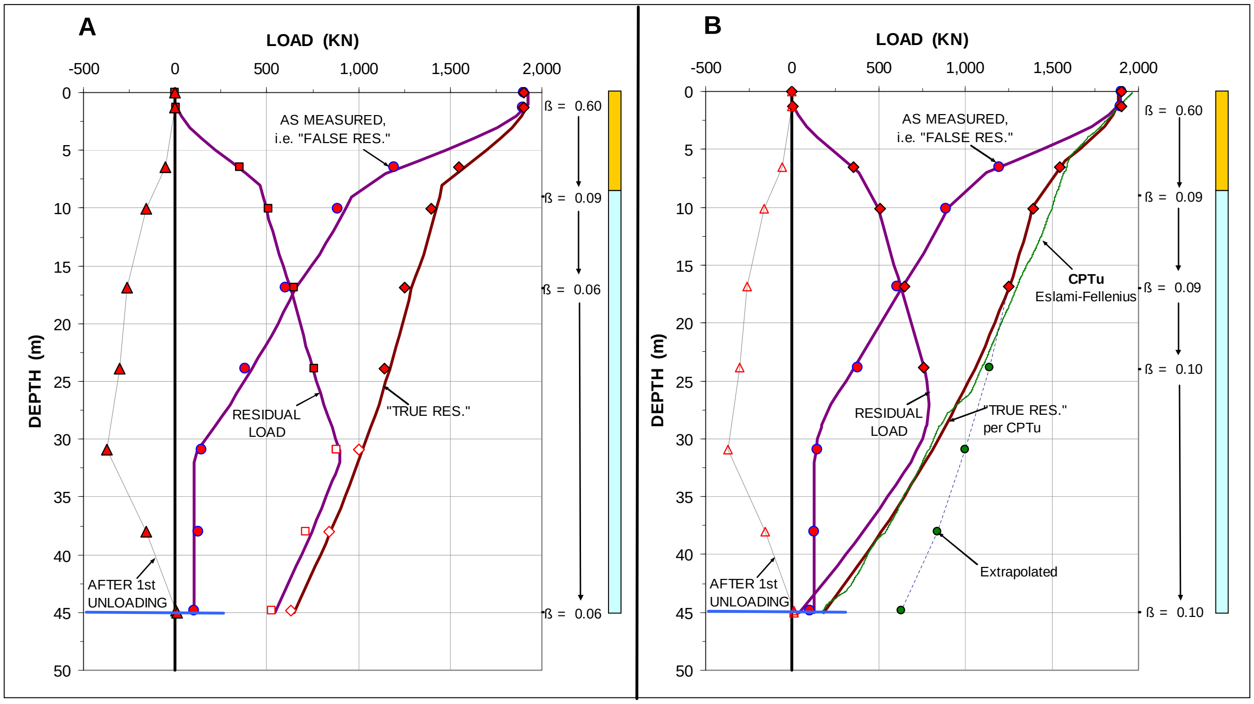

Fig. 6 Measured and Fitted Load Distributions with Beta-Coefficients

distribution to that shown by the CPTu-method by applying suitable beta-coefficients to the effective stress distribution. Figure 6B shows the results. Which of the two "true" distributions that is correct cannot be definitely stated. For what it is worth, the pile toe resistance of the CPTU-distribution in Figure 6B is 1.3 MPa, which a bit more realistic than that in Figure 6A of 5 MPa. However, the purpose of showing the two analysis results was not to find the correct distribution, but to demonstrate the ease of searching for the correct distribution by means of the "what-if" ability provided by the software.

The test records allow an evaluation of the resistance as a function of movement. Figure 7 shows the average unit shaft resistance between the strain-gage levels, calculated as difference in measured load divided by the shaft area between the gage levels. Because the corresponding "residual movement", small or large, is not known, the curves are not corrected for residual effects. The measured shear stress-movements indicate that the ultimate unit shaft resistance was obtained when the imposed movement between the pile and the soil was about 5 mm, whereafter a slight trend to post-peak softening followed.

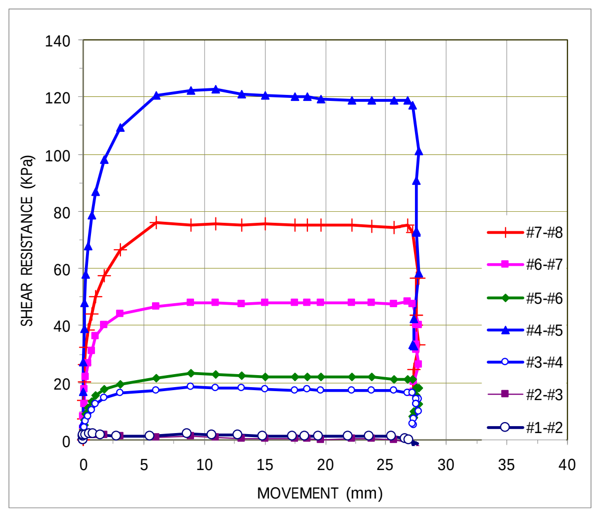

Fig. 7 Average shear resistance between gage levels

A unit shaft shear resistance vs. movement relations is called a "t-z function", which is a mathematical relation (Fellenius 2012). For unit toe resistance it is called a "q-z function". On input of a function representative for the soil layers, UniPile can calculate the pile load-movement response, i.e., simulate a load-movement curves of a static loading test. Figure 8 shows two such functions used in the fitting of the

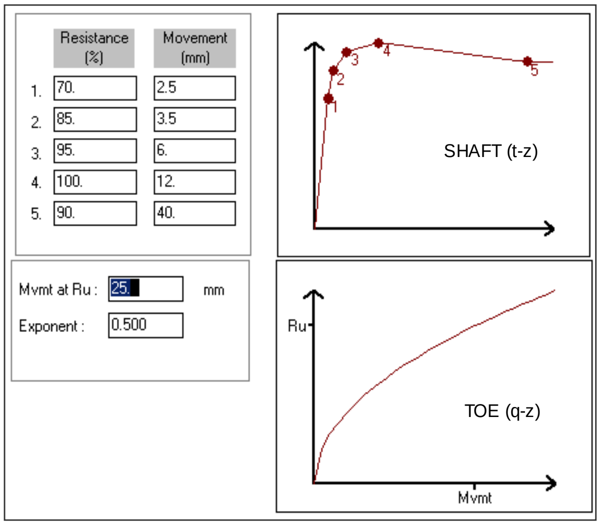

Fig. 8 Custom-made t-z and q-z Functions for Unit Shaft and Toe Resistances

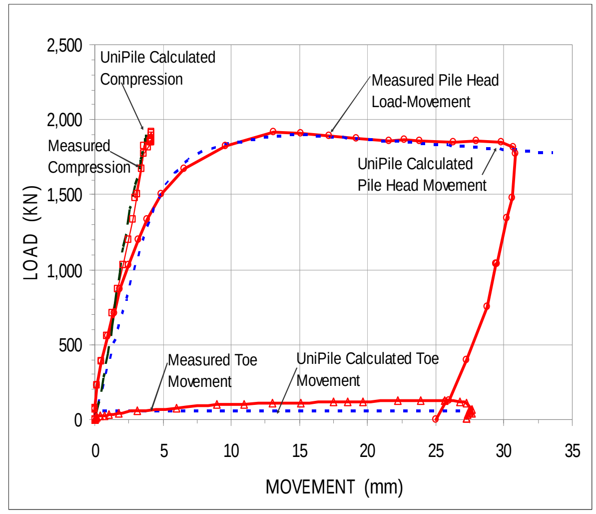

Fig. 9 Load Movements Calculated with Residual Load Compared to Measured

test pile load-movement curves for shaft and toe. For the shaft response, the measured responses shown in Figure 7 were fitted to a custom-made t-z curve, same for all elements. In view of the strain-softening response, the 80-% function could have been used instead. Other functions, such as the Hyperbolic, Ratio, and Exponent functions would have been less suitable for this case, however. The pile toe response assumed a "Ratio" function fitted to the residual load corrected toe resistance versus measured toe movement.

The new highway leading up to the bridge will include a 5 m thick embankment, which will mean an increase of stress by about 40 KPa and renewed soil settlement. The increase of effective stress results in an increase of capacity to about 2,500 KN. It will also result in an increase of the maximum load in the pile to about 1,600 KN, still an acceptable load. However, the renewed settlement caused by the embankment will impart downdrag on the piles that, potentially, could result in excessive settlement of the bridge pier foundation.

Figure 9 shows the resulting fit of the load-movement curves for the pile head, pile shaft, and pile toe as measured and as calculated assuming presence of residual load distribution per the distribution fitted to the CPTu-distribution. The figure also includes the pile toe movement and pile shortening. Note that the figure is produced from the load distribution with the evaluated distribution of residual load and the evaluated t-z and q-z relations in a simulation of the test.

The design assumed that the project piles would be the same as the test pile and be assigned a working load (dead load) of 700 KN/pile, which, for unchanged conditions, places the neutral plane (the force equilibrium) at a depth of about 15 m. The maximum drag load is about 600 KN. Thus, adding the 700 KN dead load, the maximum axial load will be about 1,300 KN, which is well within the structural strength of the pile.

The bridge pier will be placed on 15 piles and the footprint of the pile cap is 1.5 m times 15 m. Figure 10 shows the distribution of settlement calculated by UniSettle (Goudreault and Fellenius 2011) using soil profile of the UniPile calculations after input of soil compressibility values, the increased highway thickness, and pile group geometry and loads accordance with the recommendations by Fellenius (2012).

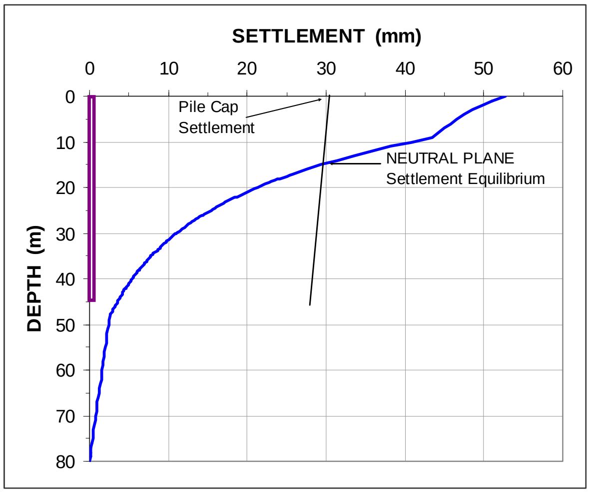

Fig. 10 Distribution of Long-Term Settlement for the Bridge Abutment

CONCLUSIONS

The load distribution evaluated from the test on the instrumented driven pile indicated that the pile was affected by a significant amount of residual load. The software enabled analysis of the distributions of true and the residual load applying two approaches for the distribution of resistance below the upper zone, the zone where the residual load is from fully mobilized shaft shear. In one approach, the assumption was made that the shaft resistance below this depth followed the same values of beta-coefficient as in the upper zone. In the second approach, the assumption was made that the resistance agreed with that calculated by the result of an adjacent CPTu sounding. The analysis results demonstrated the flexibility of the software.

The relations of unit shear vs. movement obtained from the test data established t-z and q-z relations, which then were used as input to the software to calculate pile head and pile-toe load-movement curves. The simulated curves agreed well with the measured curves.

Input of the working load intended for the piled foundation established a depth to the neutral plane. Input of the planned embankment heights with soil compressibility data gave predicted long-term settlements for the piled foundation.

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