# RESEARCH ON GROUND ANCHORS IN NON-COHESIVE SOILS

## Etude de tirants scellés dans des sols pulvérulents

by

H. OSTERMAYER and F. SCHEELE

Institute of Soil Mechanics and Foundation Engineering, Technical University, Munich/Fed. Rep. Germany

### SOMMAIRE

Des essais d'ancrages en vraie grandeur sur chantier ont été effectués dans des massifs de sables graveleux compactés à différentes densités. Des jauges de déformation collées sur des barreaux d'acier donnent la distribution des contraintes dans la zone fixe d'ancrage. Ces dernières augmentent progressivement avec la force appliquée en tête d'ancrage jusqu'à la rupture. Afin d'estimer par des essais standards in situ, l'influence du type et de la densité du sol sur la capacité portante du tirant d'ancrage, on a présenté le rapport entre la charge critique et le nombre de corps d'essais de pénétration (marteau de 50 kg).

### SUMMARY

Full scale field tests on anchors have been performed in one and the same gravelly sand compacted to various densities. During the pull-out tests the distribution of skin friction along the fixed anchor length was determined from strain gauge measurements. The results of 5 series of tests comprising 30 anchors complement a previously published design chart and give an additional correlation between carrying capacity of anchors and number of blows of penetration tests. The variation of skin friction along fixed anchor length with increasing load or with load kept constant over a longer period of time, helps to explain the influence of soil density and fixed anchor length on the carrying capacity and longterm behaviour of anchors.

## INTRODUCTION

The last Conference on Diaphragm Walls and Anchorages (London 1974) and the Seminar on the same topic (London 1976) showed that current practice in the field of ground anchors is ahead of theory and that there is urgent need for a proper understanding of the behaviour of the anchors and the surrounding ground under working conditions (C.P. Wroth, 1975). Design charts, which are based on field test results of about 300 anchors (H. Ostermayer, 1975) may help to estimate carrying capacity of anchors in relation to fixed anchor length in certain soil conditions. However, the main factor of soil density influencing the carrying behaviour of anchors in non-cohesive soil has not yet been tackled systematically in field tests.

In 1975-1976 a major research program was carried out in Munich to examine the influence of soil density on the carrying capacity of anchors. In order to have a better insight into the carrying behaviour of anchors, research aimed at investigating the distribution of stresses for different anchor lengths. The effect of time on the variation of stresses was also studied. The so sought information should then help to explain the various important factors that influence the carrying capacity and long-term behaviour of anchors.

## TEST PROGRAM

On a test site five series of six anchors each were installed and pull-out tests were performed. A schematic arrangement of the test pit is shown in Fig. 1. The dimensions of the test pit were about 5 × 10 × 10 meters. A rigid concrete wall was used as abutment for the pulling jack.

The anchors of the first series were installed in the existing soil, which was sandy gravel of high density. For the following four test series the soil was replaced by gravelly sand. The grain size distribution curve of the test soil is given in Fig. 1, the coefficient of uniformity was U = 8 to 10. The density of the sand was varied for each series. After every test series the soil was removed and compacted again in layers of about 30 cm height to a desired uniform density with the help of vibrators. The compacted sand carried a surcharge of about 2 m gravel. For each series the soil density was checked by 8 standard penetration tests (SPT) and 4 dynamic penetration tests (50 kg hammer weight and 15 $cm^2$ cone area). In addition unit weight and density index were determined for at least 6 samples.

A temporary anchor construction (Type A) was used with a total length of 9 m and an inclination of

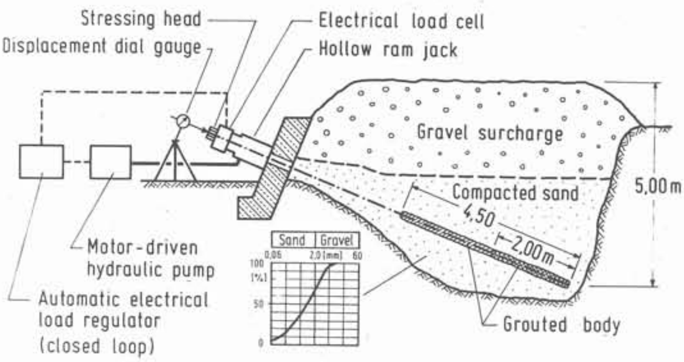



Fig. 1. — **General arrangement of full-scale anchor test set-up.**

about 20°. As shown in Table 1 series 2, 3 and 4 had bond lengths of tendon $L_v$ of 2.0 m and 4.5 m respectively. The bore hole diameter of 89 mm and the grouting pressure of about 0.5 MN/m² was kept constant. The anchors of series 1 and 5 had a bond length of 3.0 m. For these anchors different bore hole diameters (76 mm to 114 mm) and different grouting pressures (0 to 5 MN/m²) were used.

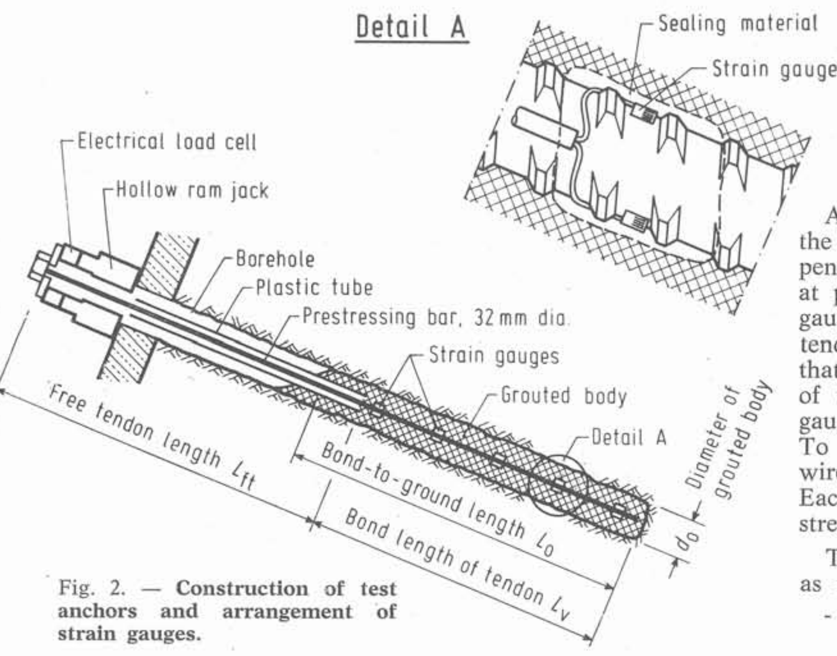



Fig. 2. — **Construction of test anchors and arrangement of strain gauges.**

TABLE 1

**Anchor and soil data for 5 series of tests with 6 anchors each**

| | Test series No | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| Anchor data | Prestressing steel tendon | 4 × 16 mm dia. | 32 mm dia. | | | |
| | Casing diameter (mm) | 89 | | | | 76.8 114 |
| | Grouting pressure (MN/m²) | 0 to 5.0 | 0.5 | | 0.5 and 1.0 | 0.5 and 2.0 |
| | Bond length of tendon (m) | 3.0 | 2.0 and 4.5 | | | 3.0 |
| Soil data | Soil type | sandy gravel | gravelly sand | | | |
| | Density index D | 1.1 | 1.14 | 0.76 | 0.28 | 0.8 |
| | Dynamic penetr. test (N/10 cm) | > 80 | 76 | 20 | 2 | 30 |
| | Standard penetr. test (N/30 cm) | > 130 | 120 | 43 | 11 | 60 |

A total of 9 anchors were specially prepared for the pull-out tests. In the laboratory temperature compensating strain gauges were attached to the steel bar at pre-selected points. A typical arrangement of the gauges along the bond length of tendon and the free tendon length is shown in Fig. 2. It is pointed out that the gauges were mounted on the profiled sides of the bar, which were carefully treated before the gauges were cemented to the steel-bar (see Detail A). To protect against the grouting, the strain gauges and wire connections were given several waterproof coatings. Each gauge was calibrated in the laboratory by stressing the steel bar.

The procedure of the installation of the anchors was as follows:

- the steel casing was driven to the depth of about 8 m;

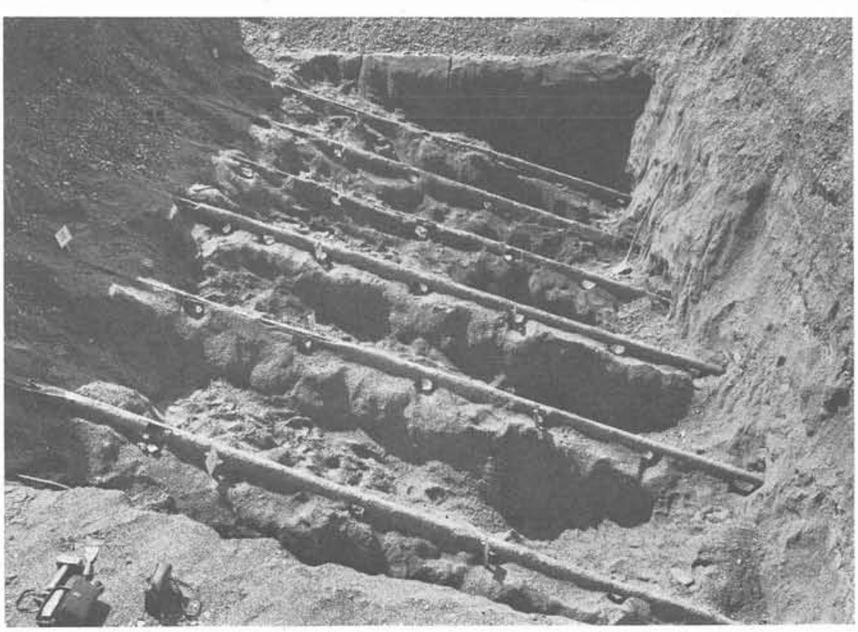


Fig. 3. — **One test series in sand dug out for careful examination.**

- the anchor bars with fixed spacers (along the bond length) were inserted into the casing;
- the bore hole was grouted under constant pressure along the planned bond length while retracting the casing simultaneously;
- in the upper part of the bore hole the remaining cement suspension was flushed out with water;
- the pull-out tests were performed after a period of about 14 days.

The pulling force was applied in steps by a hollow ram jack until the failure load was reached. Between each step, loading and unloading cycles were applied according to the German Standards for Fundamental Tests (DIN 4125).

For each loading step, the displacements of the anchor head were measured manually through the readings of the dial gauge, while the measurements of the strain gauges were recorded automatically.

At certain loading steps, the force was kept constant with the help of an electric-hydraulic regulator for a period up to 2 months.

All the anchors were dug out after the pull-out tests (see Fig. 3) and the grouted bodies were carefully examined. The diameters and lengths of the grouted bodies were measured and the surrounding soil conditions were checked.

## TEST RESULTS

### Load Carrying Capacity

In Fig. 4 the failure loads (ultimate load carrying capacity $T_f$) of the five test series are presented in relation to the length of the fixed anchor (bond-to-ground length $L_o$). The results confirm the validity of the previously published design chart (H. Ostermayer, 1975) and supplement the chart with additional curves for loose gravelly sand and very dense sandy gravels. The diagram shows the smallest linear increase of carrying capacity with increasing bond length for loose sands. Contrary to this in case of dense sands, the greatest increase is encountered for smaller lengths which then tapers off steadily with increasing length. With lengths of more than 6 to 7 m the increase of carrying capacity per m length will probably be the same whether the anchors are installed in loose or in dense sands and gravels. The reason for this behaviour is a progressive failure mechanism which will be investigated in the following paragraphs.

In addition it should be noted that compared to the large influence of the soil density on the carrying capacity the influence of grouting pressure (minimum pressure 0.5 MN/m²) as well as the influence of the diameter of the grouted body (diameter of 10 to 15 cm) seems to be negligible.

### Distribution of Skin Friction

As a result of strain gauge measurements at anchors in dense sand a typical distribution of tensile forces in the steel tendon is represented in Fig. 5. The decrease of forces from the front part to the rear of the bond length corresponds with the load transmission from the tendon into the grouted body. As

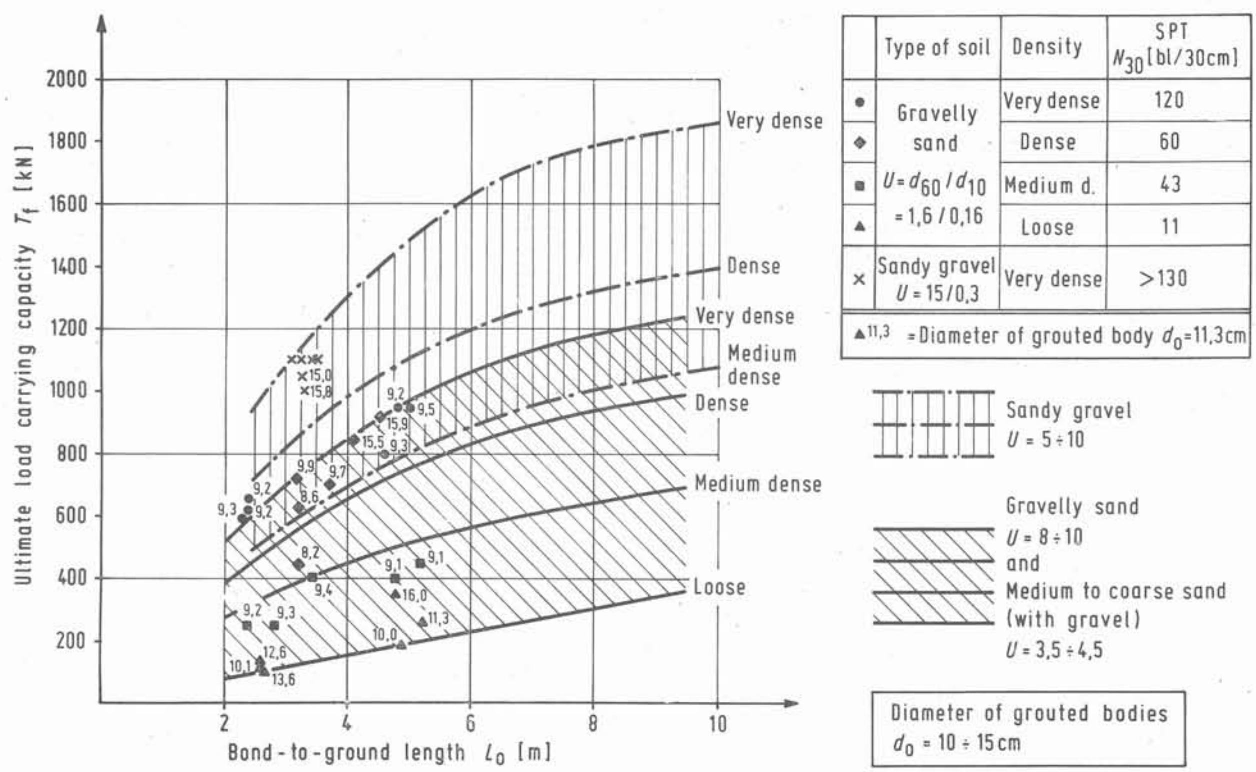



Fig. 4. — **Carrying capacity of anchors in sandy gravel and gravelly sand showing influence of soil type, density and bond-to-ground length.**

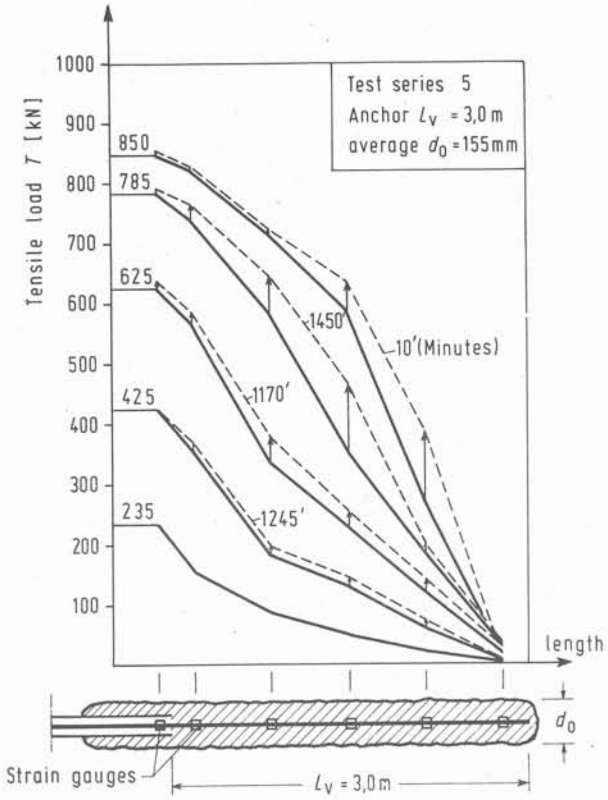



Fig. 5. — **Distribution of tensile load along bond length of tendon (grouted body in dense sand).**

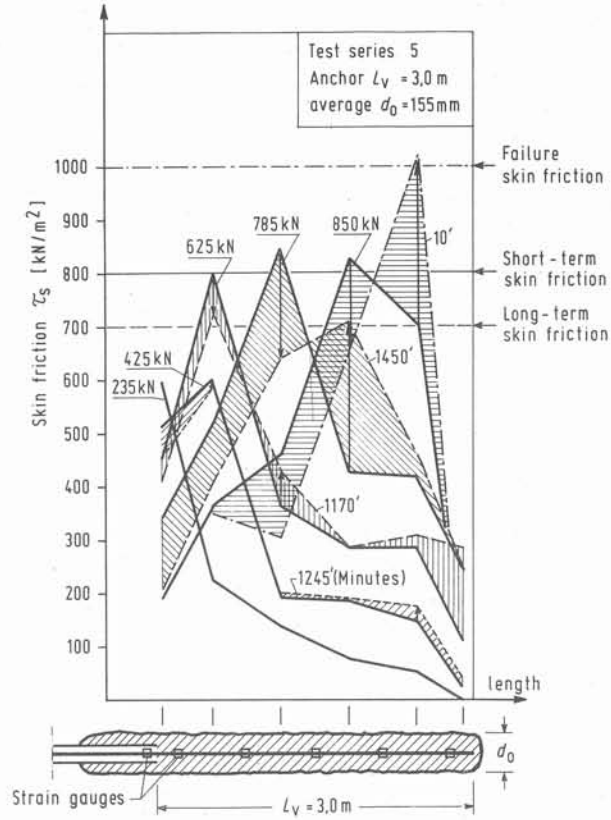



Fig. 6. — **Distribution of skin friction in the soil/grout interface along bond length (grouted body in dense sand).**

shown in Fig. 5 the forces in the tendon increase not only when the applied test load at the anchor head is increased (here 5 loading steps), but also when the load is kept constant for a period of time (in the example of Fig. 5 three loading steps were kept constant for about one day).

The difference in the values of forces measured at two adjoining points divided by the circumference area of the grouted body (grout-soil interface) gives the value of «skin friction». These calculated values are shown in Fig. 6. Obviously there are maximum skin friction values, the location of this maximum moves from the front part of bond length towards the anchor end when the test load is increased. The reason is that the elastic deformations of the steel tendon cause progressive displacements in the grout/soil interface. This progressive displacement causes the shear resistance of the dense sand to shift beyond the peak point into the region of lower residual shear values.

It is worth noting that at each loading step the same maximum friction value is reached for a short time (maximum «short-term skin friction») and that this value tapers off with time until a certain «long-term skin friction» value is not exceeded at any point along the bond length. As the applied test load is kept constant during one loading step, a decrease of skin friction in the front part of bond length will result in a corresponding increase of skin friction in the rear part. This kind of balancing could not be attained for the last loading step of 850 kN, so that failure occurred within 10 minutes.

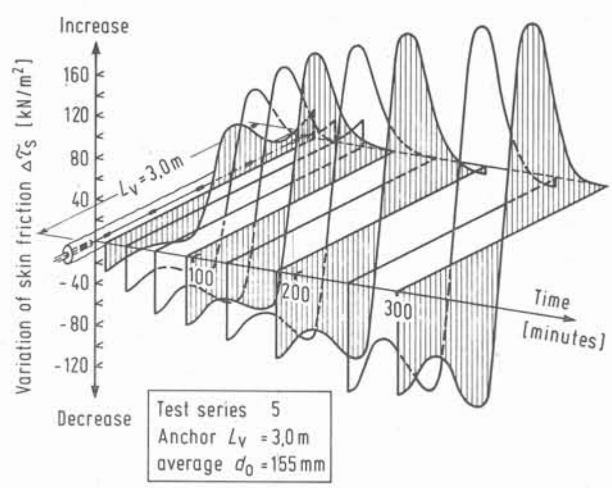



Fig. 7. — **Variation of skin friction along bond length over a period of 300 minutes (load of 785 kN was kept constant).**

For the load of 785 kN this balancing over a period of 300 minutes is illustrated in the three-dimensional plot of Fig. 7. The decrease and increase of skin friction along the bond length is shown for several points of time and demonstrated with the aid of shaded areas. It is pointed out that for any particular time the shaded area showing friction decrease in the front part is equal to the shaded area showing friction increase in the rear part.

When evaluating skin friction $\tau_s$ in the grout/soil interface from the bond stresses $\tau_b$ in the steel bar/grout interface (Fig. 8) it must be taken into consideration that the bond-to-ground length $L_o$ is longer than the bond length of tendon $L_v$. In the front part of the grouted body there is no steel/grout bond (due to the plastic tube), so that the forces resulting from skin friction $\tau_s$ in this area are transmitted further back and cause a concentration of bond stresses $\tau_b$ in the front part of bond length of tendon. These very high bond stresses which are obtained by strain gauge measurements are schematically shown in Fig. 8, where the existing $\tau_b$ is converted to an equivalent skin friction $\tau'_s$ through the factor $d_b/d_o$, ($\tau_s' = \tau_b \,.\, d_b/d_o$).

The actual skin friction is achieved by equalizing the equivalent value over the entire length including the front part of the grouted body (see area $\Delta$ F) in order to get the actual skin friction $\tau_s$.

In the case of dense sands the limit values of skin friction, max $\tau_s$, are effective along a relatively short length. For short anchors this length will correspond almost with the whole bond-to-ground length (Fig. 8 a). For long anchors this length of max $\tau_s$ is only a part of the bond-to-ground length. The location of this part shifts towards the anchor rear when the test load is increased (distribution of skin friction near failure load is shown in Fig 8 b). Assuming that the limit value max $\tau_s$ is identical for different bond-to-ground lengths, the mean values (mean $\tau_s$) for long anchors are smaller than for short anchors. This has already been presented in the chart of Fig. 4 in terms of carrying capacity versus bond length.

For the last loading step before failure load was reached Fig. 9 shows the «long-term skin friction» values of all test anchors in sand, which were equipped with strain gauges (diameter of grouted body being 9 to 12 cm).

For dense and very dense sand the skin friction values obtained experimentally fit very well with the qualitative distribution of skin friction of Fig. 8, thereby confirming the assumption made. The limit values, max $\tau_s$ of shorter anchors ($L_v = 2$ m) however are likely to exceed the corresponding values of longer anchors ($L_v = 4$ m). The difference may be partly traced back to the larger radial confining pressures in the front part of anchors.

In loose and medium dense sand the skin friction is found to be more or less constant along the whole bond-to-ground length. This corresponds with the stress-strain-behaviour of the sand for these densities.

The decisive influence of soil density is obvious in these tests when for example in the case of long anchors one compares the limit values of skin friction for loose and medium dense sand (about 150 and 300 kN/m$^2$ respectively) with the limit values for very dense sand (about 800 kN/m$^2$).

These high values of skin friction are mainly the result of an interlocking or wedging effect due to the dilatation of soil (E. Wernick, 1977). The peak values of up to 1 300 kN/m$^2$ do not represent the actual skin friction but the equivalent skin friction as already explained in Fig. 8.

## Penetration Tests and Carrying Capacity of Anchors

As the density of non-cohesive soils is in current practice very often indirectly determined by penetrometer tests, it was decided to plot a diagram showing

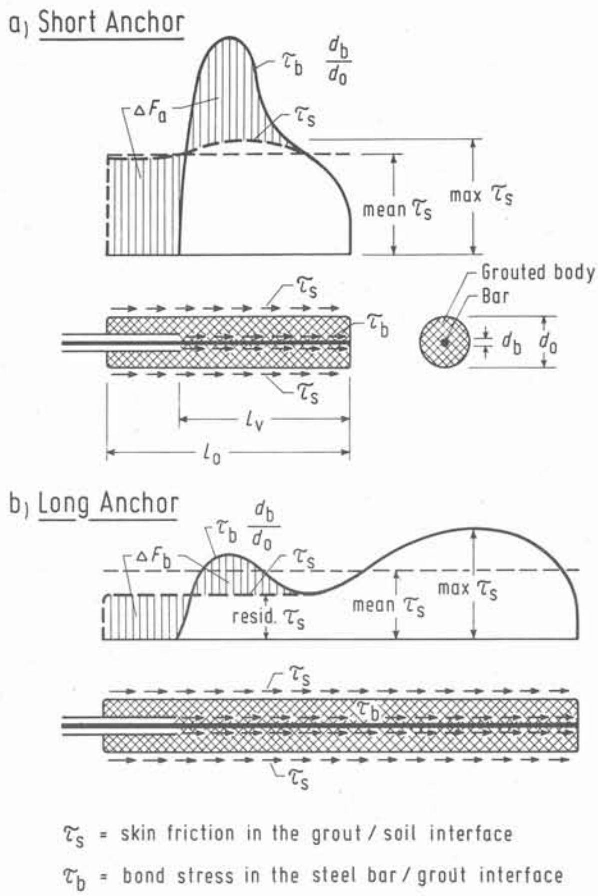



Fig. 8. — **Qualitative distribution of skin friction and bond stresses for short and long anchors in dense ground at ultimate load.**

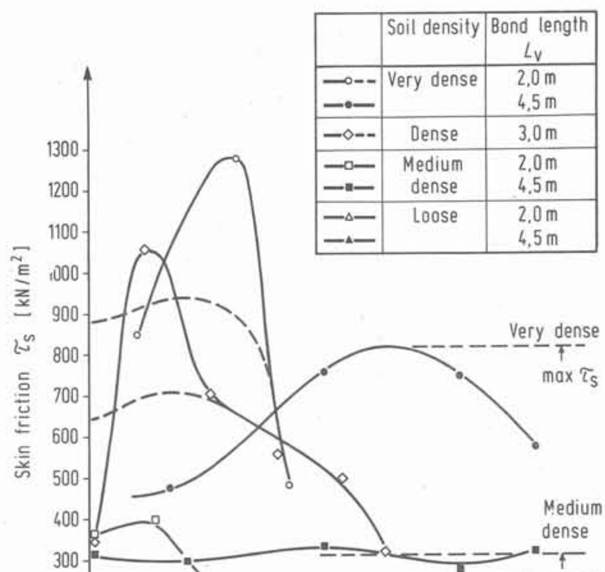
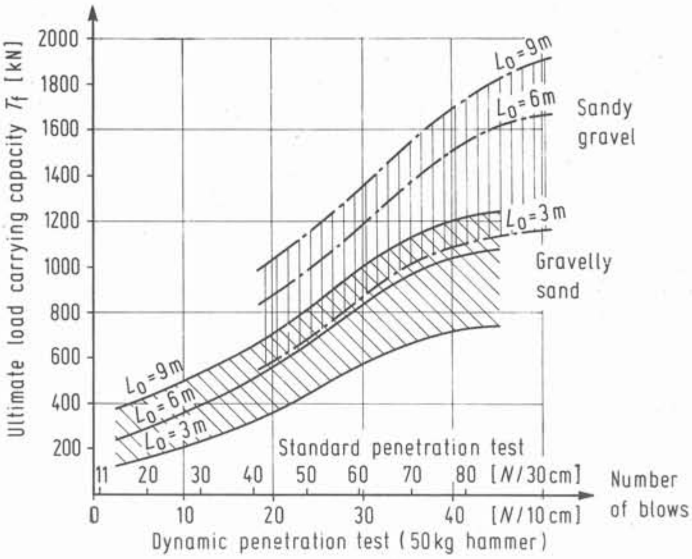



Fig. 9. — **Distribution of long-term skin friction $\tau_s$ at ultimate load in relation to bond length $L_v$ and soil density.**
**Grouted bodies ($d_o$ = 9.1 — 12.6 cm) in gravelly sand.**




carrying capacity of anchors in relation to penetration resistance. Fig. 10 shows the results of the 30 test anchors supplemented by additional results of other *in situ* tests (fundamental tests of different anchor systems). This chart may be used for a rough estimation of the carrying capacity of anchors which are properly installed. It must be emphasized however, that only for the 30 anchors in the test pit both dynamic penetration tests (50 kg hammer) and standard penetration tests (SPT) have been carried out. For the other *in situ* tests only dynamic penetration tests were used. The chart may be adjusted and extended for sandy gravel soils depending upon the results of additional future test data.

Fig. 10. — **Relationship between carrying capacity, bond length of anchors and dynamic penetration resistance in two types of non-cohesive soils.**

## CONCLUSIONS

To investigate the important influence of the density of non-cohesive soils on the carrying behaviour of anchors it was for the first time that full scale field tests were performed in one and the same soil compacted to different densities. Under these controlled conditions the exact distribution of skin friction along the bond length could be calculated from strain gauge measurements.

On the basis of the results of the 5 series of tests comprising 30 anchors, the original design chart of 1975, showing carrying capacity versus bond length for different soil conditions, has now been complemented. Furthermore a diagram is presented from which it is possible to estimate carrying capacity of anchors with different bond lengths from the number of blows of standard penetration tests (SPT) and dynamic penetration tests. When using one of these charts it must be borne in mind that certain fluctuations in test results are possible due to the inhomogeneity existing in the soil at site, even when the anchors have been properly installed.

The different shapes of distribution of skin friction which were derived through measurements help to give an explanation for the influence of bond length of anchors and density of soils on the carrying capacity as shown in the design chart (Fig. 4).

The different shapes of distribution of skin friction of skin friction along the bond length with increasing load paved the way for the inclusion of valid assumptions in the calculation of carrying capacity in terms of soil constants.

In addition the variation of skin friction with respect to time was measured for several loading steps. The results (shown for only one anchor in Fig. 7) will provide a basis for anticipating the long-term behaviour of anchors.

## ACKNOWLEDGEMENTS

The tests were performed at the Institute of Soil Mechanics and Foundation Engineering, Technical University, Munich/Germany, guided by Prof. Dr-Ing. R. Jelinek. The authors are grateful to Bundesministerium für Raumordnung, Bauwesen und Städtebau, to Bundesministerium für Verkehr, to Innenministerium des Landes Nordrhein-Westfalen and to Hauptverband der Deutschen Bauindustrie for financial support.

## REFERENCES

Deutsche Normen. — « Verpressanker für vorübergehende Zwecke im Lockergestein : Bemessung, Ausführung und Prüfung ». DIN 4125, Blatt 1, Beuth Verlag, Berlin (1972).

OSTERMAYER (H.). — «Construction, Carrying Behaviour and Creep Characteristics of Ground Anchors», Proc. Conf. on Diaphragm Walls and Anchorages, London 1974, *Institution of Civil Engineers,* London, pp. 141-151 (1975).

WERNICK (E.). — «Stresses and Strains on the Surface of Anchors», 9th Int. Conf. SMFE, Specialty Session 4: Ground Anchors (1977).

WROTH (C.P.). — Discussion on Papers 18-21 (Report). Proc. Conf. on Diaphragm Walls and Anchorages, London 1974, *Institution of Civil Engineers,* London, pp. 165-169 (1975).