Dynamic Pile Testing and Finite Element Calculations for the Bearing Capacity of a Quay Wall Foundation - Container Terminal Altenwerder, Port of Hamburg

F. Kirsch, B. Plaßmann, T. Huch, W. Rodatz
Institute for Foundation Engineering and Soil Mechanics
Technical University of Braunschweig, Germany

ABSTRACT: In April 1999 the construction of a new quay wall in the port of Hamburg started. In a first stage two new berths with an overall length of 1400 m will be built. In order to proof the bearing capacity of the foundation dynamic pile testing is performed to a great extent. To date more than thirty different piles were tested by the Institute for Foundation Engineering and Soil Mechanics of the Technical University of Braunschweig. In order to check for possible set-up effects redriven tests were performed. For comparison reasons static load tests were also carried out, one of which was done with an instrumented pile to allow skin friction and end bearing to be evaluated separately. The CAPWAP results proofed to be very helpful especially in those cases where the testing situation differed from the later service condition. Special questions called for detailed finite element analysis of the load-settlement behaviour of single piles in certain construction stages.

1 CONTAINER TERMINAL ALTENWERDER

1.1 *The project*

The port of Hamburg is one of the biggest and most important amongst the harbours in Europe. Its geographical and infrastructural position makes it very interesting as a gate from the North Sea to Europe. This stands especially for the east of the continent since the Baltic Sea cannot be shipped with the big container ships of the latest generation. Since 1980 the total freight handling in Hamburg has grown from 63,1 million tons to 75,8 million tons in 1998. In the same time the share of contained freight has grown from 11 % to 48 % (Freie und Hansestadt Hamburg 1999). This underlines the importance to provide of sufficient space for containers and container ship berths.

As early as 1973 first planning started to realise the extension of the port of Hamburg in the area of Altenwerder, which is situated south of the well known Köhlbrand-Bridge, one of the landmarks of Hamburg. It did cost a lot of political endeavor until in April 1999 the construction of the quay wall itself could commence. In a first phase two new berths with a length of approximatetely 800 m will be built until the year 2001. Finally a total length of 1400 m will give four modern container ships the opportunity to unload their goods.

1.2 *Site conditions*

The construction of the quay wall is done from the dry. The 24,5 m draught necessary for modern container ships will be achieved only after major dredging works (Fig. 1). Therefore it is necessary to install all load bearing elements of the quay wall by an extensive pile driving operation with depths to approximately 30 m.

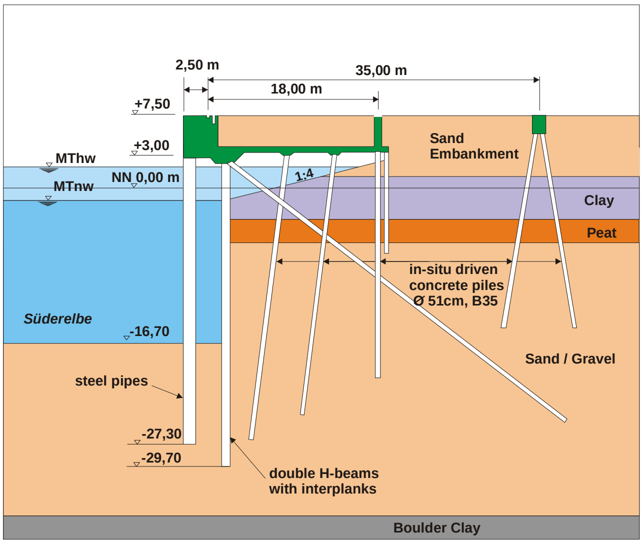

Figure 1. Cross section (Freie und Hansestadt Hamburg 1999).

Figure 2. View on the construction site (Freie und Hansestadt Hamburg 1999).

In the first phase approximately 1300 cast-in-place concrete piles with a total length of 32 km are driven into the ground to support the quay slab and the beam of the gantry crane tracks. In order to speed up the construction of the wall and in particular the front piles these structural elements are placed into 32 m deep trenches and are then driven a further five meters to final position. Figure 2 gives a view on the construction site in Summer 1999.

Subsoil conditions in the north of Germany are characterized by the effects of the last glacial period. Under deposits of marine sand and gravel with layers of silt tertiary boulder clay with different stiffnesses is encountered. Typical soil parameters as revealed by the site investigation programme and as chosen for design purposes are given in Table 1.

Table 1. Soil parameters.

Type	γ	γ'	φ'	c'	E_S
	kN/m^3	kN/m^3	°	kN/m^2	MN/m^2
Landfill	19	11	27,5	0	8,0
Sand	18	10	32,5	0	40,0
Clay	17	7	25,0	10	2,0
Mould/Peat	14	4	17,5	10	1,5
Gravel	19	10	35,0	0	100,0
Boulder clay	22	12	30,0	20	25,0

Approximately half way along the quay wall axis the depth of the boulder clay reduces and the structural elements of the foundation are embedded into it. This was not addressed as a problem since the soil investigation gave reason to expect sufficient bearing capacity. Figure 3 gives the geological situation along the axis of the quay wall.

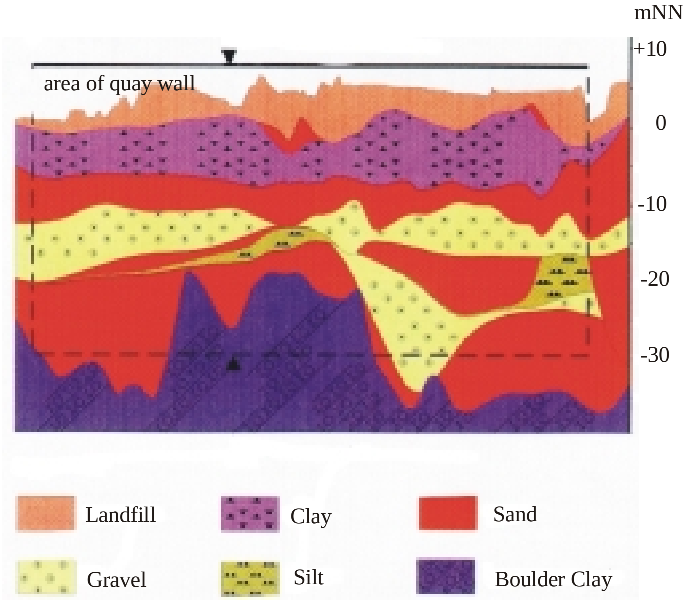

Figure 3. Geological situation (Miller 1999).

1.3 *Construction details*

The quay slab rests on tubular steel piles (∅ 1219×16 mm) and a mixed sheet pile wall with double planks (HZ 975 A) and intermediate planks (AZ 18-10) as well as on a total of 1300 driven cast-in-place concrete piles ∅ 51 cm. Horizontal stability is ensured by 46 m long inclined steel piles (HTM 600/136) driven at 2,27 m intervals. The rear crane rail, at a distance of 35 m from the front rail, is founded on a row of inclined driven cast-in-place concrete piles (Wittwer & Krefft 1999).

1.4 *Pile installation*

The steel pipes and the double planks are placed into a slurry trench filled with a cement-bentonite suspension. This mixture should hardens and achieves approximately the same qualities as the surrounding soil. Afterwards the steel profiles are driven by a 15 tons Menck MHF 10-15 hammer for a further five meters into the subsoil to ensure sufficient bearing capacity. For the driving of the anchor piles and the cast-in-place concrete piles hydraulic hammers of appropriate capacity are used.

2 DYNAMIC PILE TESTING

2.1 *Test results*

Up to now 24 restrike tests were carried out on different concrete piles. All test results were obtained by the CAPWAP procedure. Figure 4 gives a graphical overview of the load calculated capacities split into skin friction and end bearing.

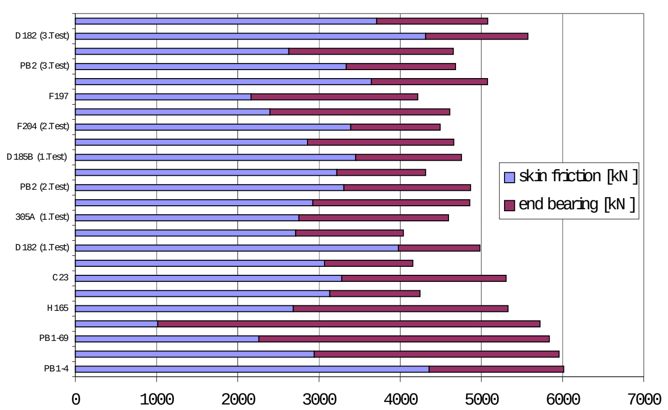

Figure 4. CAPWAP results of the driven cast-in-place piles.

A field test allowed the comparison between the CAPWAP results and the results of a previously performed instrumented static load test, where skin friction and end bearing were measured separately. The instrumentation consisted of a load cell at the end of the pile and stress measurement sections in different depths along the pile axis. Figure 5 shows the load settlement curves calculated by CAPWAP and those measured during the static load test.

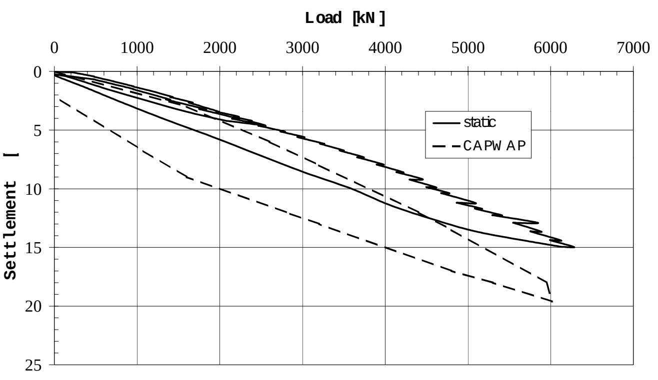

Figure 5. Load-settlement response of a concrete pile.

The tests at the driven steel profiles and tubular steel piles respectively were done at the end of the driving operation and also as a restrike test three weeks after installation. Up to now a total of 23 tests with CAPWAP evaluation were carried out. Figure 6 shows an example.

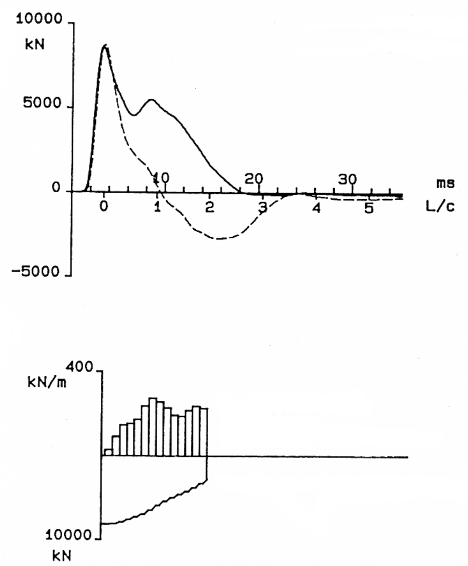

Figure 6. CAPWAP result of a tubular steel pile.

The reason for the need to gain information about the skin friction distribution along the pile axis by CAPWAP analysis lies in the specific construction of this quay wall foundation and the regulations of the competent authorities. Each individual pile has to be able to carry its load with an appropriate factor of safety even in the case of a failure of the wall. Due to possible horizontal movement of the wall the soil behind fails by developing shear bands. Thus it may loosen and subsequently reduce its frictional capacity. Therefore all resisting forces acting along the pile above an assumed line of failure must not be taken into account for the determination of the bearing capacity. This assumed line of failure starts at the toe of the sheet pile wall with an inclination of approximately 1:2 and goes in upward direction into the backing soil.

It is obviously important to distinguish between skin friction and end bearing in cases where piles are being excavated and exposed to water later on.

2.2 *Special soil condition*

As shown in Figure 3 parts of the foundation are embedded in the boulder clay which was addressed by the soil investigation as a relatively stiff clay. In contrast the systematic pile tests revealed a significant reduction in the bearing capacity of all structural elements in that area. It became obvious that the clay was far softer than expected. Careful adaptation of the piles design became necessary and different structural solutions were investigated by means of dynamic pile testing. In case of the concrete piles their lengths were extended as well as the redrive method was adopted. For the steel piles a series of different pile shoes with wings and filling sheets was tried to cope with the situation (Fig. 7).

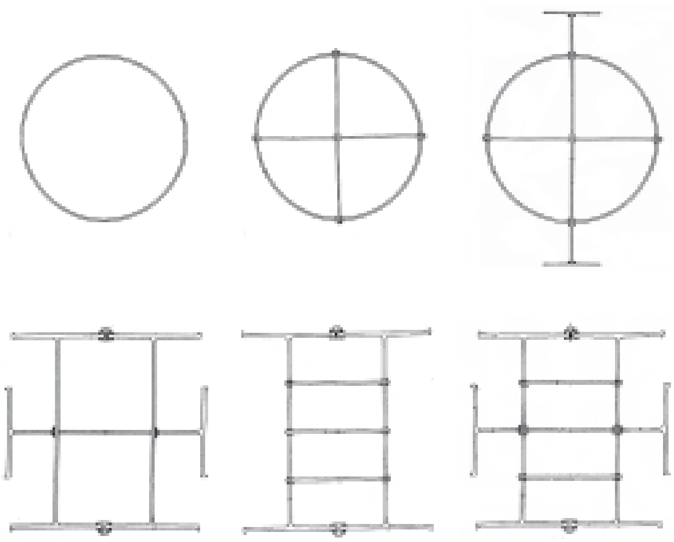

Figure 7. Different pile shoe designs.

In this critical phase the dynamic pile testing proofed its usefulness. Fast performance and the relatively low costs of dynamic pile tests made the redesign of the foundation during construction possible. In the end this observational method using the different CAPWAP results produced an optimised foundation.

3 SET-UP EFFECTS

For all types of piles a certain time dependent development of the bearing capacity was expected. As the design especially for piles embedded in the boulder clay turned out to be difficult, more effort was spent in the investigation of possible set-up effects.

Since the bentonite slurry hardens with time the increase of the skin friction along the steel profiles is evident. In fact the resisting forces became so high in some cases that it was impossible to gain any vertical displacement during the restrike tests.

For the concrete piles a gain in bearing capacity of 10% to 20% within two month after pile installation was encountered (Fig. 8).

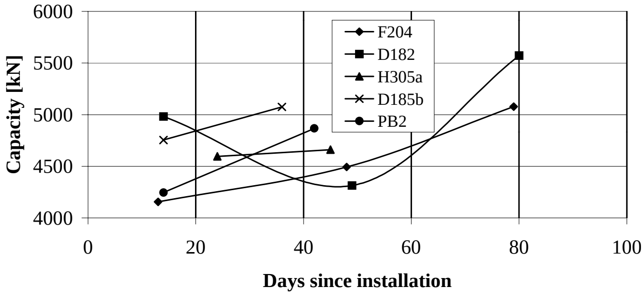

Figure 8. Set-up of the driven cast-in-place piles.

The reduction in the bearing capacity and its subsequent regain of pile D182 was object of intense studies. In order to proof that the reduction could be explained not by a general loss of strength of the surrounding soil but by the disturbance of the ground adjacent to that special pile by the recent cutting of the slurry trench, numerical simulation of the construction process was carried out.

4 FINITE ELEMENT ANALYSIS

4.1 *General remarks*

Numerical simulation in geotechnics today is object of profound investigation. The characteristics of the soil as a material led to special solutions on the side of constitutive laws for geomaterials as well as in the practice of finite element calculations. Numerous publications deal with the numerical simulation of piled foundations and their problems (e.g. El-Mossallamy 1999, Maybaum et al. 1999). For the finite element analysis of the subject case the ANSYS program was used. In the last years specific implementations into the program were realised at the Institute for Foundation Engineering and Soil Mechanics of the Technical University of Braunschweig to adjust it to geotechnical problems (e.g. Vittinghoff et al. 1997, Plaßmann et. al 1999).

4.2 *Numerical modelling*

The geometrical modelling of the problem was guided by the use of specially designed macros, which allow the easy variation of different parameters. Due to the spatial nature of the situation a three-dimensional discretisation was necessary. Using the symmetrical nature of the problem the inclined pile D182 was cut in half and the adjacent slurry trench with the surrounding soil and appropriate boundary conditions was modelled as shown in Figure 9. To allow for the calculation of the primary stress field as well as for the different construction stages double elementation became necessary. To model the soil-pile interaction properly areas with high stress gradients were meshed finer than other zones. The finite element mesh consisted of a total of 1988 brick elements with second order shape functions.

The soil behaviour is simulated by an elastoplastic constitutive law. The Drucker-Prager yield criterion governs the plastic deformations. The influence of the choice of the constitutive law was subject of former studies (Maybaum et. al 1999). Material properties of the slurry and the concrete were chosen according to prior investigation (Heinrich 1998).

The simulation of the construction process is done by a step-by-step analysis beginning with the in-situ stresses, then modelling trench and pile installation and finally the stepwise loading of the pile. The load was applied in steps of 200 kN up to failure.

Two different situations were investigated and the findings were compared with the results of two dynamic pile tests. One took place before and one after the cutting of the slurry trench adjacent to pile D182.

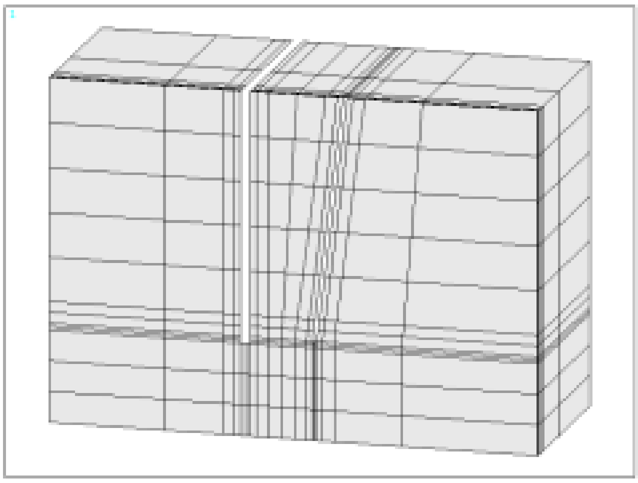

Figure 9. Finite element mesh for pile D182 with adjacent trench.

4.3 *Results*

With the finite element analysis it was possible to proof that the reduction in the bearing capacity was solely caused by the installation of the slurry trench, which occurred in this particular case after the pile driving and is therefore only a local effect in time. In general all pile installation took place after the cutting of the trench and therefore no reduction of the bearing capacity had to be expected.

Figure 10 shows the influence of the open trench on the surrounding soil. The shaded areas represent the horizontal displacements of the soil towards the slurry filled trench before pile installation. It is apparent that the pile is located in the area of influence.

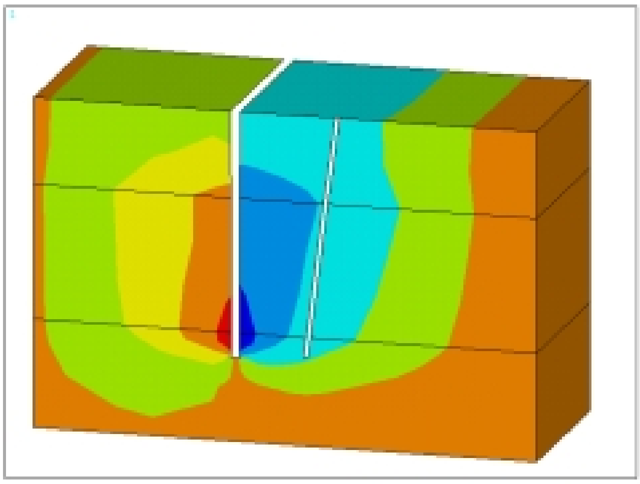

Figure 10. Horizontal displacement due to open trench before pile installation.

Figure 11 shows the vertical displacement of the soil around the axially loaded pile in the state of failure.

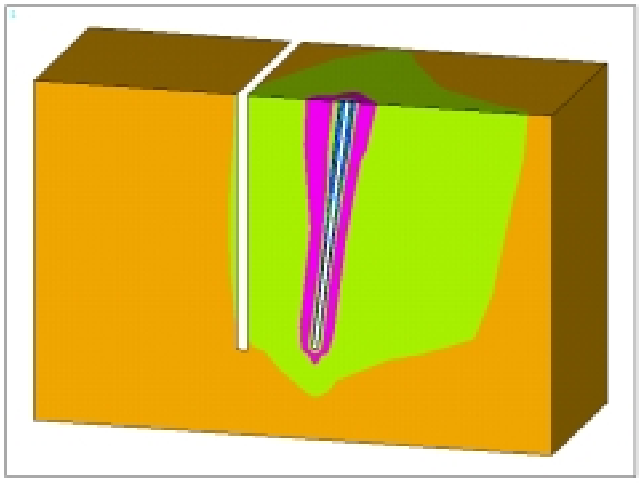

Figure 11. Vertical displacement due to loaded pile.

The load-settlement curves gained by finite element analyses as well as obtained by the dynamic pile tests before and after the slurry trench installation are given in Figure 12.

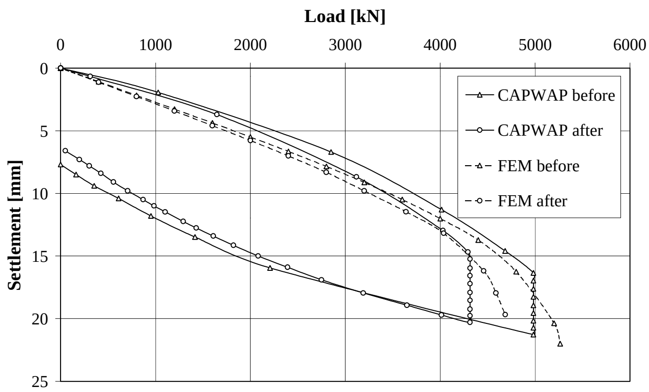

Figure 12. Load-settlement curves of pile D182.

5 CONCLUSIONS

The subject project is an example of extensive use of dynamic pile testing. Adaptation of the pile geometry due to the change of the soil properties became necessary when pile tests revealed insufficient bearing capacity. The quick and economical nature of the dynamic pile testing allows a relatively easy redesign of a piled foundation. The results of dynamic pile tests can be evaluated using a three-dimensional finite element analysis. This allows the explanation of local effects due to different construction stages.

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