Applicability of 1-g model tests in three cases

L'applicabilité d'essais sur modèles 1-g dans trois cas

E.FRANKE, Professor of Civil Engineering, Technical University, Darmstadt, FRG

SYNOPSIS: Among building materials soil is the most complex. Therefore model testing is in soil mechanics often the most accurate tool in order to derive load-deformation-laws. As scale effects are affecting the results lg-model tests (under simple gravity conditions) are applicable only in special cases. 3 examples are shown, esp. the pile group action problem for horizontal load.

Since LADANYI (1960) and DE BEER (1965) it is known that the bearing behaviour of soils is dependent on the mean normal stress.

$$\sigma_m = (\sigma_x + \sigma_y + \sigma_y)/3 \qquad (1)$$

This dependence is causing scale effects in model testing well known now, but mostly disregarded in the past. The need to control scale effects was the main impact to develop centrifuge model testing. But as centrifuge testing is rather expensive it may be of interest to show that conventional lg-model tests are applicable with the same accuracy at least in three cases.

Case 1:
Derivation of an ultimate limit state equation: If the failure mechanisms in the soil adjacent to a foundation element (behaving rigid in this state) are the same in model and prototype (e.g. like for active earth pressure) then the same limit state equation (accounting for the different geometrical and shear parameters) is valid for both cases. Scale effects do not matter.

Case 2:
Derivation of geometry-factors: The load-deformation-behaviour of foundation elements is affected by geometrical boundary conditions. The deviation in behaviour caused by changes in geometrical conditions can be referred to characteristic cases, i.e. they can be expressed as ratios or percentages of the behaviour in characteristic reference cases, e.g. by shape factors, by the slope inclination, by group action factors etc. Deriving these percentages experimentally by comparing the behaviour of foundation elements in the same scale, the results are valid independent of doing the tests in model or full scale, i.e. scale effects do not matter. The important presumption in this case is that the interesting load-deformation results obtained experimentally must be depictable by power functions with the same constant exponent resp. by parallel straight lines in log-log-scale.

Case 3:
Derivation of load-deformation-laws for prototypes by model tests:
The scale effect caused by equ.(1) now must be taken into account. With lg-tests this is only possible by investigating a model family, i.e. a group of foundation elements of the same dimensionless, but different absolute magnitudes. From the experimental results then the law can be derived acc. to which the behaviour is changing with magnitude accounting for equ. (1). The presumption is again like in case 2 that the interesting load-deformation functions must be parallel straight lines in log-log scale.

One example for each case show the applicability of these principles.

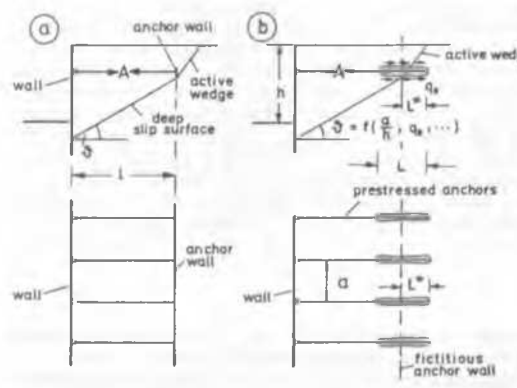

Fig. 1

Example 1 corresponding to case 1:
Quay walls and walls of pits of large width are normally back anchored. For the calculation of the required length l of anchors the failure mechanism of fig.1 is used. Originally this mechanism was derived for an anchor wall parallel to the quay or pit wall (see fig.1a and KRANZ, 1936). For prestressed anchors or anchor piles the same mechanism is used as an approximation assuming a fictitious anchor wall at the centre of the friction transferring part L of the anchor, i.e. L* = L/2 (see fig.2b). This assumption is more or less reasonable for prestressed anchors which have a limited friction transferring length L<l like in fig.2, but for anchor piles with friction transfer over the whole length l its accuracy was doubted. Indeed model tests showed that the actual length L* is more

or less deviating from L/2.

The derivation of L^* is shown in fig.2 in connection with the derivation of the minimum value of the allowable anchor force A_{min} for an anchor length l=16 m and L=8 m. The required friction transferring length L is calculated beforehand using the anchor force A from the wall calculation and the allowable friction resistance $q_{s,all}$ (always obtained by load tests)

$$L = A/\pi \cdot d \cdot q_{s,all} \qquad (2)$$

Then deep slip surfaces (e.g. 0...6) are drawn. For each of them the force polygon must be drawn (acc. to the Culmann-method) to find out for which polygon the minimum anchor force A_{min} is obtained. As shown in fig.2 in the force polygon (big drawn for the slip surface 3) the part of the anchor force acting outside the failure body

$$A_{0..VI} = L^*_{0..6} \cdot \pi \cdot d \cdot q_{s,failure} \qquad (3)$$

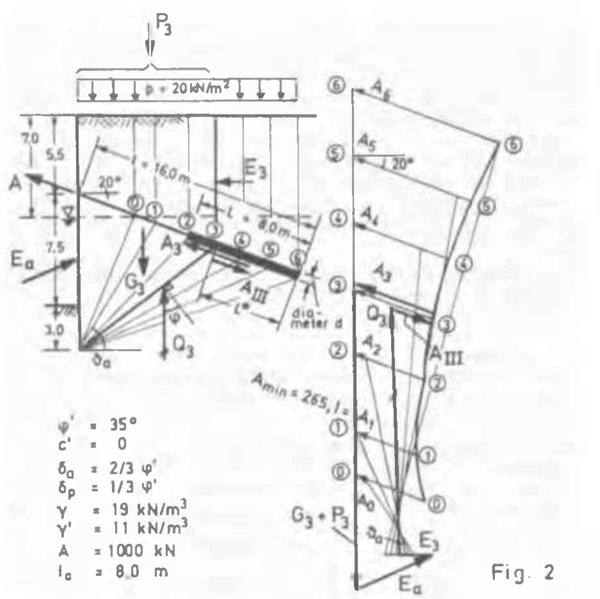

Fig. 2

must be taken into account. A_{min} is the minimum value of the curve connecting $A_0...A_6$.

In the example of fig.2 the minimum value A = 265.1 kN/m is showing that the slip surface 1 acc. to the active earth pressure is connected with the extremum, leaving the whole friction transferring length L outdside the friction body, i.e. $L_1^* = L$ and the assumption of the fictitious anchor wall at $L^* = L/2$ would be completely wrong.

This investigation has to be done repeatedly for different lengths l of the anchor to find out the required minimum of the length, however, always using the same values of L. The required safety factor is F = min A/A ≥ 1.5.

Another result of these model tests was the justification to neglect deviations from the plain strain case.

About 60 model tests were done approving that the limit state equation acc. to fig.2 is working. The result is valid for model tests as well as for prototypes. For more details and the analytical formulations see Heibaum (1987).

Example 2 corresponding to case 2:
Large bored or driven piles are mostly installed vertically because of costs; then horizontal loads on pile groups must be transferred by the lateral soil support of the piles. In comparison with single piles the interaction between the group piles deteriorates the load-deflection behaviour. This is of interest up to deflections of 0.1·d (d = pile diameter) which can be approximated by the power function:

$$\frac{u_0}{d} = A \cdot \left(\frac{H_0}{H_{ref}}\right)^n$$

$$\log \frac{u_0}{d} = \log A + n \cdot \log \left(\frac{H_0}{H_{ref}}\right) \qquad (4)$$

u_0/d, H/H_{ref} are the normalized movement resp. load at the pile head.

Experimental evidence shows that the 2 cases of fig.3 (which are extreme limitations) must be distinguished: rigid piles and ∞ long flexible piles. (In the latter case of elastic piles in non-linear behaving soil it is defined that the deflections do not reach the pile tip, the deflection curves for increasing loads then are self-similar to each other,s. Dietrich, 1982. Only in these 2 cases constant exponents n resp. n_∞ do exist.) By fig.4 to 11 the expeimental results are shown and the model test rules are derived.

Fig.4 to 7 approve that the test results are straight lines in log-log-scale acc. to equ.(4).

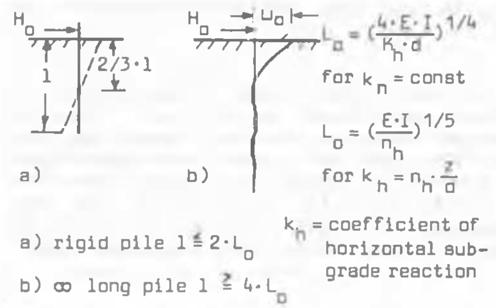

Fig. 3

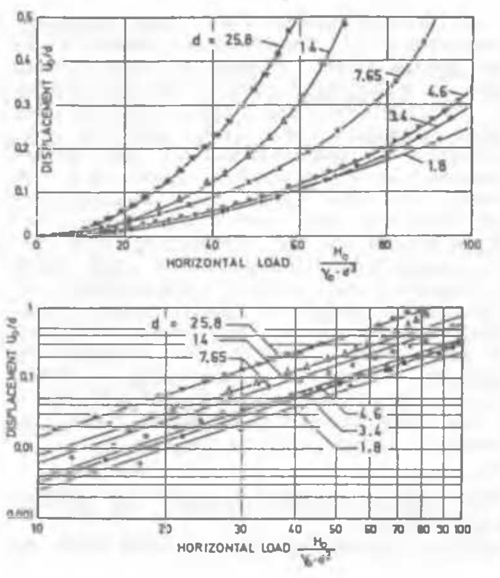

Fig. 4

On fig.4 the test results of a model family of rigid piles are shown which had equal dimensionless magnitude l/d, but different absolute diameter (d = 1.8 to 25.8 cm). With increasing d (and length l) σ_m (see equ.1) increases causing the shown change in load deflection behaviour resp. the parallel deviations in log-log-scale (see example 3, too).

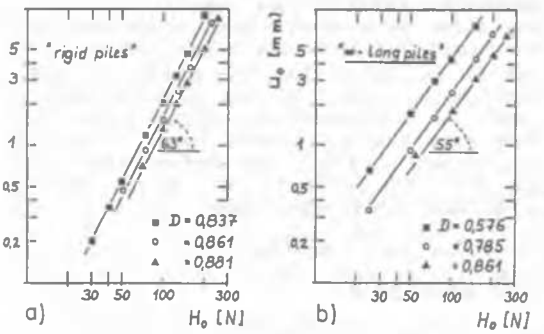

Fig. 5

On fig.5 it is shown that the exponents are constant, but different for rigid (n = tan 63°) and ∞ long piles (n_∞ = tan 55°). (For elastic piles with lengths shorter than in the defined case of ∞ length no straight lines in log-log-scale do occur; n is decreasing with increasing load between n and n_∞.) Furthermore it is seen that changes in density D are causing only parallel deviations in case of n resp. n_∞ beeing constant in log-log-scale. From these (and other) test results it can be concluded that n resp. n_∞ are constant (in homogeneous soil) for each soil type and independent of the actual soil condition expressed by D (or I_C for cohesive soils).

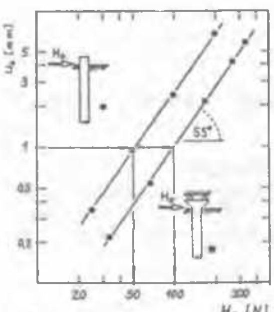

Fig. 6

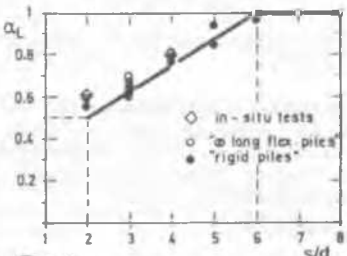

Fig. 8

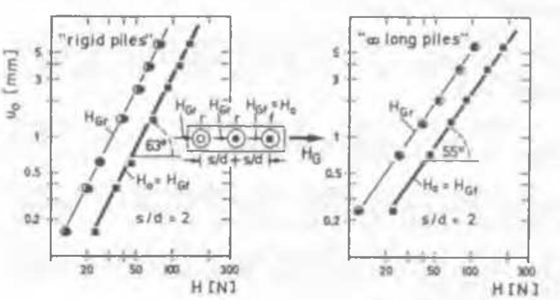

Fig. 7

On Fig.6 it is shown that the different behaviour of piles with free movable and fixed head is expressed by parallel deviations in log-log-scale again.

On fig.7 results of tests on a pile row with equal normalized pile distances s/d acted on by the group force H_G are shown. It revealed that the front pile behaves always like a single pile whilst all rear piles take the same but smaller load shares of H_G than the front pile at the same deflection (independent of the number of rear piles). The distance s/d is causing parallel deviations in log-log-scale again.

On fig.8 the results of all tests (including prototype tests of Schmidt 1986) with pile rows like the ones of fig.7 are gathered and shown by the load factor α_L = f(s/d) with

$$\alpha_L = H_{Gr}/H_0 \qquad (5)$$

H_0 = load of a single pile at the deflection u_0
H_{Gr}= load share of a rear group pile at the deflection $u_G = u_0$

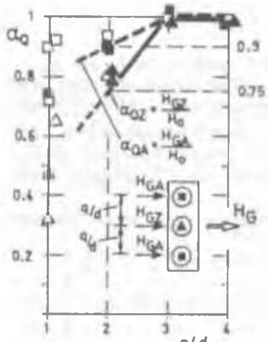

Fig. 9

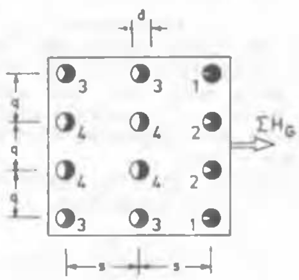

Fig. 10

On fig.9 the results of tests on pile rows across load direction with distances q/d are shown. Now it must be distinguished between the corner piles (index A) and the centre piles (index Z). It was experienced that the centre piles took smaller load shares of the group force H_G than the corner piles in comparison with a single pile of the same deflection ($u_0 = u_G$) independent of the number of centre piles, dependent on the pile distance q/d only. This is expressed by the load factors

$$\alpha_{QA} = H_{GA}/H_0 \qquad (6a)$$

$$\alpha_{QZ} = H_{GZ}/H_0 \qquad (6b)$$

From tests with groups consisting of pile rows along and across the group force direction results like on fig.11 were obtained. From these results it was learned that only four pile types do exist, schematically shown on fig.10:

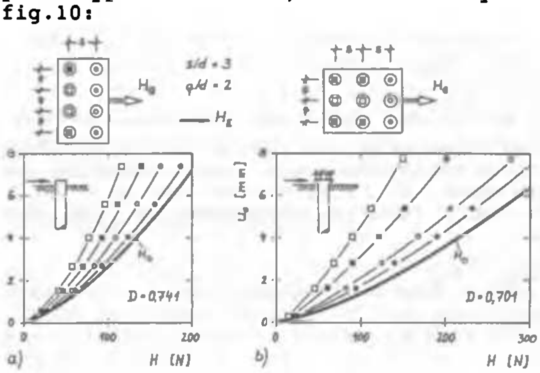

Fig. 11

- Type 1 $H_{GA} = \alpha_{QA}\cdot H_0 = \alpha_1\cdot H_0$
- Type 2 $H_{GZ} = \alpha_{QZ}\cdot H_0 = \alpha_2\cdot H_0$
- Type 3 $H_{GrA} = \alpha_{QA}\cdot\alpha_L\cdot H_0 = \alpha_3\cdot H_0$
- Type 4 $H_{GrZ} = \alpha_{QZ}\cdot\alpha_L\cdot H_0 = \alpha_4\cdot H_0$

For a individual group pile i the load factors $\alpha_i = \alpha_{1...4}$ can be gained by simple superposition. Generally it is

$$H_i = \alpha_i \cdot H_0 \tag{7}$$

The group action factor is consequently in case of j piles

$$R = \Sigma\alpha_i\cdot H_0/j\cdot H_0 = \Sigma\alpha_i/j \tag{8}$$

and the load share of a group pile is

$$H_i/H_G = \alpha_i\cdot H_0/\Sigma\alpha_i\cdot H_0 = \alpha_i/\Sigma\alpha_i \tag{9}$$

for $u_G = u_0$ independent of the magnitude of u_G.

It was learned in addition that the α_i-values are independent of the boundary conditions at the pile head and -at least approximately- of the bending rigidity, i.e. no significant difference was obtained for α_i-values of rigid and ∞-long piles.

Summing up all these results it was shown that in log-log-scale

$$\log\left(\frac{u_G}{d}\right) = \log f_1\left(\frac{\sigma_m}{\sigma_K}\right) + \log f_2(D, I_C) + \log f_3\left(\frac{s}{d}\right) + \log f_4\left(\frac{q}{d}\right) + \log f_5\ (\delta) + \dots + n\cdot\log\left(\frac{H_i}{H_{ref}}\right)$$

with σ_K as a suitable chosen reference value and δ = degree of fixation of pile head. Now conclusions can be drawn towards a model test rule: Equation (4) can be rewritten in the form

$$\frac{u_0}{d} = \underbrace{f_1\left(\frac{\sigma_m}{\sigma_K}\right)}_{\text{scale effect}}\cdot\underbrace{f_2(D, I_C)}_{\text{soil conditions}}\cdot\underbrace{f_3\left(\frac{s}{d}, \frac{q}{d}\dots\right)}_{\text{geometrical conditions}}\cdot\left(\frac{H_0}{H_{ref}}\right)^n \tag{7a}$$

Comparing test results of group and single piles of the same type, in the same soil (with regard to type and actual condition) and in the same scale, then f_1 and f_2 are the same and from equ.(7a) it follows

$$\frac{u_G}{u_0} = \frac{f_{3G}}{f_{30}} \cdot \left(\frac{H_i}{H_0}\right)^n \tag{7b}$$

i.e. f_1 and f_2 are cancelled, the scale effect and the effect of the soil conditions vanishes, results are valid for model tests as well as for prototypes. It is seen that for $H_i = H_0$ a constant ratio reveals independent of the exponent n

$$\frac{u_G}{u_0} = \frac{f_{3G}}{f_{30}} = \Delta_u = \text{const} \tag{7c}$$

For $u_G = u_0$ one obtains dependent on n

$$\frac{H_i}{H_0} = \Delta_u^{-1/n} = \Delta_H = \text{const} \tag{7d}$$

The values α_L, α_{QA}, α_{QZ} and its combinations are of the Δ_H-type. The neglection of their dependence on n resp.n_∞ is content of the mentioned approximation. Further tests of more accuracy may show refinements with different α_i-values for rigid and infinitely long piles.

At least it is shown that the moduli of subgrade reaction k_h (acc. to the Winkler medium theory) must be decreased for group piles applying once more the α_i-values: For $k_h = n_h\cdot z/d$ and ∞-long piles ($l>4\cdot L_0$) the deflection for the single pile is (with L_0 of fig.4 and C_y e.g. acc. to Matlock/Reese, 1960)

$$u_0 = C_y \cdot L_0^3 \cdot H_0/E\cdot I$$

and for the group pile

$$u_G = C_y\cdot L_{0Gi}^3\cdot(\alpha_i\cdot H_0)/E\cdot I$$

Introducing now $L_{0Gi} = (E\cdot I/n_{hGi})^{1/5}$ and $u_0 = u_G$

$$n_{hGi} = \alpha_i^{5/3}\cdot n_h \tag{8a}$$

For rigid piles ($l < 2 \cdot L_0$)

$$n_{hGi} = \alpha_i \cdot n_h \tag{8b}$$

Interpolation for $4 \cdot L_0 < l < 2 \cdot L_0$

For k_h = const the corresponding values are for ∞-long piles

$$k_{hGi} = \alpha_i^{4/3}\cdot k_h \tag{8c}$$

For rigid piles

$$k_{hGi} = \alpha_i\cdot k_h \tag{8d}$$

For more details see Klüber (1988), Franke (1988).

Example 3 corresponding to case 3:
This example was already shown in connection with fig.4. The derivation of a load-deflection-law from these test results was shown by Franke/Muth (1985).

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