Notwithstanding these observations, the American standard penetration test is so valuable that an attempt should be made to establish a quantitative relationship between the two methods. Alternatively British boring standards could be changed and hole diameters reduced to comply with American practice. The latter will eventually overtake British engineers in any event because of the changing quality of manual labour in handling heavy weights.

THE CORRELATION OF CONE SIZE IN THE DYNAMIC CONE PENETRATION TEST WITH THE STANDARD PENETRATION TEST

(Mohan, D., Aggarwal, V. S. & Tolia, D. S. (1970). *Géotechnique* **20**, No. 3, 315–319)

G. Meardi, Politecnico di Milano, Italy

The Writer began to use the dynamic penetrometer with a conical point in 1955 and since then it has been widely adopted in Italy. The penetrometer is the one suggested by Terzaghi and Peck (1948, pp. 280–281). It has a 51 mm diameter at the point and the 73 kg hammer

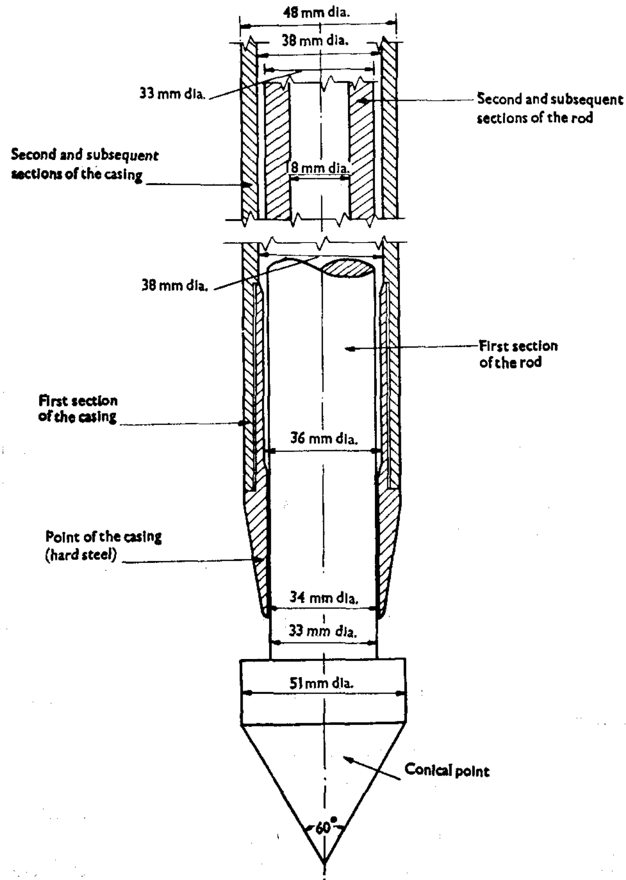


Fig. 1. Conical point penetrometer 51 mm dia. with casing 48 mm dia.

falls 75 cm. It is widely used not only because the test is cheap but also because, when alluvial deposits are concerned, it is always necessary to get as continuous a penetration record as possible (Terzaghi and Peck, 1948, p. 317).

The Writer realized that the tests might be misleading, especially with clayey soils, if the casing is not used. The resistance to driving due to adhesion or friction along the rod of the penetrometer soon proved to be prevailing over the resistance of the point, with the result that soft soils seemed very resistant (Meardi, 1957, Fig. 6). In the Writer's opinion the soil displaced by the point, if it is sandy, may partly find place in the surrounding soil and increase its density, but if it is clayey the water which fills the pores of the soil prevents the increase in density of the surrounding soil, so that the soil displaced by the point causes the soil above to shrink around the rod which follows the point itself. Therefore ever since adopting this penetrometer the Writer has always used a casing. The casing sometimes meets strong resistances, and in some circumstances dictates stopping the test even in the upper strata of subsoil. To reduce this resistance a casing is used with an outer diameter of 48 mm, instead of 51 mm which is the diameter of the point. The technical difficulties due to the small space available for the casing have been satisfactorily solved by making proper threaded junctions between the elements of the casing.

The records of the resistance to driving of the casing are also useful because, when the soil is sandy or gravelly, the resistance of the casing increases far less than in a clayey soil, especially if it is soft. This gives indicative information on the nature of the soil.

The Writer has studied the relation between the resistances of the point measured and the resistances of the standard penetration test and has ascertained that in the clayey soils such a relation is variable. The resistances measured by means of a conical penetrometer in clayey and silty clayey soils are therefore only indicative and require further survey by means of borings or vane test.

In sandy soils there is a fairly constant relation or practical equality at all elevations between the resistances measured with the conical penetrometer (i.e. the casing) and those measured with the standard penetration test. This equality is being verified in a study by Dott. Ing. P. Meardi, a colleague of the Writer, on the various types of penetrometer used in Italy. He compares several pairs of tests made at a spacing ranging from 1 to 2 m, with a conical penetrometer and standard penetration tests. The results are shown in Figs 2–6 which also show the profile of the soils.

The Writer obtained the same result in sandy or sandy-gravelly soils, including soils with particles up to 4/5 cm diameter.

However, the results of the tests given by the Authors prove that with sandy soils the friction along the rod causes an increase in the resistance to penetration such as to indicate that it is advisable not to drive deeper than 6 m. Using the casing the Writer was able to carry out satisfactory penetrometric tests down to 60 m. An example is the survey relative to the foundation on piles with large diameter of the bridge over the river Po for the Bologna–Padova motorway. When the casing met too great a resistance (which happened especially driving through silty-clayey strata) the test was interrupted and a drilling made with a diameter slightly greater than that of the casing. Bentonitic mud filled the space, and then the penetrometer was redriven with its casing into the enlarged hole.

With this procedure, on resuming the test, the resistance met by the penetrometer for about one metre was often much smaller than that recorded at the end of the test on the previous portion. This is important for a correct evaluation of the resistances measured at the bottom of a hole.

It can be concluded that it is not necessary to modify the 51 mm diameter of the conical penetrometer suggested by Terzaghi and Peck (1948) because in sandy soils, using the casing, the same resistance is obtained as can be recorded by the standard penetration test and surveys can be made down to depths much greater than the 6 m stated as the deepest elevation

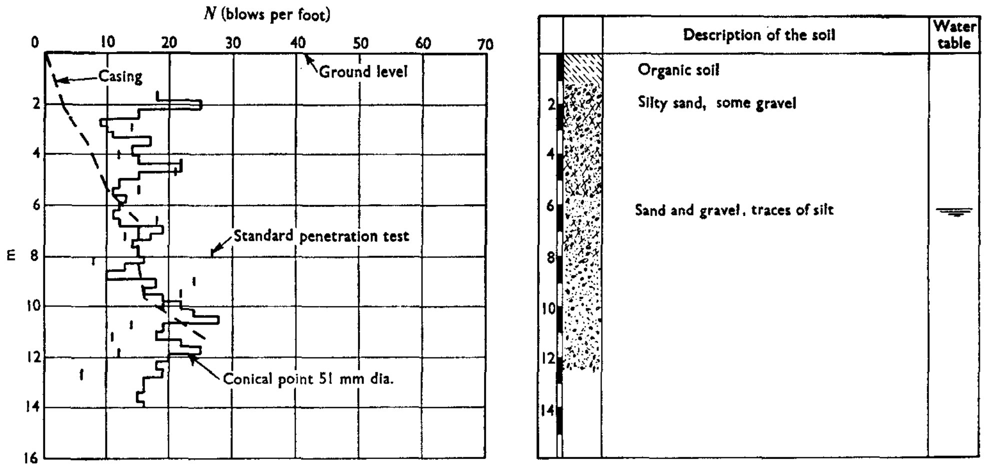


Fig. 2

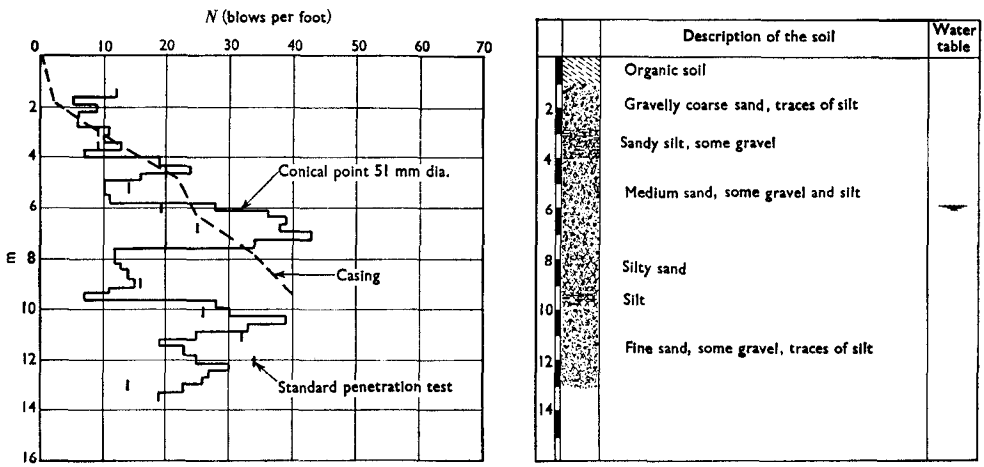


Fig. 3

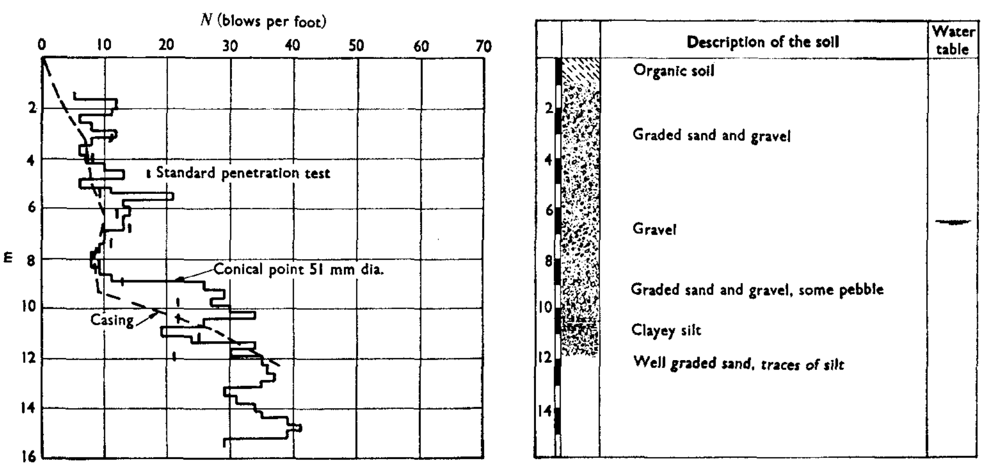


Fig. 4

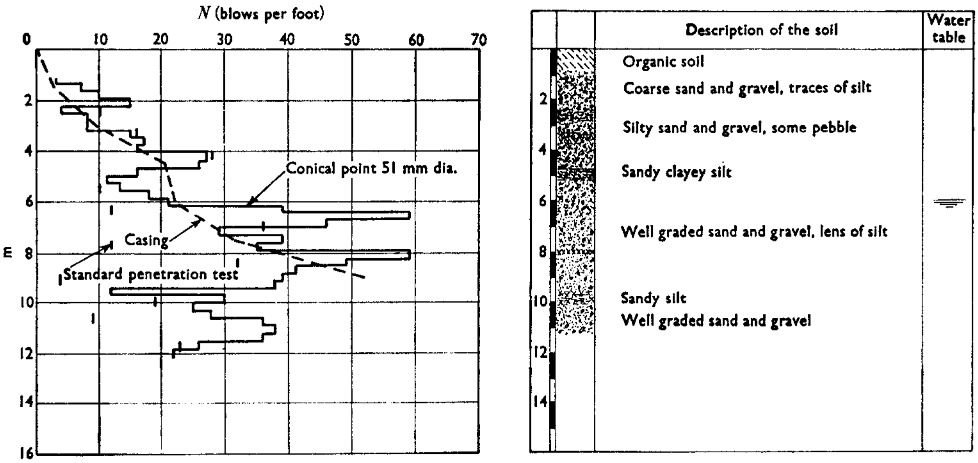


Fig. 5

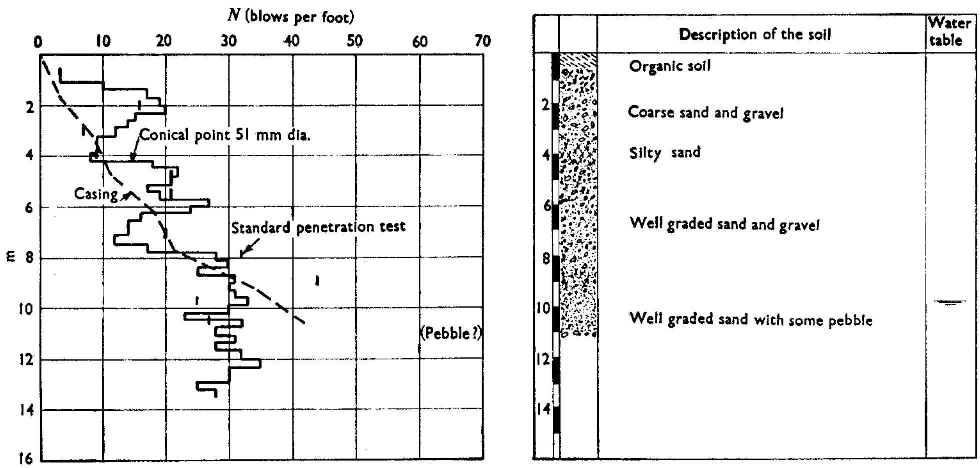
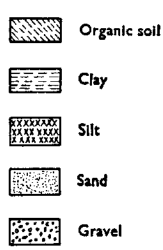


Fig. 6



Figs 2–6. Penetrometric diagrams of dynamic tests carried out with conical point and casing with the sampling spoon of the standard penetration test. Lateral profiles of the tested soils

by the Authors. This depth is inadequate for nearly all the surveys for foundations if the upper layers of the ground are not firm and especially if piles are to be used.

The Writer has reached depths greater than 60 m, but it is common in Italy to make tests down to 25–30 m, even when the resistance is so strong as to call for more than 700 blows per foot. In these cases when N is greater than 80 the layer can be penetrated by reducing the fall of the hammer to about 10 cm if necessary. In this way there is no damage to the joints of the equipment. The number of the blows per foot is not easily computed with respect to the number of blows necessary with the ordinary fall, but when the resistance calls for more than 100 blows per foot, the accuracy of their measurement is no longer of consequence, because it is always a matter of compacted sand or gravel that can bear maximum loads.

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J. W. Gadsby, C.B.A. Engineering Ltd, Vancouver

The Authors' results on shallow depth dynamic cone tests indicate significantly the effects of skin friction and cone area upon the blow count. Their Fig. 6 shows that the blow count decreases as the cone sizes increase from 43·75 to 62·5 mm. The Writer suggests that this effect can be attributed to a reduction in skin friction on the drill rods. However, the in-

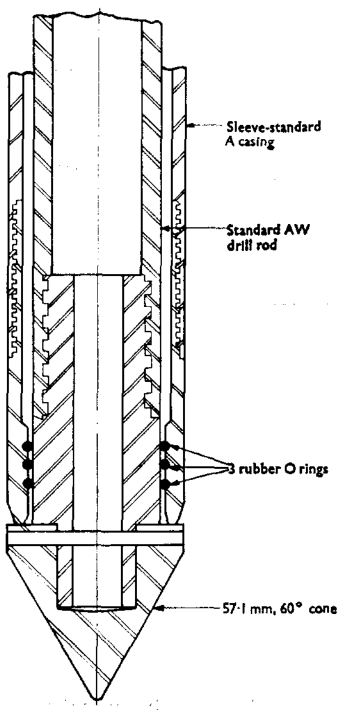

Fig. 1

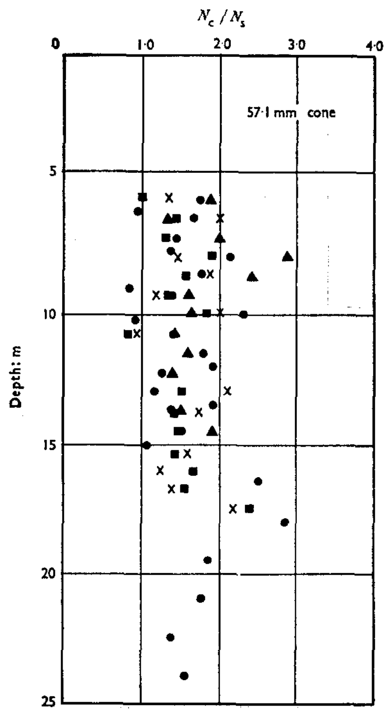

Fig. 2

crease in blow count with cone sizes of 68·7 and 75·0 mm can be attributed to the greater effect of cone area upon the driving resistance than the skin friction.

As standard penetration tests are made in holes supported by casing or drilling fluid, there is normally no effect of skin friction on the drill rods. It would therefore be more useful to make cone tests where skin friction on the rods is minimized for correlation with standard penetration tests.

For this reason a sleeved dynamic cone has been used on two sites in Vancouver in uniform sand deposits. The apparatus is shown in Fig. 1. The cone is 57·1 mm dia. with a 60° apex angle. The test procedure consists of driving the cone 60 cm ahead of the casing. The casing is driven to no less than 10 cm from the cone and the driving procedure is repeated. A 140 lb hammer is used with a 76 cm free fall. After driving the cone and casing, generally to a depth of about 10 m, a B size casing is washed down over the A casing to reduce the skin friction between casing and soil. By this procedure the cone has been driven to depths of 25 m.

The ratio of N_c/N_s, the blow count on the dynamic cone divided by the standard penetration test blow count, is shown in Fig. 2 for five tests made at two sites in Vancouver. All tests reported are below the water table. The results for tests above the 5 m depth are not included as the materials are not uniform sands.

The results shown in Fig. 2 indicate that there is a good correlation between sleeved cone and standard penetration tests up to 25 m depth penetrated. The N_c/N_s ratio for the 57·1 mm cone is approximately 1·5.

* * *

D. Mohan, V. S. Aggarwal and D. S. Tolia

The Authors are in general agreement with Gadsby for the explanation of the behaviour of the blow count with the cone sizes given in Fig. 6 of the Paper. The initial decrease in the blow count is due to the increase in cone size from 43·75 to 62·5 mm consequent upon the reduction in skin friction due to the increasing gap between the shaft and the base of the cone. Further increases in blow count for cone sizes of 68·7 and 75·0 mm are due to the relative dominance of the effect of increased cone sizes over the frictional resistance.

A study of the dynamic cone penetration test has been carried out (Mohan and Sen Gupta, 1970) at the Central Building Research Institute, Roorkee, India, where friction is eliminated on the shaft by circulating bentonite slurry. There is a good agreement between the dynamic cone and the standard penetration test. This has been checked on a number of construction sites (Fig. 1).

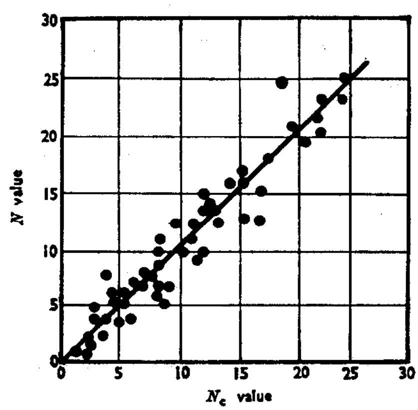

Fig. 1

The Authors are glad that Gadsby also obtained a ratio of $N_c/N_s = 1{\cdot}5$ with his sleeved dynamic cone. It agrees with the ratio given in the Paper for shallow depths. The Authors have further observed that it may be difficult to push the sleeved cone in dense sands. In one study it was difficult to penetrate even a simple cone and the blow count was about 60–70 over each 30 cm penetration.

REFERENCE

MOHAN, D. & SEN GUPTA, D. P. (1970). The dynamic cone penetration test. *Civil Engineering* **40**, No. 2, 49–50.

FULLY DEVELOPED PLASTIC SHEAR FLOW OF GRANULAR MATERIALS

(MANDL, G. & FERNÁNDEZ LUQUE, R. (1970). *Géotechnique* **20**, No. 3, 277–307)

A. J. M. Spencer, University of Nottingham

This Paper is a valuable contribution to the understanding of the flow of granular materials, and clarifies a number of points. In particular the discussion of non-coaxiality of stress and strain-rate tensors gives a new and illuminating approach to this long-standing problem in the mechanics of granular materials.

Of particular interest to the Writer is the analysis in Appendix 2 of the model of a granular material proposed by de Josselin de Jong (1958, 1959). The Authors show that this model leads to the equation (equation (63a) of the Paper[1])

$$\left(\frac{\partial v_x}{\partial x}-\frac{\partial v_y}{\partial y}\right)\sin 2\theta-\left(\frac{\partial v_x}{\partial y}+\frac{\partial v_y}{\partial x}\right)\cos 2\theta-\left(\frac{\partial v_y}{\partial x}-\frac{\partial v_x}{\partial y}-2\frac{\partial \theta}{\partial t}\right)\sin\rho=0 \quad . \quad (1)$$

where v_x and v_y are velocity components referred to rectangular Cartesian co-ordinates (x, y), θ is the angle of inclination of the largest compressive principal stress axis to the x axis and ρ is the angle of internal friction, here assumed constant. As the Authors point out, this equation differs slightly from one proposed by Spencer (1964), which in the present notation is

$$\left(\frac{\partial v_x}{\partial x}-\frac{\partial v_y}{\partial y}\right)\sin 2\theta-\left(\frac{\partial v_x}{\partial y}+\frac{\partial v_y}{\partial x}\right)\cos 2\theta-\left(\frac{\partial v_y}{\partial x}-\frac{\partial v_x}{\partial y}-2\frac{\mathrm{D}\theta}{\mathrm{D}t}\right)\sin\rho=0 \quad . \quad (2)$$

where $\mathrm{D}\theta/\mathrm{D}t$ denotes the convected derivative

$$\frac{\mathrm{D}\theta}{\mathrm{D}t}=\frac{\partial\theta}{\partial t}+v_x\frac{\partial\theta}{\partial x}+v_y\frac{\partial\theta}{\partial y} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Equations equivalent to equations (2) and (3) have been given earlier by Mandel (1947).

The purpose of this discussion is to draw attention to the fact that equation (1) is not a possible constitutive relation for any material because it fails to satisfy the essential and fundamental physical requirement of being independent of a superimposed uniform rigid body translational motion. Thus suppose that

$$\left.\begin{aligned} v_x &= f(x, y, t) \\ v_y &= g(x, y, t) \\ \theta &= h(x, y, t) \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

satisfy equation (1). Then equation (1) should also be satisfied by any velocity field which

[1] An error in the Paper in the sign of $\partial\theta/\partial t$ has been corrected.