Combined Cast-In-Place and Precast Piles for the Reduction of Negative Friction Caused by Embankment Fill

Pieux moulés dans le sol et pieux moulés d'avance réduisent le frottement négatif causé par un remblai

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SUMMARY

For a road and railway bridge at Moosmatten in Switzerland a pile foundation using combined cast-in-place and precast piles was adopted in order to reduce negative skin friction on piles. The paper gives a description of the soil conditions and the adopted solution for the pile foundation. It attempts to calculate negative friction forces and gives results of a loading test on one pile.

SOMMAIRE

Pour un pont route-rail à Moosmatten en Suisse, une fondation a été adoptée utilisant des pieux moulés dans le sol et des pieux moulés d'avance pour assurer le minimum de friction latérale négative sur les pieux. Le présent article décrit les caractéristiques du sol, la solution adoptée pour la fondation des pieux, le mode de calcul du frottement négatif et les résultats d'un essai de charge sur un pieu.

A NEW ROAD linking the town of Biel in Switzerland with the national highway N-1 (Berne–Zurich) (Fig. 1) had to cross an existing main road and a narrow-gauge railway line at Moosmatten near Schönbühl, 8 km north-east of Berne. The project consisted of 3 separate single-span bridges, each approximately 18 m long, carrying respectively the main road, the railway line, and a secondary road over the new access road (Fig. 2). All the bridges were to have pile foundations.

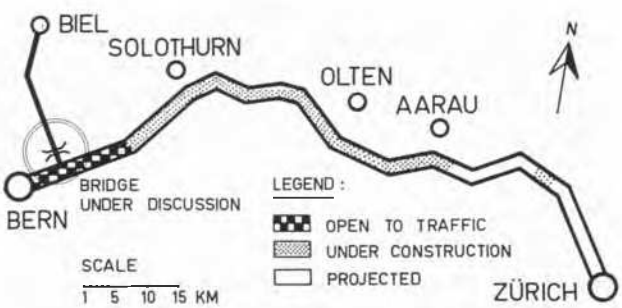

FIG. 1. Layout of National Highway N-1 and access road to Biel.

SOIL CONDITIONS AND PROBLEM OF BRIDGE FOUNDATIONS

The bridge site is situated in a typical diluvial glacier valley. The ground therefore consists of hard morainic silty gravel, which is partly very sandy and slightly clayey. These diluvial deposits are covered by a 3-to-5-m-thick layer of postglacial soft silty clay and, on top, a 1-to-2-m-thick layer of lacustrine marl and peat. Some of the borings carried out at the bridge site are shown in Fig. 2, and Table I gives some soil properties determined from undisturbed samples.

Because of traffic requirements it was not possible to build up the embankments adjoining the bridge in advance of the pile work. Consequently large negative friction forces on the piles were anticipated as soon as the compressible clay layers started consolidating under the weight of the fill. Therefore the Laboratories for Hydraulic Research and Soil Mechanics (annexed to the Swiss Federal Institute of Technology in Zurich) proposed a combined pile foundation system, which would give both the high bearing capacity of large bored cast-in-place piles and the small friction forces of smooth precast piles.

DESCRIPTION OF THE PILING SYSTEM USED

The manufacture of the composite piles was based on the Hochstrasser-Weise method (Ledergerber, 1961). Liners, 90 cm in diameter, were sunk through the soft soil layers and embedded 3 m in the solid morainic gravel (Fig. 3). Following spoil removal the liner was filled with concrete up to the level of the top of the morainic layer. When the liner was later withdrawn this concrete plug formed the base of the pile which, because of its large diameter and rough surface, provided the desired high loading capacity.

The cylindrical concrete pile, 50 cm in diameter, was placed in the sunken liner prior to the liner's withdrawal. The bond between the two pile types was ensured by reinforcement bars protruding at the lower end of the prefabricated pile shaft which was pressed into the unset concrete of the pile foot. The space between the pile shaft and the boring tube was then filled with loose fine sand and the boring tube withdrawn. The smooth cylindrical pile shafts and the surrounding fill of loose sand are thought to reduce to a minimum the negative frictional forces caused by the consolidation of the clay layer.

NEGATIVE SKIN FRICTION ON THE PILES

The friction forces caused by the consolidation of the soft clay layers were evaluated both for an ordinary cast-in-place pile of 90 cm diameter (pile type B) and for the combined pile with smooth surface in an artificial sand sheathing (pile type A). The Laboratory for Soil Mechanics proposed the following three methods for determining negative friction forces: (1) estimation of negative skin friction according to Terzaghi and Peck (1948); (2) a theoretical method proposed by Zeevaert (1959); (3) an empirical method, Elmasry (1963). The results of comparative calculations are given in Table II.

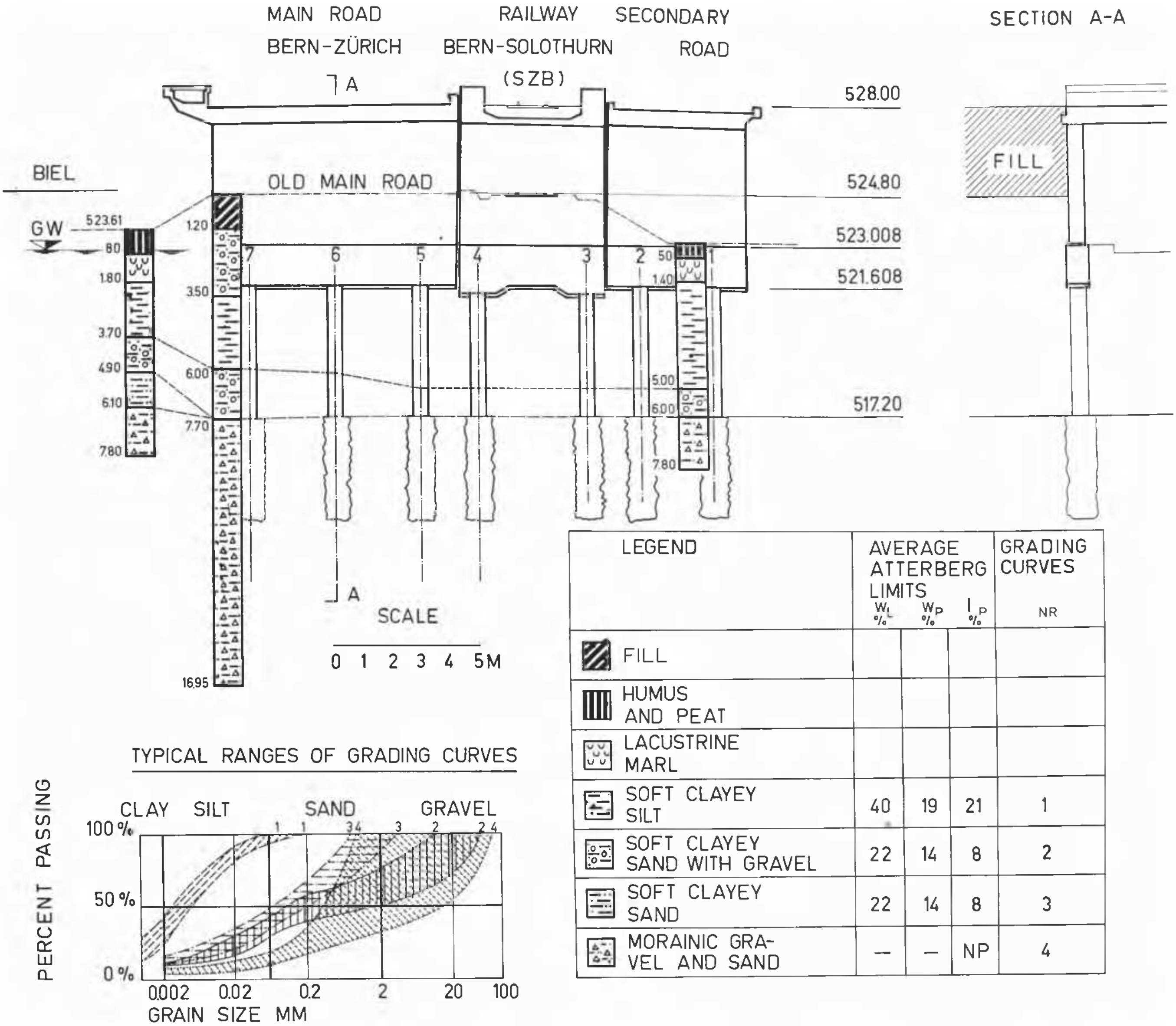

FIG. 2. Sections through bridges and soil profiles.

TABLE I. SOIL PROPERTIES OF SOFT CLAY MEASURED ON UNDISTURBED SAMPLES

	Elevation (m above sea level)	Grading curve per cent passing 0.002 mm.	0.06 mm.	2 mm.	Atterberg limits W_L	W_P	I_P	Natural moisture content, w (per cent)	Unit weight, γ (gram/cu. cm.)	Shear strength q_u (ton/sq.m.)	c' (ton/sq.m.)	ϕ' (deg)
Boring outside old fill	520.5	16	98	100	46	21	25	34.3	1.87			
	519.5	37	100		41	19	22	33.1	1.92		1.0	24
	518.5	39	100		44	18	26	35.9	1.88			
Boring under old fill	520.0	10	92	100	39	17	22	25.3	1.91	3.7	1.5	22
	519.3	14	97	100	40	16	24	25.1	1.93	4.3	1.5	22.5
	518.5	12	90	100	40	17	23	25.0	1.94			

TABLE II. COMPARISON OF NEGATIVE SKIN FRICTION FORCES AND ALLOWABLE PILE LOADS

	Negative skin friction (ton/pile)			Corresponding allowable pile load (ton/pile)		
	Method 1	Method 2	Method 3	Method 1	Method 2	Method 3
Pile type A (combined pile)		14.9			137.5	
Pile type B (ordinary bored pile)	63	56.7	69.5	61	72.9	49.2

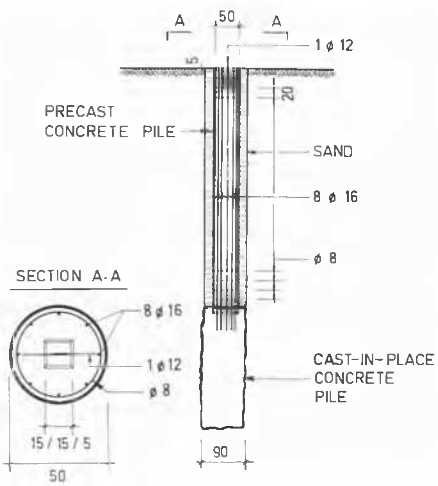

FIG. 3. Diagram of combined pile.

Basic Assumptions

The calculation was based on the following average numerical values:

Fill

Thickness, $h_{\text{fill}} = 2.6$ m
Unit weight, $\gamma_{\text{fill}} = 2.2$ ton/cu.m.
$p_c = \gamma_{\text{fill}} \cdot h_{\text{fill}} = 5.72$ ton/sq.m.

Compressible Clay

Thickness of compressible layer, $h = 4.4$ m
Unit weight (submerged), $\gamma' = 0.92$ ton/cu.m.
Dry unit weight of soil, $\gamma_d = 1.52$ ton/cu.m.
Unit weight of solids, $\gamma_s = 2.69$ ton/cu.m.
Average moisture content, $w_n = 25.2$ per cent
Unconfined compressive strength, $q_u = 4.0$ ton/sq.m.
Cohesion, $c' = 1.5$ ton/sq.m.
Angle of friction, $\phi' = 22°$

Pile Foundation

Number of piles per unit area, n'
Perimeter of one pile, $U = \pi \cdot D$
Ultimate shear strength along pile shaft:
for sand $s = K_0 \tan \phi' \, p_v$, where $K_0 \tan \phi' \approx 0.25$
for clay $s = c' + \tan \phi' \, p_v$

Calculations

Using Method 1 (Terzaghi and Peck, 1948) for pile type B only the part Q'' of the drag force has to be taken into account as the piles do not penetrate the fill causing the negative friction:

$$Q''_{\max} = Lhs/n \approx 63 \text{ tons,} \tag{1}$$

where L = circumference of the pile cluster = 50 m; h = thickness of the compressible layer = 4.4 m; s = average shear strength of the clay = $\frac{1}{2}q_u$ = 2.0 tons/sq.m.; n = number of piles in the cluster = 7. A source of considerable error which this method shares with Method 2, but not with Method 3, is the necessity of estimating the area of influence of one pile.

For Method 2 (Zeevaert, 1959) the procedure outlined has been calculated in two ways: introducing the overburden pressure p_o at the pile head, i.e. for $z = 0$ (both pile types); introducing the cohesion c' in the expression for the shear strength of the clay along the ordinary pile (type B).

Pile type A (combined pile)

The transfer of pressure, assuming the pile rigid compared with the soil, is expressed by the following formula:

$$d(p_{oz} - p_{vz})/dz = n'Us \tag{2}$$

where p_{oz} = initial effective vertical pressure in soil, p_{vz} = reduced effective vertical pressure in soil, z = depth below pile head, n' = number of piles per unit surface, U = perimeter of one pile, s = ultimate shear strength along the pile shaft. The shear strength s can be represented as a function of p_{vz}, in the case of cohesionless sand, e.g.:

$$s = K_o p_{vz} \tan \phi',$$

where K_0 = coefficient of earth pressure at rest, ϕ' = angle of friction of sand. With the abbreviation $m_1 = n' \, U \, K_0 \tan \phi'$, and introducing $p_{oz} = p_o + \gamma' z$, whence $dp_{oz}/dz = \gamma'$, the differential Eq 2 becomes

$$dp_{vz}/dz + m_1 p_{vz} = \gamma'. \tag{2a}$$

Its solution must fulfil the boundary condition

$$p_{vz}(z = 0) = p_o. \tag{3}$$

By differentiation it can be seen that the expression

$$p_{vz} = (\gamma'/m_1)(1 - e^{-m_1 z}) + p_o e^{-m_1 z} \tag{4}$$

is a solution to Eq 2a satisfying the boundary condition 3, and therefore represents the reduced vertical pressure in the soil after the load transfer due to consolidation of the soil surrounding the piles has taken place.

The negative friction acting on the pile shaft can be represented as:

$$F_n = \int_0^h Us dz = \frac{m_1}{n'} \int_0^h p_{vz} dz. \tag{5}$$

Integration of Eq 5 will lead, after some transformations, to:

$$F_n = (1/n')(p_{oh} - p_{vh}) \tag{6}$$

where h = thickness of compressible layer, p_{oh} = initial pressure at depth h, p_{vh} = reduced pressure at depth h, n' = number of piles per unit area.

Pile type B (ordinary pile)

In a clay, the shear strength can be assumed to be

$$s = c' + \tan \phi' p_{vz}. \tag{7}$$

We introduce the following abbreviations:

$$m_2 = n' \cdot U \tan \phi'$$

$$\gamma_{id} = \gamma' - n'Uc'.$$

			PILE TYPE "A"	PILE TYPE "B"
PILE DIAMETER	D	cm	50	90
PILES PER UNIT AREA	n'	PILES/m²	0.151	0.076
FRICTION FACTORS	m_1	m^{-1}	0.0594	——
	m_2	m^{-1}	——	0.0867
UNIT WEIGHT OF SOIL	γ'	t/m^3	0.92	0.92
	γ_{id}	t/m^3	——	0.81
PRESSURE REDUCTION AT z=h	Poh-Pvh	t/m^2	2.26	4.30
NEGATIVE FRICTION FORCE	Fn	t/PILE	14.9	56.7
REDUCED PRESSURE AT z=l	Pvl	t/m^2	13.7	11.6
ALLOWABLE PILE LOAD	Q'pa	t/PILE	137.5	72.9

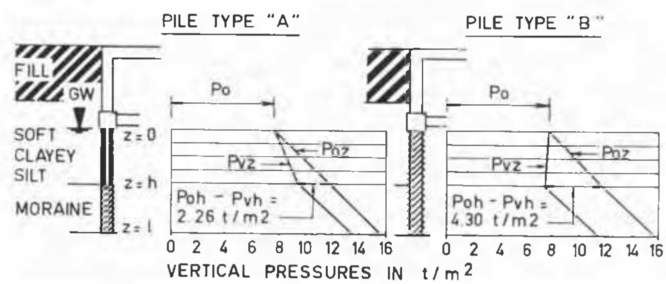

FIG. 4. Pressures p_{vz} and p_{oz} for piles type A (combined pile) and type B (ordinary 90-cm-diam bored pile).

The basic differential Eq 2 is then transformed into:

$$dp_{vz}/dz + m_2 p_{vz} = \gamma_{id} \tag{8}$$

which is identical in form with Eq 2a. Therefore the solution will be identical in form to Eq 4:

$$p_{vz} = (\gamma_{id}/m_2)(1 - e^{-m_2 z}) + p_o e^{-m_2 z}. \tag{9}$$

For the two pile types considered and with the numerical values indicated above, the pressure distributions have been calculated and represented in Fig. 4, together with the resulting friction forces F_n. The values for F_n are further given in Table II.

Method 3 is an empirical solution developed by M. A. Elmasry at the Laboratory for Soil Mechanics (annexed to the Swiss Federal Institute of Technology in Zurich). It is only applicable for pile type B. He proceeded in the following way: tests with a model pile, 50 mm in diameter and equipped with strain gauges, were carried out in clayey silt soil. In several series of tests the consolidation pressure, the thickness of the compressible layer, the unit weight, and moisture content of the soil were varied one at a time. The drag force on the pile and its distribution along the pile shaft were measured. By means of a dimensional analysis and the Buckingham-π-theory (Buckingham, 1915) an empirical formula for the negative friction force was derived.

The formula obtained by this procedure is

$$F_n = F_{n-s} + 0.416\ (2\phi_1 - 0.70 \cdot \lambda^2 \phi_2) \tag{10}$$

where F_n = total drag force (negative skin friction) for one pile; $F_{n-s} = U\ h\ s \ldots\ldots s = 0.3\ q_u$; $\phi_1 = U\ h\ p_c$; $\phi_2 = w_u\ p_c^3/\gamma_s\ \gamma_d$; $\lambda = \gamma_d\ h/p_c$. Using the numerical values indicated above, we obtain for pile type B:

$$F_{n-s} = 2.83 \times 4.4 \times 0.3 \times 4.0 = 14.9 \text{ ton/pile}$$

$$F_n = 14.9 + 0.416\ (142.4 - 11.0) = 69.5 \text{ ton/pile.}$$

BEARING CAPACITY AND ALLOWABLE PILE LOADS

Actual Pile Loads

The three bridges give rise to average pile loads of 78 tons. The most heavily loaded piles have to carry 94 tons each.

Allowable Pile Load

The allowable pile load can be calculated according to Zeevaert (1959) in the following way:

$$Q'_{pa} = (AN_r p_{vl}/F) - F_n, \tag{11}$$

where A = cross-section of pile foot, p_{vl} = confining pressure at pile foot, N_r = bearing factor, F_n = negative friction force on pile, F = factor of safety. The shear parameters for the morainic underground have not been measured, but N_r = 35 is thought to be a conservative estimate of the bearing factor for the soil conditions encountered.

To be able to compare the three methods used, the corresponding vertical pressure p_{vh} has been calculated by means of Eq 6 for each method. Thus we obtain for pile type B with a safety factor $F = 2$:

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$$\text{Method 1: } Q'_{pa} = \tfrac{1}{2}[0.64 \times 35 \times (7.0 + 4.2)] - 63 = 61.5 \text{ ton/pile;}$$

$$\text{Method 2: } Q'_{pa} = \tfrac{1}{2}[0.64 \times 35 \times (7.4 + 4.2)] - 56.7 = 72.9 \text{ ton/pile;}$$

$$\text{Method 3: } Q'_{pa} = \tfrac{1}{2}[0.64 \times 35 \times (6.5 + 4.2)] - 69.5 = 49.2 \text{ ton/pile.}$$

An ordinary 90-cm-diameter bored pile was therefore not able to carry the load of P = 94 tons.

Had the influence of negative friction not been taken into account, one might have predicted the bearing capacity erroneously as follows:

$$Q_{pa} = \tfrac{1}{2}(0.64 \times 35 \times 15.9) = 177.5 \gg 94 \text{ tons.}$$

On the other hand the combined pile type A has, again with a safety factor of $F = 2$, the following allowable pile load

$$Q'_{pa} = \tfrac{1}{2}[0.64 \times 35 \times (9.5 + 4.2)] - 14.9 = 137.5 \text{ tons.}$$

In this case only Method 2 can be used.

Test Pile

The bearing behaviour of one pile was checked by a loading test. The settlement of the test pile at the nominal pile load of P = 94 tons is 0.5 cm; at a load $P + F_n = 94 + 14.9 = 109$ tons, it would be 0.7 cm. As the piles stand in one line with an average spacing of 2.7 m, i.e. 3 times pile diameter, the group action will be small. The settlement of the pile foundation is therefore estimated to be approximately 1 cm.

CONCLUSIONS

Three separate single-span bridges, each approximately

18 m long, carrying a main road, a narrow-gauge railway line, and a secondary road over a newly built access road to national highway N-1 (Berne–Zurich) were founded on a total of 14 piles. A pile foot of 90 cm in diameter bored pile was combined with a prefabricated pile shaft of 50 cm in diameter in order to reduce negative skin friction. This combination has increased the net allowable pile load from 72.9 tons to 137.5 tons or by 88 per cent. With ordinary bored piles of a constant diameter of 90 cm over their whole length, the negative skin friction would have increased the number of piles to at least 20. This would have caused not only considerable additional cost, but also reduced the pile spacing below 3 times pile diameter.

The method of calculation proposed by Zeevaert (1959) has proved to be easily adaptable to various soil conditions, as long as the shear characteristics of the soil are known in terms of effective stresses. Another recently developed formula (Elmasry, 1963) is only applicable to clayey silts for which the unconfined compressive strength, unit weight, and moisture content is known.

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REFERENCES

BUCKINGHAM, E. (1915). Model experiments and empirical equations. *Trans. American Society of Civil Engineers*, Vol. 37.

ELMASRY, M. A. (1963). The negative skin friction of bearing piles. Thesis presented to the Swiss Federal Institute of Technology, Zurich.

LEDERGERBER, R. (1961). Pressbeton-Bohrpfähle System Hochstrasser-Weise. *Schweizerische Bauzeitung*, No. 7.

TERZAGHI, K., and R. B. PECK (1948). *Soil mechanics in engineering practice*. New York, John Wiley and Sons.

ZEEVAERT, L. (1959). Reduction of point bearing capacity of piles because of the negative friction. *First Pan-American Conference on Soil Mechanics and Foundation Engineering* (Mexico) Vol. 3, pp. 1145–52.