Static and cyclic tests on bored piles for the foundation of a tunnel and slab tracks

H.-G. Kempfert
Institute of Geotechnique, University of Kassel, Germany

M. Raithel
Kempfert+Partner Geotechnik, Würzburg, Germany

H. Quick
Prof. Dipl.-Ing. H. Quick Ingenieure und Geologen, Darmstadt, Germany

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ABSTRACT: As part of the new high-speed railway line Nürnberg - Ingolstadt, a tunnel is under construction at the town of Offenbau. The tunnel and parts of the track will be founded on bored piles in clay formations with different bearing capacities. Due to water pressure acting on the tunnel floor and traffic loads, some of the piles will be subjected to cyclic loading. To verify the design principles and to optimize the pile lengths a total of 4 load tests on bored piles with a diameter of 120 cm and a length of 21 to 26 m were carried out. To simulate traffic loads, two test piles were loaded by cyclic load sequences. In this paper the concept of the pile load tests and the relevant results of the pile load tests will be presented.

1 INTRODUCTION

At present the Deutsche Bahn AG builds a new high-speed railway line with a length of about 89 km between Nürnberg and Ingolstadt in southern Germany. It runs mainly parallel to the Federal Highway A9 Nürnberg-München. A total of 8 viaducts, 38 bridges and 9 double-track tunnels with a total length of approx. 25 km will be constructed. The high-speed line will be equipped with a slab track which allows a train speed of 300 km/h.

The line is divided into 3 planning sections (North, Middle and South). Part of the section North is the construction of a tunnel with a length of 1,340 m near the town of Offenbau, some 45 km south of Nürnberg.

2 GEOTECHNICAL CONDITIONS AT OFFENBAU TUNNEL

2.1 *Subsoil*

Underneath 2 to 3 m fill, quarternary sandy clay and silt of low to medium plasticity with a thickness varying between 5 and 20 m was encountered. In some areas the clay and silt layers alternate with quarternary sand layers with variable thickness. In general the cohesive soils have a stiff consistency. At the base of the quarternary formation very stiff cohesive soils are present. Locally the clay and silt layers were macerated due to groundwater.

The underlying Aalenium rock formation is made up of Opalinus claystone (so-called Opalinuston), of which the degree of weathering decreases with increasing depth. The completely weathered or residual top layers mainly consist of compressible sandy clay and silt of medium to high plasticity. The thickness of the weathering zone varies between 1 and 7 m. Upon unloading or dehydrating and subsequent soaking, the soils show a potential for swelling (*v. Wolffersdorf et al*). The fresh to moderately weathered Opalinus clay is jointed and appears as thin foliated schistose sandy claystone and partially as margelite claystone.

Most parts of the Offenbau tunnel will pass through the quarternary layers and weathered Opalinus clay (Fig. 1).

The relevant soil mechanical properties are given in table 1.

2.2 *Groundwater*

The groundwater conditions are characterized by two aquifers. The top aquifer is made up of the quarternary sandy soils, in which the groundwater level is close to the ground level. The joints of the fresh Opalinus claystone and margelite claystone make up the 2nd aquifer.

The two aquifers are separated from each other by the quarternary cohesive soils and the clay and silt of the weathering-zone. Since these layers are impermeable the groundwater in the 2nd aquifer is confined and partially artesian (see also Fig. 1).

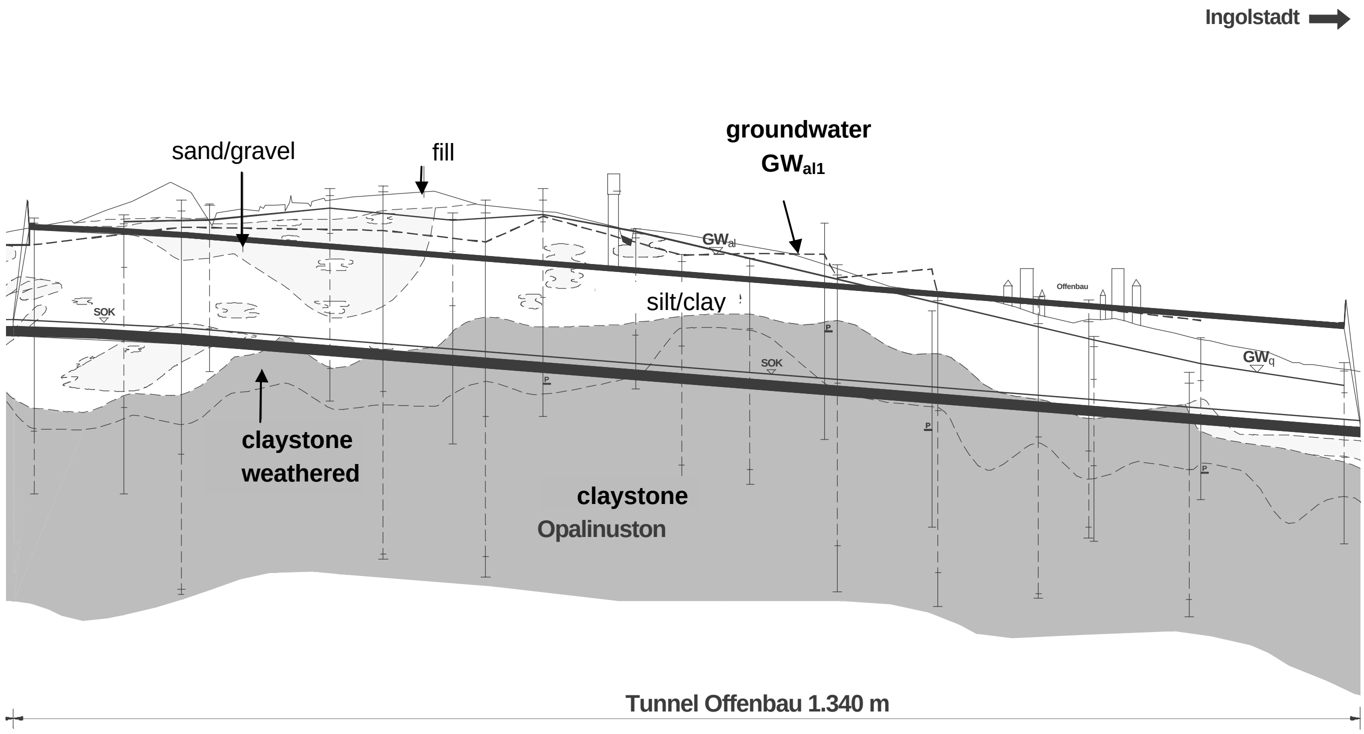

Figure 1. Geotechnical Profile of Tunnel Offenbau

Table 1. Soil Mechanical Properties

layer		weight γ/γ' [kN/m³]	angle of internal friction φ' [°]	cohesion c'/c_u [kN/m²]	constrained modulus E_s [MN/m²]	
Quaternary	cohesive soils	20/10	22,5	10/60	10	
	sandy soils	20/10	27,5	0/0	15	
Aalenium	weathered Opalinus clay	20,5/10,5	20	15/75	12,5	
	fresh to moderately weathered Opalinus clay	21/11	22,5	20/100	q_c*	E_s
					3 - 5	10
					5 - 10	15
					10 - 20	20
					> 20	40

* Cone resistance (in MN/m^2) measured with cone penetration test

3 FOUNDATION CONCEPT

3.1 *Offenbau Tunnel*

Based on the first geotechnical investigations, initially it was planned to use the cut and cover technique to construct the tunnel Offenbau. After the execution of additional geotechnical investigations it was clear, that the required extensive dewatering would result in large differential settlements of the neighboring motorway A9 and the town of Offenbau due to the presence of compressive soil layers with variable thickness. A foundation of the tunnel floor on the compressive layers would also lead to unacceptable deformations of the track under dynamic traffic loading.

To reduce the negative impacts of a temporary dewatering, the Offenbau tunnel will now be constructed via the top-down method, using secant bored-pile walls and compressed air (Fig. 2):

1. excavation up to concrete cover level
2. installation of the secant bored-pile walls (bored piles with a diameter of 1.2 m)
3. construction of the concrete cover and refilling the cut
4. excavation of the tunnel under compressed air with a pressure < 1 bar and partial relaxation of the confined groundwater in the 2nd aquifer by means of relaxation wells
5. construction of the preliminary arched shotcrete tunnel floor
6. concreting the tunnel floor and the final inner concrete lining under atmospheric pressure

During construction the upward loads will be transferred from the tunnel cover (compressed air) and from the arched shotcrete shale (water pressure and swelling of the underlying Opalinus clay) to the bored piles. This will result in a tensile loading of the piles.

After construction the upward loads against the tunnel floor due to water pressure and swelling, and the downward loads due to the construction weight, the cover and traffic load will be transferred to the piles. This will result in a compressive loading of the bored piles.

Since the pressure head of the confined groundwater in the 2nd aquifer north and south of the Offenbau tunnel is less than at the tunnel, both ramps will be constructed within sloped building-pits. During construction the confined groundwater in the 2nd aquifer will be lowered. The trough constructions will be founded on bored piles, since compressible layers with insufficient bearing capacity are present.

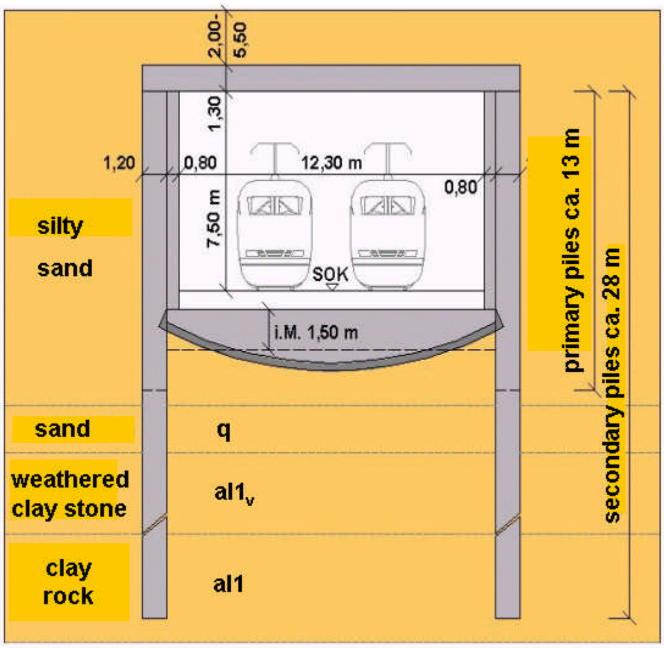

Figure 2. Typical Cross Section of the Offenbau Tunnel

3.2 *Slab Track Section Lohen - Auer Berg*

South of the Offenbau tunnel, originally the construction of embankments with heights up to 7 m was planned, by carrying out locally a shallow soil replacement and by using vertical drains to accelerate the consolidation process. Based on the results of the additional geotechnical investigations, this foundation concept had to be changed, since it became obvious, that an extensive soil replacement would have to be carried. Furthermore, to meet the strict deformation requirements for slab track (see *Kempfert et al.*) a deeper soil replacement would have been required. This would have meant an extensive dewatering of the 2nd aquifer, which would have led to settlements of neighboring constructions. Also a hydraulic contact between the two aquifers couldn't be ruled out, taking into account the great soil replacement depths and the use of vertical drains.

Therefore, for this section a foundation on bored piles was decided.

4 PILE LOAD TESTS

4.1 *Objectives*

For the preliminary foundation design, values of the shaft friction and the base resistance were determined in accordance with the German standard DIN 4014, which distinguishes between values for soil and rock. Since a clear distinction between soil and rock was not possible in most cases due to the different weathering stages of the Opalinus clay, the determination of the bearing capacity of the piles was uncertain.

To verify the predicted pile load capacity and to optimize the pile lengths, it was decided to carry out pile load tests. Since the geology at the Offenbau tunnel and the elevated track south of the tunnel is more or less similar, it was necessary to plan the pile load tests in such a way, that the results of the pile load tests can be transferred to all sections with piled foundations.

4.2 *Concept and Plan*

The majority of the piles will be subjected to cyclic compressive loading. In areas with small construction weights or thin cover layers, the piles will be subjected to cyclic tensile loading due to water and swell pressure acting on the tunnel floor. During a train passage, these piles will be loaded by compression, which will result in alternating loading of some piles.

To simulate the actual load combinations as accurate as possible a total of 4 bored piles with a diameter of 120 cm were test loaded. At 2 piles only static loads were applied. The other 2 piles however were preloaded with cyclic load prior to applying the static load increments.

To obtain information about the base resistance in layers with different bearing capacity, the test piles were penetrated into layers with cone resistances $q_c > 20$ MN/m^2 (South) and into layers with cone resistances between 10 MN/m^2 < q_c < 20 MN/m^2 (North).

The loads were applied to the piles by jacking against spreader bars. As reaction piles 2 bored piles with a diameter of 120 cm were used. To ensure a sufficient reaction force, the reaction piles were penetrated deeper than the test piles (Fig. 3). The clear distance between the test pile and the reaction piles was equal to 3 m, i.e. 2.5 pile diameters.

Prior to the construction of the test piles, the actual subsoil conditions were determined using cone penetration tests (CPT) and core borings. Especially the results of the CPT's were used to determine the length of the test piles, the load increments and the installation depths of the measuring devices.

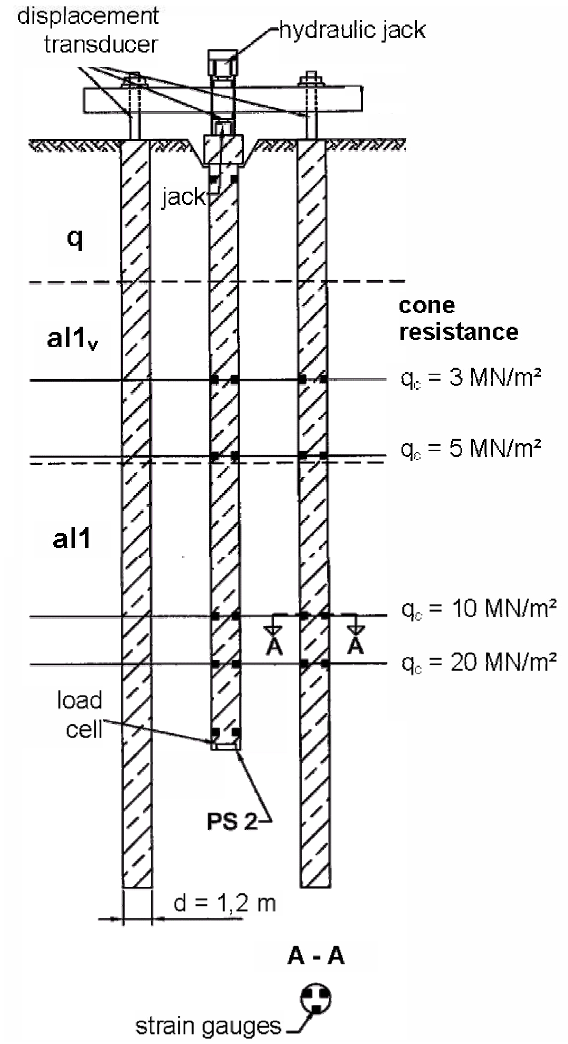

Figure 3. Typical Test Setup

4.3 *Instrumentation of Test Piles and Test Setup*

The test and reaction piles were constructed in the same way as it is foreseen for the working piles, i.e. using a temporary casing and drilling under water.

To measure the shaft friction in different soil layers and the base resistance the test piles were equipped on site with several strain gauges and a load cell at the base, which were welded to the reinforcement (Fig. 4).

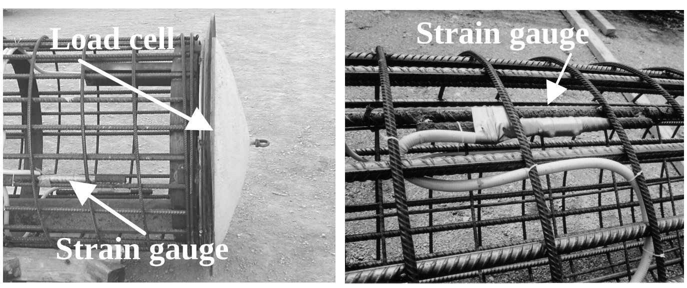

Figure 4. Instrumentation of Test Piles

For the reaction frame, which was designed for a maximum test load of 14 MN, 4 steel beams HL 1000 x 455 were used.

The compressive load was applied by hydraulic jacks (Fig. 3 and 5). The tensile load was introduced by GEWI rods installed in the piles, using hydraulic jacks on top of the reaction frame.

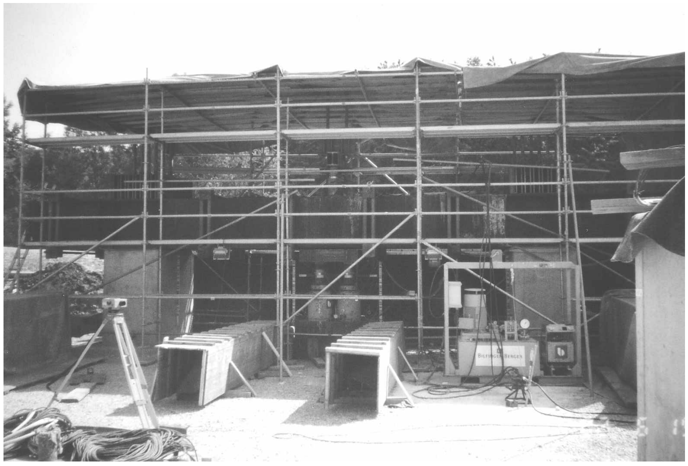

Figure 5. General View of Test Site

4.4 *Test Implementation*

The pile load tests were conducted using load increments. It was planned to reach a settlement of at least 10% of the pile diameter (i.e. 12 cm).

At the cyclic load tests firstly several cyclic load increments were applied, after which these piles were loaded incrementally up to 14 MN (Fig. 6).

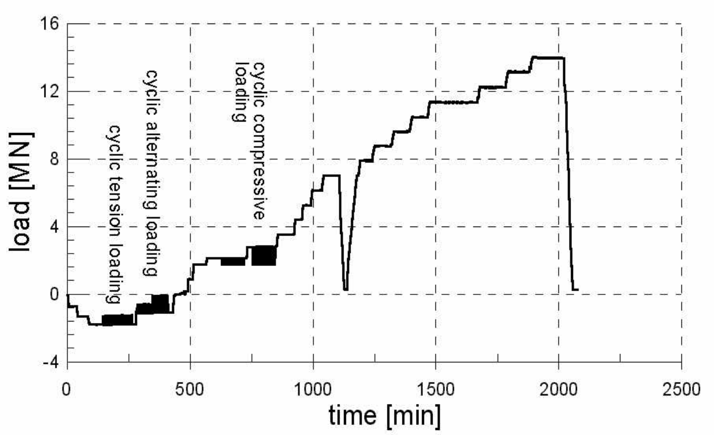

Figure 6. Load Stages of Cyclic Load Test

4.5 *Test Results*

4.5.1 *Introduction*

The results of the pile load tests were evaluated in accordance with the “Recommendations for Static and Dynamic Pile Tests” (1998) of the German Society for Geotechniques (DGGT) with regard to:

- ultimate bearing load
- shaft friction in relation to the measured cone resistance
- base resistance for cone resistance between 10 MN/m^2 and 20 MN/m^2 and for cone resistance q_c > 20 MN/m^2

- load-settlement/heave curve of the test piles
- load-settlement/heave curve of the reaction piles

To verify the effect of cyclic traffic loading on the bearing capacity and settlement behavior of the piles, the results of the cyclic load tests were compared with those of the static load tests. Also the results of the cyclic load tests were extrapolated to obtain knowledge of the settlement behavior under further cyclic loading.

4.5.2 *Ultimate Bearing Capacity and Resistance-Settlement Curves*

The ultimate bearing capacities reached during the tests and extrapolated to the limit settlement of 10% of the pile diameter (i.e. 12 cm) are presented in table 2 and the corresponding load-settlement curves are plotted in Fig. 7.

Table 2. Derived Ultimate Bearing Capacity

Test Pile	Loading	Measurements after last load increment		Ultimate bearing capacity of load test	
		Settlement [cm]	Load at pile top [MN]	Ultimate settlement [cm]	Ultimate bearing capacity [MN]
PN2 North	static	12.17	6.3	12.0	6.25
PN5 North	cyclic/ static	9.8	6.25	12.0	6.5
PS5 South	static	2.5	14.0	12.0	20.4[1)]
PS2 South	cyclic/ static	5.5	14.0	12.0	19.9[1)]

[1)] extrapolated on the safe side using the "Hyperbel-method"

During the load tests at location South, the limit settlement of 12 cm was not reached. The ultimate bearing capacity of these piles was determined by extrapolating the measurements using the so-called Hyperbel-method. Since the extrapolation is based on a small part of the measured resistance-settlement curve only, this extrapolation method is subjected to a certain inaccuracy. Therefore an estimation of the minimum and maximum ultimate bearing capacity was carried out, which resulted in an ultimate bearing capacity between approx. 15 and 26 MN. Hence the extrapolated values given in table 2 can be regarded as a safe average.

The ultimate bearing capacities according to table 2 exceed the predicted bearing capacity according to the geotechnical report by approx. 25% (location North) and approx. 175% (location South).

The cyclic pile load tests show hardly no effect of the cyclic loading prior to the static load increment on the ultimate pile bearing capacity, i.e. for the cyclic loaded piles almost the same ultimate bearing capacity was obtained as for the fully static loaded piles. The small differences are probably due to the geology.

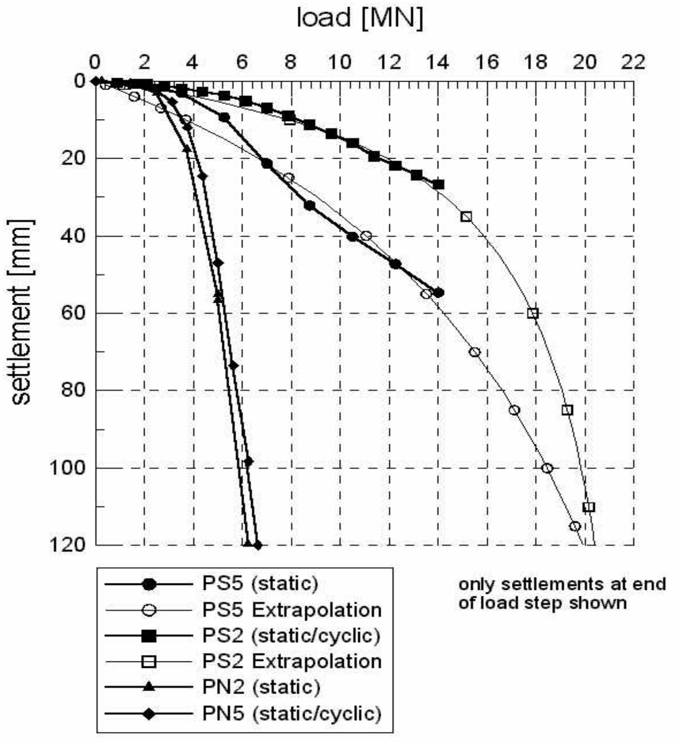

Figure 7. Load-Settlement-Curves

4.5.3 *Skin Friction an Base Resistance*

In Fig. 8 the measured base resistance of test pile PN5 and the derived skin friction are plotted as a function of the settlement of the pile top. In Fig. 9 the skin friction derived for test pile PS2 is plotted as a function of the pile load and depth.

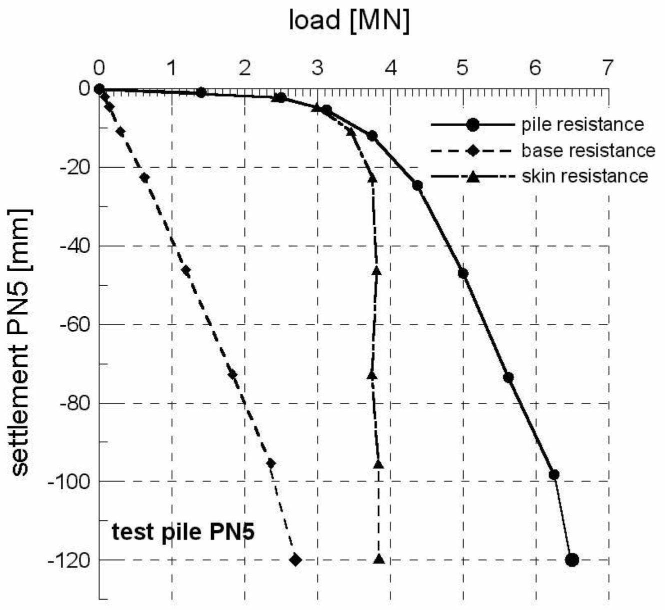

Figure 8. Skin-Resistance- and Base-Resistance-Settlement Curves of Test Pile PN5

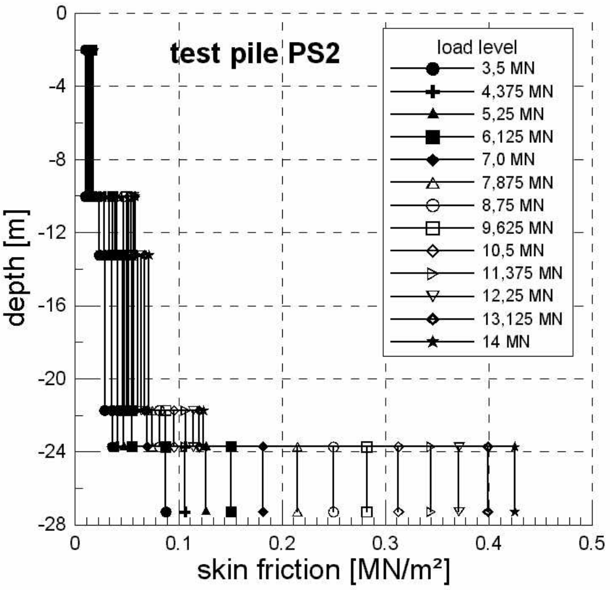

Figure 9. Skin Friction of Test Pile PS2

The derived ultimate skin friction of all 4 test piles show an almost equal distribution of the skin friction along the pile. In general, the skin friction increases considerably for cone resistances $q_c > 20\ MN/m^2$. For cone resistances $q_c > 20\ MN/m^2$ the base resistance also exceeds the values mentioned in the geotechnical report, thus explaining the high bearing capacities obtained at location South (especially with respect to location North).

4.5.4 *Deformations under Cyclic Loading*

It was already stated, that the cyclic loading prior to the static loading had only a small effect on the ultimate pile bearing capacity. Since only a limited number of cyclic load sequences could be carried out, an extrapolation of the bearing capacity under further cyclic loading (100,000 to 1,000,000 load cycles) was carried out based on the measurements obtained during cyclic loading.

To determine the influence of the number of alternating loads on the settlement, the settlements were plotted in a semi-logarithmic scale. Usually this results in a straight line, which makes it possible to extrapolate the settlement due to further cyclic loading. During the execution of the test however, it was noted, that the settlement measurements were partially influenced by variable solar radiation, despite adequate protection. The falsification of the measurement results due to irregular solar radiation on the instrumentation, partially exceeded the settlement increment due to alternating loading. Since the incremental settlements were very small, an interpretation of the measurement results was difficult.

As an example the extrapolated settlement due to a cyclic tensile load is shown in Fig. 10 for test pile PN5.

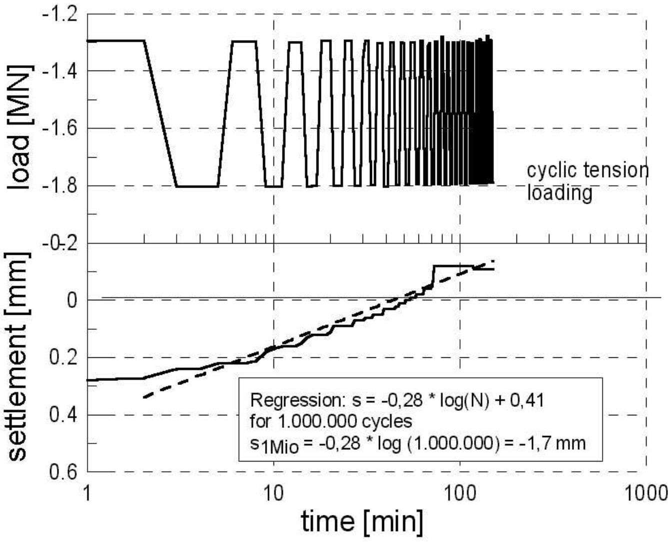

Figure 10. Extrapolation of Settlements due to Cyclic Tensile Loads (Test Pile PN5)

The extrapolation of the settlements measured during the cyclic loading sequences, as described above, resulted in the following estimation of the maximum settlement increment after 1,000,000 load alternations:

- 1.1 mm for cyclic compression loads
- 1.3 mm for alternating loads
- 1.7 mm for cyclic tension loads

4.6 *Conclusions*

4.6.1 *Ultimate Skin Friction and Base Resistance*

Based on the results of the pile load tests the characteristic values of the ultimate base resistance and skin friction as given in table 3 were recommended. With respect to the uncertainties by extrapolation the load-settlement curve of the test piles at location South, the ultimate skin friction and base resistance for cone resistances $q_c > 20\ MN/m^2$ were determined on the safe side, resulting in an ultimate bearing capacity of approx. 16 MN for piles PS2 and PS5.

Table 3. Characteristic Values of Ultimate Skin Friction and Base Resistance [1)]

Cone resistance	Ultimate skin friction τ_{mf} [MN/m²]	Ultimate base resistance σ_{sf} [MN/m²]
$0\ MN/m^2 < q_c < 3\ MN/m^2$	0.03	-
$3\ MN/m^2 < q_c < 10\ MN/m^2$	0.04	-
$10\ MN/m^2 < q_c < 20\ MN/m^2$	0.12	2.5
$q_c > 20\ MN/m^2$	0.35	6.0

1) assuming a penetration depth of at least 2 times the pile diameter in the relevant cone resistance range

4.6.2 *Characteristic Load-Settlement/Heave Curves*

For the verification of the serviceability (limit state 2) characteristic load-settlement/heave curves were specified for tensile and compressive piles. The specification of a uniform load-settlement curve was not possible, since the load-settlement curves are influenced by the skin friction and the base capacity. Therefore, in accordance with DIN 4014 guidelines were specified, which allow the construction of load-settlement/heave curves.

Fig. 11 shows that the settlements calculated with the characteristic values given in the following are slightly larger than the measured settlements, i.e. the verification of the serviceability is on the safe side.

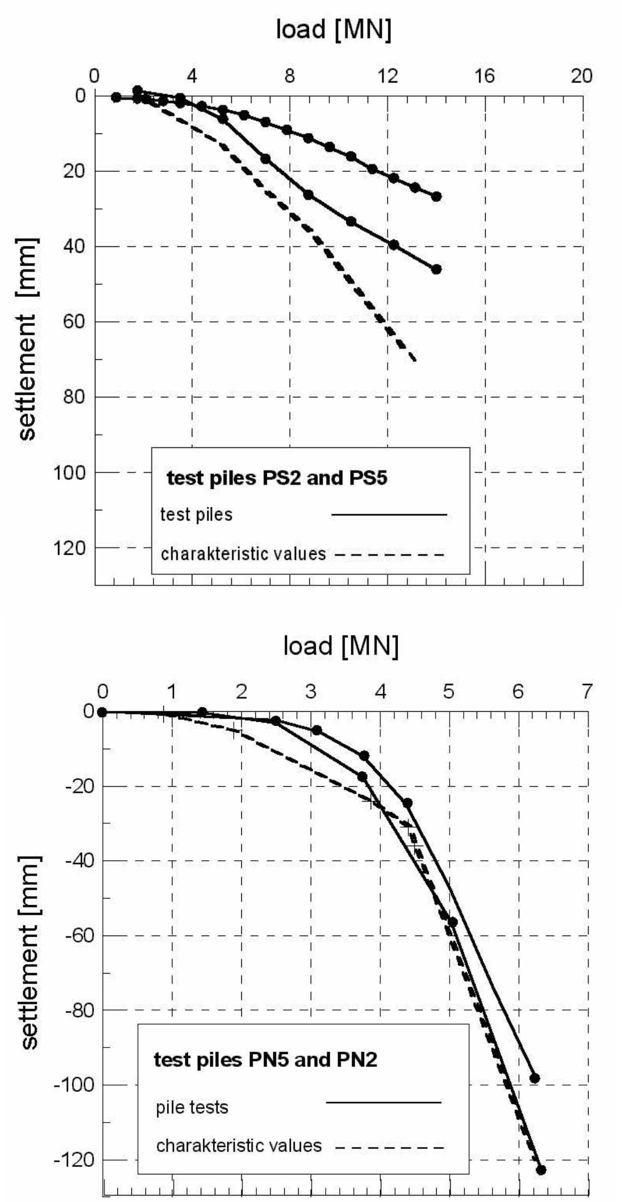

Figure 11. Characteristic Resistance-Settlement Curves

To describe the development of the skin friction, the settlements at 25%, 50% and 100% of the ultimate skin friction were specified. To determine the characteristic settlement s_r (in cm) as a function of the ultimate skin friction $Q_{r,g}$ (in MN) the following equations were derived (for tension piles the calculated settlement has to be multiplied by 1.5 to obtain the heave):

$$s_{r\,(0.25 \cdot Qr,g)} = 0.02 \cdot Q_{r,g}$$
$$s_{r\,(0.50 \cdot Qr,g)} = 0.15 \cdot Q_{r,g}$$
$$s_{r,g\,(1.0 \cdot Qr,g)} = 0.75 \cdot Q_{r,g} + 0.5$$

The base resistance was specified in accordance with DIN 4014 as a function of the ratio s/D = 0.02, 0.03 and 0.10 (D = pile diameter) as given in table 4.

Table 4. Base Resistance

Penetration in cone resistance range	Ratio s/D	Measured base resistance [MN/m²]		Design value of base resistance [MN/m²]
q_c > 20 MN/m²	0.02	PS5: 1.0	PS2: 3.1	1.5
	0.03	PS5: 2.0	PS2: 4.9 1)	2.5
	0.1	PS5: 7.9 1)	PS2: 8.3 1)	6.0
10 MN/m² < q_c < 20 MN/m²	0.02	PN5: 0.6	PN2: 1.3	0.8
	0.03	PN5: 0.8	PN2: 1.6	1.0
	0.1	PN5: 2.4	PN2: 3.0	2.5

1) extrapolated

4.6.3 *Cyclic Loading*

It was estimated, that the settlement increment after 1,000,000 cyclic load alternations would be maximal 1 to 2 mm. An influence of the cyclic loading on the ultimate bearing capacity was not observed. Therefore the specified characteristic values of the skin friction and base resistance and the characteristic load-settlement/heave curves is not corrected for cyclic loading, as long as the ratio between the cyclic load and the static load does not exceed the ratio which was tested.

It was noted however, that for the serviceability limit state an additional settlement/heave up to approx. 3 mm due to traffic loading had to be expected.

5 CONSTRUCTION OF THE PILES

At the Offenbau tunnel bored piles with a total length of approx. 70,000 m will be constructed with approx. 8,700 tons of reinforcement and 56,000 m^3 concrete. The about. 27 m long primary piles and the 10.7 m long secondary piles will be constructed under water using a temporary casing. To meet the working schedule it is necessary to carry out the foundation works with up to 12 drilling rigs (Fig. 12).

Since the tunnel will be excavated under compressed air, at least 4 cm overlapping of the piles at the bottom of the primary piles is required. To verify the correct installation of the piles and to locate areas with possible irregularities the verticality of the piles will be monitored with an inclination measurement device developed by the contractor (Bilfinger + Berger AG).

Figure 12. General View of Construction Site

To avoid irregularities due to sedimentation of the enriched solids in the water, the used water has to be pumped out and the pile casing has to be filled with fresh water. This will result in a water demand of approx. 2.5 times the volume of the pile, taking into account some water losses. Since this required amount of fresh water is not available and the used water pumped out of the pile casings cannot be infiltrated at site, a water treatment plant was build. Along the tunnel water pipelines with a total length of approx. 9 km were laid out (*Raithel et al*).

6 CONCLUSIONS

To verify the design principles of the pile foundation of the Offenbau tunnel and the following elevated track and to optimize the pile lengths, load tests on bored piles with a diameter of 120 cm were carried out. To simulate the actual working loads as accurate as possible, 2 test piles were loaded by cyclic load sequences.

An influence of the cyclic loading on the ultimate bearing capacity was not observed.

The derived characteristic values of the ultimate skin friction and ultimate base resistance exceeded the values predicted in accordance with DIN 4014 significantly. Based on the results of cone penetration tests, these values can be transferred to the entire section of the Offenbau tunnel and the following elevated track. These values also can be used in areas with comparable geotechnical conditions.

The design of the pile foundation of the Offenbau tunnel and the elevated track is based on the results of these pile load tests. Bored piles with a total length of about 70,000 m have to be installed.

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