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# Dynamic Cone for Shallow In-Situ Penetration Testing

**REFERENCE:** George F. Sowers and Charles S. Hedges, "**Dynamic Cone for Shallow In-Situ Penetration Testing,**" *Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils, ASTM STP 399*, Am. Soc. Testing Mats., 1966, p. 29.

**ABSTRACT:** Field calibration of a portable dynamic cone penetrometer was made to determine a penetration resistance relationship with the standard penetration resistance. The penetrometer has been found useful in the inspection of footing foundations and for light field exploration where the standard penetration range of limits is generally known. The test data show that it is capable of approximating the standard penetration resistance for the virgin soils of the southeastern United States.

**KEY WORDS:** soil (material), field tests, penetrometers, penetration resistance, cone penetrometers, footings

Penetration tests have long been used to evaluate soil consistency and density. The primitive builder may have sounded the ground with a pointed stick or his heel, as can be seen in some tribal villages today. The skilled workman forced the point of a pick or drove a rod into the ground with a mallet of known weight. Today there are numerous penetrometers of standardized design, but all based on the same principle; the penetration of an object into the soil, forcing the soil aside and developing a shear displacement similar to a bearing capacity failure of a foundation [*1-4*][3]. The relationship between soil strength and penetration resistance is a function of the shear pattern. This can be determined by a plastic analysis of the shear zone or by empirical correlation with laboratory tests. In each case the results depend on the shape of the penetrometer, which varies with the type of soil and its consistency and density.

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[3] The italic numbers in brackets refer to the list of references at the end of this paper.

Various shapes of penetrometers are in use, including flat-tipped rods, cones of different sizes and shapes, augers with cone-shaped tips, and cutting edges of thick-wall samplers. Although there are few comparative data on the effect of shape, there is some belief that the cones yield more consistent results than the others. Two types of loading are used: static and dynamic. Static loading simulates the shear developed in laboratory testing and can be easily adapted to continuous penetration and automatic recording [5]. Dynamic loading is adapted to a very wide range of soil strengths but introduces the variable effect of dynamic shear and shock or vibration. The personal experiences of those who make and interpret the test results rather than any well-defined merits of any one method or device appear to be the factor determining selection and use of the various devices [6].

## Dynamic Portable Penetrometer Genesis

The senior author developed a lightweight portable dynamic cone penetrometer in 1959 to be used in field exploration and for verifying individual footing foundations during construction. The device, as with most field tools used in foundation evaluation, should never be used as the sole means for determining foundation conditions. It must be used in conjunction with previously established field and laboratory data: standard split-barrel penetration resistance, density, shear strength, and consolidation data. Some investigators have gone to great lengths to develop sophisticated techniques for the correlation of penetrometer design and penetration depth using standard applicator energies with unconfined compression strength or bearing capacity of deep foundations [7]. The heterogeneous variation of most natural soil masses is not favorable to the use of such rigorous techniques, except in very localized areas of relatively homogeneous soils. The dynamic penetrometer described in this paper was developed primarily as a verification or control penetrometer to check individual foundations during construction where a subsurface investigation has been made utilizing standard split spoon penetration methods, and laboratory shear strength and consolidation tests and analyses have been performed on undisturbed samples. A secondary use is the field investigation of subsurface conditions for lightly loaded structures where local experience from previous field investigations and laboratory analysis have established narrow limits of the strength parameters and consistencies; here again it is a verification tool to be used for an economical foundation analysis.

The device is a dynamic portable cone penetrometer utilizing a 15-lb. steel ring weight falling 20 in. on an E-rod slide drive (Fig. 1). The cone point is enlarged to minimize shaft resistance during testing. The penetration test is made through an augered hole from 4 to 6 in. in diameter using

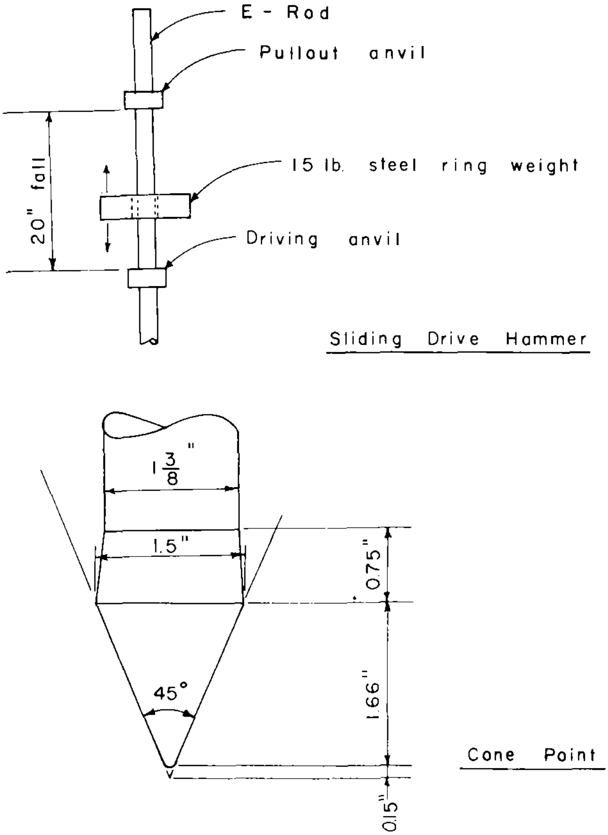



FIG. 1—*Dynamic portable penetrometer.*

the auger cuttings to identify the soil. This is essential because the interpretation varies with the soil type.

After augering to the test depth, the cone point is seated 2 in. into the undisturbed bottom of the hole to be sure the cone is completely embedded. The cone point is further driven 1¾ in. using the ring weight hammer falling 20 in. These blows are counted and recorded. If need be, a second and third penetration test can be made by driving the cone point additional 1¾-in. increments. Beyond this distance the effect of side friction of the shaft may become apparent, and the shape of the shear zone may be altered and jeopardize the value of the blow count readings. The penetrometer can effectively be used in auger holes to depths of 15 to 20 ft. Beyond this it is difficult to handle the weight of rods by hand, and also it is possible the penetration blow resistance count is affected by the dynamic energy loss in overcoming the rod inertia.

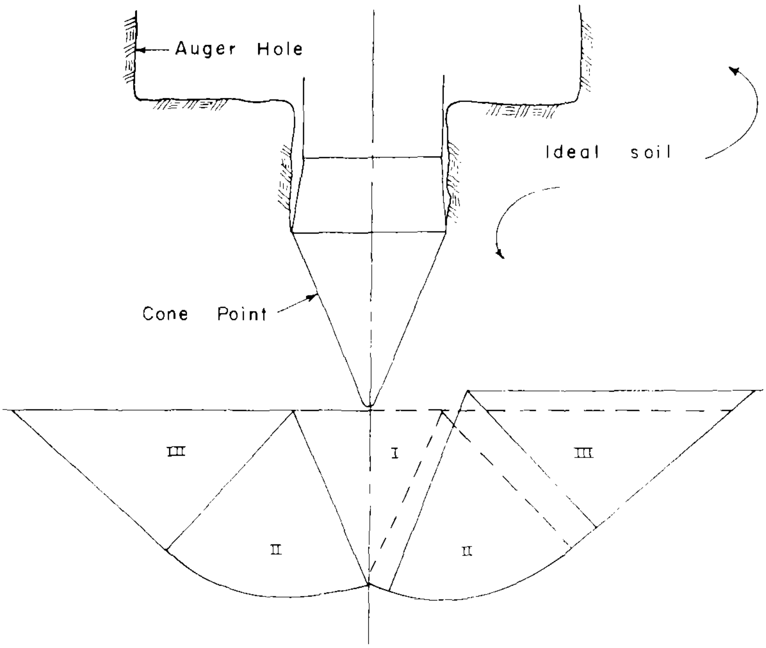



FIG. 2—*Theoretical boundaries of plastic failure.*

## Theoretical Principle

Basically the theoretical aspect of the successive penetrations caused by the hammer drop is that outlined in the classic study of bearing capacity failure by local and by general shear [*8*, *9*]. Before the cone point is forced into the level of the soil to be tested, the soil is in a state of elastic equilibrium. When the cone point is forced to the test level the soil passes into a state of plastic equilibrium with the cone point becoming the element forming part or all of Zone I, Fig. 2. Assuming an ideal soil and a smooth cone point, the zone of plastic equilibrium is subdivided into a cone-shaped zone (later displaced by the penetrometer point), an annular zone of radial shear emanating from the outer edges of the cone, and an annular passive Rankine zone. The dashed lines on the right-hand side of Fig. 2 indicate the boundaries of Zones I to III at the instant of failure or penetrometer movement, and the solid lines represent the same boundaries after the cone point has moved into the level being tested. The foregoing explanation is brief; it describes the general condition that exists during the cone point penetration and is not meant to be a complete or precise rationalization. As mentioned before, it is not necessary and almost impossible to form a working hypothesis of the cone point penetration mechanism because of the macro and micro variations within a real soil mass.

## Penetrometer Resistance—Shear

The punching resistance of an ideal plastic medium as first described by Prandtl [1] can be expressed by

$$q_o = N s \qquad (1)$$

where $q_o$ is the average punching or penetration stress, $s$ is the shearing resistance of the medium, and $N$ a coefficient which depends on the geometry of the point and surface it penetrates. As modified for expressing soil bearing capacity, $N$ depends on both the geometry of the point and the surface and on the angle of internal friction. For clays exhibiting no apparent internal friction and for cone angles of 45 to 60 deg., $N$ appears to be approximately 7.

The static penetrometers, such as the "Dutch Cone" and the Swedish Geotechnical Laboratory cone, apply a static force to the point sufficiently great to produce shear failure. Thus, the soil bearing capacity for a foundation the same size and shape of the cone is measured directly at that depth below the surface, and the soil shear strength could be found by rewriting Eq 1 as

$$s = \frac{q_o}{N} \qquad (2)$$

provided $N$ can be found theoretically or by experiment. This procedure lends itself to continuous measurement of resistance with increasing depth by merely advancing the cone and measuring the necessary force, and automatic recording might minimize the human factor.

There are three serious inherent shortcomings, however. First, wide variations in resistance within a short distance provide a zigzag record that is difficult to average and often more difficult to interpret. Second, a very hard but thin layer that may contribute little to the strength of the soil mass may distort the picture. Third, in penetrating soils of widely varying resistance, the force may be limited in hard materials to the weight of the equipment or the integrity of some anchoring device, while in soft materials the error inherent in the measuring system may obscure the soil's resistance.

The dynamic penetrometer has none of these shortcomings (although it has a few of its own). A measured increment of work, $\triangle W$, is applied to the penetrometer and this is dissipated in the energy necessary to force the penetrometer a distance $\triangle s$

$$\triangle W = \triangle s \, q_o \, A \qquad (3)$$

where $A$ is the penetrometer area.

By driving the penetrometer a fixed distance, the variations in $q_o$ are automatically averaged. If a hard spot is encountered, the work applied can be increased simply without increasing the weight of the equipment.

The ordinary dynamic penetrometer, including the one under discussion, employs a simple falling weight for a controlled source of energy. Thus the measuring system can be simple and relatively foolproof. If a hard layer is encountered, the force increases as $\Delta s$ decreases, so the device adjusts itself to some extent to the material hardness. All this is possible with a light, simple, unsophisticated device.

The only inherent disadvantage is from the effects of a dynamic force on some soils. The dynamic resistance of a loose, saturated, fine-grain, cohesionless soil is likely to be lower than the static resistance; conversely, the dynamic resistance of a very dense, saturated, fine-grain, cohesionless soil is likely to be higher than the static. Therefore, the results of dynamic penetration testing must be utilized judiciously with proper engineering interpretation of the results. The indiscriminate use of any test result is fraught with danger, and this test is no exception [9].

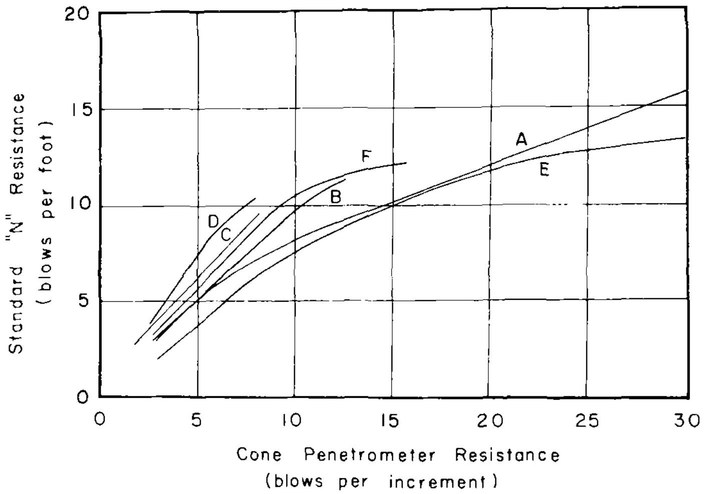
## Application and Behavior of Penetrometer

The dynamic penetrometer described in this paper has been used with much success by the authors in four geologic regions encompassing ages from Precambrian to Recent and almost all types of soils: the Piedmont, Blue Ridge, Appalachian Valley and Plateau, and Coastal Plain geologic provinces of the southeastern United States. Its primary correlations have been with results from the ASTM Method for Penetration Test and Split-Barrel Sampling of Soils (D 1586-64), on a blow-count basis for their respective increments of driving.

The soils in which the penetrometer has been most reliably calibrated with reference to Method D1586 resistances are the sandy micaceous silts and clayey sandy micaceous silts of the Piedmont geologic province; the silty sands, clayey sands, and interbedded and intermixed sandy, silty, clayey soils of the Coastal Plain province; and the silty clays and clayey silts and sandy clays of the Appalachian Valley province. It has also been calibrated for compacted fills made of the above soils.

The dynamic portable penetrometer in virgin soils of the Piedmont province has shown a consistent correlation between penetrometer resistances and Method D1586 resistances. Curve *A* in Fig. 3 was compiled from a variety of tests on virgin Piedmont soils in Georgia, South Carolina, and North Carolina. The ratio of Method D1586 resistance to the penetrometer blows varies from 0.9 to 1.0 for material with low resistances to 0.3 to 1.0 for material with high blow resistance. These ratios are for individual data points and may not exactly coincide with ratios taken from the various curves.

The use of the penetrometer in compacted fill soils of the Piedmont origin shows that the calibration ratio of Method D1586 penetration resistance to the penetrometer blows varies from 0.9:1 for low-density




Curve A - Virgin Piedmont soils
B - 95% Compacted soil
C - 90% Compacted soil
D - 85% Compacted soil
E - Coastal Plain soils
F - Piedmont alluvium

FIG. 3—*Penetration relationships.*

(85 per cent of maximum by ASTM Methods D698) fill to 0.66:1 for high-density (95 per cent of maximum by ASTM Methods D698) fill. Three curves, *B*, *C*, and *D*, shown in Fig. 3 are for tests on compacted fills and their different densities.

The two remaining curves, *E* and *F*, show the relationship between Method D1586 resistance and penetrometer resistance for the marine and estuarine Coastal Plain soils (Curve *E*); and alluvial soils of the Piedmont, which are silty micaceous sands and sandy micaceous silts of recent deposition. The ratio of Method D1586 resistance to penetrometer blows for the Coastal Plain soils varies from 0.5:1 for materials of high resistance to 1:1 for materials of low penetration resistance. The ratio for alluvial soils of the Piedmont varies from 0.6:1 to 2:1.

## Conclusions

The conclusions reached from the many tests and calibrations by the authors and their associates are that the dynamic portable penetrometer is a useful tool for construction control and field exploration for lightweight structures where value does not justify the cost of a drilling rig or where access prohibits a drilling machine. The use of the penetrometer is not too valid in alluvium of Piedmont origin, in that the calibration ratios vary without specific pattern. It is probable that this variation is due to (1) the effect of pore pressure irregularities caused by the usually high water contents of such soils, (2) the vast irregularities in deposition and grain size of the Piedmont alluvium, and (3) the general unconsolidated state of such recent deposits.

In general the penetrometer produces the best correlations between 4 and 30 blows. Below 4 blows, for the required 1¾-in. penetration increment, the soils are too soft or loose to produce significant results; these soils under any circumstances should be tested by other means, such as unconfined compression or triaxial shear tests from undisturbed samples, or by field vane shear methods. Above 30 blows per penetration increment the correlations are quite variable. This change may be caused by the local hard layers of partially weathered soils in the Piedmont, or the grain size variation usually associated with high penetration resistances in the Coastal Plain soils. The penetrometer is generally limited to soils in which all the gradation is smaller than fine gravel or very coarse sand.

In order to utilize the portable dynamic penetrometer for construction control it must be calibrated for each project. This can be done during the exploratory work. For exploratory work the penetrometer must be used in areas where the limits of the soil properties are generally known, with the aid of unconfined compression tests or triaxial shear tests on undisturbed samples. The penetrometer can be used for verification of penetration resistances from Method D1586 once a calibration has been established; however, in soils that are highly micaceous the soil rebound associated with excavation of the footing may show up in a reduction of the number of blows recorded. When checking such conditions the test should be made through an auger hole to the footing level immediately prior to footing excavation or beside the footing in the unexcavated portion. Several penetration tests are needed at different depths below the footing level to qualify the inspection results. It is wise to test between the first foot level below the footing and the level at a depth equal to the width of the footing. The penetrometer does not work well below the water table unless the bore hole is stabilized to prevent inflow and soil softening.

The use of the penetrometer in estimating in-place density of compacted fills is not valid because the penetration resistance varies with both density and moisture content. In fill control work it is used to supplement density testing and to determine areas where relative consistency or density are radically different. Areas thus detected can be checked by standard density test methods.

The use of this type of dynamic penetrometer with its sliding weight presents conditions which can result in injury to the operator's fingers unless maximum attention is maintained during operation.

The ratios as well as Fig. 3 are intended to show qualitative rather than quantitative information. More field and laboratory work is necessary before this procedure can be developed into a rational method of control.

## Acknowledgments

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# DISCUSSION

*Nicholas Chryssafopoulos*[1] —The authors have presented in this paper some interesting correlations between values obtained by means of a dynamic cone penetration test they have devised and standard penetration test results ($N$-values). These correlations were established with the primary purpose of determining densities of fills already in place at variable depths. The results reported primarily covered cohesive soils and mixtures of cohesive and noncohesive materials. To reach the depth at which density checks are to be carried out, a hole is advanced to the desired depth, and then the dynamic cone penetration resistance is measured.

In view of the well-established lack of reliability of $N$-values in cohesive soils, the use of the correlations established by the authors may result in estimates of densities which may not be representative of the true density of the soil being checked. The writer wonders whether better results could not be obtained, once the hole is advanced to the desired depth, by testing the density and strength of samples of the soil obtained by pushing or driving into the ground 6-in. long sections of a 2-in. Shelby tube. It would appear that this latter method, which has been used often, would not require special equipment or longer time for taking the samples. On the contrary, actual measurements of density and strength would be made instead of estimates based on correlations that may prove to be not too reliable, when dealing with cohesive soils.

*Messrs. Sowers and Hedges* (*authors*)—Mr. Chryssafopoulos apparently misunderstands the authors' intentions when he states that the primary purpose of the correlations is to determine density of fills. Instead, the paper states that such a use is invalid except to quickly detect doubtful areas where density tests should be made. We agree with the discusser that a direct measurement of density is necessary. We do not agree that it is faster (the sampling may be but the ensuing testing takes time). We do not agree that a 2-in. thin-walled tube is a valid sampling device for fill density tests even though some may use it.

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