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DEPARTMENT OF CIVIL AND STRUCTURAL ENGINEERING

FURTHER STUDIES ON THE REPEATED LOADING OF PILES IN SAND

by

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TO

THE MEMORY OF MY FATHER

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SUMMARY

The work presented in this thesis concerns the behaviour of isolated piles subjected to repeated loading and placed at various depths in a medium dense sand upon which either static or cyclic surcharge acted. The piles, which were of laboratory scale, were instrumented by strain gauged load cells located along the inner surfaces of the pile shafts. The behaviour of tension as well as compression piles was examined. It was found that the behaviour of the pile was governed to a large extent by the repeated load level, the number of load cycles and the initial boundary stress conditions existing along the pile shaft. In compression, the pile life-span decreased when the embedment depth increased while the reverse trend was observed for tension piles. The movement of both tension and compression piles decreased when the surcharge pressure was increased or was cycled, and it was of a minimum value when the upper repeated load acted in-phase with the higher surcharge pressure. For tests performed with static surcharge pressure, repeated loading was found to decrease the bearing capacity and the pulling resistance of the pile. The higher percentage of reduction was recorded for the tension pile. In contrast, after cyclic surcharge tests the pile capacity always increased.

At any depth of embedment or surcharge pressure, as the number of load cycles was increased the shaft load of a compression pile increased up to a peak value then decreased gradually until it reached a limiting value. This limiting value increased when the load level, the pile depth or the surcharge pressure was increased and it was independent of the pile loading history.

For a tension pile the shaft load decreased progressively as the number of cycles increased until failure occurred.

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CHAPTER 1

INTRODUCTION

1.1 The Need for the Investigation

The loads on piles, in general, have two main forms, dead and live. The former comprises the self weight of the structure, foundation, and all other loadings that are independent of time, whilst the latter includes loadings which are time dependent such as wind and wave pressures and other moving loadings.

The repetition and ratio of live to dead load may in certain circumstances be so small that the pile experiences undetectable changes in behaviour. In contrast, the drastic variations in the system of forces that act upon off-shore structures and the high live to dead load ratio plus the large number of loading cycles may cause a sudden change in the performance of foundation piles (McClelland 1974). Not only off-shore structures encounter high live to dead ratios but also narrow tall buildings, transmission towers, long span suspension bridges, and silos. During the life-time, each of these structures is liable to a loading randomly repeated in one or two directions with different amplitudes. Therefore the stability of these structures might also be severely affected. Failure of piles under repeated loading was the subject of several authors whose works are described in the following paragraphs.

Chan (1976) described the state of failure in compression model piles subjected to repeated loading as follows. Initially the pile was stable, but after a certain number of cycles, which increased with decrease of the load amplitude, the rate of pile movement began to increase rapidly until another stable stage was reached. In the case of tension piles, he found that the rate of movement initially decreased then after a stable stage it began to increase until the pile was pulled out.

Matlock (1979) indicated that the resistance along the upper portion of a pile subjected to tensile repeated loading was progressively lost. The field observations by Kraft et al (1981) showed that the load transfer characteristics of piles were affected when the loading was repeated. Even the theoretical study by Poulos (1981) revealed that under the influence of repeated loading both the ultimate capacity and the load transfer of the pile are affected.

A search through the literature indicates that only a very limited area of this wide subject of repeated loading has been investigated.

1.2 The Scope of the Investigation

In view of the limited published information the following topics have been considered in the present investigation:-

(A) The influence of pile depth on the life span of compression and tension piles when:-

(i) The pile is subjected to repeated loading while the sand surface is acted upon by a constant surcharge.

(ii) The pile is subjected to sustained loading while the sand surface is acted upon by a surcharge cycled between two limits.

(iii) The pile is subjected to repeated loading while the sand surface is acted upon by a surcharge cycled between two limits. This type of loading consists of the following three modes of testing.

a) The application of the cyclic surcharge on the sand surface is independent of the repeated load level of the pile;

b) the application of the high cyclic surcharge pressure is in phase with the high repeated load level of the pile, and

c) the application of the high cyclic surcharge pressure is in phase with the low repeated load level of the pile.

(iv) Different combinations of the previous states of loading.

(B) The influence of the previous states of loading on the load displacement response of the pile.

(C) The influence of the previous states of loading on the load transfer characteristics of the pile.

To carry out such a research programme there are three main approaches:-

(A) Theoretical approach.

(B) Full-scale field tests.

(C) Laboratory-scale model tests.

Unfortunately, soil is a very intricate medium. It is neither homogeneous nor isotropic, "it is inherently a particulate system" as Lambe and Whitman (1969) said when they differentiated between solid, fluid and soil mechanics. Therefore, theoretical approaches which treat the pile-soil system as a rigid plastic, perfectly elastic or elastic up to failure and then yielded as a plastic material are unlikely to lead to a proper solution. Although the second approach is the more reliable for quantitative results it has the following limitations:-

(i) Time of conducting the previous research programme.

(ii) Difficulties in conducting such a research programme.

(iii) The cost.

Therefore the only approach which can be adopted is the model one. Model tests, in general, involve certain limitations. These limitations are mostly due to the similarity between the state of stress and strain in the model and that in the prototype, Rocha (1957), Roscoe and Poorooschasb (1963) De Beer (1963, 1965). Hence the results of model pile tests will be of quantitative value only if the corresponding elements of soil in the prototype and the model are subjected to identical strains. However, there exists three types of model testing:-

(i) Free-stressed sand surface.

(ii) Pre-stressed sand surface.

(iii) Sand subjected to centrifuge forces.

Concerning the first type of testing, this may be assumed as the worst case of modelling. Piles tested by these methods are usually subjected to a stress-level much lower than that of the prototype and hence neither the load nor the deformation of the model can be related to the prototype.

In the second type, the sand surface is acted upon by a surcharge pressure that keeps the sand in a state of stress similar to the average stresses of the prototype. Lack in similarity is very limited as compared with the first type.

Although the centrifuge is assumed to be the best testing technique it also suffers from certain defects such as:-

(i) Noises and vibration during the operation may affect both the sand density and the instrumentations.

(ii) The ratio between the length of the sample (the pile depth) to the average radius of the centrifuge may affect the results.

(iii) The sample actually is subjected to an acceleration equal to the resultant of two accelerations, the gravitational and the centrifuge. Therefore the model is not perfectly tested in a stress-path identical to that of the prototype.

(iv) The influence of the particle size of the soil. That is, when a model is subjected to an acceleration of N times the gravitational acceleration, the size of each particle in the model becomes in effect N times as large as the original size, which is equivalent to anticipating the behaviour of piles in sand from that in boulders.

(v) Any slight changes in the density of the sand would be magnified N times.

(vi) Due to unexpected changes or a sudden stop of the centrifuge motor, long-term tests such as those of the present investigation cannot be conducted.

(vii) The cost of the centrifuge per hour is relatively high as compared with the other types of test.

Therefore, it was decided to carry out the present investigation tests in a model of the pre-stressed sand surface. The topics have been studied experimentally on isolated model piles driven into a dry clean medium dense sand. The piles were instrumented by load-cells located along the inner surface of the shaft. The load transfer during any test was monitored and recorded by a data logger.

CHAPTER 2

REVIEW AND DISCUSSIONS OF SIGNIFICANT RELATED WORK

2.1 Introduction

In this chapter, the available literature is divided into four main parts. In the first part the influence of repeated loading on metals is considered. The second part deals with the behaviour of sand under static and repeated loading. The third part, the response of foundations other than piled to repeated loading is reviewed. The last part is concerned with the general behaviour of a single pile under both static and repeated loading.

2.2 Repeated Loading On Metals.

A repeated load is a force that is applied many times to a member, causing stress in the material that continually varies, usually through some definite range. The fatigue strength of a material is often used to indicate its strength in resisting repeated stress. Early in the study of strength of materials, it was found from experience and tests that members usually failed under repeated loads that were considerably smaller than similar static loads that were required to cause failure.

The mechanism of fatigue failure of a ductile member caused by repeated loads, and as described by Osgood (1970), is a gradual or progressive fracture. The fracture seems to start at some point in the member at which the stress is highest, usually at a point where the stress is concentrated or highly localised by the presence of a fillet, groove or hole, or some other abrupt discontinuity. As the load is repeated, a small crack may start and gradually spread until the member ruptures without measurable yielding of the member as a whole.

2.3 The Behaviour of Sand

The available information about the behaviour of sand will be reviewed under the following two main sections.

2.3.1. Under Static Loading

Roscoe (1970) demonstrated the following fundamental principles of soil:-

(i) Results of triaxial compression tests on soils tested under active and passive conditions indicated that the soil behaviour was strongly dependent on the loading path. (Fig. 2.1).

(ii) Tests on instrumented model retaining walls indicated that the behaviour of the soil was highly dependent on strain path.

(iii) Different granular materials of initially different voids-ratio tests in a simple shear apparatus results in the following observations:-

- (a) Once the rupture surface had been formed all the subsequent shear strains occurred within that surface.
- (b) The failure plane consisted of a thin band of not more than 10 grains thickness. Through that band the void ratio attained the critical state and beyond that it decreased in magnitude as shown in Fig. 2.2.
- (c) Even the densist sand packing was found to be compacted first before being dilated.

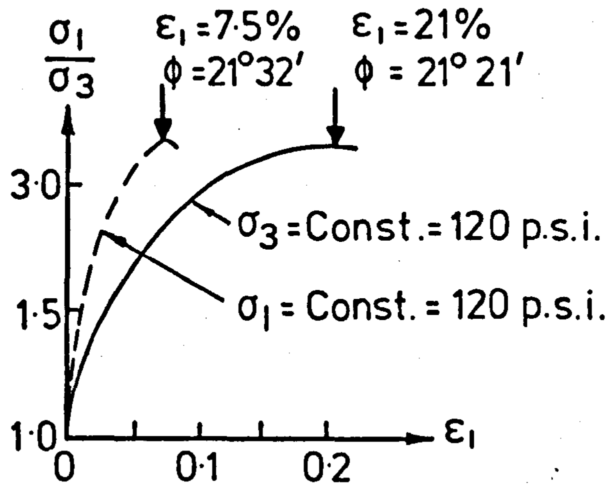


FIG. 2·1 INFLUENCE OF STRESS PATH ON STRAINS TO DEVELOPE PEAK STRESS RATIO OF NORMALLY CONSOLIDATED WEALD CLAY (AFTER ROSCOE 1970)

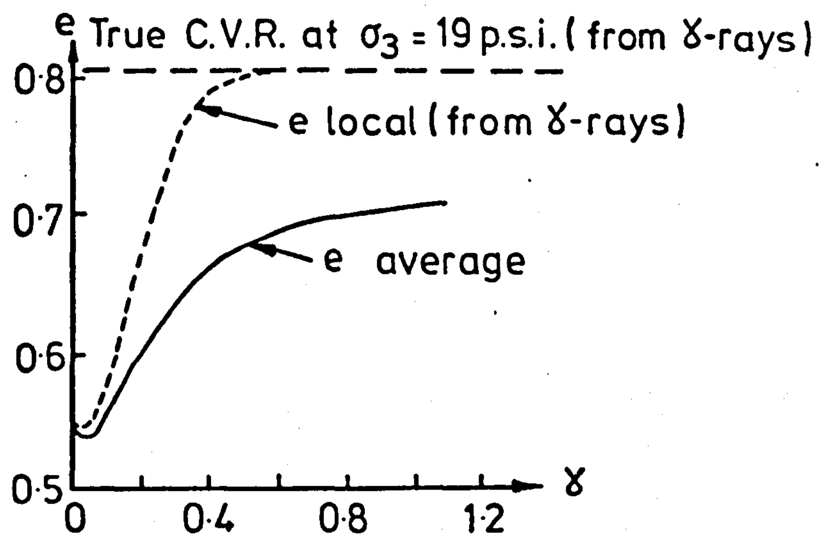


FIG. 2·2 VOIDS RATIO VERSUS AVERAGE SHEAR STRAIN (γ) MEASURED AT THE CENTRAL THIRD OF DENSE SAND TESTED IN SIMPLE SHEAR APPARATUS (AFTER ROSCOE 1970)

In an attempt to explain the phenomenon of the critical void ratio of sand, Youd (1970) suggested that in randomly packed systems both loose and dense arrays of particles are to be expected. Smaller shear strains are required to collapse loose arrays than to dilate dense arrays. After larger strains, the loose arrays of particles have for the most part collapsed and the denser arrays dilate. Dilation, in turn generates additional loose arrays which may eventually collapse. Thus, at some stage of strain, the effect of expansion and collapse of particle arrays counter balances each other and a constant volume state is attained.

Based on triaxial compression drained tests, Lee and Seed (1967) concluded that:-

(i) The state of critical void-ratio was a function of the confining pressure. Very dense sand may compress during shearing if the confining pressure is high enough.

(ii) The phenomenon of volumetric change ceased when the cell pressure was equal to a critical confining pressure.

(iii) There was an unique relationship between the critical void ratio and the critical confining pressure. Critical void ratio of loose sand was found to vary considerably with small changes in confining pressure. In contrast, small changes in void ratio of dense sand corresponded to large changes in critical confining pressure.

(iv) The phenomenon of dilation was dependent on the strength of the sand grains.

(v) The failure strength of the sand was not only dependent on friction and volumetric change, it was also dependent on the crushing and re-arrangement of the grains, Fig. 2.3.

An investigation on the angles of friction between sand and plane surfaces has been reported by Butterfield and Andrawes (1972). From that study the following conclusions can be drawn:-

(i) The static coefficient of friction is dependent on the relative movement between the solid and the sand, microscopic toughness and hardness of the solid material. The gradation, density and angularity of the grains and their packing are also effected i the coefficient.

(ii) The static coefficient of friction was always greater than the kinetic coefficient by a magnitude dependent on the material, the speed of sliding and the stress level.

(iii) A phenomenon of "stick-slip" associated with a non-steady deformation was noticed during the failure stage of the tests. This phenomenon was attributed to the difference between the static and kinetic friction, which was found to depend on the stiffness and damping of the loading system and the speed of testing.

2.3.2. Under Repeated Loading

The behaviour of sand under the influence of repeated loading is very complex as compared with that under static loading.

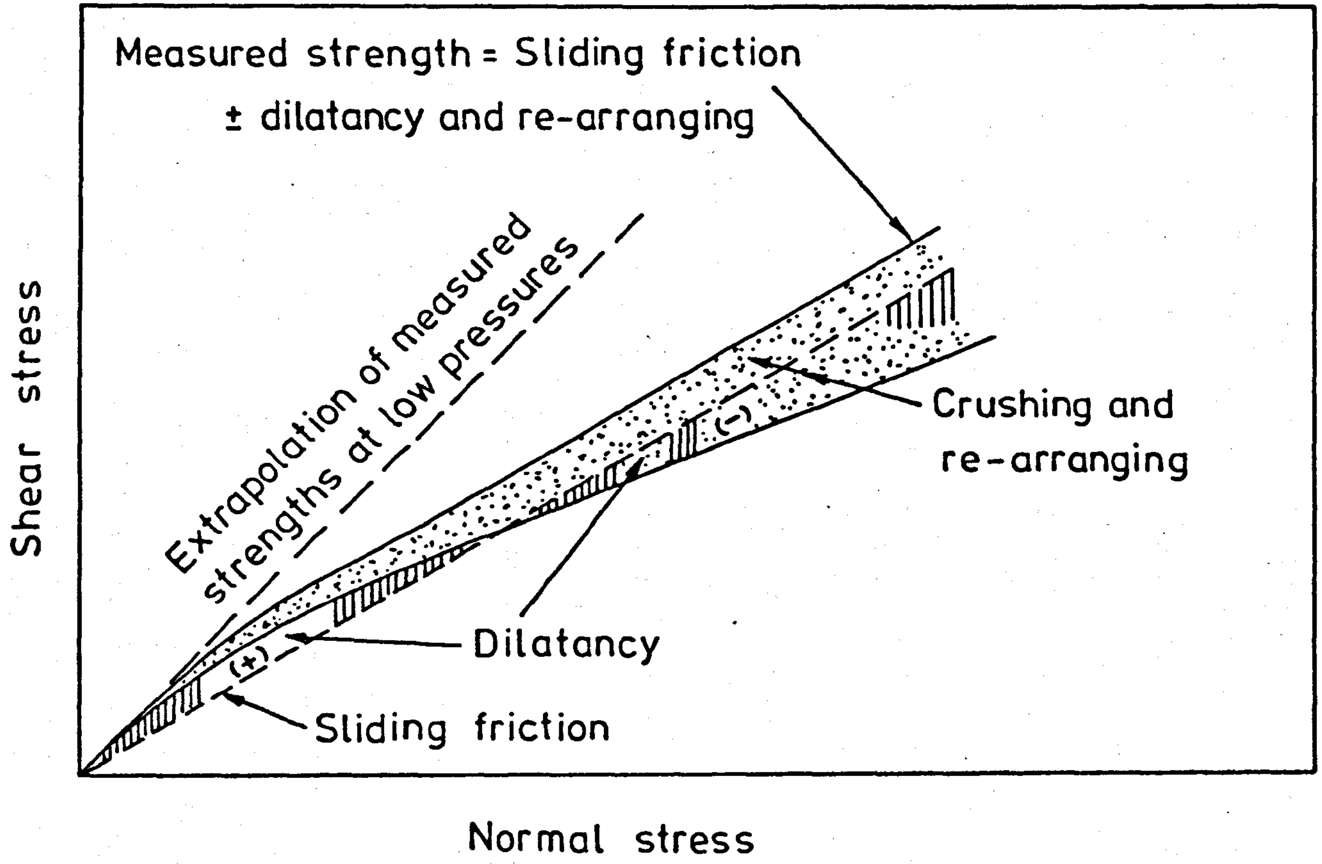


FIG. 2·3 SCHEMATIC ILLUSTRATION OF CONTRIBUTION OF SLIDING FRICTION, DILATANCY AND CRUSHING TO THE MEASURED MOHR ENVELOPE FOR DRAINED TESTS ON SAND (AFTER LEE AND SEED 1967)

Parameters such as the amplitude of stress, number of stress cycles, frequency and duration of the repeated stress, stress history, stress level and density of the sand are important.

From the results of cyclic loading tests on the surface of sand samples tested in a large oedometer, Al-Mosawe (1979) concluded that the compaction of the samples increased with the number of load cycles at a rapidly decreasing rate. Most of the compaction took place within the first few load cycles. Ko and Scott (1967) reached the same conclusion from a few loading cycles on samples tested in the triaxial apparatus. They concluded that most of the potentially unstable arrays of particles got a chance to be re-arranged into a more stable configuration within the early loading cycles. The investigation of Morgan (1966) on sand samples subjected to 2,000,000 cycles of deviator stress confirmed the conclusion of Al Mosawe.

The observations of Silver and See (1971) of the behaviour of sand under repeated loading in simple shear test indicated that the sand sample was compacted by an amount dependent on the number of strain cycles and the repeated shear strain level.

To explain the phenomenon of shear strength reduction associated with vibration of sand, Youd (1970) postulated that when a sand mass is subjected to stress fluctuated between two limites, the inter-particle contact stresses would also fluctuate. Repeating of the normal component of the contact stresses results in fluctuations in the contact area. Separation between particles due to the shearing stress component along the contact surfaces may therefore be expected. Re-arrangement of the sand grains will continue until a stable state is reached. The void ratio of this stable state is equivalent to the critical void ratio. If the sand is acted upon by increasing static loading during this stable state, no volume change will be detected. The interlocking between particles will no longer contribute to the overall shearing strength of the sand. The influence of confining pressure on the previous reduction of shearing strength was also discussed by Youd. He argued that increasing the confining pressure induces an increase in the initial interparticle normal stresses which in turn increase the frictional resistance along the interparticle surface.

The reduction in shearing strength of saturated sand was extensively studied by many researchers such as Silver and Seed (1971a, 1971b), Seed et al (1977), Peck (1979) Nemat-Nasser and Shokooh (1979), Martin and Seed (1979), Blazquez et al (1980). The main conclusion was that during the application of repeated shear-strain the sand volume tends to decrease. Pore water pressure would consequently increase by an amount depending on the relative density, the drainage condition of the sand, as well as the number of cycles, frequency and amplitude of the repeated shear-strain. The effective stresses, therefore, decreases which leads to a reduction in the sand shearing strength.

2.4 Repeated Loadings On Foundations Other Than Piled

During its lifetime, a structural foundation may be subjected to loading which fluctuates between different levels. This fluctuation is mainly caused by the variation of the live-loading with time. Repeated loading might severely affect the stability of the structure depending on the amplitude, the level of the repeated loading, the stress-history and desnity of the soil as well as the number of loading cycles.

From a laboratory investigation Carr (1970) reported that the ultimate pulling resistance of a plate anchor increased after testing under repeated loading. In addition the movement of the anchor increased with increase in the number of loading cycles. The behaviour of plate anchors under repeated loading was studied extensively by Hanna et al (1978). The tests were conducted in dry sand of different over consolidation ratios. The main findings of that study were that:-

(i) The irrecoverable displacement was much greater than that developed during static loading tests.

(ii) The over-consolidation ratio of the sand was found to have no effect on the accumulation of the irrecoverable movement.

(iii) Even for the high level of repeated load which had been used and the large number of loading cycles applied no failure was observed.

(iv) The life-span of the anchor when subjected to alternating loading was shorter than that for repeated loading.

Based on the results of the tests performed on anchors, Al-Mosawe (1979) supported the finding of Hanna et al. (1978) and reported that:-

(i) The irrecoverable movement of the alternating loading, in contrast to repeated loading, increased at an increasing rate, leading to failure.

(ii) The ultimate load capacity of the anchor after a repeated loading test increased but it always decreased after an alternating loading test.

(iii) The performance of the anchor depended mainly on the type and magnitude of the previous loading.

(iv) When surcharge pressure was cycled, the anchor system was always stiffer than that of an anchor under static surcharge.

(v) Pre-stressing the anchor was always associated with a longer life-span.

The results of repeated loading tests on instrumented reinforcing steel strips embedded in dry medium dense sand have been reported by Al-Ashou (1981). He found that:-

(i) After an initial stable state, the strip within a relatively small number of load cycles, was pulled out.

(ii) The life-span of the reinforcing strip was found to be a function of the amplitude and level of the repeated loading.

(iii) Increase in surcharge pressure always increased the life-span of the reinforcing strip.

(iv) The ultimate pulling resistance decreased by as much as 35% after the strip was subjected to repeated loading.

(v) Even after $2X10^5$ cycles of surcharge, failure had not been reached when the strip had been subjected to a sustained loading as high as 0.78Qt. Qt being the static pull-out load.

(vi) Testing the strip under different combinations of repeated loading and cyclic surcharge indicated that the most severe loading condition occurred when the upper load level coincided with the lower surcharge pressure.

Prevost et al (1981) conducted tests on a gravity platform foundation bearing on normally consolidated silt. The tests were carried out in a centrifuge. The foundation was subjected to repeated, inclined and eccentric loading conditions (not more than 15 cycles). The results of those tests showed that the foundation accumulated permanent vertical, horizontal and tilting movements and that the accumulation of movement decreased as the number of cycles of loading increased.

2.5 The Behaviour Of Isolated Piles

2.5.1. Under Static Loading

Based on the results of compression tests carried out on H-bearing piles driven through sand and gravel, D'Appolonia and Romualdi (1963) suggested that, during loading, pile deformation will be resisted by shear-stresses mobilised along the pile/soil interface surface. These stresses increase with increase in the relative displacement between the pile and the soil and are a maximum near the ground surface. Initially the soil sticks to the pile surface and the relationship between pile top loading and load transfer, as represented by the average shear stress on the pile, is approximately linear. Once the loading reaches a certain level, the pile begins to slip along the soil and a shear plane starts from the ground surface and progressively extends towards the pile base. Any subsequent increment of loading will be transferred to the pile base in as much as the pile is slipping along its entire length. The failure load will then be reached when the base starts to punch into the soil and settle continuously.

The mechanics of load mobilisation have been comprehensively studied by Hanna (1969). He confirmed most of the previous study and postulated that the failure surface does not necessarily coincide with the pile/soil interface surface. Concerning the pile load transfer, Hanna stated the following fundamental concepts:-

(i) The shaft friction and base resistance do not act independently, they are interrelated and strain dependent.

(ii) Unloading friction is generated from the ground surface and propagated down to the pile base such as that which takes place during the pile loading process.

(iii) After unload, the pile will be subjected to a system of stresses (residual) distributed in such a manner that keep the pile in a state of equilibrium.

(iv) The behaviour of the pile in the subsequent stages of loading is, to a large extent, dependent on these residual stresses.

Field and model tests carried out by Vesic (1967) on instrumented piles embedded in sand of different relative densities indicated that both the unit ultimate point resistance and the average skin friction attain a limiting value beyond a certain depth of penetration. To explain this phenomenon Vesic suggested that when a pile is loaded, the sand beneath the base is compressed whilst that around the shaft tends to move downward. The latter action causes the original horizontal stress which acts at any given point along the pile shaft to become inclined by an angle dependent on the magnitude of the settlement and the depth of the investigated point. If the depth of the pile is great enough, then a condition of constant inclination after a certain critical depth will be attained. This critical depth was found to vary from 10 times the pile diameter for loose sand to 20 times the pile diameter for dense sand. Vesic then reached the following conclusion. There is arching created along the pile shaft such as that observed above a horizontal trap door in a silo. This arching affects the state of effective vertical and horizontal stresses around the shaft as shown in Fig. 2.4.

A later published paper by Vesic (1969) stated that "very little is known about the actual stress condition around piles and this problem will remain open".

The phenomena of arching and the limiting values of point and shaft stresses have been supported by many researchers such as Tan (1971), Ooi (1980), Touma and Reese (1974).

Cooke (1974) described the initial strain field around loaded piles as follows. The soil deforms in such a way that it consists of a series of concentric cylinders centred on the pile. Shear strains decrease with increasing distance from the pile surface.

Holmquist et al (1976), Parry and Swain (1977) attributed the postulation of Hanna (1969) that the failure surface and the pile/soil interface surface not being coincident to the reduction in the shear strength of the soil in the direct vicinity of the pile.

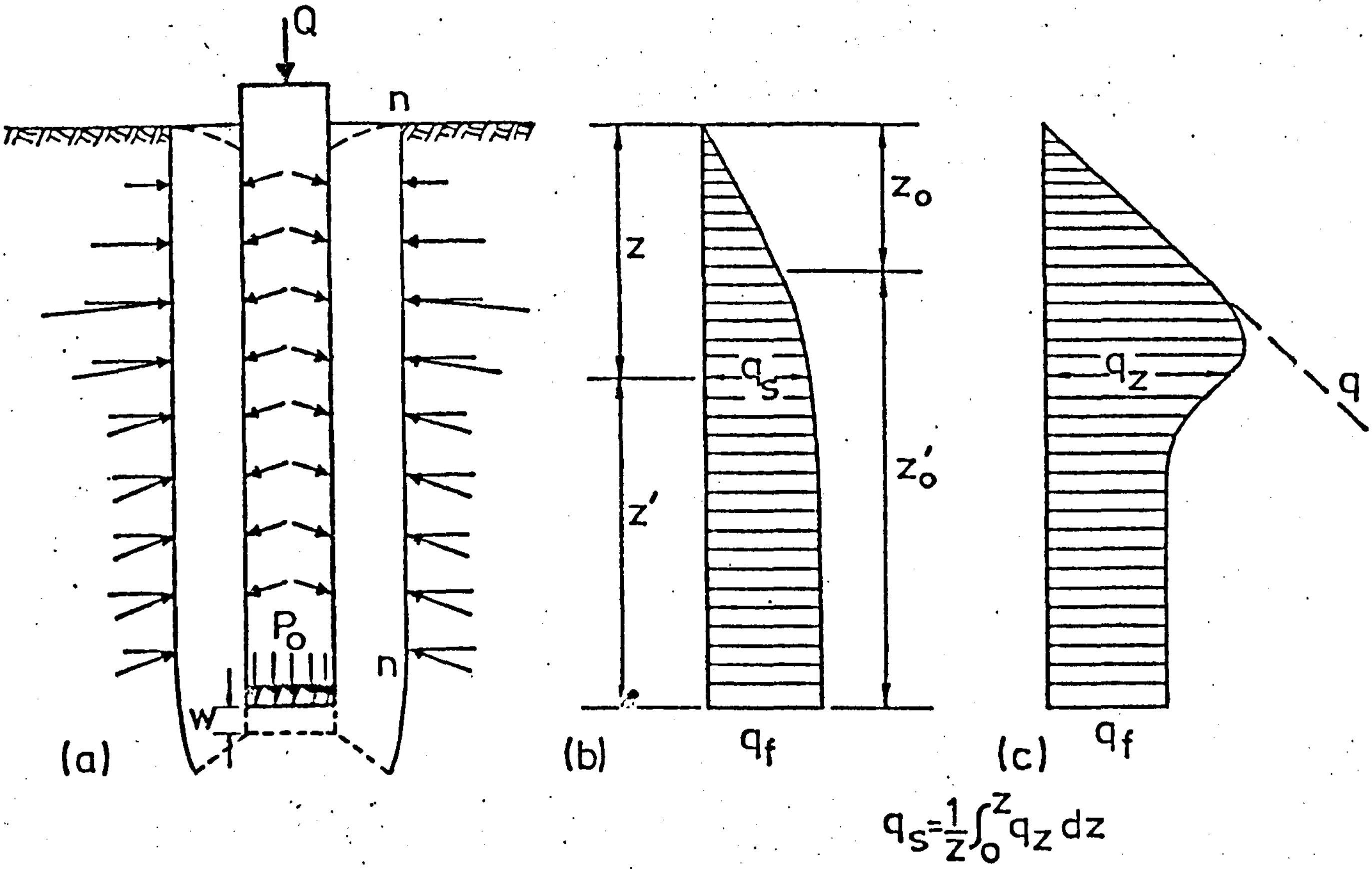


FIG.2-4 STRESS DISTRIBUTION AROUND A DEEP FOUNDATION IN SAND — (AFTER VESIC 1967)

Gallagher and St. John (1980) suggested that the coinciding of the two surfaces depend on the surface roughness of the pile. Thorburn and Buchanan (1979) restricted the limiting value of the skin friction to driven piles only. This conclusion was based on the following failure mechanism. During the process of pile driving, sand in the immediate vicinity to the pile dilates due to the large applied shear strains. In contrast, small shear-strains cause a compaction to the soils located a short distance from the pile shaft, thus creating a thin, looser annulus of sand along the shaft which affects the resistance and distribution of stresses along the pile shaft.

From field tension and compression tests carried out on instrumented steel piles of 0.457m diameter driven into boulder clay to 9.2m depth, Gallagher and St. John (1980) concluded that the pile shaft capacity in tension was less than that in compression. A similar conclusion was reported by Hunter and Davisson (1969).

2.5.1.1. Residual Stresses

These stressed are generated along the piles during and after the process of installation and have a magnitude and direction dependent on many factors such as the depth, method of installation and flexibility of the pile. The important influence of these residual stresses on the load-settlement behaviour of a pile has been quantified by Hanna and Tan (1971). They carried out a series of large scale laboratory pile tests. Two methods of test preparation were followed. In one, the pile was permitted to float whilst in the other method the top end of the pile was fixed and thus axial movement at the pile was prevented. In this manner two extreme initial residual stresses were created. The result of compressive

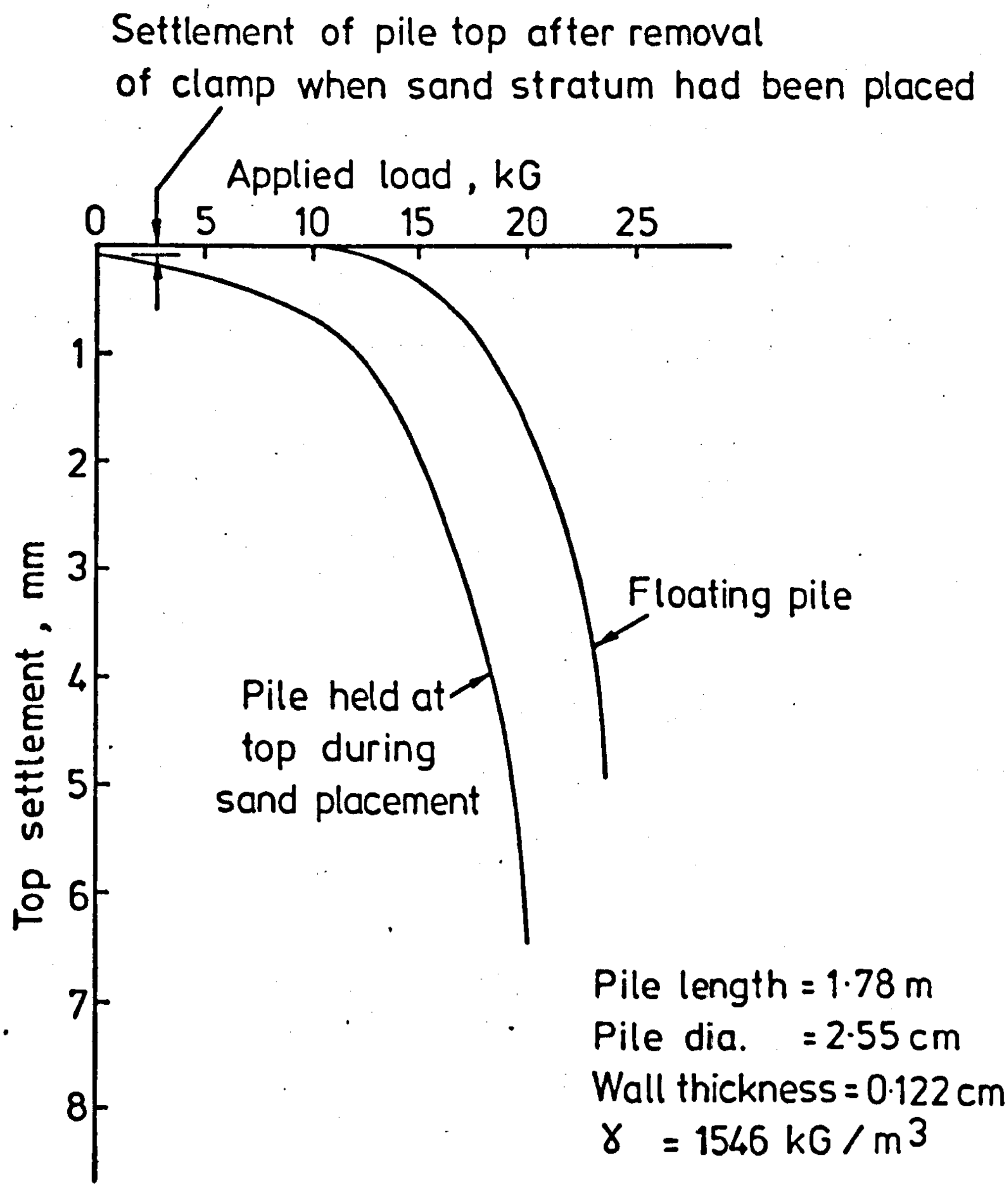


FIG. 2-5 LOAD - SETTLEMENT CURVES FOR COMPRESSION LOADING (AFTER HANNA AND TAN 1971)

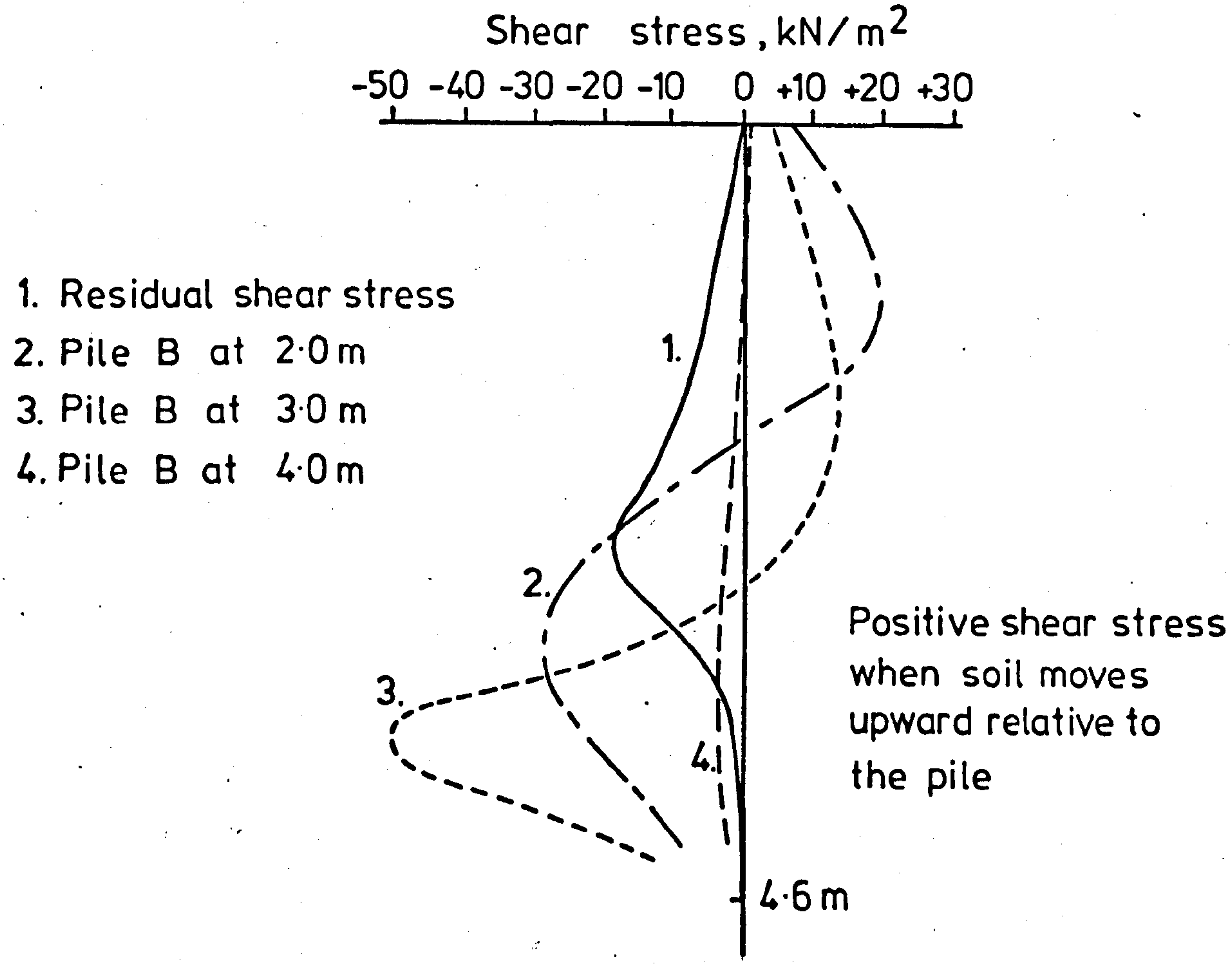


FIG. 2-6 CHANGES IN THE DISTRIBUTION OF SHEAR STRESSES ON THE SHAFT OF PILE A CAUSED BY SOIL MOVEMENT SET UP DURING THE INSTALLATION OF PILE B (AFTER COOKE ET AL 1979)

tests, Fig. 2.5. indicated that the residual stresses greatly affected the load carrying capacity and the settlement of the tested piles.

Hunter and Davisson (1969) concluded that the evaluation of load transfer based on zero residual loads may seriously be in error. Measurements of load distribution (Cooke et al 1979) of instrumented tubular steel piles embedded in London clay revealed that the state of residual stresses of a given pile was also affected by the installation of a nearby pile, Fig. 2.6.

2.5.2. Under Repeated Loadings

Repeated tension loading tests on H-piles in sand have been reported by Begemann (1973). Those tests revealed that the pile tended to pull out of the ground when a high alternating load was applied. For repeated loading less than 35 per cent of the static pull-out resistance, no significant increase in movement was observed. Begemann also highlighted the effect of load history on the subsequent pile performance.

An extensive test programme to study the behaviour of a single pile under repeated loading was conducted by Chan(1976). The pile and the apparatus of testing is similar to that employed in the present investigation except that Chan's tests were only performed on sand subjected to a constant surcharge pressure. The main findings of that work were:-

(i) The behaviour of the pile was found to be dependent mainly on the amplitude of the repeated loading and the number of loading cycles.

(ii) For compression repeated loading, initially the pile was stable, but after a certain number of cycles which increased with decrease of the load amplitude, the rate of movement began to increase rapidly until another stable stage was reached. Repeated loding was found to bring about a re-distribution of pile load between the shaft and the base.

(iii) For tension repeated loading, the pile initially moved with a decreasing rate then after a stable stage, the rate increased until eventually the pile was pulled out. The intial compressive residual axial loads changed to tensile during the early stages of the repeated loading. These tensile residual loads were very small at the final stage.

(iv) Failure due to repeated loading was attributed to a reduction of normal stresses that act upon the pile shaft.

(v) Repeated loading was found to decrease the ultimate load capacity of the pile. This observation has been confirmed by Bogard and Matlock (1979), Poulos (1981).

(vi) Increasing the surcharge pressure resulted in a pile of longer life-span when subjected to repeated loading.

Holmquist and Matlock (1976) conducted repeated loading (not more than 100 cycles) on 25mm instrumented aluminium tube piles tested in a drum 750mm diameter containing remoulded clay.

The results of those tests indicated that the alternating loading caused a severe reduction in skin friction as compared with that of unidirectional repeated loading. Increasing the confining pressure from 68kN/m^2 to 340 kN/m^2 corresponded to an increase in the life-span of the pile from 25 cycles to 75 cycles of repeated loading. When comparing the life-span of driven and bored piles they concluded that the latter had the longer life.

Although different techniques have been employed in testing, the studies of Madhloom (1978) and Ooi (1980) confirmed the finding of Chan (1976). Ooi indicated that the life-span of a pile decreased when it was subjected to failure loading before being tested under repeated loading.

Matlock (1979) concluded that the resistance along the upper portion of a pile subjected to tensile repeated loading was lost progressively

Bogard and Matlock (1979), after a cyclic loading test carried out on an axially loaded pile, suggested a mechanistic model of what may happen in a typical annulus of soil around a pile. Prior to repeated loading, the distribution of shear strength, shear stress and soil deformation are as shown in Fig. 2.7A. After repeated loading the gradient of shear strength of the soil in the vicinity of the pile may give rise to failure at a short distance from the pile wall. The band in which failure is initiated is shown hatched in Fig. 2.7B.

To solve the problem of a foundation pile subjected to repeated loading many theoretical approaches have been proposed. (For review see Smith 1979). Each of them attempted to produce a model which combines some of the phenomenon observed in element

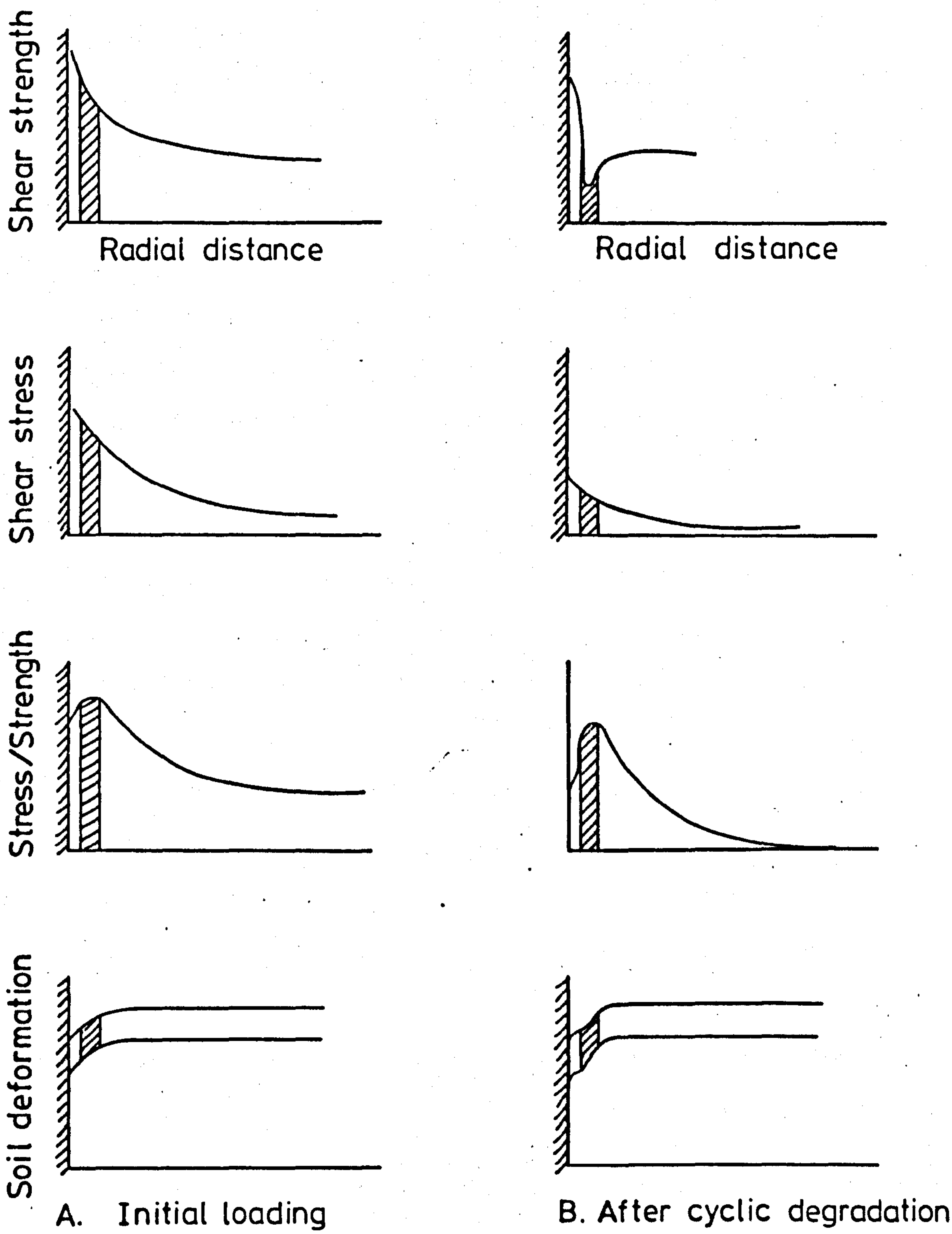


FIG. 2-7 MECHANISTIC INTERPRETATION OF CYCLIC DEGRADATION PROCESS IN AXIAL PILE-SOIL INTERACTION (AFTER BOGARD AND MATLOCK 1979)

behaviour such as a reduction in shearing strength. Most of these solutions assumed the pile and the soil as an elastic continuum and the problem was then solved either by the solid or by the transfer function approach. Gallagher and St. John (1980) did not give credibility to any of the previous design approaches unless it was used with an unrealistically high factor of safety.

Theoretical investigations by Poulos (1981) indicated that the cyclic failure begins at the top of the pile and progresses downwards as the number of load cycles and the level of the repeated load are increased. This failure caused the load transfer to increase gradually in the lower parts of the pile. Moreover, Poulos found that the increase in the soil shear strength results in a pile of longer life.

Recently, field investigation of Kraft et al (1981) on instrumented (356mm) open-end steel piles, embedded in clay and subjected to unidirectional repeated loading revealed that the load history and cyclic loading affected the load-deformation response of the pile. Concerning the load transfer of the pile these authors found that initially the load was transferred to the deeper sections but after several cycles of loading, the trend was reversed and the upper portion of the pile began to carry more of the load again.

CHAPTER 3

LABORATORY INVESTIGATION OF THE BEHAVIOUR OF A PILE ELEMENT EMBEDDED IN A TRIAXIAL SPECIMEN

3.1 Introduction

One of the main parameters studied in the test programme presented in this thesis is the pile shaft friction. The available information has indicated that this friction depends on the load level and history, surface roughness, pile material and rigidity as well as the soil properties and many other factors, Hanna (1963, 1969), Coyle and Sulaiman (1967), Vesic (1967), Touma and Reese (1974). Therefore to establish a better understanding of this friction, a pile shaft was tested in a modified triaxial cell. This was done in such a manner that the base of the pile was freely extended out of the sand sample and thus all the resistance to loading was taken by shaft friction. Two levels of confining pressure were used in these tests, 50 and 100 kN/m^2. These pressures were chosen to simulate the conditions of the main research programme. Both tension and compression piles were investigated. The properties of both the pile and the sand that were used in this investigation are the same as those used in the main research programme.

3.2 The Test Apparatus And Procedures

A large triaxial cell of 300mm diameter and 600mm high was employed. The apparatus, shown in Fig. 3.1., consist of a cylindrical sand sample of initially 200mm diameter and 497mm high supported by a perspex pedestal of 200mm diameter and 50mm high. The pedestal, which is rigidly connected to the base of the triaxial cell, contains a hole of 25mm diameter located along its own axis and through which the 19mm diameter pile passed. The diameter of

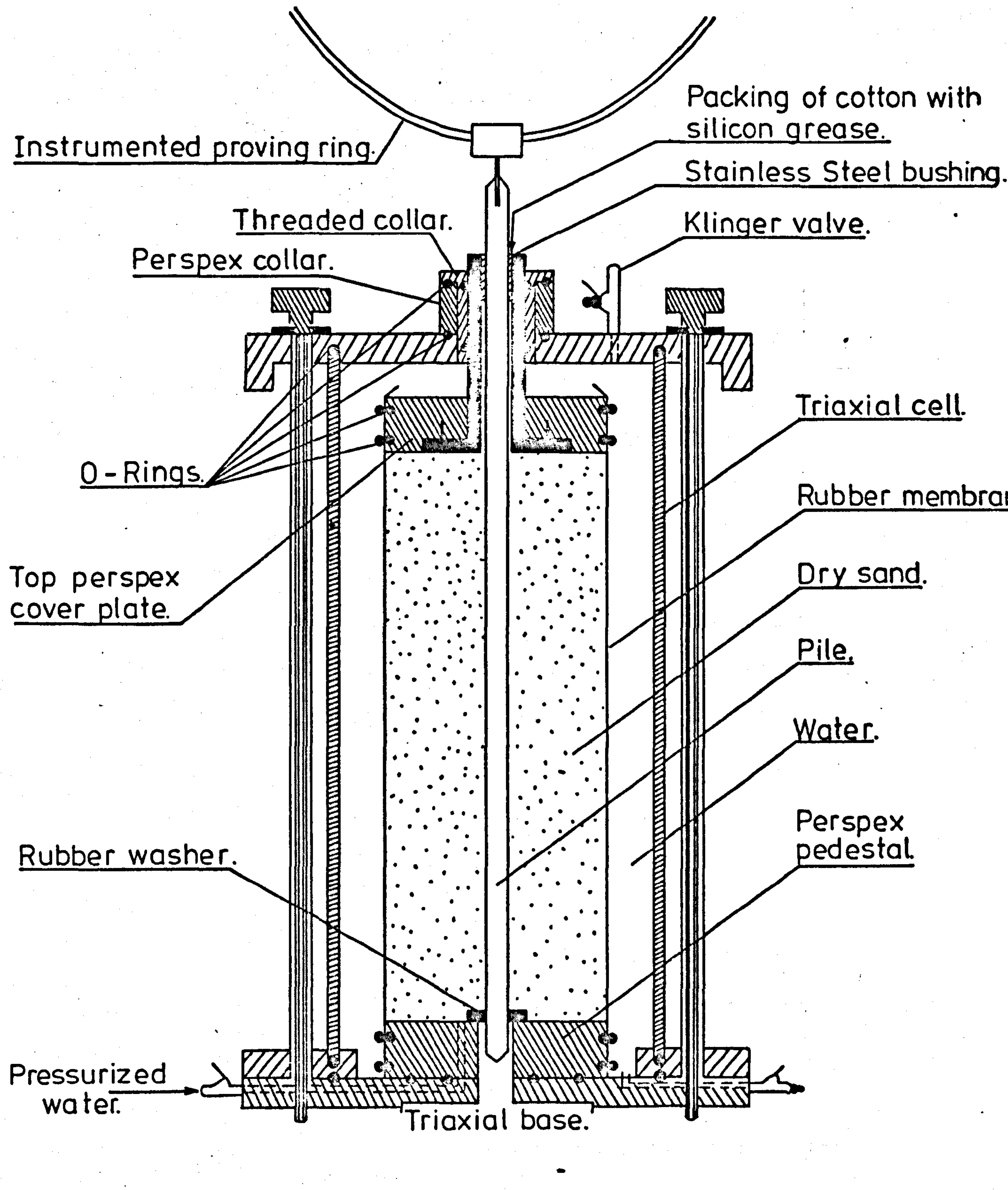


FIG. (3·1). LABORATORY MINIATURE PILE TEST APPARATUS.

this hole was larger than that of the pile and ensured that in no case would the pile surface touch the pedestal.

In order to fix the bottom of the pile in its proper position during installation and testing and to prevent the sand grains from falling through the gap between the pile and the pedestal, a rubber washer of 10mm thickness was stuck on the pedestal so that the centre of the washer hole coincided with the centre of the pedestal hole. A stainless-steel bushing 25mm inner diameter was rigidly connected to the top cover plate. The outer diameter of the bushing, 38mm was tapered at the top segment so that during erection of the apparatus the bushing could easily and smoothly pass through the cell cover plate.

The "model" pile was a tube of high strength aluminium alloy 19mm outer diamter, 750mm in length, and a wall thickness of 1.6mm.

The soil was uniform, air-dried, medium dense sand. The properties of this sand are stated in section 5.1. During the sample preparation, the sand was placed by a raining technique to give an average density of 1.557 Mg/m^3 which is equivalent to 51.2% relative density.

Before the first test was started many preliminary tests were carried out to check the repeatability of the test and the vertical displacement of the sand mass after applying the back pressure as well as that during testing of the pile. It was found that the tests do repeat to within approximately 5%. Regarding the vertical displacement, this was found to vary from approximately 8.0mm to 20.0mm. Therefore an average of 14.0mm was considered when preparing the samples to give a net depth of pile embedment equal to 475mm which corresponded to a depth to diameter ratio of 25. The vertical displacement was detected by a dial gauge mounted on the top end of the bushing.

Before starting the test each individual component of the apparatus was cleaned. The groove of the cell base, the hole of the rubber washer and the inner surface of the threaded collar were coated with silicon grease.

During the process of sand placement the pile was held at the lower end by the rubber washer. The upper end which was held by four sets of thin plastic bands was adjusted to set the pile vertical and hence coincide with the centre of the sand sample. The plastic bands bridge the pile to four free-standing rods which were firmly connected to the cell base and forming a cross-shape.

After the mould was filled with sand to a depth of 497mm and the surface of the sand was levelled off, the top cover plate with extreme care was placed.

In order to apply a back pressure (not more than 40 kN/m^2) the clearance between the pile surface and the inner surface of the bushing was gently packed by layers of cotton and silicon grease, Fig. 3.3. The upper part of the triaxial cell, which was suspended vertically from a crane, was lowered very slowly and carefully guided so that the bushing could smoothly pass through the collar outside the cell.

When the two parts of the triaxial cell were lumped tightly together the cell as a whole was carefully lifted up and positioned on the base of the triaxial machine. Water under low pressure (not more than 5 kN/m^2) was allowed to enter and fill the chamber. The back pressure was then cut off and the cell pressure was raised steadily to the required testing pressure. A load test was conducted using the arrangement shown in Fig. 3.4.

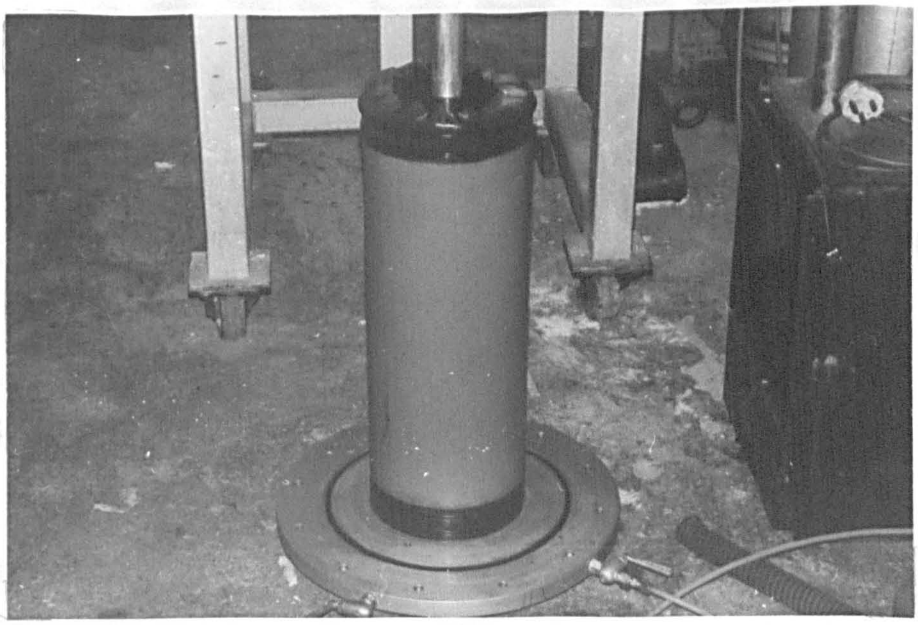


FIG. 3.2. THE SAND SAMPLE AFTER APPLYING BACK PRESSURE AND REMOVING THE MOULD.

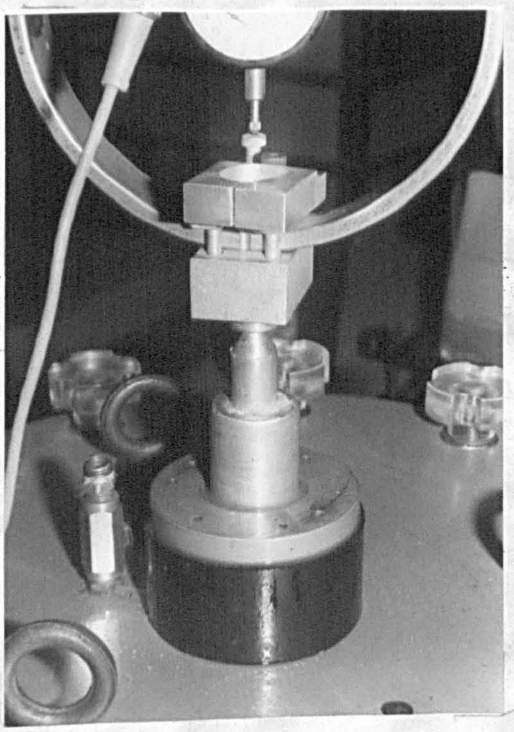


FIG. 3.3. PROCEDURE OF LOADING.

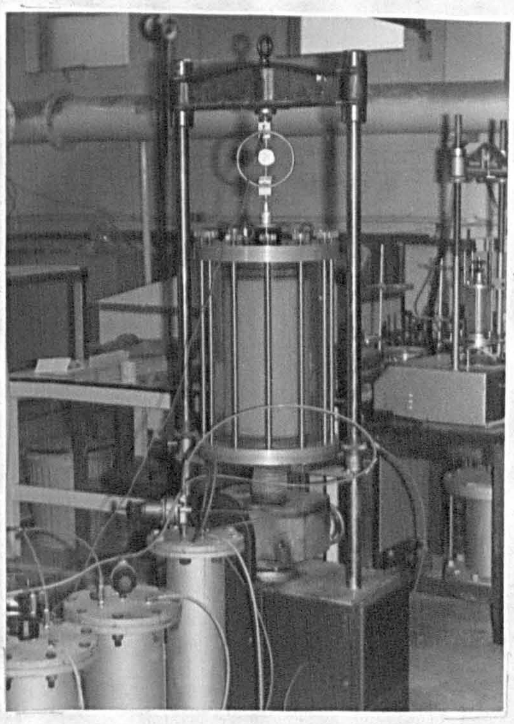


FIG. 3.4 GENERAL VIEW OF THE APPARATUS.

Load readings were taken each 30 seconds from the instrumented proving ring which was monitored by a Solartron data logger. The displacements were checked by dial gauge. The rate of displacement was 1.36mm per minute. Confining pressures of 50 and 100 kN/m^2 were used in this investigation. During the compression phase of the study the load was transmitted through a 25mm steel ball mounted on the top of the pile. For the tension phase, a threaded metal bar was used for this function, Fig. 3.3. Tension loadings were generated by reversing the direction of movement of the triaxial machine, after the triaxial base was rigidly connected to the base of the machine.

3.3. Test Programme

The test programme comprised nineteen tests performed on two sand samples. The first five tests which were carried out on the first sample, studied the shaft friction in compression loading. On the second sample fourteen tests have been conducted through which the shaft friction in tension loading was examined. The detailed test programme and sequence of tests are shown in table 3.1.

3.4 Presentation And Discussion Of Results

Fig. 3.5 shows a plot of load versus pile movement (average skin friction versus pile movement) of the tests that were carried out on the first sample. For all tests, at relatively small displacements the skin friction increased rapidly until a peak value was attained. The peak skin friction of the test T1, which was conducted under 100 kN/m^2 confining pressure, reached 40 kN/m^2. Beyond that the friction continuously decreased until it reached a value of 34.4 kN/m^2. These values of skin friction correspond to an angle of friction between the pile material and the sand equal to 22^o and 19^o respectively. Moreover, the peak skin friction was found

Table 3.1

Test Programme

Sample No.	Test No.	Type of loading	Cell Pressure kN/m^2	Remarks
1	T1	Compression	100	
	T2	"	50	
	T3	"	100	
	T4	"	"	
	T5	"	50	
2	T6	Tension	50	
	T7	"	"	
	T8	"	"	
	T9	"	"	
	T10	"	"	
	T11	"	"	
	T12	"	100	
	T13	"	"	
	T14	"	"	
	T15	"	"	
	T16	"	50	
	T17	"	100	
	T18	Compression	"	
	T19	Tension	"	

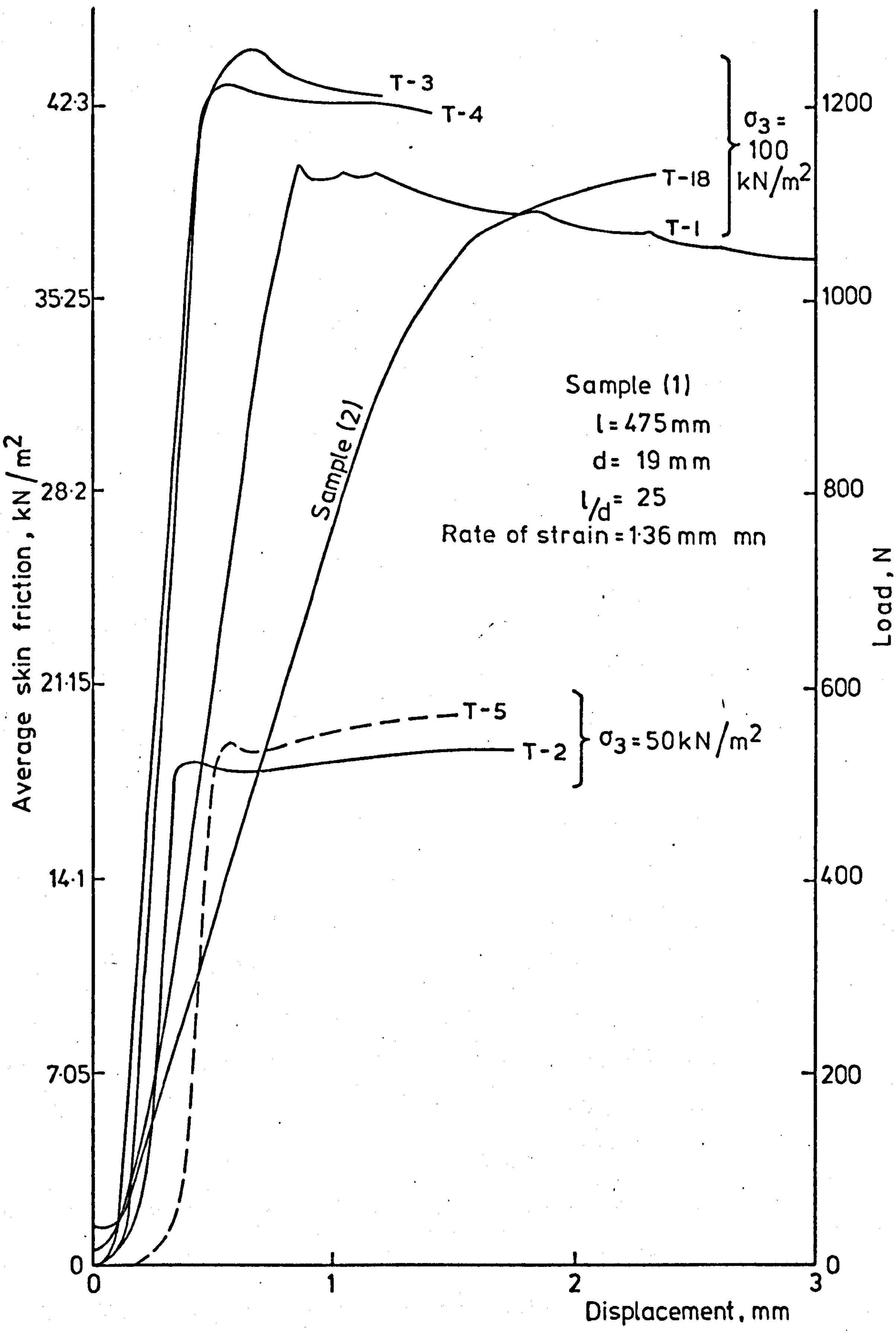


FIG. 3-5 LOAD Vs. DISPLACEMENT (COMPRESSION)

occur at a displacement of 0.858mm which is equivalent to 0.05 of the pile diameter or o.18% shear strain.

The angle of friction between a reinforcing strip and the sand as measured by Al-Ashou (1981) using a load control method and for a wide range of normal stresses, was 20.5°. This value may be assumed equal to the average of the two aforementioned angles (19° and 22°).

The test T2 was performed on the same sand sample but after the confining pressure was decreased from 100 kN/m^2 to 50 kN/m^2. The results of this test showed the peak skin friction was 18.34 kN/m^2 after which the skin friction dropped, then slightly increased as the pile displacement was increased. The peak value was reached when the pile moved 0.388mm which is equivalent to 0.02 of the pile diameter. The reason for this smaller displacement as compared with that of test T1 is believed to be due to the stiffening the sand had gained due to the previous test T1 where it was tested under 100 kN/m^2 confinement.

When the cell pressure was raised again to 100 kN/m^2 and test T3 conducted the peak skin friction reached a value equal to 44.3 kN/m^2 which is much higher than that of test T1. This value was attained when the pile head settled 0.655mm which corresponds to 0.034 of the pile diameter. The higher value of the peak skin friction as compared with that of test T1 may be related to the compaction of the sand sample which took place after each change in the confining pressures Ko and Scott (1967).

The peak skin friction decreased and the load displacement relationship became brittle when the test T3 was repeated as shown by test T4, Fig. 3.5. A decrease in the peak skin friction due to repeated loading has been reported by many authors, Chan (1976), Ooi (1980), Al-Ashou (1980) and Bogard and Matlock (1979). The first three authors indicated that the normal stresses which act on the shaft surface decreased due to repeated loading while the latter related it to the reduction in shear strength of the soil layer located very close to the pile shaft surface.

Again when the cell pressure was decreased to 50 kN/m^2 and the test T5 carried out, the peak skin friction was greater than that of test T2. This increases in the peak value may also be related to the compaction effect that took place after changing the cell pressure.

It may be argued that if an arch is generated in a horizontal plane it will affect the value of the horizontal effective stress which acts on the pile shaft surface and therefore, the value of the shaft friction will also be affected, Thorburn and Buchanan (1979) Vesic (1967). Thus, the value of the shaft friction due to an increase or decrease in cell pressure from and to 50 kN/m^2 respectively should not significantly be affected. From Fig. 3.5 it can be seen that the shaft friction of tests T1, T3 and T4 was always greater than twice that of tests T2 and T5 which were conducted under 50 kN/m^2 confinement. These results may indicate that the sand grains did not form an arch around the pile shaft. A similar conclusion may be drawn from the tension tests series as shown in Fig. 3.7.

At the start of each test, the load-displacement relationship was non-linear. This behaviour may be attributed to the state of residual stresses which are generated after each loading test as well as during the process of sand placement, Tan and Hanna (1974). The intial displacement may therefore cause a relative movement between the pile shaft and the soil, so that all stresses along the shaft become in one direction such as that described by Hanna (1969).

Coyle and Sulaiman (1967) carried out a similar investigation on 25.4mm steel stube, instrumented model pile placed in a poorly graded silty sand sample 150mm diameter and tested in a modified triaxial cell under a wide range of cell pressures. The results of two different initial densities tested under 70 kN/m^2 (10 psi) by the authors together with the result of test T1 of the present investigation are presented in Fig. 3.6. This plot shows the relationship between the average skin friction and the displacement ratio (displacement/diameter of the pile) of the pile. It will be seen that there is close agreement between the displacement ratio of the two investigations. With respect to the skin friction, the peak value of the denser sample was much closer to the respective value of the present investigation.

Fig. 3.7 shows the load displacement relationship of the other test series which was carried out on the second sand sample and designed to examine the shaft friction under tensile loading.

The general trend of this family of curves is similar to that of the first test series (Fig. 3.5). The test, T6 which was the first test in this series, was conducted under 50 kN/m^2 cell pressure. The peak skin friction of this test was about 14.1 kN/m^2 and attained at a displacement 0.021 of the pile diameter (0.13% shear strain). Beyond that the skin friction increased slightly as the pile displacement increased and reached 17.6 kN/m^2 at the end of the test.

If the value of the post failure skin friction of the compression test T1 (which was carried out on the first sand sample) and that of the tension test T6 (which was carried out on the second sample) are taken as a ratio of the applied confining pressure, the following values of this ratio are obtained, 0.35 and 0.352 respectively. This

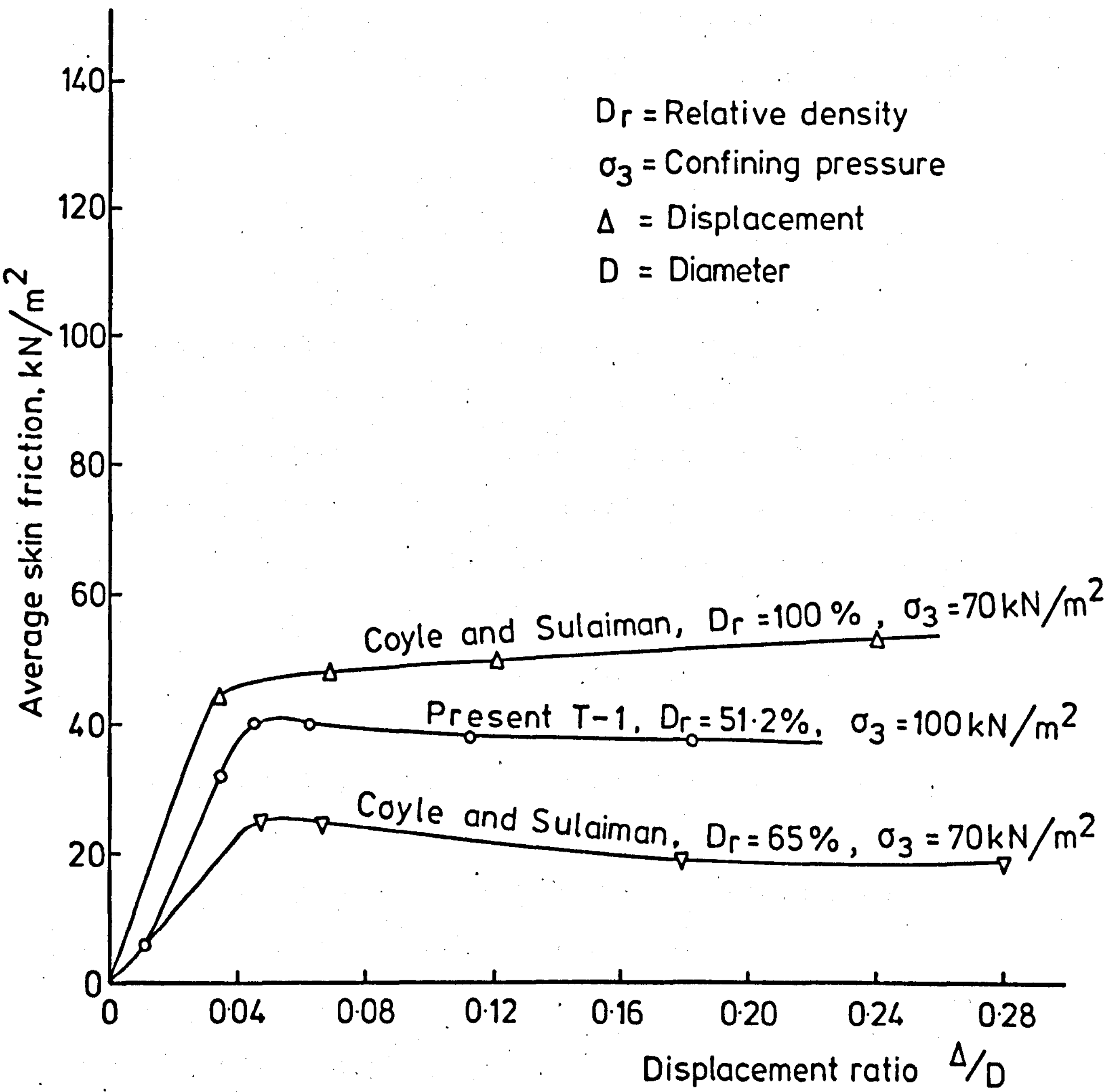


FIG. 3-6 STRESS - STRAIN RELATIONSHIP OF COMPRESSION TESTS

revealed that, at post failure and after a large pile displacement, the conventional coefficient of shaft friction in compression loading and that in tension was the same and was independent of the cell pressure.

When the test T6 was repeated six times, the peak skin friction was affected and the load displacement behaviour of any test became stiffer than the preceding test.

During the first loading cycle, test T7, the peak skin friction increased to 16.7 kN/m^2 and then continuously decreased after each loading cycle until it reached a value of 15.45 kN/m^2 during test T11. The peak skin friction of test T7 was higher than that of test T6 and is believed to be caused by the change in the direction and magnitude of the residual stress which resulted after the test T6 such as that demonstrated by Hanna (1969). A similar behaviour has been reported by Tan (1971) and Ooi (1980).

When the cell pressure was raised to 100 kN/m^2 and the test T12 performed the peak skin friction reached a value of 46.27 kN/m^2. This value also decreased when the test T12 repeated three times and reached a value of 39.32 kN/m^2 during test T15.

After the cell pressure was decreased to 50 kN/m^2 and test T16 conducted the peak skin friction was higher than all the values of the previous tests which had been carried out under 50 kN/m^2 confining pressure, as shown in Fig. 3.7.

The compression loading test T18 was carried out directly after the tension test T17. The load displacement characteristic of this test was more ductile as compared with the other compression tests, as shown in Fig. 3.5. Moreover, in contrast to the other tests which were conducted at 100 kN/m^2 the post failure skin friction of

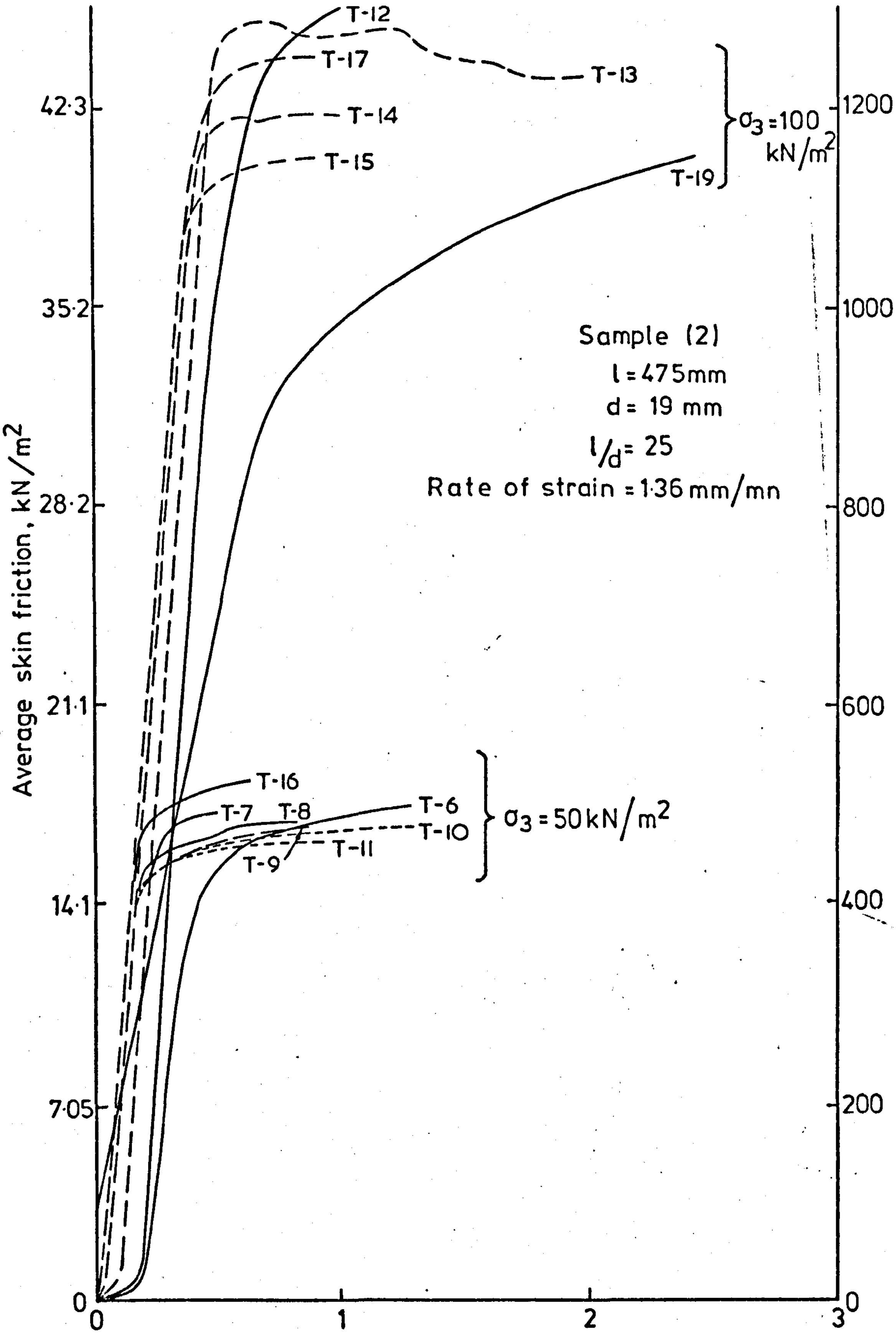


FIG. 3-7 LOAD Vs DISPLACEMENT (TENSION)

test T18 increased.

A similar trend was observed when the tension test T19 was performed directly after the compression test T18 - see Fig. 3.7. The change in behaviour of the tests T18 and T19 may also be attributed to the state of the residual stresses as reported by Tan (1971) and Hanna (1969). On examination of the mechanics of load mobilisation in friction pile, the latter author found that the pulling resistance of a pile which has not been subjected to a previous compression loading is larger than an identical pile but loaded in compression.

3.5 Conclusion

From these few tests and within the testing limits, the following conclusions may be drawn:-

(1) For small pile displacements, the skin friction increased linearly with increases of pile displacement.

(2) During the post failure stage and after a large pile displacement the conventional coefficient of shaft friction in compression loading and that in tension loading was the same and was independent of the cell pressure.

(3) The shear-strain required to attain the peak skin friction increased as the confining pressure increased.

(4) At a given confining pressure, the peak skin friction was increased when the confining pressure changed and, then, returned back to its original value.

(5) The peak skin friction decreased and the load displacement became more brittle when the loading test was repeated.

(6) Within these few tests and under such conditions, the influence of arching in a horizontal plane and around the pile shaft was not detected.

(7) Alternating loading greatly affected the behaviour of the pile shaft. The skin friction decreased and the displacement required to mobilise a given skin friction increased due to this type of loading.

CHAPTER 4

THE TEST APPARATUS AND MATERIALS

4.1 Arrangement Of The Apparatus

Three identical rigs have been employed in this investigation. The arrangement and detail section through one of these rigs are shown in Figs. 4.1, 4.2 and 4.3 respettively. The pile was placed in a cylindrical steel container filled with dry sand. A double hanger and lever arm system acting together with a reciprocating machine were used for repeated pile loading. In each minute during testing, the machine steadily lifted and then released the lower hanger which carried the amplitude of the repeated loading.

The mechanical advantage of the lever arm was 11.0 and 10.0 for compressive and tensile loads respectively.

The stress condition around the pile was controlled by applying a surcharge pressure on the sand surface.

The method of testing and the apparatus adopted in this study was similar to that of Madhloom (1978)except some modification regarding the cyclic surcharge system which was found necessary to fulfill the requirement of the test programme.

4.2 The Sand Container

The sand container was made of steel tube 380mm diameter, 965mm high and 17.5m thick. The diameter of the container was 20 times the diameter of the pile whilst the minimum depth of sand beneath the pile base after installation was 18.5 times the pile diameter. In order to prepare a suitable space for the pressure plate the sand was filled up to 920mm of the container height. The influence of any external vibrations through the ground was minimised by introducing a 12mm thick rubber sheet betwen the floor and the

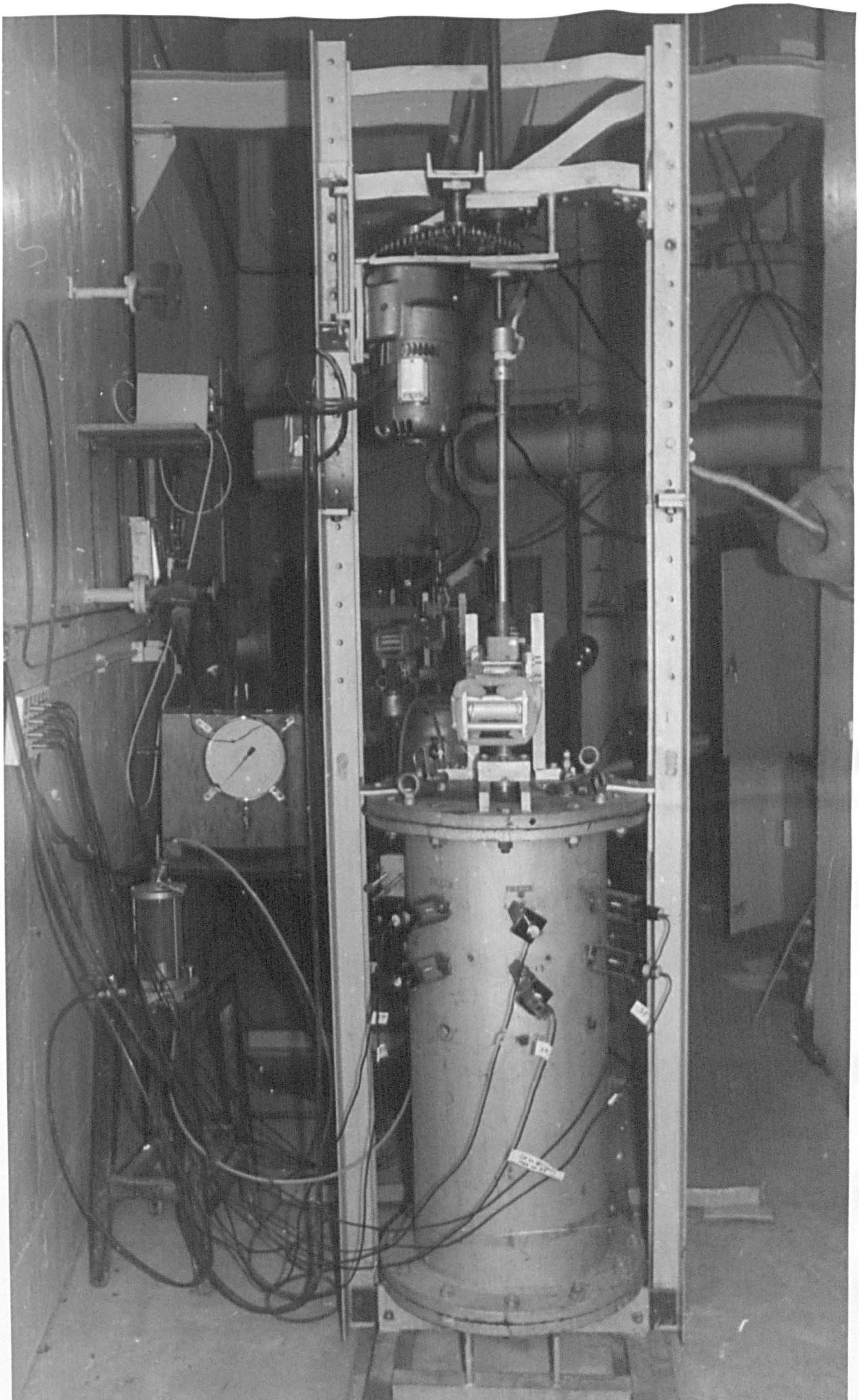


FIG. 4-1 THE TEST APPARATUS

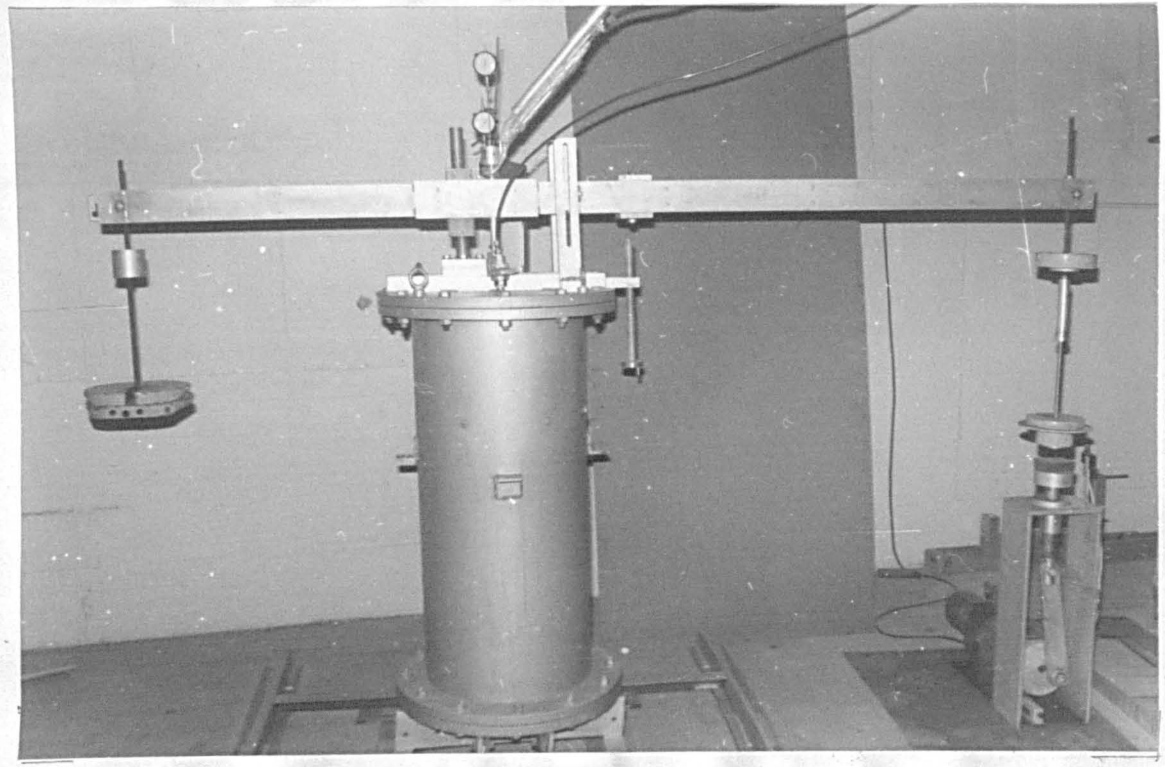


FIG. 4.2 OVERALL VIEW OF APPARATUS FOR REPEATED LOADING TESTS.

(After Chan 1976).

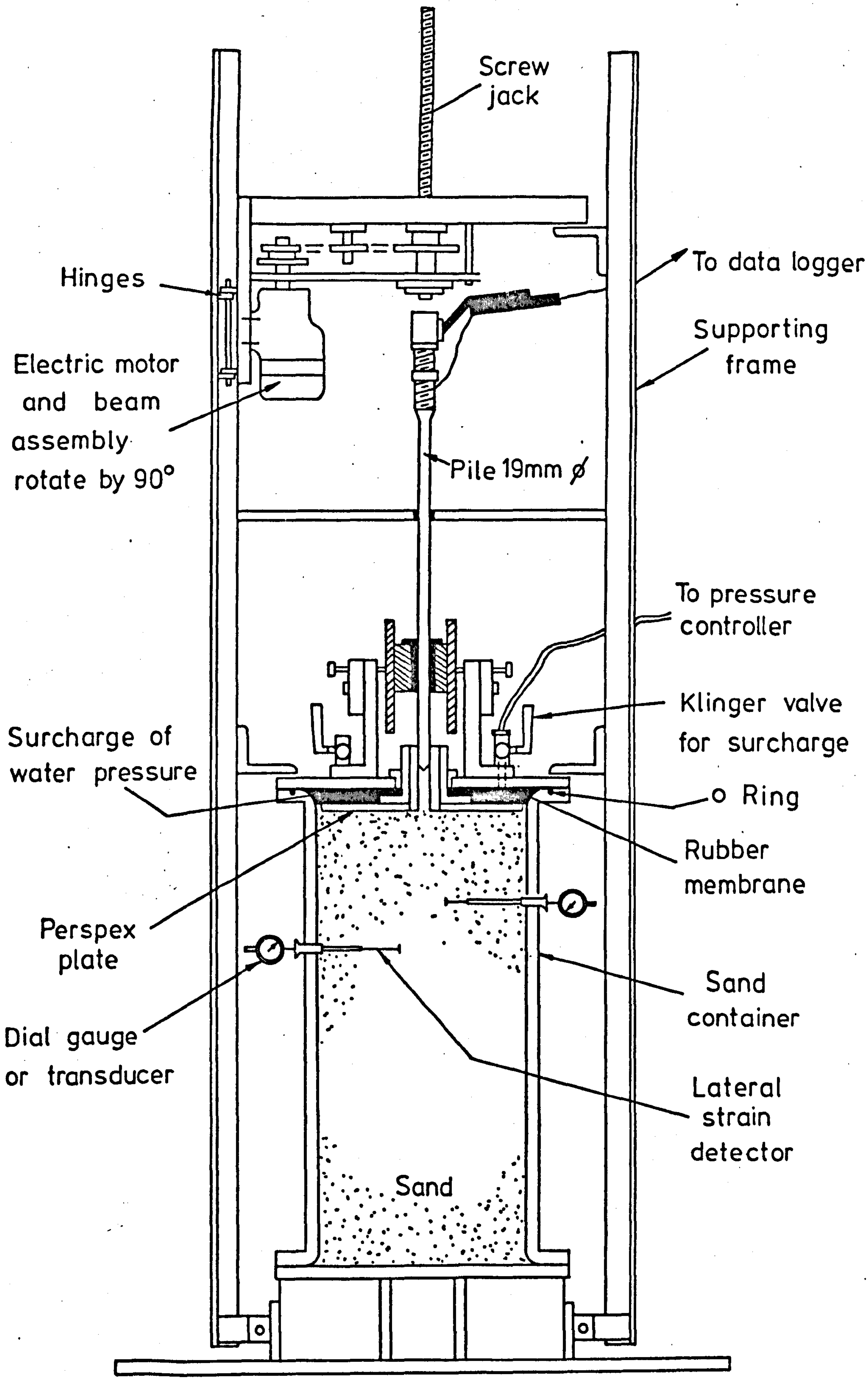


FIG. 4-3 SECTION THROUGH TEST RIG

container structure.

Preliminary tests indicated that the ultimate load capacity and the corresponding settlement of the pile were greatly affected and became non-repeatable when the container wall surface was not coated with silicone grease.

The influence of the container size on the state of stresses in the soil mass during the process of penetration was calculated by Chan (1976). He found that the major principal stresses decreased rapidly with increase of the radial distance from the shaft and the vertical distance from the base. These stresses became of negligible value near the boundary of the container.

4.3 The Sand Hopper

The sand hopper was made of 12mm thickness plywood with 40 x 40cm cross-section and 60cm depth. The outlet of the hopper was a segment of steel tube of 44mm outer diameter connected to a 50mm flexible hose. The other end of the hose was fitted with a stainless steel wire mesh of 3.3 x 3.3mm size through which the sand was allowed to flow. Hoses of two different lengths were used during the process of sand placement, one of them was 70cm length and the other was 40cm.

4.4 Arrangement Of Repeated Loading

The system by which the pile loading was repeated is shown in Fig 4.4. It consists of a lever arm, two hangers connected together through a ball bushing guide and a reciprocating electric machine. The frequency of the repeated loading was one cycle per minute for all tests except one in which the frequency was 3 cycles per minute. An adjustable press button relay was fixed to the skeleton of the reciprocating machine by which the machine could be stopped when the lever arm became inclined due to the

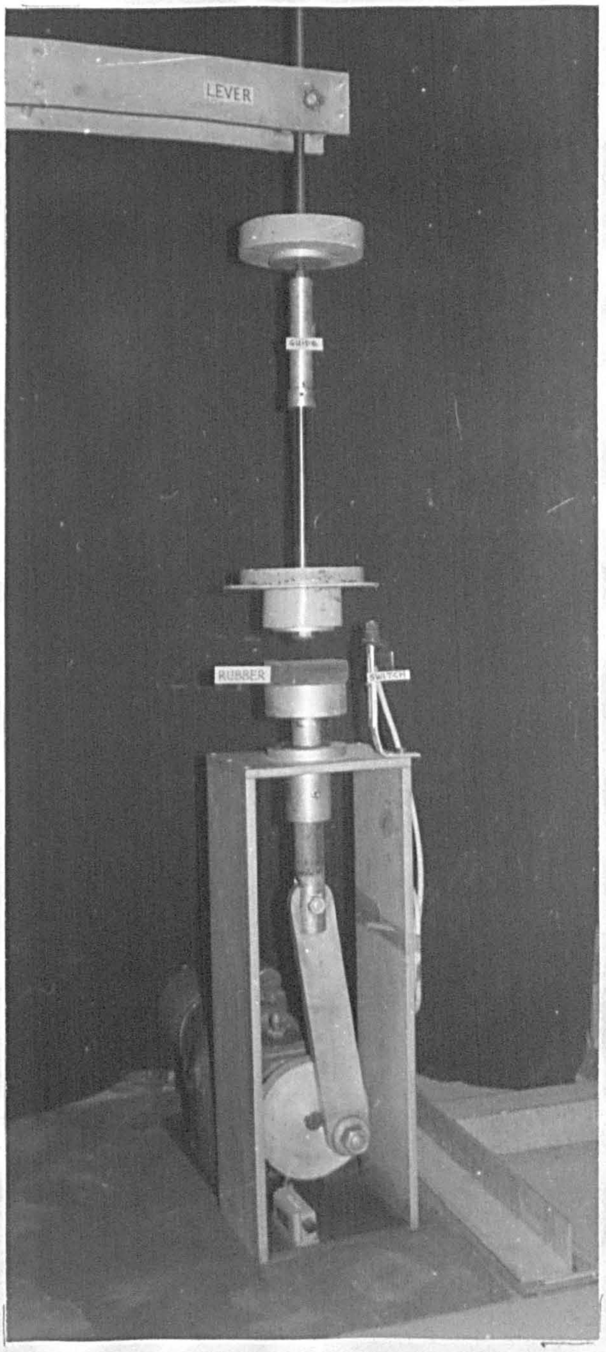


FIG. 4.4 ARRANGEMENT FOR APPLYING REPEATED LOADS.

(After Chan 1976).

displacement of the pile by more than a certain amount. Scanning of the top load cell of the pile during the 60 seconds duration of the loading cycle indicated that the pile loading was held constant for approximately 23 seconds while the transition period of increasing or decreasing the loading was about 7 seconds. This relative period of the pile loading was kept approximately constant in each cycle by re-levelling the lever arm as discussed in section 5.7.

The influence of impact on the pile during application of the repeated load was minimised by introducing rubber dampers at the surface of contact between the pile and the lever arm and at the top of the ram of the reciprocating machine. By use of these dampers the stiffness of the lever-hanger system was artificially decreased.

The problem of lateral movement of the lever arm due to eccentric load application was overcome by providing the bottom of the hanger with a small semi-spherical steel element Fig. 4.4 and by two frictionless roller guides located at the top of the container as shown in Fig. 4.5. These guides also prevented any undesirable lateral movement of the load lever. Measurement of the friction in the guide between the lower and the upper hangers indicated that this friction was negligible.

4.5 The Surcharge Pressure System

4.5.1. General

Three types of surcharge pressure have been used in this investigation.

(i) Constant surcharge pressure.

(ii) Surcharge pressure cycled between two limits

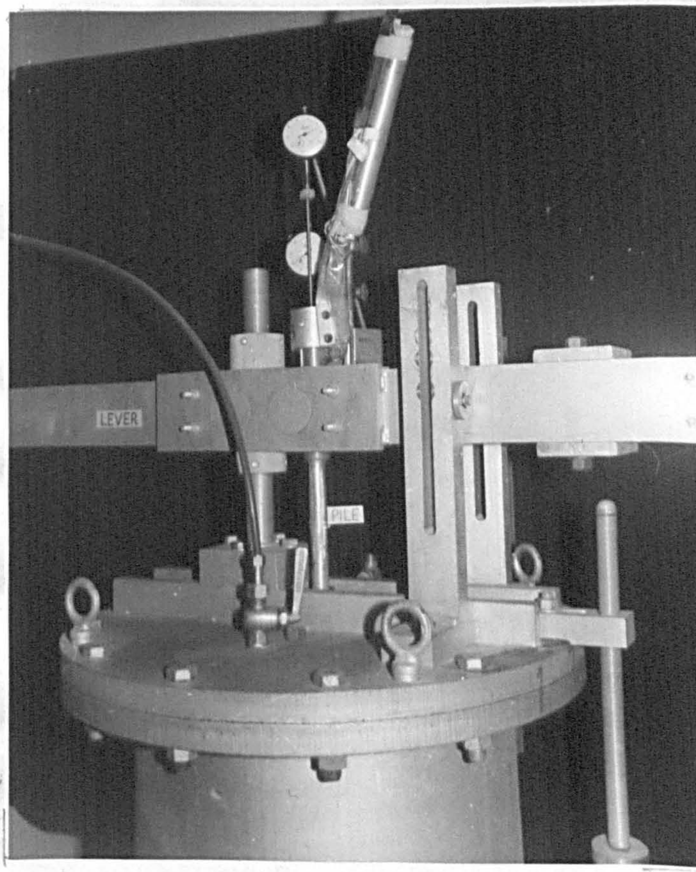


FIG. 4.5 ROLLERS FOR PREVENTING ACCIDENTAL LEVER MOVEMENT.

(After Chan 1976)

and independent of the pile repeated load level, and

(iii) the cycle of the surcharge pressure was conditioned by the pile loading level. This type consisted of two modes of testing:-

a) The high surcharge pressure was in phase with the high repeated load level of the pile.

b) The high surcharge pressure was in phase with the low repeated load level of the pile.

Since the available system which had been built by Madhloom (1978) could only fulfill the first two types of surcharge pressure the system was subjected by the writer to a simple modification through which the third type of surcharge could be applied. Fig. 4.6 shows the technique by which the surcharge pressure on the sand surface was controlled. The high pressure of the air which was supplied by the compressor was regulated through the pressure controller, then applied on the water surface in the "change-over pot". The pressurised water then, through the pressure plate, acted upon the sand surface as a surcharge.

4.5.2. The Pressure Controller

Schematic diagrams of the pressure controller, by which the magnitude and the time of application of the surcharge pressure were regulated are shown in Fig. 4.6. The pressure diagram, Fig. 4.6b consist of:-

(i) Two Norgren regulators, type R13 - 400 RNBM

(ii) Two Solenoid valves, type 454TA

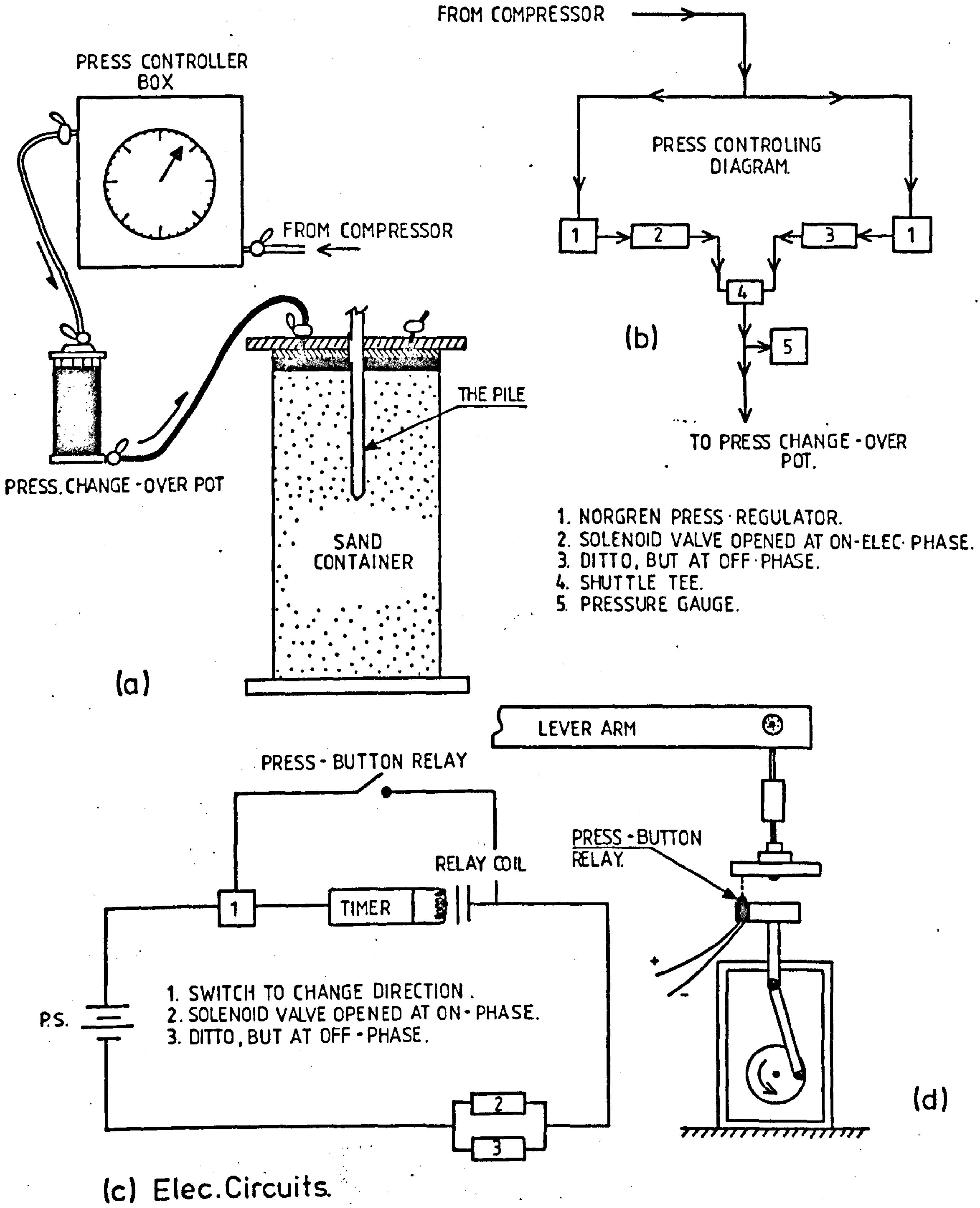


FIG. (4·6). SCHEMATIC DIAGRAM OF PRESS·CONTROLLER.

and 457TA. The first one is open when the electrical current is "on" whilst the other is open when the current is "off".

(iii) Shuttle T which was provided with two valves. When one is opened the other is shut.

According to this diagram the frequency of the cyclic surcharge is dependent on the frequency of the electrical current. From the electrical circuits diagram, Fig. 4.6c it will be seen that the current is controlled either by the timer through which the current is repeated regularly at a given frequency or by the press-button relay. This relay was attached to the head of the reciprocating ram as shown in Fig. 4.6d. Therefore, the electric current may be "on" or "off" depending on the contact between the ram and the bottom of the lower hanger.

4.5.3 The Magnitude Of The Surcharge Pressure

In reality this research programme is a part of a continuing programme to study the behaviour of piles under repeated loadings. In 1976 Chan published the results of the first part of that programme in which the sand surface was subjected to a constant over-burden pressure equal to 100 kN/m^2. Madhloom (1978) presented another set of results in which the surcharge was cycled between 50 to 100 kN/m^2. In the present investigation, therefore, the surcharge pressure was either constant at 100 kN/m^2 or cycled between 50 and 100 kN/m^2. Several additional tests were carried out under 0.0 kN/m^2 and 200 kN/m^2 to study the influence of the surcharge pressure on the behaviour of the pile.

4.6 The Test Piles.

4.6.1. General

Four model piles have been used in this investigation. They were made of high strength aluminium alloy tube of 19mm outer diameter and 15.8mm inner diameter. They were of three different lengths, the longer of 1,000mm, two of 700mm and the shorter pile of 600mm. Three depths of penetration were examined namely, 570, 380 and 285mm which corresponded to 30, 20 and 15 times the pile diameter respectively. The instrumentation and details of the piles are shown in Fig. 4.7 and Fig. 4.8.

In order to facilitate the gauging work, the shaft of each pile was divided into segments. These segments were connected together by means of threaded internal collar joints as shown in Fig.4.9. The pile base consisted of conical segment of 60 degrees apex angle, and was threaded onto the lowest pile segment. The pile head, which comprised a thick tubular section, was welded to the shaft. The two threaded collars, located near the pile head, were used to transfer the load from the lever arm to the pile head, Fig. 4.8a.

4.6.2. Measurement Of Pile Movements

The pile top and base movements were directly measured by dial gauges, Fig. 4.10. A "tell-tale" strain rod was used to detect the movement of the pile base. This rod was threaded to the pile base and extended out of the pile top. Through the pile body, the rod was separated from the strain gauge wires by a tube of relatively greater diameter.

4.6.3. Measurement Of Pile Forces

Measurements of the axial load distribution along the pile depth were made by strain gauges mounted on the inner surface of the pile

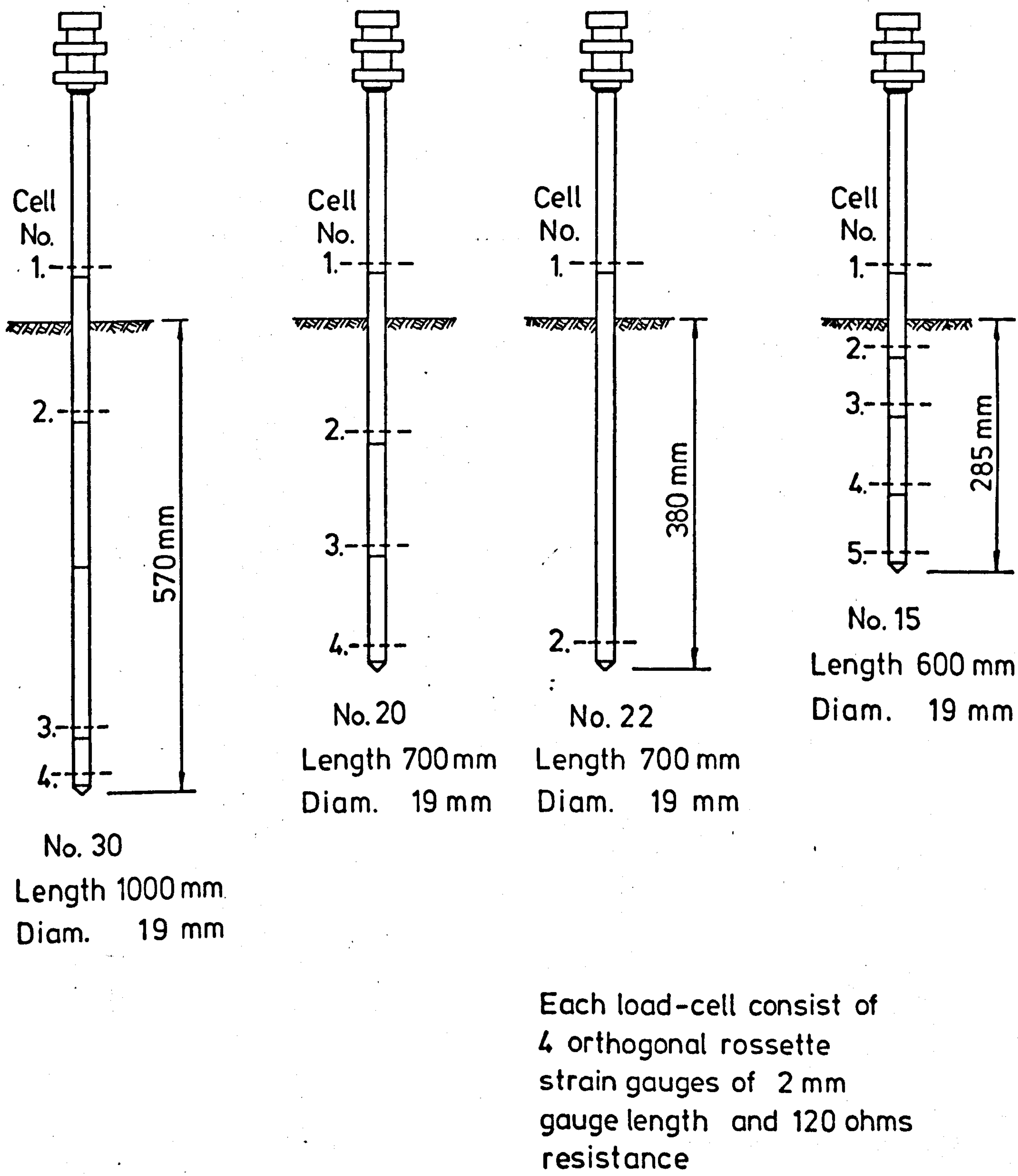


FIG. 4-7 MODEL PILES

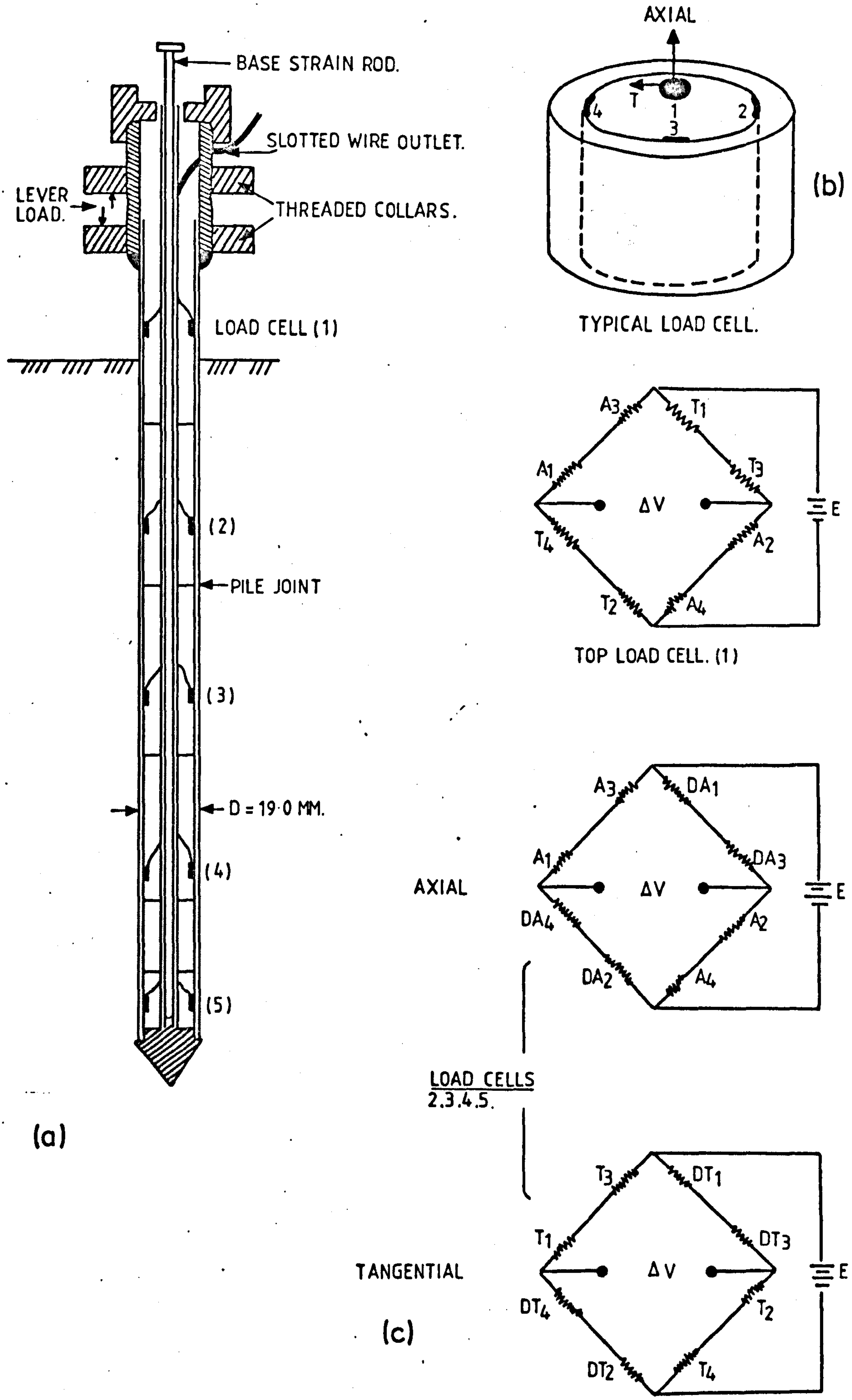


FIG.(4·8). TYPICAL STRAIN GAUGE CIRCUITS.

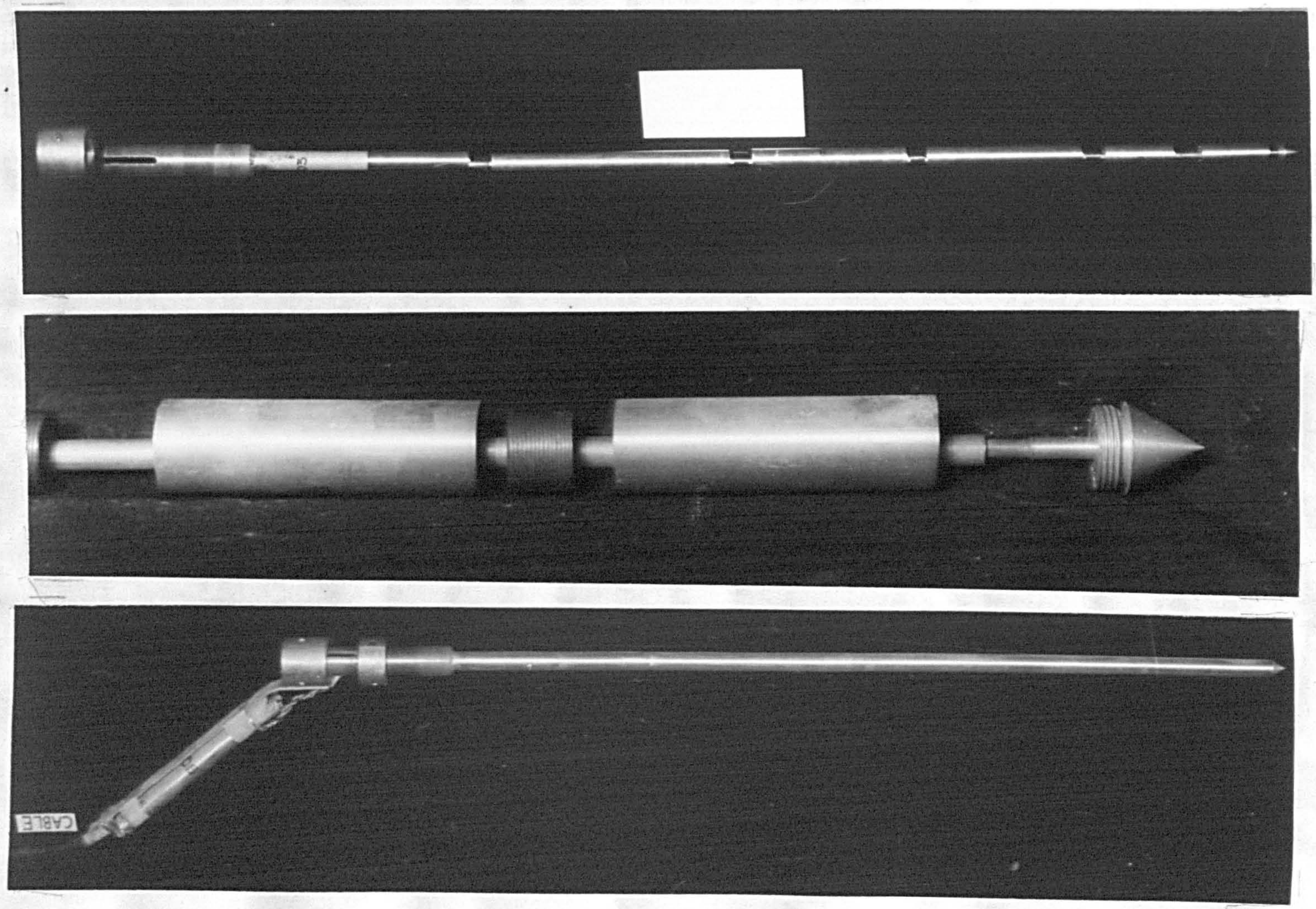


FIG. 4.9 MODEL PILE BEFORE AND AFTER ASSEMBLY

shaft. Therefore, each instrumented segment of the pile was actually acting as a load-cell. Each load-cell consisted of four tem prature compensated strain gauge rosettes. The individual rosette comprised two orthogonal foil strain gauges type FCA-2 manufactured by the Tokyo Sekki Kenkyujo Company. The length, the nominal resistance and the gauge factor of each strain gauge were 2mm, 120 ohms and 2.01 respectively. Except the top load cell which was always located above the sand surface, the axial and tangential strains at the position of the load cell were evaluated by two independent full bridges. These bridges were combined into a single bridge at the top load-cell of each pile Fig. 4.8.

The analysis by which the pile loading at any given depth, in terms of the axial and tangential strains, was calculated is presented in Appendix A. The influence of the lateral soil pressure was taken into account in this analysis.

To increase the sensitivity of the load cells, the inner shaft surface at the location of the strain gauges were machined to a thickness which varied from 0.635mm at the base cell to 0.89mm at the top cell location. The length of the machined part was designed so that the shaft was always safe from local buckling, and the positions of the strain gauges were always far enough from the zones at which the shaft wall thickness were changed. This length was found to be equal to the pile diameter.

4.6.4. Gauging Technique

Before mounting, the loads of each strain gauge were extended by soldering a sufficient length of 0.25mm wires in order to be longer than the length of the pile segment under consideration. The pile segments were then assembled and the outside position of each strain

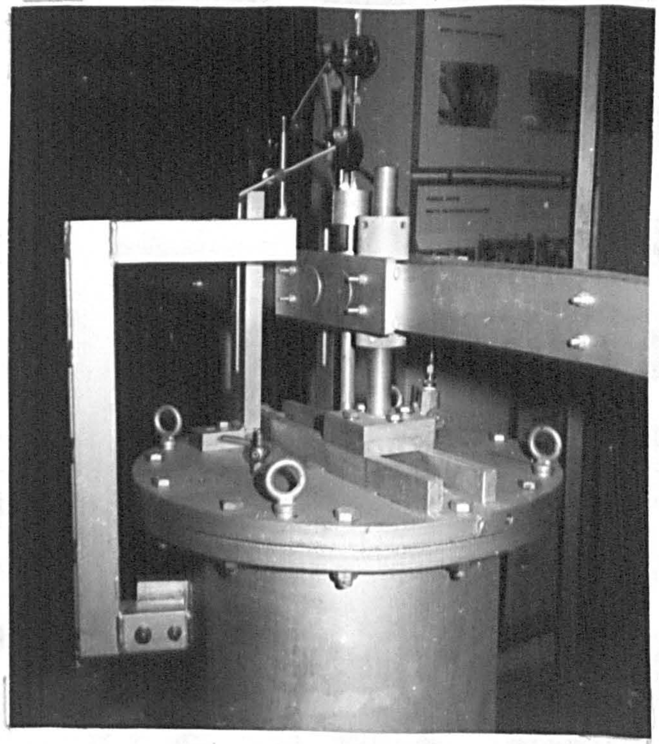


FIG. 4.10 MEASUREMENT OF PILE MOVEMENTS

(After Chan 1976)

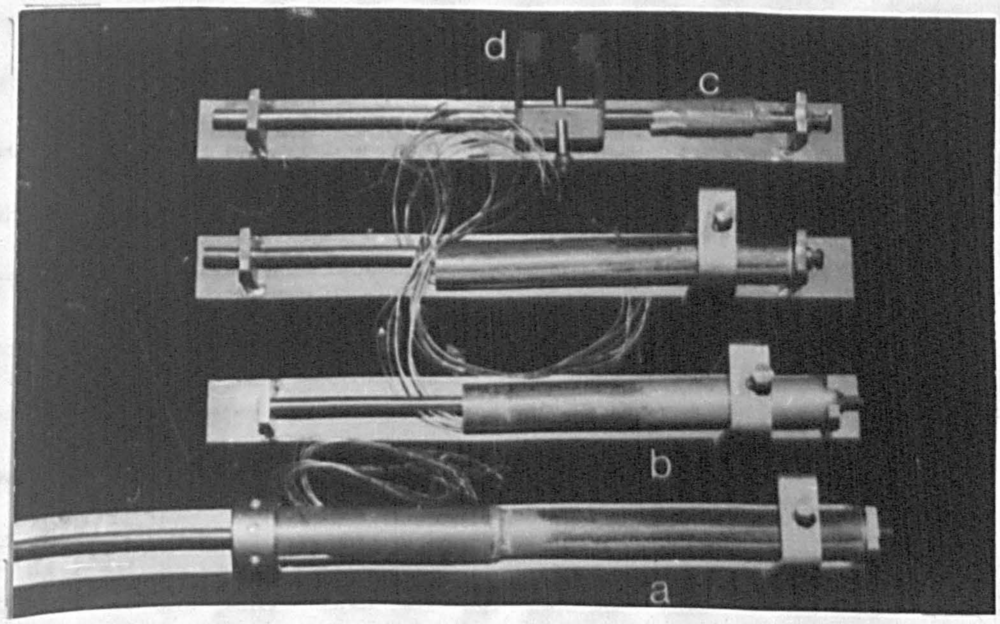


FIG. 4.11 STRAIN GAUGE MOUNTING DEVICE

a) Pile top section.

b) Intermediate pile section.

c) Holding down clamp.

d) Strain gauge mounting rubber pad.

(After Ooi 1980).

gauge was marked before dismantling for cleaning and gauging. A perspex plate on which the rosette axes have been marked was used to prepare the rosettes so that they became suitable for handling. Double side sticky tape was first placed on the plate, and after tracing the rosette axes a hole was then cut from the tape. The back of the rosette was then carefully positioned on that hole so that the axes on the tape coincided with those of the rosette. The leads were lined up along the tape length and coated with a layer of adhesive to insulate them. The rosette as well as the leads were covered by a layer of sticky tape. The latter layer also acted as an extra precaution to separate the wires from each other. The rosette, after cutting and shaping the holding tapes, became ready for mounting on the pile shaft surface. A strain gauge mounting device which consisted of a central rod on which a metal plate was stuck as shown in Fig. 4.11. This plate was a segment of a cylinder 25mm length and had an inner diameter equal to the diameter of the rod and an outer diameter equal to the inner diameter of the pile shaft. On the outer surface of this segment a rubber pad which had been marked with the axes of the rosette, was stuck. A part of this device was the sliding clamp which was used to apply the pressure on the rosette during the process of mounting. The method of mounting a strain gauge on the inner shaft surface was as follows. After covering the rubber pad with silicone paper, the rosette with the backing facing upwards was carefully aligned and positioned on the pad. With the central rod inside the pile segment and after applying the CN adhesive to the back of the rosette, the pile segment was slid into place and clamped down on to the rosette. A suitable pressure was applied by the sliding clamp to keep the rosette in contact with the pre-determined position of the inner shaft surface. The assembly was left two hours to cure before another rosette was mounted. To release the residual stresses which were generated during and after the rosette placement, each load-cell was subjected to many cycles of loading-unloading in a hydraulic testing machine. Through these cycles, the load response of the cell was checked. The stability of the load cell over a period of two weeks was also checked before the calibration test was carried out on that cell.

4.6.5. Calibration

The calibration of each load-cell was conducted by using dead weights as shown in Fig. 4.12. A typical result of a calibration is presented in Fig. 4.13. from which it can be seen that the response of the load cell was linear. Unloading readings were within 0.5% of those of the loading which was in full agreement with that quoted by Chan (1976). A repeated calibration test after the pile had been acted upon by a large number of loading cycles indicated that the calibration factor was not affected appreciably.

4.7 The Data Logger

The wires of the load cells were connected to the data logger through terminal boxes. These boxes were provided with an excitation voltage from a King Shill constant voltage power supply. Therefore all the output of the load-cells of the four piles can be recorded directly by the data logger. This Solartron Compact 33 data logger comprises four elements:-

(i) The analogue scanner.

(ii) The control unit. Through this unit the various mode of operation can be

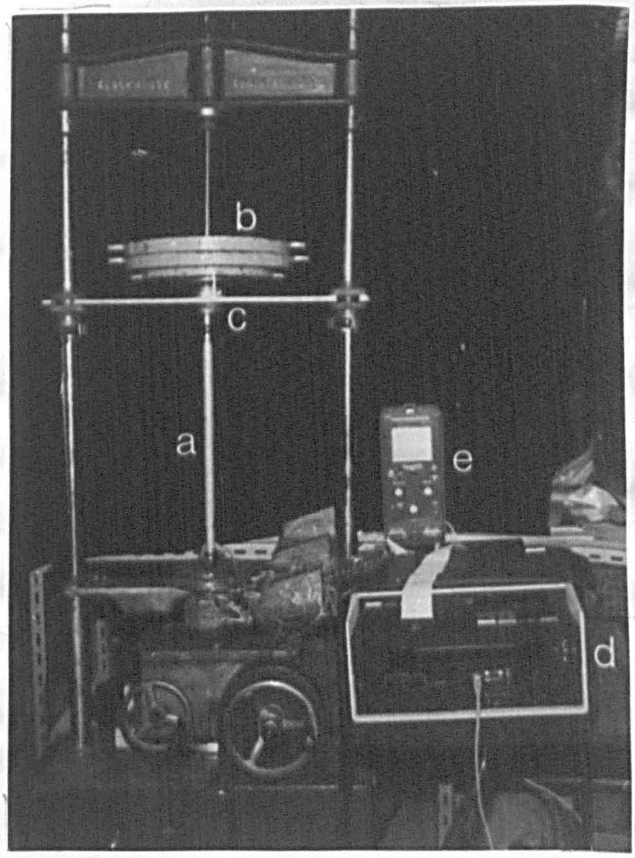


FIG. 4.12 GENERAL ARRANGEMENT OF CALIBRATION OF LOAD-CELL.

a) Pile section.

b) Dead weights.

c) Guide.

d) Portable logger.

e) Power supply.

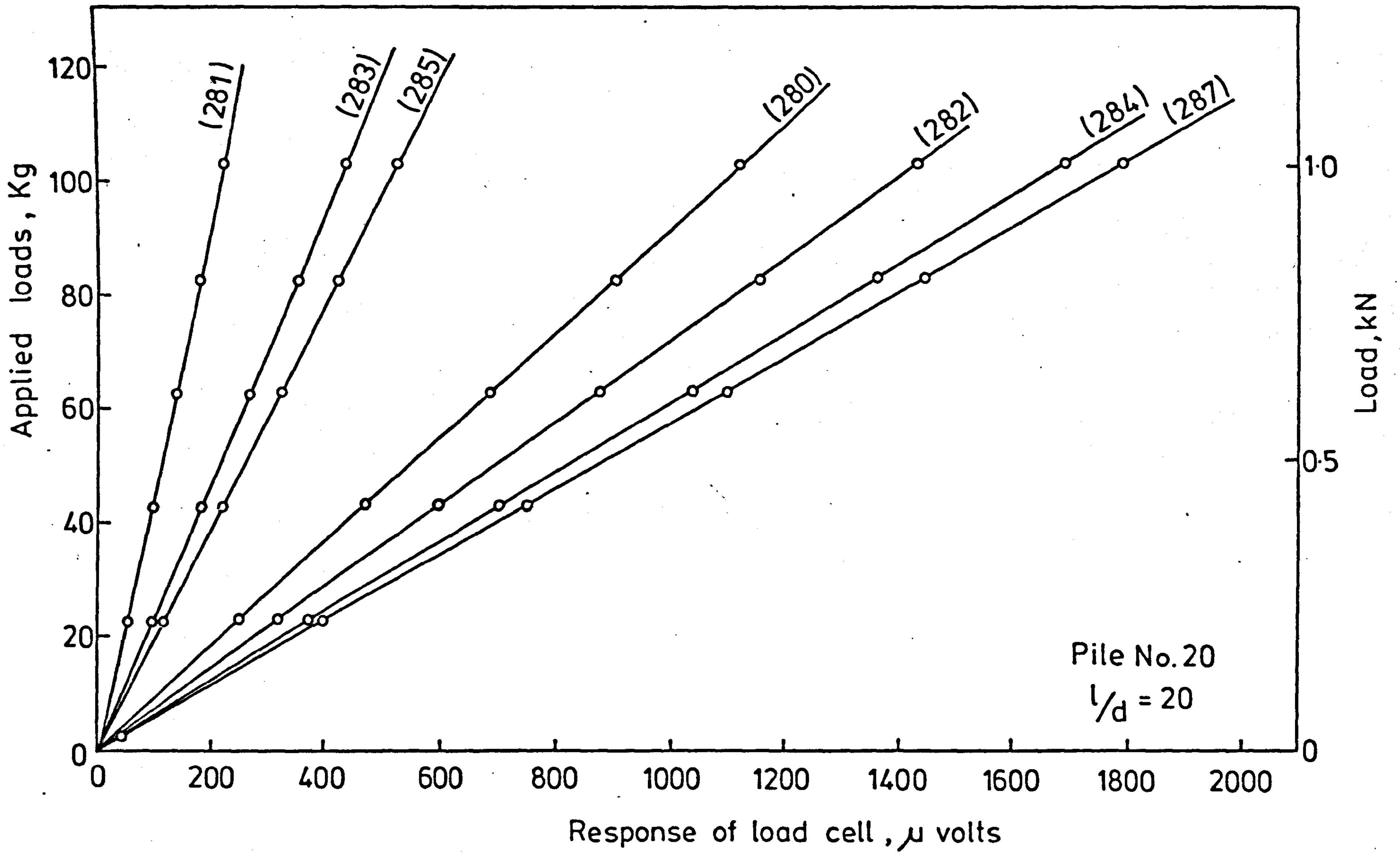


FIG. 4-13 TYPICAL CALIBRATION RESULTS FOR LOAD CELLS

selected.

(iii) The digital Voltmeter with the display screen, and

(iv) An auto-mode switching out-put unit which prints on paper and/or punches on tape.

CHAPTER 5

EXPERIMENTAL PROCEDURES AND TESTING PROGRAMME

5.1 Properties And Sand Placement

The sand used in this investigation was the same as that used by Chan (1976) and Modhloom (1978). In order to clean the sand it was washed with running water and dried in an oven. Prior to testing, the sand was sieved between No. 10 and 100 sieves to remove the coarser and finer materials respectively. According to the particle size distribution curve shown in Fig. 5.1. the sand is a medium, uniformly graded sand. The sand was in an air-dry state with a moisture content of 0.2%. The specific gravity of the sand grains was 2.64 and the maximum and minimum densities, obtained following the method of Kolbuszewski (1948) were 1.84 and 1.34 Mg/m^3. These densities corresponded to void ratios of 0.434 and 0.971 which were slightly different from those reported by Chan.

Preliminary tests indicated that the relative density of the sand was a function of both the intensity of raining and the height of fall, Walker and Whitaker (1967), Vesic (1967). For a 40cm height of fall with 3.3 x 3.3mm mesh and 50mm hose diameter, the density was 1.557 Mg/m^3 which corresponded to a void ratio of 0.696 and a relative density of 51.2%.

Before starting a test the sand container wall was coated with a thin layer of silicone grease. The hopper, which was filled with sand, was lifted by a crane to be at a suitable height and suspended directly above the container. This height was determined so that the mesh was 40cm above the container bottom. To obtain a uniform

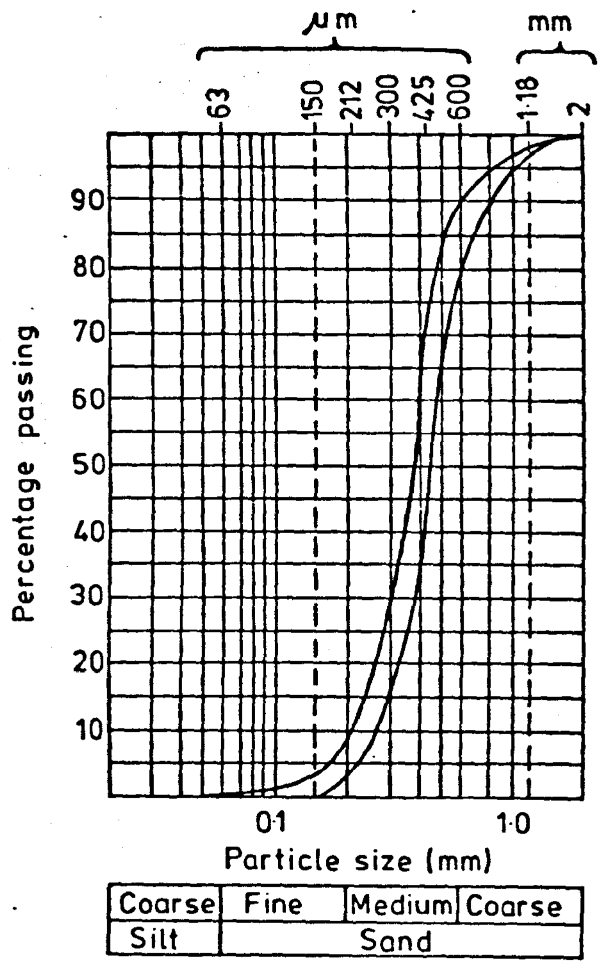


FIG. 5-1 PARTICLE SIZE DISTRIBUTION OF SAND

embedment the sand was distributed uniformly over the surface of placement while the hose was continuously tapped gently by hand. The height of fall was controlled by a small steel washer suspended from the mesh, which was kept always a few millimetres above the sand surface.

5.2 Assembly of Apparatus And Application Of Surcharge Pressure.

When the container was filled, the sand surface was levelled with a hand rotating scoop which also provided a suitable space for the pressure plate. After the pressure plate had been positioned and its bushing coated with a layer of silicone grease, the cover plate which was lifted by a crane, was gently lowered, positioned and bolted to the container. The lever was placed, levelled and supported on the stopper. With great care the upper loading frame was mounted and firmly bolted to the lower steel frame.

In order to set the pile vertical and to adjust the jack so that its centre line became vertical and coincided with the centre line of the pile, two plumbs were used. By means of these plumbs the verticality in two perpendicular planes could be checked. Initially the pile was set randomly and was supported laterally by two special guides located at two different elevations. The first guide was at the lever arm while the other was at the top of the bushing of the pressure plate. Both guides rested in a piston-fit fashion in their holes which allowed only vertical movement of the pile during the driving process. When the lever arm moved in a horizontal plane, the pile which was supported by the lever-guide, rotates in a given vertical plane about its base. Therefore, the verticality of the pile in this vertical plane was adjusted by adjusting the horizontal alignment of the lever.

After the pile was set vertical in this plane the lever was clamped to the roller supports. Since the container axis was originally set vertical and the pile base was located on this axis, hence the vertical axis of the pile coincided with that of the container. To align the axis of the loading jack with that of the sand container the jack was driven down until its lower end became close to the pressure plate bushing. After the verticality of the jack was checked and adjusted the centre lines of the jack, the pile and the container were made coincident. After holding firmly, the jack was redriven up and the pile then repositioned before the jack was brought into contact with the ballbearing located at the top of the pile. Water at a pressure not greater than 10 kN/m^2 was introduced into the space between the pressure plate and the container cover. To flush out any air bubbles which might be trapped underneath the container cover a large amount of water was allowed to run through. The bleed valve was then closed and the chamber pressure increased to the required level. This level was monitored by a regulator mounted on a pressure cylinder. The surcharge pressure was maintained constant at 100 kN/m^2 in most of the penetration tests except for a few tests in which the pressure was either 0 or 200 kN/m^2 as shown in the test programme, table 5.1.

5.3 Depth Of Embedment

Three different depths of embedment have been examined in this investigation. They were 570, 380 and 285mm which are equivalent to 30, 20 and 15 times the pile diameter.

5.4 Pile Driving

The driving process was carried out by a jack of 1000mm length and 25mm diameter driven by an electrical motor of 2HP capacity with a constant rate of penetration of 36mm per minute. Both the motor and the jack were carried by a rigid steel frame, Fig. 5.2. Because of the large resistance encountered during driving additional lateral supports were provided specially in those tests where the longer pile was tested and/or a high surcharge pressure was applied to the sand surface.

Since the length of the longer pile was greater than the space between the lever arm and the cross beam of the upper loading frame, the cross beam was designed in such a manner that it could be rotated through 90° as shown in Fig. 5.3.

5.5 Static Loading Tests

These tests were carried out to study the load-displacement response of the pile before and after the pile was tested under repeated loading. The method of testing was by incremental loading, Whitaker and Cooke (1961), in which each load increment was maintained until settlement had ceased before adding the next load increment. The top and base displacement of the pile were measured by dial gauges. After the pile movement had substantially ceased, scannning of load-cell readings was carried out for each increment of load.

5.6 Repeated Loading Tests

The arrangement of such loading, which was discussed in section 4.4 is shown in Fig. 4.4. The repeated load was generally taken as a percentage of ultimate capacity of the pile and applied in the form of dead weights.carried by the load hanger. The lower repeated loading was placed on the upper part of the hanger while the lower part of the hanger carried the amplitude of the repeated load. Thus, in a complete loading cycle the reciprocating machine carried the lower hanger up, then down to the original position in a 60 seconds period. The measurements made during each test were

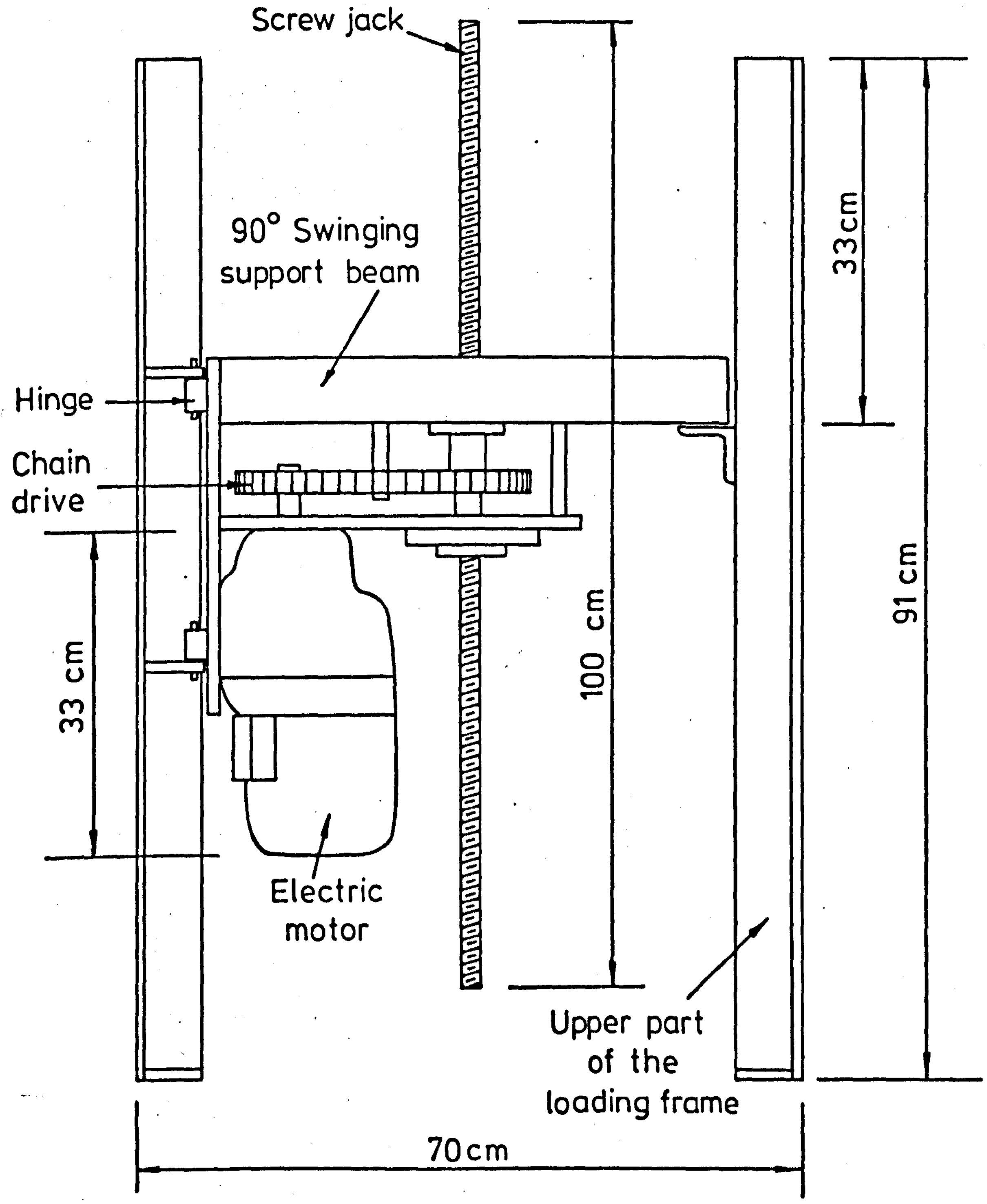


FIG. 5-2 DETAILS OF THE SWINGING JACK AND ELECTRIC MOTOR

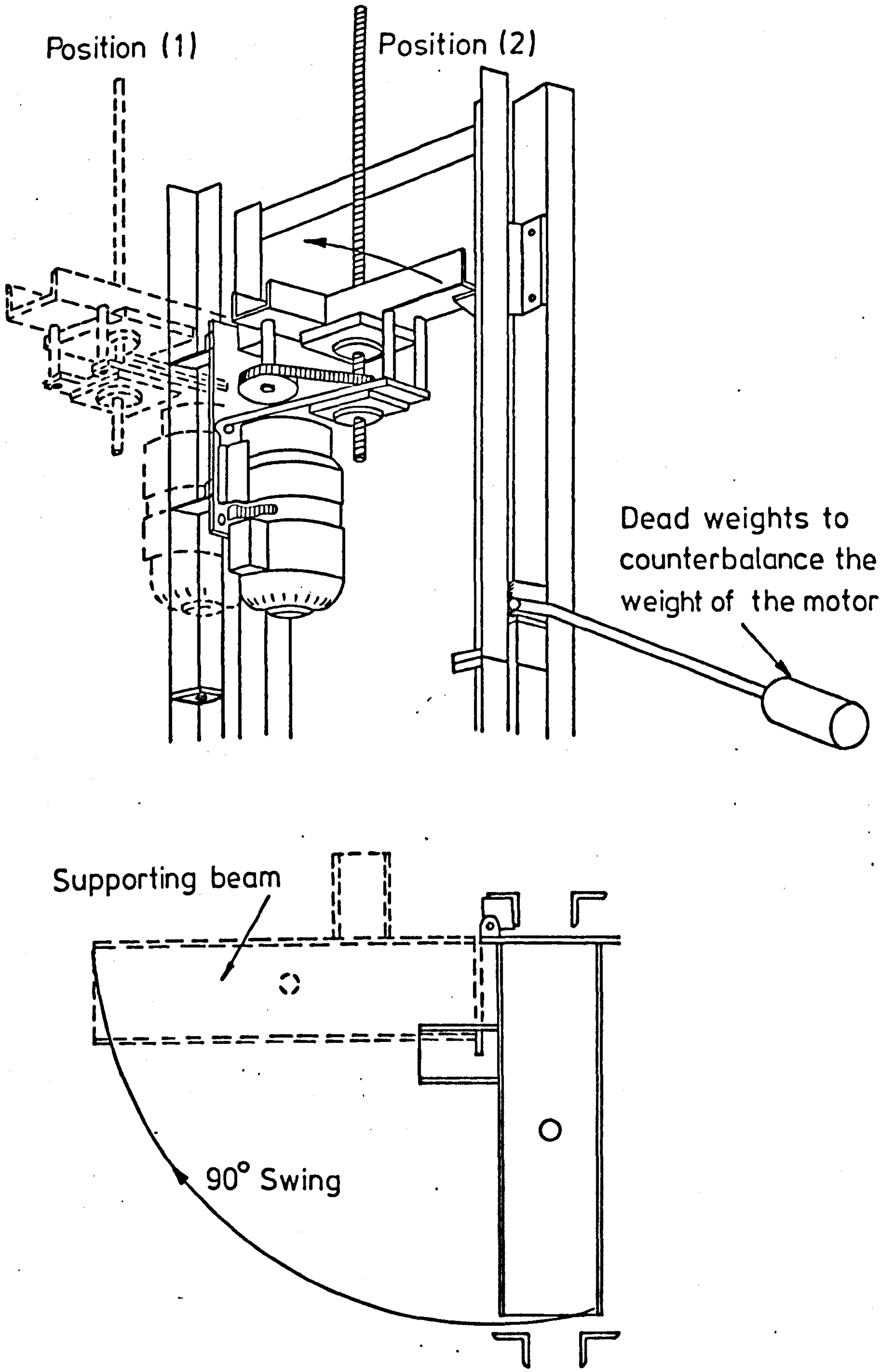


FIG. 5-3 TWO POSITIONS OF THE JACK ALLOWING FOR PILE INSERTION

the reading of the load-cells and the pile movement which were taken at the end of the following cycles, 1, 2, 5, 10, 20, 50, 100, 200. In each of these cycles measurements were taken when the repeated loading was on the high load level and when it was on low load level. When the pile moved a large amount (generally more than 0.7 of the pile diameter) or when the rate of movement of the compression pile changes from unstable to another stable stage, the repeated load test was terminated.

5.7 Re-levelling Of The Lever Arm

With increase in the number of cycles the pile movement increased and the lever arm was no longer horizontal. The re-levelling of the lever was achieved by the aid of the thrust bearing screw Fig. 5.4 and was done slowly and smoothly during the lower loading phase. The horizontality of the lever was checked frequently by a water level mounted on the top edge of the lever.

5.8 Collecting Of Data And Its Analysis

From the data logger output and the dial gauge readings together with a computer programme the following were calculated :-

(i) Variation of the rate of movement with the number of load cycles.

(ii) Variation of the axial pile loading with the number of load cycles.

(iii) Distribution of the average skin friction along the pile depth and the variation of this friction with the number of load cycles during the high and during the low repeated loading.

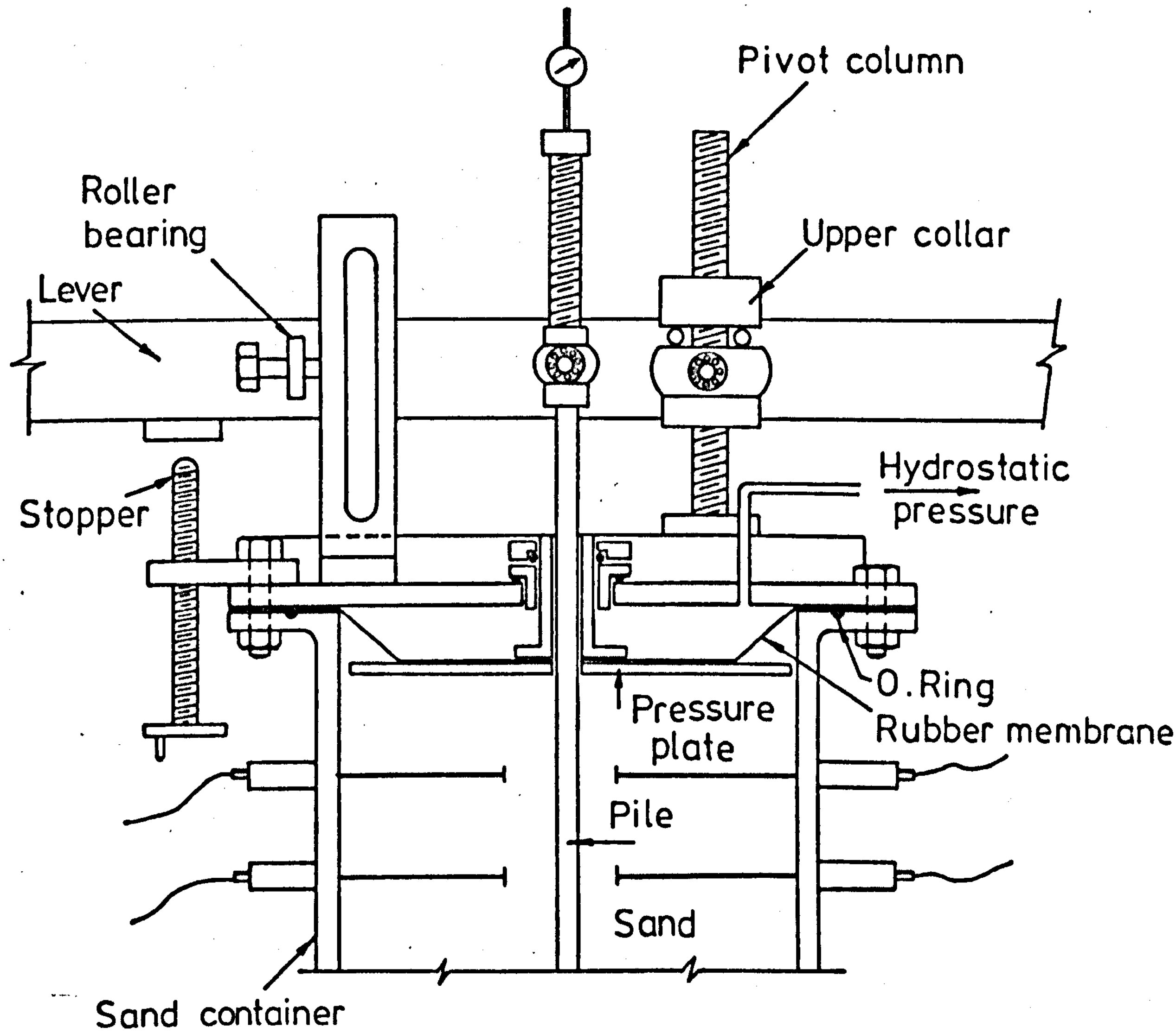


FIG. 5-4 DIAGRAMMATIC DETAIL OF THE LOADING LEVER

5.9 Test programme

The test programme was concerned with the behaviour of both compression and tension piles when subjected to different types of repeated loading. Variation in the state of the effective stresses around the pile was also considered in this programme. The test programme was divided into the following main parts :-

Part A, which deals with the general behaviour of piles under the condition of static surcharge pressure, table 5.1.

Part B, which deals with the general behaviour of the pile when the sand surface is subjected to cyclic surcharge, table 5.2.

TEST 5.1

STATIC SURCHARGE TEST PROGRAMME

Test			Type of Loading	Range of Repeated Loading	Past Load History	Surcharge Pressure kN/m^2	$\frac{\ell}{d}$	Remarks
Series	No	Name						
1	1	-	Compression	-	-	100	15	
	2	-	"	-	-	0	20	Preliminary
	3	-	"	-	-	100	"	
	4	-	"	-	-	200	"	Tests for
	5	-	"	-	-	100	30	
	6	-	Tension	-	-	100	15	static-
	7	-	"	-	-	0	20	
	8	-	"	-	-	100	"	capacity
	9	-	"	-	-	200	"	
	10	-	"	-	-	100	30	determination
11	11	3015	Compression	0.3Qc/0.0**	V+	100	30	
	12	3013	"	"	A++	"	"	
	13	2005	"	"	V	"	20	
	14	2202	"	"	A	"	"	
	15	1507	"	"	V	"	15	
	16	1505	"	"	A	"	"	
	17	3023	"	0.5Qc/0.0	V	"	30	
	18	3017	"	0.7Qc/0.0	V	"	"	
	19	2003	"	0.5Qc/ 0.2Qc	A	"	20	
	20	2015	"	0.5Qc/0.0	V	"	"	
111	21	3020	Tension	0.3Qt/0.0***	V	100	30	
	22	3016	"	"	A	"	"	
	23	2210	"	"	V	"	20	
	24	2205	"	"	A	"	"	
	25	1508	"	"	A	"	15	
	26	2016	"	0.5Qt/0.0	V	"	20	
	27	2209	"	0.5Qt/ 0.2Qt	V	"	"	
1V	28	2009	Compression	0.3Qc/0.0	A	0	20	
	29	2008	"	"	A	200	"	
	30	2207	Tension	0.3Qt/0.0	A	0	"	
	31	2010	"	"	A	200	"	

TABLE 5.1 (cont'd)

Test		Name	Type of Loading	Range of Repeated Loading	Past Load History	Surcharge Pressure kN/m^2	$\frac{\ell}{d}$	Remarks
Series	No							
V	32	3017	Compression	0.7Qc/0.0 0.5Qc/0.0 0.3Qc/0.0	V	100	30	
	33	3018	"	0.3Qc/0.0 0.5Qc/0.0 0.7Qc/0.0	V	"	30	
	34	2007	"	0.3Qc/0.0 0.5Qc/0.0 0.99Qc/0.0 0.3Qc/0.0	V	"	20	
	35	2008A	"	0.3Qc/0.0 0.35Qc/0.0 0.3Qc/0.0 0.4Qc/0.0 0.3Qc/0.0 0.5Qc/0.0 0.3Qc/0.0	A	200	20	
Vl	36	2011	Alter	0.15Qc/0.15Qt	V	100	20	
	37	2012	"	.15Qc/.3Qt	V	"	"	
	38	2013	"	.5Qc/.15Qt	V	"	"	
Vll	39	2015	Compression Alter	0.5Qc/0.0 .5Qc/.3 Qt	V	100	20	
	40	3025	Compression	0.7Qc	A	100	30	Creep Test

* $\frac{\ell}{d}$ denotes ratio of the embedment depth to the diameter of the pile.

** Qc denotes static ultimate load in compression.

*** Qt denotes static ultimate load in tension.

\+ V denotes test carried out after installation.

++ A denotes test carried out after ultimate loading test.

TEST 5.2

CYCLIC SURCHARGE TEST PROGRAMME

Test Series	Test No	Test Name	Type of Loading	Range of Repeated Loading	Past Load History	Surcharge kN/m² Max - Min	Surcharge Frequency c.p.m.	$\frac{\ell}{d}$	Remarks
Vlll	41	3014	Compression	0.7Qc	A	100/50	1	30	ℓ=depth of embedment
	42	3019	"	"	V	"	1	"	
	43	2206	"	"	V	"	1	20	d=diameter of pile
	44	3021	Tension	0.7Qt	V	"	1	30	
lX	45	2204	Compression	0.3Qc/0.0	V	100/50	1	20	High surcharge pressure coincides with high load level.
	46	2208	"	0.5Qc/0.0	V	"	1	"	
	47	2004	"	0.3Qc/0.0	V	100/50	1	"	High surcharge pressure coincides with low load level.
X	48	2006	Compression	0.3Qc/0.0	V	100/50	1.5	20	) Surcharge
	49	2014	"	"	V	"	3	"	) frequency
	50	3022	Tension	0.3Qt/0.0	A	"	3	30	)
	51	3024	Compression	0.3Qc/0.0 (3 c.pm.)	V	"	1	30	Loading frequency.
Xl	52	2208A	Compression	0.5Qc/0.0 0.3Qc/0.0 0.3Qc/0.0 0.5Qc/0.0	V	100/50	1	20	Different combinations
	53	2014A	Compression	0.7Qc/0.0 0.7Qc 0.9Qc	V	100/50	3	20	
	54	3022A	Tension	0.3Qt/0.0 0.5Qt/0.0 0.5Qt 0.5Qt/0.0	A	100/50	3	30	

TABLE 5.2 (cont'd)

Test			Type of Loading	Range of Repeated Loading	Past Load History	Surcharge kN/m^2		$\frac{\ell}{d}$	Remarks
Series	No	Name				Max - Min	Frequency c.p.m.		
X1	55	3023A	Compression	0.5Qc/0.0 0.5Qc 0.5Qc/0.0	V	100 100/50 100	1	30	
	56	2016A	Tension	0.5Qt/0.0 0.5Qt 0.5Qt/0.0	V	100 100/50 100	1	20	

CHAPTER 6

PRESENTATION AND DISCUSSION OF STATIC-SURCHARGE TEST RESULTS

6.1 General

This chapter is mainly concerned with the results of tests carried out on piles subjected to repeated loadings and embedded in a sand on which a constant surcharge pressure acted. Out of forty tests in this part of the investigation ten tests, series 1, were performed to study the static load-displacement characteristic of both compression and tension piles.at different depths of embedment. Through the second and the third test series, the influence of depth and load level on the behaviour of piles subjected to repeated loads was examined. The effect of surcharge pressure was examined in the fourth series. Different combinations of repeated loadings were studied in the other three test series. In the following discussion standard terms used are defined as follows:-

(i) Qc and Qt are the ultimate bearing capacity and the ultimate pulling resistance of the pile respectively.

(ii) The load amplitude is the difference between the upper and the lower repeated load levels.

(iii) The pile movement is the irreversible accumulated displacement of the pile which is produced from repeated loading. It was measured in that half cycle at which the magnitude of the repeated load was at the lower level.

(iv) The pile life-span is the number of cycles beyond which the pile movement exceed a certain allowable limit. For comparison, this limit was taken 10mm in this investigation.

(v) The rate of movement is the pile movement per load cycle.

(vi) The depth ratio is the ratio of the embedment depth to the pile diameter.

(vii) The base load is the load that is applied on the pile base when the repeated load is at its upper level.

(viii) The residual base-load is the load that is applied on the pile base when the repeated load is at its lower level.

(ix) Virgin pile is a pile that has not been subjected to any loading after installation.

6.2 Series 1: Preliminary tests.

These tests were performed to determine the magnitude of the ultimate capacity of the pile from which the repeated load levels were established. They were also used to check the performance of the test equipment and the repeatability of the results. The influence of depth of penetration as well as the surcharge pressure on the driving resistance were also examined in this test series.

6.2.1 Driving resistance

The variations in driving resistance of the pile base with depth are presented in Fig. 6.1. These tests were conducted with 100 kN/m^2 surcharge pressure. It is clear that the resistance was independent of the length of pile employed in the test, that is at 100mm penetration, for example, the driving resistance of the longer pile and that of the shorter pile were approximately the same. Initially the pile base resistance increased rapidly with depth until it reached a maximum limiting value after a penetration which varied between 5 to 10 times the pile diameter. Beyond that it decreased slightly and reached an almost constant final value. Tests carried out with 200 kN/m^2 surcharge pressure, Fig. 6.2, revealed the same trend but the maximum driving resistance was attained within a penetration of 4 to 7 times the pile diameter. At 0.0 kN/m^2 surcharge pressure the resistance steadily increased at a rate which decreased as the penetration increased. A comparison between the three curves of Fig. 6.2, indicated that the depth at which the pile base resistance reached a maximum value (critical depth) decreased as the surcharge pressure increased. The present observations at 100 kN/m^2 surcharge pressure are supported by Chan (1976) and Madhloom (1978). They attributed the high initial driving resistance to the presence of the rigid pressure plate which restricted the upward movement of the soil during the shearing stages. The driving resistance, therefore, increased due to the change in the pattern of the slip failure surface, Hanna (1963). Regarding the decrease in the penetration resistance after the peak, Chan stated that this decrease was mainly related to the reduction in the magnitude of the effective vertical pressure which was caused by the container wall friction. The base penetration resistance of the pile has been investigated by Biarez and Foray (1977). They

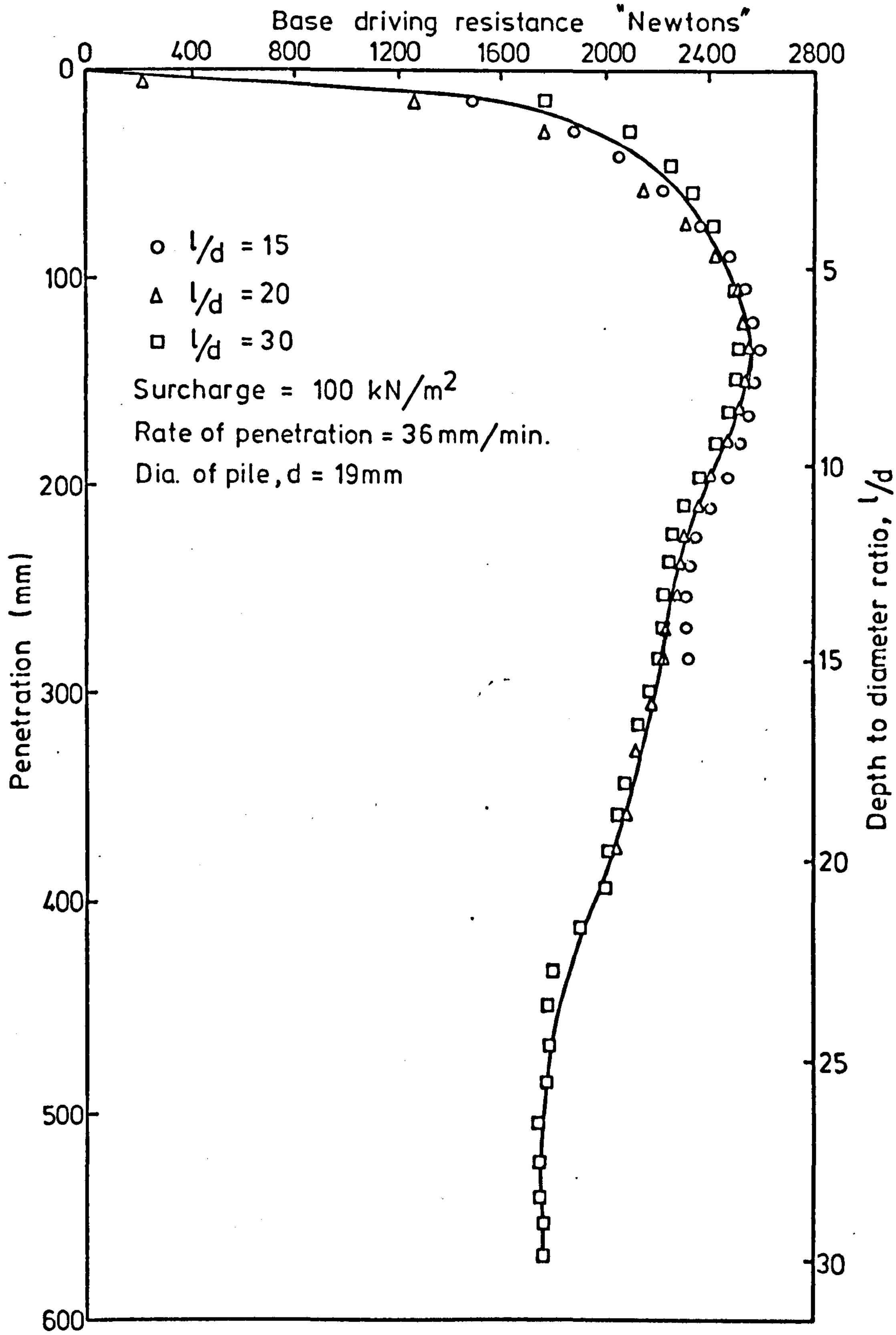


FIG. 6-1 VARIATION IN DRIVING RESISTANCE OF PILE BASE WITH DEPTH

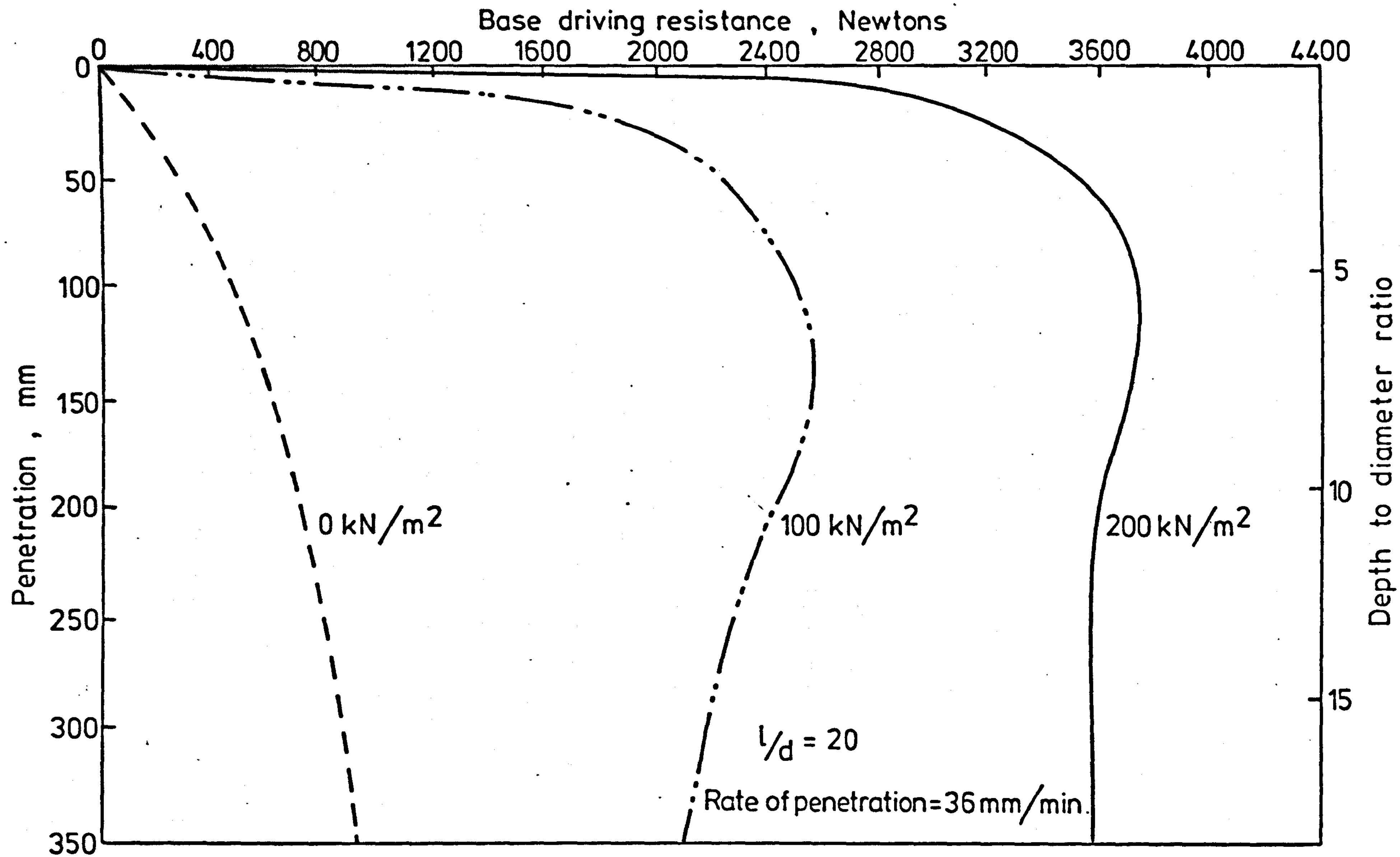


FIG. 6-2 INFLUENCE OF SURCHARGE PRESSURE ON THE BASE PENETRATION RESISTANCE

found that, with zero surcharge pressure, the resistance always increased with penetration but, when a surcharge prssure was applied on the sand surface, the resistance reached a maximum at a certain critical depth and then decreased as the penetration was increased. Moreover, the critical depth was found to decrease as the surcharge pressure increased. To explain the phenomenon of limiting value of penetration resistance, Biarez and Foray postulated that, for a homogeneous sand medium the limiting value was related to the critical confining pressure, (Lee and Seed, 1967). At shallow depths of penetration, the mean applied stress under the pile point was smaller than the critical confining pressure and therefore the sand dilates. As the penetration increases the mean stress increases too and hence, after a certain depth, this stress becomes equal to or higher than the critical confining pressure and thus the sand compresses. Beyond the critical depth the point resistance, therefore, becomes constant. The phenomenon of limiting resistance attracted the attention of many researchers. Vesic (1967) attributed this phenomenon to some kind of arching assumed to take place along the pile depth and hence affecting the state of stresses around the pile shaft. Meyerhof (1976) indicated that the soil compressibility crushing and stress history may also contribute to such a limiting value. Based on field observations Meyerhof (1977) reported that the critical depth, which ranged from 5-15 times the pile diameter depended on the sand density.

The values of the skin friction during identical driving tests were much less repeatable than the base resistance as shown in Fig. 6.3. At the beginning of penetration, the average skin friction was relatively high but after a short penetration, generally not

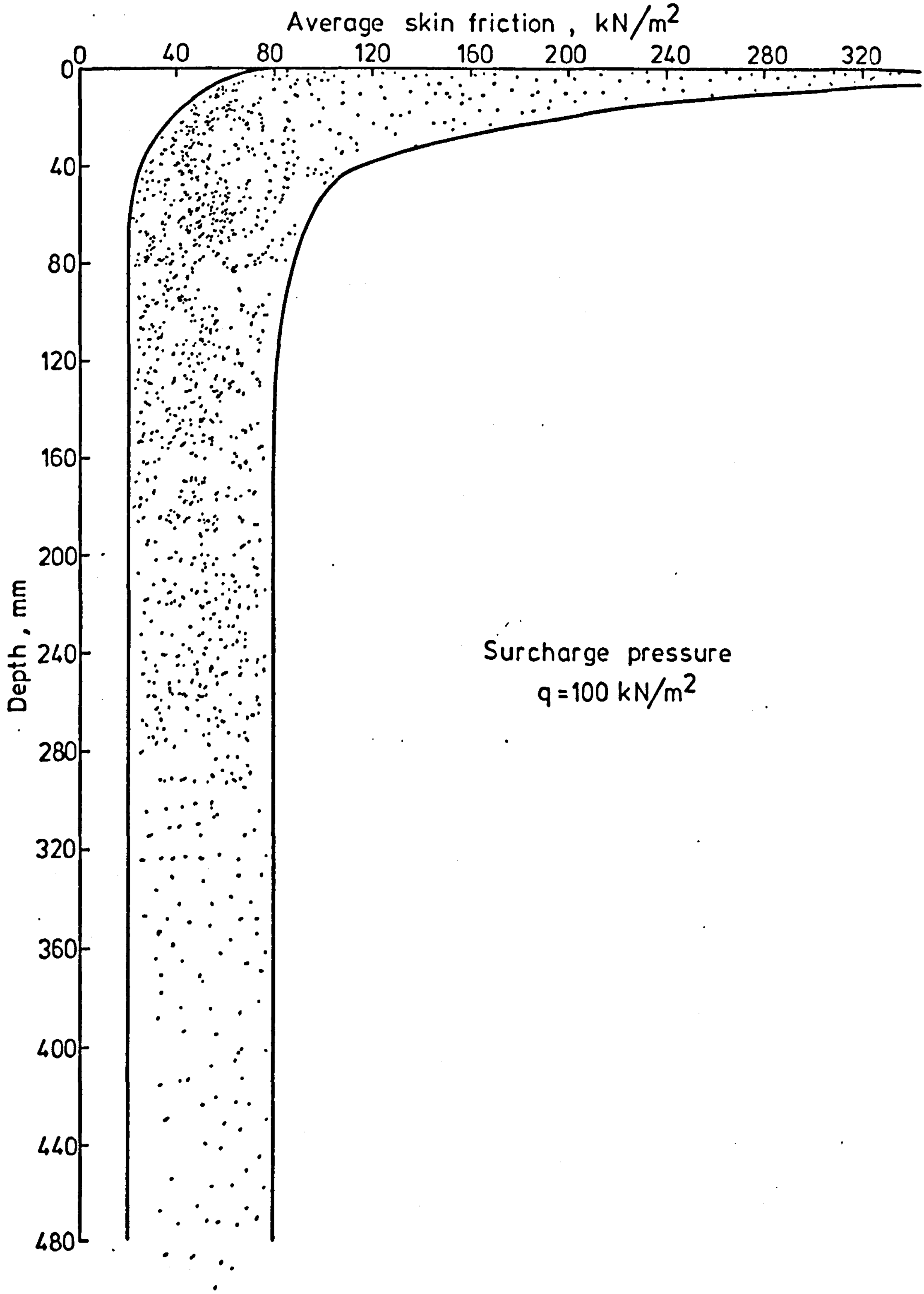


FIG. 6-3 VARIATION OF SKIN FRICTION WITH DEPTH OF PENETRATION

more than two pile diameters, it decreased to a value between 20 to 80 kN/m^2. It is believed that the early high skin friction was due to the presence of the pressure plate as concluded by Chan (1976) and Madhloom (1978). The scattering in skin friction during the subsequent penetration stages is thought to be attributed to the random change in properties and state of motion that took place in the sand during the process of pile penetration. The local skin friction at any given depth, therefore, depends on the void ratio as well as the magnitude and direction of the relative movement between the soil and the pile shaft at that depth. Results of similar tests carried out by Madhloom (1978) revealed that the value of the skin friction did not repeat at any given depth when the penetration test was repeated. Furthermore, the scattering in many times was greater than that quoted in this investigation. The estimation of the pile shaft capacity by means of the Cone Penetration Test is generally based on the cone resistance rather than the local sleeve friction, Thorburn and Buchanan (1979). This is because the local sleeve friction is not repeatable such as the cone resistance (Beringen et al (1979).

6.2.2. Behaviour of piles under static loadings

Results of typical static compression load tests carried on piles embedded at various depth ratios under the influence of 100 kN/m^2 surcharge pressure are presented in Fig. 6.4. It can be seen that the load-displacement relationship of the pile became stiffer and the ultimate capacity was increased as the depth increased. Piles tested at a depth corresponding to 30 times the pile diameter failed at 3.13 kN at which the recorded ultimate base resistance and shaft friction were 1.60 and 1.53 kN respectively. At 20 diameters

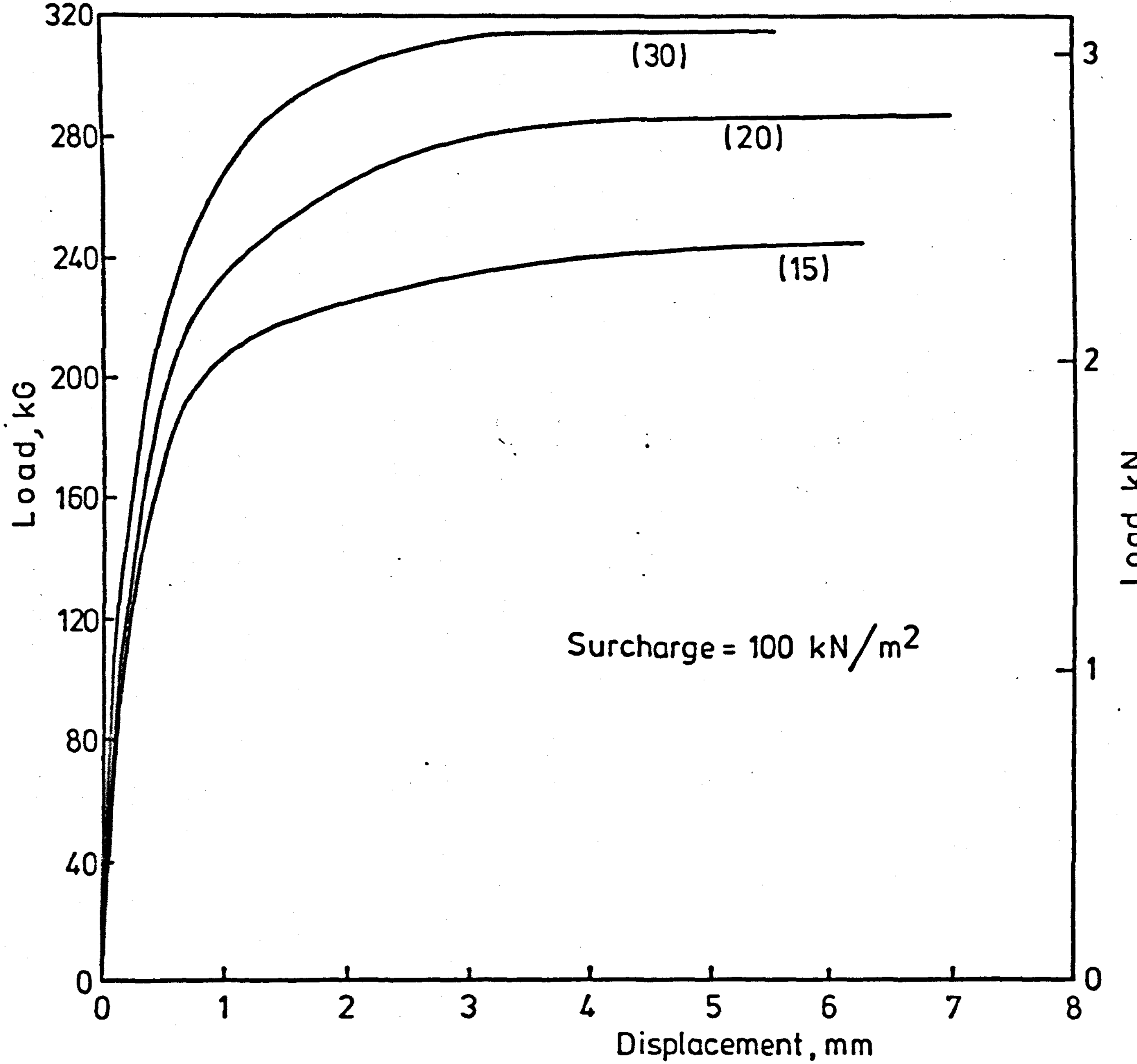


FIG. 6-4 LOAD - DISPLACEMENT BEHAVIOUR FOR COMPRESSION LOAD TESTS

depth the ultimate total, base and shaft friction were 2.75, 1.75 and 1.00 kN respectively, while they were 2.45, 1.72 and 0.73 kN respectively when the pile was tested at 15 diameters. The variation of these capacities with depth is shown in Fig. 6.5. The curve of the total ultimate capacity indicated that this capacity increased with depth but at a decreasing rate, which is in agreement with Vesic (1967). Although the state of equilibrium involved in static tests is different than that of driving tests, Yong (1970), the variation of the ultimate base resistance with depth had a similar trend in both modes of testing. The maximum limiting value was smaller and was attained at a greater depth in the case of the static test. The results of driving and static tests conducted by Chan (1976) revealed that the final penetration resistance of the pile base was approximately 2350 newtons.. This value dropped to 2100 newtons when the pile was subjected to a subsequent static load test. The reduction in the ultimate base resistance after the pile was placed in dense sand was also reported by Yong (1970). He related this phenomenon to the relaxation of the pile. Since dense sand tends to dilate during shearing and the static load test is generally performed many days after the placement of the pile therefore, due to stress relaxation, under subsequent static loading the pile will experience a smaller base resistance. The ultimate shaft friction was found to increase approximately linearly with depth as shown in Fig. 6.5. This variation implies that the average unit skin friction did not change with depth. In an attempt to explain this phenomenon the following is suggested. After the dense sand is sheared during the process of penetration by the pile base it receives high monotonic shear strains (36mm/min) from the pile shaft.

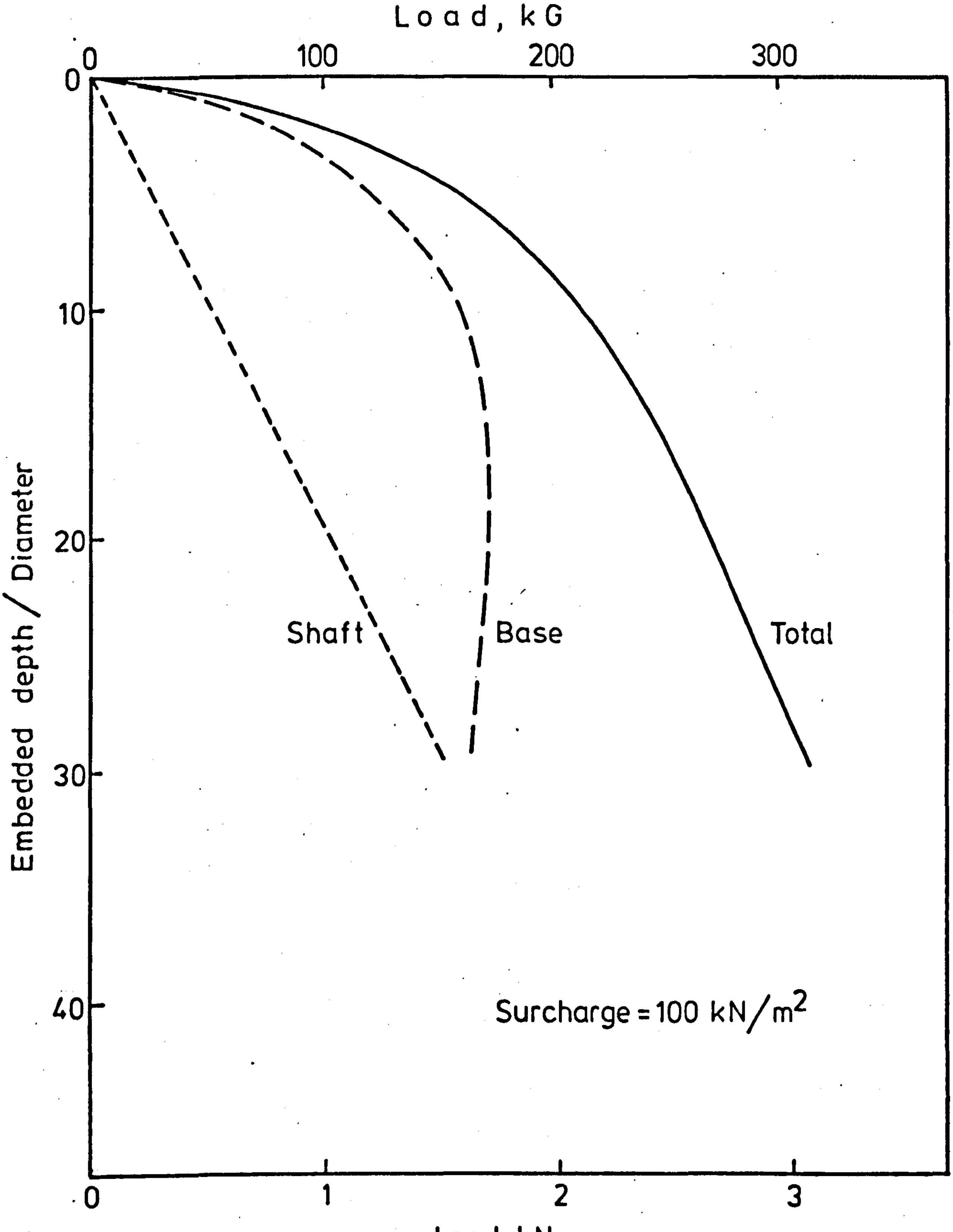


FIG. 6-5 VARIATION OF ULTIMATE RESISTANCE WITH DEPTH

These strains decrease in magnitude as the radial distance from the pile increases, Cook (1974). The sand grains along this distance will, therefore, be set-up in different packings. Close to the pile shaft and due to the high shear strains the grains are arranged into a loose packing, Thorburn and Buchanan (1979) or more precisely into a state of critical void ratio (Youd (1970). This state covers a distance of approximately ten grains and beyond that the void ratio decreases as the distance from the failure surface increase, Roscoe (1970). Since the sand was initially homogeneous and subjected to a relatively high surcharge pressure, the critical void ratio will not change with depth. Because the behaviour of the pile at failure is governed by the strength and deformation properties of the small volume of soil close to the pile shaft, Gallagher and St John (1980), therefore the variation of the ultimate skin friction with depth under these conditions of testing will be constant.

Fig. 6.6 shows the load-displacement curves of pull-out tests conducted on piles at different depth ratios and tested under the influence of 100 kN/m^2 surcharge pressure. At each depth the test was repeated to assess the effect of the residual stresses on the behaviour of the pile. This family of curves revealed that the ultimate pulling resistance of the pile increased as the depth of embedment was increased. They were 1.20, 0.55 and 0.40 kN when the pile was first tested at 30, 20 and 15 depth ratios respectively. The residual base load after penetration at the depth ratios mentioned above were compression of 318, 182 and 93 newtons respectively, and in tension of -42, -33 and -20 newtons respectively after the first tests had been performed. In fact, the residual base load after testing the

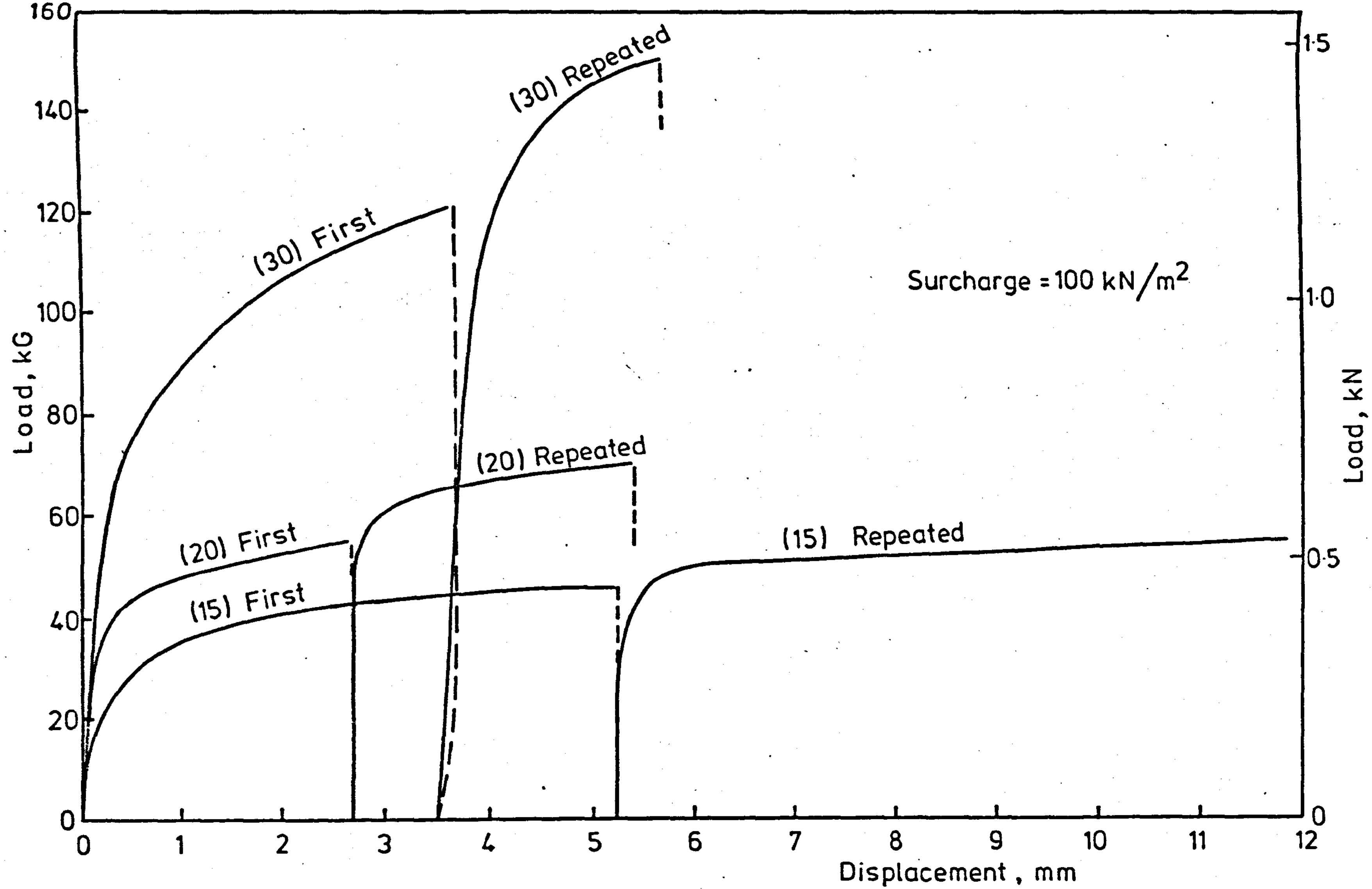


FIG. 6-6 LOAD-DISPLACEMENT BEHAVIOUR FOR TENSION LOAD TESTS

pile in tension must be zero. However, because the base load cell was actually one pile diameter above the base, the recorded load represents the friction of the last one pile diameter segment of the pile. When the test at each depth ratio was repeated, the resulting load-displacement relationship became stiffer and the ultimate pulling resistance was increased. This increase in the pile capacity may be related directly to the change in state of the residual stresses. At the beginning of the first test the pile was entirely subjected to compressive residual stresses. These stresses became entirely tensile at the beginning of the repeated test. To compare the failure shaft friction during the compression loading with that during tension loading, the results of the corresponding tests are drawn together in Fig. 6.7. From these curves it may be concluded that the shaft friction in compression was always greater than that in tension by a magnitude dependent on the depth of embedment and the state of the residual stresses. The influence of surcharge pressure on the load-displacement response of compression piles tested at 20 diameters depth is shown in Fig. 6.8. It can be seen that the increase in surcharge pressure produced a stiffer load-displacement relationship. The ultimate load capacity of the pile increased as the surcharge pressure increased. When the results of these tests were plotted together with those of tension tests conducted on the same pile as shown in Fig. 6.9 it was noticed that the ultimate shaft friction in compression tended to reach a limiting value beyond which it became independent of the surcharge pressure. In contrast, the tension shaft friction, which was initially of smaller value as compared with that of the compression test, was continued to increase when the surcharge pressure increased. The

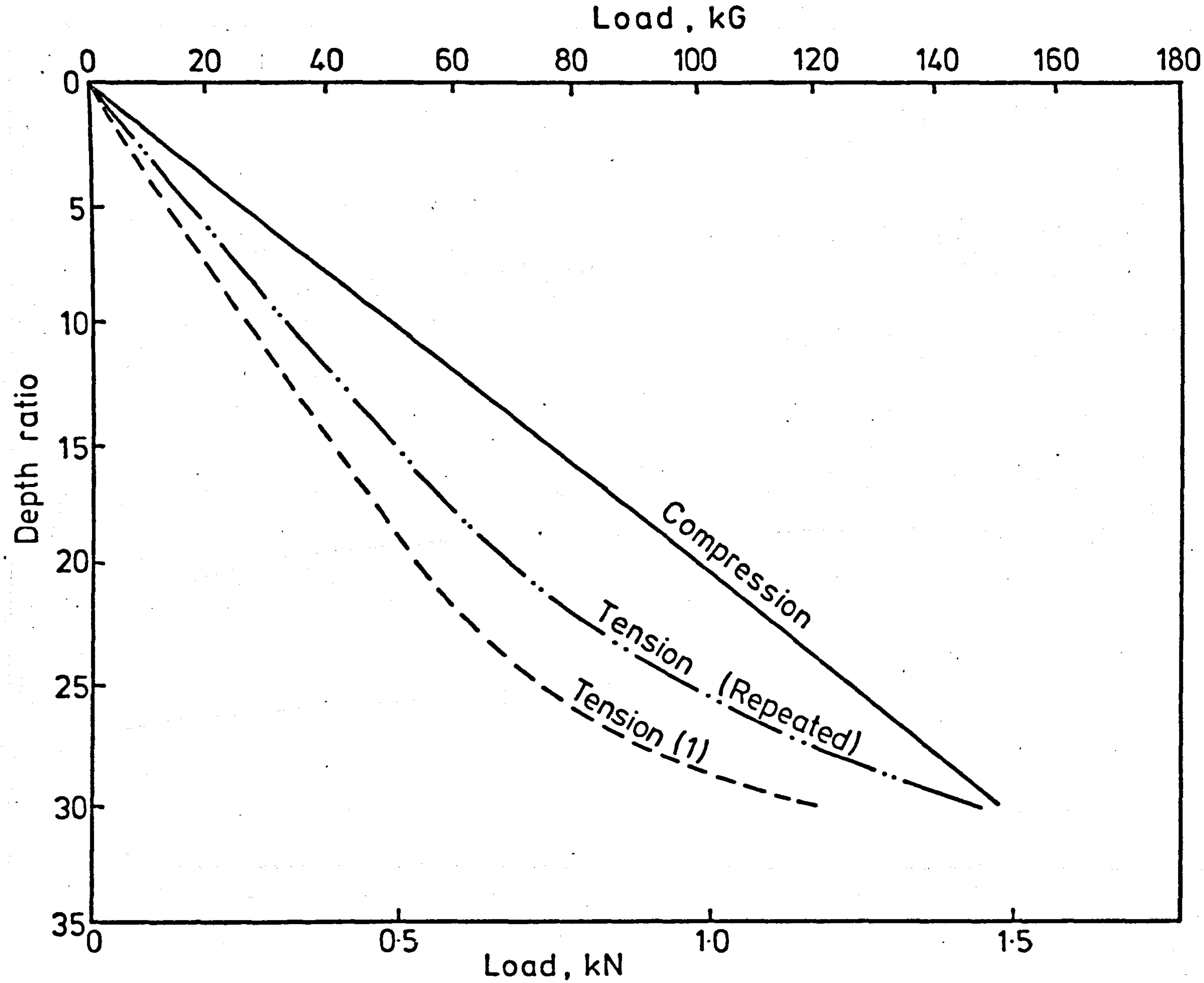


FIG. 6-7 VARIATION OF ULTIMATE SHAFT FRICTION WITH DEPTH

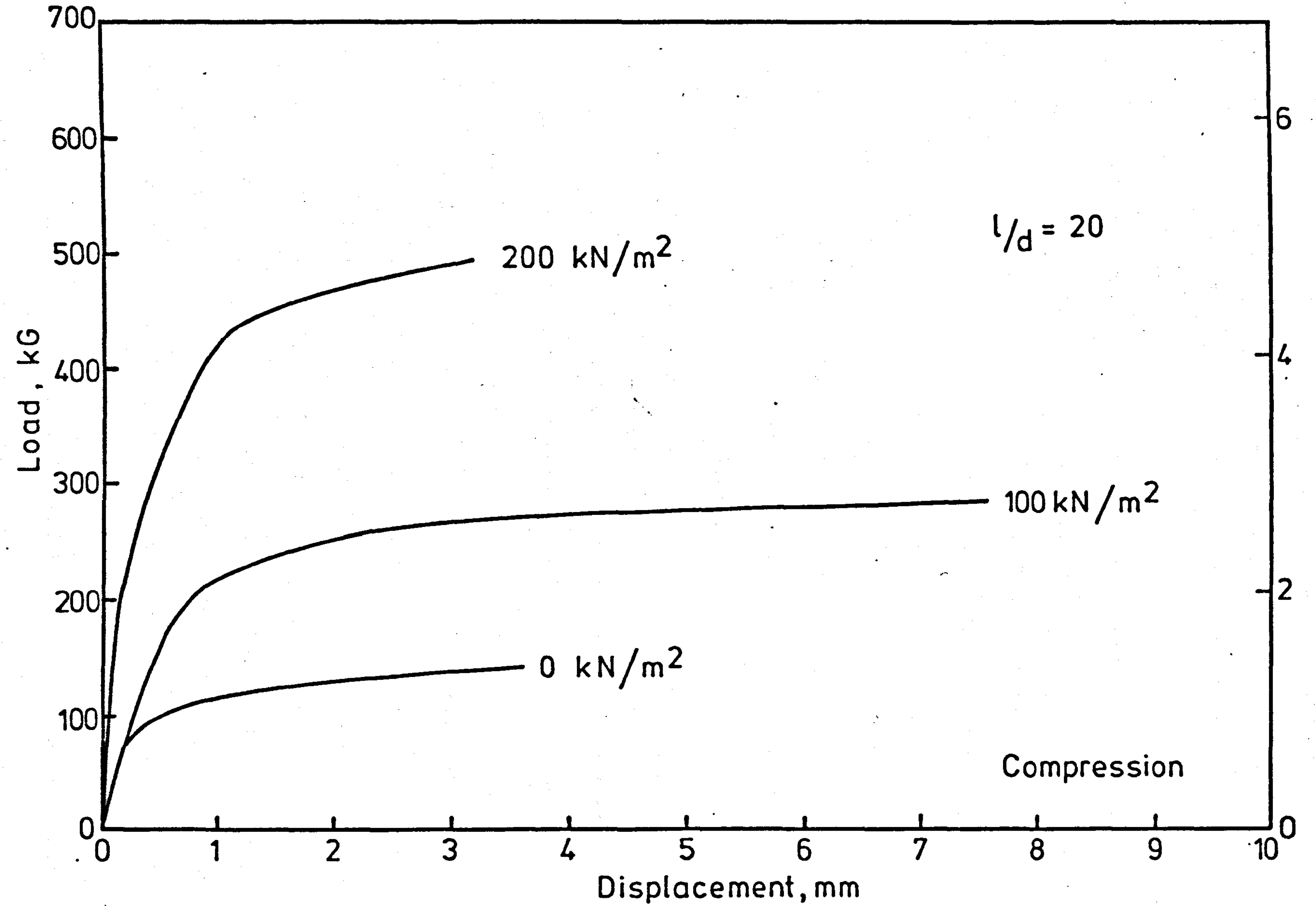


FIG. 6-8 INFLUENCE OF THE SURCHARGE PRESSURE ON THE LOAD-DISPLACEMENT BEHAVIOUR OF THE PILE

difference between the trend of the compression and that of the tension friction may be attributed to the residual stresses as indicated above. The effect of surcharge pressure seems to increase the ultimate base resistance at an increasing rate as shown in Fig. 6.9.

In Fig. 6.10 the residual base load as a percentage of the ultimate base resistance is plotted against the surcharge pressure and against the pile depth ratio. It is clear that the residual base load increased at a decreasing rate when the surcharge pressure was increased, and rapidly reached a limiting value. In contrast, with increase of the depth ratio the residual base load increased at an increasing rate up to a depth ratio of 20. Beyond that it began to decrease gradually which indicated that the residual base load was also tending to reach a limiting value.

6.3 Series 11: Compressive repeated loads with a constant surcharge pressure equal to 100 kN/m^2

As mentioned earlier, the present investigation is part of a continuing research programme to study the behaviour of piles under repeated loads. The first part which was conducted by Chan (1976) was concerned with the behaviour of piles embedded at 30 diameters depth in sand upon which a constant surcharge pressure of 100 kN/m^2 was acting. The behaviour of compression as well as tension piles was examined under a wide range of load amplitudes. Therefore, in this section, as well as in the subsequent sections of this chapter the investigation was concentrated on the influence of both the depth of embedment and the surcharge pressure on the performance of piles subjected to repeated loads. The repeated loads chosen for this purpose were 0.3 Qc/0.0 or 0.3Qt/0.0 which means that the pile load was cycled between zero and 0.3 of the ultimate compression and the ultimate pulling resistance of the pile respectively. The

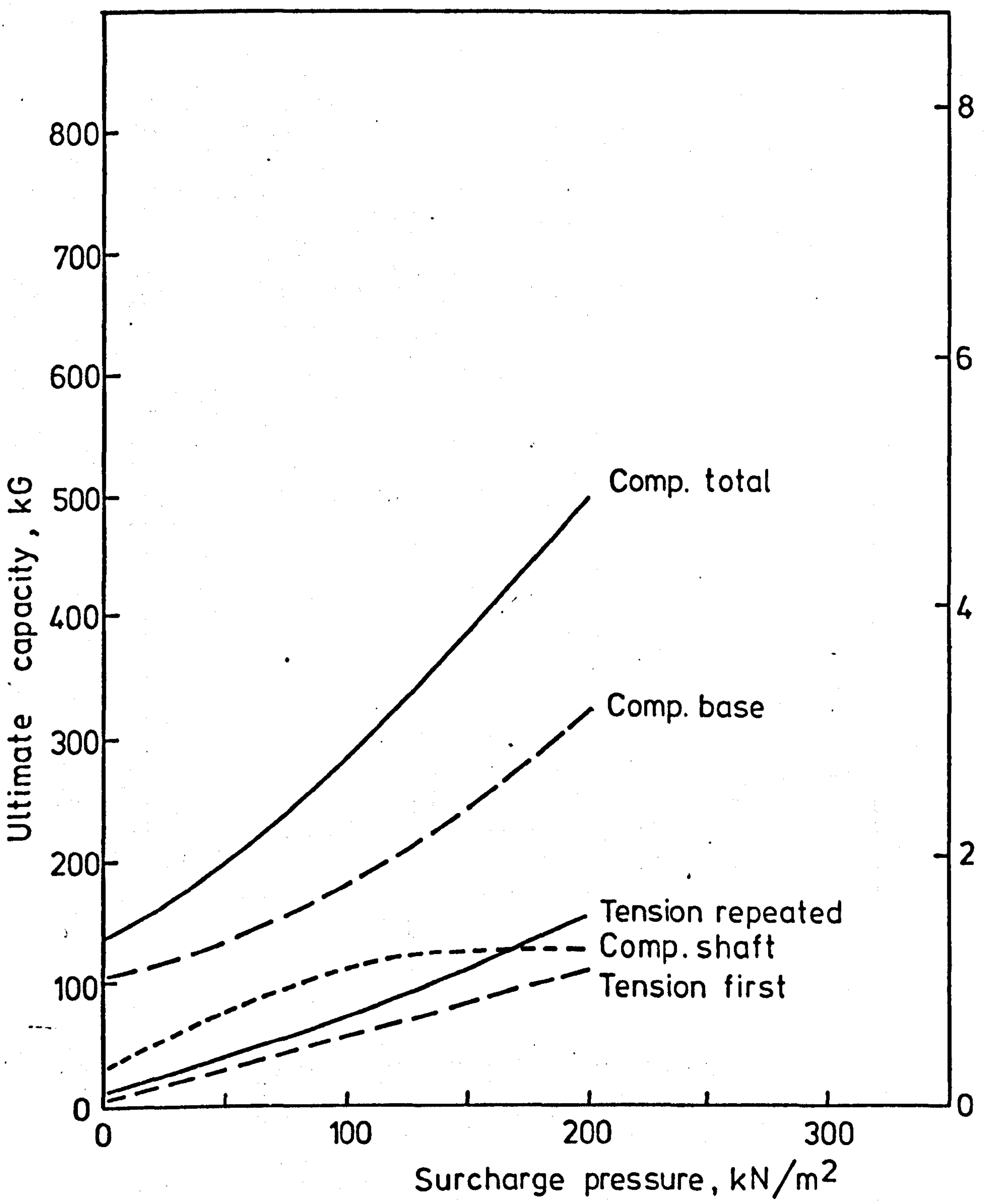


FIG. 6·9 VARIATION OF ULTIMATE CAPACITY WITH SURCHARGE PRESSURE

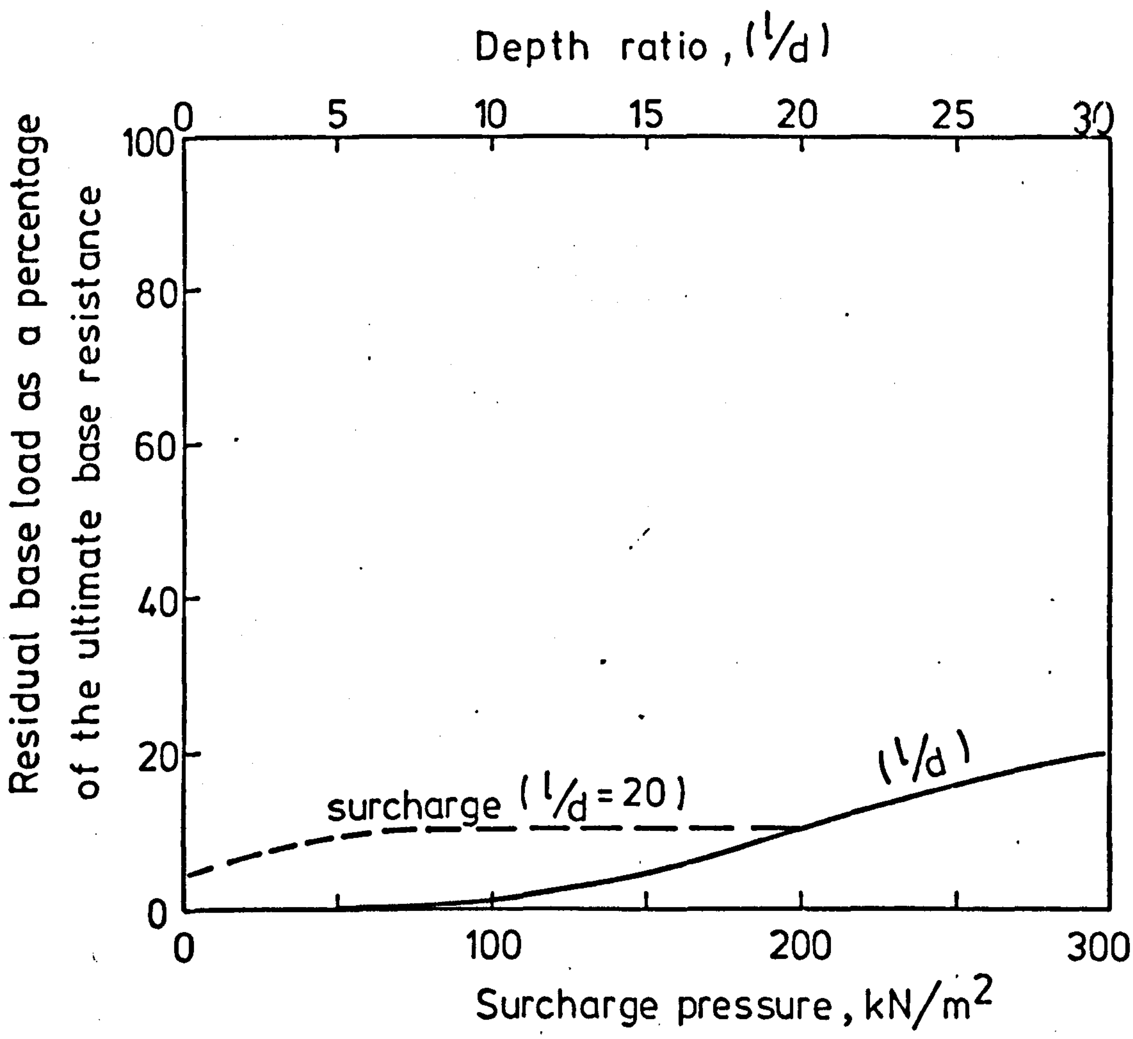


FIG. 6-10 VARIATION OF RESIDUAL BASE LOAD WITH SURCHARGE PRESS. AND WITH DEPTH

reason behind the choice of such repeated loads is that the design load of a foundation pile is, in general, limited within this range. Piles were also subjected to a range of loading conditions in this part of the investigation.

6.3.1. Effect of embedment depth

The movement of piles examined in Tests 11, 13 and 15 at 30, 20 and 15 depth ratio respectively and subjected to 0.3Qc/0.0 repeated loads are shown in Fig. 6.11. Those of Tests 17 and 20 which were at 30 and 20 depth ratios respectively under a repeated loading of 0.5Qc/0.0 are presented in Fig. 6.12. It is concluded that the life-span of the pile decreased when the embedment depth increased. The variations of the rate of movement and the pile base load against the logarithm of number of load cycles of Tests 11, 13 and 15 are shown in Fig. 6.13. At any depth, the pile was stable during the initial stage of repeated loading. After a certain number of load cycles the rate began to increase until it reached a maximum value then decreased as the number of cycles was increased. The value of the maximum rate appeared to increase as the depth of the pile increased. It was 0.0115, 0.0062 and 0.0043mm/cycle for 30, 20 and 15 diameters depth respectively at approximately the same number of cycles (around the 1300th cycle). During the intial few cycles an upwards movements were observed before the pile began to penetrate into the sand. These movements may be attributed to the change in state of the residual stresses from those generated after penetration (failure loading) to those corresponding to a loading of 0.3Qc. Because the upper repeated load level was always 0.3Qc the difference between this load and the base load was equal to the embedded shaft friction. Similarly and because the lower repeated load level was zero in each cycle the residual base load was equal to the residual shaft friction but with negative sign. As shown in Fig.

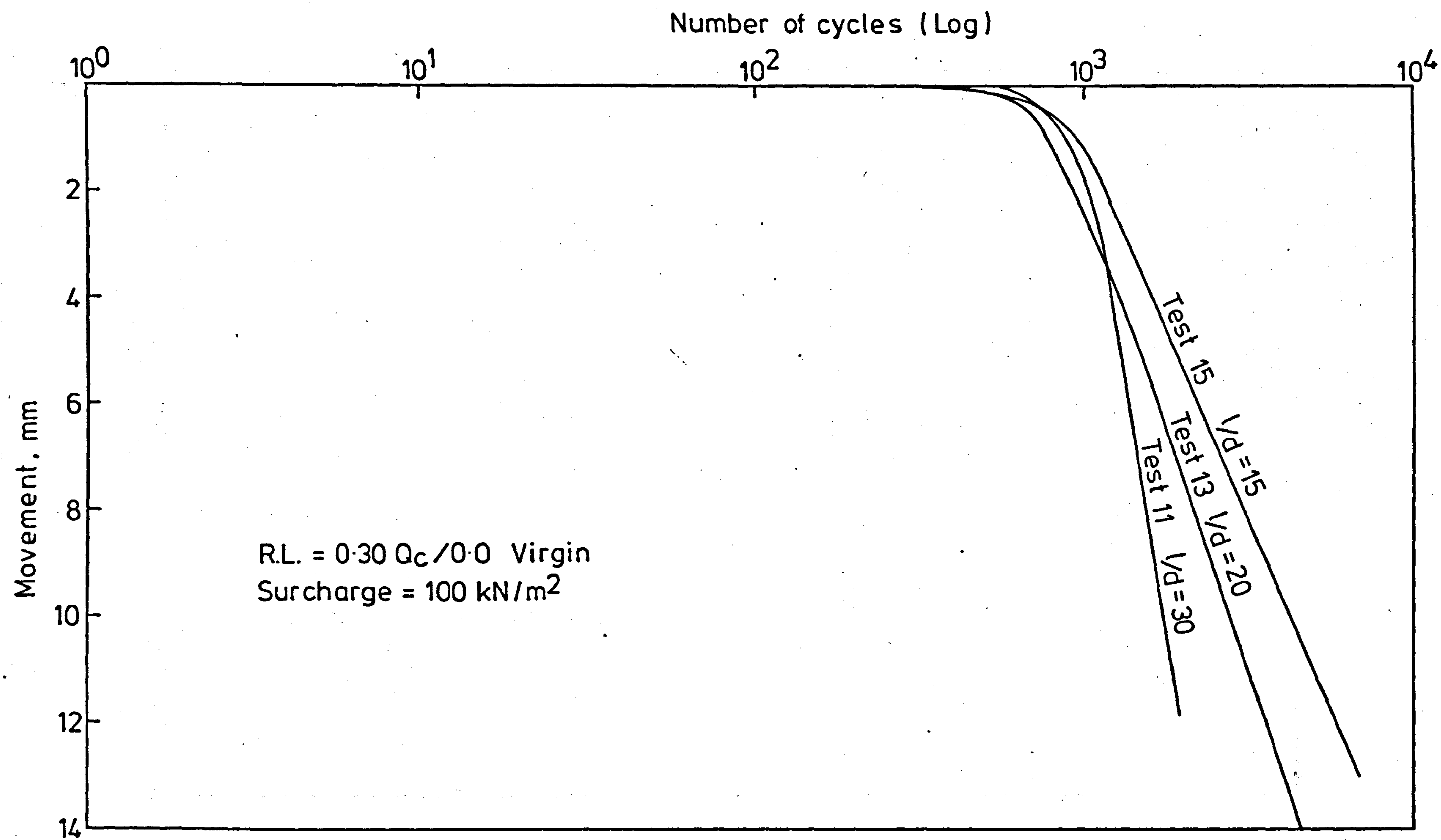


FIG. 6-11. INFLUENCE OF EMBEDMENT DEPTH ON THE PILE LIFE SPAN

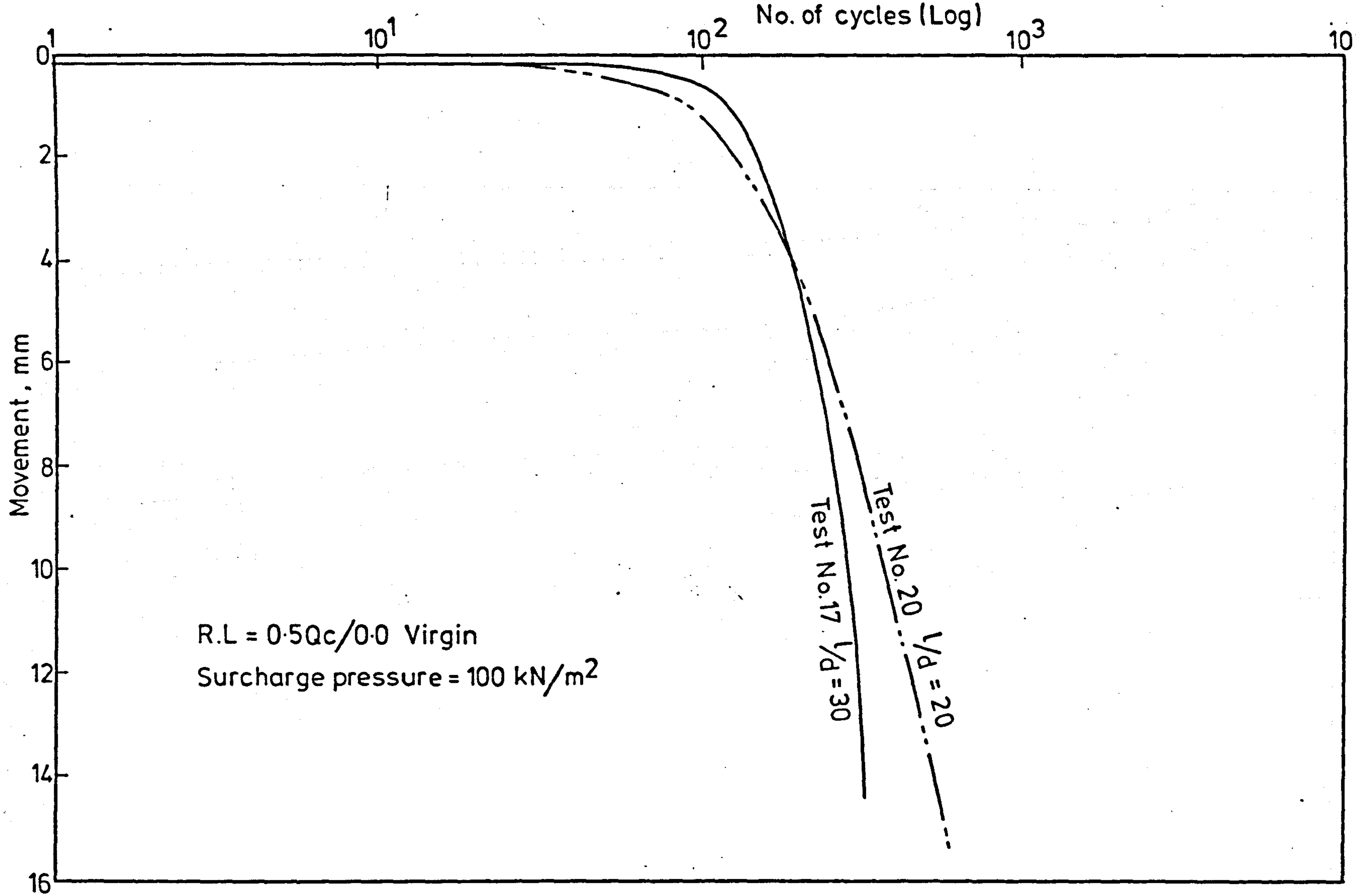


FIG. 6-12 INFLUENCE OF EMBEDMENT DEPTH ON THE PILE LIFE-SPAN

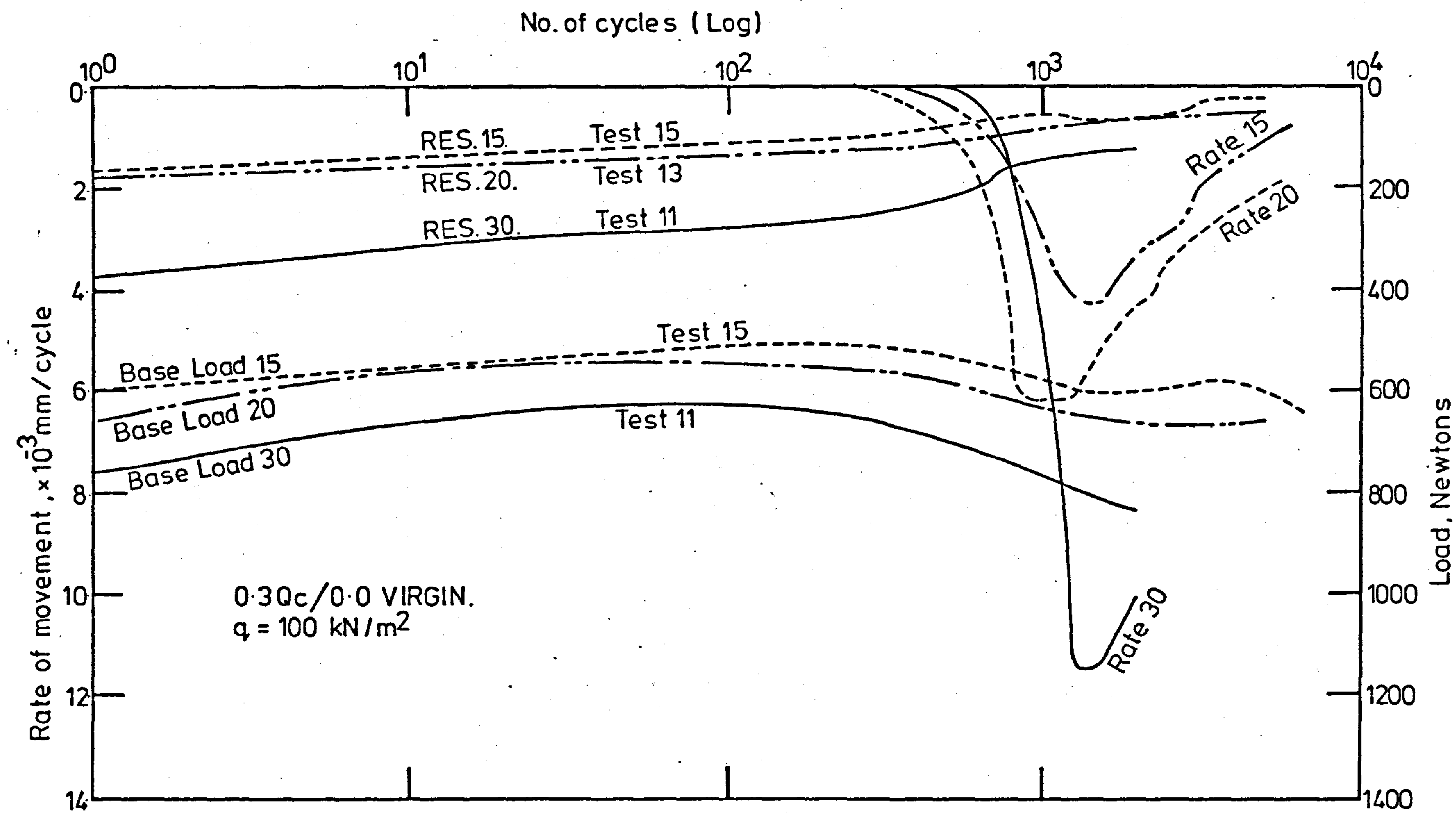


FIG. 6-13 VARIATION OF PILE LOAD AND RATE OF MOVEMENT WITH LOG NO. OF CYCLES.

6.13, at any depth, when the number of load cycles increased the shaft friction increased up to a peak value after which it decreased progressively and reached a limiting value. This limiting value increased as the embedment depth of the pile was increased. The number of cycles through which the shaft friction dropped from the peak to the limiting value appeared to increase when the pile was placed at a greater embedment. If the rates of movement of each test are related to the shaft frictions it will be seen that the rate began to increase at the cycle in which the shaft friction started to decrease from its peak value and continued to increase so long as the value of the shaft friction was decreasing.

The distribution of axial loads and residual axial loads with the corresponding skin friction during three selected cycles of Test 11 are presented in Fig. 6.14. The selected cycles were the first cycle, the cycle at which the shaft friction reached its peak value, and the last cycle in the test. As can be seen in Fig. 6.14B, initially and up to the cycle number 50, the skin friction of all points along the pile shaft increased. Beyond that cycle it decreased progressively as the number of cycles was increased until it reached almost a constant value along the upper half of the pile. Along the lower half the skin friction increased as the depth of the point was increased, and became equal to that of the first cycle for points located within the last three diameters of the pile. The distribution of the residual skin friction, unloading phase, revealed a continuous decrease from the first cycle until the end of the test. In contrast to the loading phase the repeated loading reduced the residual skin friction of the lower half of the pile significantly.

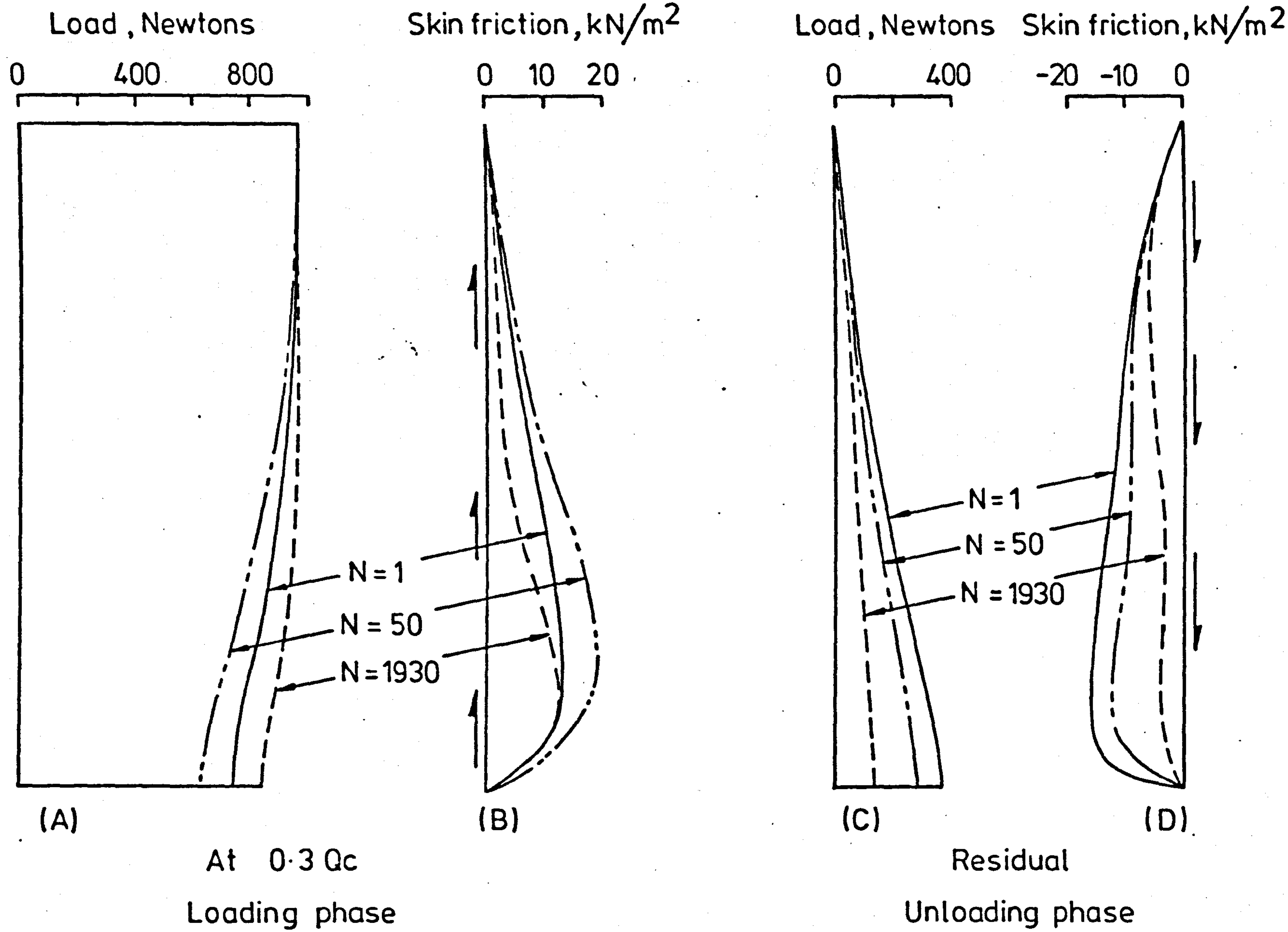


FIG. (6-14) DISTRIBUTIONS OF AXIAL LOAD AND SKIN FRICTION DURING SELECTED CYCLES OF TEST 11

6.3.2. Effect of static failure loading on the subsequent behaviour of piles subjected to repeated loads.

The results of Test 12 which was performed on a pile of 30 diameters depth and subjected to a static failure load test before being tested under repeated load of 0.3Qc/0.0 are presented in Fig. 6.15. When the movements and the rates of movement of this test are compared with that of Test 11 as shown in Fig. 6.16, it will be seen that the pile life-span was reduced when it was previously subjected to a static failure loading.. The load transfer characteristic was also affected by the previous failure loading. Fig. 6.17 shows the load transfer of this pile together with that of a virgin pile, Test 11. At the beginning of the test, the shaft load of the previously loaded pile was a maximum while it was a minimum in the case of the virgin pile. After about 600 cycles the shaft load of both piles reached a limiting value which appeared to be independent of the pile loading history. The residual shaft load of the previously loaded pile was always smaller than that of the virgin pile. The difference was a maximum at the first cycle and a minimum at the end of the test. Similar conclusions were drawn when the results of Test 14 and Test 16 in which the pile had 20 and 15 diameters depth respectively, are compared with those of Test 13 and Test 15 respectively.

6.3.3. Various load levels and amplitudes

Fig. 6.18 shows the pile movements for Tests 11, 17 and 18 against the logarithm of the number of load cycles for various repeated load levels applied to a pile of 30 diameters depth. It is clear that the life-span of the pile reduced when the load level was increased. The number of cycles required to develop the state of maximum rate of movement decreased from 1300 to 400 when the

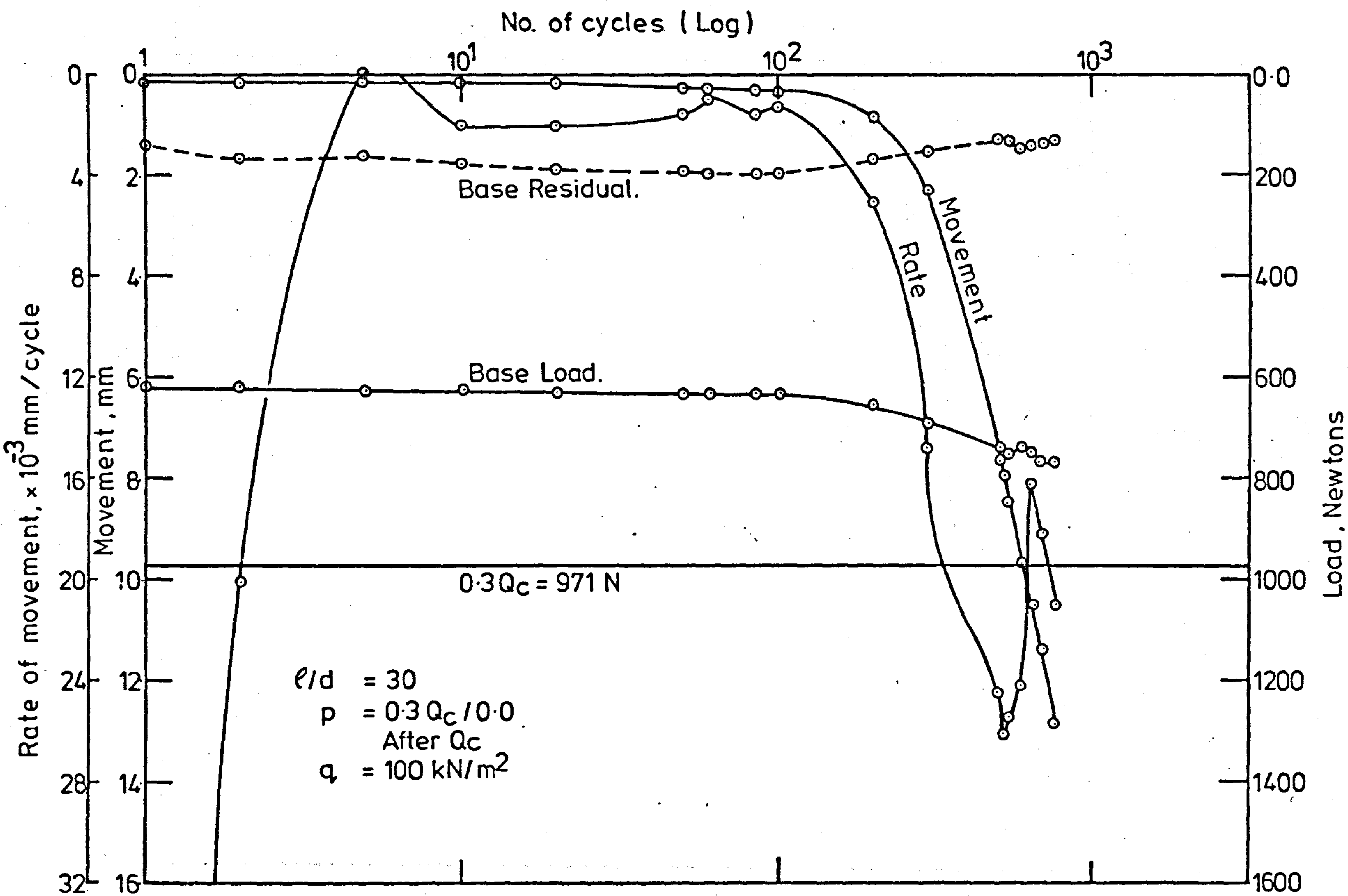


FIG. 6-15 PILE LOADS, MOVEMENTS AND RATE OF MOVEMENT AGAINST LOG CYCLES. ($\ell/d = 30$)

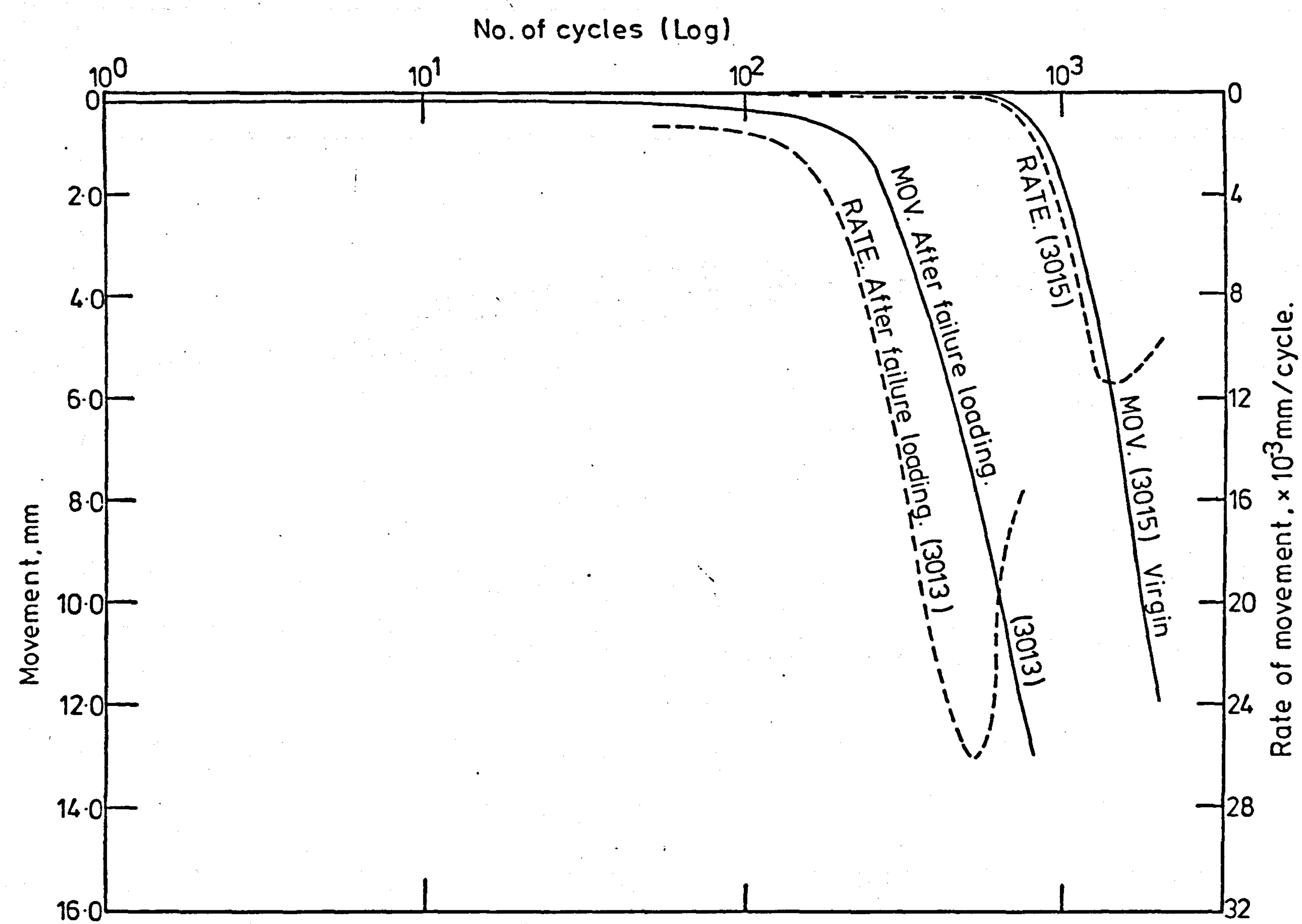


FIG. 6-16 EFFECT OF STATIC FAILURE LOADING ON THE SUBSEQUENT BEHAVIOUR UNDER REPEATED LOADING. ($\ell/d = 30$)

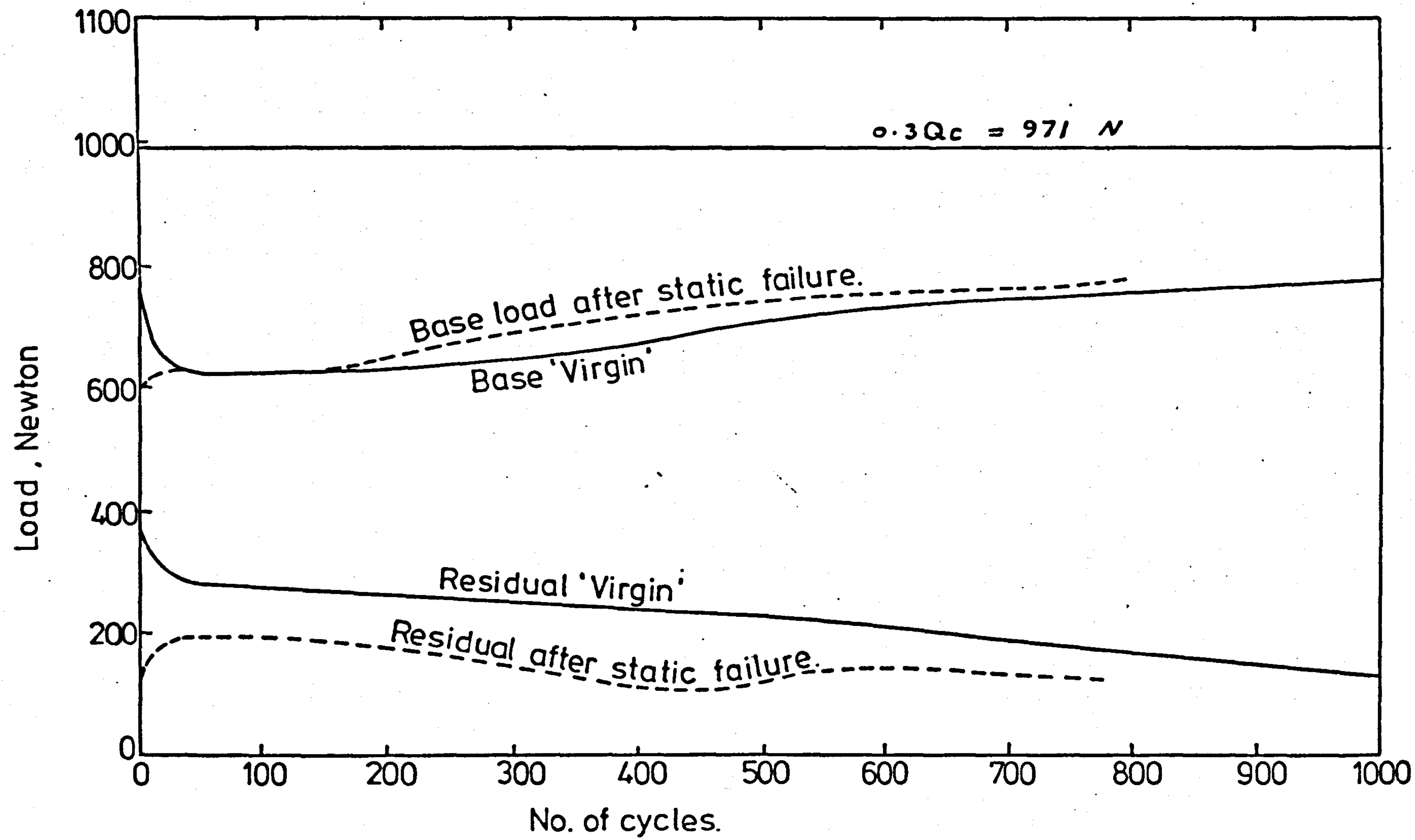


FIG. 6-17. EFFECT OF STATIC FAILURE LOADING ON THE LOAD-TRANSFER OF THE PILE DURING SUBSEQUENT REPEATED LOADING. ($l/d = 30$)

load level increased from 0.3Qc to 0.5Qc. At 0.7Qc the rate of movement increased continuously and the pile did not last more than 80 cycles. A similar behaviour was observed when the load level on a pile of 20 diameters depth was increased from 0.3 Qc to 0.5Qc as shown in Fig. 6.19.

The movements and the rates of movement of Test 19 together with those of Test 14 are shown in Fig. 6.20. Although the load amplitude was the same for both tests the pile tested under 0.5Qc/0.2Qc repeated loads experienced a longer life-span than that subjected to 0.3Qc/0.0 repeated load. The reason behind this behaviour may be related to the state of residual stresses. In test 14 the skin friction alternating in sign from positive (upward) during the application of the upper repeated load to negative during the lower repeated load, 0.0, whereas it was always positive in Test 19. Since alternating stresses severely affected the life-span of the pile, Chan (1976), then the movements which resulted from 0.5Qc/0.2Qc loading should be smaller than those of o.3Qc/0.0 repeated loads. A similar conclusion was reported by Al-Ashou (1981) when the results of 0.7Qt/0.2Qt loading were compared with those of 0.5Qt/0.0 repeated load.

The influence of 0.5Qc/0.2Qc repeated loading on the static load displacement response and on the load transfer characteristics of the pile are clearly demonstrated in Fig. 6.21. The results of this test confirmed the conclusion stated in section 6.3.2. that both the ultimate shaft friction and the ultimate base resistance decreased after the pile was tested under repeated loads. The load-displacement relationship became stiffer as compared with that of a virgin pile.

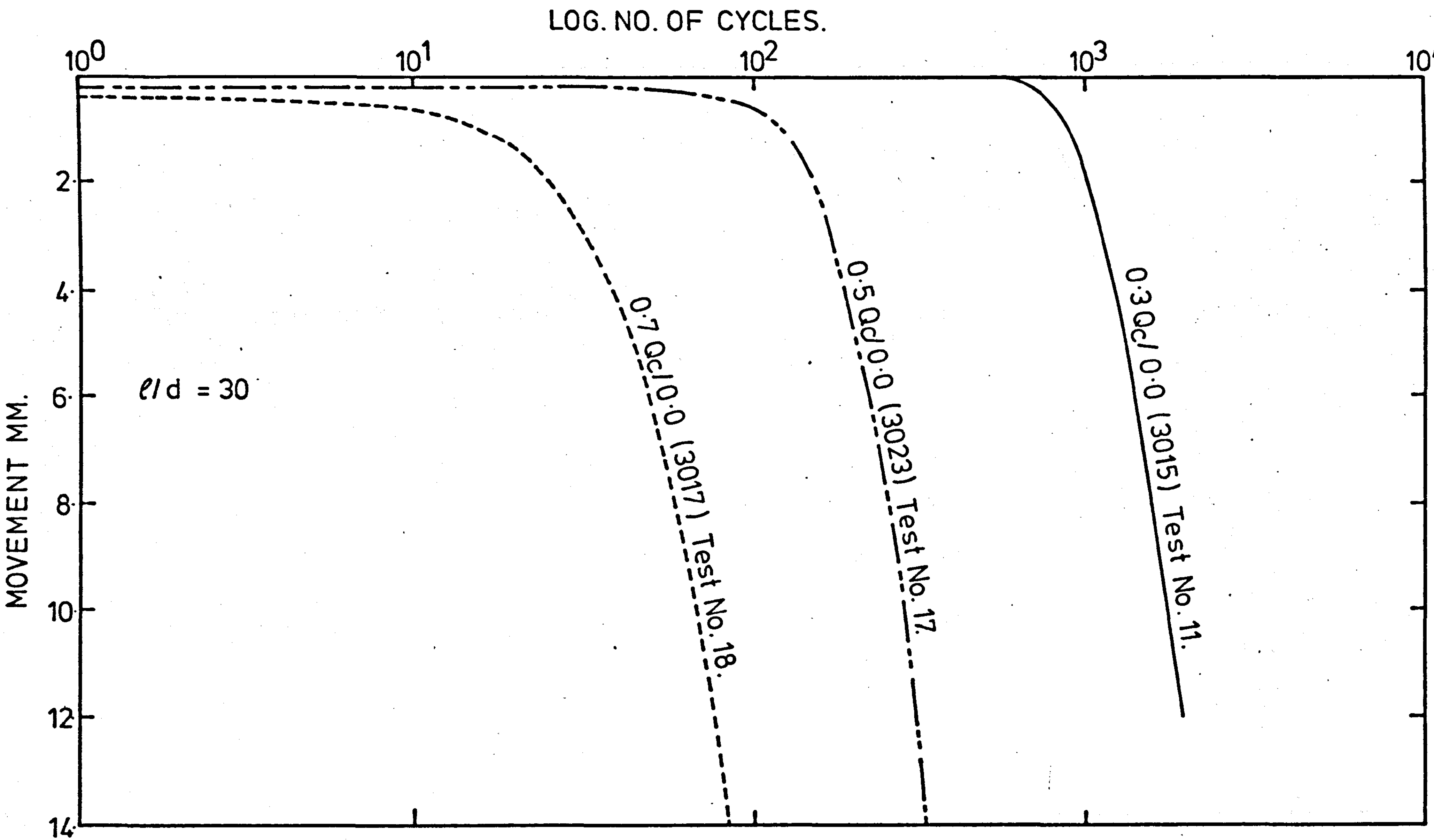


FIG. (6·18). MOVEMENT AGAINST LOG. NO. OF LOAD CYCLES. ($\ell/d = 30$)

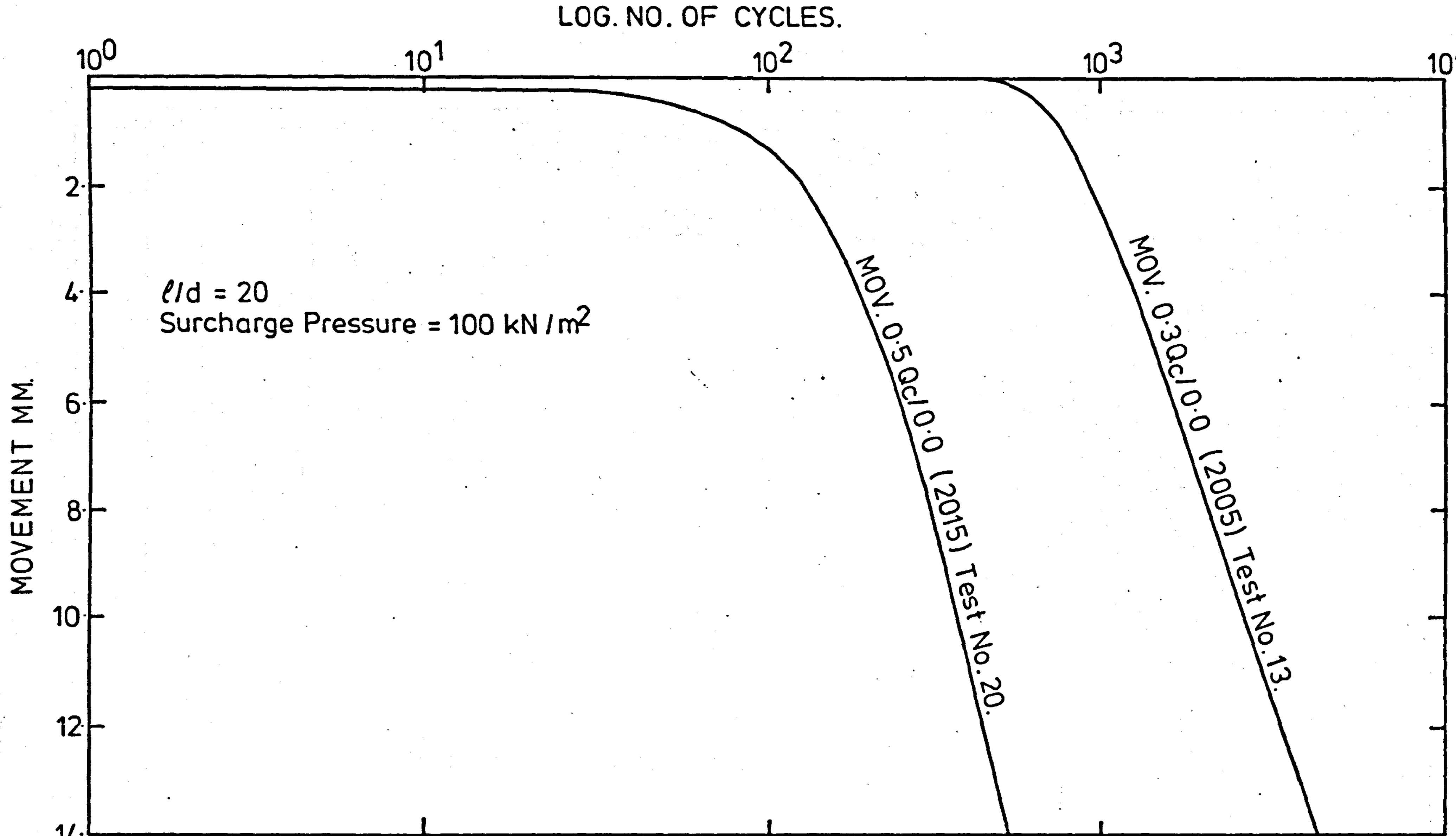


FIG. (6·19). MOVEMENT AGAINST LOG. NO. OF LOAD CYCLES. ($\ell/d = 20$)

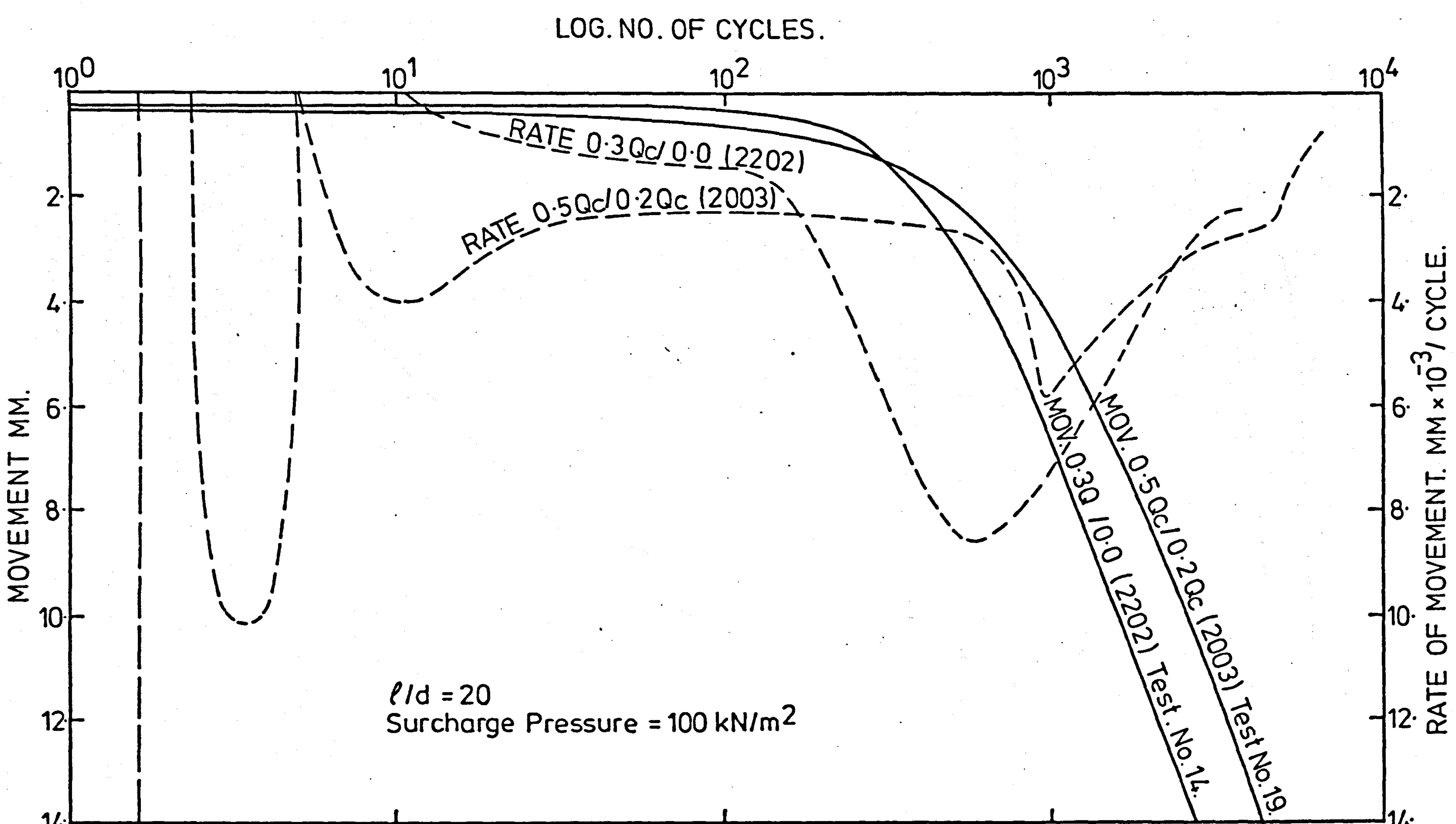


FIG. (6·20). MOVEMENT AND RATE OF MOVEMENT AGAINST LOG. NO. OF LOAD CYCLES. (ℓ/d = 20)

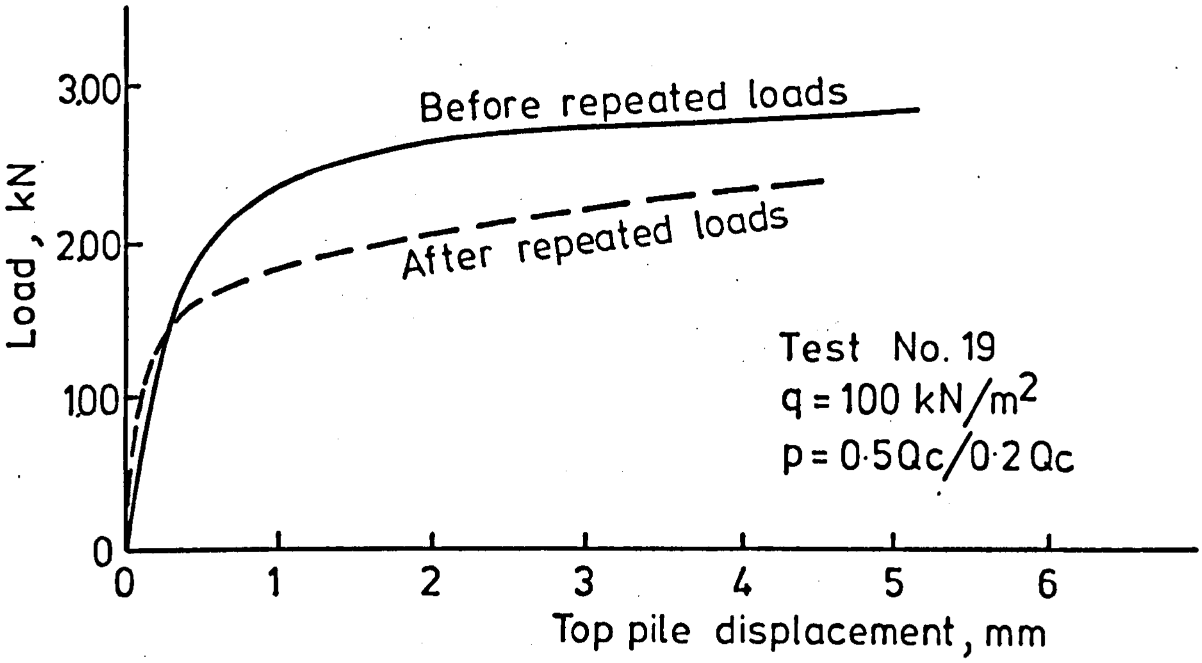


FIG. 6-21a. LOAD - DISPLACEMENT BEHAVIOUR ($l/d = 20$)

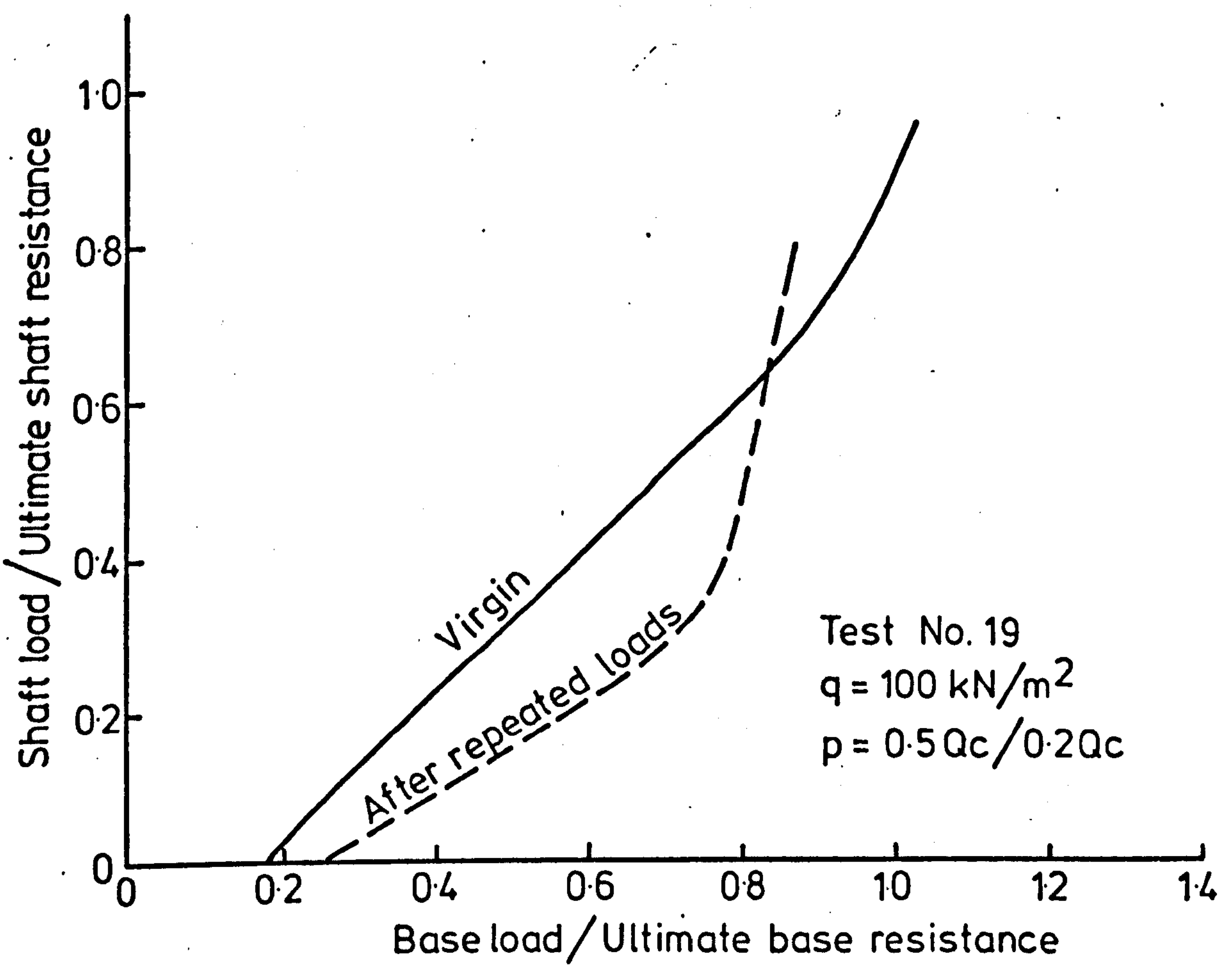


FIG. 6-21b. LOAD TRANSFER DURING STATIC LOAD TESTS ($l/d = 20$)

6.4 Series III: Tensile repeated loadings

This section deals with results of test carried out on piles embedded at various depths and subjected to repeated tensile loads. The influence of previous static failure loading on the subsequent behaviour of piles under repeated loads is also examined.

The results of Test 21 which were performed on a pile of 30 diameters depth and subjected to 0.3Qt/0.0 loading are presented in Fig. 6.22. The curve of the movement shows that initially the movement increased at a decreasing rate. After a stable stage, which was between the cycle number 200 and 40,000, a drastic change in the pile behaviour took place. The movement began to increase at a rate which increased very rapidly as the number of load cycles was increased. It was found that in none of the tensile repeated loading tests the pile was entirely pulled-out. However, even at large movements the rate of movement still had a finite value.

The changes in the pile base loads with the number of load cycles are also shown in Fig. 6.22. During the initial stage of the repeated load, and because the pile base was subjected to a compressive load (due to penetration) the shaft load was greater than the applied repeated load. As the pile load was repeating the compressive base load decreased gradually and became tensile after 1900 cycles. This tensile load then increased and reached a peak value of 80 newtons around the 40,000th cycle. Beyond that it decreased as the pile progressively pulled-out. As stated earlier although the true base load cannot have negative values in a dry sand the tensile load recorded was due to the fact that the base load cell was one pile diameter above the base. The trend of the residual base load was similar to that of the base load as shown in the same figure.

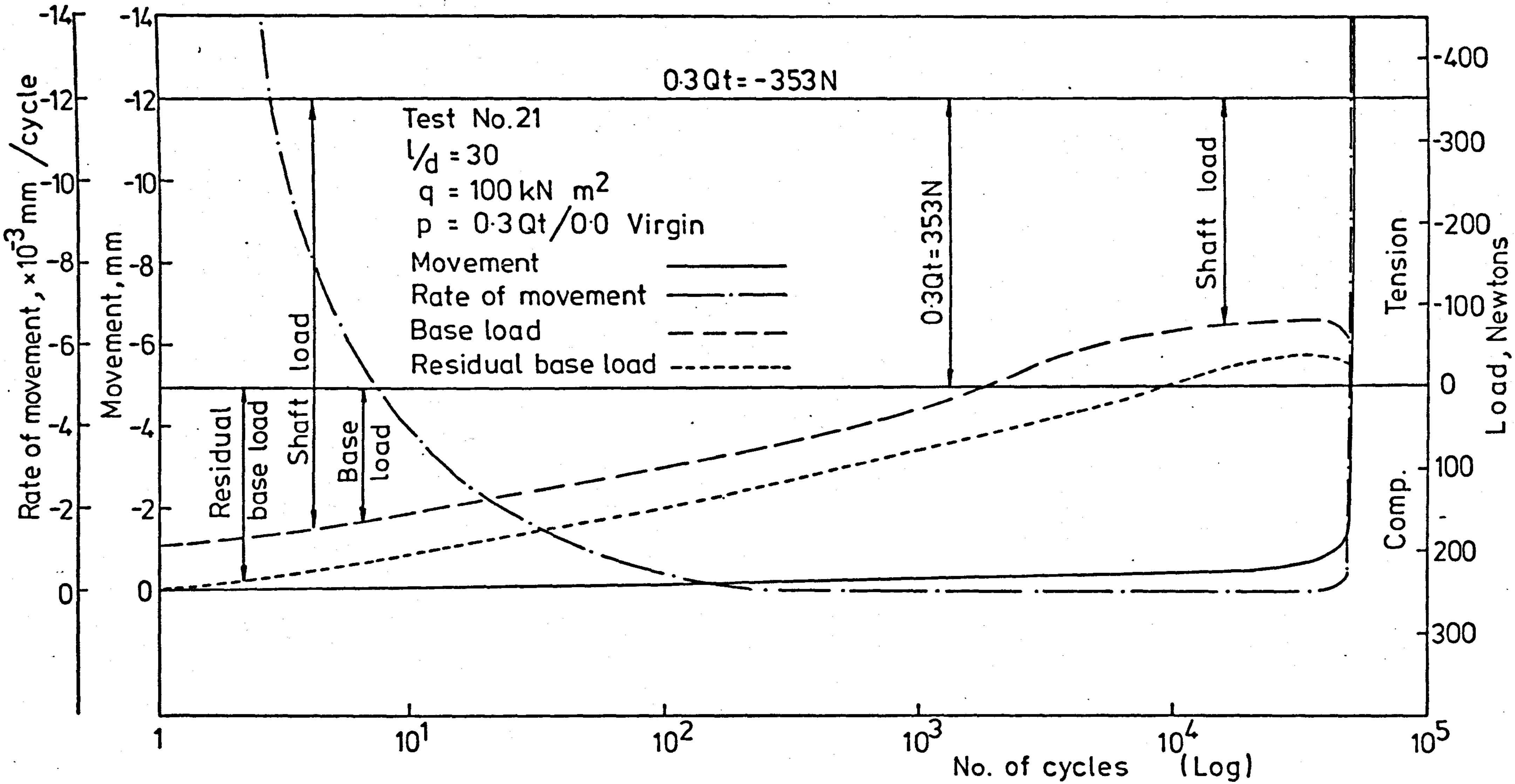


FIG. 6-22 VARIATION OF LOADS, MOVEMENTS AND RATES OF MOVEMENT AGAINST No. OF CYCLES

The distribution of axial pile loads and the corresponding skin friction durind the loading and unloading phases of selected cycles of Test 21 are shown in Fig. 6.23. These cycles were the first, a cycle during the stable stage, and the last cycle in the test. Before testing, the pile was subjected to compressive residual loads which developed after the process of penetration, Fig. 6.23A. These loads increased as the depth of embedment was increased. During the first cycle the compressive residual load along the upper portion of the pile became tensile while that of the lower portion remained compressive but decreased from 450 to 194 newtons. The corresponding skin friction revealed an increase at all points located along the pile shaft, Fig. 6.23B. Due to repeated tensile loading and up to cycle number 835 the compressive load of the lower portion was decreased progressively. The skin friction of the upper half of the pile decreased whereas it increased along the lower half during this period. The corresponding residual skin friction, Fig. 6.23D showed a continuous decrease. This decrease was greater along the upper half and it became even of positive sign. During the loading phase of the last cycle, number 50275, the skin friction along the upper part of the pile was not altered while it decreased progressively along the lower part. The residual skin friction during this cycle experienced the same trend.

The results of Test 23, in which the pile was at 20 diameters depth and subjected to 0.3Qt/0.0 repeated load, are illustrated in Fig. 6.24. When these results are compared with those of Test 21, Fig. 6.22, it will be seen that the general feature of the pile behaviour did not alter when the depth of embedment decreased from

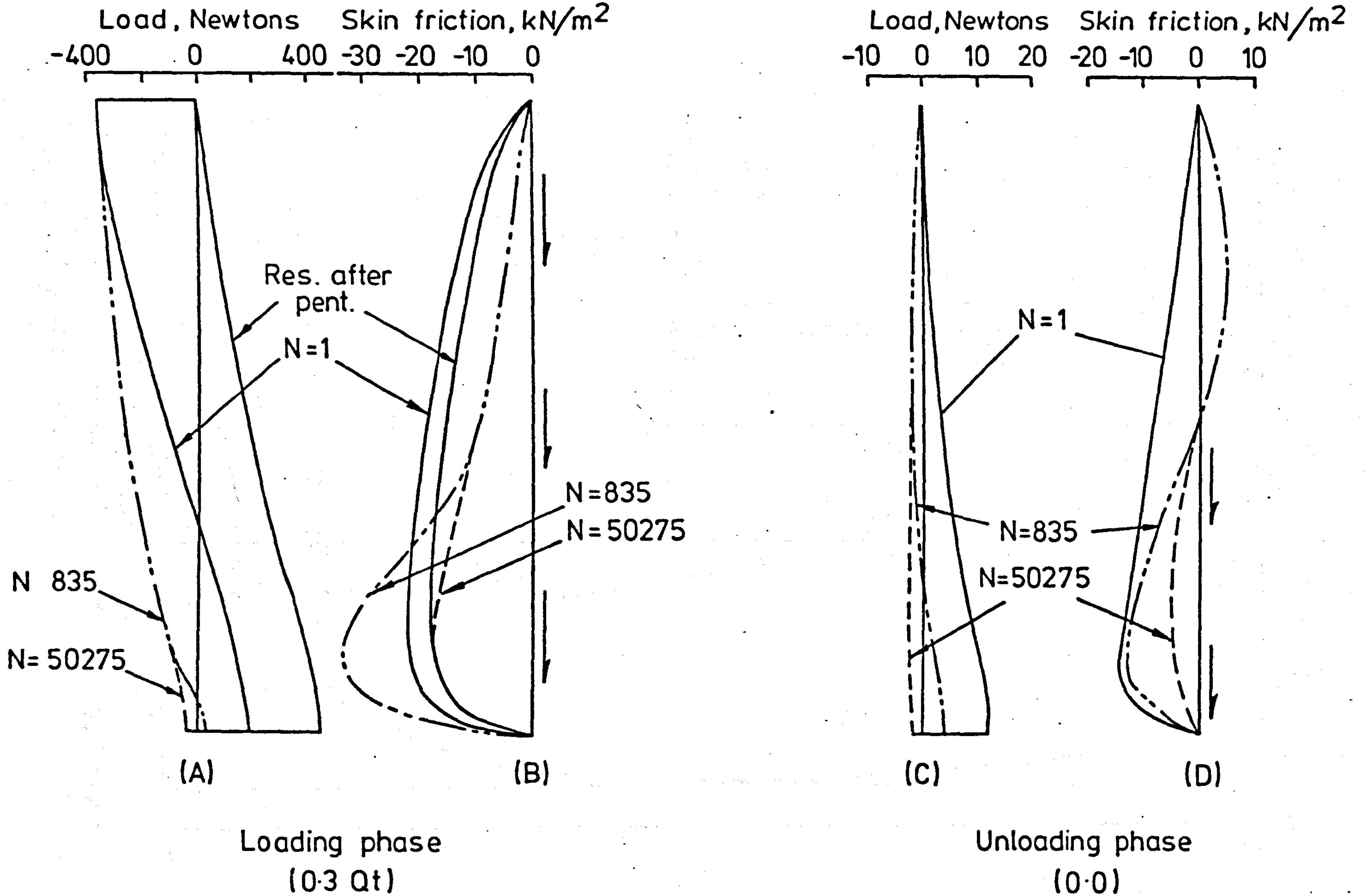


FIG. (6-23) DISTRIBUTION OF AXIAL LOAD AND SKIN FRICTION DURING SELECTED CYCLES OF TEST 21

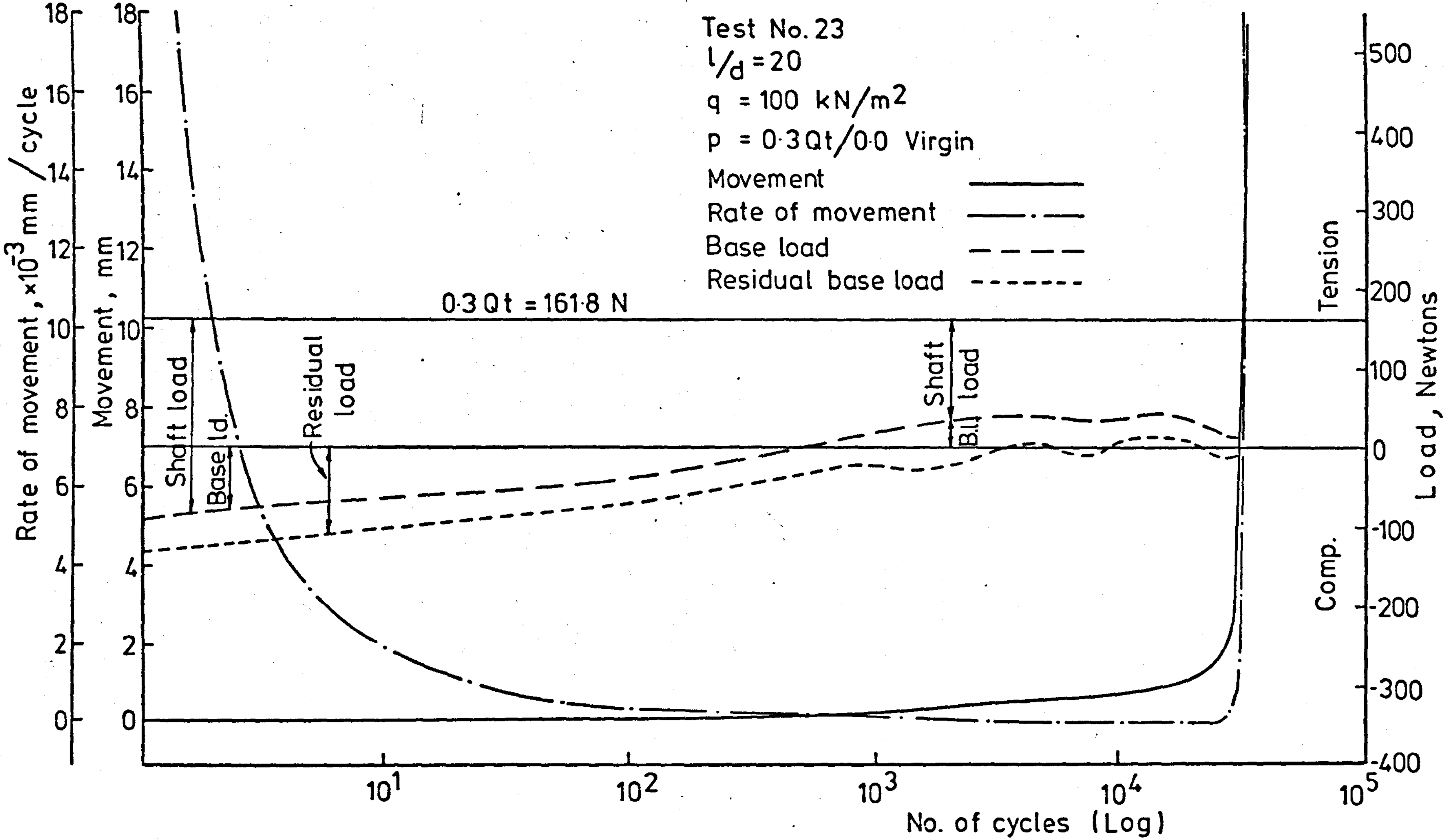


FIG. 6-24 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT AGAINST No. OF CYCLES

30 to 20 diameters. It may also be concluded that the pile life-span increased when the depth of embedment increased. Moreoever, the number of cycles at which the pile base experienced zero load was increased from 500 to 1800 when the pile depth increased from 20 to 30 diameters.

As concluded in section 6.3.2. for compressive piles, the life-spans of tension piles were also reduced when the piles were previously subjected to failure static loadings before being tested under repeated loads. For piles of 30 diameters depth it was reduced from about 50000 cycles, Test 21, Fig. 6.22, to about 8000 cycles in Test 22, Fig. 6.25. A similar behaviour was observed at 20 diameters depth and the pile life-span was reduced from about 30000 cycles, Test 23, Fig. 6.24, to about 6000 cycles, Test 24, Fig. 6.25.

Fig. 6.26 shows the results of Test 26 in which the pile was subjected to 0.5Qt/0.0 repeated load and embedded at 20 diameters depth. When these results are compared with those of Test 23, Fig. 6.24, in which the pile was subjected to 0.3Qt/0.0 loading, it will be found that the pile life-span was reduced more than 16 fold due to this increase in the repeated load level. The pile base load, Test 26, reached a peak tensile value of 80 newtons before dropping to 20 newtons at the end of the test. It was also observed that the decrease in the base tensile load from its peak value coincided with an increase in the rate of movement.

In contrast to that observed in compression tests, when the repeated tensile load levels were increased from 0.3Qt/0.0 to 0.5Qt/0.2Qt Test 27, the pile life-span decreased from about 30000 cycles, Fig. 6.23 to about 18000 cycles as shown in Fig. 6.27. This change in

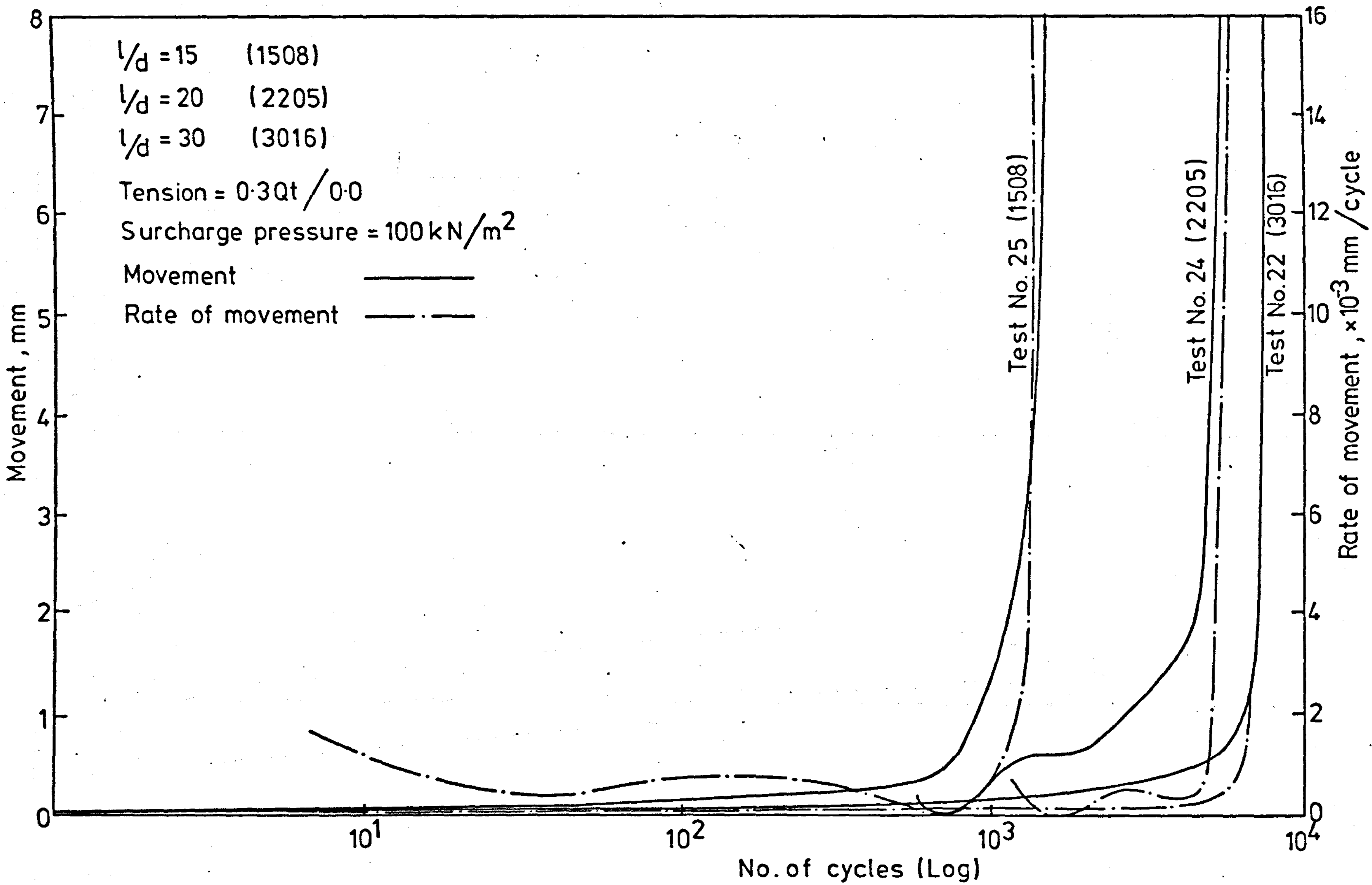


FIG.6-25 INFLUENCE OF EMBEDMENT DEPTH ON THE LIFE SPAN OF TENSION PILES

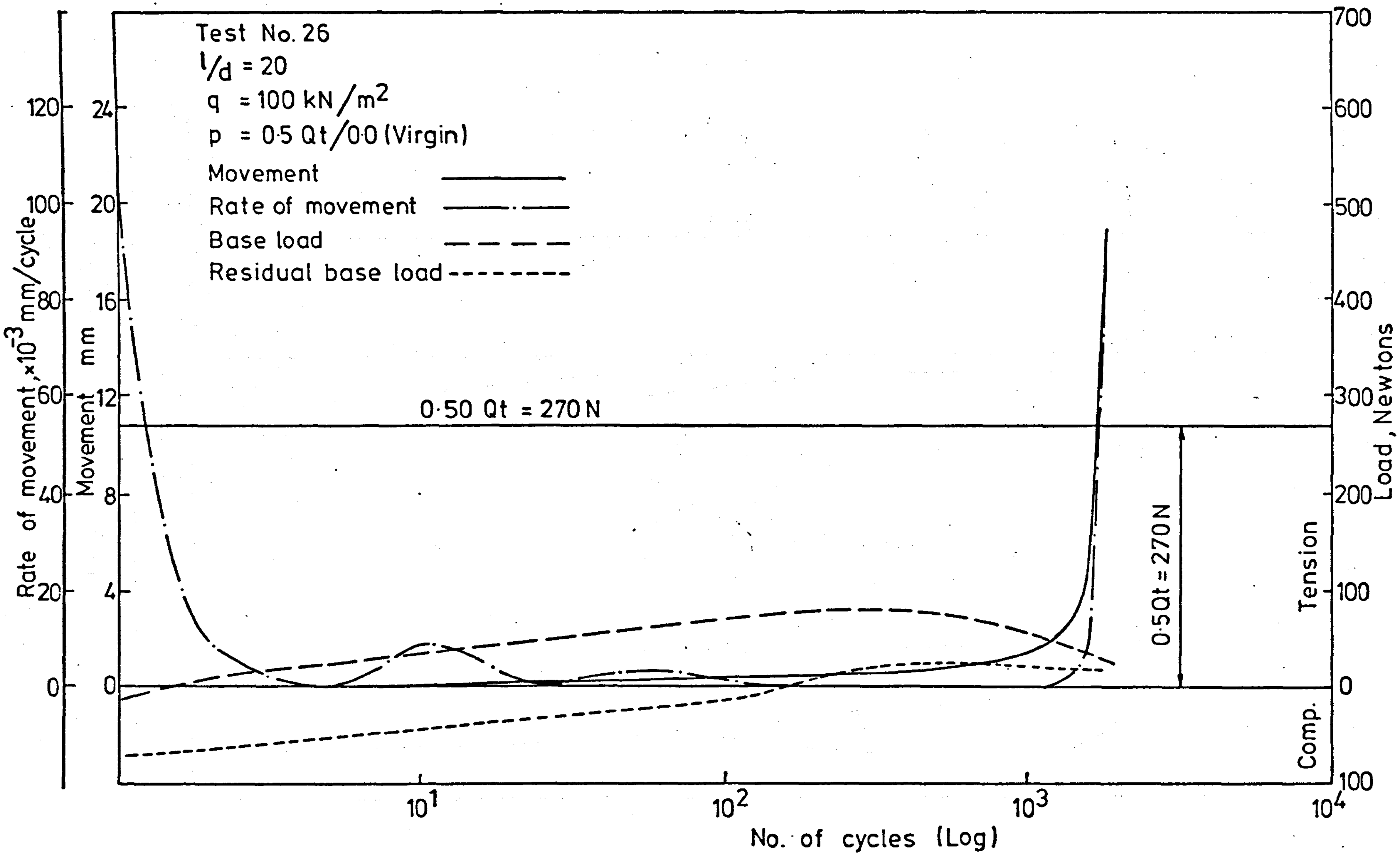


FIG. 6-26 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT AGAINST No. OF CYCLES

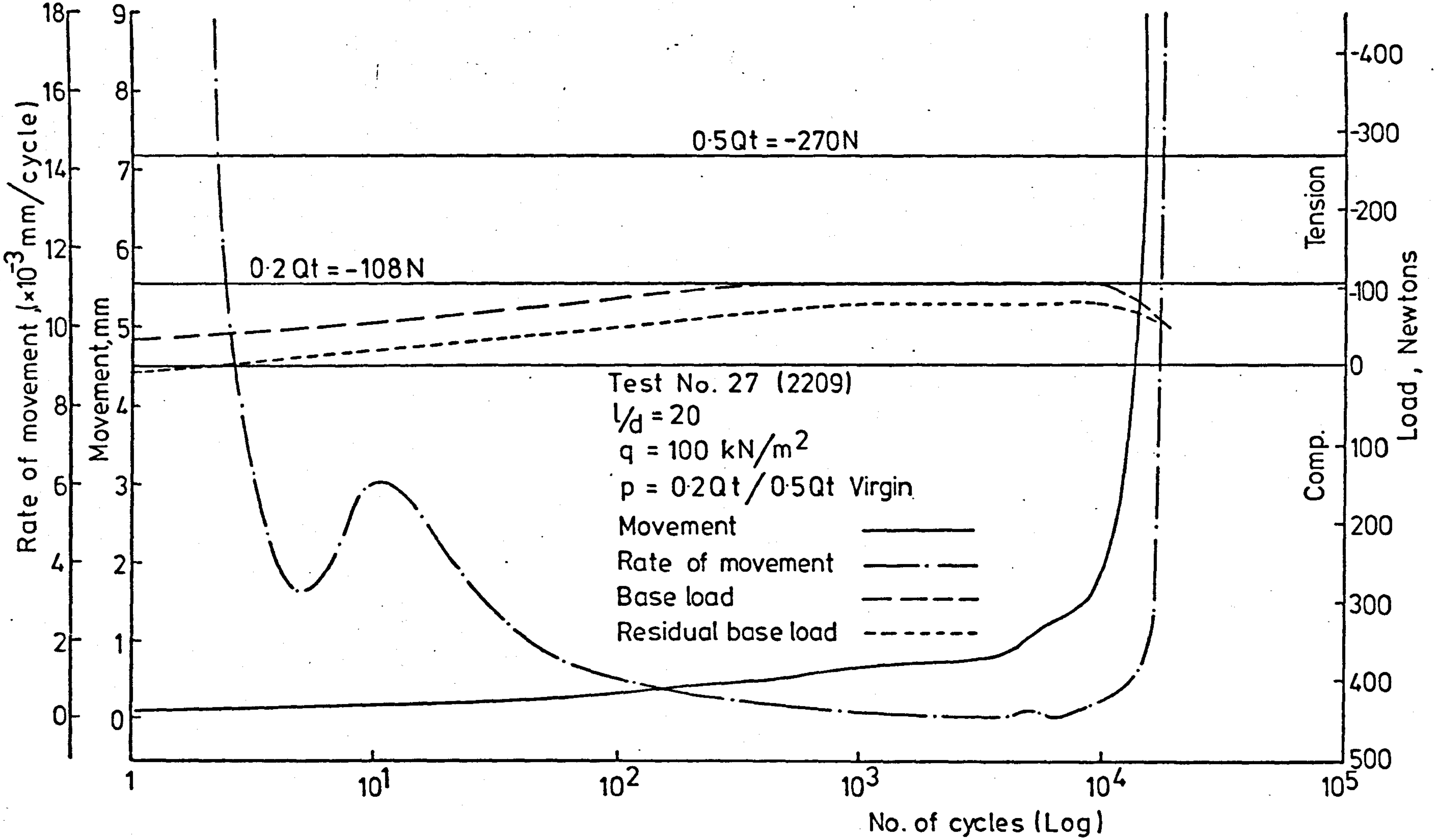


FIG. 6-27 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT AGAINST No. OF CYCLES

behaviour may be related to the state of the residual stresses. After penetration the direction of the residual stresses was downward and remained so during the repeated loading. Therefore the pile life-span will be controlled by the load level, Chan (1976). Measurements of the rates of movement during the failure stage, that is from the cycle number 4000 to the end of Test 27, are presented in Fig. 6.28. It is clear that the rate did not increase regularly but fluctuated in value as the number of cycles was increased.

The influence of repeated tensile loads on the static load-displacement behaviour of the pile is shown in Fig. 6.29. It can be concluded that repeated loading decreased the ultimate pulling resistance of the pile. The reduction is greater in the case of tension piles than in the case of compression piles. It was greater than 50% for the former piles whereas it seldom reached 30% for the latter piles.

6.5 Series lV: Effect of surcharge pressure

From the results of two identical tests performed on piles subjected to repeated compression loads of 0.3Qc/0.0 and placed in a sand acted upon by a constant surcharge pressure of either 0.0 or 100 kN/m^2 Chan (1976) concluded that the effect of increasing the surcharge pressure was to delay the onset of the critical stage of maximum rate of movements, but the main features of pile behaviour were not significantly altered. Based on repeated tension load tests conducted on reinforcement bars embedded in a sand upon which a surcharge pressure not more than 100 kN/m^2 acted, Al-Ashou indicated that the surcharge pressure caused an increase in the life-span of the reinforcement but, otherwise it had little or no influence on the

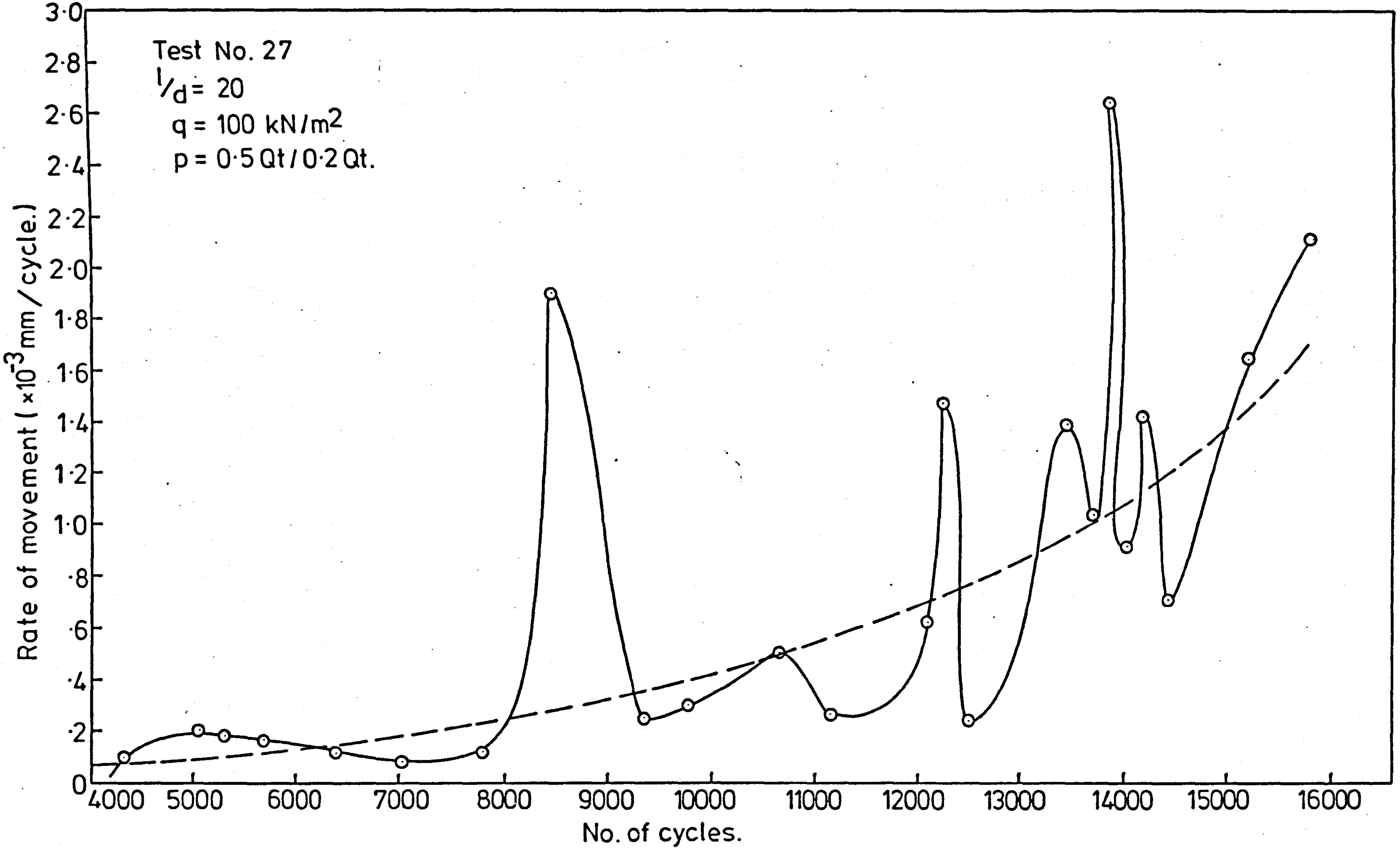


FIG. 6-28 VARIATION OF THE RATE OF MOVEMENT WITH THE NUMBER OF LOAD CYCLES.

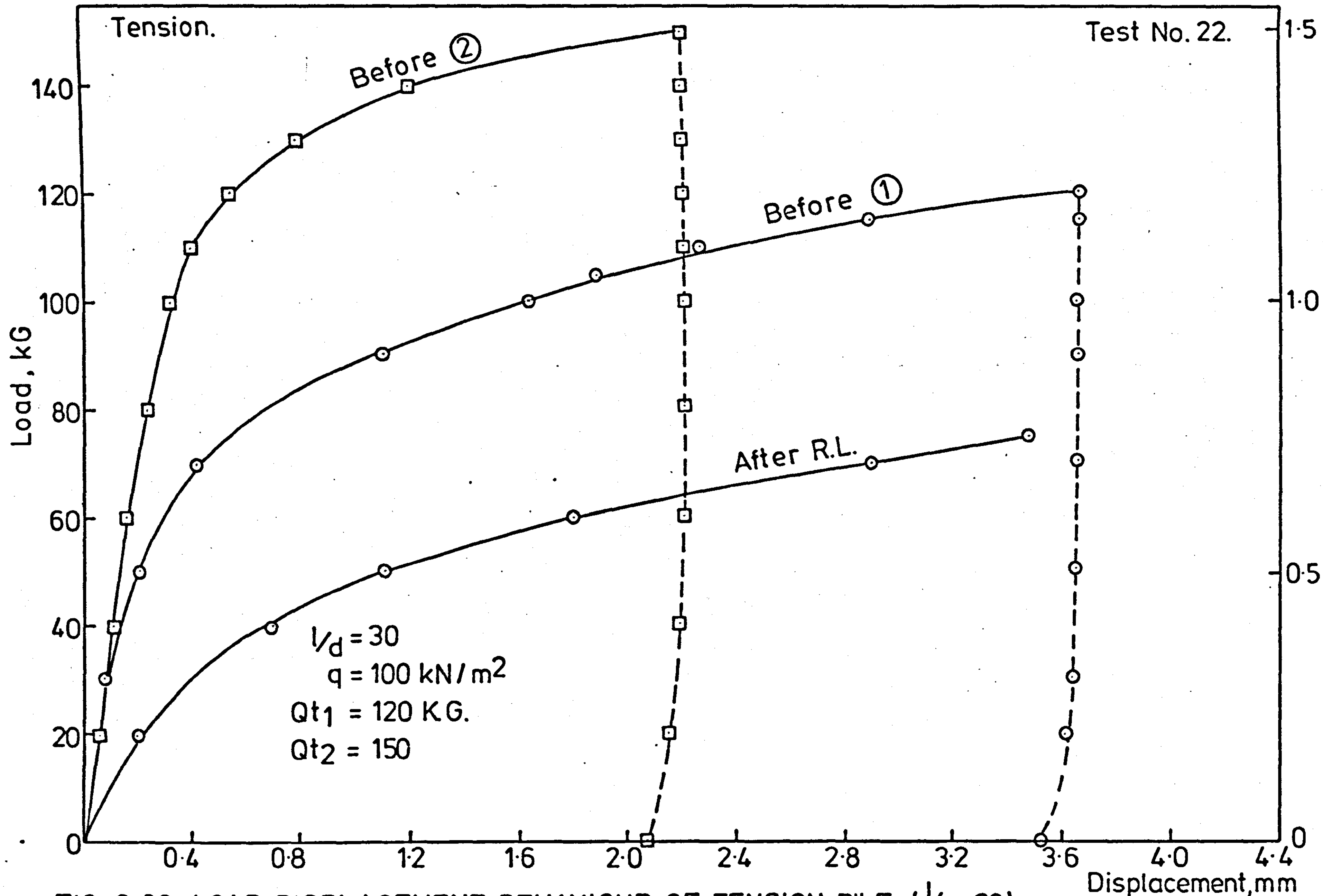


FIG. 6-29. LOAD DISPLACEMENT BEHAVIOUR OF TENSION PILE. ($l/d = 30$)

behaviour of the reinforcement. In view of these studies it was decided to examine the behaviour of both compression and tension piles embedded at shallower depth, with a surcharge pressure which ranged from 0.0 to 200 kN/m^2.

Fig. 6.30 shows the movements and the rates of movement of Tests 14, 28 and 29 versus logarithm of the number of load cycles for piles subjected to repeated loads of 0.3Qc/0.0 at 0.0, 100 and 200 kN/m^2 surcharge pressure respectively. The number of cycles at which the rate of movement reached its maximum value increased from 600 to 850 cycles when the pressure was increased from 100 to 200 kN/m^2 and it was only 200 cycles at 0.0 kN/m^2 pressure. The number of cycles at which the pile moved 6.0mm also increased when the surcharged pressure increased. It was 300, 900 and 1800 cycles at 0.0, 100 and 200 kN/m^2 pressure respectively. A similar conclusion can be drawn from Fig. 6.31 which shows the results of Tests 24, 30 and 31 where the pile was subjected to tension repeated loads of 0.3Qt/0.0 with 0.0, 100 and 200 kN/m^2 surcharge pressure respectively. The pile life-span increased from 5800 to 36000 cycles when the surcharge pressure was increased from 100 to 200 kN/m^2. At 0.0 kN/m^2 pressure the pile did not last more than 50 cycles.

Fig. 6.32 shows that the ultimate bearing capacity of the pile also decreased after having been tested under repeated loads with both 0.0 and 200 kN/m^2 surcharge pressure.

6.6 Series V: Varying load amplitudes

A foundation pile is not always subjected to a repeated load cycle between two fixed limits. In many cases these limits vary with time. In order to assess the influence of such kinds of

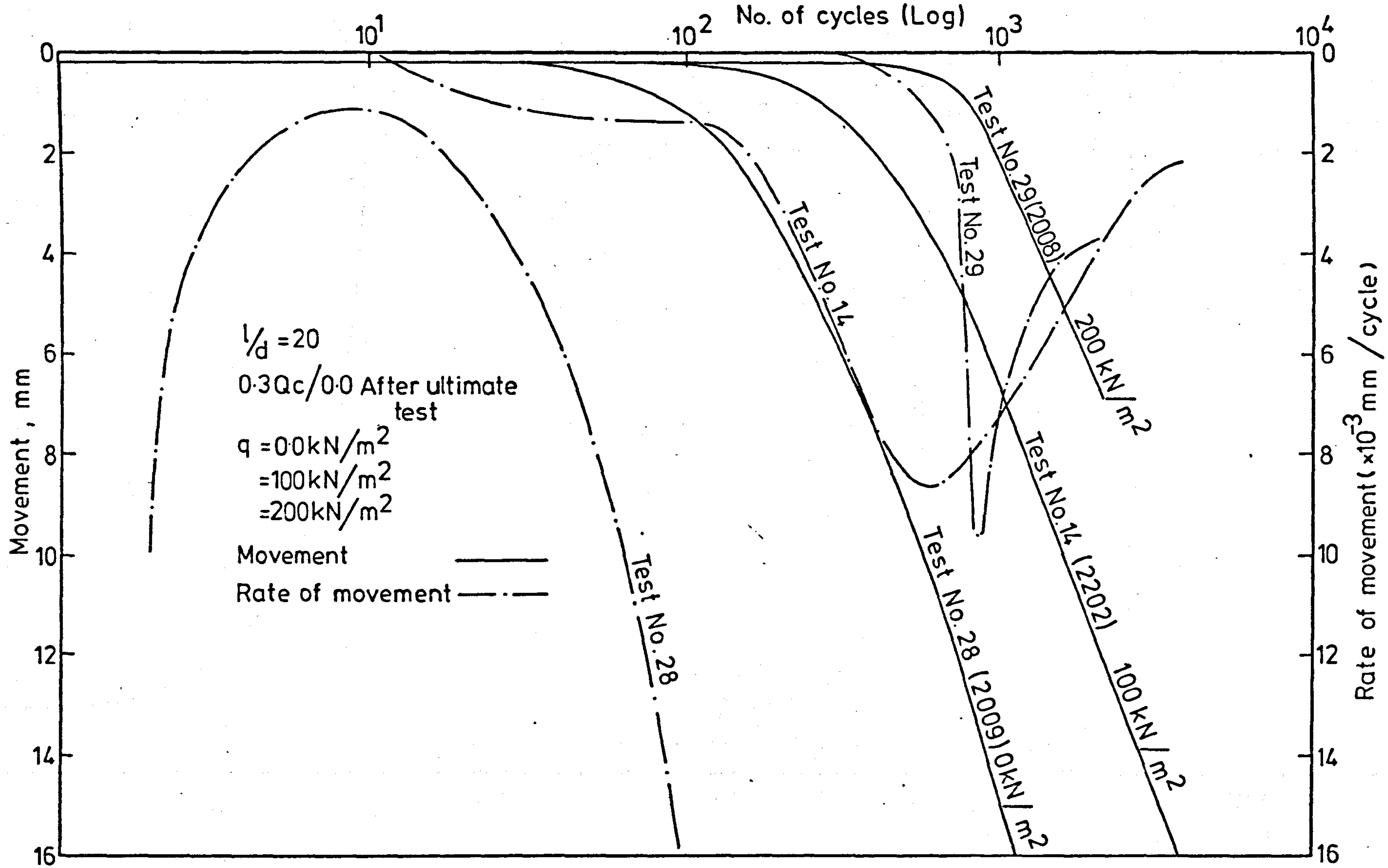


FIG. 6-30 INFLUENCE OF SURCHARGE ON THE BEHAVIOUR OF PILE SUBJECTED TO REPEATED COMP. LOADS

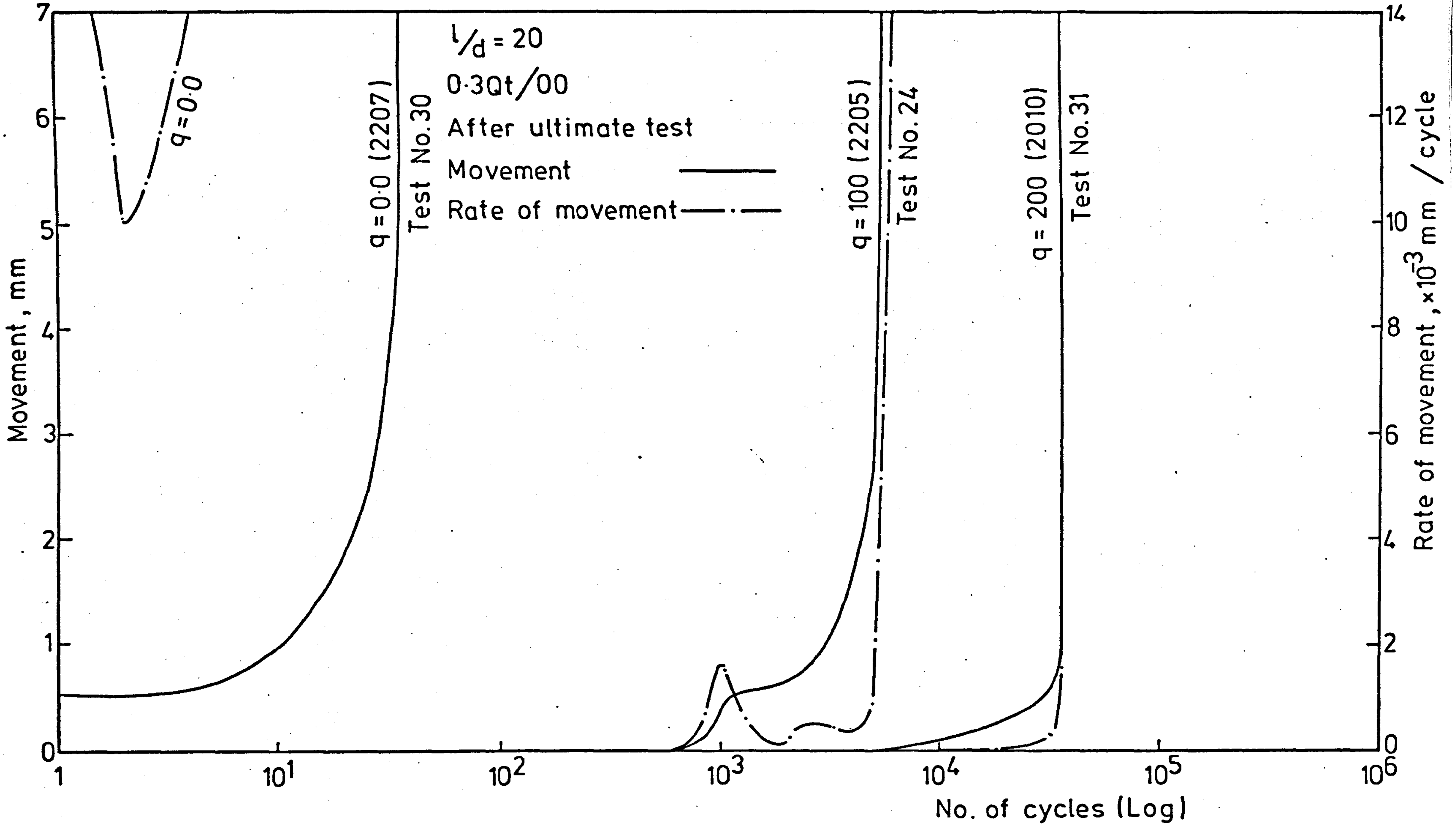


FIG. 6-31 INFLUENCE OF SURCHARGE PRESSURE ON THE LIFE-SPAN OF TENSION PILES

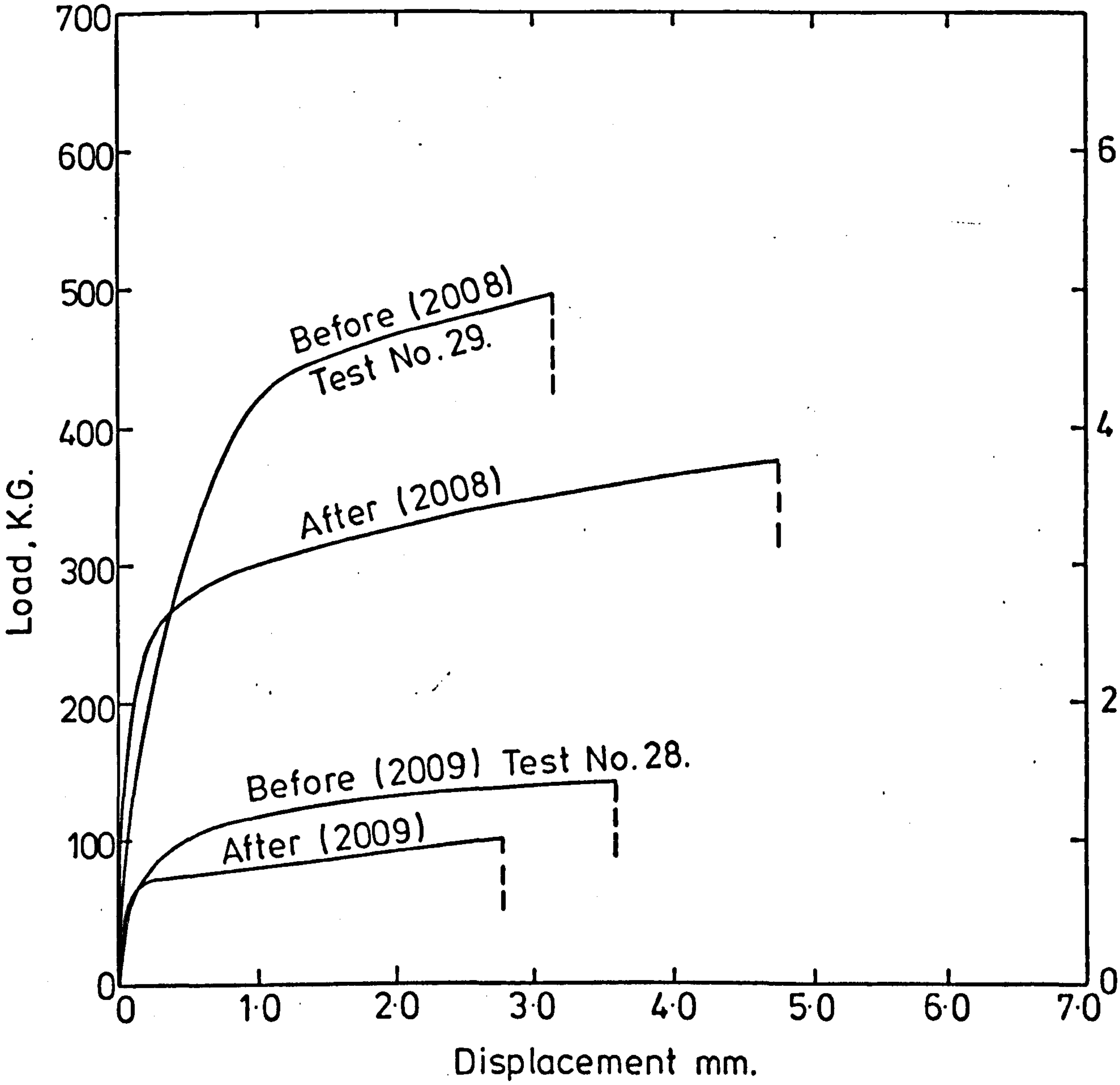


FIG. 6-32 LOAD - DISPLACEMENT RELATIONSHIP. (l/d = 20)

loading on the behaviour of piles four main tests were conducted within this series of tests. The influence of load amplitude, subsequently decreased or increased on the behaviour of piles placed at 30 pile diameters has been investigated in Test 32 (3017) and Test 33 (3018) respectively. The sequence employed in the first test was as follows:-

(32 - a) 0.7Qc/0.0

(32 - b) 0.5Qc/0.0

(32 - c) 0.3Qc/0.0

In the second test the reverse sequence was applied to the pile, that is :-

(33 - a) 0.3Qc/0.0

(33 - b) 0.5Qc/0.0

(33 - c) 0.7Qc/0.0

During each of these tests the surcharge pressure was maintained at 100 kN/m^2. Each individual test was stopped when the rate of movement began to decrease after having reached a maximum value or when it decreased progressively as the number of load cycles increased.

To appreciate the effect of previous repeated loads on the subsequent behaviour of the pile subjected to repeated load, the results of Test 33-a together with those of Test 32-c are plotted in Fig. 6.33. It is clear that the pile which was being tested under higher repeated load levels experienced a smaller rate of movement and therefore had a longer life when compared with the virgin pile. The amount of the maximum rate of movement decreased from 0.0115mm/cycle for the virgin pile to 0.0021mm/cycle for the previously loaded pile. After 2000 cycles the displacement of the virgin pile was four times that obtained from the other pile.

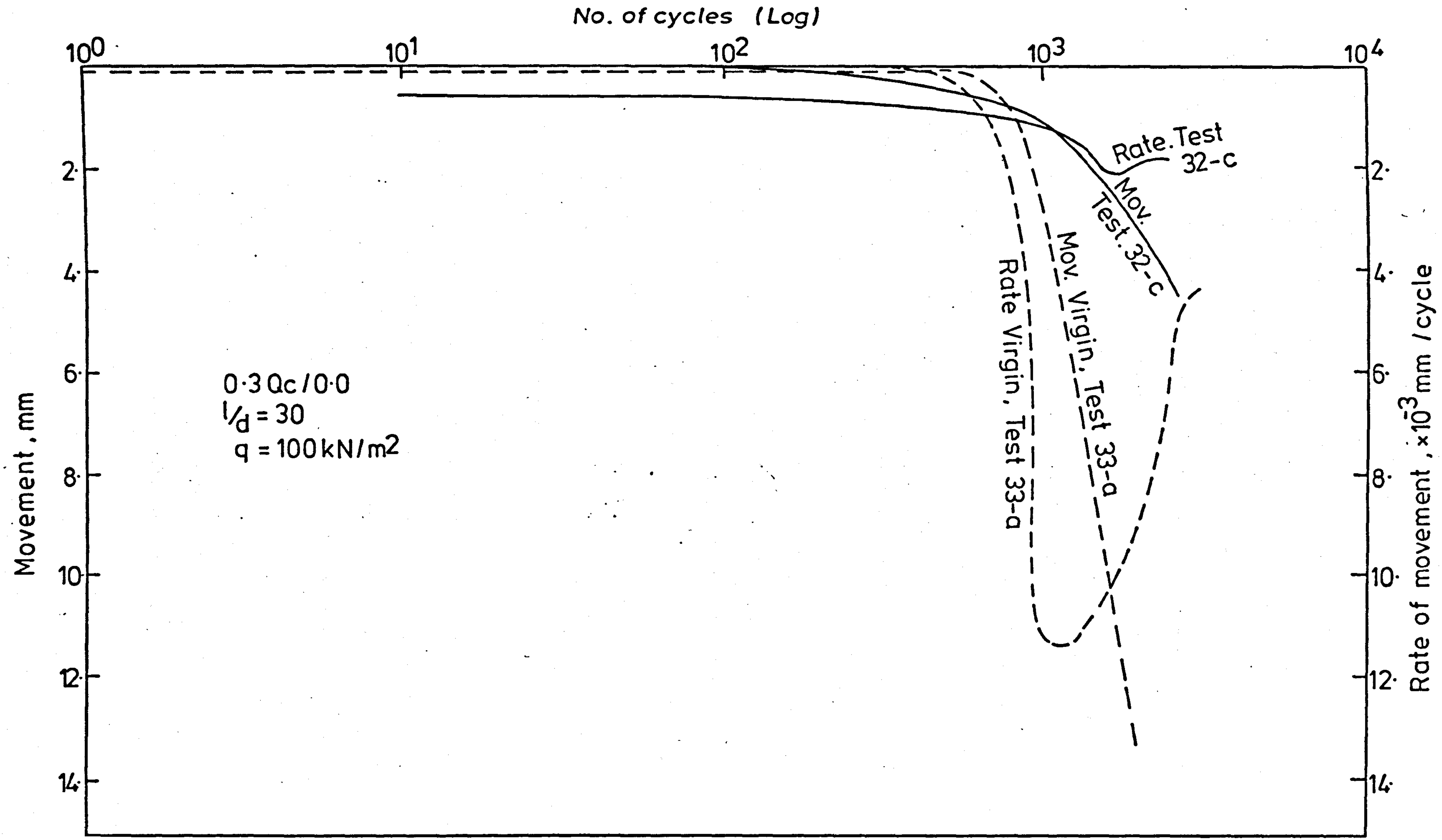


FIG. 6-33. VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH LOG. NO. OF CYCLES.

This change in behaviour is probably due to the stiffening and densification of the sand beneath the pile base caused by the previous repeated loading. When the movements and the rates of movement of Test 32-a and Test 33-c in which the repeated load level was 0.7Qc are compared as shown in Fig. 6.34 the reverse trend will be seen. The previously loaded pile had a shorter life-span. This behaviour is attributed to the initial movement and to the trend of the rate of movement. Because repeated loads decrease the ultimate capacity of the pile therefore the initial movement of the previously loaded pile was larger than that of the virgin pile. On the other hand, the rate of movement of the previously loaded pile being a maximum at the beginning of the test and decreased as the number of load cycles increased while it was a minimum for the virgin pile and then increased during the advancement of the test. Therefore the virgin pile experienced the smaller movement. Based on the results presented in Figs. 6.33 and 6.34 it may be concluded that the life-span of a pile previously loaded depends on the load level in the succeeding test. If it was low, 0.3Qc, the previous load resulted in a pile of longer life, whereas a shorter life was caused by a higher load level such as 0.7Qc.

A similar behaviour can be seen in Fig. 6.35 in which the movements and the rates of movement of tests 32-b and 33-b together with those of Test 20 are plotted. It may also be concluded that, the higher the previous load level the smaller will be the rate of movement of the succeeding test. The influence of the previous loading on the pile shaft load and its residual values is shown in Fig. 6.36. Both of these loads decreased due to the previous loading. The higher the previous load level the greater was the reduction.

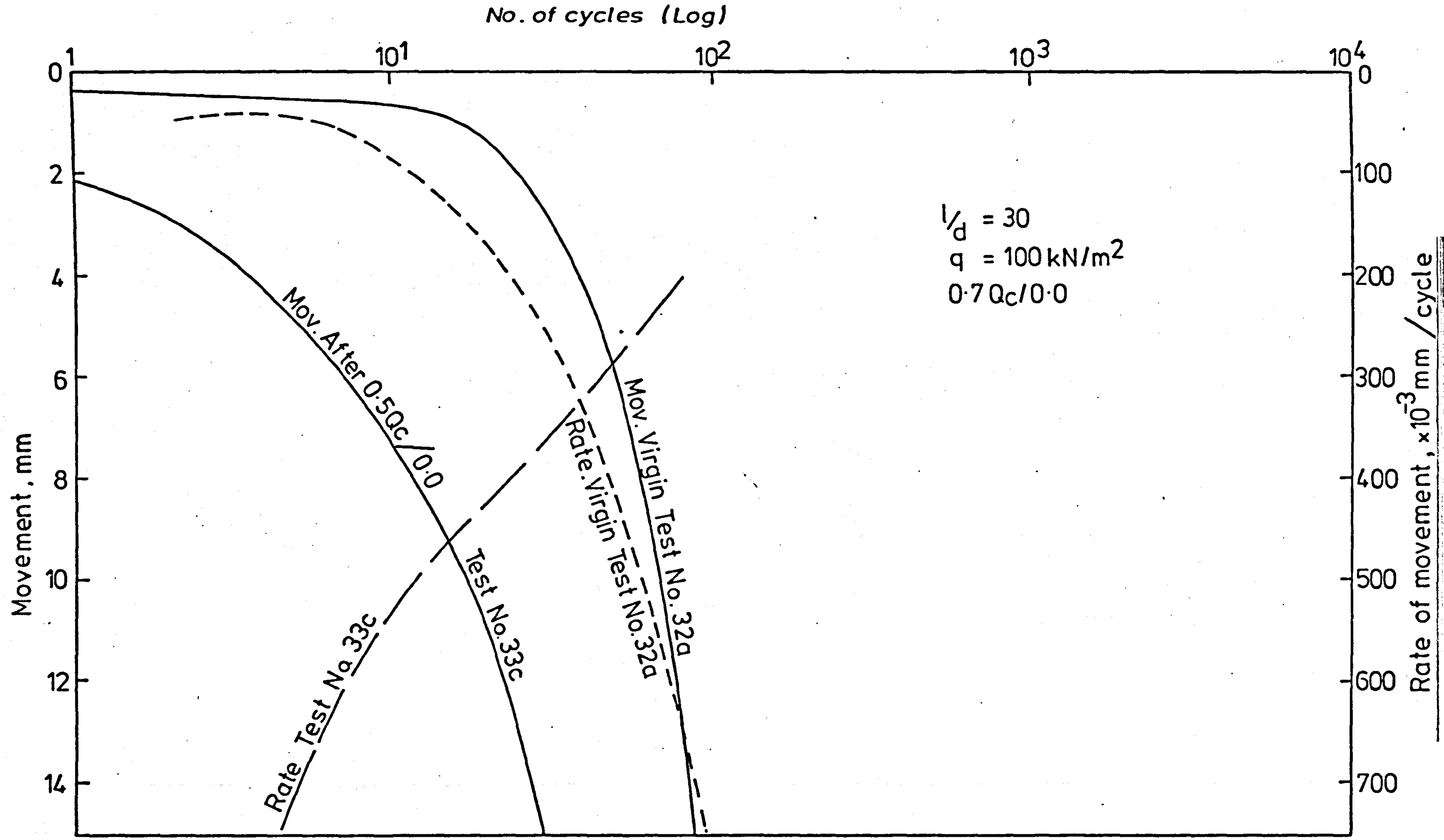


FIG. 6-34 VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH LOG NO. OF CYCLES. ($l/d = 30$)

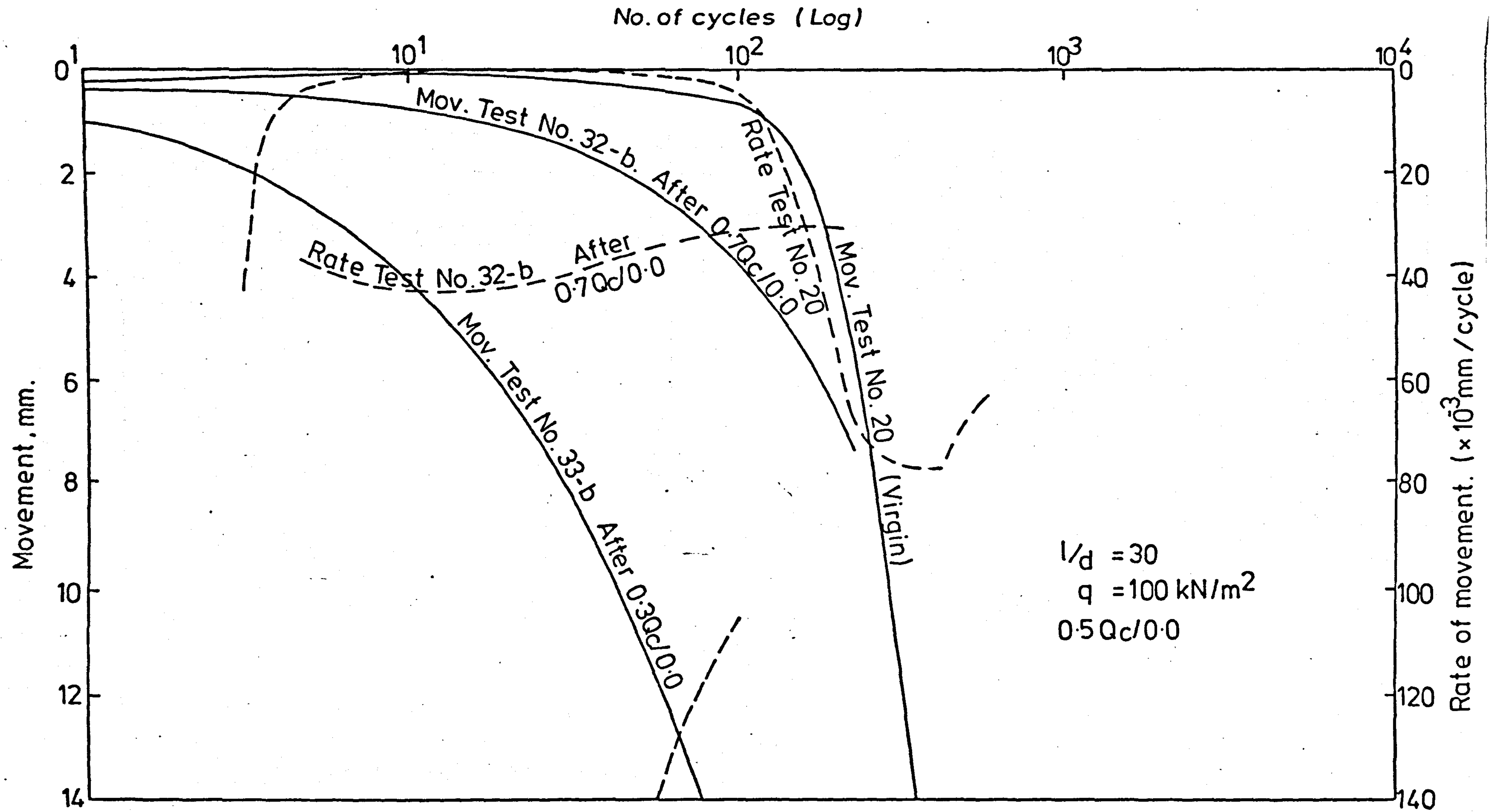


FIG. 6-35 VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH LOG NO. OF CYCLES.

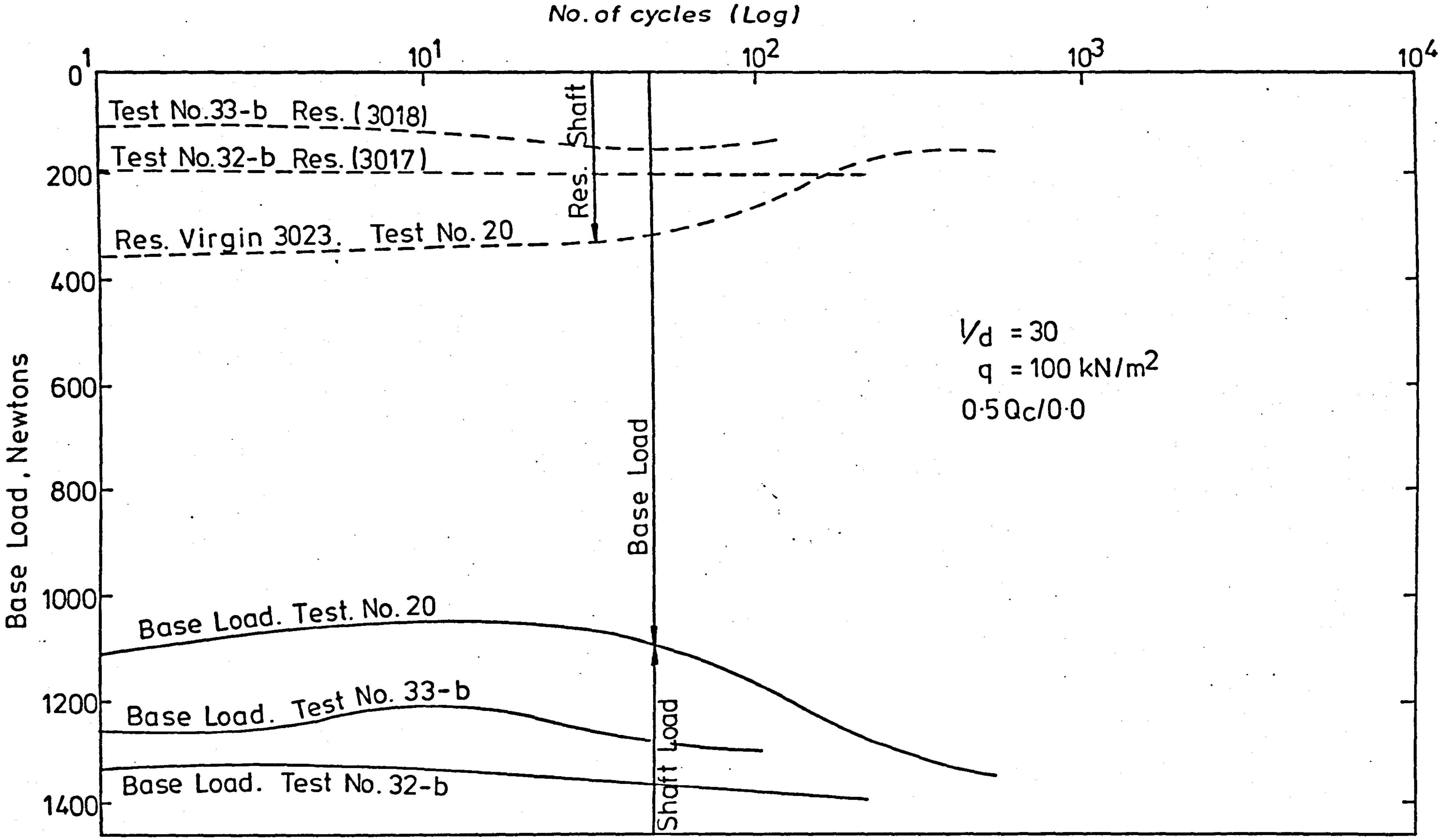


FIG. 6-36. VARIATION OF BASE LOAD WITH LOG. NO OF CYCLES.

The magnitude of the shaft load and its residual load tended to reach a limiting value which seemed to be independent of the previous loading history of the pile. The same conclusion may be drawn from Fig. 6.37 and 6.38 in which the succeeding loads were 0.3Qc/0.0 and 0.7Qc/0.0 respectively.

Fig. 6.39 shows the variation of skin friction along the pile shaft during the two phases of Tests 20, 33-b and 32-b in which the repeated loads were 0.5Qc/0.0 and Tests 32-b and 33-c where the loading was 0.7Qc/0.0. Three load cycles were chosen for comparison. The first cycle, an intermediate cycle and the last cycle of each test. The results shown supported the conclusion stated earlier that repeated loading brought about a redistribution of loads along the pile length which resulted from a deterioration of the frictional resistance.

In Test 34 the depth ratio and the surcharge pressure were 20 and 100 kN/m^2 respectively. The pile was subjected to the following sequence of loadings:-

(34 - a) 0.3Qc/0.0

(34 - b) 0.5Qc/0.0

(34 - c) Static compression load test

(34 - d) 0.99Qc/0.0

(34 - e) 0.3Qc/0.0

The results of this test, Figs. 6.40 and 6.41 confirmed the conclusions drawn from Test 33, that is the rate of movement decreased as the load level of the previous loading increased and that the pile life-span of a previously loaded pile depended on the load level in the succeeding loading.

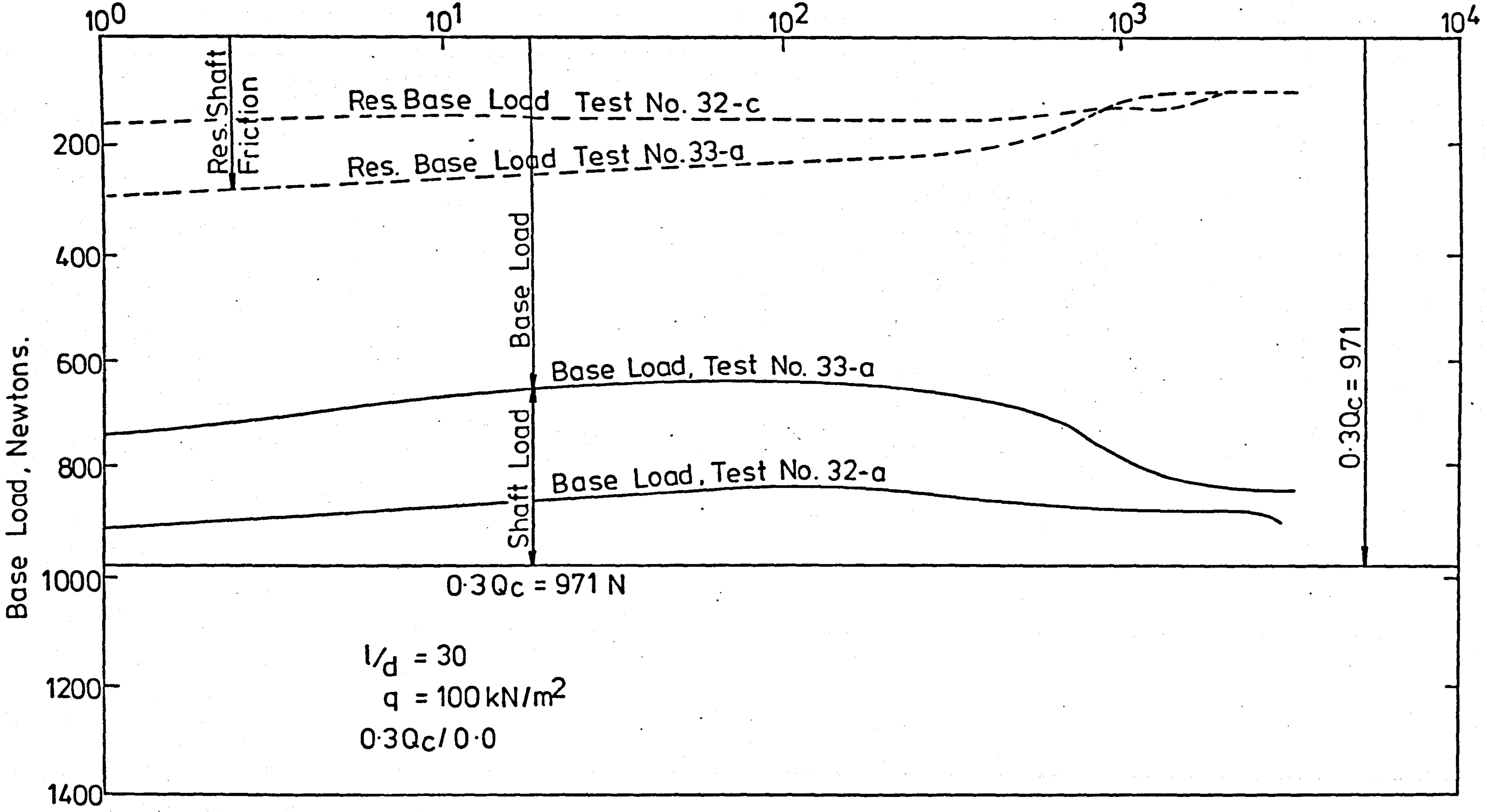


FIG.(6-37) VARIATION OF THE PILE LOADS WITH LOG. NO. OF CYCLES.

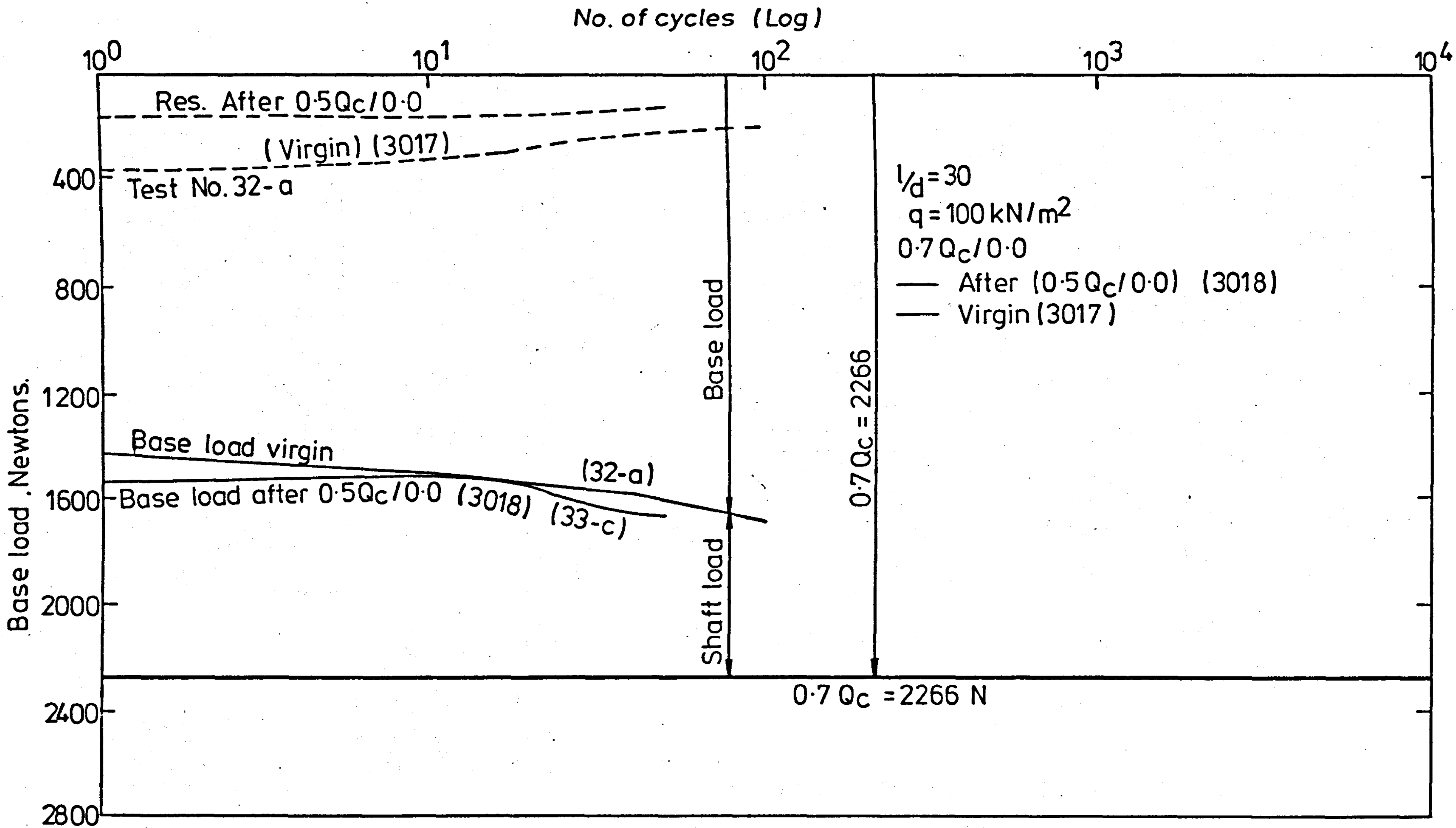


FIG.(6·38). VARIATION OF THE PILE LOADS WITH LOG NO. OF CYCLES.(l/d = 30)

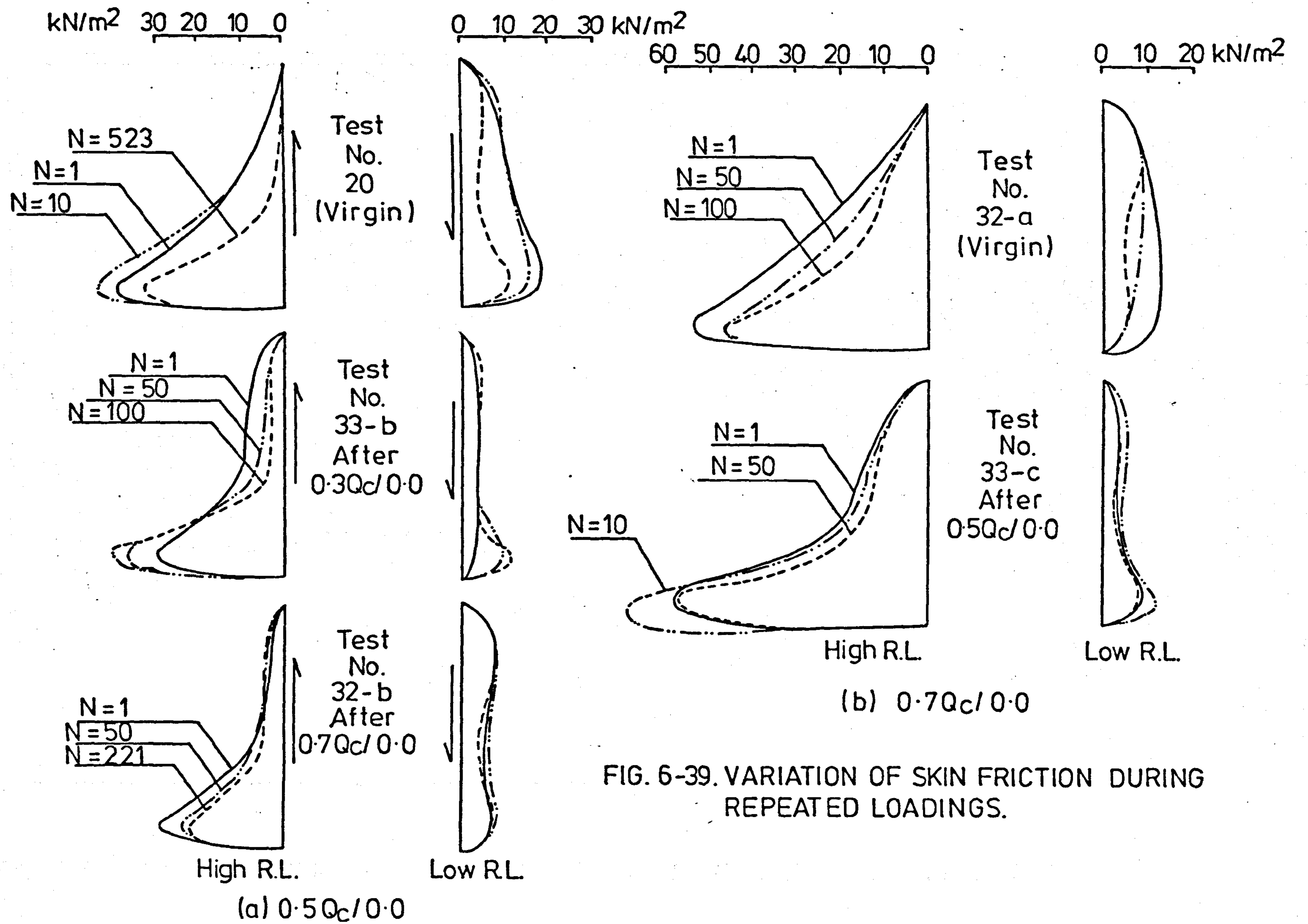


FIG. 6-39. VARIATION OF SKIN FRICTION DURING REPEATED LOADINGS.

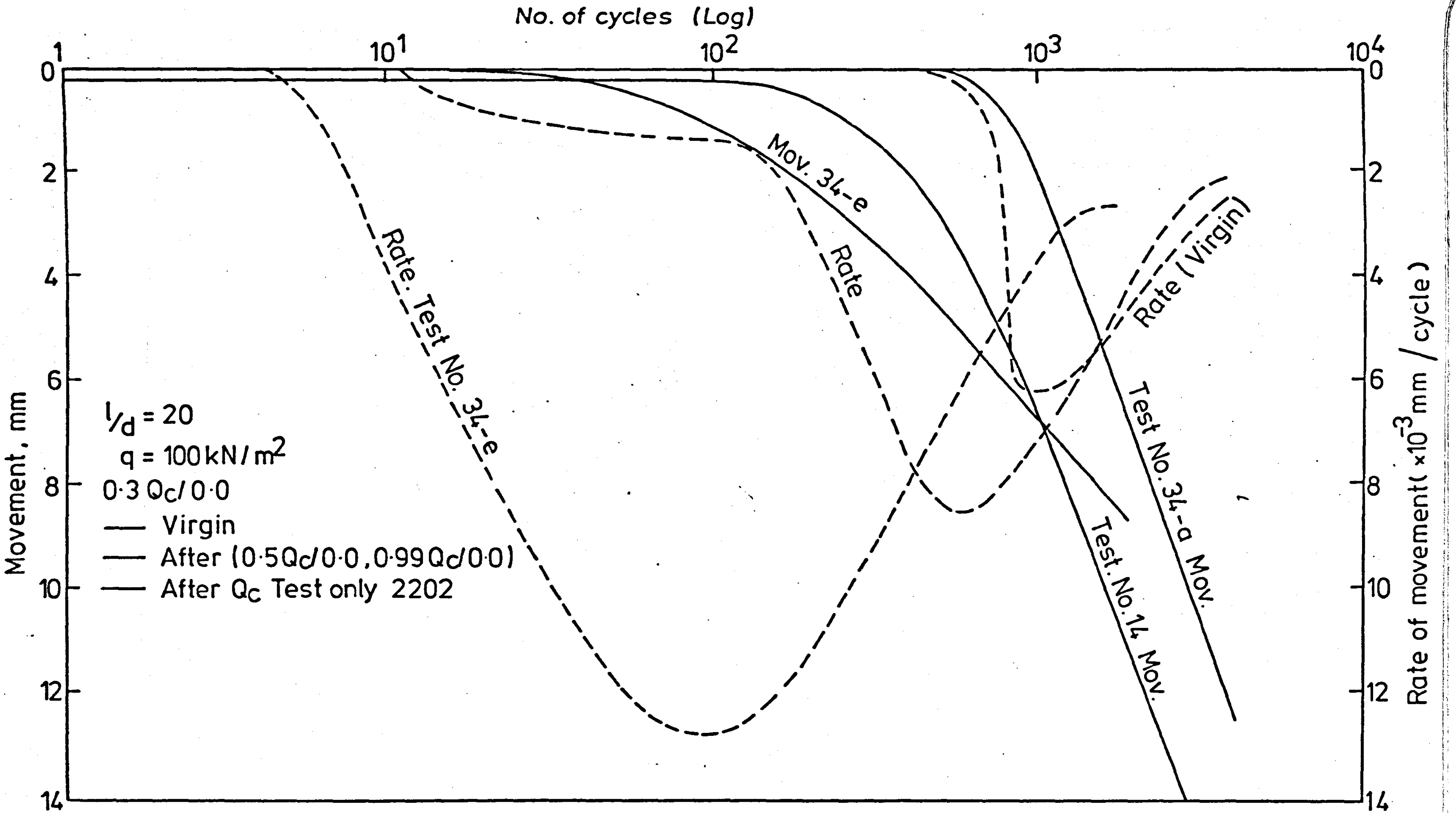


FIG. (6-40) VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH No. OF CYCLES

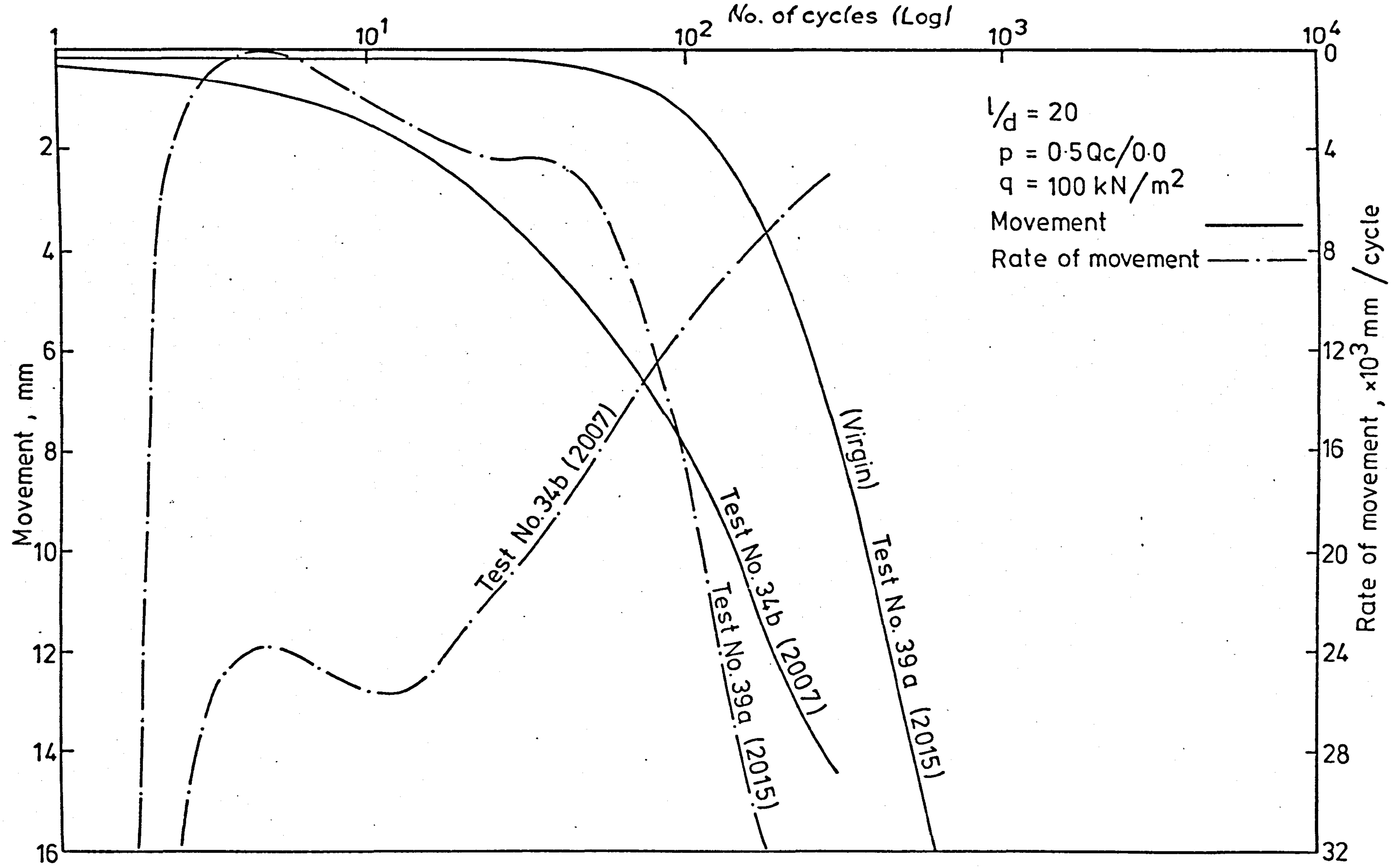


FIG. (6-41) VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH No. OF CYCLES

The results of Test 35 in which the surcharge pressure was 200 kN/m^2 and the pile of 20 diameters depth, revealed that the main features of the pile behaviour did not change when the surcharge pressure increased from 100 kN/m^2 to 200 kN/m^2. It is of interest to note in this test that the residual base load reached a limiting value which appeared to be independent of the repeated load level as shown in Fig. 6.42.

6.7 Series Vl: Compressive-to-tensile repeated loads

This section deals with the results of Tests 36, 37 and 38 in which the repeated loadings were 0.15Qc/0.15Qt, 0.15Qc/0.3Qt and 0.5Qc/0.15Qt respectively. The piles were tested at 20 diameters depth with a surcharge pressure of 100 kN/m^2.

The movements and the rates of movement versus the number of load cycles of these tests together with those of Tests 23 and 39 are presented in Fig. 6.43. In Test 36 the initial stage of the pile life was characterised by a very small or even zero downward movement. After a certain load cycle, and as the number of cycles was increased the rate of movement increased and reached a maximum value of 0.014mm/cycle around the 1700th cycle. The behaviour of the pile during this period was similar to that of test Series II.. However, after the pile had moved more than 12mm the direction of the pile movement was reversed with a rate which increased very rapidly as the repeated loading progressed. When the test was terminated after 200 cycles of upward movements the pile head had risen more than double the downward movement.

When the repeated load level of the tension phase was increased to 0.3Qt, Test 37 (2012) the downward movement ceased. The behaviour of the pile in this test was similar to that of Series III. If a comparison is made between the life-span of this pile and that of Test 23, in which the load was repeated between zero and

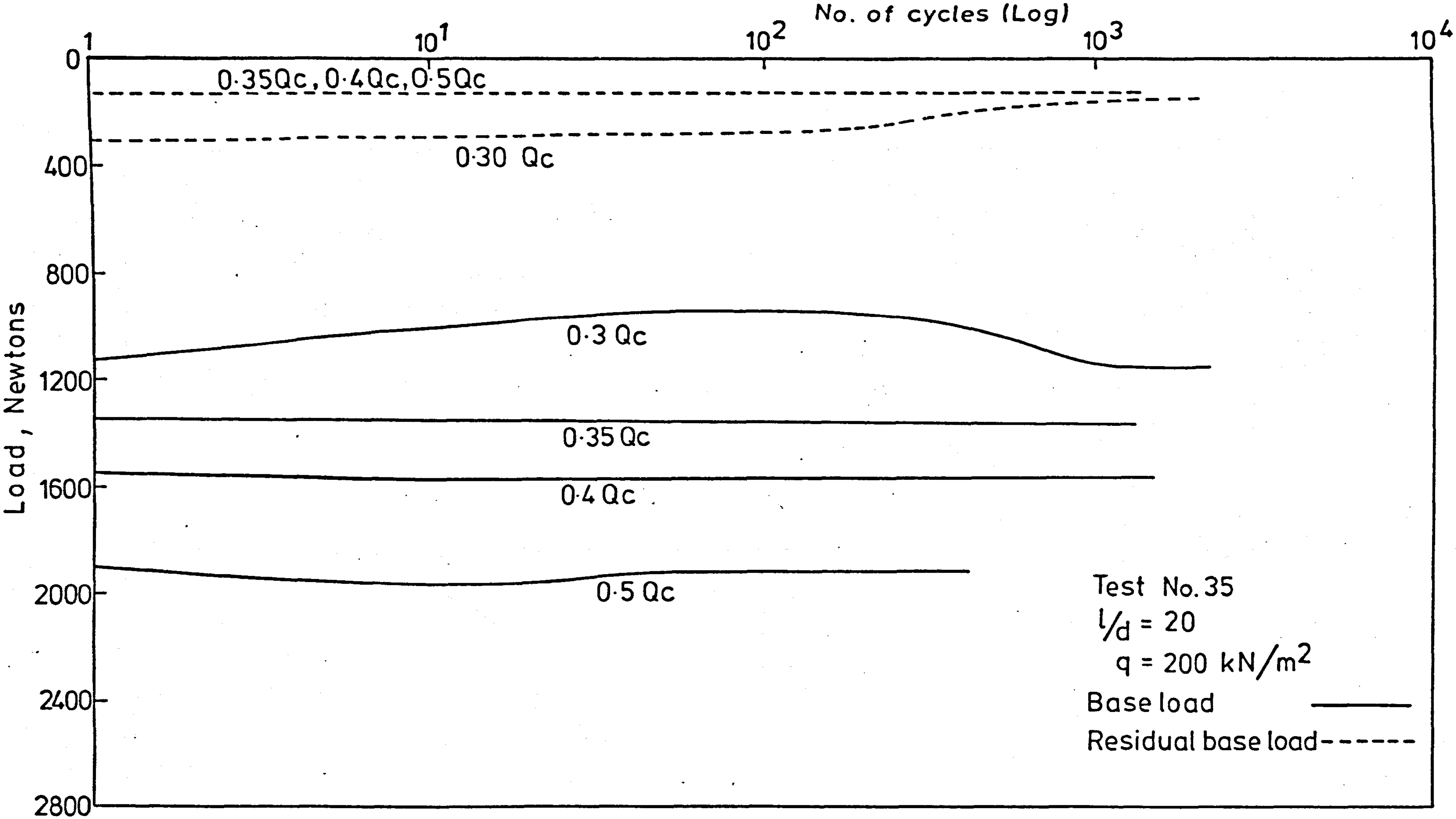


FIG. (6·42) LOAD TRANSFER DURING REPEATED LOAD OF VARYING LOAD-AMPLITUDES

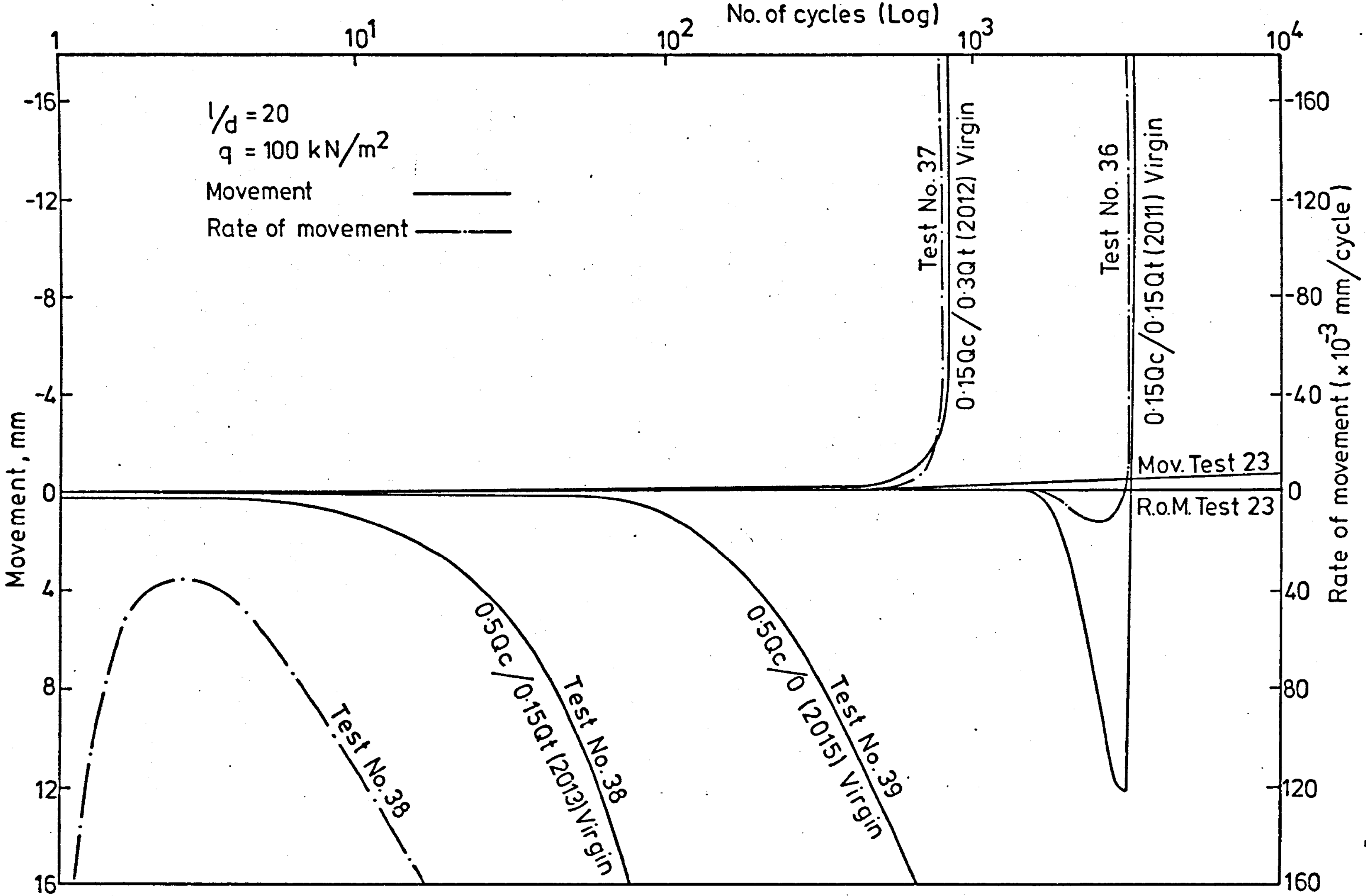


FIG.(6-43) VARIATION OF MOVEMENTS AND RATES OF MOVEMENT WITH No. OF CYCLES

0.3Qt it will be seen that the latter pile had longer life. The small component of compressive repeated load, 0.15Qc, caused a reduction in the life-span of the pile from 30000 cycles to 800 cycles. The upward movement ceased and the pile penetrated continuously into the sand when the compression phase of the repeated loads was increased from 0.15Qc to 0.5Qc in Test 38. The behaviour of the pile in this test was similar to that of Series II. The life-span of this pile also decreased, as compared with that of Test 39 (2015) due to the presence of the small component of tensile repeated load, 0.15Qt.

Fig. 6.44 shows the variation of the skin friction during the first, two intermediate and the last cycle of Test 36. The redistribution caused by repeated loading is well demonstrated in this figure. Along the upper third of the pile and after a few cycles, the direction of skin friction during both phases was upward while along the lower third it was alternating as the load was repeated. Over the middle third the skin friction was always downward and fluctuated in value with the progress of the repeeated loading.

6.8 Series V11: Miscellaneous

In this section the results of Tests 39, and 40 are discussed. During Test 39 the pile, which was of 20 depth ratio was first tested under repeated loads of 0.5Qc/0.0 and after the rate of movement began to decrease from its maximum value and became relatively small the pile was immediately subjected to a repeated load fluctuated between 0.5Qc/0.3Qt. This test was aimed at studying the influence of the previous compression-to-zero repeated loads on the succeeding compression/tension repeated loads.

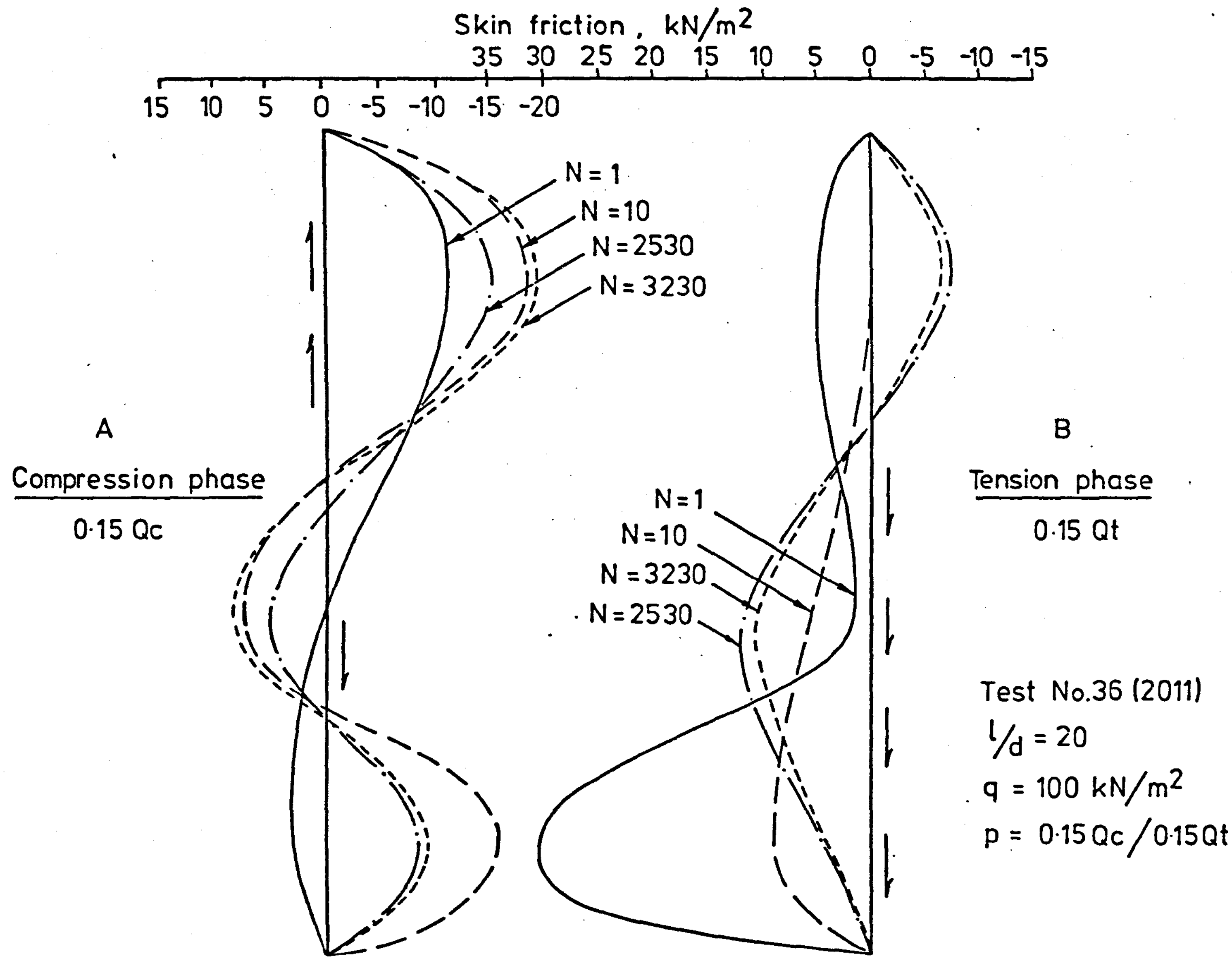


FIG.(6-44). DISTRIBUTION OF SKIN FRICTION DURING 0·15 Qc / 0·15 Qt REPEATED LOADING

In Test 40 the creep characteristic of the pile under 0.7Qc was examined. This test was conducted to evaluate the influence of creep on a pile subjected to a sustained load of the same magnitude while the sand surface was acted upon by a surcharge pressure cycled between 50 and 100 kN/m^2 such as those of Series I Table 5.2.

The variations of loads, movements and rates of movement with the logarithm of the number of load cycles of Test 39 are presented in Fig.6.45. It can be seen that the behaviour of the pile during the first part of the test, in which the repeated loads were 0.5Qc/0.0 was similar to those of Series II. Once the pile was subjected to the second part of the test, in which the repeated loads were 0.5Qc/0.3Qt, a change in its behaviour took place. In spite of the compression repeated load being the same, 0.5Qc, the rate of movement which was decreasing during the previous part began to increase rapidly as the number of cycles of the succeeding repeated loads was increased. After a maximum value was reached the rate of movement again decreased with increase of load cycles. The maximum rate of movement of the previous repeated loads, which was 0.0366mm/cycle and attained after 281 cycles, became 0.052mm/cycle and was attained after 100 cycles. At the beginning of the second part of the test the base load, during the compression phase of the repeated loads, increased as the rate of movement was increased. After a maximum value the base load gradually decreased and returned back to the value that had been reached at the end of the first part of the repeated loading, as shown in the figure. When the movement of the pile in the second part

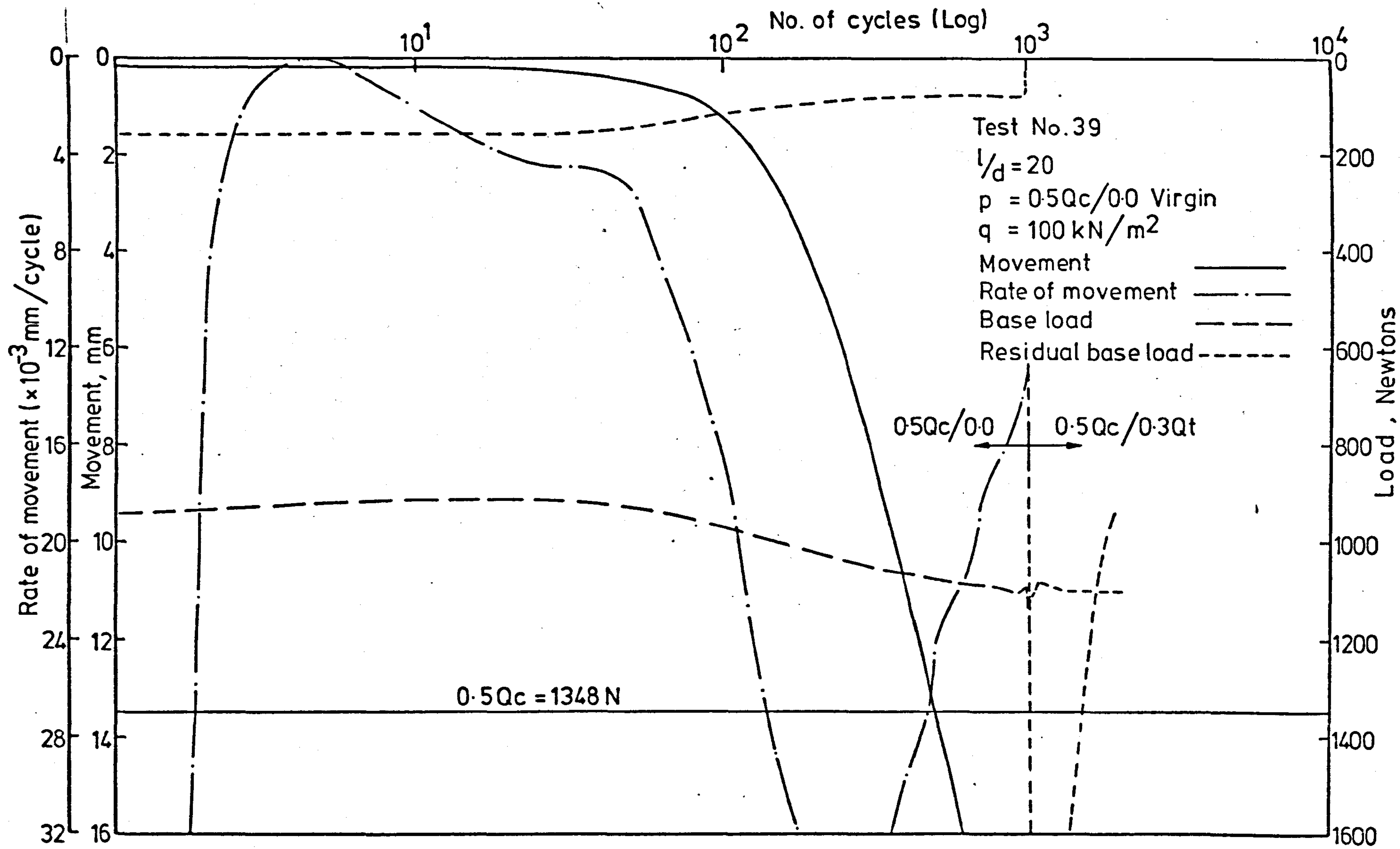


FIG.(6-45) VARIATION OF LOADS MOVEMENTS AND RATES OF MOVEMENT AGAINST No. OF CYCLES is compared with that of Test 38, despite the tensile repeated loading being higher in Test 39, it will be seen, Fig. 6.46. that a longer life was obtained from Test 39. The reason for this behaviour is probably due to the stiffening effect which resulted from the previous repeated loads of Test 39.

Fig. 6.47 shows the movement versus logarithm of the time in hours of a pile subjected to a sustained load of 0.7Qc. Initially the movement increased at a rate which decreased rapidly. After a few minutes, the rate of movement with respect ot the logarithm of the time in hours reached more or less constant value. At the time of writing the thesis, and after about 4510 hours, the pile had not settled more than 1.45mm.

6.9 Conclusion

From this part of the investigation the following conclusions may be drawn:-

(1) Preliminary tests carried out on piles at various depths of embedment with a constant surcharge pressure of different values indicated that:-

(a) The driving resistance of the pile base increased rapidly with the depth of penetration, reached a maximum then decreased slightly to a limiting value. The maximum value increased and was attained earlier when the surcharge pressure was increased.

(b) During each identical driving test, the value of skin friction at

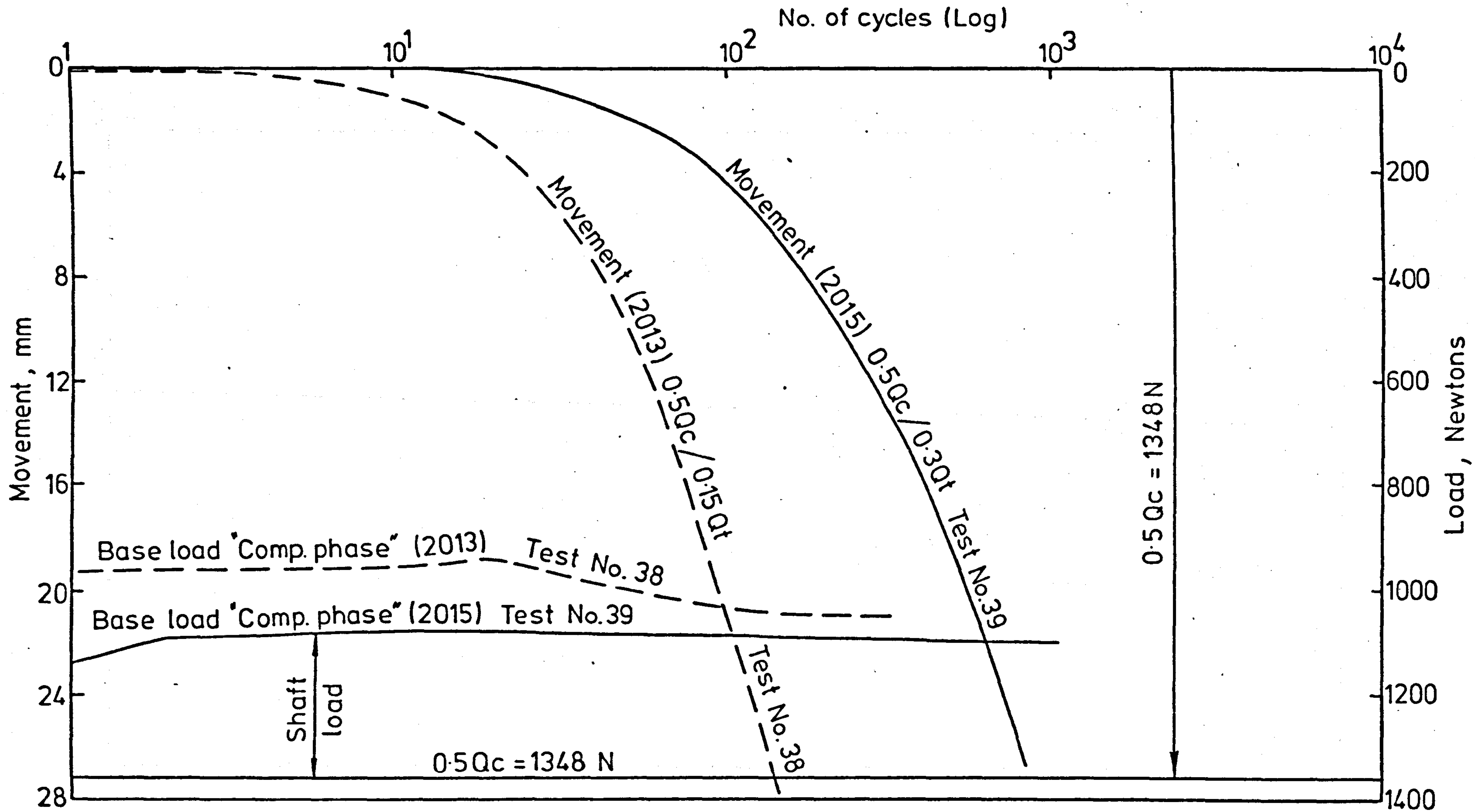


FIG. 6-46 VARIATION OF LOADS AND MOVEMENTS WITH No. OF CYCLES ($l/d = 20$)

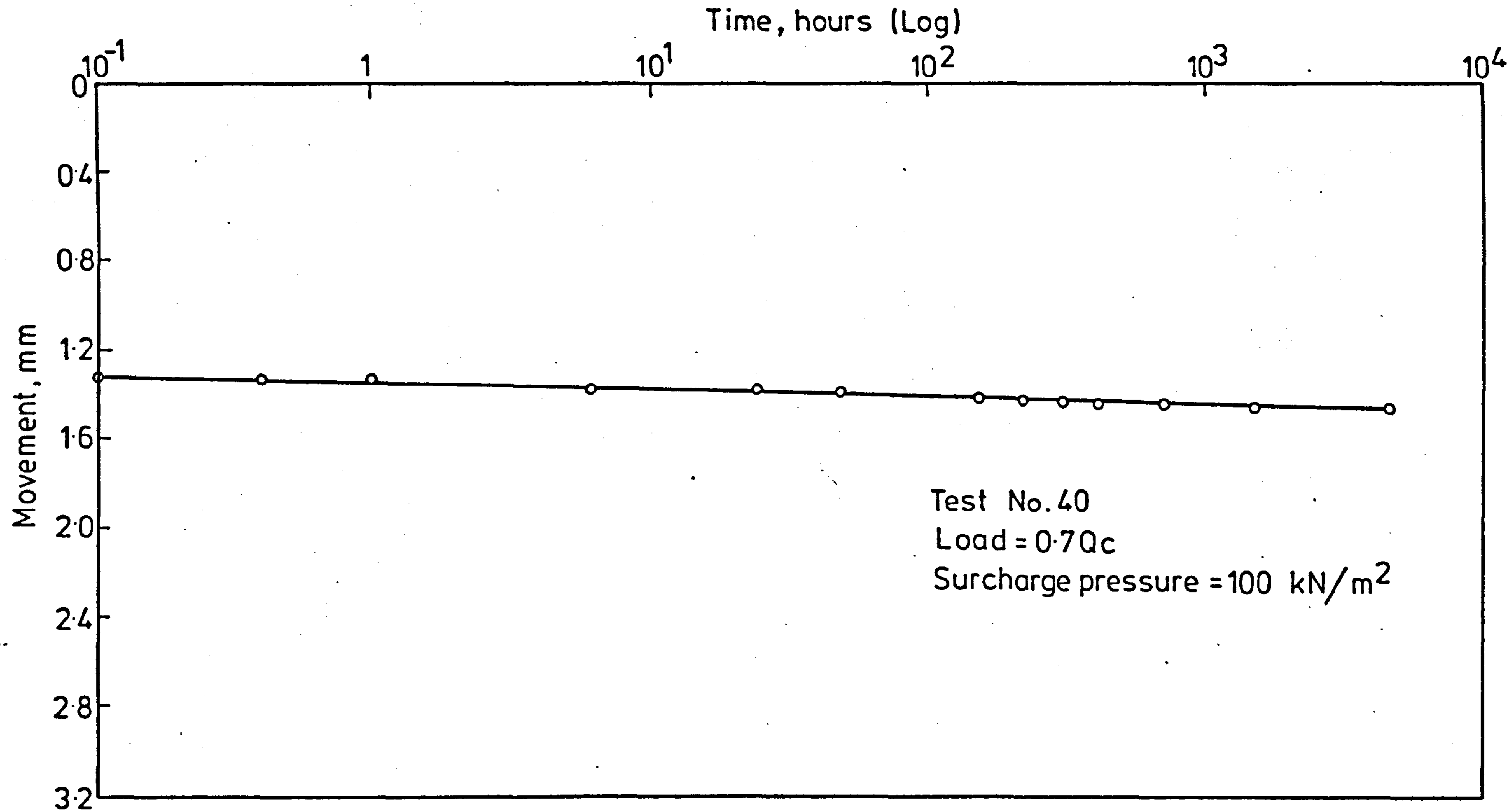


FIG.(6-47). RESULTS OF CREEP TEST

any given depth below the sand surface was much less repeatable than the pile base resistance.

(c) The ultimate static load capacity of the pile increased and the load-displacement became stiffer when the depth of embedment and/or the surcharge pressure was increased.

(d) The variation of the ultimate base resistance with depth had the same trend as that during penetration. The peak value was smaller and was attained at a greater depth in the case of the static loading.

(e) The ultimate capacity and the load displacement behaviour of the pile were affected by the state of the residual stresses before testing.

(f) The shaft friction in compression was generally greater than that in tension loading by a magnitude dependent on the depth of embedment, the surcharge pressure and the magnitude and direction of the residual stresses before testing.

(2) The results of repeated comoression loading tests performed on piles placed at various depths revealed that:-

(a) Initially the pile was stable, but

after a certain load cycle, it began to move at a rate which increased rapidly to a maximum then decreased as the number of cycles increased.

(b) For a given percentage of loading, 0.3Qc, the pile life-span decreased when the embedment depth increased .

(c) The number of cycles at which the rate of movement reached a maximum value appeared to be independent of the pile depth. It was around cycle number 1300.

(d) At any depth, as the number of cycles was increased, the shaft load increased up to a peak value then decreased gradually to a limiting value. This limiting value increased when the depth of embedment was increased.

(e) The reduction in shaft load from its peak value was associated with an increase in the rate of pile movement. This increase reached a maximum value at the cycle during which the shaft load reached its limiting value.

(f) The rate of movement increased and the life-span was reduced when the pile was subjected to a static failure loading before being tested under repeated loadings.

(3) Tests performed on tension piles at different depths of embedment showed that :-

(a) The pile moved initially at a decreasing rate. After a stable stage the movement began to increase at a rate which increased very rapidly until failure occurred.

(b) For a given percentage of loading, 0.3Qt, the pile life-span increased when the depth of embedment increased.

(c) At a given pile depth and a given loading the pile life-span reduced when it was previously subjected to static failure loading.

(4) For both compression and tension piles and at any given depth of embedment the life span increased when the load amplitude and/or the load level was decreased.

(5) For both compression and tension piles tests at any depth the life span increased when the surcharge pressure was increased.

(6) Previous repeated loading did affect the behaviour of the pile under the succeeding loading. Smaller rates of movement and longer life-spans generally resulted. Both the pile shaft load and its residual value decreased due to the previous repeated loading, the higher the previous load level the greater the reduction. At the last stage of the repeated loading, both the shaft friction and its residual

value reached a limiting value which appeared to be independent of the loading history of the pile.

(7) Compressive-to-tensile repeated loading greatly reduced the pile life-span as compared with that of the unidirectional repeated loading.

(8) The repeated loading was found to decrease the bearing capacity and the pulling resistance of the pile for all depths of embedment and all surcharge pressures examined in this part of the investigation. The highest reduction was observed in the case of tension piles, being more than 50% sometimes, whereas in compression piles this reduction seldom exceeded 30%.

CHAPTER 7

PRESENTATION AND DISCUSSION OF CYCLIC SURCHARGE TEST RESULTS

7.1 General

In the tests presented in chapter 6 the sand surface was subjected to a constant pressure while the effect of repeated loading on the pile performance was investigated. At some locations the soil surface may be acted upon by a cyclic surcharge varying in magnitude regularly or randomly. An example of such a condition of loading may be encountered with offshore piles. The frequencies of the cyclic surcharge pressure on the sand and the repeated loadings on the pile may not necessary be the same. Thus a phase difference might be expected. The influence of such complex conditions of stress on the behaviour of a pile was examined and the test results are presented in this chapter. Madhloom (1978) carried out a series of tests on compression and tension piles of 30 diameters depth embedded in a sand upon which a cyclic surcharge varying between 50 and 100 kN/m^2 acted. He found that:-

(i) The application of the cyclic surcharge caused compaction of the sand over a limited number of cycles beyond which there was little or no further compaction. The ultimate bearing capacity and the pulling resistance of the pile were, therefore, increased.

(ii) Under a static compressive loading test two trends of the pile movement were observed. At 0.6Qc or less, the pile moved at a decreasing rate as the number of surcharge cycles was increased. In contrast, at 0.8Qc load the rate was continuously increasing. In the case of static tensile loading, even at 0.55Qt loading, after a stable stage the pile movement increased by a rate which increased rapidly until failure occurred.

(iii) The behaviour of the pile subjected to repeated loading and with cyclic surcharge was similar to an identical pile but with a constant surcharge.

(iv) Schematic diagrams indicated that the most severe situation occurs when the load on the pile is at a maximum while the cyclic surcharge is at its minimum. value.

7.2 Series V111: Piles subjected to static loading.

In order to continue the research programme on the behaviour of piles under repeated loading the two limits of the cyclic surcharge pressure adopted by Madhloom (1978) have been used in this investigation.

The variation of the loads, the movements, and the rates of movement against logarithm of the number of surcharge cycles for Test 42 are shown in Fig. 7.1. The pile examined in this test, had a depth ratio of 30 and was subjected to a static compression load of 0.7Qc where Qc is the ultimate bearing capacity at 100 kN/m^2 surcharge pressure. The pile loading was applied in four increments while the surcharge pressure was kept constant at 100 kN/m^2.

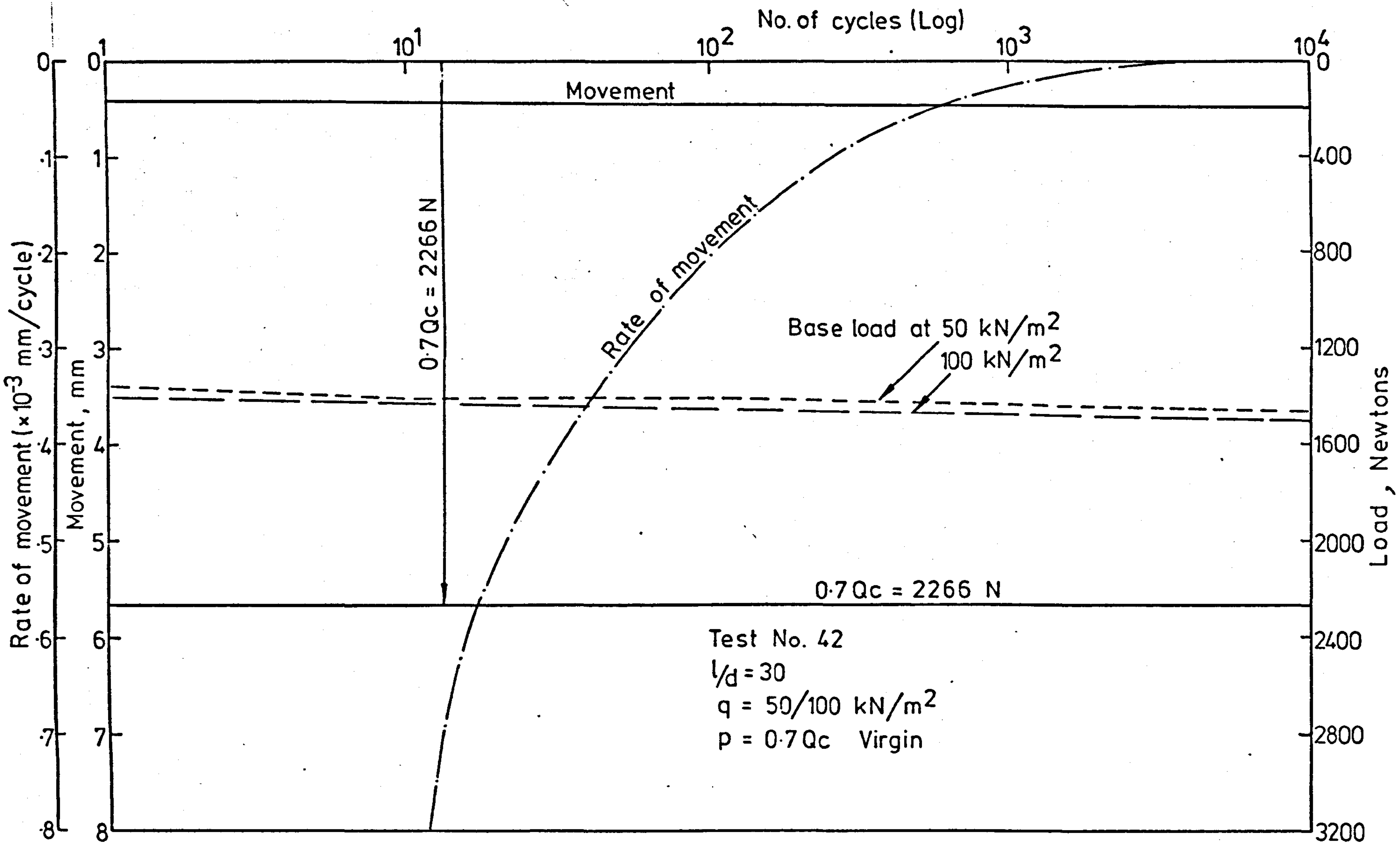


FIG. 7-1 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

To minimise the effect of creep on the observed movements, the test did not start until at least 10 hours after the load had been fully applied to the pile. At the initial stage of the cyclic surcharge and during the lower phase, 50 kN/m^2, the pile moved at a rapidly decreasing rate. The reason behind this movement is probably that at the lower pressure the pile loading, which was o.7 of the ultimate capacity at 100 kN/m^2 surcharge pressure, became close to or even higher than that at 50 kN/m^2 pressure. Regarding the decrease in the rate of movement, this may be explained as follows. Initially in each surcharge cycle and under the influence of the higher pressure the sand mass was compressed. This compression consisted of two components, reversible elastic and irreversible plastic movements. The first component resulted from the elastic deformation of both the sand grains and the arrays of grain, while the plastic movements were caused by rearrangement of the potentially unstable grains which slip into more stable locations and/or crushing, Ko and Scott (1967), Lambe and Whitman (1969). When the pressure was decreased to its lower value only the elastic movement was recovered. With the repetition of surcharge cycles the number of unstable grains decreased rapidly, Ko and Scott. This implies that the sand grains were arranged in such a way that they became approximately stable during the two phases of the cyclic surcharge. The rate of movement, therefore, would consequently decrease as the number of cycles increased.

The distribution of the axial pile loading during the two phases of the first, the 100th and the last cycle (32643) in this test are shown in Fig. 7.2. The axial load at any given point

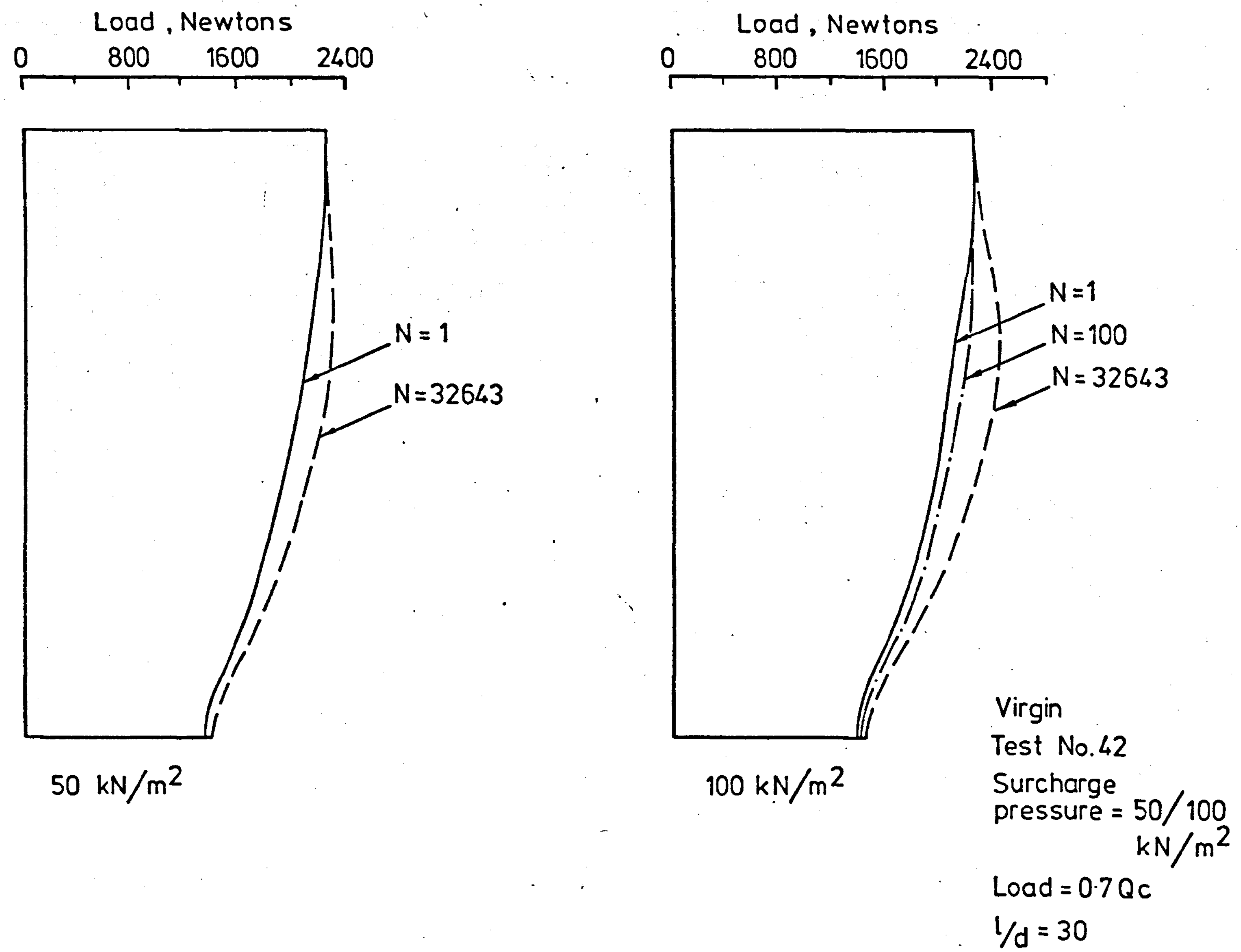


FIG. 7-2 DISTRIBUTION OF AXIAL LOADING DURING CYCLIC SURCHARGE

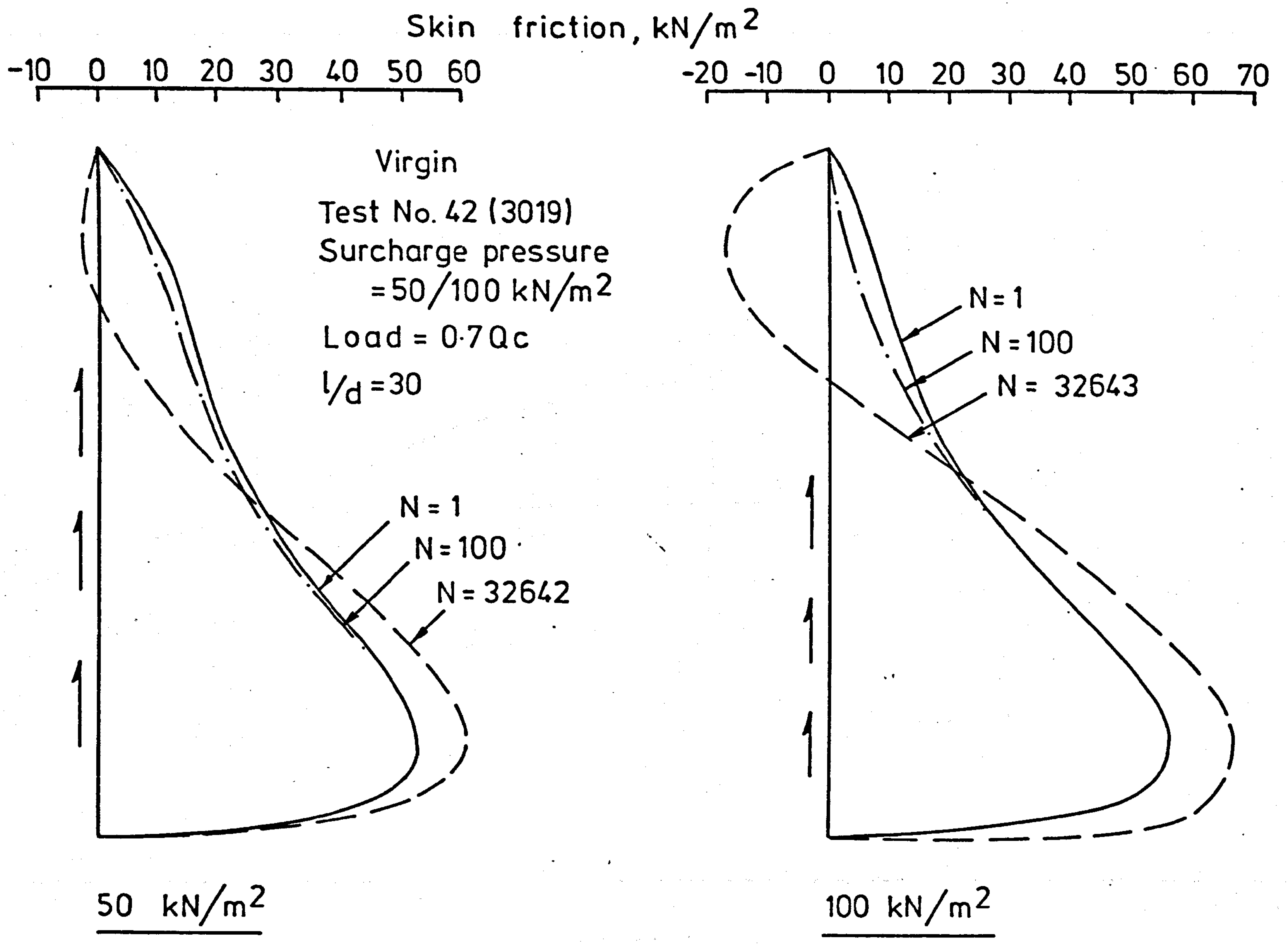


FIG. 7-3 DISTRIBUTION OF SKIN FRICTION DURING CYCLIC SURCHARGE

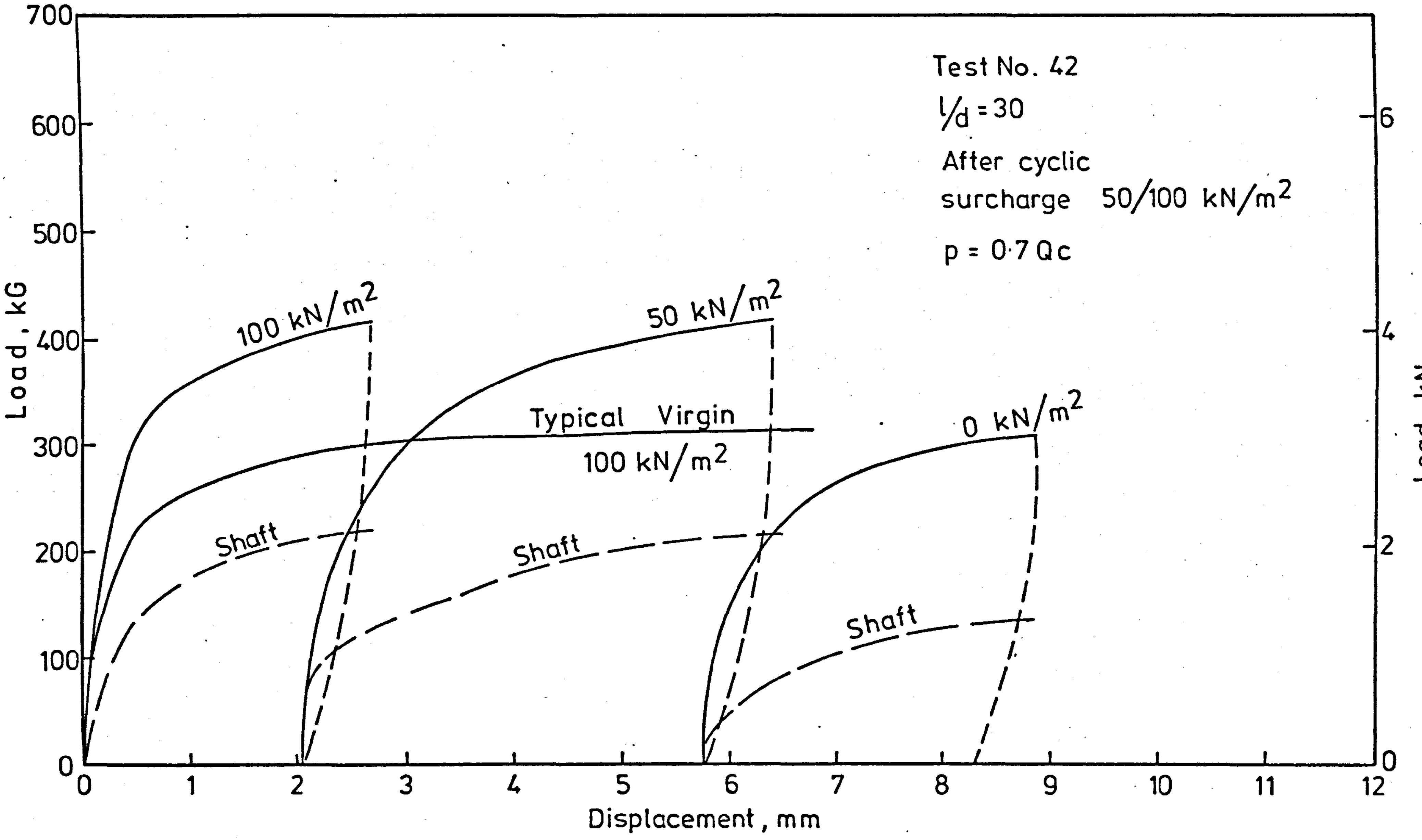


FIG. (7-4) LOAD — DISPLACEMENT

along the embedded shaft increased as the number of surcharge cycles increased. This load was greater at 100 kN/m^2 than that at 50 kN/m^2 pressure. At the end of the test, the axial load at points located within the upper third of the pile increased and became even larger than the applied pile loading. Therefore a negative skin friction developed along the pile at these points as shown in Fig. 7.3. The skin friction along the upper half of the pile decreased and along the lower half it increased when the number of cycles increased. At the end of the test, a load test was carried out on the pile at 100 kN/m^2 surcharge pressure and then repeated at 50 and 0.0 kN/m^2 pressure. Some interesting results were obtained as shown in Fig. 7.4. The ultimate capacity of the pile at 100 kN/m^2 pressure increased and it was approximately the same for both 100 and 50 kN/m^2 surcharge pressure. Even at zero surcharge pressure the pile load capacity increased and became close to that of the virgin pile at 100 kN/m^2 pressure. This phenomenon may be attributed to the structure of the sand grains. As stated above, after each pressure cycle the sand grains moved into more stable locations. With increase in the number of cycles, the structure of the sand grains, which supported the pile became more stable. Therefore at the end of the test the static load capacity of the pile at 50 kN/m^2 will be close to that at 100 kN/m^2 surcharge pressure.

From the results of Test 41, in which the pile was load tested before the beginning of the cyclic surcharge test, similar conclusions may be drawn as shown in Figs.7.5, 7.6 and 7.7. When the results of this test are compared with those of Test 42 it will be seen, Fig. 7.8, that the rate of movement decreased smoothly to zero during the virgin pile test whereas it fluctuated and reached a state of

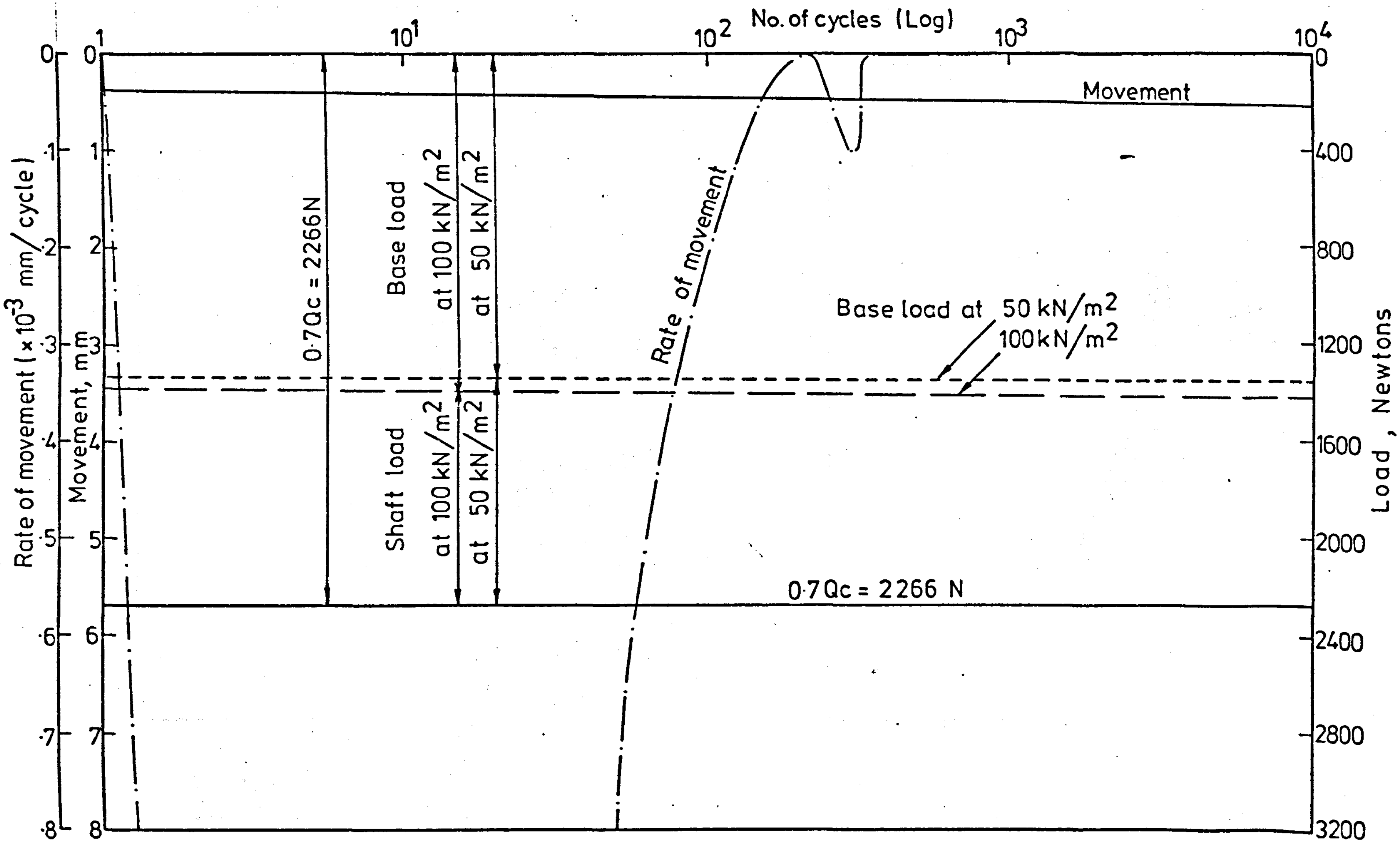


FIG. 7-5 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

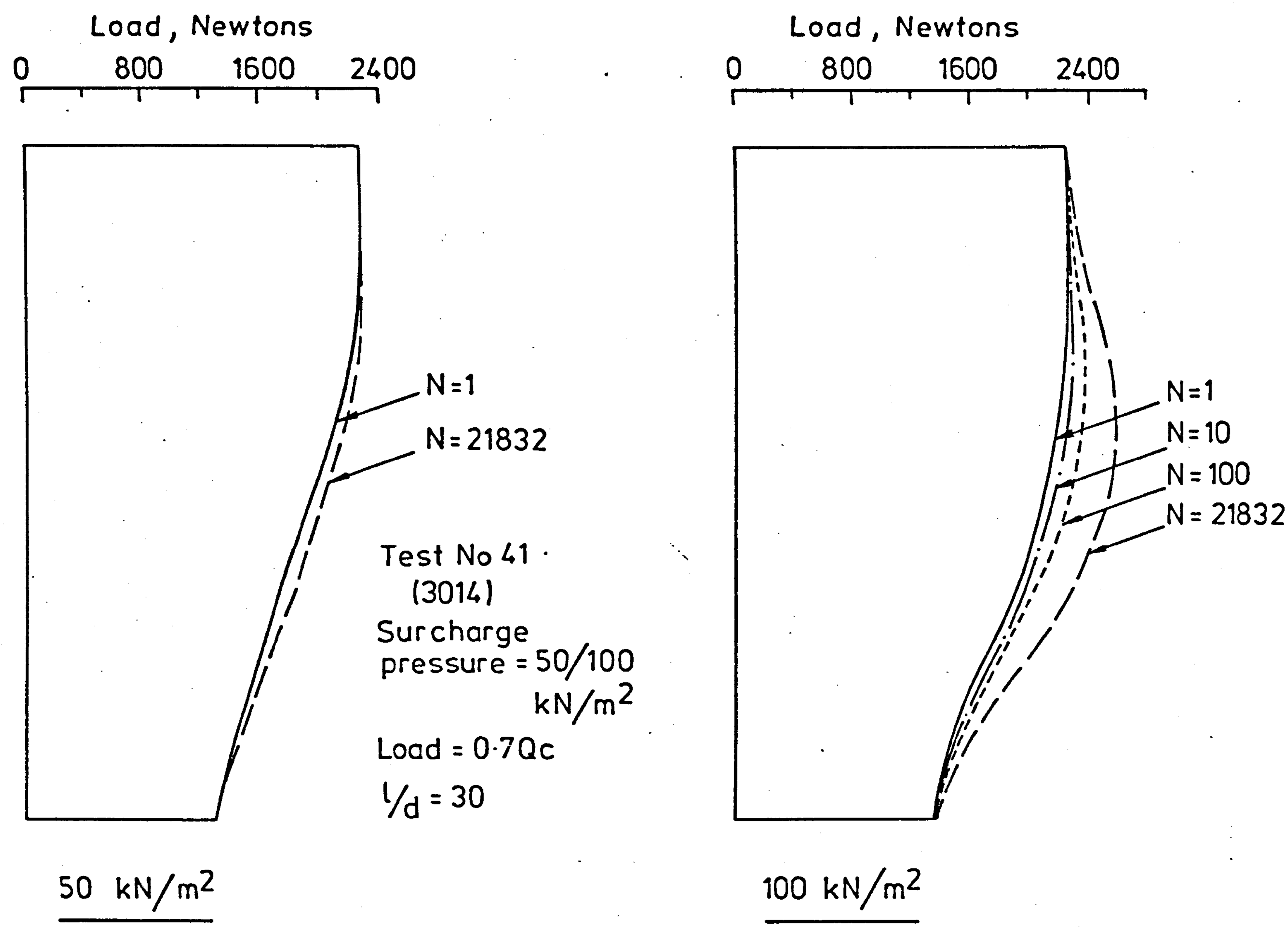


FIG. 7-6 DISTRIBUTION OF AXIAL LOADING DURING CYCLIC SURCHARGE

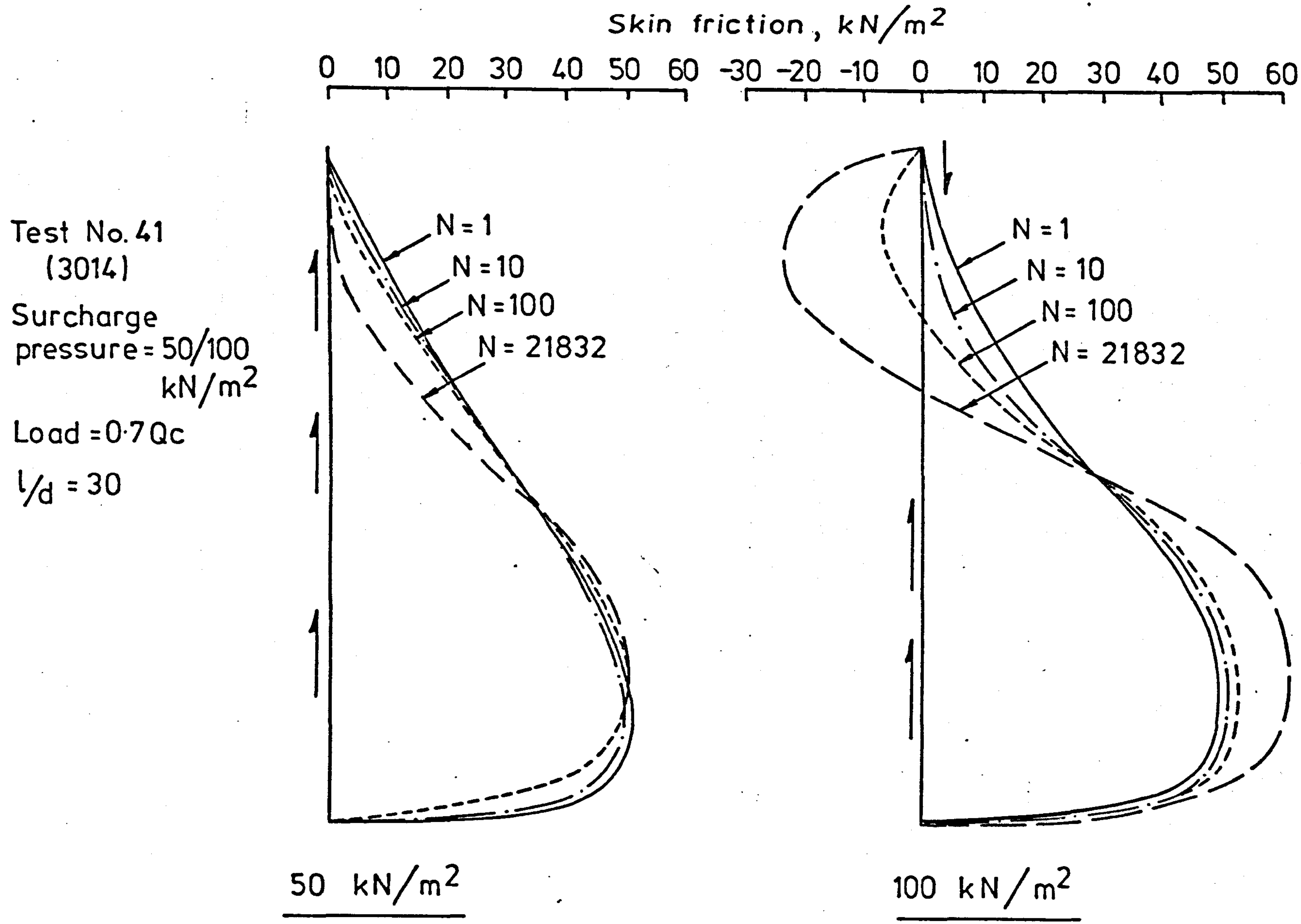


FIG. 7-7 DISTRIBUTION OF SKIN FRICTION DURING CYCLIC SURCHARGE

zero value rapidly in the case of the previously loaded pile. The number of surcharge cycles at which the rate of movement of Test 42 attained zero value was approximately ten fold of that of Test 41. The figure shows also that greater load was transferred to the pile base during Test 42, virgin pile, as compared with that of Test 41.

Fig. 7.9 illustrated the variation of loads, movements and rates of movement of Test 43, in which the pile was also subjected to 0.7 of the ultimate capacity at 100 kN/m^2 surcharge pressure but buried at 20 diameters depth. When the results in this figure are compared with those of Test 42, Fig. 7.1, in which the pile was embedded at 30 diameters depth, it may be concluded that both the pile movement and the rate of movement decreased as the depth of embedment was decreased. Concerning the pile base load it can be seen that the difference in this load as registered during the higher and the lower surcharge pressure of any given cycles was more pronounced as compared with those of the longer pile. This is because the relative movements between the sand and the pile, due to the change in surcharge pressure was a maximum at the sand surface and decreased as the depth of the sand was increased.

The influence of cyclic surcharge on the behaviour of a tension pile subjected to a static load of 0.7Qt, where Qt is the ultimate pulling resistance of the pile at 100 kN/m^2 surcharge pressure, and embedded at 30 diameters depth was investigated in Test 44. The results of this test presented in Fig. 7.10 revealed that the general feature of the pile behaviour was similar to that observed in section 6.4 where the tension piles subjected to repeated loadings

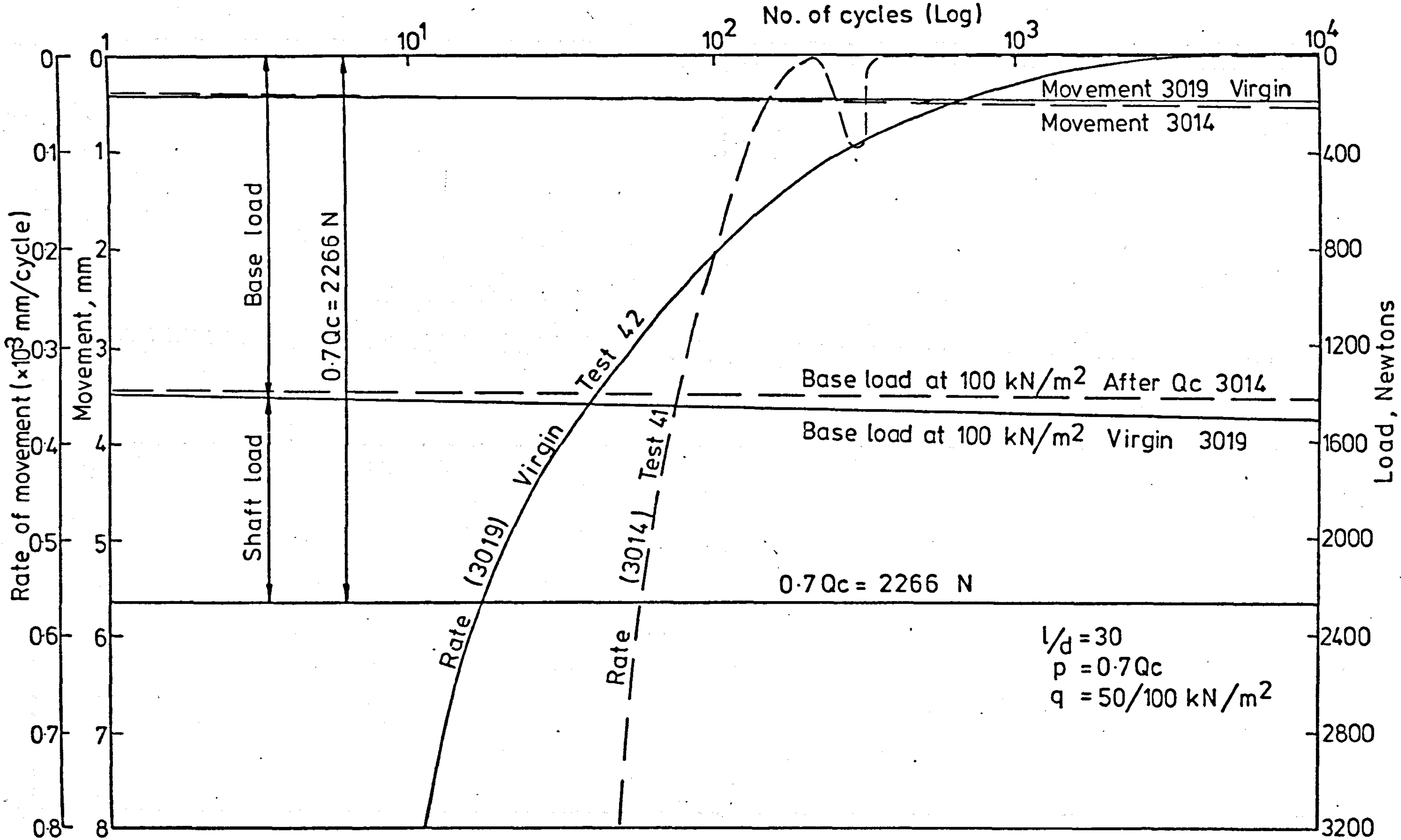


FIG.7-8 INFLUENCE OF STATIC FAILURE LOADING ON THE SUBSEQUENT BEHAVIOUR OF THE PILE

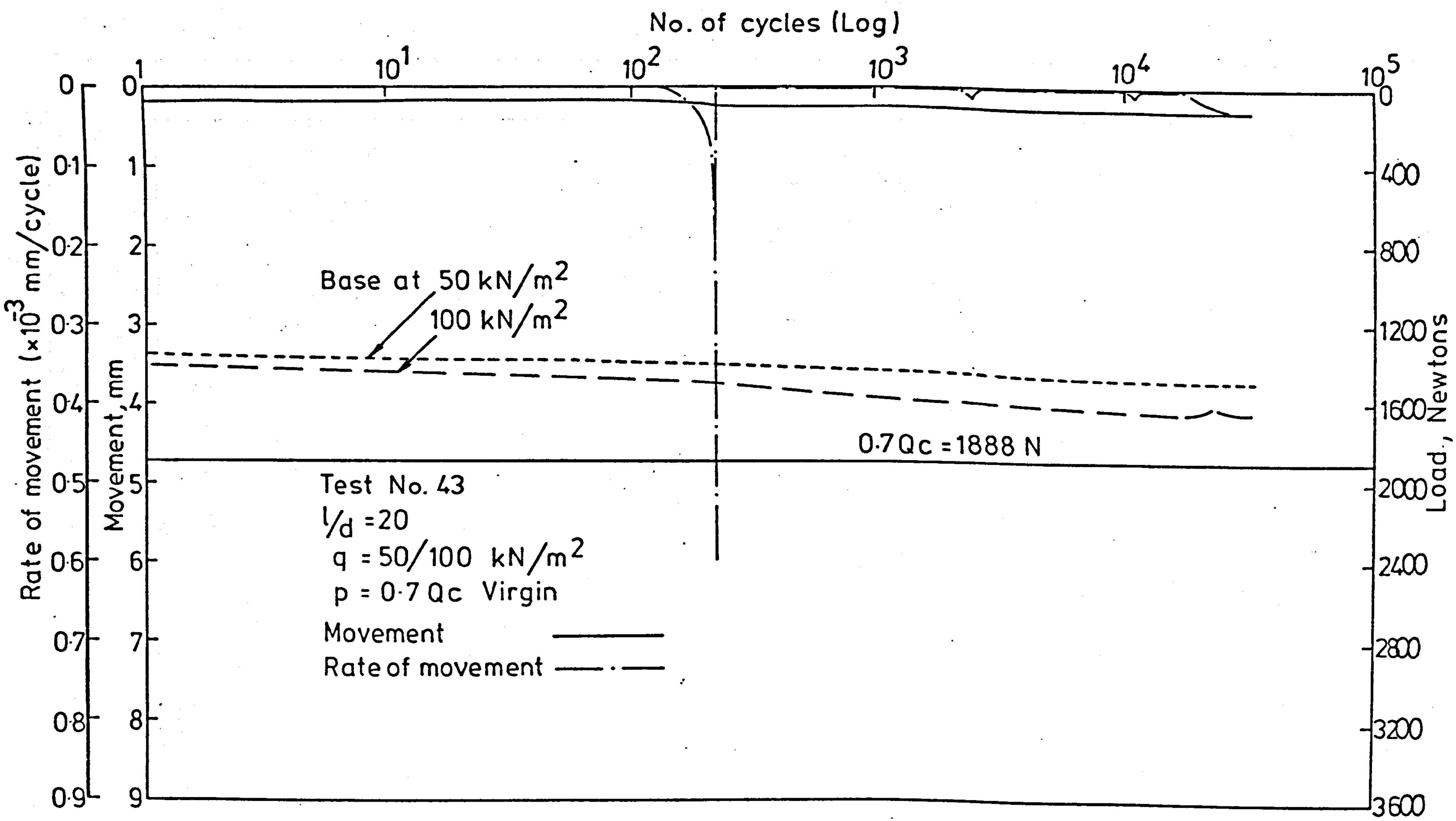


FIG. 7-9 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

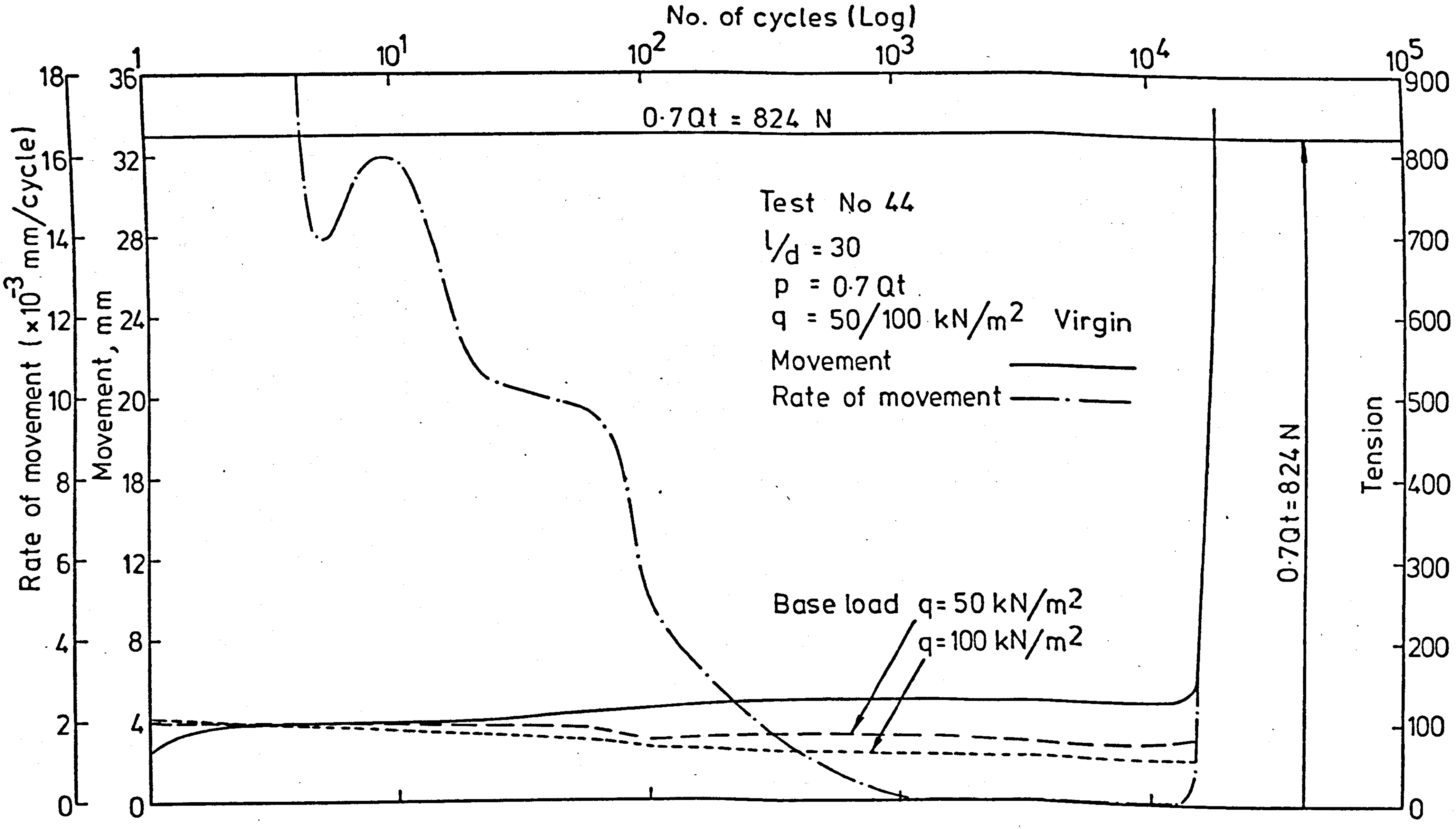


FIG. (7-10) VARIATION OF LOADS MOVEMENTS AND RATES OF MOVEMENT WITH No. OF CYCLES with static surcharge pressure.

The distributions of skin friction during each phase of the cyclic surcharge of the first, the 10th, the 1070th and the last cycle, cycle number 18300 are illustrated in Fig. 7.11. It can be seen that the residual compression skin friction mobilised after penetration decreased and became entirely tensile when the loading 0.7Qt was fully applied to the pile. During the first cycle, therefore, the skin friction was entirely in tension with a value which increased as the depth of the pile increased until it reached a maximum value at a point located 3.5 diameters above the base. Beyond this it decreased rapidly and became zero at the lower end of the pile. Except for the initial few cycles of the lower surcharge pressure, where the pile was progressively pulled out, in each cycle of the surcharge pressure the negative skin friction along the upper half of the pile increased whilst along the lower half it decreased. The results of a static pulling load test carried out on the pile after the end of the cyclic surcharge test at 100 kN/m^2 and then repeated at 50 kN/m^2 surcharge pressure are shown in Fig. 7.12. For comparison typical load-displacement curves at 100 kN/m^2 pressure are also presented in this graph. It may be concluded that the pulling resistance of the pile increased after it had been tested under cyclic surcharge. Although the resistance at 50 kN/m^2 pressure was relatively high it was smaller than that at 100 kN/m^2 pressure.

7.3 <u>Series 1X: Repeated loads in-phase with the cyclic surcharge.</u>

Through this test series the pile which embedded at 20 diameters depth, was subjected to repeated compressive loads. These loads acted upon the pile in-phase with the cyclic surcharge prrssure. Two modes of testing were therefore employed within this series.

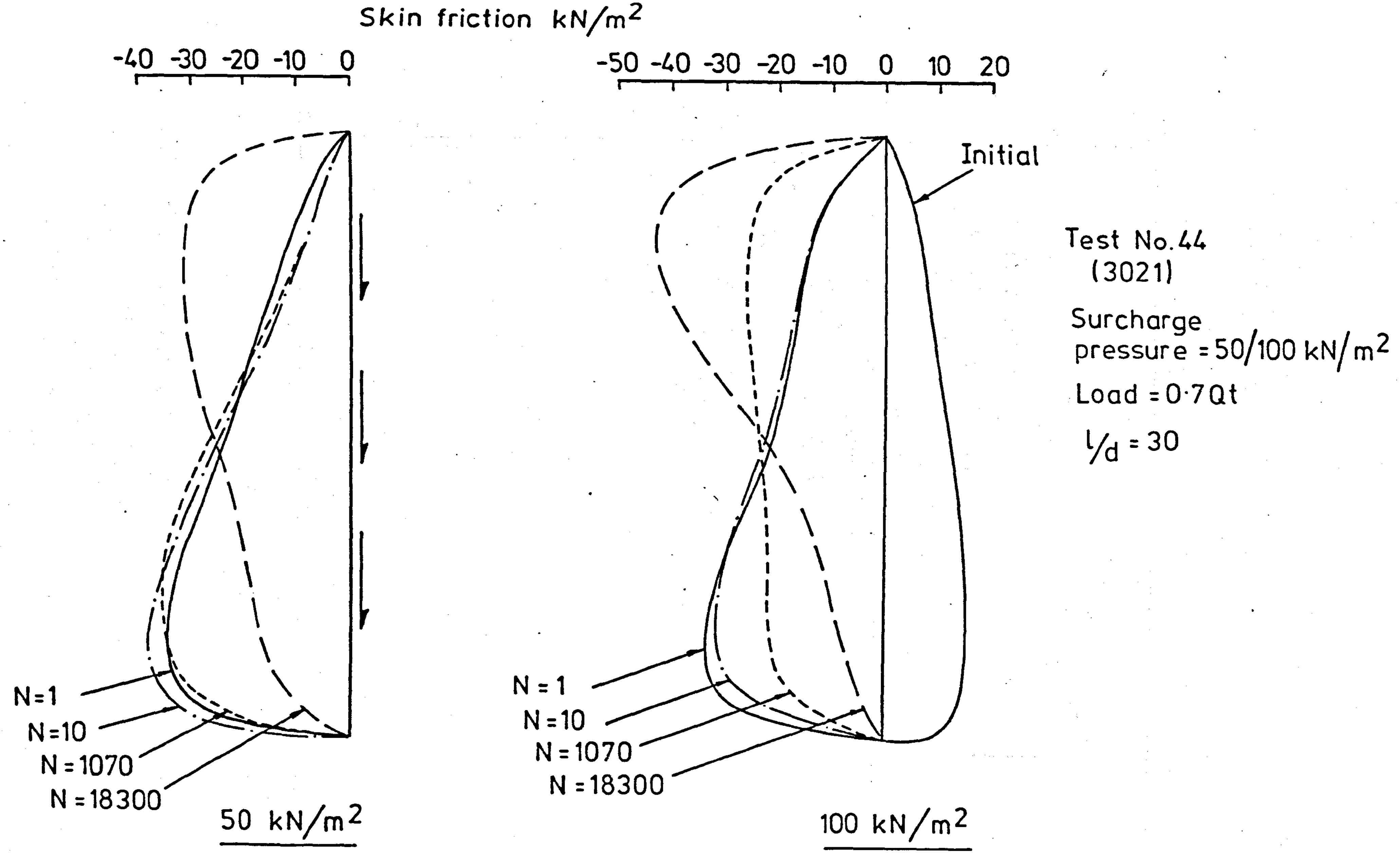


FIG. 7-11 DISTRIBUTION OF SKIN FRICTION DURING CYCLIC SURCHARGE

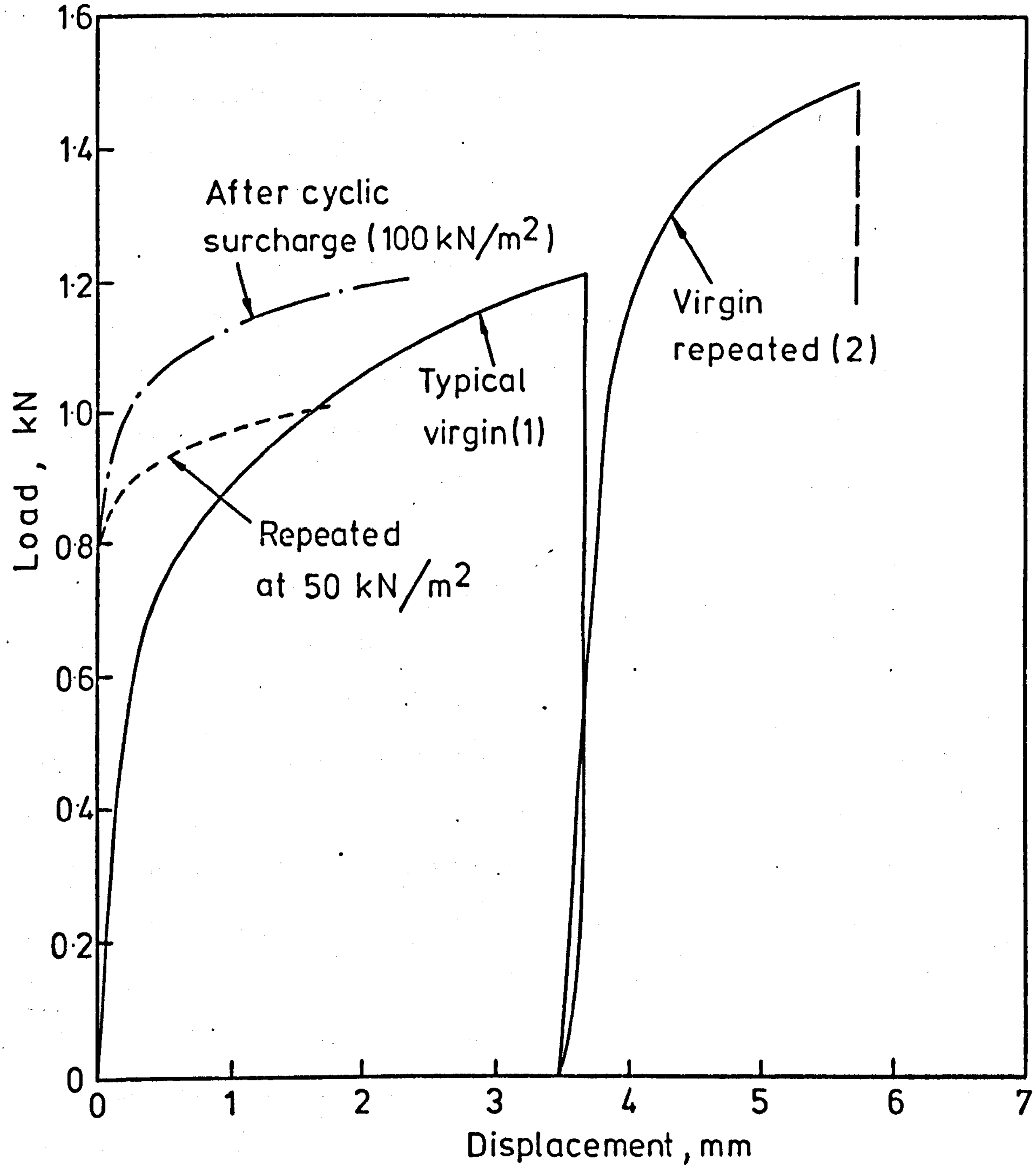


FIG. 7-12 LOAD – DISPLACEMENT BEHAVIOUR OF TENSION PILES

In Tests 45 and 46 the upper repeated load was in-phase with the higher surcharge pressure, 0.3Qc/100 whereas in Test 47 the upper repeated load was in-phase with the lower surcharge pressure 0.3Qc/50. The results of these three tests are presented in Figs. 7.13, 7.14 and 7.15 respectively. These results indicated that the pile experienced the same behaviour as that under static surcharge pressure. During the initial stage of the repeated loading the pile moved slowly into the sand but after a certain number of load cycles the rate of movement began to increase until it reached a maximum value beyond which it decreased as the number of load cycles was increased. The pile shaft load first increased and reached a peak value before decreasing to a limiting value.

To assess the effect of such a condition of loading on the performace of the pile, the movements of Test 45 and 47 together with those of Test 13 in which the surcharge pressure was static and equal to 100 kN/m^2 are drawn in Fig. 7.16. It is clear that the pile life-span increased when the surcharge was cycled and it was longest when the upper repeated load acted in-phase with the high cyclic surcharge pressure, 0.3Qc/100. This latter conclusion is in line with that reported by Al-Ashou (1981) and also with the analysis suggested by Madhloom (1978). The magnitudes of both the maximum rate of movement and the maximum shaft load were also affected by the state of the surcharge pressure as shown in Fig. 7.17. The value of the maximum rate decreased when the sand was subjected to cyclic surcharge and it was the smallest in the case of 100/0.3Qc. As demonstrated in this figure, the value of the maximum shaft load was largest in the case of 100/0.3Qc.

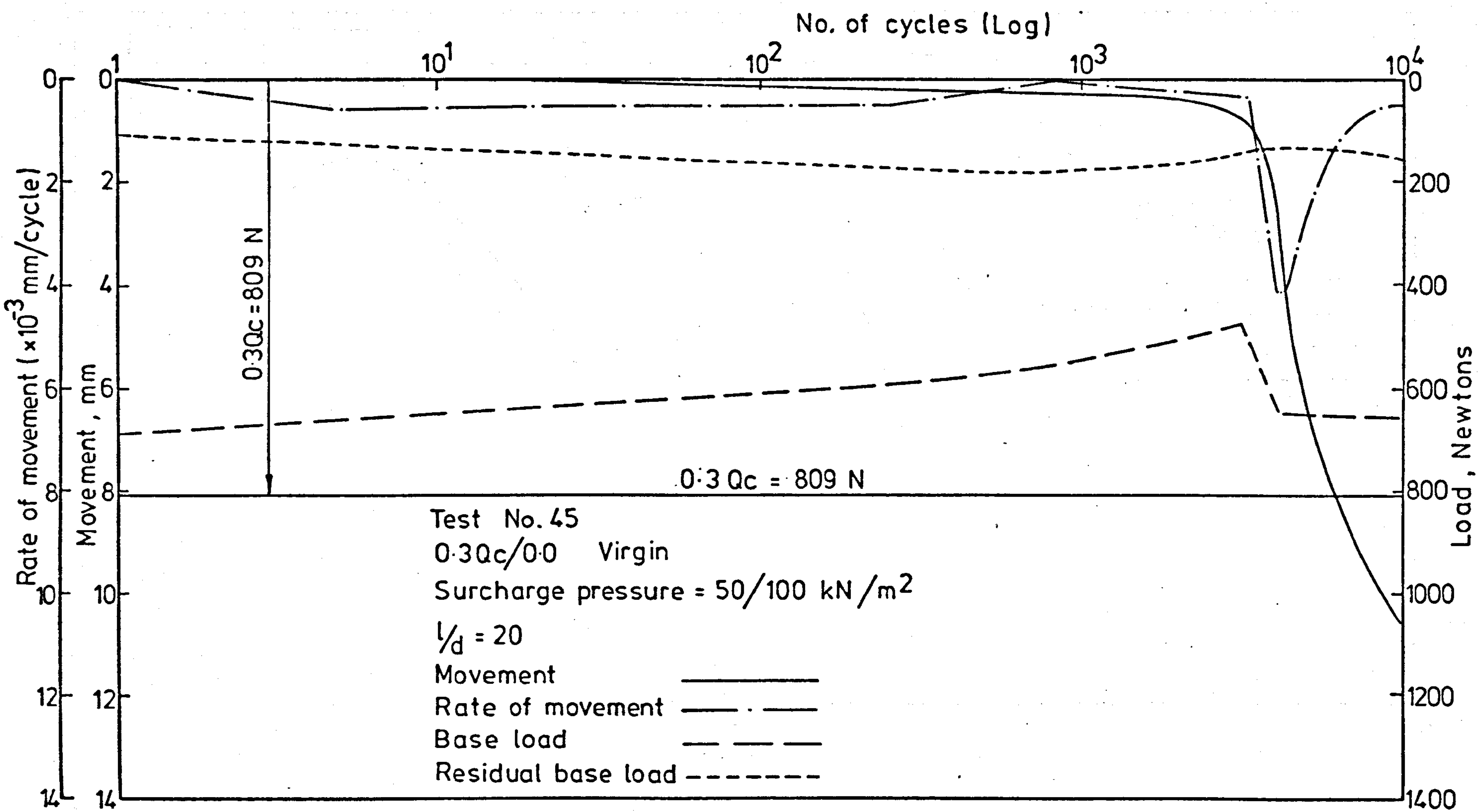


FIG. 7-13 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENTS WITH No. OF CYCLES

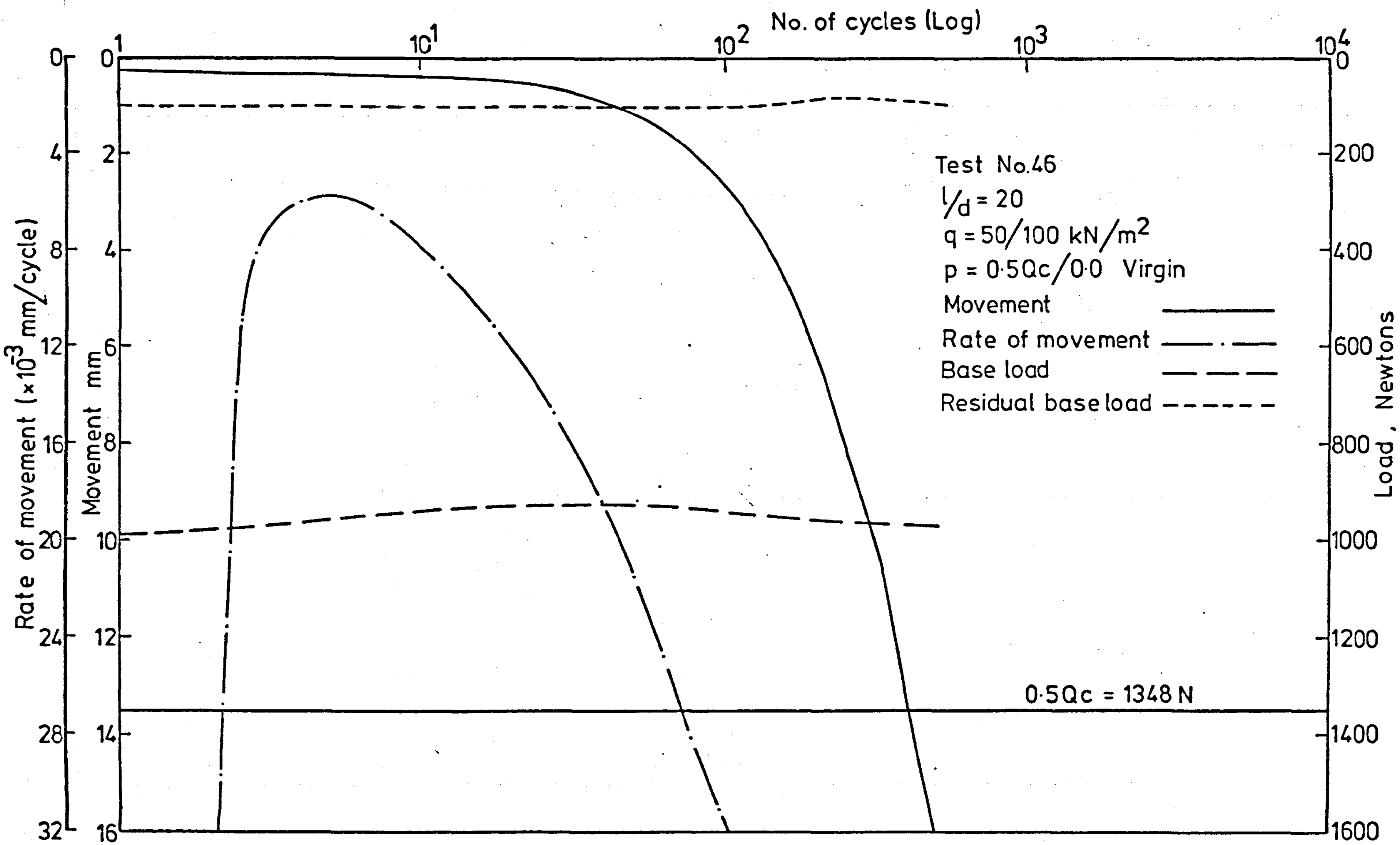


FIG. 7-14 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

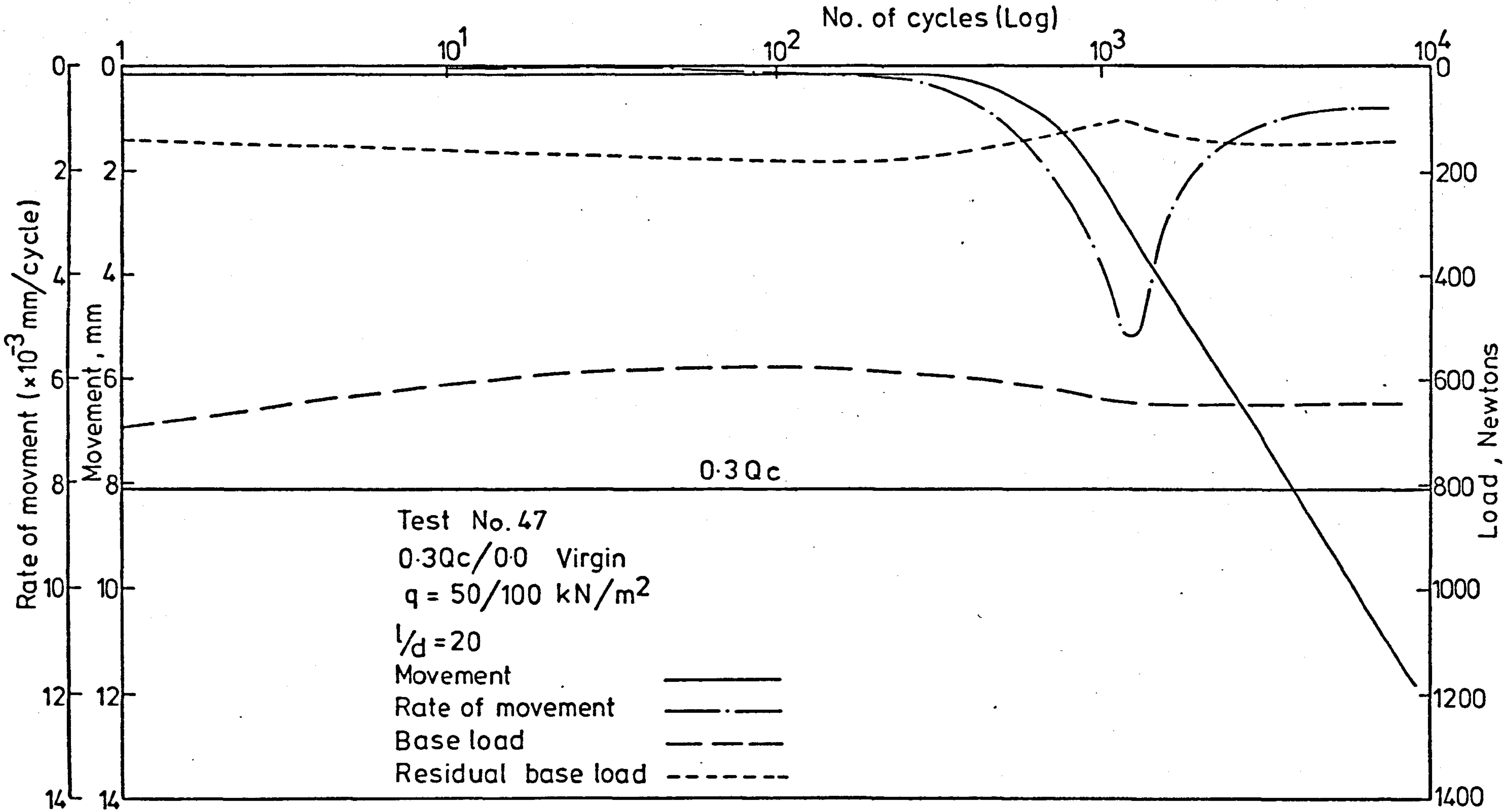


FIG.7-15 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENTS WITH No. OF CYCLES

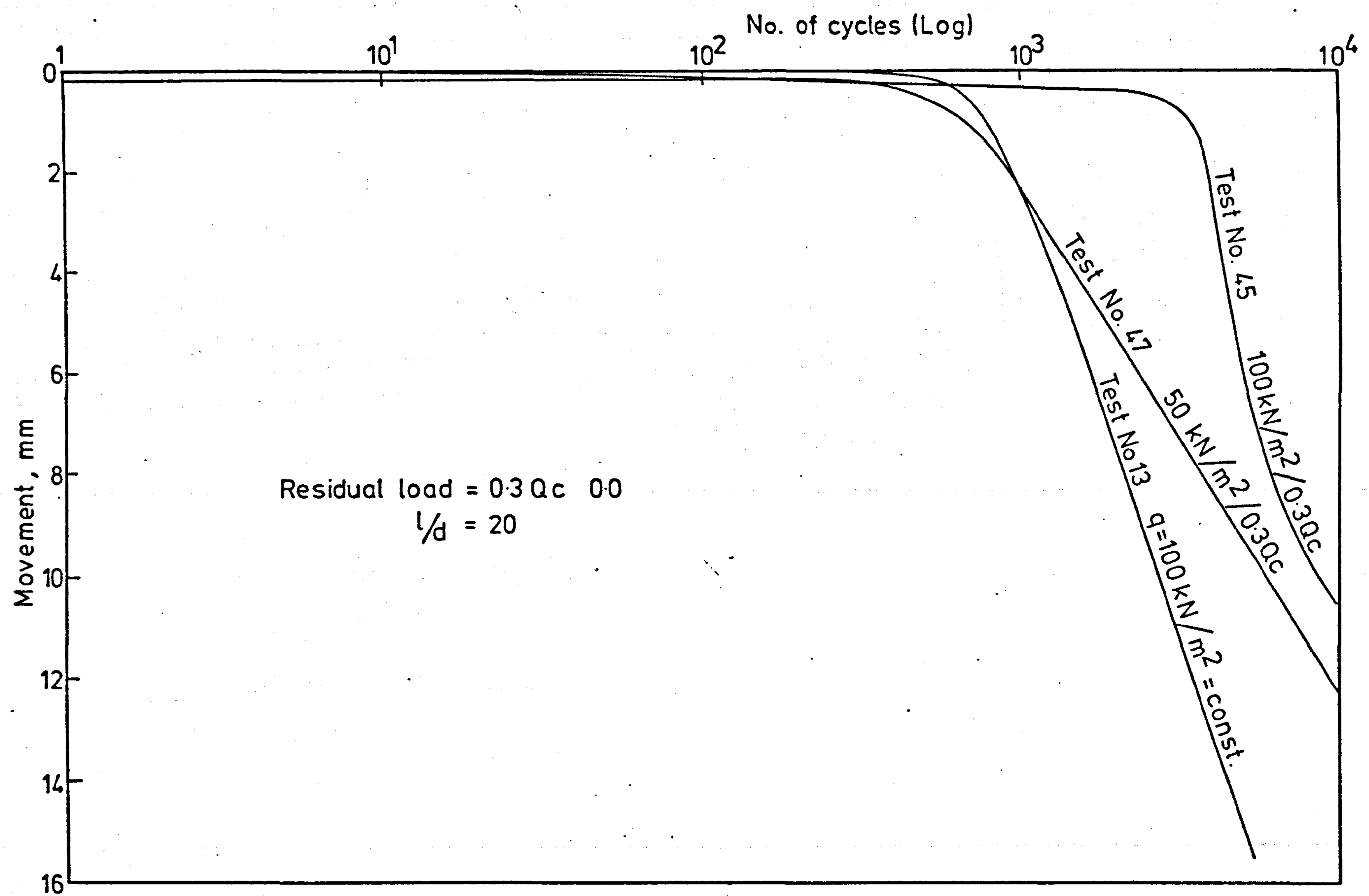


FIG. 7-16 INFLUENCE OF CYCLIC SURCHARGE ON THE LIFE-SPAN OF A PILE SUBJECTED TO REPEATED LOADING

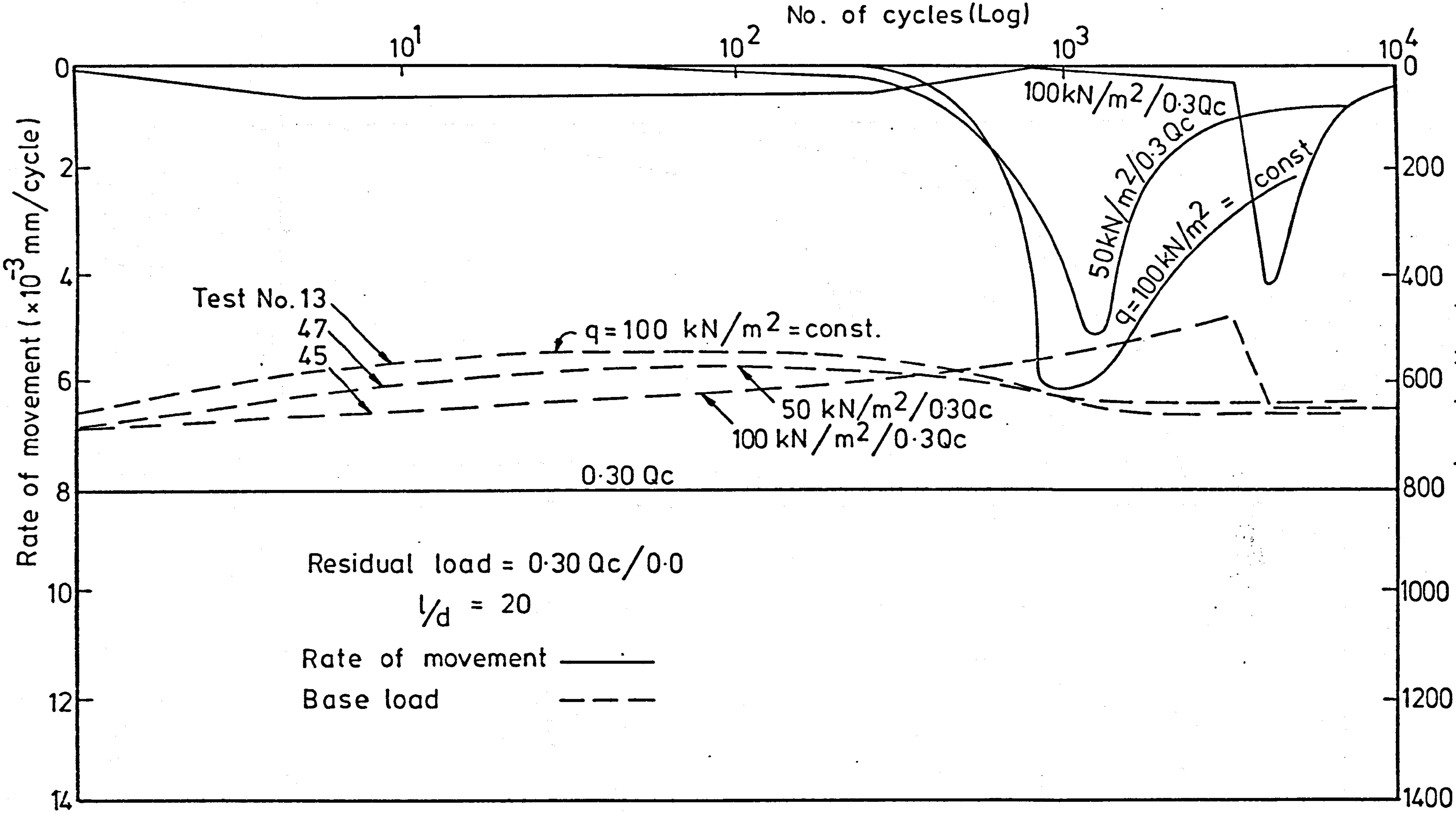


FIG. 7·17 VARIATION OF LOADS AND RATES OF MOVEMENT WITH No. OF CYCLES

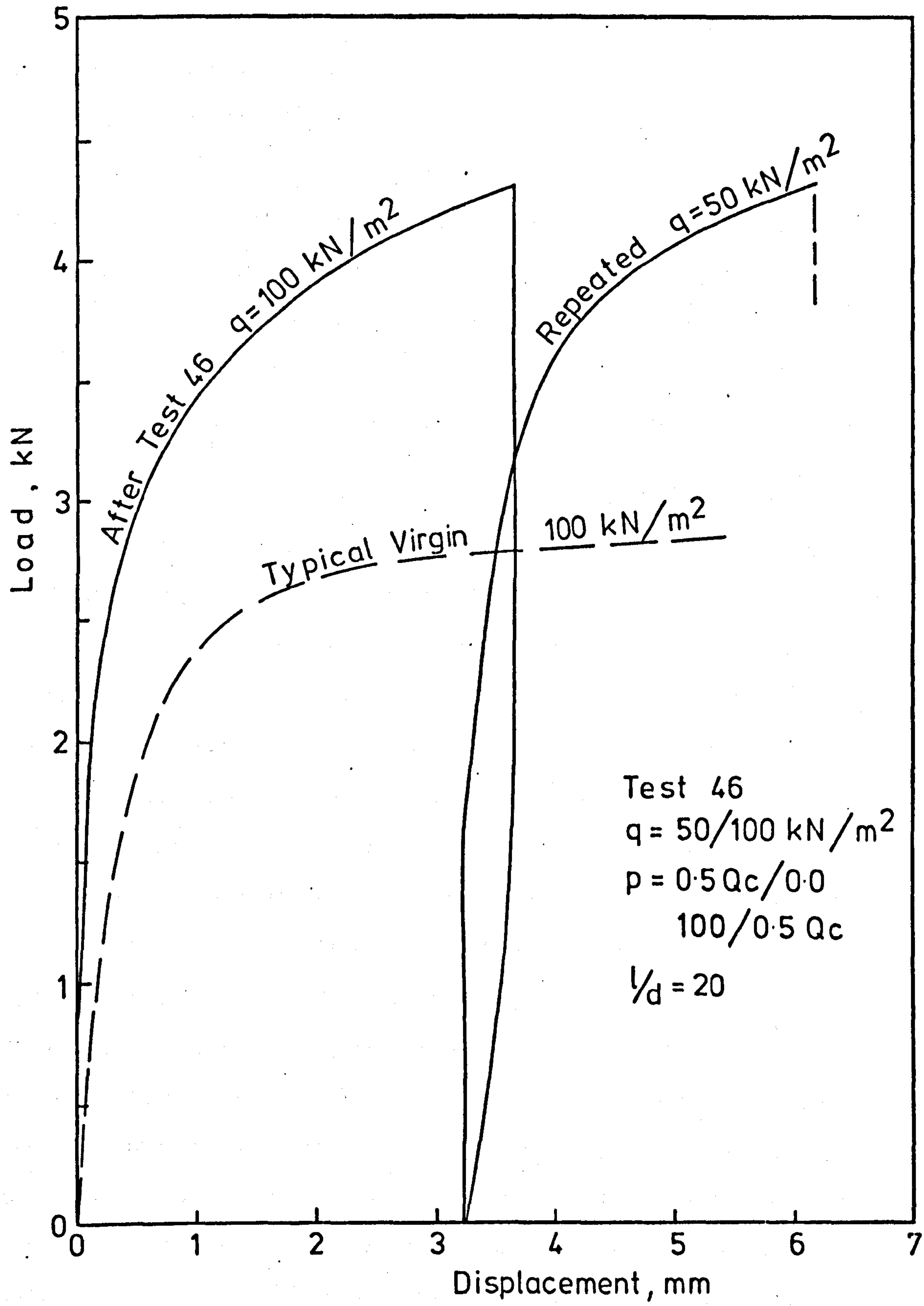


FIG. 7-18 INFLUENCE OF CYCLIC SURCHARGE ON THE LOAD-DISPLACEMENT BEHAVIOUR OF PILES

During the last stage of the repeated loading, the shaft load reached a value which appeared to be independent of the state of the surcharge pressure.

Fig. 7.18 confirmed the previously stated conclusions which indicated that the ultimate capacity of the pile increased after being tested with cyclic surcharge and that the capacity at 50kN/m^2 static surcharge pressure was very close to that at 100 kN/m^2 pressure.

7.4 Series X: Influence of surcharge and pile loading frequencies on pile behaviour.

Based on the results of tests carried on sand samples tested in a simple shear apparatus and subjected to cyclic shear-strain, Youd (1972) reported that there was little or no effect on the amount of compaction when the frequency increased up to 115 c.p.m. Similar tests conducted by Lee and Focht (1975) revealed that the cyclic loading shear strength of sand was slightly affected when the frequency increased up to 10 H_z. Because the stress-paths and the strain-path involved in the case of a pile subjected to repeated loadings or tested with cyclic surcharge are different from that of a simple shear condition it was decided to check whether the frequency had any effect on the performance of the pile. In this respect, three tests were conducted, namely Tests 48, 49 and 50. In the first two tests the influence of the cyclic surcharge frequency on the behaviour of a pile at a depth ratio of 20 and subjected to 0.3Qc/0.0 repeated loads was examined. In the latter test the surcharge pressure was maintained constant and equal to 100 kN/m^2 while the effect of the repeated loading frequency on the behaviour of a pile had 30 diameters depth was investigated.

The variations of movement and rate of movement with logarithm of the number of surcharge cycles of Test 48 and 49 are shown in Fig. 7.19. In spite of the frequency in Test 49 being two times that in Test 48 but the difference between the movement and the rate of movement of the two tests was relatively small. However, these minor differences indicated that after any number of surcharge cycles a smaller amount of movement resulted from the low frequency test. The measurements of base loads of the two piles during the two phases of the cyclic surcharge, Fig. 7.20 also revealed minor differences. More improvement in the pile shaft resistance was observed during the test of the low frequency. The difference in pile behaviour of these two tests may be attributed to the fact that with low frequency the sand grains had a longer time to arrange into a more compacted and stable structure.

In contrast, when the frequency of the repeated loading was increased from 1.0 c.p.m. in Test 11 to 3 c.p.m. in Test 50 the pile movement was slightly larger during the test of low frequency as shown in Fig. 7.21. The pile base load did not alter significantly when the frequency increased three times. However, it may be concluded that from the practical point of view and within the tested limits, the frequency of both the cyclic surcharge and the repeated loading did not affect significantly neither the movement nor the load transfer characteristics of the pile.

7.5 Series X1: Various combinations of loading and different states of surcharge pressure.

This section deals with the results of Test Series X1 which consists of five main tests. In these tests the performance of the pile was examined under various load histories and with cyclic and/or static surcharge pressure. During the first test in this

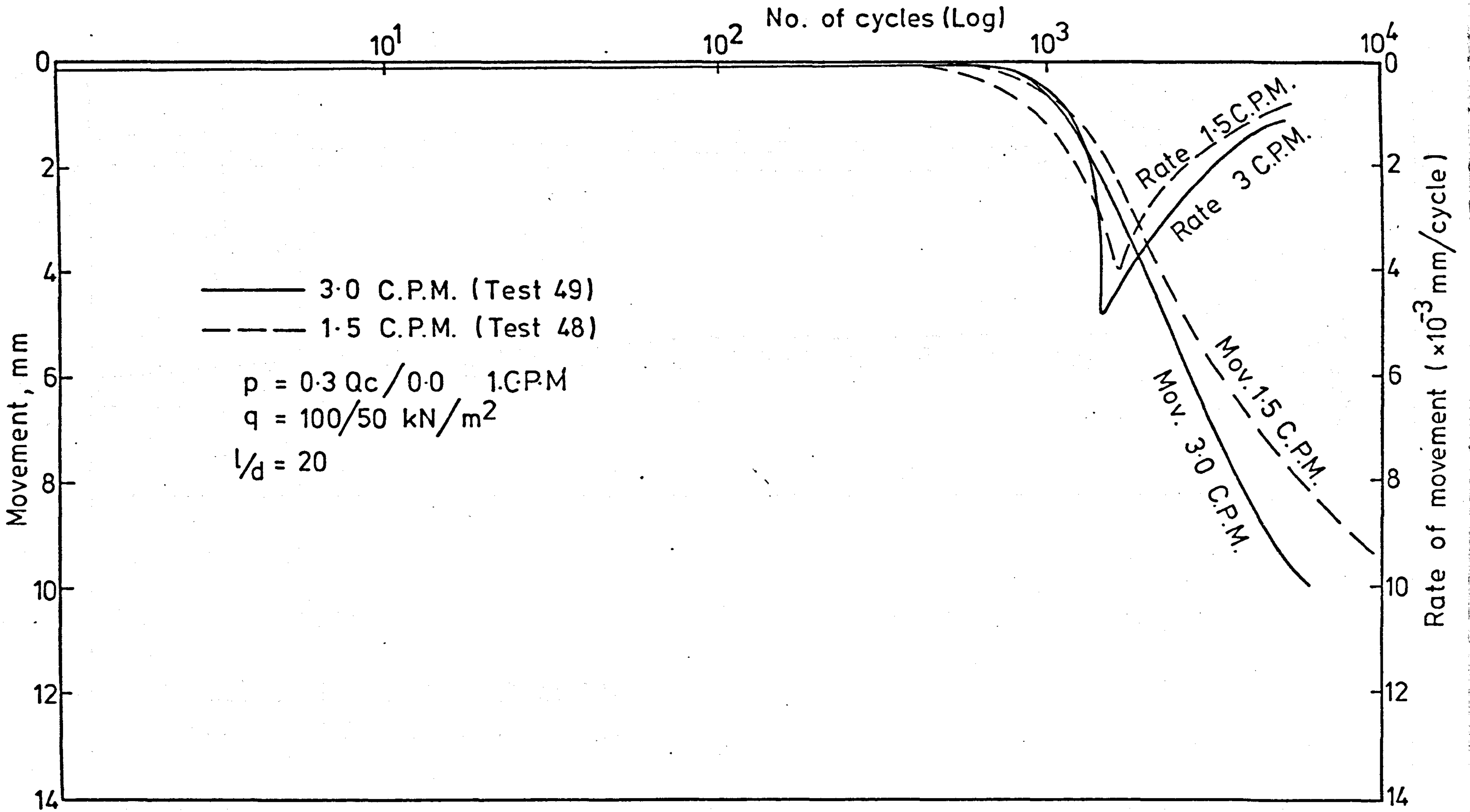


FIG.7-19 INFLUENCE OF FREQUENCY ON MOVEMENT AND RATES OF MOVEMENT

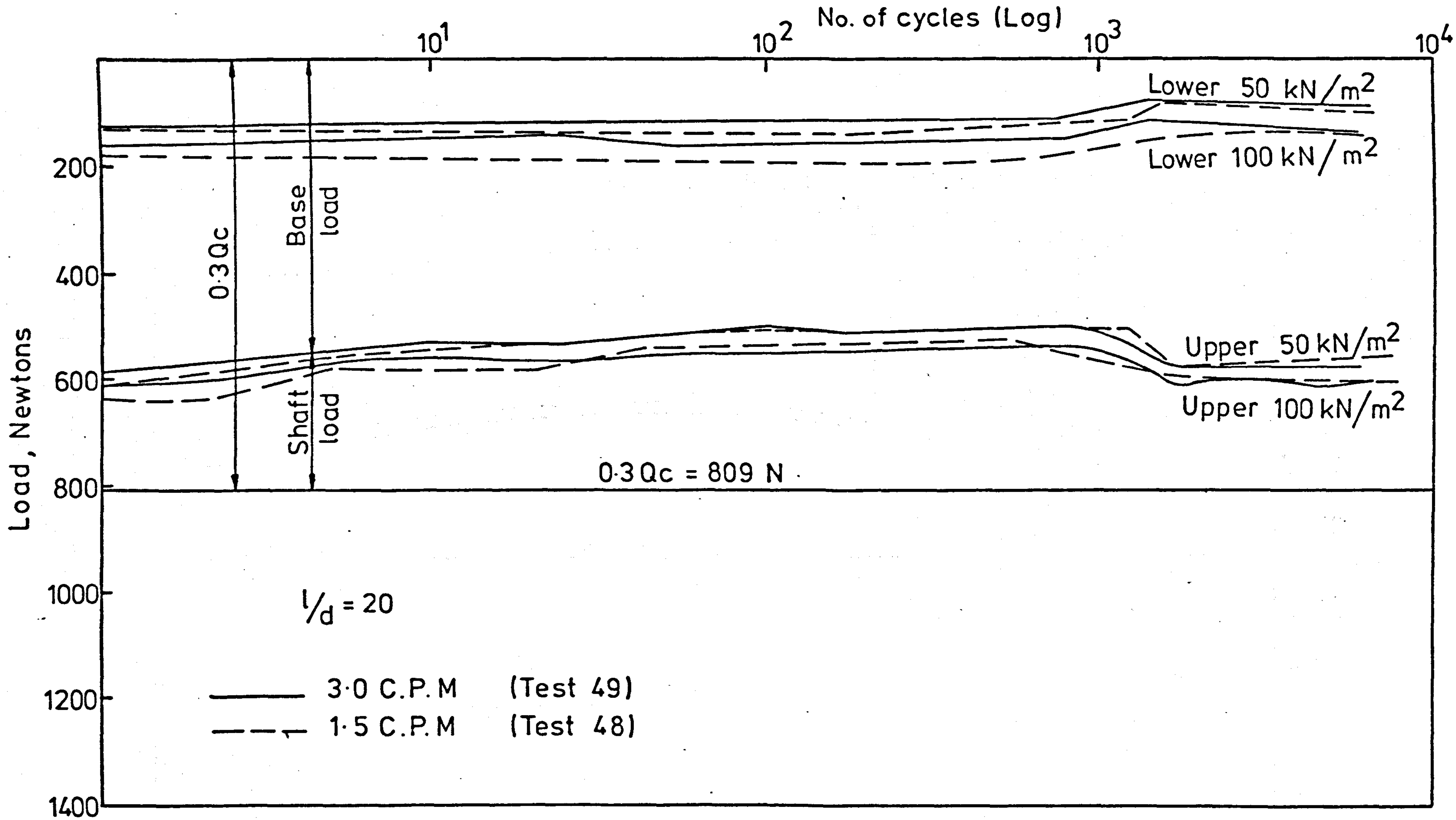


FIG. 7-20 INFLUENCE OF FREQUENCY ON THE LOAD-TRANSFER BEHAVIOUR

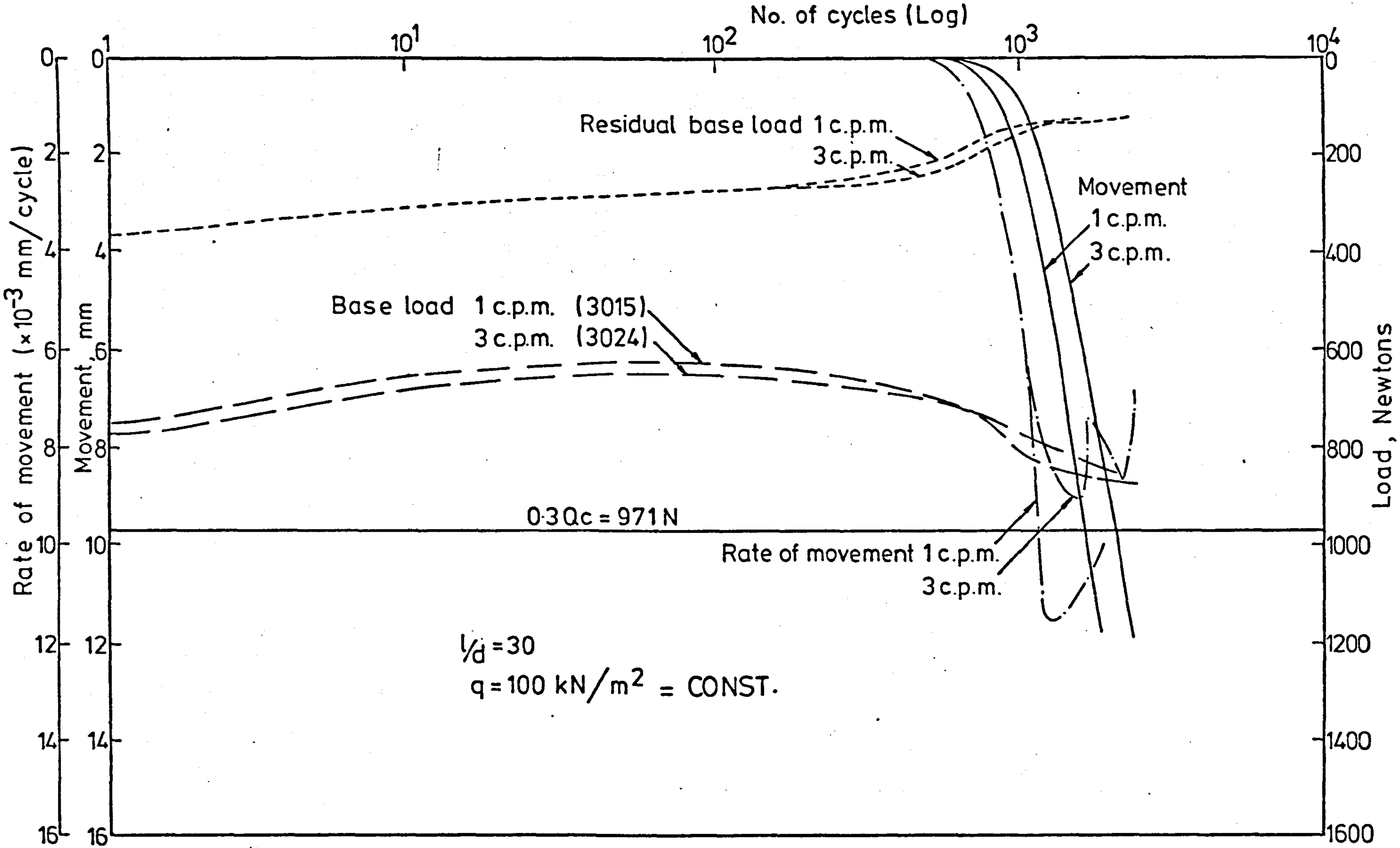


FIG. 7-21 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

series, Test 52 the pile, which was embedded at 20 diameters depth, was subjected to 0.5Qc/0.0 repeated loading and after the rate of movement had reached a maximum value and began to decrease, repeated loads of 0.3Qc/0.0 were then applied. During testing the surcharge pressure was cycled between 50 and 100kN/m^2 so that the high pressure was applied in-phase with the upper repeated load. When the results of the latter part of the test are compared with those of Test 45 it will be seen, Fig. 7.22 that the life-span of the previously loaded pile is much longer than that of the virgin pile, Test 45. The value of the maximum shaft load was larger in the test of the virgin pile while the limiting value appeared to be the same for both piles. These two conclusions are in line with those observed under constant surcharge pressure in section 6.6.

In Test 53 the pile was subjected to the following sequence of loading:

53 - a	0.3Qc/0.0
53 - b	0.5Qc/0.0
53 - c	0.7Qc/0.0
53 - d	0.7Qc
53 - e	0.9Qc

The surcharge pressure in all these parts of the test was cycled every 20 seconds. The results of part 53-b together with those of Test 46 are plotted in Fig. 7.23 as a function of the logarithm of the number of load cycles. As in the case of static surcharge, Fig. 6.35, when the repeated load became as high as 0.5Qc the virgin pile exhibited a longer life-span. During the initial stages of repeated loading the shaft load of the virgin

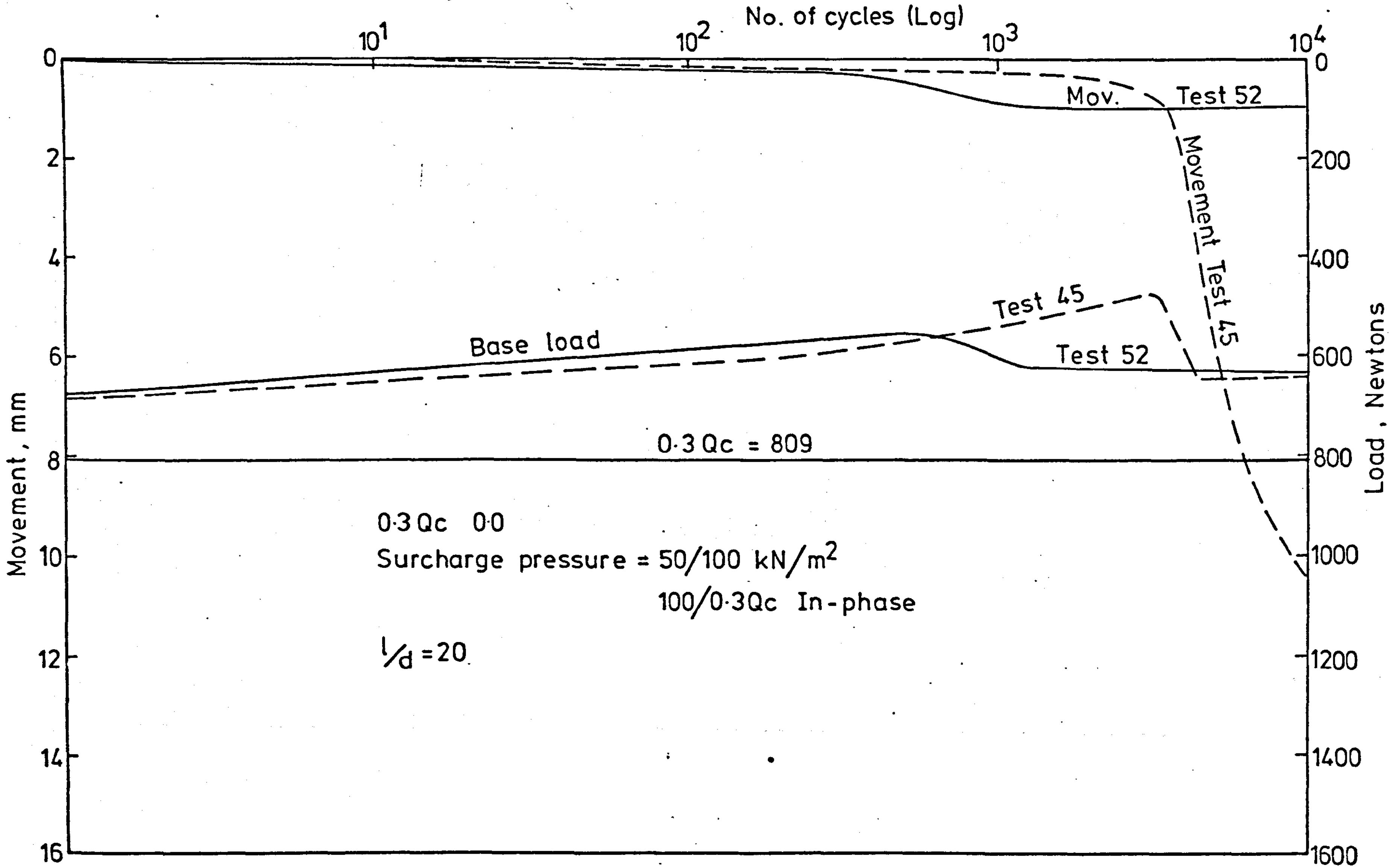


FIG. 7-22 INFLUENCE OF PREVIOUS LOADING ON THE BEHAVIOUR OF PILES

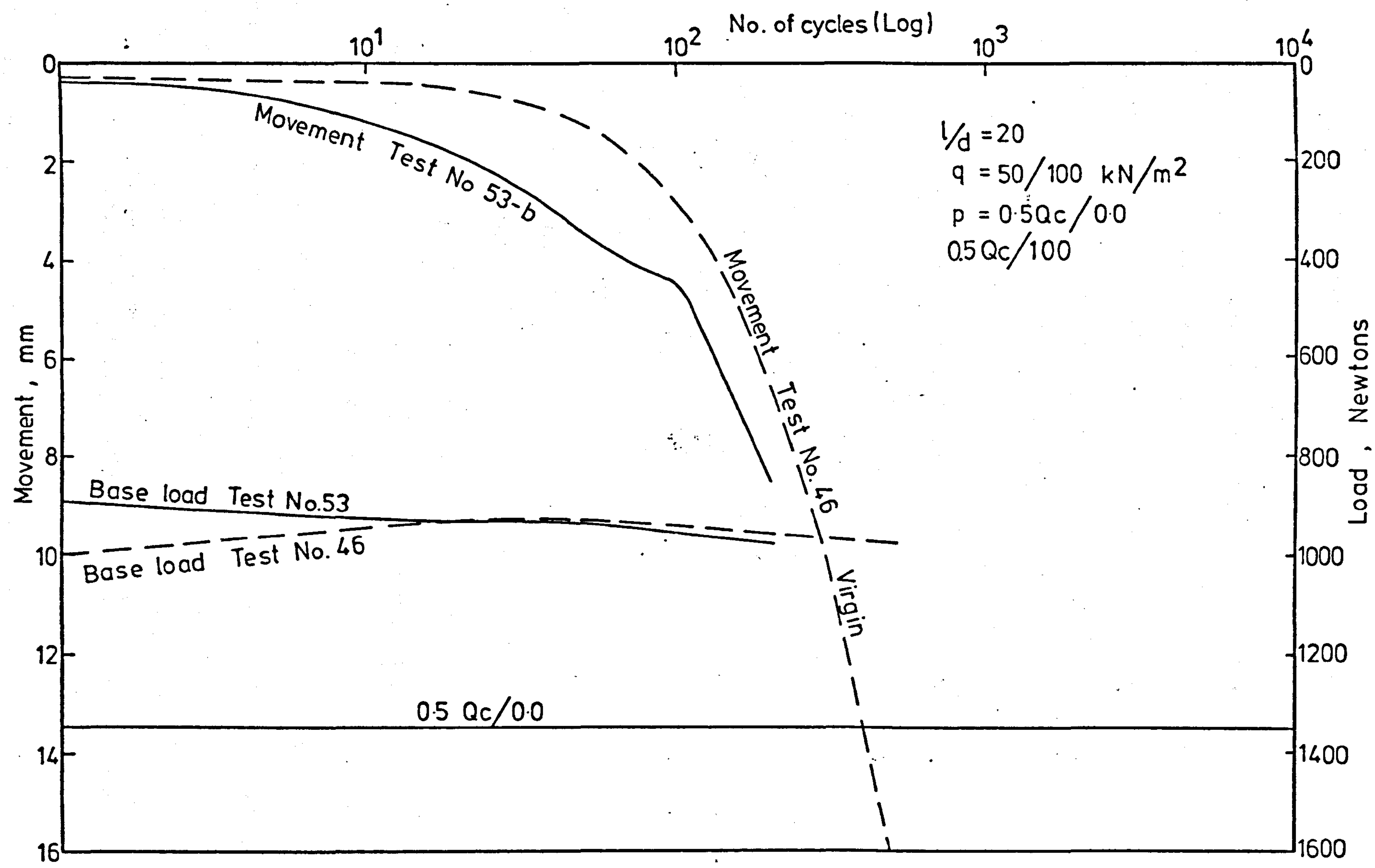


FIG. 7-23 INFLUENCE OF PREVIOUS LOADING ON THE BEHAVIOUR OF PILES

pile, Test 46, was smaller than that of the previously loaded pile, but during the last stage the shaft load of both piles reached approximately the same value.

Fig. 7.24 compares the movements of part 53-d with that of Test 43 in which the virgin pile was first loaded with 0.7Qc. It will be seen that the virgin pile experienced smaller settlement at the beginning of the test. During the cyclic surcharge the movements of the virgin pile were always smaller than those of part 53-d.

The results of part 53-e shown in Fig. 7.25 indicate that even under a high static load level, the pile moved at a decreasing rate. Initially, the pile settled 1.5mm and after 7043 cycles the settlement did not exceed 2.7mm.

Test 54 was directed to a study of the influence of cyclic surcharge on the performance of a pile subjected to repeated tensile loads. During testing the following sequences of loading were applied:

(54 - a) 0.3Qt/0.0

(54 - b) 0.5Qt/0.0

(54 - c) 0.5Qt

(54 - d) 0.5Qt/0.0

The surcharge pressure was cycled between 50 and 100 kN/m^2 during all these parts. In part (54-a) the pile was only subjected to 8700 load cycles. At the end of this part the pile was in a stable state. For comparison the movement and its rate of this part together with those of Test 22, in which the surcharge pressure was constant and equal to 100 kN/m^2 are presented in Fig. 7.26. It can be seen that when the pile in Test 22 had failed, the pile in part (54-a) was still in a stable condition. This result confirmed the previously stated conclusion that cyclic surcharge increased the pile life-span.

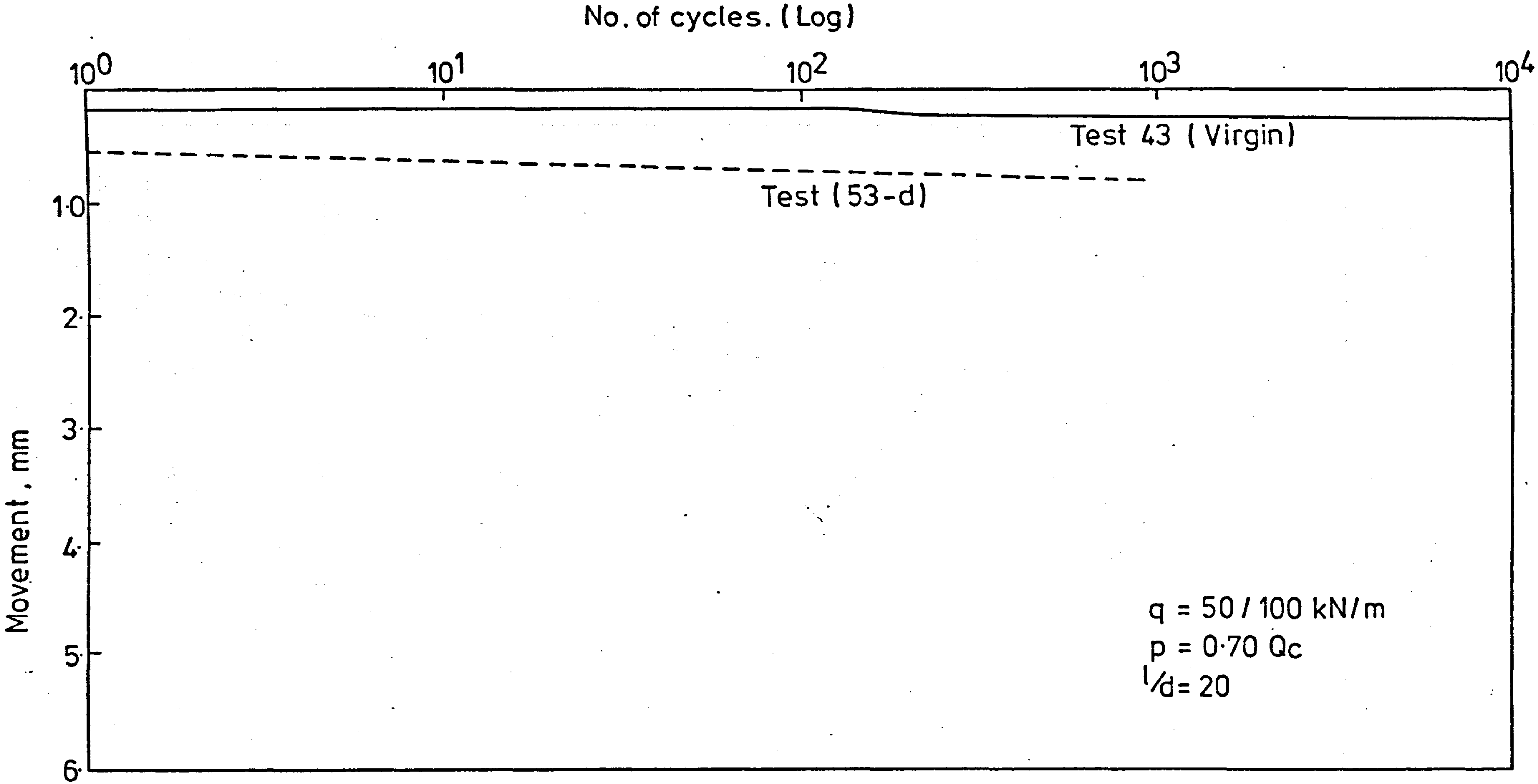


FIG. 7-24. VARIATION OF MOVEMENT WITH LOG NO. OF CYCLES.

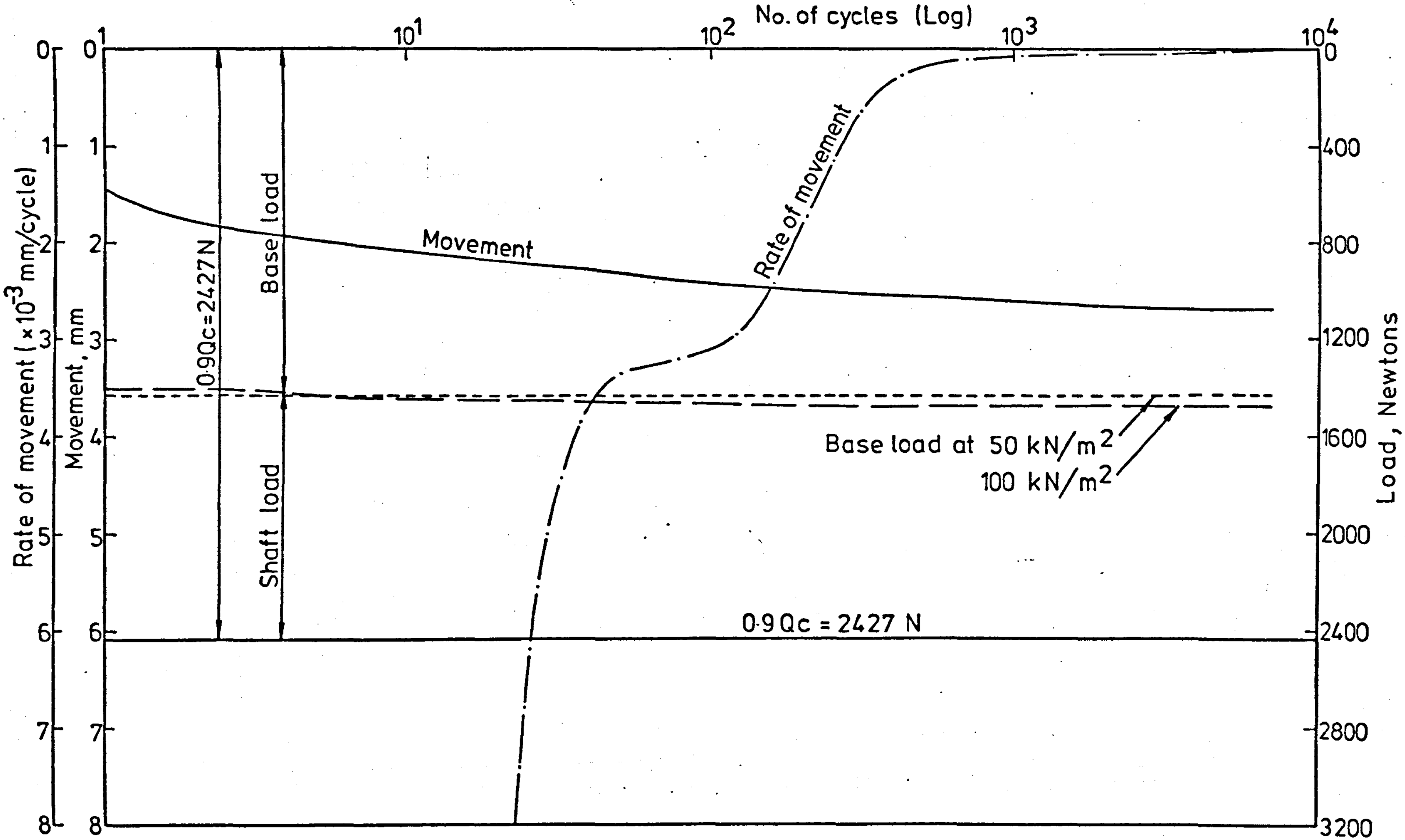


FIG. 7-25 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

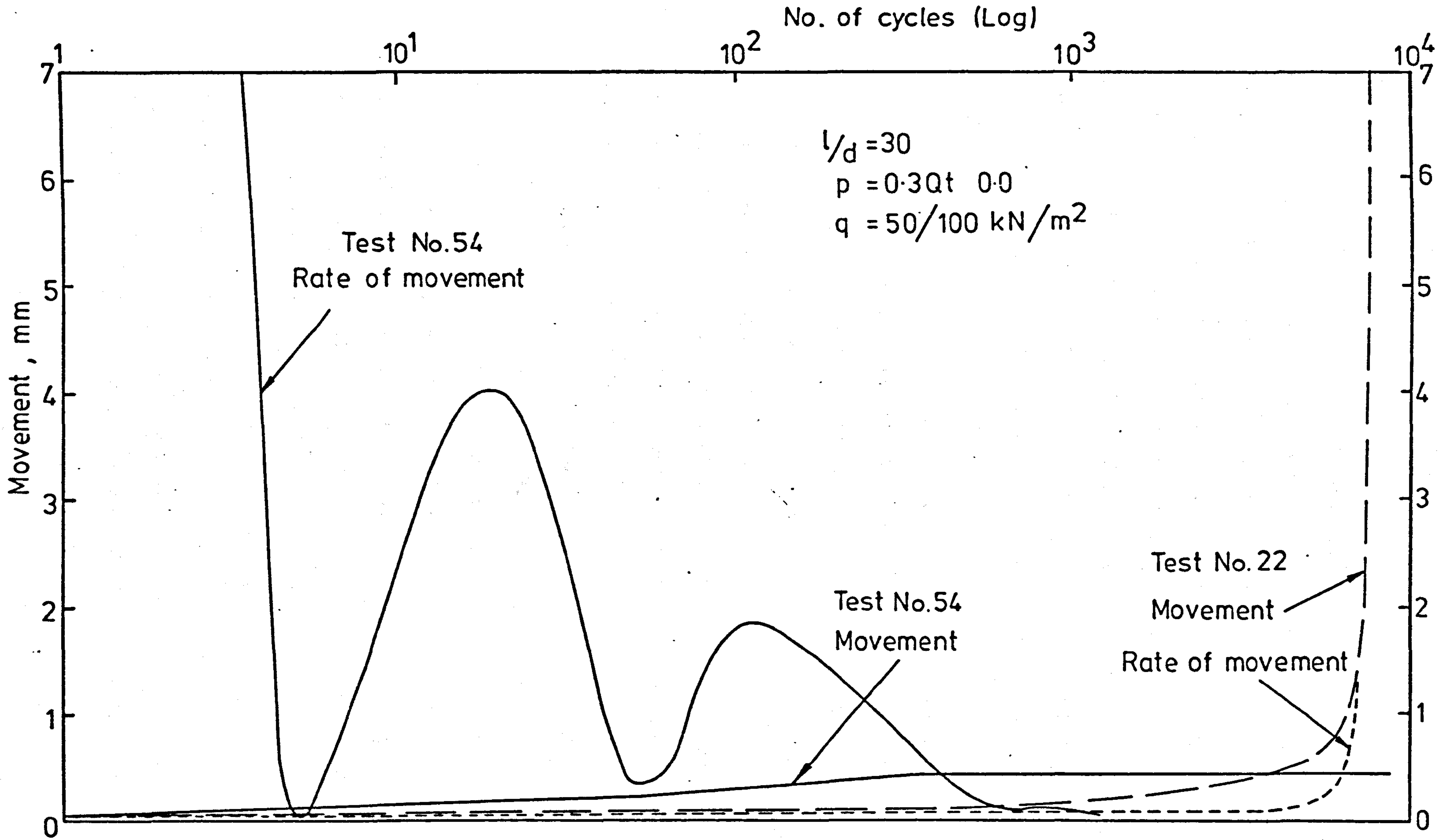


FIG.7-26 INFLUENCE OF CYCLIC SURCHARGE ON THE LIFE SPAN OF PILES

When the pile was subjected to 0.5Qt/0.0 repeated load, part (54-b), the movement increased at a decreasing rate. After cycle number 60 the rate of movement increased rapidly with increase in the number of load cycles. At cycle number 430, the last cycle in this part, the pile had pulled out a distance of 10.74mm at a rate of 0.043mm per cycle, as represented by points B_1 and B in Fig. 7.27. The pile loading was then held constant and equal to 0.5Qt whereas the cyclic surcharge was continued, part (54-c). After 1900 surcharge cycles the pile loading was then repeated, part (54-d). It was found that the rate of movement reversed its trend and began to decrease as the number of load cycles was increased. When the test was terminated after 100 load cycles, the rate of movement had decreased from 0.043 to 0.017mm per cycle as shown by point C.

The distribution of skin friction along the pile shaft during the first and the last cycle of each part of Test 54 is shown in Fig. 7.28. In part (54-a) the skin friction along the upper half increased whereas along the lower half it decreased which is similar to that observed in section 7.2. During part (54-b) where the loading was 0.5Qt/0.0 the reverse trend was observed. The 1900 cycles of surcharge pressures, part (54-c), caused an improvement in friction along the upper half of the pile. This improvement again deteriorated when the pile was subjected to repeated loading of 0.5Qt/0.0, part (54-d).

Fig. 7.29 shows the results of Test 55 in which the influence of cyclic surcharge on the behaviour of a pile subjected to compressive repeated loads and embedded in a sand upon which a static surcharge pressure of 100 kN/m^2 acted.

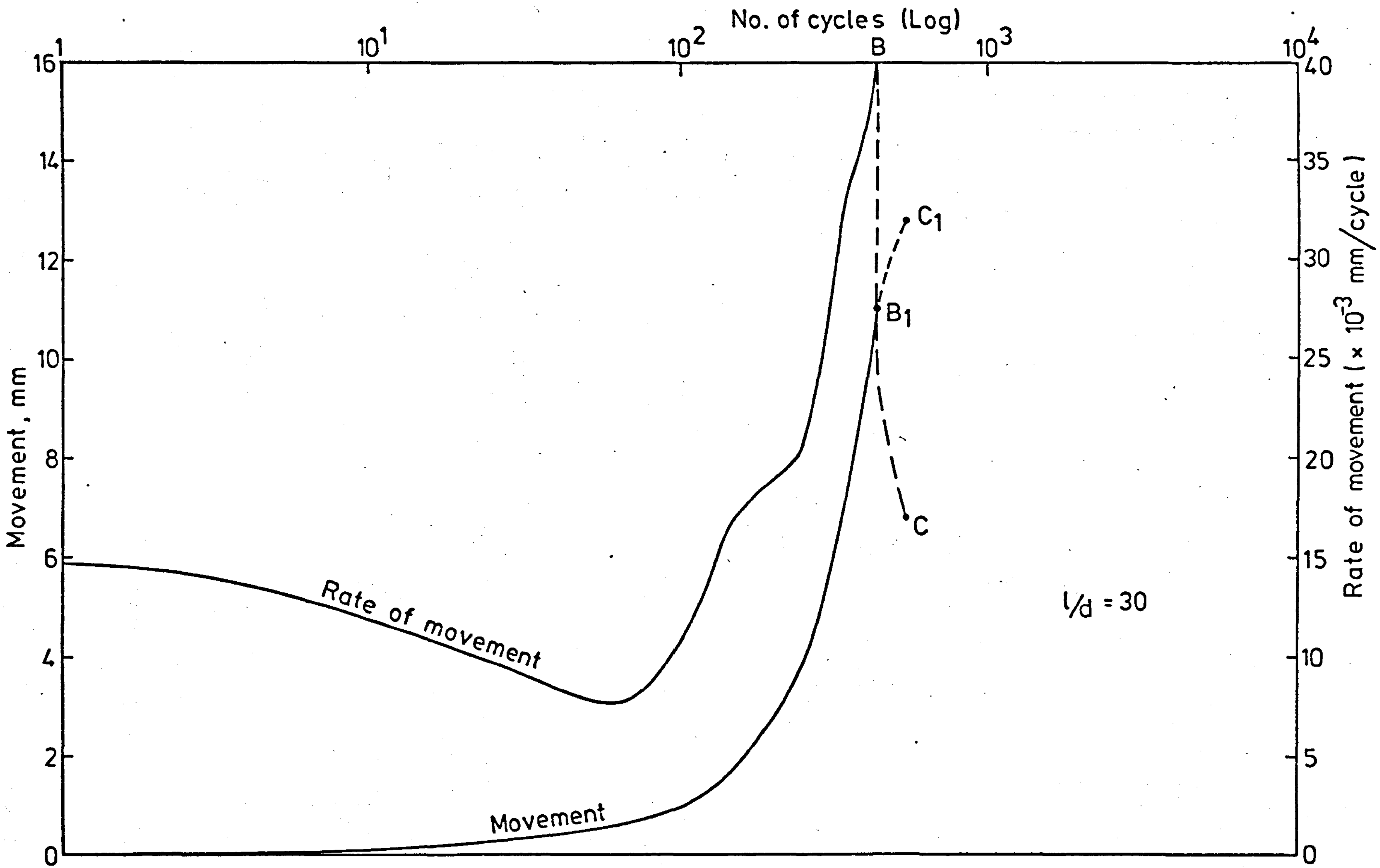
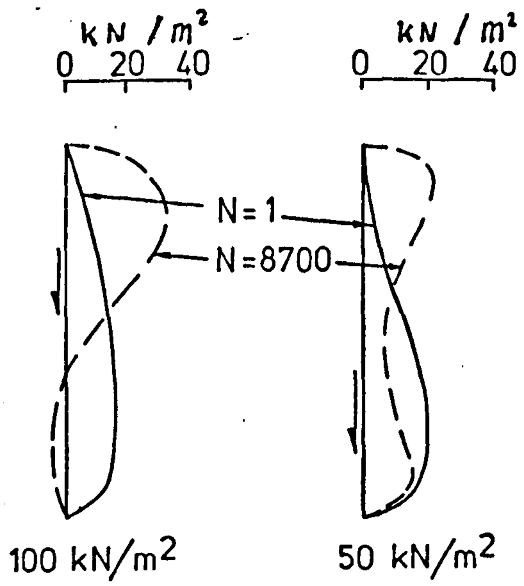
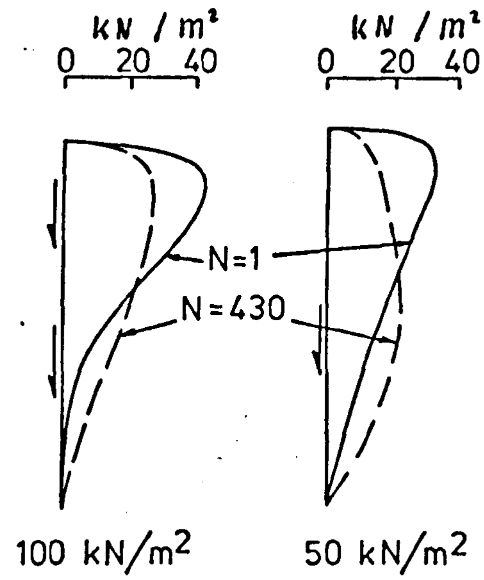


FIG. 7-27 VARIATION OF MOVEMENT AND RATE OF MOVEMENT WITH No. OF CYCLES



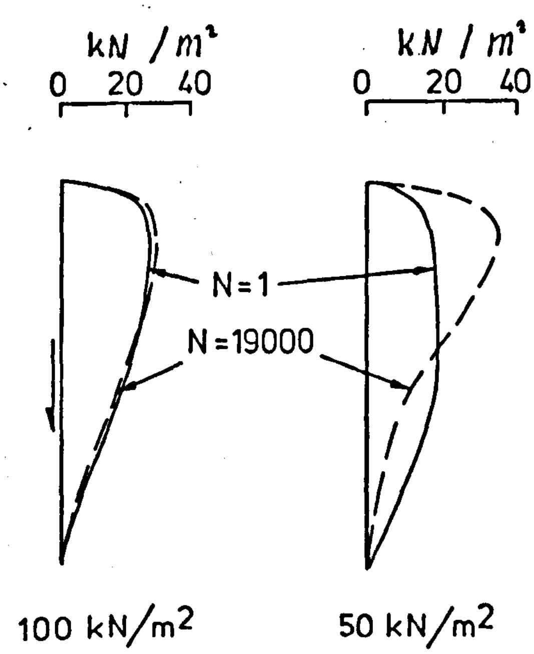
(a) 0·3Qt/0·0

During the upper repeated load level

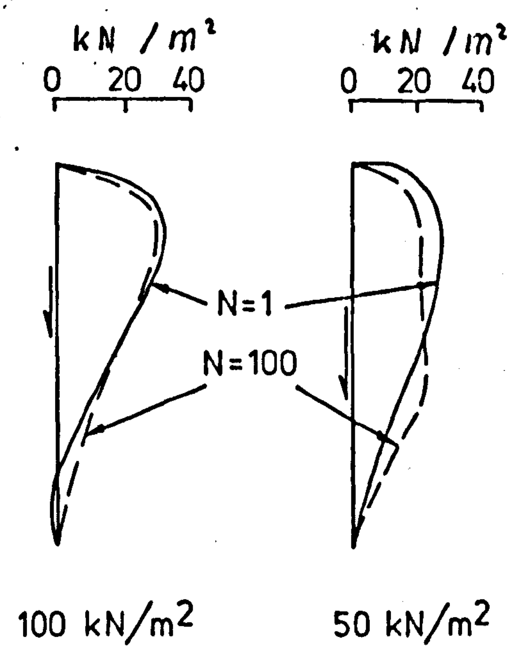


(b) 0·5Qt/0·0

During the upper repeated load level



(c) 0·5Qt



(d) 0·5Qt/0·0

During the upper repeated load level

FIG. 7-28 DISTRIBUTION OF SKIN FRICTION (l/d = 30 , TEST No.50)

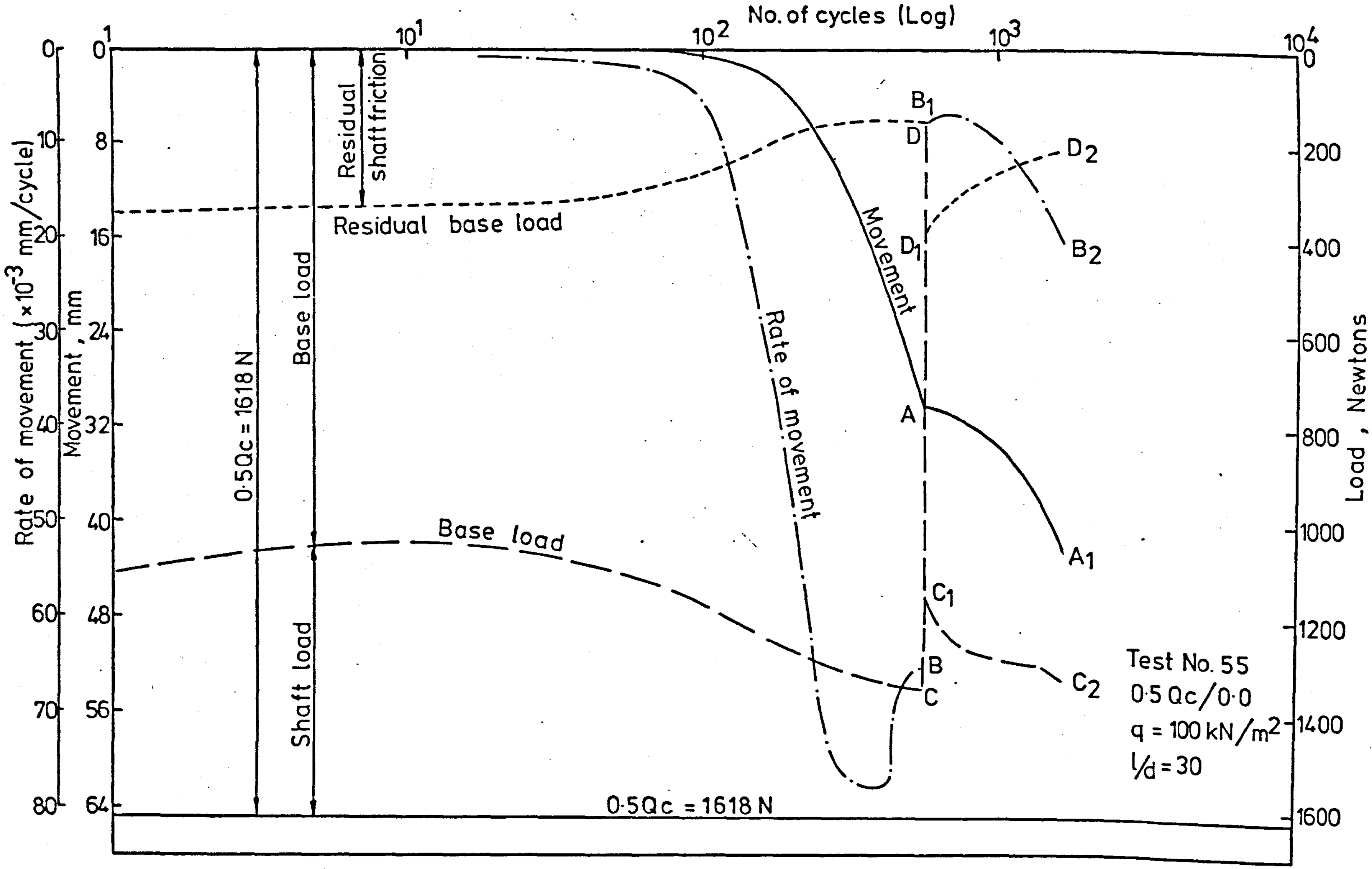


FIG.7-29 VARIATION OF LOADS MOVEMENTS AND RATES OF MOVEMENT WITH No. OF CYCLES

The pile was first subjected to 0.5Qc/0.0 loading and after the rate of movement had reached its maximum value and began to decrease the load was held constant and equal to 0.5Qc. After 2300 cycles with the surcharge cycled between 50 and 100 kN/m^2 the pile loading was then repeated while the surcharge pressure was held constant and equal to 100 kN/m^2. As illustrated in Fig. 7.29, when the surcharge was cycled the shaft load and its residual value increased as represented by the path C-C_1 and D-D_1 respectively. During the last part of the test, when the pile loading was repeated these values decreased from C_1-C_2 and from D_1-D_2 respectively. On the other hand, a sudden change in trend of the pile movement resulted after the cyclic surcharge as represented by the path A-A_1. The corresponding rate decreased from 0.065 to 0.007mm per cycle, B-B_1, then began to increase during the last part of the test until it reached 0.020, point B_2, at the end of the test.

Cyclic surcharge loading also improved the performance of a pile subjected to a tensile repeated load of 0.5Qt/0.0 and with a constant surcharge pressure of 100 kN/m^2, Test 56, as shown in Fig. 7.30. During the first part, after the pile had pulled out 18.71mm at a rate of 0.083mm per cycle the load was then held constant and equal to 0.5Qt. After 1300 cycles of cyclic surcharge was appllied to the sand surface the pile loading was then repeated. It was found that the rate of movement decreased from 0.083 to zero mm/cycle as shown by the path B-B_1. The pile behaviour, therefore, entered another stable stage which lasted more than 2700 cycles before the rate began to increase. When this number of cycles compared with that of the first part, 1300 cycles, it will be found that the pile life-span had increased by

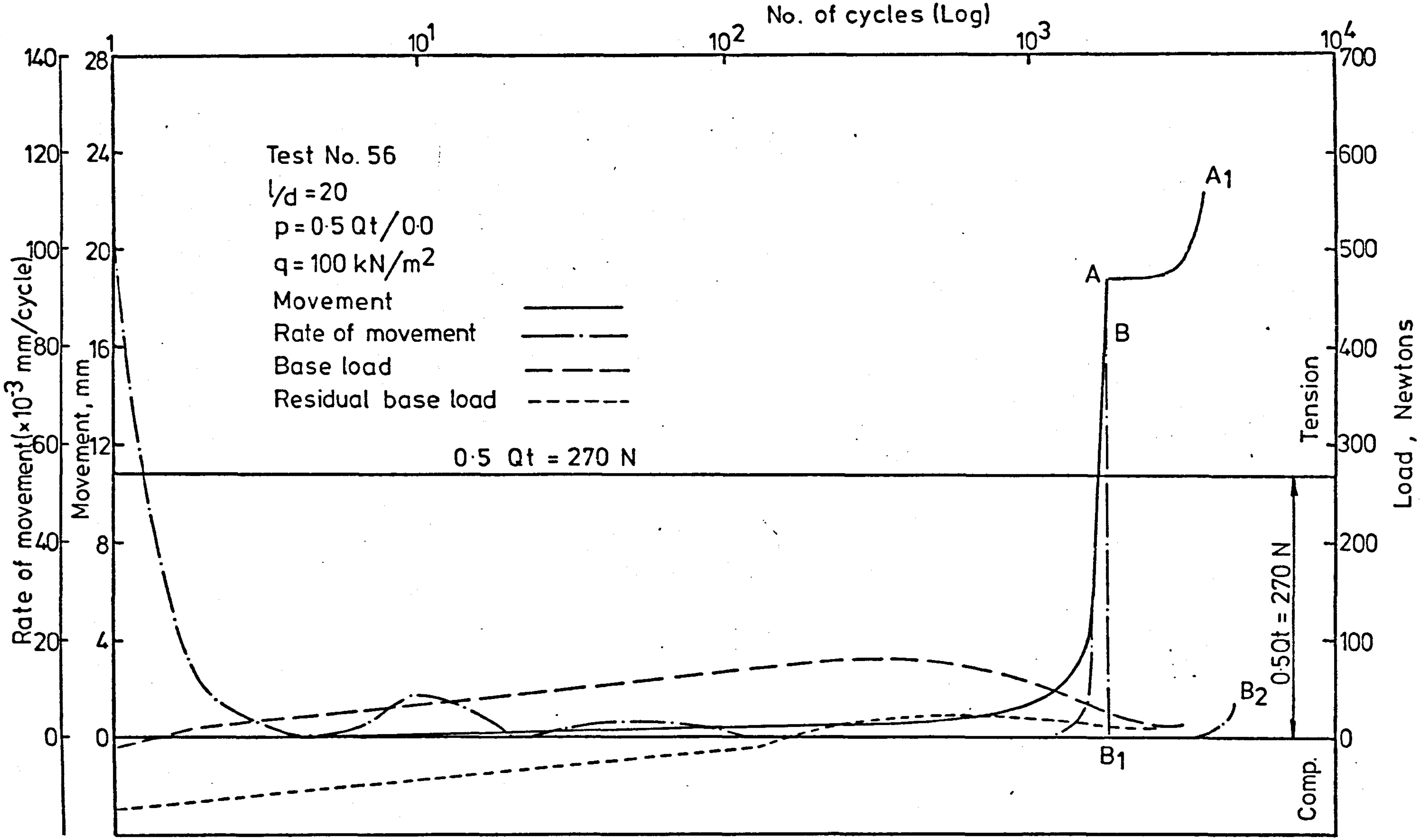


FIG. 7-30 VARIATION OF LOADS, MOVEMENTS, AND RATES OF MOVEMENT WITH No. OF CYCLES

more than two fold.

7.6 Conclusion

The following conclusions may be drawn from the results of the tests performed with cyclic surcharge pressure:-

(1) When the pile is subjected to static loading and a cyclic surcharge acted upon the sand surface the following were observed:

(a) For compression piles, the movement increased at a decreasing rate until it reached an approximately constant value. For a given percentage of loading, the pile movement increased when the depth of embedment increased. Due to cyclic surcharge the axial load at any given point along the pile depth increased. This increase was greater along the upper part of the pile. After a certain number of surcharge cycles, the axial loads of points located along the upper part of the pile became larger than the applied pile loading and thus a negative skin friction developed along the pile at these points.

(b) For tension piles, the pile movement first increased at a decreasing rate then, after a stable stage, the rate of movement increased very rapidly until failure occurred. Cyclic surcharge loading caused an increase in the negative skin friction along the upper elements of the pile and a decrease in that of the lower elements.

(2) From repeated loading tests, the following conclusions were reached :-

(a) The behaviour of piles subjected to repeated loadings with cyclic surcharge was similar to that of identical piles but tested with a static surcharge pressure. For compression piles, the rate of movement after having reached a maximum value decreased when the number of load cycles was increased. For tension piles, the rate of movement, after a stable stage, increased rapidly until failure occurred.

(b) The cyclic surcharge resulted in a pile of longer life. This life was longest when the upper repeated load acted in-phase with the higher surcharge pressure.

(c) For compression piles, the value of the maximum rate of movement decreased when the surcharge pressure was cycled. The smallest value was observed when the upper repeated load was in-phase with the higher surcharge pressure

(d) During repeated loading, the pile shaft load increased to a peak value then decreased until it reached a limiting value. The value of the peak was largest in the case of the upper repeated load being in-phase with the higher surcharge pressure. The limiting value appeared to be independent of the state of the surcharge, i.e. whether it was in or out of phase, static at 100 kN/m^2 or cycled between 50 and 100 kN/m^2.

(e) From a practical point of view and within the tested limits, the frequency of the cyclic surcharge or the repeated loading did not alter significantly the movement nor the load transfer characteristics of the pile.

(f) The previous pile loading caused a reduction in the rate of movement of the pile during the succeeding loading.

(g) The peak shaft load of the virgin pile was generally greater than that of the previously loaded pile but, at the latter stages of the test, the shaft load of both piles attained approximately the same limiting value.

(3) The cyclic surcharge was found to increase the bearing capacity and the pulling resistance of the pile. Moreover, the pile load capacity, after a large number of surcharge cycles was not changed significantly when the pile load tested at 100 kN/m^2 or at 50 kN/m^2 surcharge pressure (the two limits of the cyclic surcharge).

CHAPTER 8

MAIN CONCLUSIONS AND SUGGESTIONS FOR FUTURE WORK

8.1 Main Conclusions

The purpose of this investigation was to study in detail the influence of repeated loading on the behaviour of isolated piles. The tests were performed on laboratory scale instrumented piles driven in a dry medium dense sand. The influence of the depth of embedment, the surcharge pressure and the loading history on pile behaviour were examined in this investigation. The following conclusions can be drawn from these tests:-

(1) The results of tests carried on a pile element embedded in a triaxial specimen, chapter 3, have shown that:-

(a) The skin friction increased linearly with increase of pile displacement up to failure.

(b) The value of the ultimate skin friction decreased and the load-displacement became more brittle when the loading was repeated.

(c) Alternating loading greatly affected the behaviour of the pile shaft. The ultimate skin friction decreased and the load-displacement became more ductile with this type of loading.

(2) The results of preliminary tests carried out with static surcharge pressure revealed that:-

(a) The driving resistance of the pile base increased rapidly with the depth of penetration, reached a peak value and then decreased slightly to a limiting value. The peak value increased and was reached earlier when the surcharge pressure was increased.

(b) The shaft friction in compression was generally greater than that in tension loading by a magnitude dependent on the depth of embedment, the surcharge pressure and the magnitude and direction of the residual stresses before testing.

(3) Repeated loading tests performed on compression piles with static surcharge pressure have shown that:-

(a) Initially, the pile was stable but after a certain number of cycles it began to move at a rate which increased rapidly to a maximum value and then decreased as the number of cycles increased.

(b) The pile life-span increased when the embedment depth decreased or the surcharge pressure increased.

(c) At any depth of embedment or surcharge pressure, as the number of load cycles was increased the shaft load increased up to a peak value and then decreased gradually to a limiting value.

(d) The rate of movement increased and the life-span reduced when the pile was subjected to failure loading before being tested under repeated loading.

(4) Repeated tension loading on piles with static surcharge pressure has indicated that:-

(a) Initially, the pile moved at a decreasing rate. After a stable stage the movement began to increase at a rate which increased very rapidly until failure occurred.

(b) The pile life-span increased when the depth of embedment and/or the surcharge pressure was increased.

(5) For both compression and tension piles and at any depth of embedment or surcharge pressure, the pile life-span was reduced when the load amplitude and/ or the load level were increased.

(b) Previous loading affected the behaviour of the pile during the succeeding loading and a smaller rate of movement generally resulted. Both the shaft load and its residual value decreased when the pile was subjected to previous loading, the higher the previous repeated load level, the greater the reduction.

(7) Compressive-to-tensile repeated loads greatly reduced the pile life-span.

(8) Repeated loading was found to decrease the ultimate bearing capacity and the pulling resistance of the pile for all depths of embedment and at all surcharge pressures examined. The largest reduction was observed in the case of tension piles.

(9) When the pile was subjected to static loading and a cyclic surcharge pressure acted upon the sand surface the following trends were observed:-

(a) For compression piles initially the movement increased as the number of surcharge cycles increased but at a decreasing rate until it reached an approximately constant value.

(b) For tension piles the pile movement first increased at a decreasing rate then, after a stable stage, the rate of movement increased rapidly until failure occurred. Cyclic surcharge loading caused an increase in the negative skin friction of the upper elements of the pile and a decrease for the lower elements.

(10) The results of tests carried out on piles subjected to repeated loading with cyclic surcharge have shown that:-

(a) The behaviour of the piles was similar to that of identical piles but tested with a static surcharge pressure.

(b) The cyclic surcharge loading results in a pile of longer life. The life was largest when the upper repeated load acted in-phase with the higher surcharge pressure.

(c) During repeated loading the shaft load increased to a peak value then decreased until it reached a limiting value. The peak value was largest when the repeated load acted in-phase with the higher surcharge pressure.

(d) From a practical point of view and within the tested limits, the frequency of the cyclic surcharge or the repeated loading did not alter significantly the movement nor the load-transfer characteristics of the pile.

(11) Cyclic surcharge loading was found to increase the bearing capacity and the pulling resistance of the pile under static loading.

(12) The test apparatus, the instrumentation and the testing techniques developed and adopted during this investigation were satisfactory and in general gave repeatability to within 3% with respect to both loads and pile movements.

8.2 Suggestions for future work

Among the most important suggestions for future investigations are the following :-

(1) The influence of sand density on the behaviour of piles subjected to repeated loading.

(2) The behaviour of a pile group under repeated loading.

(3) Combinations of horizontal and vertical repeated loading.

(4) The behaviour of piles in cohesive soil and the influence of pore water pressure set up during repeated loading are also suggested.

(5) The behaviour of belled piles subjected to repeated loading.

(6) The influence of the method of installation on the behaviour of piles subjected to repeated loading.

(7) The carrying out of some tests at full-scale or at least at a much larger scale, in order to establish the scale effect, if any, which may be present.

(8) Extending the study of the pile element in a triaxial specimen to include repeated loading. This will help in understanding the behaviour of the pile shaft under such a type of loading.

(9) The influence of the surface roughness and the slenderness ratio of a pile on its behaviour when subjected to repeated loading.

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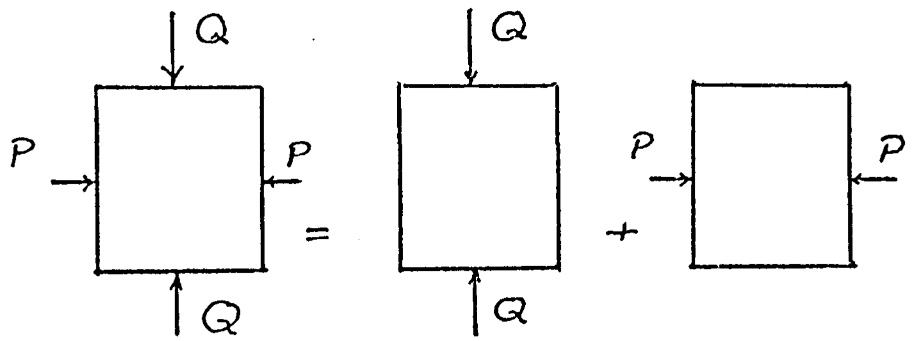
APPENDIX A

MEASUREMENT OF AXIAL LOADS IN VERTICAL PILES

A-1 Stress-Strain Relationship



$(e_a\,,\,e_t)$ $(\varepsilon_a\,,\,-\mu\,\varepsilon_a)$ $(-\mu\,\varepsilon_t\,,\,\varepsilon_t)$

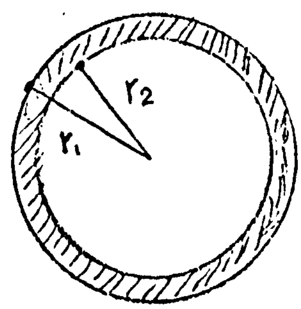


FIG. A-1

For a given load-cell embedded in sand and subjected to an axial force, Q, and a sand pressure, P, Fig. A-1, the axial and tangential strains are :-

$$e_a = \varepsilon_a - \mu\ \varepsilon_t \quad \ldots\ldots \quad \text{A-1}$$

$$e_t = \varepsilon_t - \mu\ \varepsilon_a \quad \ldots\ldots \quad \text{A-2}$$

respectively

in which:-

ε_a = axial strain due to subjecting the load-cell to axial force only.

ε_t = tangential strain due to subjecting the load-cell to lateral sand pressure only.

μ = Poisson's ratio

From equation A-2

$$\varepsilon_t = e_t + \mu \ \varepsilon_a \quad \ldots\ldots \quad \text{A-3}$$

Therefore, equation A-1 becomes

$$e_a = \varepsilon_a - \mu \ (e_t + \mu \ \varepsilon_a) \quad \ldots\ldots \quad \text{A-4}$$

or,

$$e_a = (1 - \mu^2) \ \varepsilon_a - \mu \ e_t \quad \ldots\ldots \quad \text{A-5}$$

But the load, Q = stress x area

$$Q = \varepsilon.E.\pi \ (r_1^2 - r_2^2) \quad \ldots\ldots \text{A-6}$$

$$\therefore \ Q = \frac{E.\pi \ (r_1^2 - r_2^2)}{1 - \mu^2} \ (e_a + \mu \ e_t) \quad \ldots\ldots \quad \text{A-7}$$

Where E is Young's modulus of the cell material.

When the strains are monitored at 2 points, equation A-7 becomes

$$Q = \frac{E.\pi \ (r_1^2 - r_2^2)}{1 - \mu^2} \left(\frac{e_{a1} + e_{a2}}{2} + \frac{\mu(e_{t1} + e_{t2})}{2} \right) \quad \ldots\ldots \quad \text{A-8}$$

or

$$Q = \frac{E.\pi \ (r_1^2 - r_2^2)}{2(1 - \mu^2)} \ (e_{a1} + e_{a2}) + \mu(e_{t1} + e_{t2}) \quad \ldots\ldots \quad \text{A-9}$$

Therefore the axial pile loading can be obtained by measuring the axial strain and the tangential strain separately. Such a measurement can be done by using rosette orthogonal strain gauges.

Equation A-9 is related to voltage output as follows:-

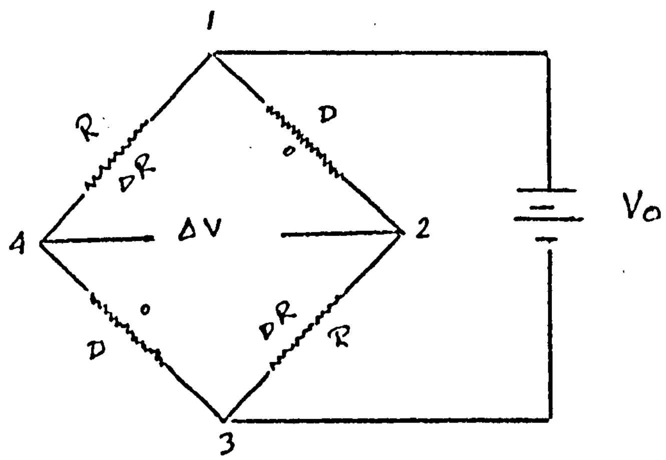


Fig. A-2

The bridge shown in Fig. A-2 consists of four identical strain gauges, two of them are active which are located on the pile surface and the other two are dummy and are located outside the pile and are supplied by an excitation voltage, Vo, under the influence of straining the resistance of the active strain gauge, R, will be changed by ΔR which corresponds to a drop in voltage ΔV between points 2 and 4. If the electrical current, i, which is equal to Vo/ (R + D), is assumed constant, then

$$\Delta R = \frac{\Delta V}{i} \quad ..... \quad \text{A-10}$$

but

$$\Delta R/R = K.e \quad ..... \quad \text{A-11}$$

where

K is the gauge factor of the strain gauge which is constant

e is the measured strain

$$\therefore \quad e = \frac{\Delta V}{R.K.i.} \qquad \ldots\ldots \qquad A-12$$

equation A-9 then becomes

$$Q = \frac{E.\pi\,(r_1^{\,2} - r_2^{\,2})}{2(1-\mu^2)\ R.K.i.} \quad (\Delta Va_1 + \Delta Va_2) + \mu\ (\Delta V_{t1} + \Delta V_{t2})$$

$$\ldots\ldots \qquad A-13$$

or

$$Q = C \times (\Delta Va + \mu\Delta V) \quad \ldots\ldots \qquad A-14$$

Where C is the calibration factor,

ΔVa is the drop in voltage of the axial strain bridge due to straining of the load-cell, which is also evaluated and recorded by the data logger.

In the case of the top load-cell, where the sand pressure is equal to zero, equation A-14 becomes,

$$Q = C.\ \Delta Va$$

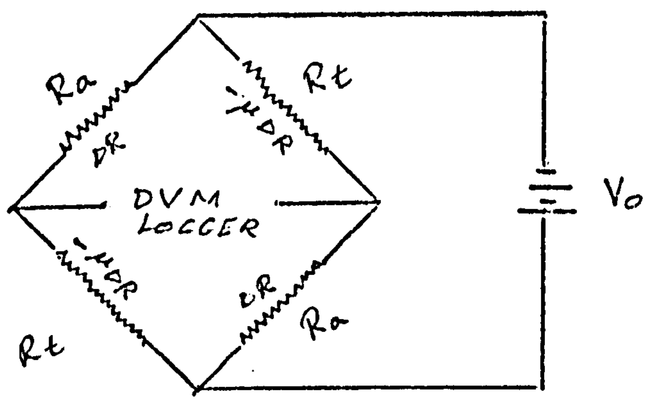


Fig. A-3

To increase the sensitivy of the top load-cell bridge and to eliminate the need for dummy strain gauges the strain gauges are connected as shown in Fig. A-2 in which the two bridges are combined in to one.