An Experimental Investigation of Factors Affecting Penetration Resistance in Granular Soils in Centrifuge Modelling

by

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ABSTRACT

Miniature penetration probes have been developed at Cambridge University over the last 7 years for assessing consistency of sample preparation and strength characteristics of cohesionless and cohesive soils. An experimental programme, consisting of 20 centrifuge tests and 12 laboratory floor tests in a rigid chamber, was conducted to investigate:

(i) the usefulness of the penetration probe as an index test to check consistency of sample preparation;

(ii) the container boundary effects on penetration resistance;

(iii) scale effects;

(iv) grain size effects;

(v) rate of penetration effects;

and (vi) the effect of stoppage of the centrifuge between penetration tests.

The results pertaining to the boundary effects are examined in relation to the body of existing data from chamber tests. The scale effects which need to be considered for proper interpretation of the probe data are analysed in terms of the stress - dilatancy concept for dense soils.

1. Introduction

In order to simulate or model the standard cone on the centrifuge, the cone has to be of a very small diameter in the enhanced inertial acceleration field. As most tests of soil structure interaction studies are performed at 80 to 100 g, it would be necessary to use cones with dimensions of 0.45 mm to 0.36 mm. Obviously the design and instrumentation of such a probe is impractical.

Ferguson and Ko (1981) modelled the standard cone on the centrifuge using different probes with diameters varying from 3.6 mm to 20 mm at acceleration levels of 10 g to 1.8 g respectively. No instrumentation was incorporated in these cone penetrometers and only the total resistance to penetration was monitored.

To incorporate instrumentation it is necessary to use probes of 7 mm to 10 mm diameter. Such miniature penetrometer probes and vanes have been developed for use on the Cambridge 10m diameter beam centrifuge to assess soil strength properties during centrifuge model testing, (Davies and Parry 1982, Almeida and Parry 1983a, 1983b). However, a 10 mm probe when used at 100 g corresponds to a 1 m diameter pile rather than a conventional cone penetrometer with an area of 10 cm^2.

One of the important considerations in the development of very small diameter probes is the grain size effects during centrifuge modelling. The grain size effects can be significant when the ratio of the probe diameter to the mean size is less than 20, (Ovesen, 1981). Size effects also have to be considered as the same probe is used at different acceleration levels.

In addition to grain size and scale effects, the penetration resistance is affected by :

(i) the proximity of rigid boundaries of the strong box,

(ii) the rate of penetration,

and (iii) stopping and restarting the centrifuge, which results in stress cycling of the soil, and in some cases overconsolidation effects.

In order to investigate the effect of these parameters on the penetration resistance an experimental programme of centrifuge and laboratory floor tests was undertaken. Piezo-probe data of experiments conducted by Springman (1987)) are reviewed in terms of the pore water pressures generated during penetration testing.

2. Review of Penetrometer Probes Developed at Cambridge University

Cheah (1981) developed the Mark I penetrometer probe for use in clay by modifying the vane apparatus of Davies and Parry, 1982. The probe, shown in Fig.1a, had a tip 10 mm in diameter with a 60° apex angle and was screw driven into the soil by means of a D.C. motor.

The shaft behind the tip was 8 mm in diameter and was coated with a thin layer of Teflon to reduce the shaft friction. Subsequent studies showed that this technique did not significantly reduce the side friction and that the load cell at the top of the shaft was measuring a combination of the tip and shaft resistance (Almeida and Parry 1983a).

Screw Driven Mark II and Mark III probes were developed to eliminate some of the drawbacks of the Mark I probe (Almeida and Parry 1983a,b). In the Mark II probe, shown in Fig.1b, two load cells were incorporated to measure the tip and total load separately. The probe was driven at a constant rate in the soil with a D.C. motor. During use of the Mark II probe it was noted that the signal outputs from the columnar load cells were low and thus susceptible to electrical noise. In order to overcome this problem, the Mark III probe was developed with a rosette load cell mounted at the top of the shaft of the probe, Fig.1c (Almeida and Parry 1983a). Use of the rosette load cell led to a seven fold increase in signal outputs, thus ensuring reliable load cell data.

A piezo-probe was also developed as part of studies dealing with embankments on soft ground (Almeida and Parry 1983b). The piezo-probe is shown in Fig.2. The porous element is located at the tip of the probe. A PDCR 81 Druck pressure transducer was used for measuring pore water pressures. The rosette load cell located at the top of the penetrometer probe measured the tip load. The piezo-probe was also used in centrifuge tests undertaken by Springman (1987) in layered soils consisting of clay over sand.

In 1986 a hydraulically actuated penetrometer probe was developed for use in granular materials, Fig.3. Unlike the Mark I to Mark III equipment, this probe is pushed in the soil at a constant rate by a hydraulic actuator. Probes of 10 mm and 20 mm diameter with 60° apex angles were fabricated. The point resistance of the probe is measured by a load cell located directly behind the tip. The load cell is sealed to prevent water entry into the probe when used in saturated soils. The depth of penetration of the probe is monitored by a rotary potentiometer located inside the bottom chamber as shown in Fig.3. Additional instrumentation consists of pressure transducers fitted at the water and nitrogen pressure inlets and a differential pressure transducer mounted in the piston.

Before the penetrometer test, downward movement of the probe is prevented by nitrogen pressure acting in the bottom chamber. At an acceleration of 100 g, a pressure of about 2000 kPa is required to counteract the weight of the probe and the piston and the head of water in the external piping. The probe is advanced into the bed of soil by feeding pressurized water from accumulators mounted near the axis of the centrifuge into the top chamber of the piston. The nitrogen is vented out of the bottom chamber via a pressure relief valve. Fig.4 shows schematically the hydraulic and pressure control systems required during centrifuge operation of the probe.

3. Apparatus and Experimental Procedures

3.1 Centrifuge Tests: Series I

The assembled package for centrifugal testing of the hydraulically actuated penetrometer probe is shown in Fig.5. The strong box is a 850 mm diameter by 400 mm high tub which has been used in the past for other soil-structure interaction studies including piles and shallow foundations.

The penetrometer is mounted on a channel section. It is possible to perform four penetration tests in the same tub by changing the position of the channel after each test. The present procedure requires stoppage of the centrifuge while the penetrometer is moved to a new location.

In Series I, penetration tests were performed using a 10 mm diameter cone in dry Fontainbleu and 14/25 Leighton Buzzard sand. All the samples were prepared by raining sand from a single-holed hopper, a technique used at Cambridge University for a number of years. The height of drop and rate of pouring was controlled to achieve the required relative density.

Table 1 summarizes the details of the experimental program undertaken in Series I. The variables investigated were: type of sand, relative density, scale effects and the effect of stoppage of centrifuge machine between penetration tests.

3.2 Centrifuge Tests: Series II.

For Series II tests the penetrometer was mounted on a beam of rectangular box section as shown in Fig.6. The new box section was designed and fabricated to allow the distance between the penetrometer and the rigid boundary of the tub to be varied. The use of the box section also eliminated the spacers required under the channel section in Series I.

Dry sand samples were prepared by pouring sand from a hopper similar to Series I. To investigate the grain size effects on the penetration resistance, a 20 mm diameter probe was commissioned for use on the centrifuge. Considerable difficulties were encountered in sealing the system against nitrogen and water leaks. The problems encountered were similar to the ones encountered during commissioning of the 10 mm probe.

A summary of the experimental program undertaken in Series II is presented in Table 2.

3.3 Laboratory Floor Tests:

Fig.7 shows the set-up for laboratory floor tests at 1 g with surcharge pressure applied to the sand surface. For these tests the 850 mm diameter tub was again used. The surcharge pressure was applied by means of a 12 mm thick steel plate resting on the top surface of the sand. The plate was reinforced by a 100 mm deep pressure frame which was loaded by the hydraulic ram in a large consolidation press. Four holes of 18 mm diameter were drilled in the steel plate through which the penetration tests were performed. Tests L1 to L4 were performed in the same tub under a surcharge pressure of 50 kPa. Tests L5 to L12 were all performed in a second tub. This was accomplished by rotating the top plate and the pressure frame through 45° after tests L5 to L8 were completed. Table 3 summarizes the details of the laboratory floor tests.

3.4 Centrifuge Tests: Others

In the series of saturated sand tests performed by Tan (1987) and Shi (1987), the samples were prepared dry as in Series I. The samples were saturated by purging with carbon dioxide and then slowly drawing deaired water into the sample under vacuum. A summary of the experimental program undertaken by Shi (1987) is presented in Table 4.

3.5 Materials

3.5.1 14/25 Leighton Buzzard Sand

The 14/25 sand passes through a 1.2 mm sieve and is retained on a 0.6 mm sieve. It has a nominal grain size diameter of 0.9 mm. Tests performed by Mak (1983) indicate the maximum and minimum void ratio of this material to be 0.80 and 0.50 respectively. The specific gravity of solids is 2.67 for this sand. Mak also performed triaxial tests on dry samples of 14/25 sand under confirming pressures of 30 kPa and 160 kPa. Results obtained by Mak are shown as the solid square points in Fig.8 in the form of a stress-dilatancy plot as proposed by Bolton (1986).

3.5.2 Fontainbleu Sand:

The Fontainbleu sand passes through a 0.4 mm sieve and is retained on a 0.08 mm sieve. It has a nominal grain size diameter of 0.17 mm.

Classification tests on this sand have been performed independently in Denmark, France and by Shi (1987) in Cambridge. The maximum and minimum void ratio values for Fontainbleu sand are found to be 0.92 and 0.55 respectively by Shi (1987). The specific gravity of solids used in calculating void ratios is 2.644.

Triaxial tests have been also performed on dense samples of this material: the results of Luong (1987) and the Danish Geotechnical Laboratory (1987) are presented as open squares on the stress-dilatancy plot in Fig.8

3.6 Data Acquisition and Processing

Electrical signals from the various instruments were monitored during the centrifuge tests and the laboratory floor tests on a 14 track Racal tape recorder. The data were digitized using an Alpha micro computer and the FLY 14 suite of software. Fig.9 shows typical FLY 14 output. These digitized data were transferred to a main frame computer and processed into engineering units. Selected portions of these data were then transferred to an IBM AT for further processing. The SYMPHONY spreadsheet environment was used to perform these calculations and to generate the required graphs.

4. Parametric Study:

4.1 Probe data as an index of sample uniformity and consistency:

The air pluviation sand pouring technique using a single holed hopper has been used for sample preparation at Cambridge University since 1960. These procedures were developed initially for use with different grades of Leighton Buzzard sand. In recent years, other granular materials, such as Fontainbleu sand, calcareous sand and flint grit, have been used in centrifuge models. These samples have been prepared using the air pluviation technique as well as other techniques such as deposition in water and vibrating the sample. Irrespective of the method of sample preparation, the relative density is estimated from the overall weight of the sample and the volume of the soil. Obviously this calculation infers the sample to be uniform and homogeneous. The validity of this inference can be now checked by using miniature probes during centrifuge modelling.

Tests 1A, 1B and 1C were all performed at a nominal acceleration of 20 g in the same strong box and the results are presented in Fig.10. The relative density of the sample was 65%. The data indicate variations in the penetration resistance of the order of 40 percent. It will be later explained that for such low relative densities this variation may be largely due to stress cycling induced by stoppage of centrifuge between the tests. Figs. 11 and 12 show data for soil samples with higher relative densities. The relative density of the Fontainbleu sample was 87% whereas the 14/25 Leighton Buzzard sand sample had a relative density of about 97%. All the tests were performed at a nominal acceleration of 20 g and the samples were prepared using the air pluviation technique. From Figs. 11 and 12 it can be concluded that the current air pluviation technique can produce very dense uniform soil samples. Fig.13 shows the variation of tip resistance with model depth for the loose Fontainbleu sand sample used in tests 2A to 2D. The results indicate some local non-uniformities in the sample. It is therefore necessary to exercise care in the preparation and handling of medium dense (D_R between 50 to 70%) and looser sand samples.

4.2 Boundary Effects:

Parkin and Lunne (1982) report results of an extensive series of tests to investigate boundary effects in flexible walled chamber tests. Tests were performed using different diameter cones and calibration chambers. The boundary conditions on the flexible sides and base were either constant stress or constant volume. The results of this study are presented in Fig.14. The term R_D in this figure is defined as the ratio of the diameter of the chamber to the diameter of the cone. These results indicate that the side boundary effects are dependent on the relative density of the sand. For loose sands with relative densities of the order of 30%, the side boundary effects are negligible. For dense sands with relative densities of the order of 90%, the chamber diameter must be at least 50 times the diameter of the cone to eliminate the effect of the side boundary on the cone resistance. In recent years this particular research finding has been extensively used to correct flexible wall chamber tests for boundary effects.

Parkin and Lunn (1982) also report the results of the experimental work performed by Last (1979) in a rigid-walled chamber. The diameter ratios in this latter study were 28.0 and 39.7. These results are also presented in Fig.14 and indicate that for rigid walled chambers the side boundary effects are not significant when the diameter ratio is greater than 28. This difference in behaviour was attributed by Parkin & Lunn (1982) to the friction mobilized on the rigid side walls resulting in low coefficient of earth pressure values.

The strong boxes used in the centrifuge tests are rigid-walled, and as such direct application of Parkin and Lunn's (1982) work on flexible boundary effects is not valid. Tests 4A to 4C and 5A to 5D were performed to investigate the effect of rigid walls on penetration resistance.

In the present series, tests in Fontainbleu sand were performed with the distance from the rigid wall over probe diameter ratio (L_D) varying from 10 to 42. The results are presented in Fig.11. The relative density of the sample was 87%. Tests in 14/25 Leighton Buzzard sand were performed at distance ratios (L_D) of 5, 10 and 21 and the results are presented in Fig.12. The data shows that side wall boundary effects are negligible even when the probe is located at a distance corresponding to a distance ratio of 5 from the wall. Small variations in the test results are likely to be due to stress cycling caused by stoppage of centrifuge between the tests.

In terms of the bottom boundary effect, it is observed that the penetration resistance increases rapidly as the tip of the probe approaches the bottom of the strong box as shown in Fig.15 for test 1B. Similar trends were observed for all the tests with the 10 mm diameter probe; these indicated the influence of the bottom boundary is felt at a distance of 10 to 12 times the diameter of the probe from the bottom of the strongbox.

4.3 Scale Effects:

Scale effects in model studies need to be considered when dealing with problems in the area of soil structure interaction. Scale effects in centrifuge testing have been studied by Ovesen (1981), and Tagaya et al (1987) while investigating the problem of pull-out resistance of anchors in granular soils. Yamaguchi et al (1977) and Abghari et al (1987) investigated scale effects in relation to the bearing capacity problem. The bearing capacity as well as pull-out capacity is dependent on the mobilized angle of internal friction, which is in turn dependent on mean effective stress and relative density. As the mean effective stress varies with the depth of the failure mechanism, the bearing capacity of different sized foundations was found to be different.

As the same size penetrometer probe is used in different centrifuge tests at a variety of g levels, scale effects have to be considered in interpreting these results. Tests 2A to 2D and 3A to 3D in the present investigation were performed to study scale effects. Fig.16 presents the results of tests carried out in 14/25 Leighton Buzzard sand. Tests 3A, 3B and 3C were carried out at nominal acceleration levels of 40 g, 20 g and 30 g respectively. These tests correspond to prototype diameters of 397 mm, 197 mm and 296 mm respectively. Presentation of the data in the form of Fig.16 is misleading as stress level effects and depth effects are not separated. Similar data for loose and medium dense Fontainbleu sand are presented in Figs. 17 and 18 respectively. In all these tests it is observed that the smaller the prototype diameter of the probe the higher the resistance, a fact which has been reported by many researchers while studying model piles at 1-g (Meyerhof, 1983).

These data are analysed in section 5 to account for dependence of the angle of internal friction on the mean effective stress level.

4.4 Grain Size Effects:

The failure mechanism, width and the extent of shear and rupture bands in cohesionless soils are all dependent on the relative magnitude of the model dimension and the grain size of the material. However in centrifuge model testing, the grain sizes used in the model corresponds in the prototype to much larger soil grains: an increase in direct proportion to the centrifuge acceleration level. Scaling or modelling of sand sized particles in a centrifuge test would require use of silt or clay size particles, but these have significantly different strength characteristics. In many cases the same granular materials are used in the model as in the prototype. The effects of grain size must therefore be investigated for different class of problems. Ovesen (1981) performed a series of pull-out tests on model anchors with the ratio of anchor diameter to the mean grain size being in the range of 25 to 128. No grain size effects were observed in these tests and it was concluded that the grain size effects were insignificant if the ratio of the model diameter to the mean grain size was greater than 25. Yamaguchi et al (1977) investigated grain size effects in relation to the bearing capacity problem. In that study the ratio of the model footing diameter to the mean grain size, (defined as grain size ratio) was 36 to 286 and no grain size related errors were observed.

The average grain size (D_{50}) of soils used in this centrifuge model test study varied from 0.17 mm (Fontainbleu) to 0.9 mm (14/25 Leighton Buzzard). As the miniature penetrometer probe has a diameter of 10mm the grain size ratio in 14/25 Leighton Buzzard sand would be about 10. As the existing literature reports the results of tests with grain size ratios greater than 25, Centrifuge tests 5A to 5D in 14/25 Leighton Buzzard sand were performed using the 20 mm probe. A comparison of tests 3A and 5A is presented in Fig.19. In interpreting these results it should be noted that the void ratios in tests 3A and 5A were 0.53 and 0.51 respectively. This difference would lead to a variation of about 20% between the probe resistance profiles shown, (Meigh, 1987). Also the 10 mm probe was rougher due to more extensive usage than the 20mm probe. In view of these differences, exact quantification of grain size effect is not possible. However the present study does indicate that probe resistance is influenced by grain size effects for grain size ratios less than 20.

4.5 Rate of Penetration Effect:

The penetrometer probe used in the present study is pushed in the sand bed at a rate of about 2-4 mm/sec. As the penetrometer is used both in the saturated as well as dry sand samples without any pore water pressure measurements, it is necessary to investigate the effect of rate of penetration on the probe resistance. Fountainbleu sand is finer than 14/25 Leighton Buzzard sand and as such is less permeable. Fig.20 shows the results of tests performed in dry and saturated Fontainbleu sand samples at comparable relative densities. As the results are in close agreement, it can be concluded that at these rates of penetration, even in fine sands like Fontainbleu sand, the probee penetration can be considered as a drained event.

Results of the piezo probe data of Springman (1987) are presented in Fig.21 as the total pore water pressure measured and compared to the static water pressure. The rate of penetration of the probe was 10 mm/sec. It is again observed that no excess pore water pressures are generated in the sand layer which has a mean grain size diameter of 0.4 mm (30/52 Leighton Buzzard Sand).

4.6 Stress Cycling and Overconsolidation Effects:

Presently the centrifuge needs to be stopped to perform penetration tests at different locations in the same soil sample. This procedure leads to stress cycling even if all the tests are performed at the same nominal acceleration level. In some tests, model tests are performed at different nominal accelerations and, depending on the sequence of testing, can also lead to overconsolidation of soil samples.

Fig.22 shows the consequence of stress cycling. Test 3B was performed at a nominal acceleration of 20 g after test 3A was performed at an acceleration of 40 g. Test 3D was performed after tests 3A, 3B and 3C. Test 3C was carried out at an acceleration of 30 g. The close agreement between tests 3B and 3D indicates that stress cycling effects are insignificant. Similar observations can be made from the data from tests 4A to 4C (Fig.11) and tests 5A to 5D (Fig.12). In all these tests the relative densities were of the order of 90% or more. It appears that the effect of stress cycling is negligible for soil samples with initial relative densities in excess of 90%.

The effect of stoppage of the centrifuge leading to stress cycling is likely to be significant for samples prepared at low relative densities, Fig.10. The data presented in Fig.17 for loose Fontainbleu sand suggests densification of the sample due to stress cycling. It would be preferable to have a moveable gantry on the centrifuge package so that penetration tests can be performed at different locations without stoppage of the centrifuge.

4.7 Laboratory Floor 1 g Tests:

The centrifuge tests were complemented by a total of 12 tests at 1 g with surcharge pressures applied at the top sand surface. The tests were performed on dense Fontainbleu Sand for surcharge pressures of 50 kPa, 100 kPa and 150 kPa. The relative density of the sample for tests L1 to L4 was 87% whereas tests L5 to L12 were performed in a soil tub with a relative density of 81%. The data from tests L1, L5 and L9 are not included due to either inadequate penetration of the probe in the soil sample or decrease in surcharge pressure during the tests. In test L3 the data recorded was inappropriately set and the data were inadvertently not recorded during test L7.

Test results are presented in Figs. 23 to 25 for surcharge pressures of 50 kPa, 100 kPa and 150 kPa. All the test results exhibit similar trends with an initial portion where the penetration resistance increases with depth. This behaviour is due to the stress relief under the opening in the top plate. The initial section is followed by a relatively constant probe resistance with depth, the magnitude of which is dependent on the surcharge pressures. The nearly constant probe resistance is then followed by an increase in probe resistance as the tip approaches the bottom boundary.

For analysing the data a particular value of probe resistance has to be inferred for correlating the probe resistance with the surcharge pressure. It is worth noting that even in the central portion of the curve where the top and bottom boundary effects are negligible the values of probe resistance do not strictly remain constant but slightly increase with depth. Similar observations are also reported by Parkin and Lunn (1982) in the flexible-walled chamber tests. Due to the variable nature of the probe resistance with depth a consistent criterion is required to interpret the data. In the present study, average plateau values have been used to correlate the surcharge pressure and the probe resistance.

Table 5 presents the comparison between laboratory floor tests and centrifuge tests at comparable relative densities. The available data indicate that the laboratory floor tests at 100 kPa are at variance with the centrifuge tests. As the data are somewhat limited and the stress conditions in the rigid chamber tests are not known, further analysis and interpretation is not possible without further testing.

5. Interpretation

The parametric study presented in section 4 demonstrates that use of the 10mm probe at different acceleration levels (in the same strong box) corresponds to different diameter prototype piles.

(Figs. 16, 17 and 18). The penetration resistance is dependent on the relative density and mean effective stress level. In order to interpret data of this type it is necessary to consider the mobilized angle of internal friction as a function of initial relative density, critical angle of internal friction and mean effective stress, as proposed by Bolton (1986).

Based on triaxial and plane strain tests on a variety of sands, Bolton (1986) proposed he follow empirical relationships for predicting the mobilized angle of internal friction. For triaxial tests:

(1) $\phi'_{max} - \phi'_{crit} = 3I_R$ (degrees) For $0 < I_R < 4$

(2) I_R (degrees) $= I_D (10 - \ln p') - 1$,

where, I_D = initial relative density, p'= mean effective stress and I_R is defined as a relative dilatancy index.

The above equations have been used in the present study to analyse the data from tests 3A, 3B and 3C as well as tests SQ18, SQ20 and SQ22 performed by Shi (1987). Tests 2A to 2D can not be analysed using this approach. Due to the loose nature of the sand sample, the I_R values are negative. In these situations considerable contraction of the sand occurs before the critical angle of internal friction is mobilized, and the above empirical correlations can lead to significant errors (Bolton, 1986).

To predict the mobilized angle of internal friction for different sized prototype probes, it was assumed that the mean effective stress is equal to the geometric mean of the probe resistance and the vertical effective stress;

(3) $p' = \sqrt{\sigma_v' q_p}$

Using equations 1, 2 and 3, mobilized angles of friction have been estimated. The results are presented in Figs. 26 and 27 for 14/25 Leighton Buzzard and Fontainbleu sand respectively. In performing these calculations, the following parameters were used

For 14/25 Leighton Buzzard sand: $\phi'_{crit} = 35^0$, $I_D = 1.90$

For Fontainbleu sand: $\phi'_{crit} = 32^0$, $I_D = 0.76$ to 0.81

Fig.26 Demonstrates the decrease in mobilized angle of friction at the probe tip as the probe is penetrating the soil. The analysis also indicates that the mobilized angle of friction is size dependent. It is therefore essential to develop correlations similar to Fig.26 for proper interpretations of data from a probe used at different accelerations at comparable relative densities.

For 14/25 Leighton Buzzard sand Fig.26 Indicates a difference of as much as 2^0 in the mobilized angle of friction due to the effect of mean effective stress at constant relative density. It is also interesting to note that the mobilized angles of friction are within $\pm 0.5^0$ when the mean effective stress is greater than 7000 kPa using eqn. 3. At these stress levels the probe is considered to cause a "cavity expansion" mode of failure and the mobilized angle of internal friction is not significantly affected by small changes in the mean effective stress.

6. Conclusions

1) Miniature penetrometer probes can be used effectively to check the consistency and uniformity of soil samples in centrifuge model tests. As the results are affected by scale effects the penetration tests should be performed at a particular g level in any one series of experiments so that consistency and uniformity of sample preparation can be assessed.

2) Boundary effects in centrifuge model tests are significantly different from those reported in the literature for flexible walled chamber tests. The present study shows no influence of the rigid boundary when the penetration test is performed at a distance greater than 5 times the diameter of the probe from the side wall. Care is recommended in extrapolating this finding when more than one side wall is close to the probe. The effect of the bottom boundary influences the results when the tip is at a distance of 10 to 12 times the diameter of the probe.

3) It is shown that the penetration resistance is affected by scale effects. If the penetrometer is used at different g levels at the same relative density, a consistent interpretation of the data may be achieved using the stress-dilatancy concept proposed by Bolton (1986). This concept can be used for interpreting data of medium dense to dense sands. It can not be used presently for loose sands where samples undergo considerable contraction in volume before ϕ'_{crit} is mobilized. A curve fitting technique may be employed for data analysis of loose sands.

4) Penetration resistance is influenced by grain size effects. The present study indicates that probe diameter to mean grain size of the soil should be 20 or more to avoid grain size related errors.

5) Currently, various miniature probes are pushed into the soil at

rates varying from about 2mm/s to 10 mm/s. The experimental evidence indicates that at these rates of penetration even in fine sands like Fontainbleu sands, the penetration test is a drained event.

6) Presently the centrifuge is stopped when the penetrometer is moved from one location to the another in the same strong box. This leads to stress cycling even if the penetration tests are performed at the same nominal acceleration level. The effect of this stress cycling appears to be insignificant for sand samples at relative densities of about 90%. The effect of stress cycling and overconsolidation of soil samples (depending on the sequence of tests) is likely to be more significant for loose to very loose samples. It is recommended that a gantry mounted penetrometer system be developed for performing tests at different locations in the strongbox without necessitating stoppage of the centrifuge.

7) Penetration tests performed at 1 g under varying surcharge pressures produce data which are at variance with the centrifuge data for comparable relative densities. Further experimental work is required to quantify these differences based on the stress dilatancy concept.

8) Because of the various factors affecting penetration resistance, standards for penetration probes should be agreed for use in centrifuge modelling. A 7 to 10 mm diameter probe with a 60^0 apex angle is suggested.

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LIST OF FIGURES

Table 1 Centrifuge Tests

TEST	1A	1B	1C	1D	2A	2B	2C	2D	3A	3B	3C	3D
Test Acceleration (nominal)	20	20	20	20*	10	20	40	80	40	20	30	20
Probe Diameter	10	10	10	10	10	10	10	10	10	10	10	10
Distance to the rigid boundary (mm)	132	132	132	132	132	132	132	132	132	132	132	132
Sand Type	F'Bleu				F'Bleu				14/25LB			
d_{50} (mm)	0.17				0.17				0.90			
Void Ratio	0.68				0.95				0.53			
Relative Density	65%				0%				90%			

* After accelerating to 100 g.

Table 2 Centrifuge Test Series - II

TEST	4A	4B	4C	4D	5A	5B	5C	5D
Test Acceleration (nominal)	20	20	20	1	20	20	20	20
Probe diameter (mm)	10	10	10	10	20	20	20	20
Distance to the rigid boundary (mm)	420	200	100	100	420	200	100	100
Sand Type	Fontainbleu				14/25LB			
d_{50} (mm)	0.17				0.9			
Void Ratio	0.6				0.51			
Relative Density	87%				97%			

Table 3: LABORATORY FLOOR TESTS.

Test	L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12
Surcharge Pressure (kPa)	50	50	50	50	100	100	100	100	150	150	150	150
Probe dia (mm)	10	10	10	10	10	10	10	10	10	10	10	10
Distance to the rigid boundary (mm)	132	132	132	132	132	132	132	132	132	132	132	132
Sand Type	F'Bleu				F'Bleu							
d_{50} (mm)	0.170				0.170							
Void Ratio	0.60				0.62							
Relative Density	87%				81%							

Notes: L1 = data not used due to inadequate penetration.

L3 = data acquisition system improperly set-up.

L5 = inadequate penetration.

L7 = data not recorded inadvertantly

L9 = inadequate penetration

Table 4 - Centrifuge Tests by Shi (1987)

Test	SQ 18	SQ 19	SQ 20	SQ 22	SQ 23
Test Acceleration (Nominal)	30	30	20	40	40
Probe Diameter (mm)	10	10	10	10	10
Distance to the Rigid Boundary (mm)	132	132	132	132	132
Sand Type	F'Bleu		F'Bleu	F'Bleu	
d_{50} (mm)	0.17		0.17	0.17	
Void Ratio	0.64		0.62	0.63	
Relative Density	76%		81%	78%	

Table 5. Comparison Between Centrifuge and Laboratory Floor Tests

Vertical Stress (kPa)	Probe Resistance (MPa)	
	Centrifuge Test (4A)	Lab Floor Test
50	20	20
100	45	30

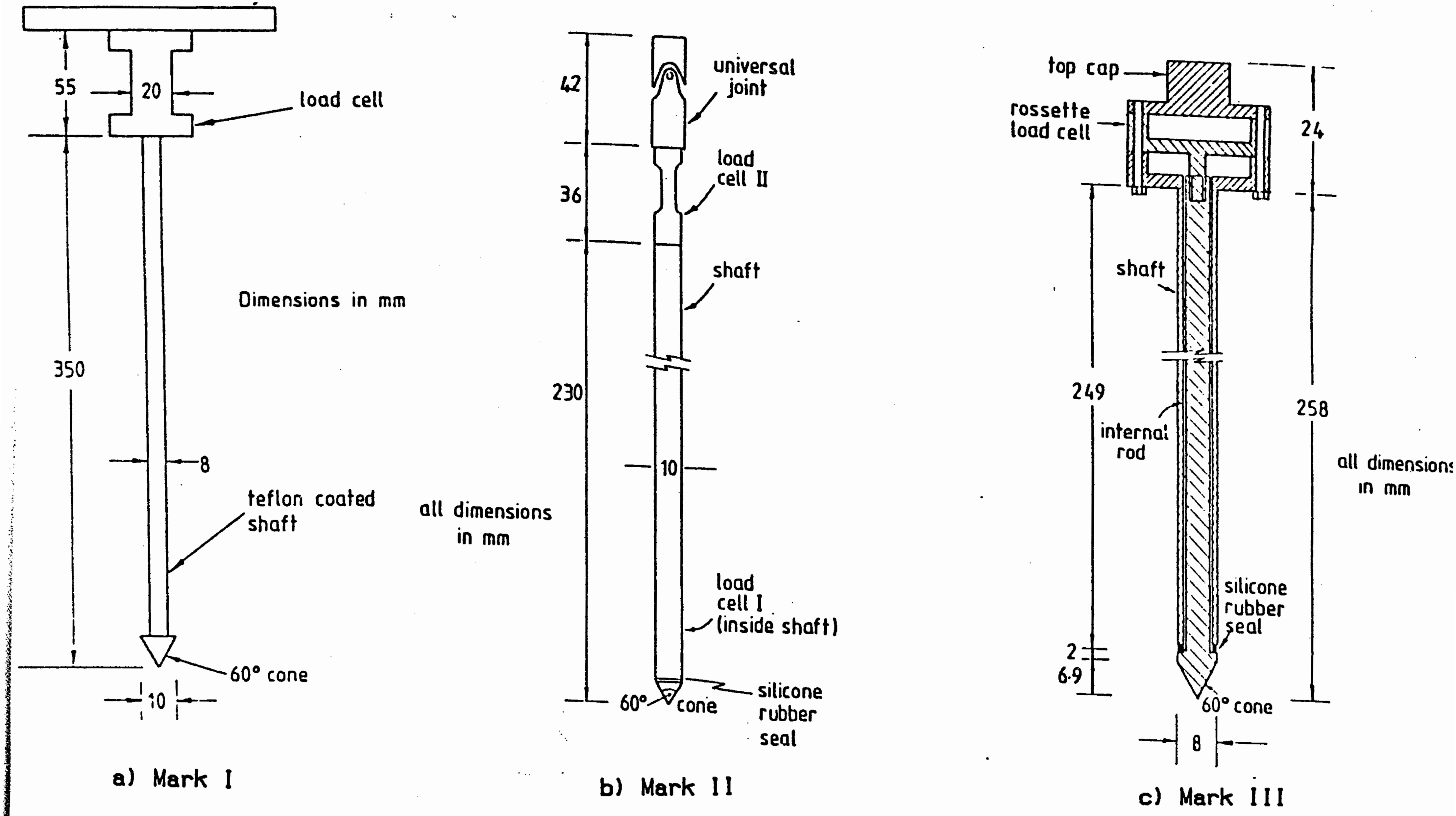


Fig. 1 Penetration Probes Marks I, II and III

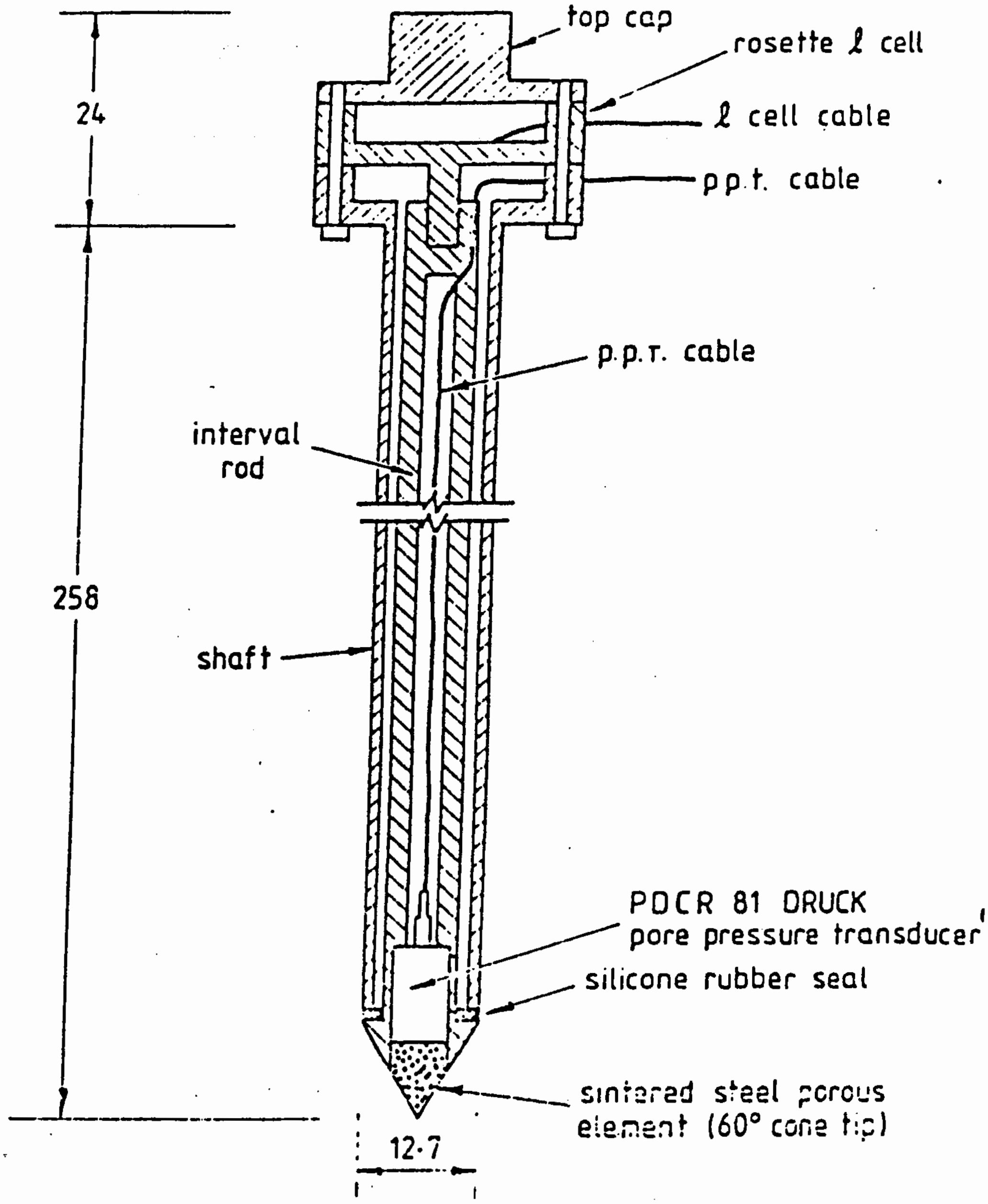


Fig. 2 Piezo-probe Mark I

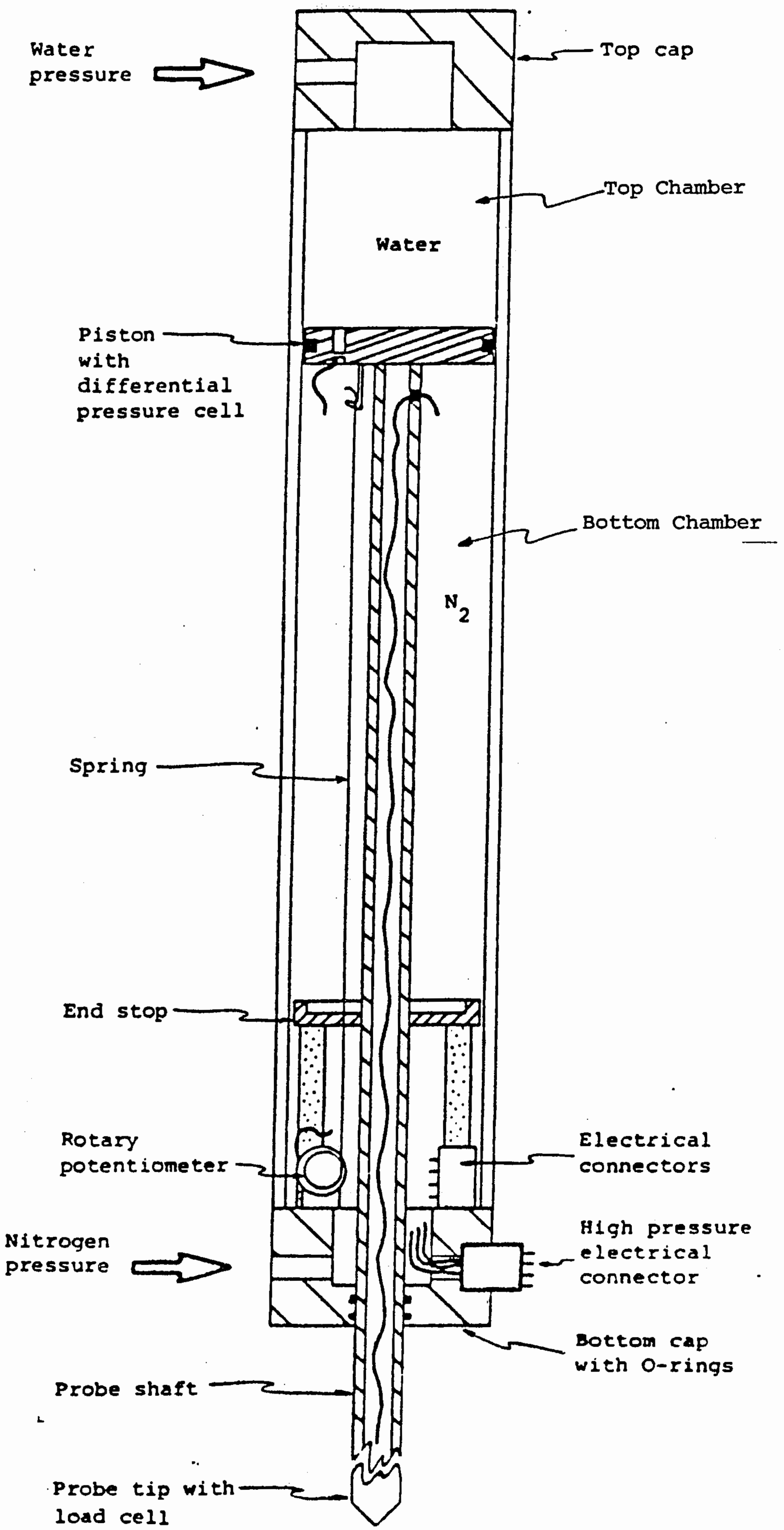


Fig. 3 Hydraulically - actuated Penetration Probe

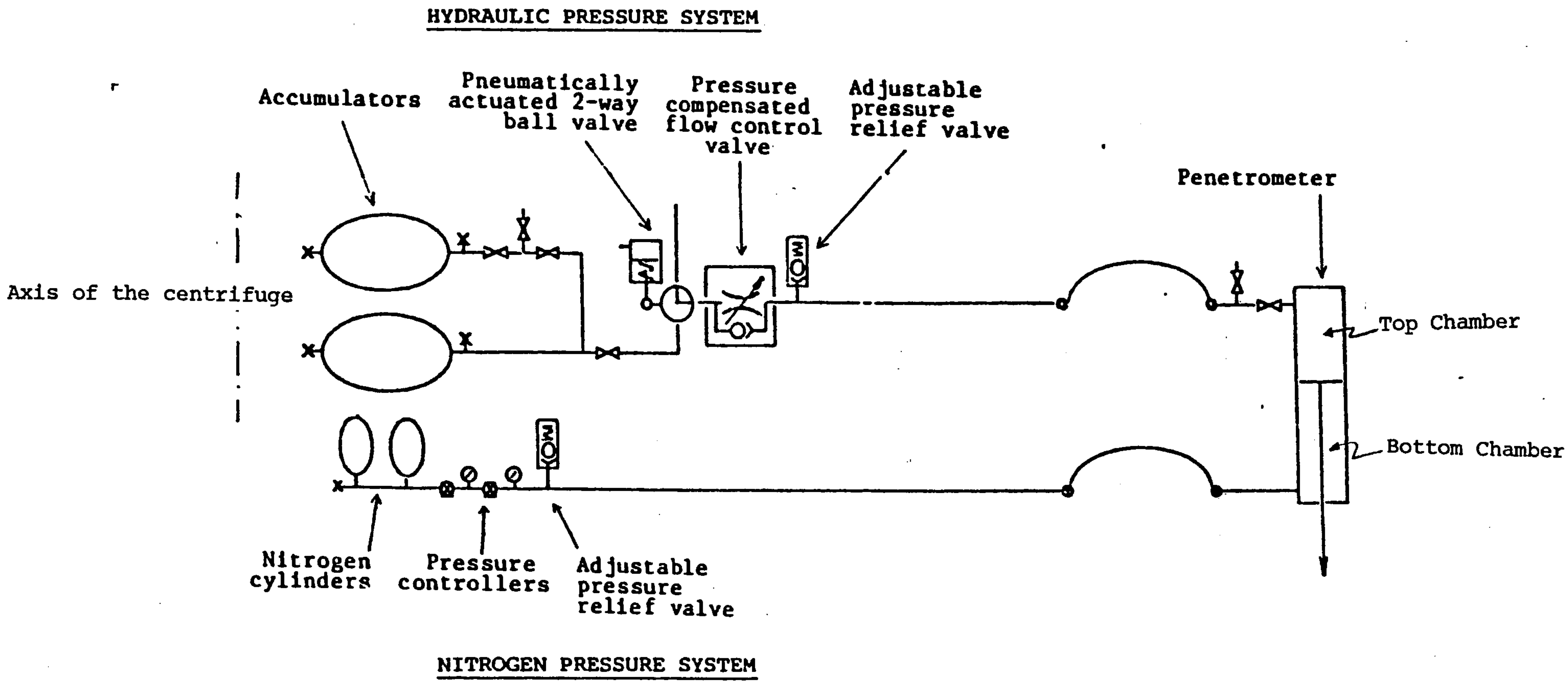


Fig. 4 External Pressure Control System for Hydraulically - actuated Penetration Probe

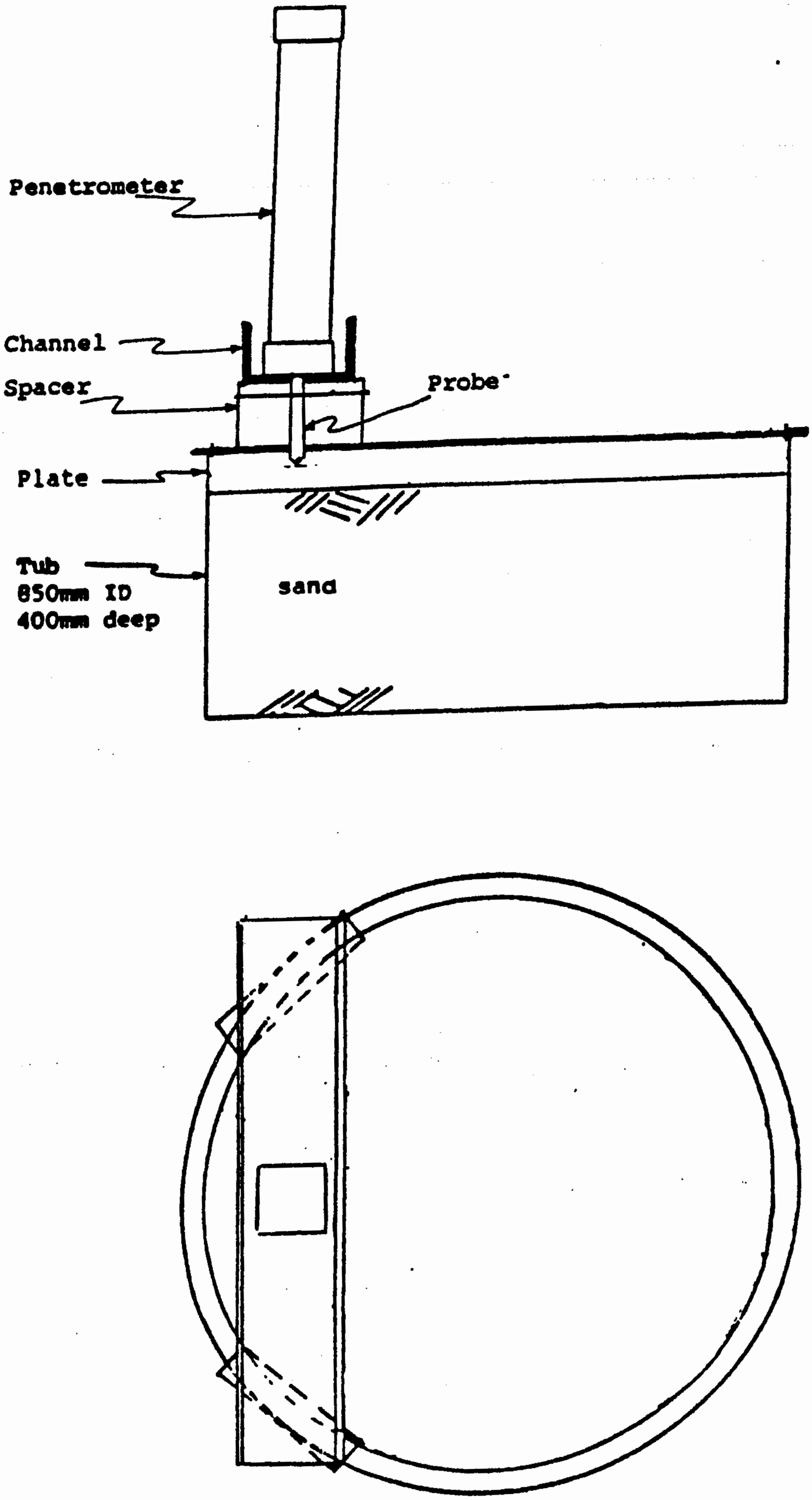


Fig. 5 Assembled Package for Centrifuge Test Series I

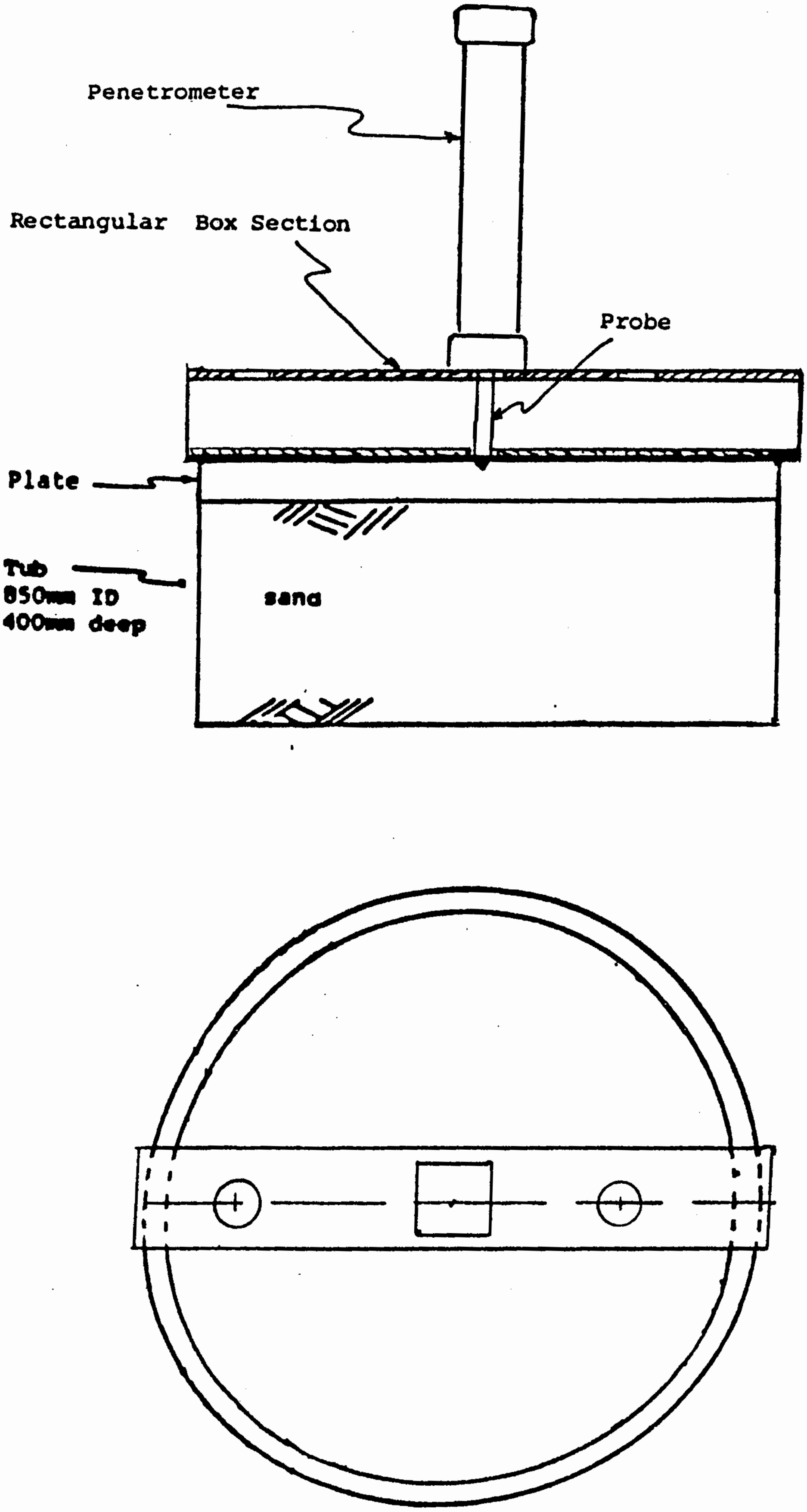


Fig. 6 Assembled Package for Centrifuge Test Series II

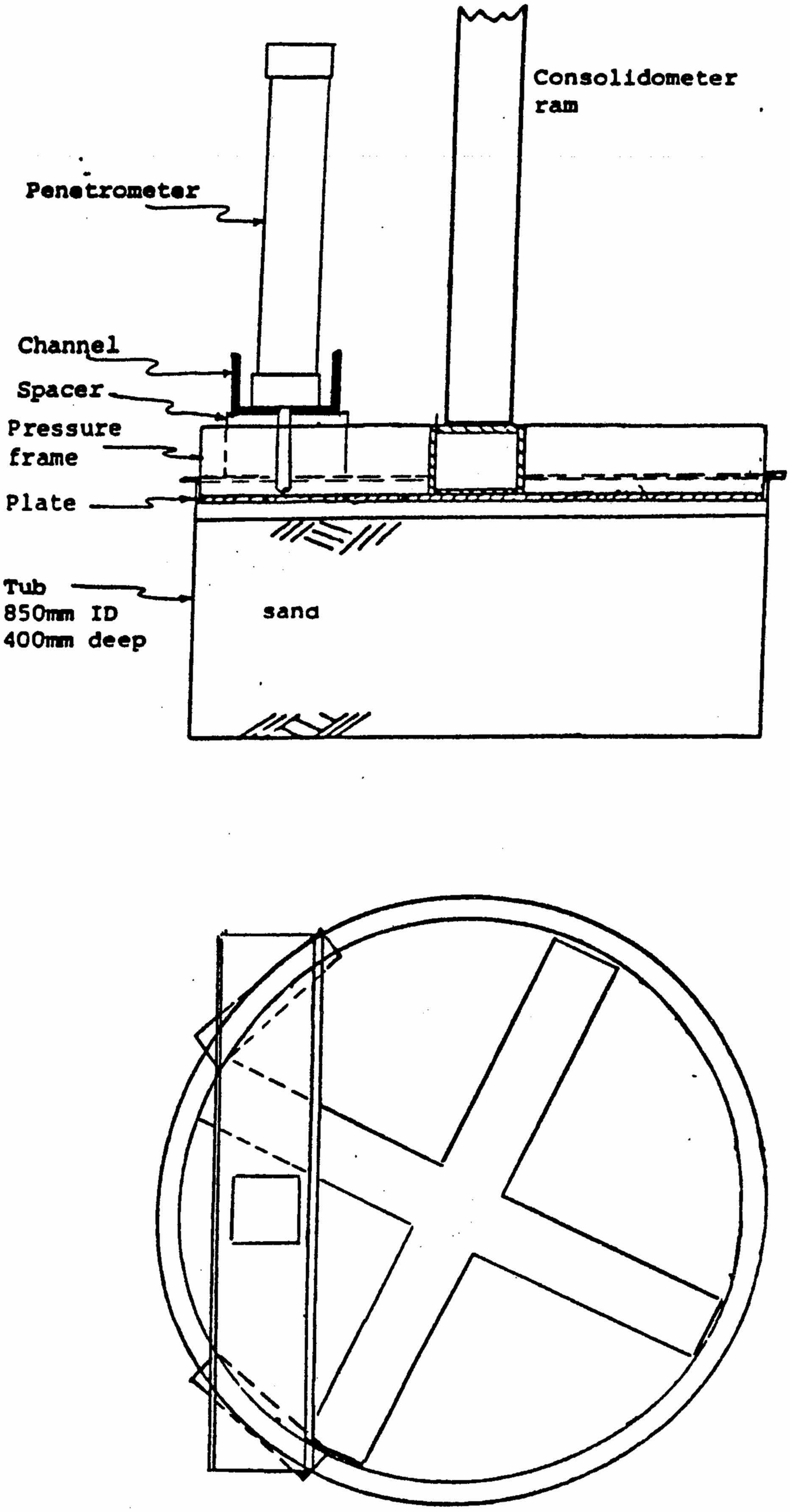


Fig. 7 Assembled Package for Laboratory 1 g test

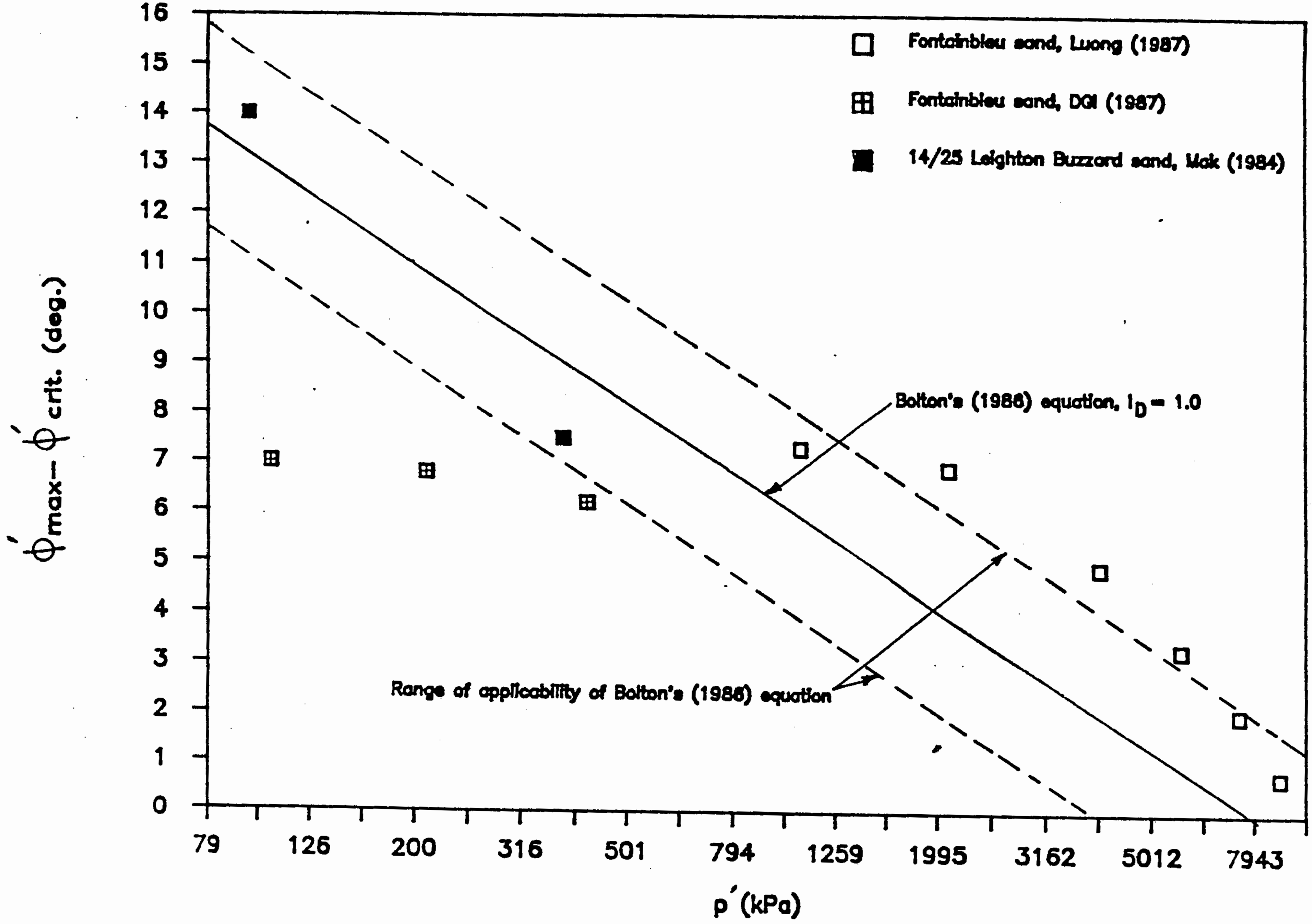
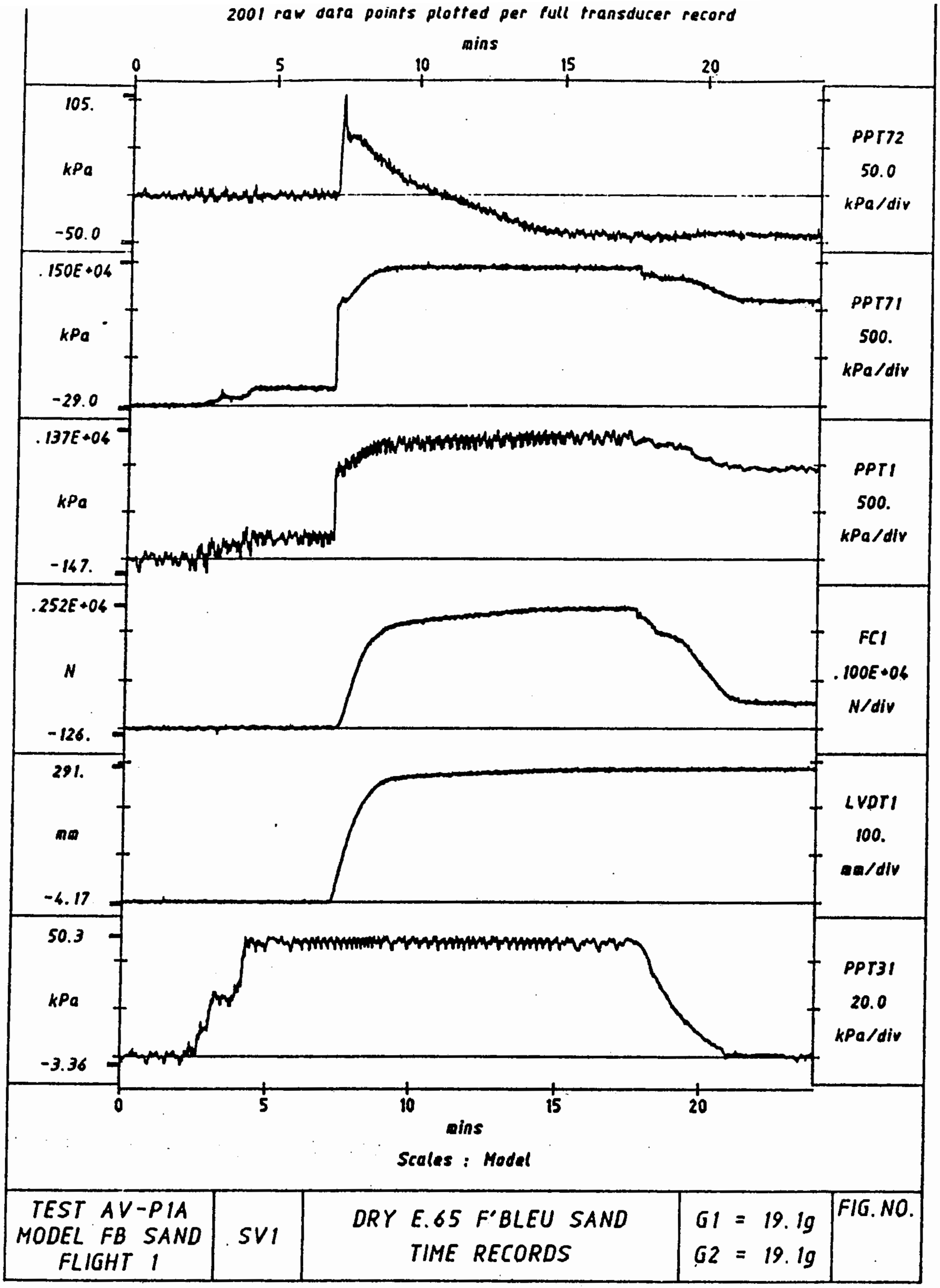


Fig. 8 Stress - Dilatancy Plot for Leighton Buzzard and Fontainbleu Sand



Instrument	ID	Units	Description
Rotary Pot.	LVDT 1	mm	Movement of penetrometer probe
Tip Load Cell	FC 1	N	Point resistance of penetrometer probe
Press. Trans.	PPT 1	kPa	Differential pressure across piston
Press. Trans.	PPT 71	kPa	Pressure at water inlet to piston
Press. Trans.	PPT 72	kPa	Pressure at nitrogen inlet to piston
G - meter	PPT 31	g	Nominal acceleration level in sample

Fig. 9 Typical Data Output from FLY 14 for Centrifuge Test

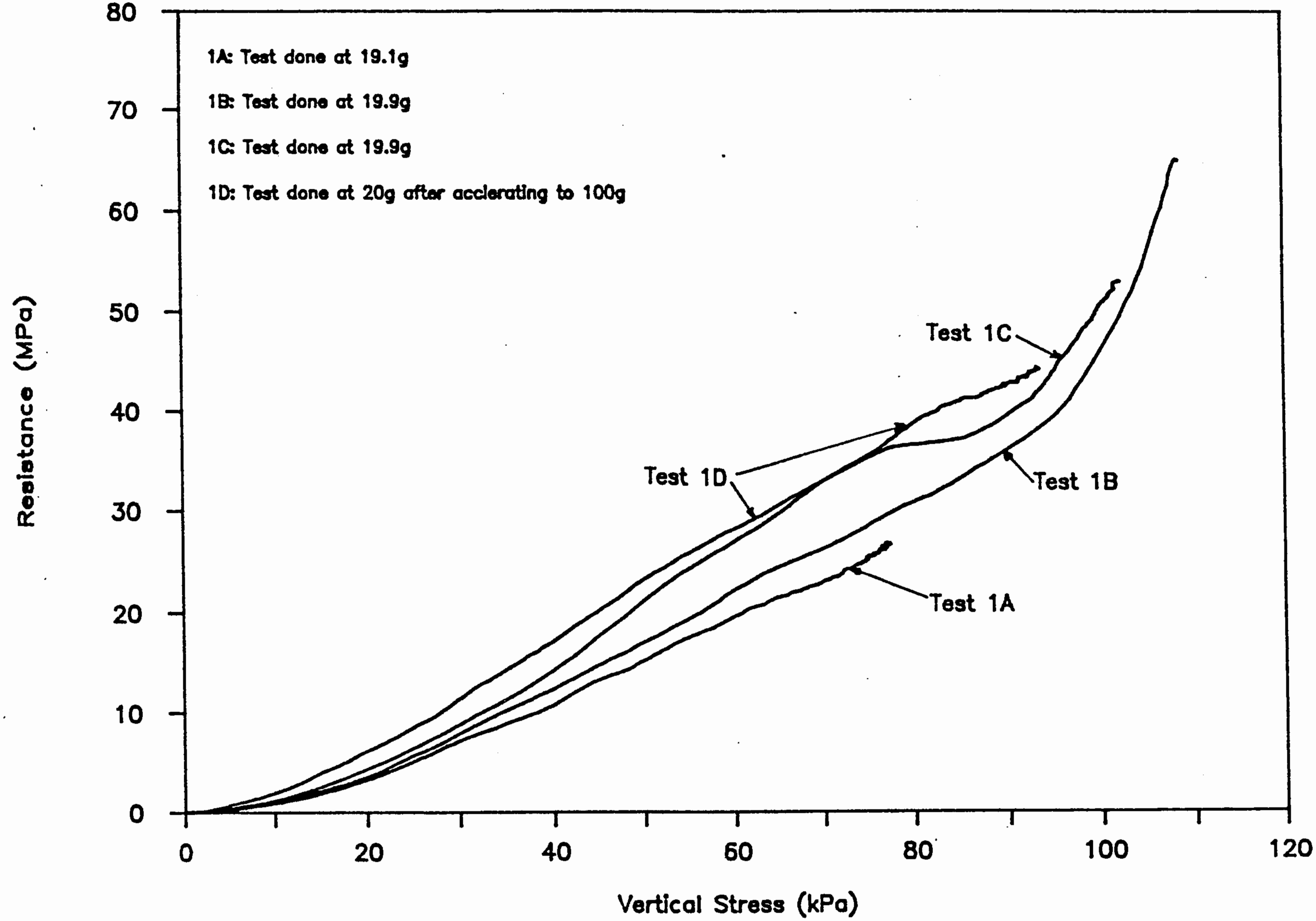


Fig. 10 Probe Resistance as a Measure of Uniformity of Sample Preparation

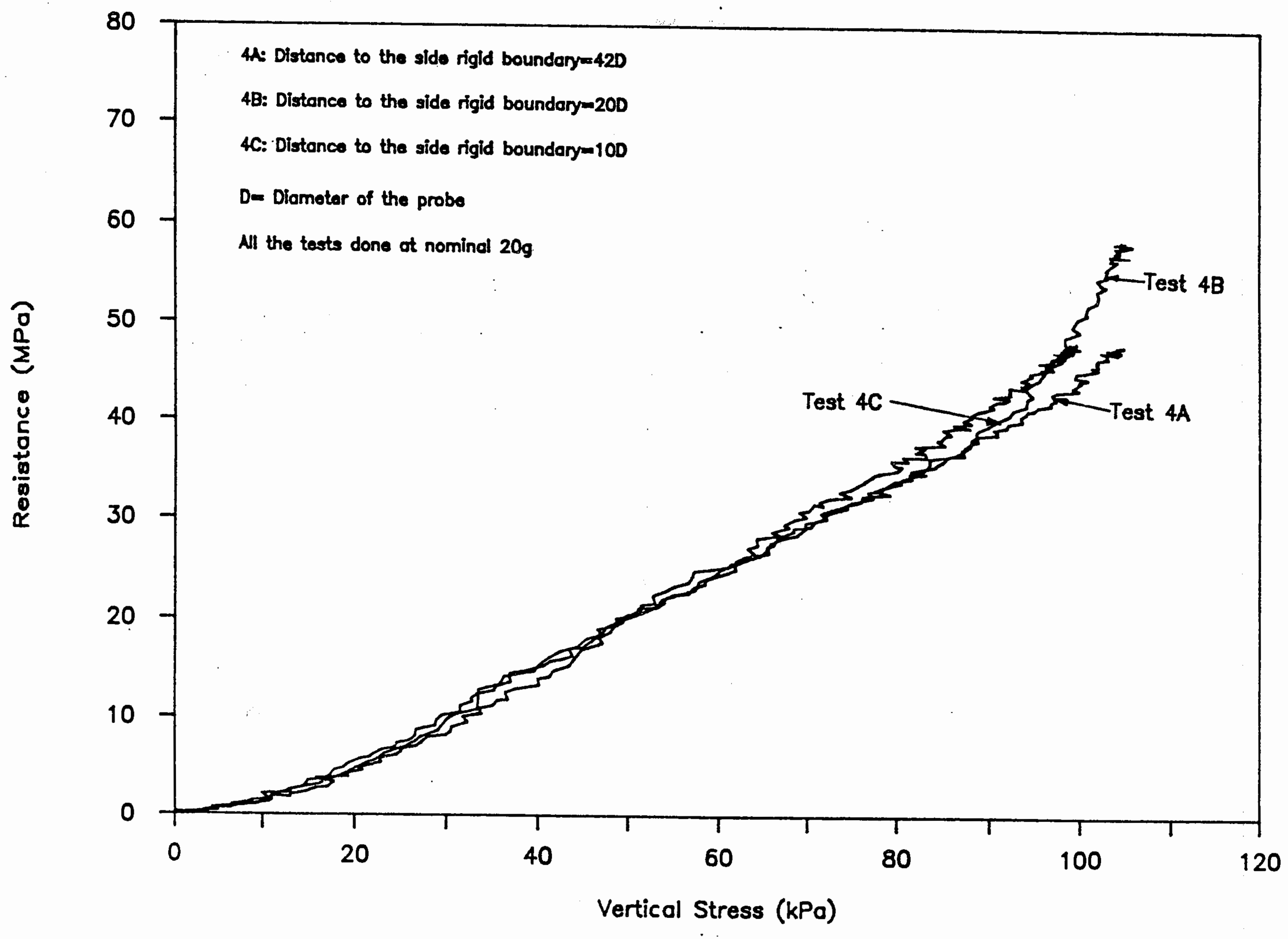


Fig. 11 Boundary Effects in Penetration Test: Dense Fontainbleu Sand

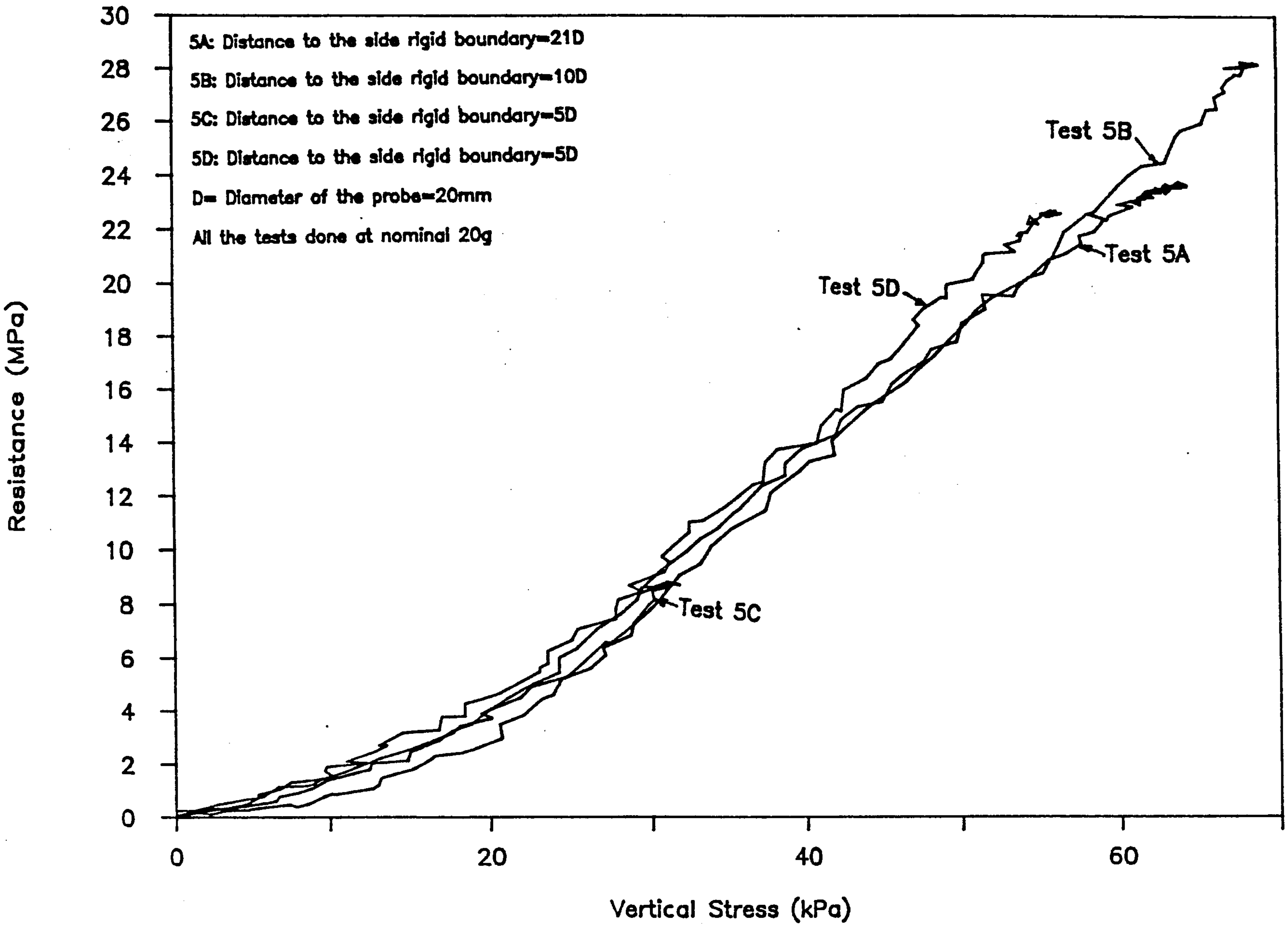


Fig. 12 Boundary Effects in Penetration Tests:
Dense 14/25 Leighton Buzzard Sand

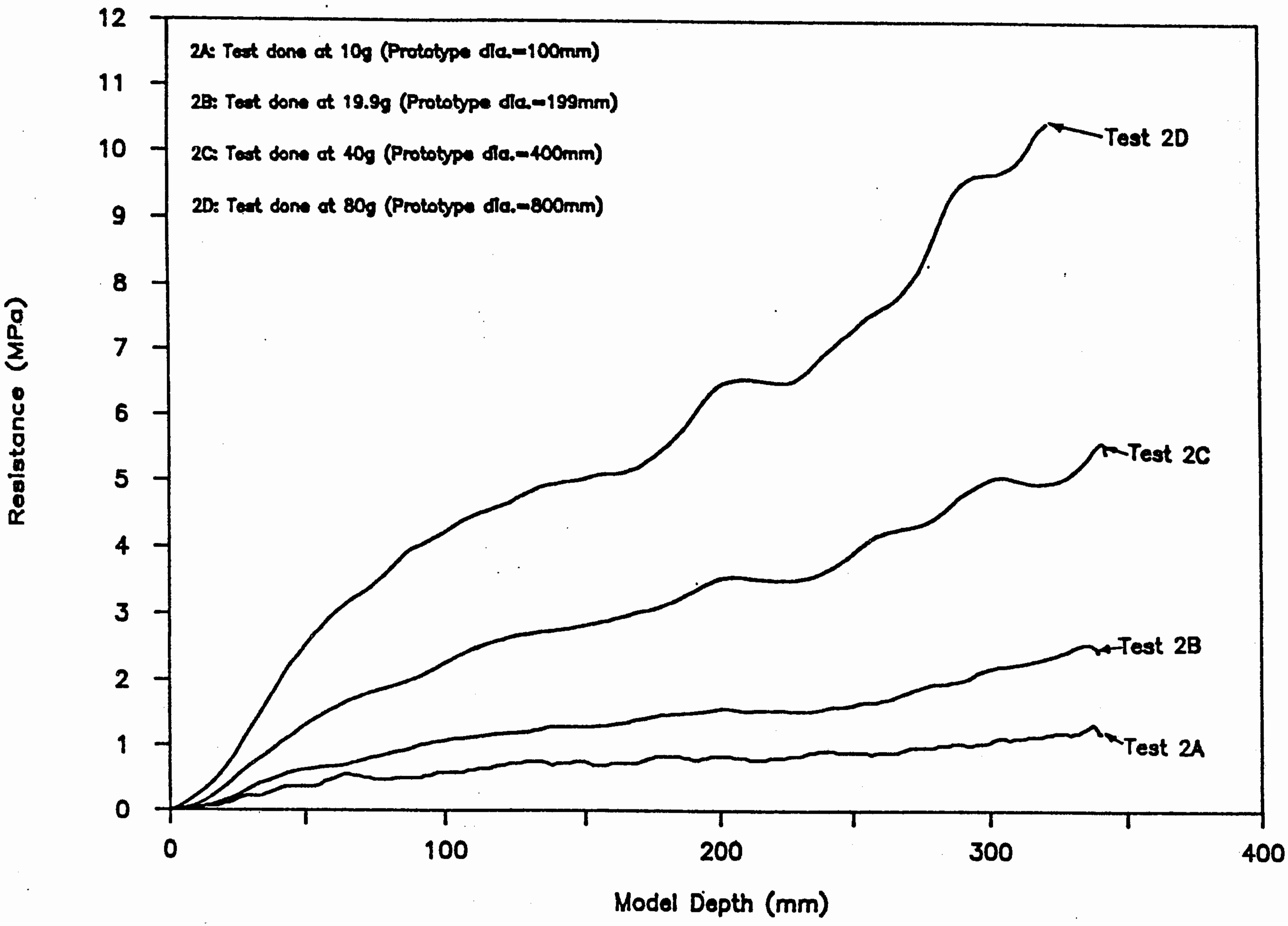


Fig. 13 Probe Resistance as a Measure of Uniformity of Sample Preparation

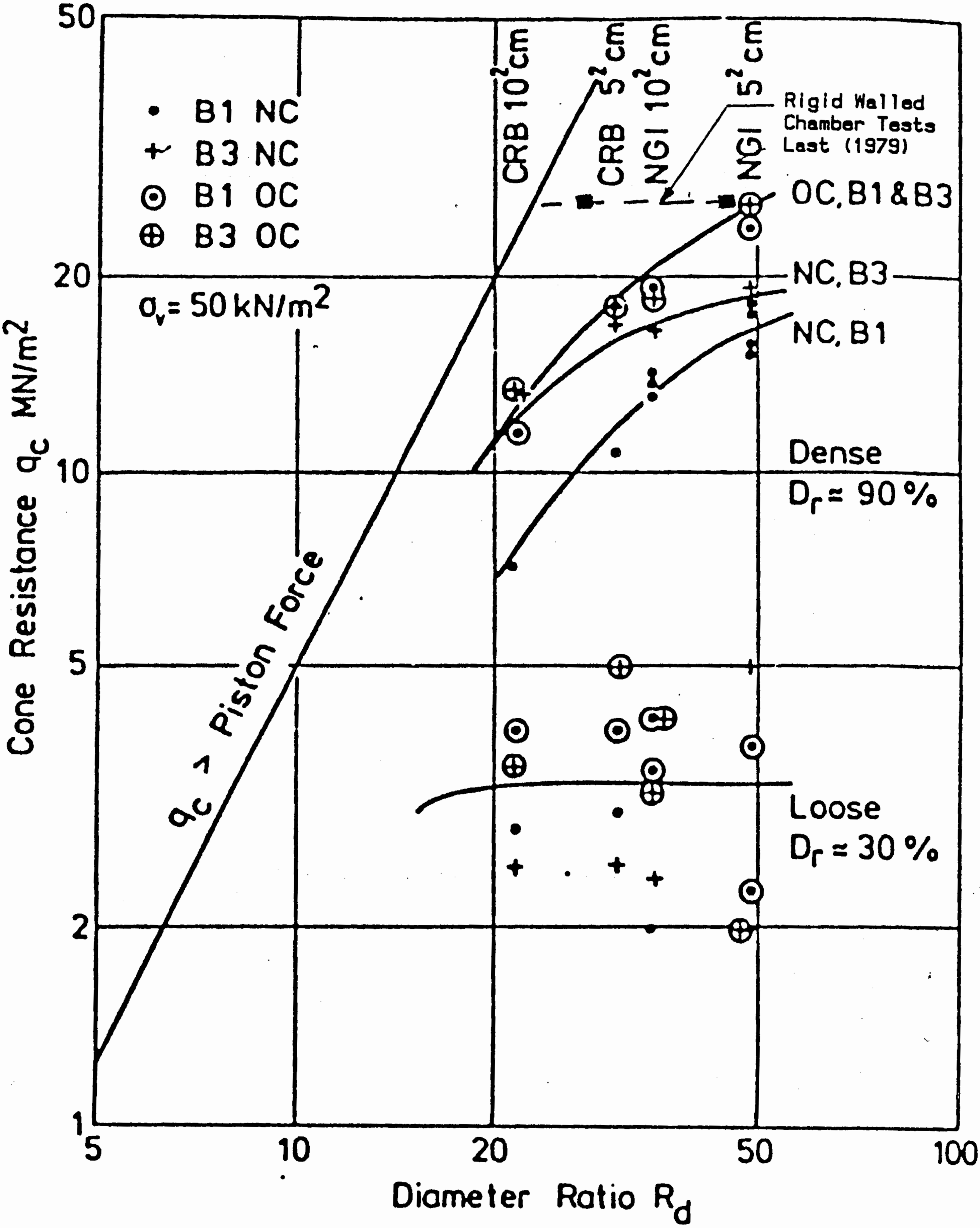


Fig. 14 Boundary Effects in Flexible and Rigid Walled Chamber Tests (After Parkin & Lunne, 1982)

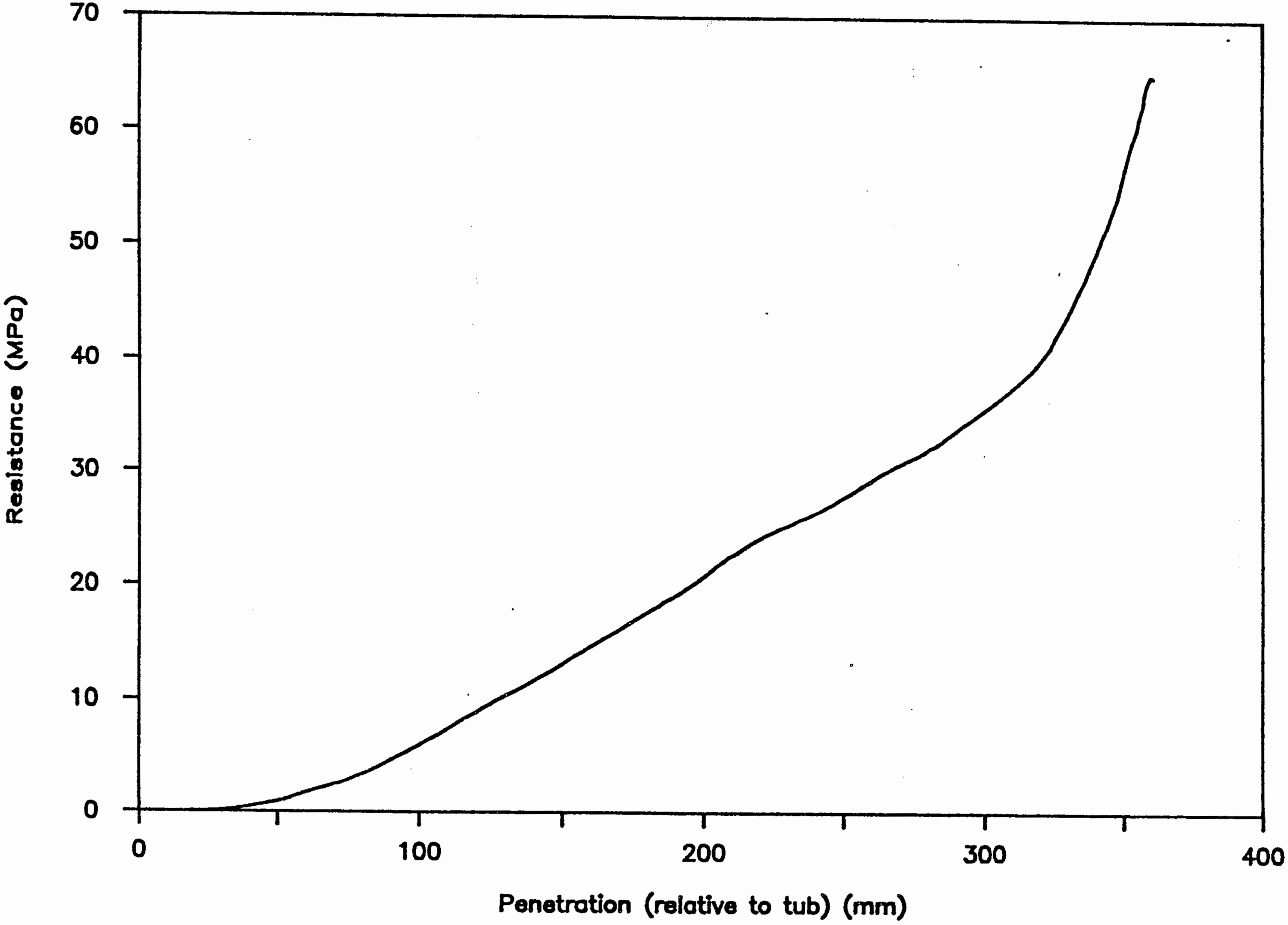


Fig. 15 Effect of Bottom Boundary on the Penetration Resistance:

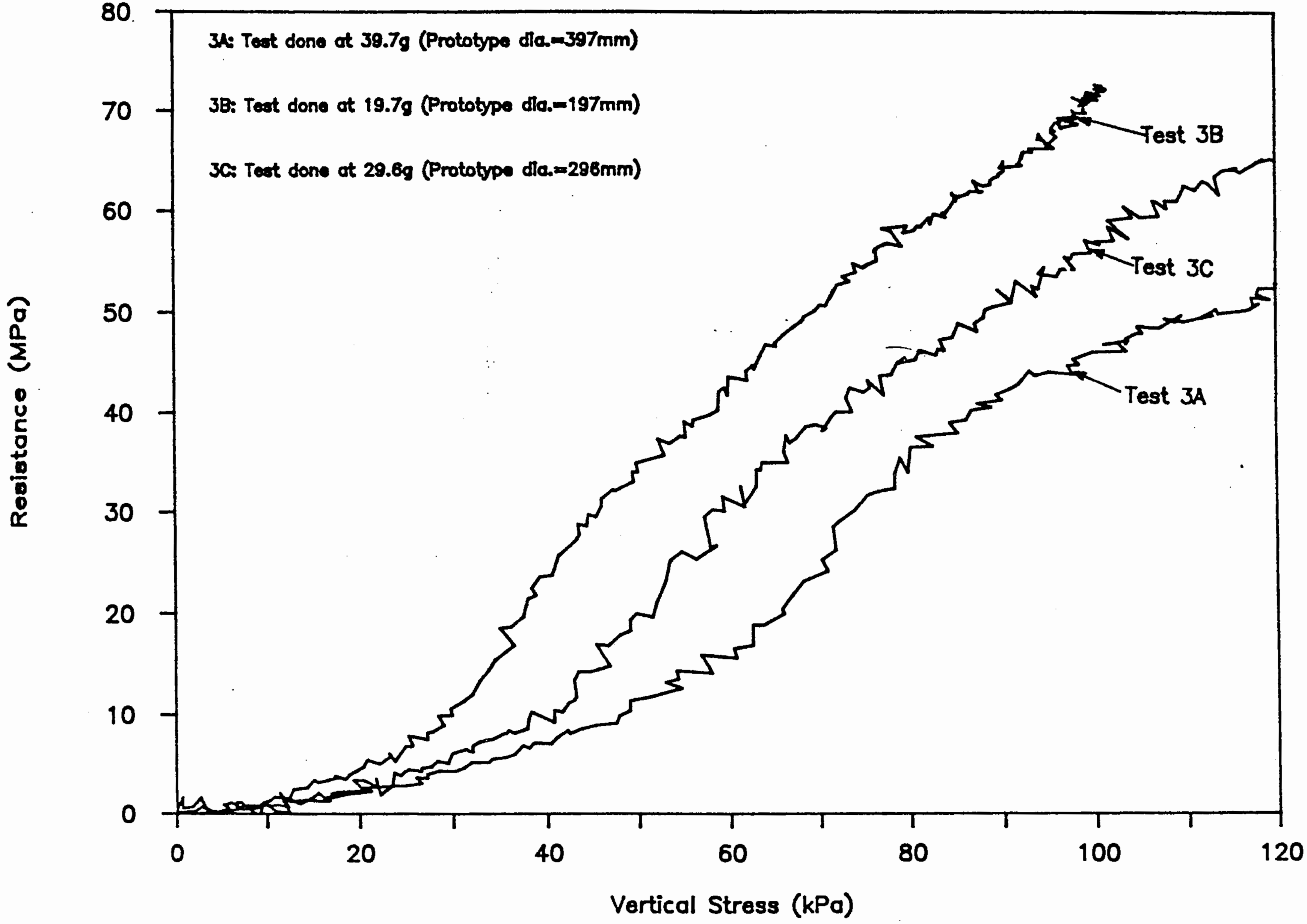


Fig. 16 Scale Effects in Penetration Tests:

Dense 14/25 Leighton Buzzard Sand

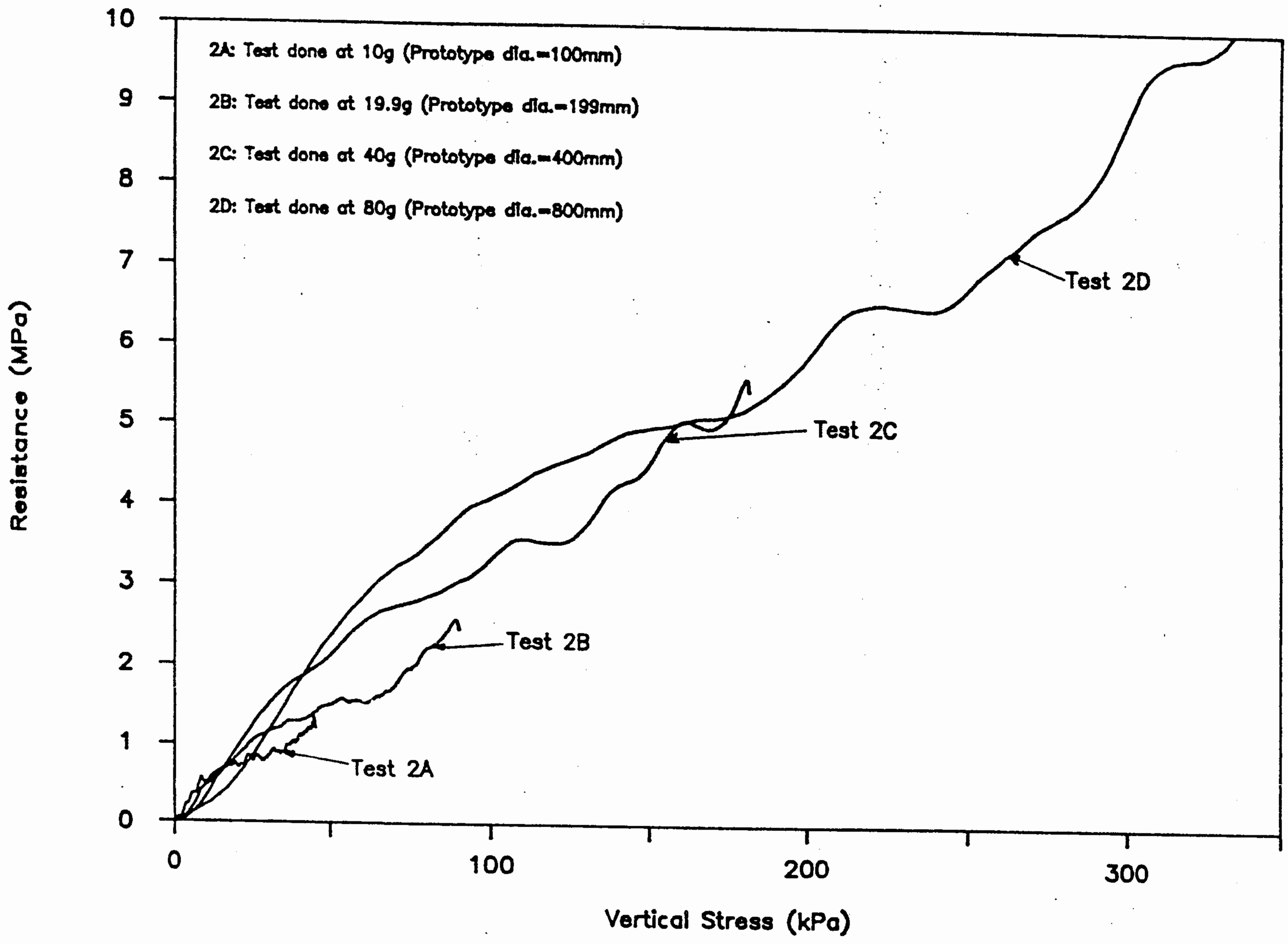


Fig. 17 Scale Effects in Penetration Tests: Loose Fontainbleu Sand

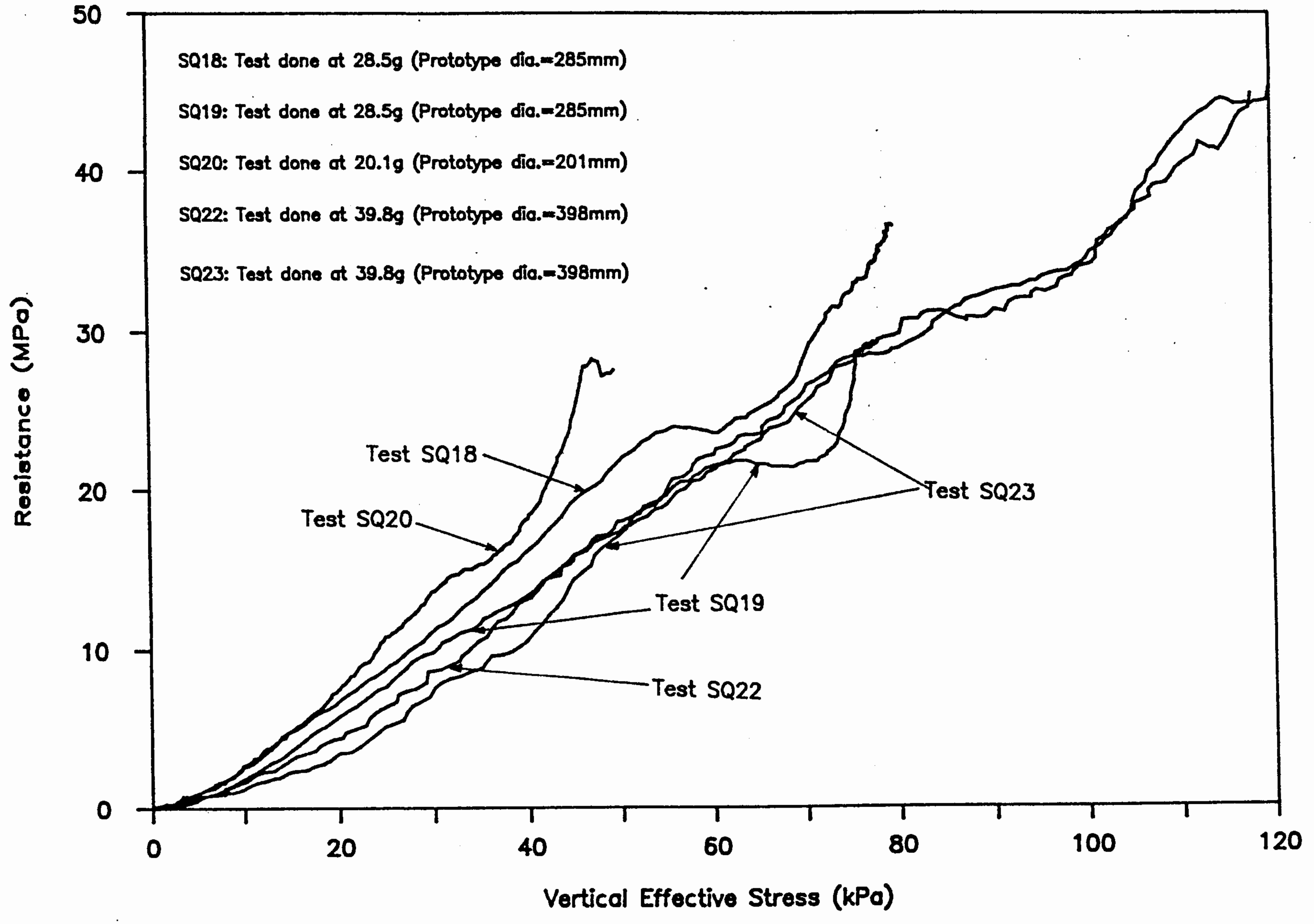


Fig. 18 Scale Effects in Penetration Tests:

Dense Saturated Fontainbleu Sand (after Shi, 1987)

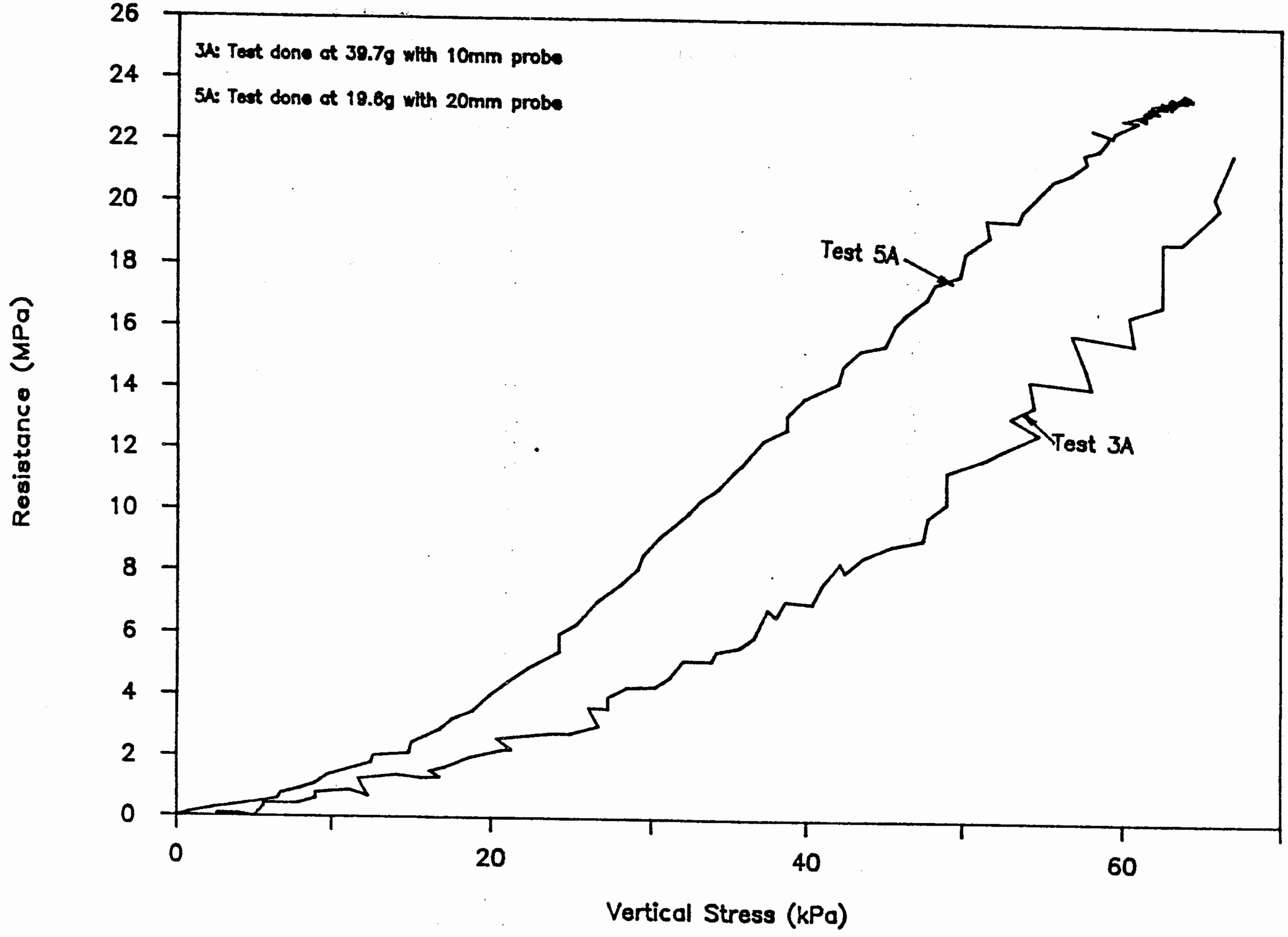


Fig. 19 Grain Size Effects: 14/25 Leighton Buzzard Sand

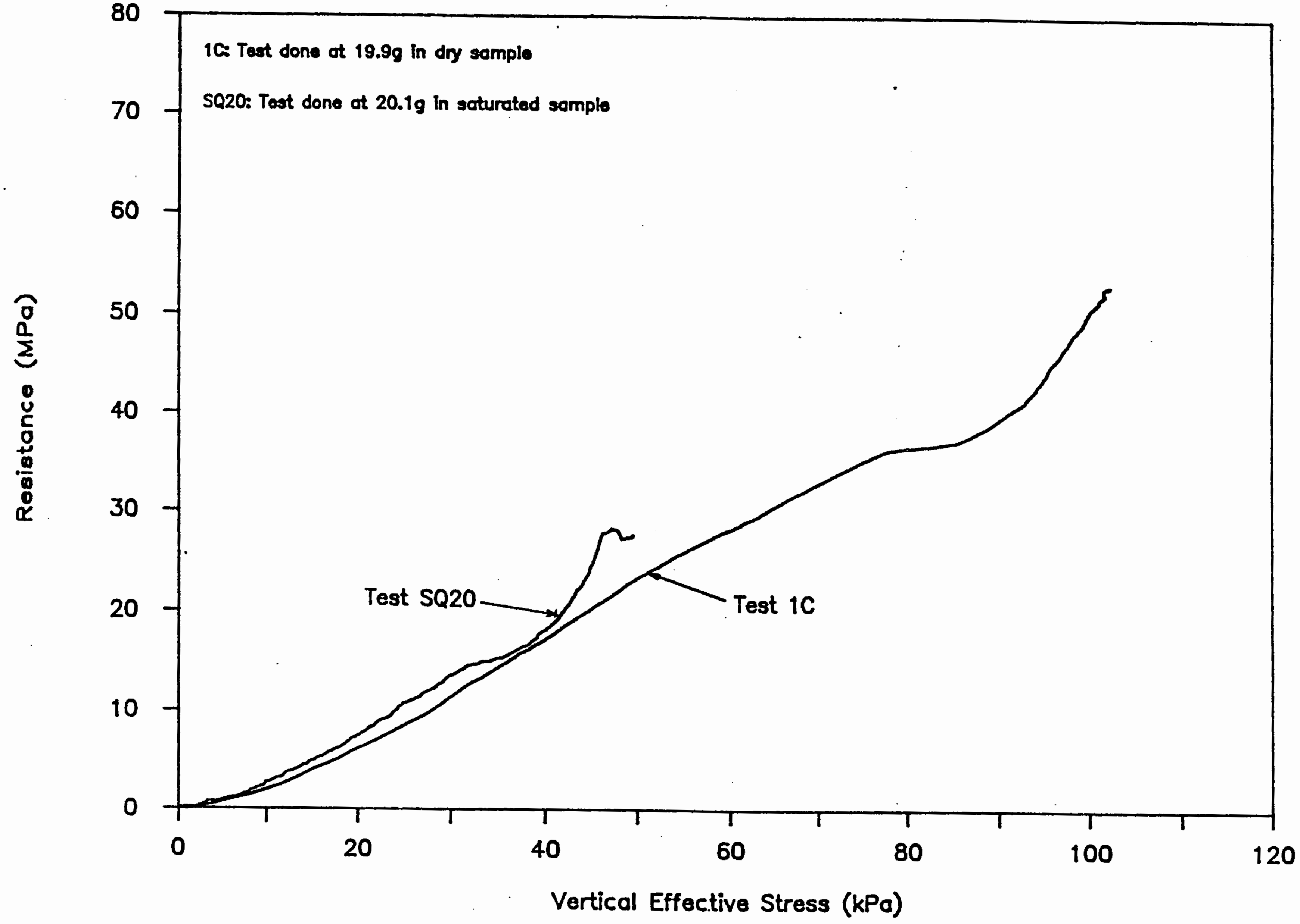


Fig. 20 Effect of Rate of Penetration of Pore Water Pressures Generated

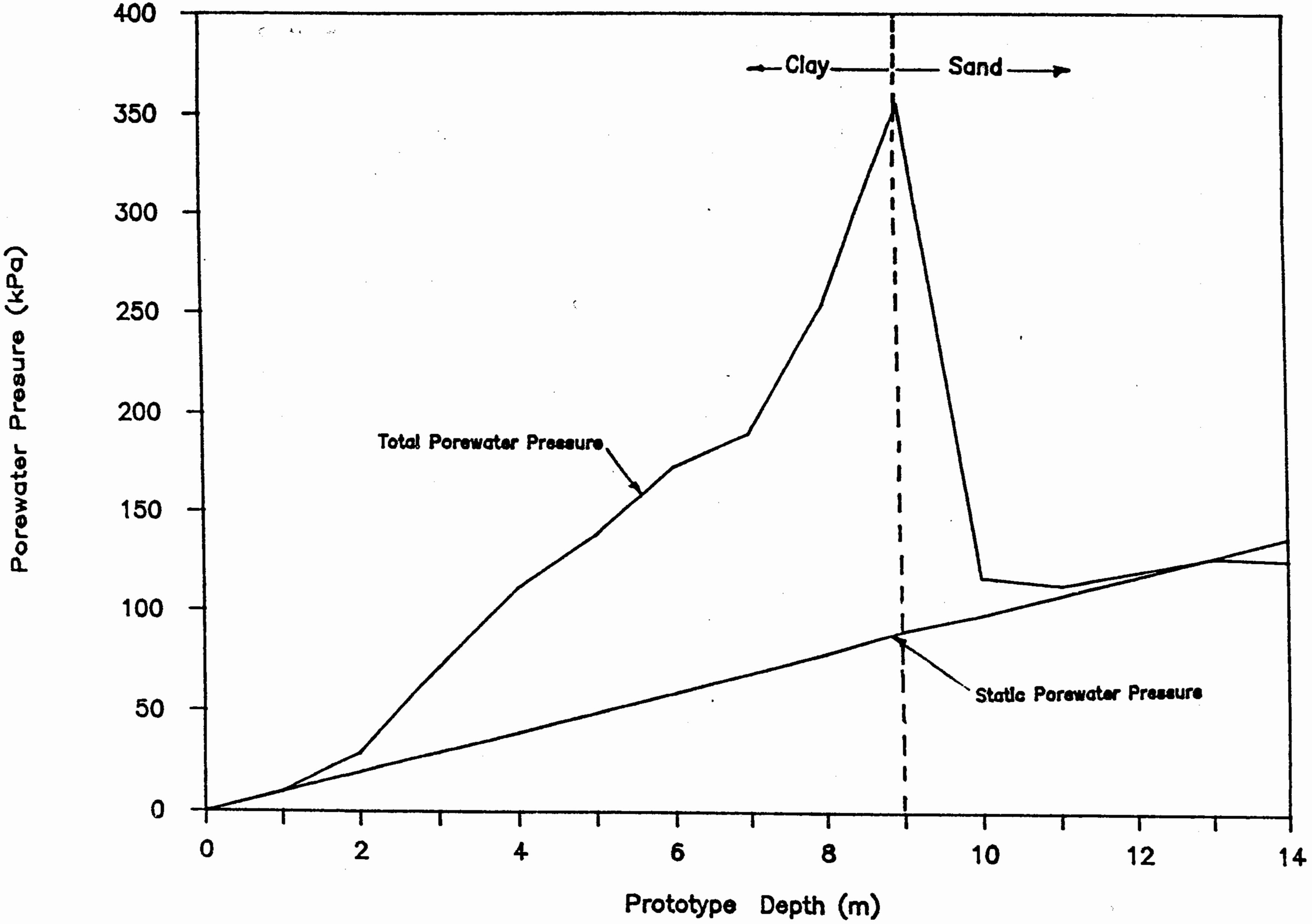


Fig. 21 Typical Data from Piezo-probe (after Springman, 1987)

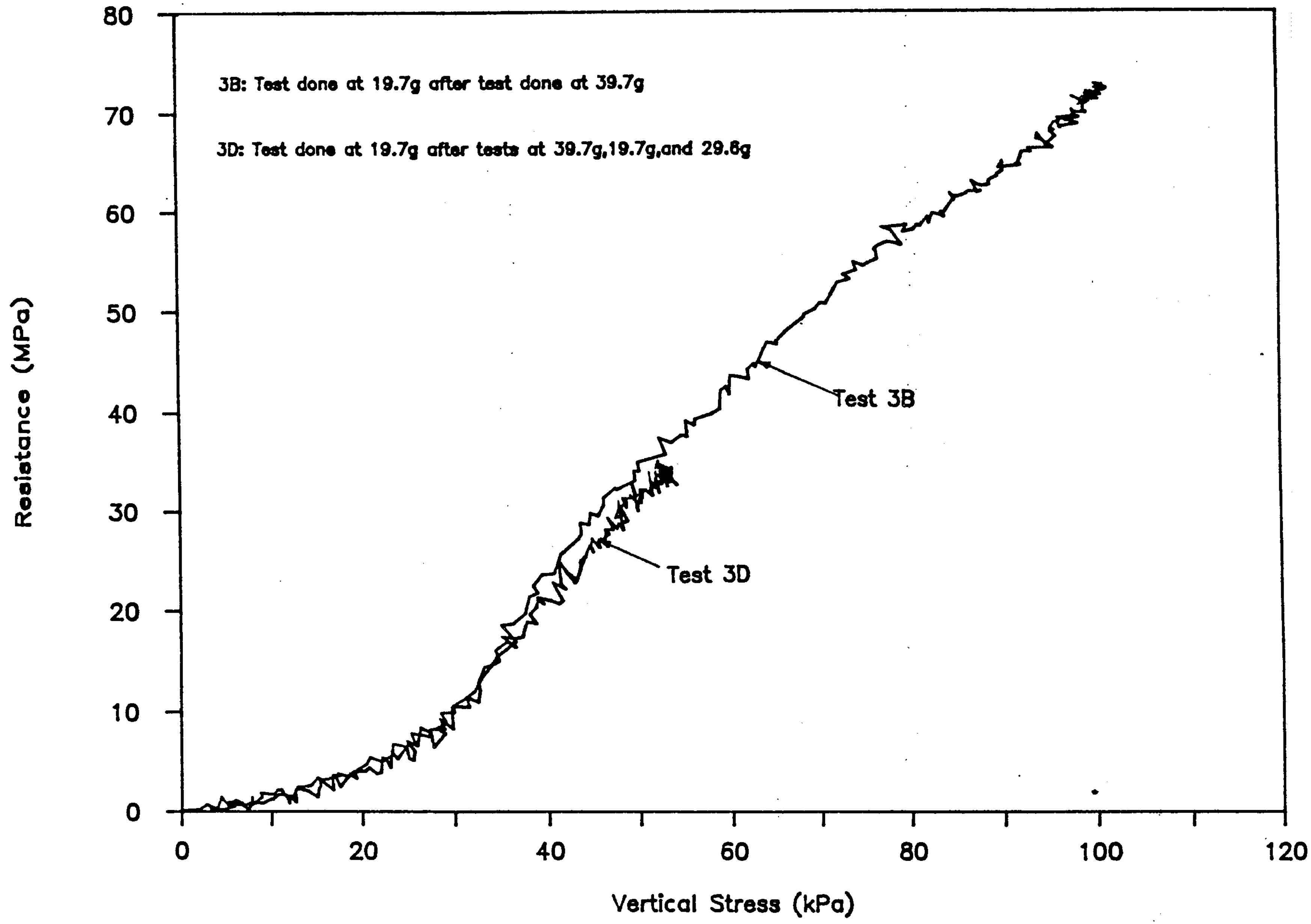


Fig. 22 Effect of Stress Cycling Due to Stoppage of Centrifuge

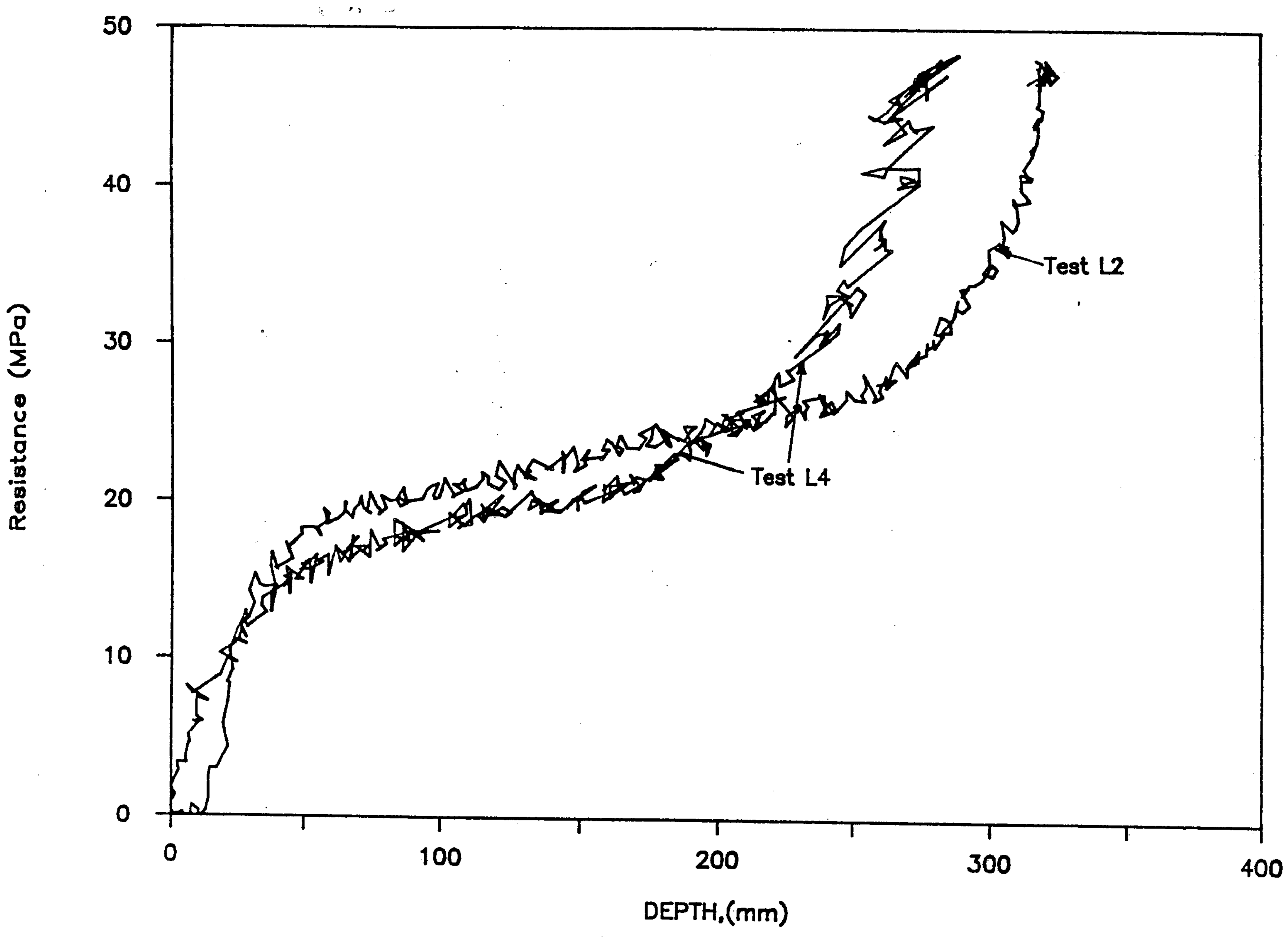


Fig. 23 Penetration Resistance for Surcharge = 50 kPa

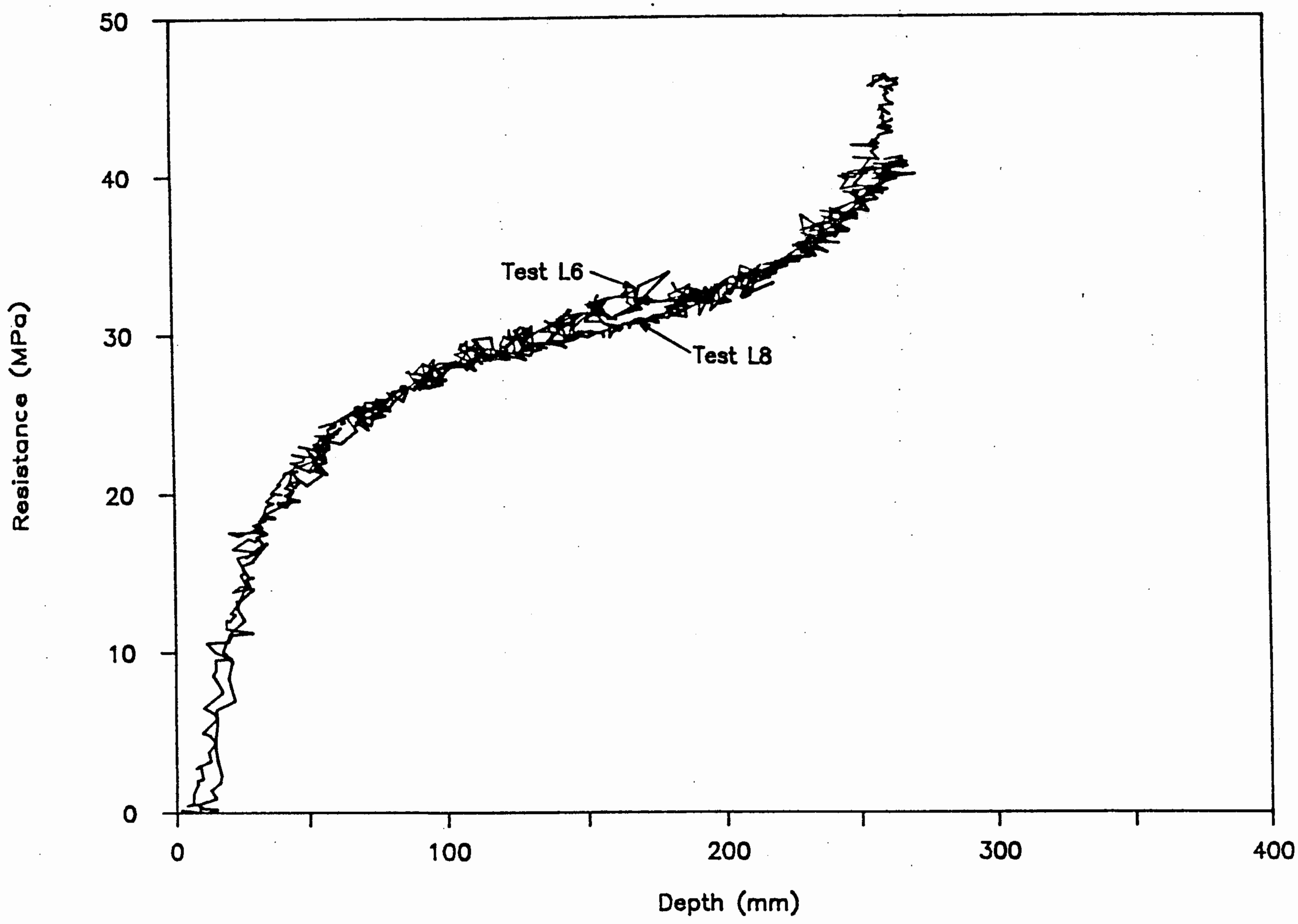


Fig. 24 Penetration Resistance for Surcharge = 100 kPa

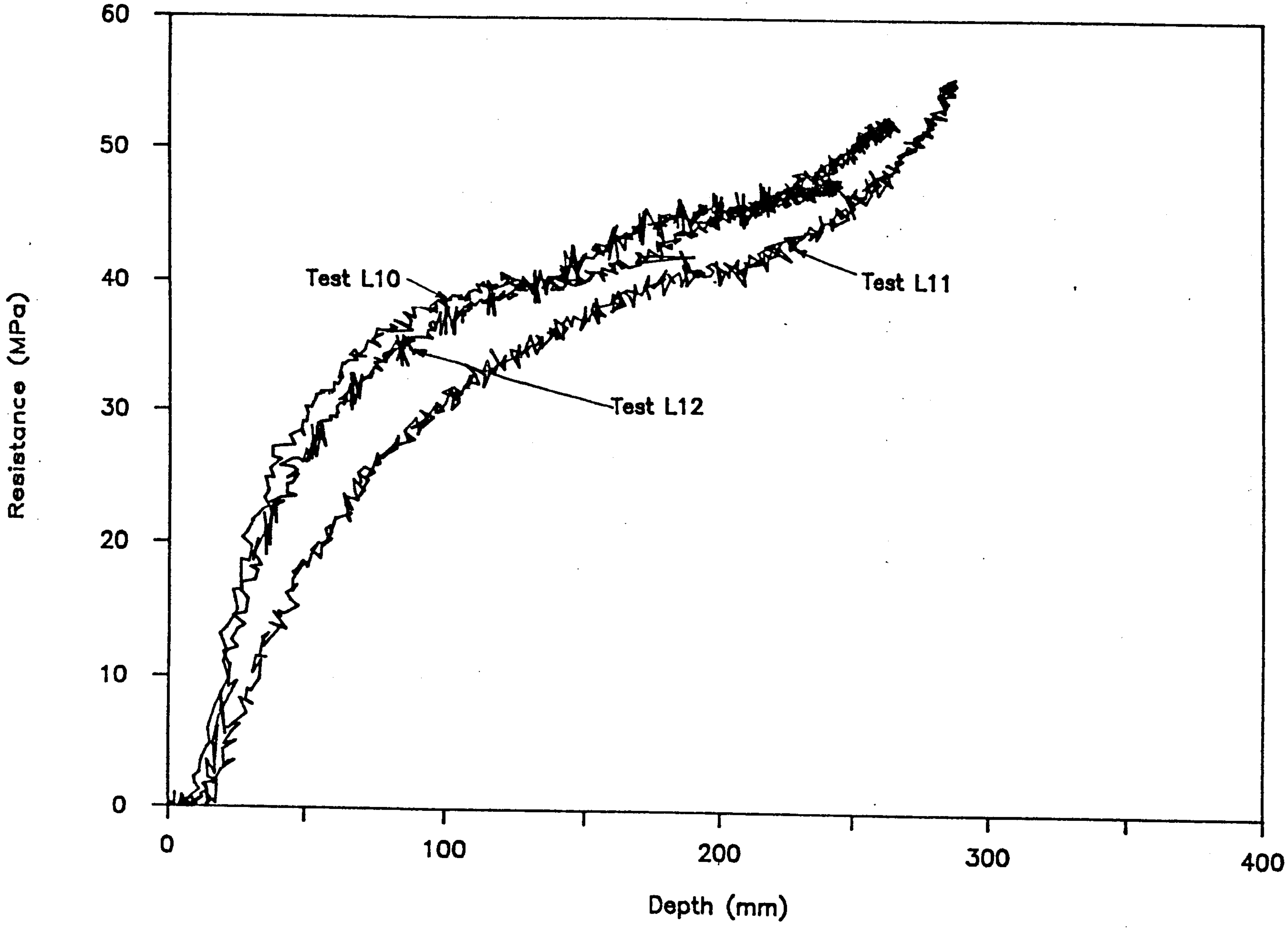


Fig. 25 Penetration Resistance for Surcharge = 150 kPa

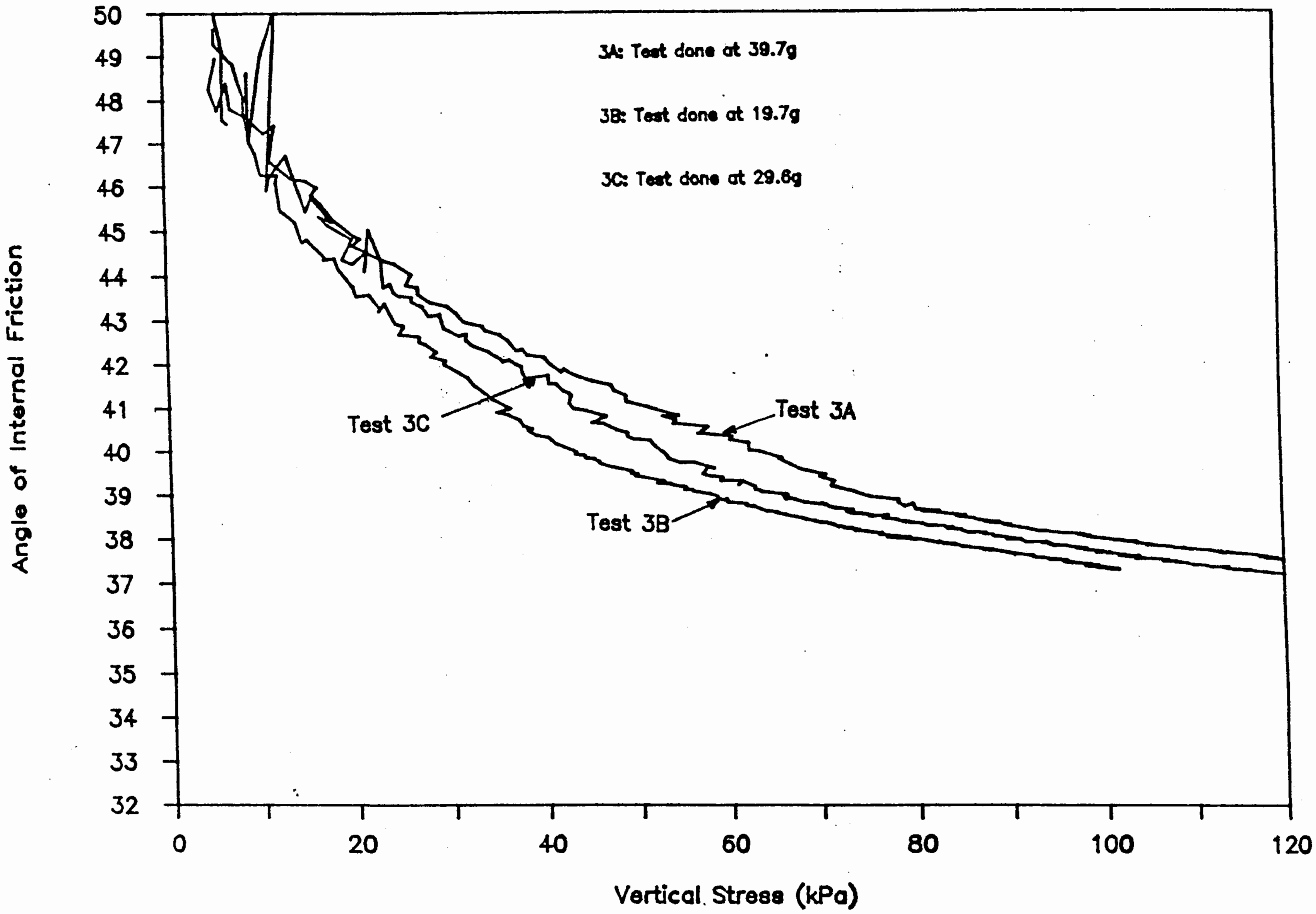


Fig.26 Size and Stress Level Effects on Mobilized Angle of Internal Friction: Dense 14/25 LB Sand

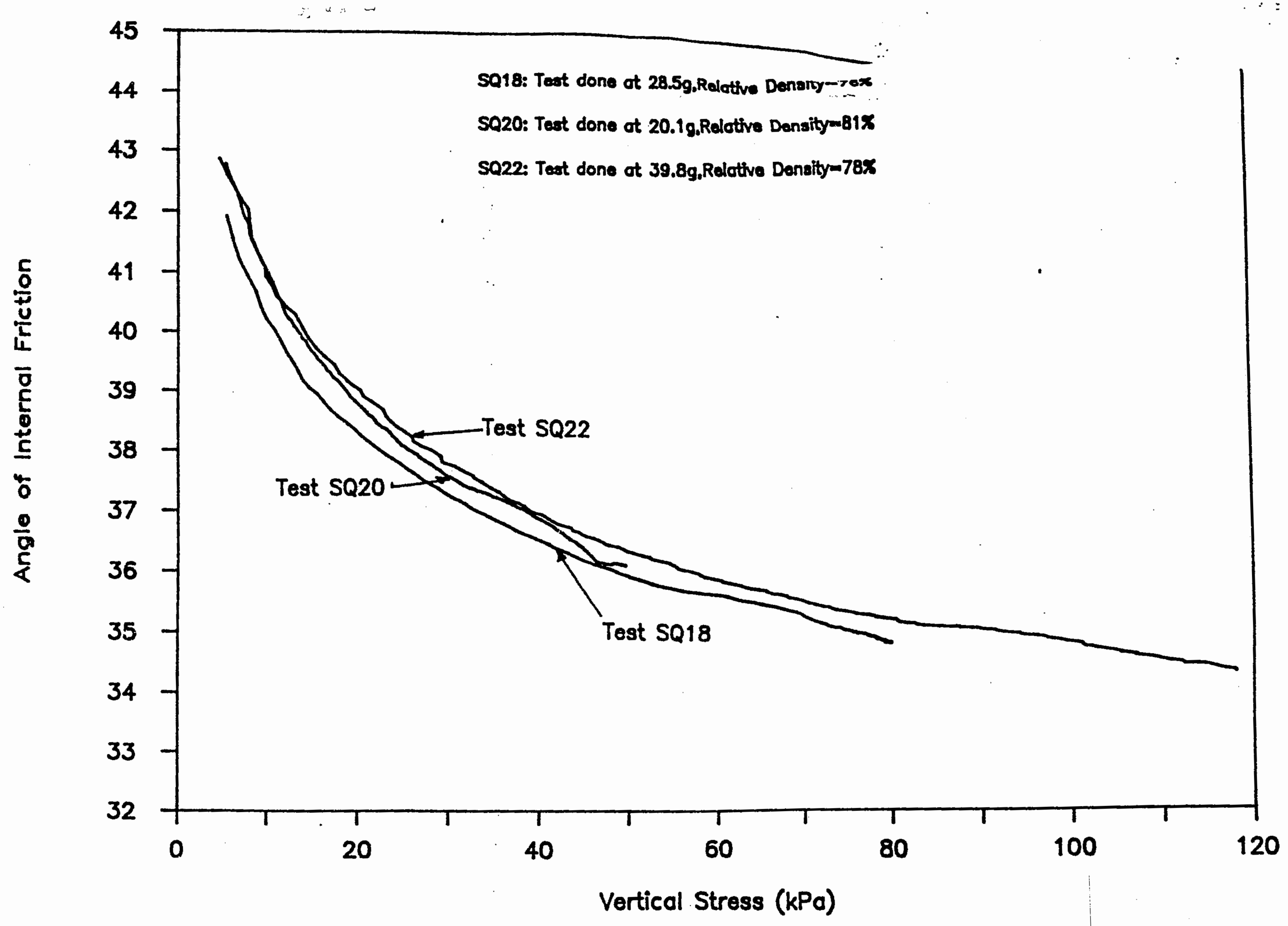


Fig.27 Size and Stress Level Effects on Mobilized Angles of Internal Friction: Dense Fontainbleu Sand