Large scale impulse loading tests on a group of cast-in-place piles on soft cohesive ground

Essais de chargement par impulsions à grande échelle, sur un groupe de pieux forés dans les sols cohésifs mous

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Synopsis: Large-scale tests have shown, that the bearing capacity of cast-in-place concrete piles founded in soft to stiff cohesive basin sediments can be considerably improved by the installation of compacted gravel cushions at the pile base. In addition to the static pile load, impulse loads with a duration of Δt = 29 sec were applied, which confirm - as has been known from dynamic load tests - that the pile stiffness is considerably higher than for static loads.

1 INTRODUCTION

In connection with the construction of the high-speed railway line Hannover-Würzburg of the German Railway Company, up to 35 m high bridges are required to cross the wide valleys of Lower Saxony. There, the subsoil consists frequently of up to 60 m deep soft soils, which become stiffer with depth.

In spite of the difficult subsoil conditions, the design criteria permit only small subsoil and foundation deformations.

As competent soil strata, which may support the loads with small settlements, are encountered only at large depth, large-scale investigations were carried out concerning the stabilization of shallower soil layers by coarse-grained stone and gravel columns. Investigations have shown that in this way, a foundation can be achieved which results only in small settlements (Drescher and Meyer, 1987). Thus, the design concept was based on this type of foundation solution.

At the tendering, an alternative foundation concept was proposed, using cast-in-place concrete piles, type Franki. The soil layer at the pile base was stabilized by compacted gravel cushions prior to pile base construction. In the present paper, the required investigations for this foundation solution will be described, as well as the settlement behaviour of the bridge, supported by this foundation system.

2 SUBSOIL CONDITIONS

The subsoil consists within the bridge area at the valley bottom predominantly of soft to stiff, 4 to 6 m thick clay layers which towards the valley slopes change into heterogeneous loess. These variable layers are underlain by 2 to 5 m thick deposits of terrace gravels and stones, which along the valley boundaries are interlayered with silty soils ("Fliesserden"). At greater depth, up to 50 m thick deposits of cohesive basin sediments are found, which are of stiff to semi hard consistency. At the bottom of these basin sediments follow mesozoic layers (Upper Bunter).

The location of the foundation test area is indicated in Figure 1, the subsoil conditions are shown in Figure 2. A comparison of the soil stiffness modulus with the cone penetration resistance shows good agreement concerning the increase of soil stiffness with depth in the cohesive basin sediments.

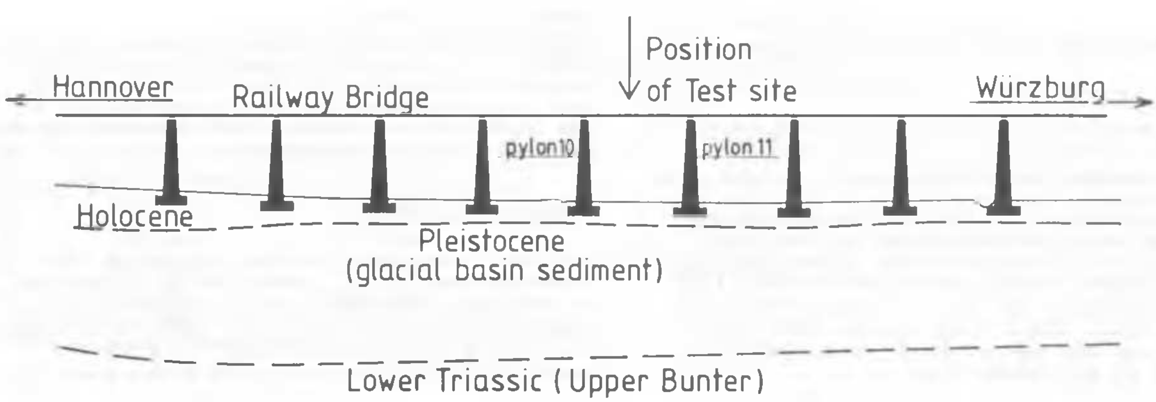

Figure 1. Geological cross section through the Aue-Valley

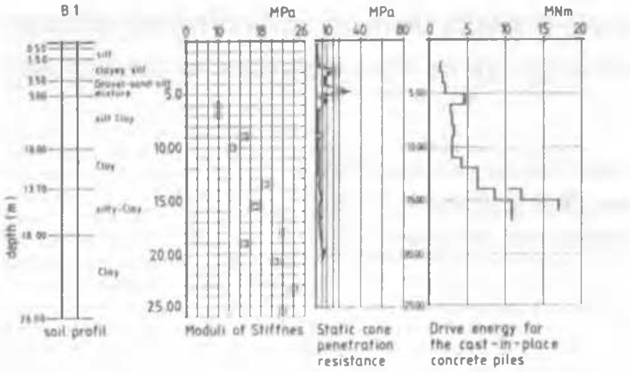

Figure 2. Subsoil conditions at the test side

3 STATIC REQUIREMENTS

The superstructure of the railway bridge was statically designed in such a way, that the bridge pylons had to support vertical structural loads as well as horizontal loads, resulting from accelerating and braking trains. The maximum compressive stresses σ_d in the continuous rails were not permitted to exceed 72 kN/mm². This required a pylon horizontal stiffness at the top of k $\geq$ 500 kN/cm, which depended on the deformation of the subsoil, the foundation slab and the pylon shaft. At a bridge span of 44 m, the vertical loads from the 33 m high pylons were

F = 33 MN structural load

P = 9 MN traffic load.

The magnitude of the horizontal traffic load as well as their distribution as a function of time are given by the superposition of two meeting trains, one of which is accelerating and the other one is braking from a maximum speed of v_{max} = 240 km/h. The resulting load-time function is shown in Figure 3.

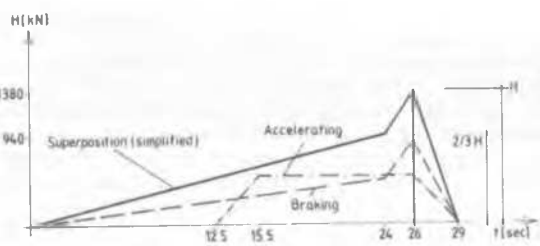

Figure 3. Resulting time dependent horizontal traffic load from braking and acceleration forces of trains for bridge pylon

The loading period of Δt = 29 sec acting on the pylons, which is determined by the load-time function, is significantly above the duration of impact loads (pendulum movements) of the pylon system (Δt = 2 to 4 sec). The relevant horizontal loads from braking and accelerating trains can therefore be considered to correspond to an impulse load.

4 FOUNDATION SYSTEM

Instead of the initially envisaged proposal for subsoil stabilization, using vibrated stone columns, the contractor proposed an alternative foundation system with modified driven cast-in-place piles, type Franki. According to this proposal, the bearing capacity of the cast-in-place piles can be improved by stabilizing the soil layer above and below the pile base using compacted gravel cushions, which do also reduce plastic deformations of the foundation system, resulting from future traffic loads.

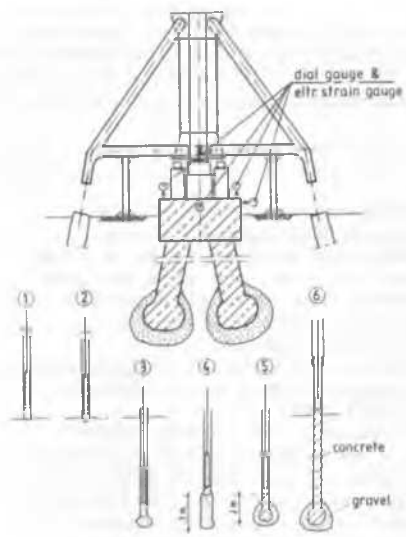

Figure 4. Schematic test arrangement and Generation of the cast-in-place concrete piles, type Franki

As there is no experience concerning this modified pile foundation system, the client required to demonstrate the attainable bearing capacity and soil deformation before giving the construction permission.

These feasibility investigations were performed by the foundation contractor, Franki-pfahl Baugesellschaft GmbH. The test arrangements, execution and evaluation of results were made according to the instructions of, and in close cooperation with the advisor of the Deutsche Bundesbahn, the Niedersächsische Landesamt für Bodenforschung.

5 TEST FOUNDATION

The load tests were carried out using four cast-in-place piles, installed at a spacing, at the ground surface, of 1,4 m and 1,6 m, respectively. The pile heads were connected by a 2,6 m x 2,6 m large and 1,5 m thick, reinforced concrete slab. The piles were inclined 5:1 and extended 17 m below the ground surface.

The loading conditions corresponded to that of a single pile in a row of compression piles, supporting the 33 m high bridge pylon, and included vertical loads and impulse loads, caused by the horizontal traffic load components.

6 LOADING TESTS

The piles were loaded using two 4 MN jacks and one 15 MN jack, to which pressure transducers were connected. The deflections of the pile system were monitored with precision dial gauges and electronic displacement sensors. The electronic signals were recorded by a plotting device. The schematic arrangement is shown in Figure 4.

The pile load was transmitted via a reaction frame to tension piles. The centric static load was generated by the 15 MN jack and the exentric impulse loads by the two 4 MN jacks.

Because of the limitations of the testing arrangement, it was only possible to simulate the horizontal impulses loads at the pile head (excentricity), but not the respective horizontal loads acting at the same time at the foundation system.

The application of the load-time function (impulse) as shown in Figure 3 was achieved by manually operating the hydraulic jacks according to prescribed load-displacement curves.

Prior to the installation of the 4 test piles, three driving tests were performed in the vicinity of the test area. Based on these trials, the four compression piles were installed to a depth of 17 m. The respective driving energy values in the relevant lower pile section are shown in Figure 2 and Tabel 1. As an example, the variation of the driving energy with depth is shown for test pile P 1 in the same diagramme.

Based on the average of the driving records, the following minimum driving energy criteria were established for the future foundation piles:

1. Driving process (gravel): E $>$ 25 MNm for the last 3 m, whereby the driving energy must be larger for the last meter than for the previous metre. The expelled gravel volume must at least be V = 0,8 m³.
2. Driving process (with concrete): E $\geq$ 32 MNm.

Tab. 1 Drive energy by Generation of the cast-in-place concrete piles, type Franki

Pile No.	Drive energy on the last 3 metres	Drive energy after Gravelinput on 2 metres
P_1	30.1 MNm	28.9 MNm
P_2	19.8 MNm	27.2 MNm
P_3	26.4 MNm	44.2 MNm
P_4	26.7 MNm	28.1 MNm
Average	25.75 MNm	32.1 MNm

6.1 Static loading

The results from the static load tests are shown in Figure 5 as load settlement curves. The fully drawn curve shows the actually measured load-settlement relationship, where the next load increment was applied after the settlement rate had decreased below s $<$ 5/1000 mm/min.

The evaluation of the load settlement curve for each load increment with respect to the final contributing settlement increment was carried out according to Rollberg, 1985, and is drawn as a dashed line, which can be considered to be a load-settlement relationship, exluding the time factor.

The future building loads, to be supported by each pile in the pile group give a maximum value of Q_2 = 1800 kN, with a corresponding settlement of s = 8,1 mm.

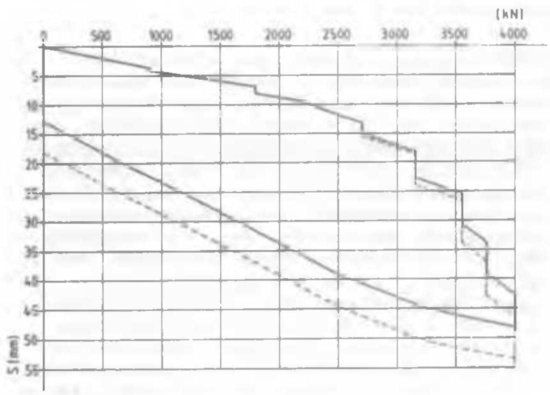

Figure 5. Load settlement curves for static loads

Although the load settlement curve increases more than linearely beyond this load level, an equilibrium is reached for each load increment up to the maximum load of Q_{max} = 4000 kN, (s = 53,4 mm). The available reaction loads from the tension piles, did not permit to reach the failure load of the piles.

After application of all impulse load cycles and static load increments, the vertical pile stiffness for the static "ultimate load" was calculated to

$$Ks = 1.800/0.81 = 2.222 \text{ kN/cm}.$$

6.2 Impulse loading

All together three impuls test series were performed. In the table 2 the corresponding loads Q and load increments ΔQ are shown together with the interpreted axial pile stiffness values k_{imp}:

Table 2. Axial pile stiffness values

Series	n	Q(kN)	ΔQ(kN)	t(min)	k_{imp} (kN/m)
1	12	1000	400	15	7039...8333
2	12	1300	500 600	25	7196...8339
3	7	1000	500	14	6643...7492

n : number of impulses
t : time between 2 Impulses.

Between the impulse load series 1 and 2, a rest period of 7 days, and between series 2 and 3 a rest period of 10 days was chosen, during which the piles were completely unloaded.

The impulse loading confirmed, that the well-known rate effect (higher foundation stiffness) from dynamic loading compared to static loading also applies to impulse loading, although the duration of the loading interval is significantly longer

7 CONCLUSIONS

The load-deformation values measured on the test pile system show that, using the modified cast-in-places piles, the static and dymanic bearing capacity and stiffness requirements can be met even in the soft to medium stiff cohesiv subsoil conditions. Based on the test results, permission was obtained to execute the project according to the proposed foundation concept.

Since the tests, and except for the railroad gravel bed, the bridge has been completed and settlement observation have been made. Assuming a logarithmic relationship between pylon settlements and corresponding time intervals, also taking into account the load from the future gravel bed, the calculated ultimate settlement for the pylon adjacent to the test area ($h \approx 33$ m) corresponds to $s = 16$ mm.

The high pylons transfer the load to a pile group of $n = 44$ piles, with a structural load of $F = 33$ MN. Each pile carries thus a vertical load component of $Q = 750$ kN. The corresponding settlements, assuming elimination of the time effect in the load-settlement diagramme, Figure 5, can be calculated to

$s_1 = 4{,}11 \cdot 750/960 = 3{,}2$ mm

from which a group factor of

$X = (16 - 3{,}2)/3{,}2 = 4{,}0.$

derives.

ACKNOWLEDGEMENT

We are obliged to the Deutsche Bundesbahn (DB-Projektgruppe Hannover-Würzburg Nord), who kindly supported this investigations generously.

REFERENCES

Drescher J., a. Meyer H. (1987)
Impact of eccentric impuls loads on footings situated on soft quarternary deposits reinforced with stone columns -large scale tests Intern. Conf. on
SOIL STRUCTURE INTERACTIONS, Paris, 47 - 54

Rollberg D.,(1985)
Die Zeit-Setzungs-Kurve von Pfählen
Geotechnik 8, H.3, 127 .. 131