Hydraulic Applications of Geosynthetics to some Filtration and Drainage Problems with Special Reference to Prefabricated Band-shaped Drains

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ABSTRACT: Some of filtration and drainage problems related to the current usage of geosynthetics have been briefly reviewed. As an example, prefabricated band-shaped drains are chosen as the subject for review and discussion to illustrate the advantage of the use of geosynthetics for the purpose of filtration and drainage of soils, and also to point out the complexity of the interactions between soil and geosynthetic material in contact when liquid flows through the soil-geosynthetic system. A wide variety of products available and their wide-spread unique uses indicate a further development and a tremendous future potential if continued research efforts, proper uses and unbiased understanding among all those concerned ensue.

1 INTRODUCTION

Numerous studies have been conducted and published on the subjects related to hydraulic applications of geosynthetics to filtration and drainage since a keynote lecture on this subject was presented at the Fourth International Conference on Geotextiles, Geomembranes and Related Products in The Hague (Gourc and Faure 1990). In this lecture, three fundamental aspects of geosynthetics leading to their hydraulic characteristics were discussed in detail, i.e., 1) properties of fibers and fibrous structures, 2) interactions of fiber and water, or the hydraulic conductivity of geosynthetics and 3) the fiber-water-soil-particles interaction as well as some of the testing procedures.

While the understanding of these scientific phases of the complex problems has no doubt advanced considerably since then, the past four years have seen more marked progress in practice-oriented knowledge of the properties and behavior of geosynthetic materials, wide-spread use of index and performance tests both in the laboratory and in the field, and versatile practical applications of wide variety of geosynthetic products to various engineering problems, particularly in the category of filtration and drainage.

In his 1992 Mercer Lecture, Koerner described the advent of geotextiles for filtration purposes as follows: "--- it is quite presumptuous to ask that a relatively thin geotextile act as a filter in place of traditional soil layer which is at least 30 cm thick and is sometimes made from a sequence of different soil gradations. Yet, this is precisely what began in the 1960s.--- Today, many types and styles of geotextiles are used in filtration designs for a wide range of soil, liquid and hydraulic conditions. (Koerner, et al 1993)."

In terms of the geosynthetic terminology, Koerner (1994) recently gave the two terms more practical definitions than those given in its second edition published in 1990, differentiating them again by changing only one word from 'across' to 'within' as follows:

Filtration: The equilibrium geotextile-to-soil system that allows for adequate liquid flow with limited soil loss <u>across</u> the plane of the geotextile over a service lifetime compatible with the application under consideration.

Drainage: The equilibrium geotextile-to-soil system that allows for adequate liquid flow with limited soil loss within the plane of the geotextile over a service lifetime compatible with the application under consideration.

2 REVIEW OF FILTRATION AND DRAINAGE PROPERTIES OF GEOSYNTHETICS

2.1 Basic principles - hydraulic conductivity

The most fundamental hydraulic property of the geosynthetics for the both functions may be expressed by the coefficient of permeability, k, in accordance with Darcy's law:

$$k = q/(A\ i) = v/i \qquad (1)$$

in which q is the rate of flow through the geosynthetic, A the area of the geosynthetic plane through which the flow percolates, i the hydraulic gradient and v the velocity of the flow.

For filtration the hydraulic conductivity of the geosynthetic normal to its plane is defined as permittivity, Ψ :

$$\Psi = k_n/t \qquad (2)$$

where k_n is the coefficient of cross-plane permeability and t the thickness of the fabric under a specified normal stress.

For drainage the hydraulic conductivity in parallel with the plane of the geosynthetic is defined as transmissivity Θ :

$$\Theta = k_p\ t \qquad (3)$$

where k_p is the coefficient of in-plane permeability.

From the above definitions, the flow rates of the cross-plane and in-plane flow rates, q_n and q_p, respectively, are given by

$$q_n = k_n\ (h/t)\ A = \Psi\ h\ A \qquad (4)$$

$$q_p = k_p\ (h/l)\ w\ t = \Theta\ i\ w \qquad (5)$$

in which l is the length of the in-plane flow path and w the width of the flow. Equations (4) and (5) indicate that once Ψ and Θ are successfully determined, the flow rates q_n and q_p do not depend on the value of t which is highly dependent on the applied pressures and therefore, is difficult to measure accurately.

2.2 Review of filter criteria

Numerous empirical criteria have been proposed for satisfactory performance of the geosynthetic filter largely based on the experience obtained from graded granular filters. It has long been recognized that an effective filter must satisfy the four basic criteria; 1) retention criterion: the largest pore in the filter should be smaller than the larger particles of the soil, retaining the soil and preventing excessive migration of soil particles, i.e., piping, 2) permeability criterion: sufficient flow should be maintained through the filter without significant flow impedance during its design life, 3) anti-clogging criterion: the majority of openings in the filter should be large enough to let the smaller particles pass through the filter so that the filter will not "clog," and 4) survivability criterion: the filter must be strong and durable enough to survive the installation process and changes in conditions during its service life (e.g., Christopher and Fischer 1992, and Luettich, Giroud and Bachus 1992). Thus, basically the filter has to satisfy the two conflicting requirements 1) and 2), retention and permeability criteria, i.e., a filter should have pore spaces or openings small enough to prevent excessive migration of soil particles but large enough to allow adequate flow of liquid.

For a special soil such as a gap-graded silty sand, however, a design methodology recommended is to select a geotextile filter which has relatively uniformly shaped holes small enough to retain the sand but large enough to release the silt, for this type of sands rarely pose settlement problems as a consequence (Fluet and Luettich 1993).

The four filter criteria were examined in detail by Christopher and Fischer (1992). Their review and proposals are summarized in the following:

1) 22 different formulas are listed as the existing retention criteria for geotextile filters, all of which are functions of various opening sizes of the filter fabric such as O_{95}, O_{90}, O_{50} and O_{15} and the diameter of soil particles such as d_{90}, d_{85}, d_{50} and d_{15}, depending mostly on the uniformity coefficient of the soil, C_U (= d_{60}/d_{10}). For instance, O_{90} and d_{90} are, respectively, the 90% opening size of the

fabric that means the diameter of beads which are 90% refused, and the "90% finer" grain size of the soil particles. Incidentally O_{95} is referred to as the apparent opening size (AOS). Most of the criteria are given in the form of the O_i/d_j ratio not exceeding a certain value or a range, in which i and j in O_i/d_j are integers.

2) The permeability criteria were indicated in terms of the coefficient of permeability of the filter fabric and the soil, k_f and k_s, respectively, i.e., a) $k_f \geqq k_s$, b) $k_f \geqq 10k_s$ and c) $k_f \geqq 0.1k_s$. A couple of other criteria were given in terms of permittivity of the fabric. The criterion a) has long been advocated by many researchers on the assumption that the geotextile needs to be no more permeable than the soil it is retaining. Carroll (1983) and Christopher and Holtz (1985) recommend, however, the criterion b) for critical soil and hydraulic conditions in which clogging has been shown to cause roughly an order of magnitude decrease in the geotextile permeability. The criterion c) was proposed by Giroud (1982) on the premises that a geotextile with only 10% of the permeability of the soil would still have a much greater flow capacity than the soil because the length of the flow path is directly related to the flow rate through a porous media.

3) The anti-clogging criteria call for soil-fabric filtration tests for critical applications, while for less critical cases some empirical criteria apply, consisting typically of the ratio of the opening size of the fabric such as O_{95}, O_{50} and O_{15} to the % finer grain size such as d_{50} and d_{15}, i.e., the O_i/d_j ratio being greater than a recommended value. A few researchers propose the use of the percent open area (POA) or the porosity of the filter fabric to be greater than a certain value.

4) As the survivability criteria, it is recommended the following tests be run as specified by a well established testing standard and each of the results should fall within a recommended range of values depending upon two field conditions, one being more severe than the other. The laboratory tests required are for grab strength, elongation, seam strength, puncture strength, burst strength, trapezoid tear and ultraviolet degradation. In addition, chemical compatibility tests and tests for biological and biochemical clogging may become necessary.

The foregoing filter criteria are too numerous and even confusing to many practitioners and also the measurement of pore size characteristics which are vital in these criteria have not been adequately standardized. It is often felt that more rigorously defined standard procedures are required for these tests in order to ensure reproducibility of test results. Also, efforts have been made to find just how much variations should be expected in test results between laboratories.

In addition, geotextiles are generally not uniform enough to be defined by a single AOS or POA value. In fact it has been pointed out that the actual value for a given geotextile usually varies between a few units of AOS due to an inherent variability in properties which depends on a number of factors including the method of manufacture and the level of quality control (Christopher and Fischer 1992, and Rowe 1993).

It is also to be noted that external factors can influence the development of the filter bridge upon which many of the criteria rely. A failure for the bridge to develop due to movement of the geotextile would allow fine-grained soil to move through the filter (Christopher and Fischer 1992). Such a phenomenon is closely related to the long term equilibrium flow rate which may result in clogging.

In this regard Koerner, et al (1993) gives the following interpretation: some soil will always embed itself on the surface of, or within the filtering geotextile. This type of 'tuning' of the geotextile to the upstream soil, its stress state, its permeating liquid, and its unique hydraulic conditions is necessary and fully expected. One can even consider that the geotextile becomes a catalyst which forces the upstream soil mass to modify itself so as to provide its own filter layer. At this point, with the soil being its own self-filter, the geotextile is serving more as a separating layer between the newly established soil filter layer and the downstream drainage system.

Based on the foregoing, 'clogging' is defined as the reduction of the geotextile's permeability to the degree where the lack of flow through it results in the system's non-performance at any point in its service lifetime. In this context 'complete clogging' is a misnomer since the clogged geotextile always retains some nominal permeability. The lower limit of the most severe type of clogging is probably the permeability of the fine fraction of the upstream soil which is often too low for a filtration system (Koerner, et al 1993).

2.3 Review of filter design

The state-of-the-art design practice is best illustrated and directed by Luettich, Giroud and Bachus (1992) pointing out that a complete geotextile filter design involves more than merely considering retention and permeability criteria. They presented a comprehensive systematic approach to solving common filtration design problems on the basis of the filter criteria such as those discussed in the preceding section.

Their proposed design methodology consists of the following nine steps; 1) Define the application filter requirements: identify the drainage material and define retention versus permeability trade-off, 2) Define boundary conditions: evaluate confining stress and define flow conditions, 3) Determine soil retention requirements: determine a steady state flow or dynamic flow conditions, define soil particle size distribution and the Atterberg limits, define soil dispersion potential and soil density conditions, and determine the maximum allowable geotextile opening ratio, O_{95}, 4) Determine geotextile permeability requirements: define the soil hydraulic conductivity, define the hydraulic gradient for the application, and determine the minimum allowable geotextile permeability, 5) Determine anti-clogging requirements, 6) Determine survivability requirements, 7) Determine durability requirements, 8) Miscellaneous design considerations to be given to: the geotextile structure, intrusion of the geotextile into the drainage layer, extrusion of fine-grained soil through the geotextile when subjected to high confining pressures, abrasion of the geotextile due to dynamic action, intimate contact of the soil and geotextile, biological and biochemical clogging factors and safety factors, and finally 9) Select a geotextile filter: make sure it has the properties required in steps 3 through 8 and if necessary, verify through conformance testing.

It goes without saying that sound engineering judgment based on experience and the up-to-date knowledge is essential in drawing a conclusion in each step and making the final selection.

2.4 Review of testing procedures

It is not the intention of this section to list up and discuss all the test procedures established by authoritative organizations and accepted widely by practitioners worldwide, nor to examine all the innovative ones being developed by many researchers. The object lies in pointing out and reviewing the problems involved in and different views on testing geosynthetics for their filtration and drainage properties. Reference may be made to quite a few papers which have been published recently describing and discussing in detail testing procedures for the filtration and drainage characteristics (e.g., Boshuk and Zhou 1992, Spence 1993, Greenwood, et al 1993, Koerner 1994).

The standardized testing procedures presently available are in general all 'index' tests. The index tests are those carried out on specimens of geosynthetics without the presence of soil. The actual field performance of geotextiles and geocomposites varies from the 'index' characteristics. Therefore for large projects where the performance of the geotextile or geocomposite is very important and sufficient relevant data on performance is not available, then 'performance' tests, field or laboratory, should be specified before the geotextile or geocomposite is selected (Corbet 1993).

For demanding filtration and drainage applications, analytical designs based on index test properties may not be adequate. By definition index testing does not account for site-specific parameters, and consequently in more critical applications, the designer must turn to other types of testing. In ascending order of relevance, these may be classified as laboratory performance tests, systems tests, large scale model tests and finally prototype or full scale field tests (Ingold 1993).

While the tests for filtration and drainage characteristics of geosynthetics may be classified into several classifications as in the preceding paragraphs, a distinction between 'short term' and 'long term' tests is of practical importance. A long term test may be able to reveal and assess at least in part potential problems by simulated modeling in the laboratory, particularly a problem like piping and clogging.

In this connection the five different approaches to this problem reviewed and summarized by Koerner, et al (1993) appear significant; these rather time-consuming tests are a) long term flow tests (also see Wayne and Koerner 1993), b) gradient ratio tests, c) hydraulic conductivity ratio tests, d) fine fraction filtration tests and e) dynamic filtration tests. Also worth mentioning is a simple performance test on the flow behavior of a soil-geotextile composite under its typical operational conditions to evaluate the long term reduction in hydraulic conductivity due to soil particle retention (Ling, et al 1993).

The long term testing is one of the major future directions which, although time-consuming, will no doubt help identify potential problems, clogging or piping or any other factors threatening our geosynthetic filter and/or drainage systems which are expected to function as designed over a long period of time. Future efforts to develop techniques to effectively accelerate long term tests are also important, however.

With regard to reproducibility of test results and variability of geotextile products, C. Lawson points out that test method variation must be less than product variation, but the situation exists with CEN and ASTM tests, for instance, that pore size, transmissivity and permeability tests have a variation in test method similar to the product variation. He also indicates that the pore size tests are not immune from equipment variations and operator variations; BS6906 Part 2 states that the test results can have a variability of ± 20% (Dixon 1993).

For an argument that O_{90}/d_{10} values were quoted as varying by factors of 5 or 10, B. Myles reported that in the ISO round robin tests which involved 18 laboratories, 4 geotextiles and 3 methods, and took over a year to complete, the results were remarkably similar, and he felt that such factors as stress levels were important when looking at flow and pore size characteristics. He also stated that the variability in pore size was usually only 10-15% and hence different index tests could be used (Dixon 1993).

2.5 Geosynthetics for filtration and drainage

A wide variety of geosynthetic products have been made available and used successfully to replace most of graded granular filters and traditional drainage systems due to their comparable performance, improved economy, consistent properties and ease of placement. Woven and nonwoven geotextiles, geonets, geocomposites, geopipes, etc., and their combinations are utilized extensively for filtration and/or drainage purposes.

Since it is impossible to describe each product and its uses, this paper deals in some depth with only one type of geosynthetic products which has both functions of filtration and drainage, focusing specifically on prefabricated band-shaped drains, the main purpose of which is to facilitate drainage from a thick layer of soft clay accelerating consolidation settlement and increasing rapidly shear strength of the clay.

Prefabricated band-shaped drains have to satisfy all the requirements for filtration and drainage as a geocomposite and in addition may be subjected to large deformations and high pressures from the surrounding soil. The hydraulic conductivity and compressibility of the soil in the immediate vicinity are greatly influenced by the installation method and subsequent performance of prefabricated band-shaped drains. It is hoped to serve as a good example to understand the filtration and drainage characteristics of geosynthetics in intimate contact with soil.

3 PREFABRICATED BAND-SHAPED DRAINS (PD)

3.1 General

Prefabricated band-shaped drains (abbreviated as PD hereinafter) have been extensively employed for the past few decades gradually taking the place of sand drains. Quite a variety of PD's are being produced by various manufacturers throughout the world. These products constitute a significant part of the geosynthetics for the purpose of filtration and drainage of soft clay strata to accelerate consolidation settlement and also to gain rapid increase in strength.

Among various prefabricated drainage geocomposites which have recently come into use, PD's have been widely used since 1960's as a direct successor of cardboard drains (or wicks) invented in 1940's. Just as various geosynthetics have greatly replaced traditional materials and methods, prefabricated drains are becoming increasingly popular, rapidly replacing sand drains which have been in extensive use since 1930's.

There are two main types of PD's. One common type consists of a thin geotextile filter sleeve which prevents fine soil particles from entering inside, but allows easy entry of porewater into the central core whose function is to act as draining channels while withstanding buckling and compressive stresses. The other type is a simple unitized strip of porous plastic having small continuous drain holes inside with sufficient strength and durability.

While the principal purpose of PD is referred to simply as facilitating drainage of soft clay strata, the function of PD is, in the geosynthetic terminology, the combination of filtration and drainage; "filtration" permits porewater in the clay to infiltrate across a band-shaped geotextile filter into continuous small holes, grooves or channels in a plastic core encased by filter fabric and "drainage" allows water to flow vertically through the longitudinal channels in the core to drainage layers of granular soil overlying and/or underlying the clay stratum.

When PD's are used as vertical drains in the same way as sand drains to stabilize soft clay, they will have to satisfy the basic requirements for filtration and drainage as geocomposites, including such problems as: a) Decreases in discharge capacity as confining pressures increase, as a PD deforms badly or bends sharply in the clay which consolidates as much as a few tens of percent, as the length of PD increases when it is extended down to great depths, and due to other causes, b) Formation of a smear zone around a driven PD and also formation of a transition filter zone around PD which causes migration of fine particles leading to clogging of filter and flow paths in the core, and c) Evaluation of the equivalent diameter of a band-shaped PD when Barron's formula is applied to the design of a PD installation.

However, the soils in contact with PD's are limited to soft clays in most cases and the durability of PD's to be expected is generally only a few years, being much shorter in terms of the service life than the other types of geosynthetics designed for permanent filtration and drainage.

3.2 Discharge capacity of prefabricated band-shaped drains (PD) in soft clay

The important hydraulic properties of PD's are the permeability of filter material (permittivity) and the discharge capacity q_w (transmissivity). The vertical drains should provide low resistance to filtration and be capable of discharging the flow vertically with low well resistance.

The discharge capacity q_w is defined as follows:

$$q_w = Q/i \qquad (6)$$

in which Q is the discharge velocity and i is the hydraulic gradient of the flow.

According to Mesri, et al (1994), well resistance refers to the finite permeability of the vertical drain with respect to that of the soil. Well resistance depends on the amount of water the consolidating soil discharges into the vertical drain, and on the discharge capacity and maximum drainage length of the drain.

Mesri and Lo (1991) defined a discharge factor, D, in terms of the horizontal permeability of the soil, k_h, and discharge capacity q_w and maximum drainage length of the drain l_m.

$$D = \frac{q_w}{k_h \cdot l_m^2} \qquad (7)$$

Based on their analysis on three major embankment projects, a threshold dis-

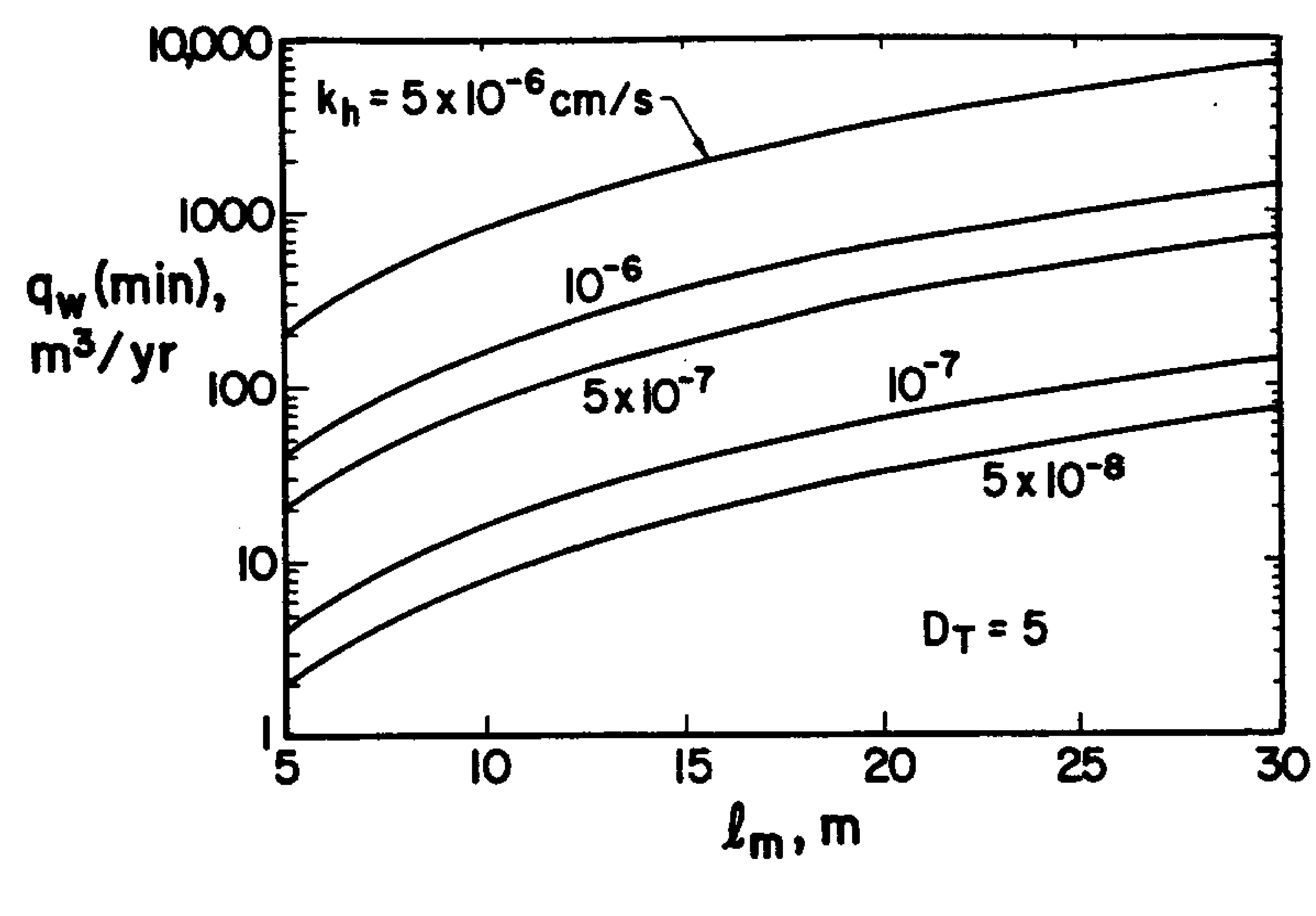

Fig. 1 Minimum discharge capacity required for negligible well resistance as a function of maximum drainage length of the vertical drain and horizontal permeability of the soft clay (after Mesri and Lo 1994)

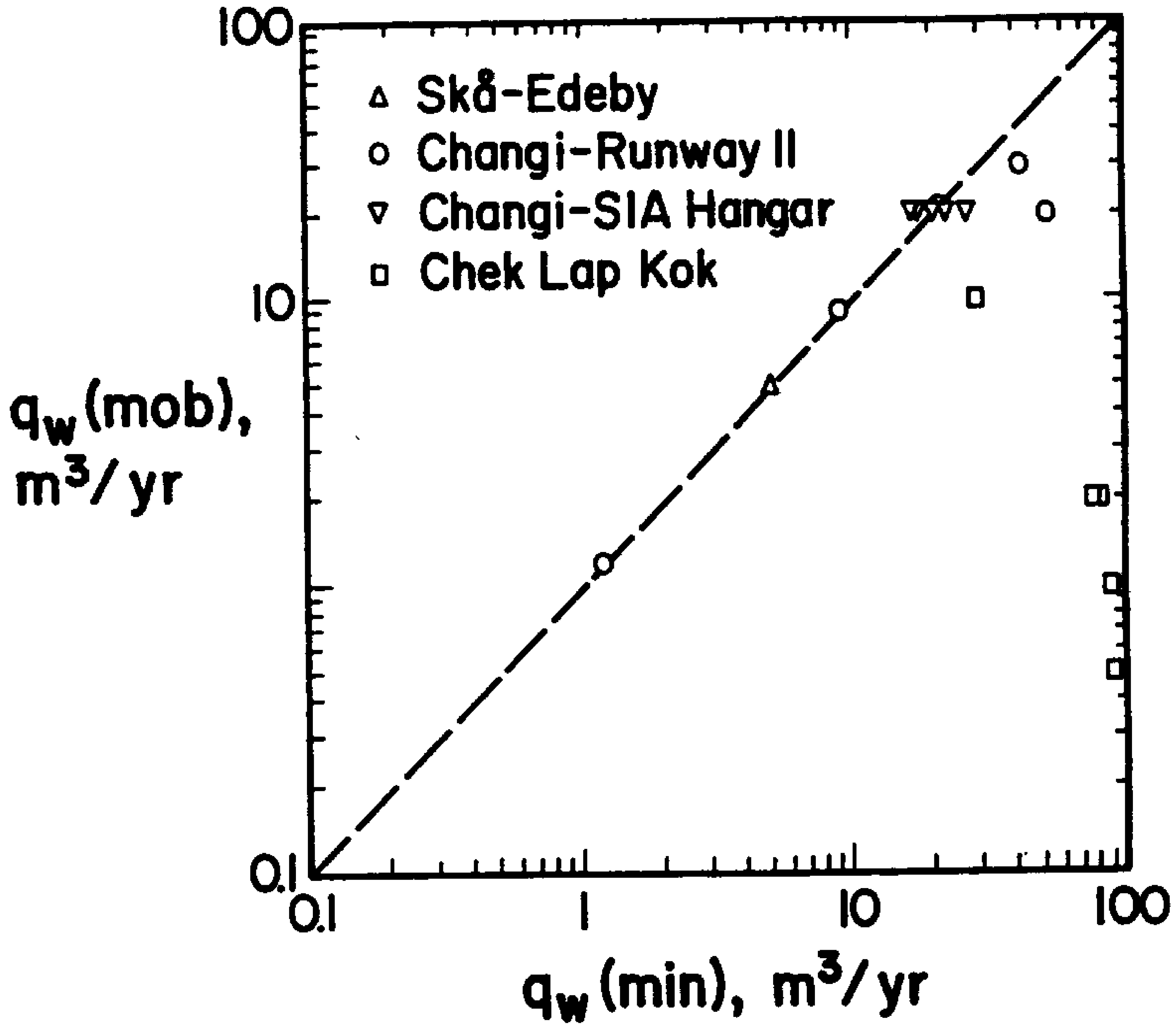

Fig. 2 Discharge capacity mobilized in field situations compared to discharge capacity required for negligible well resistance (after Mesri and Lo 1994)

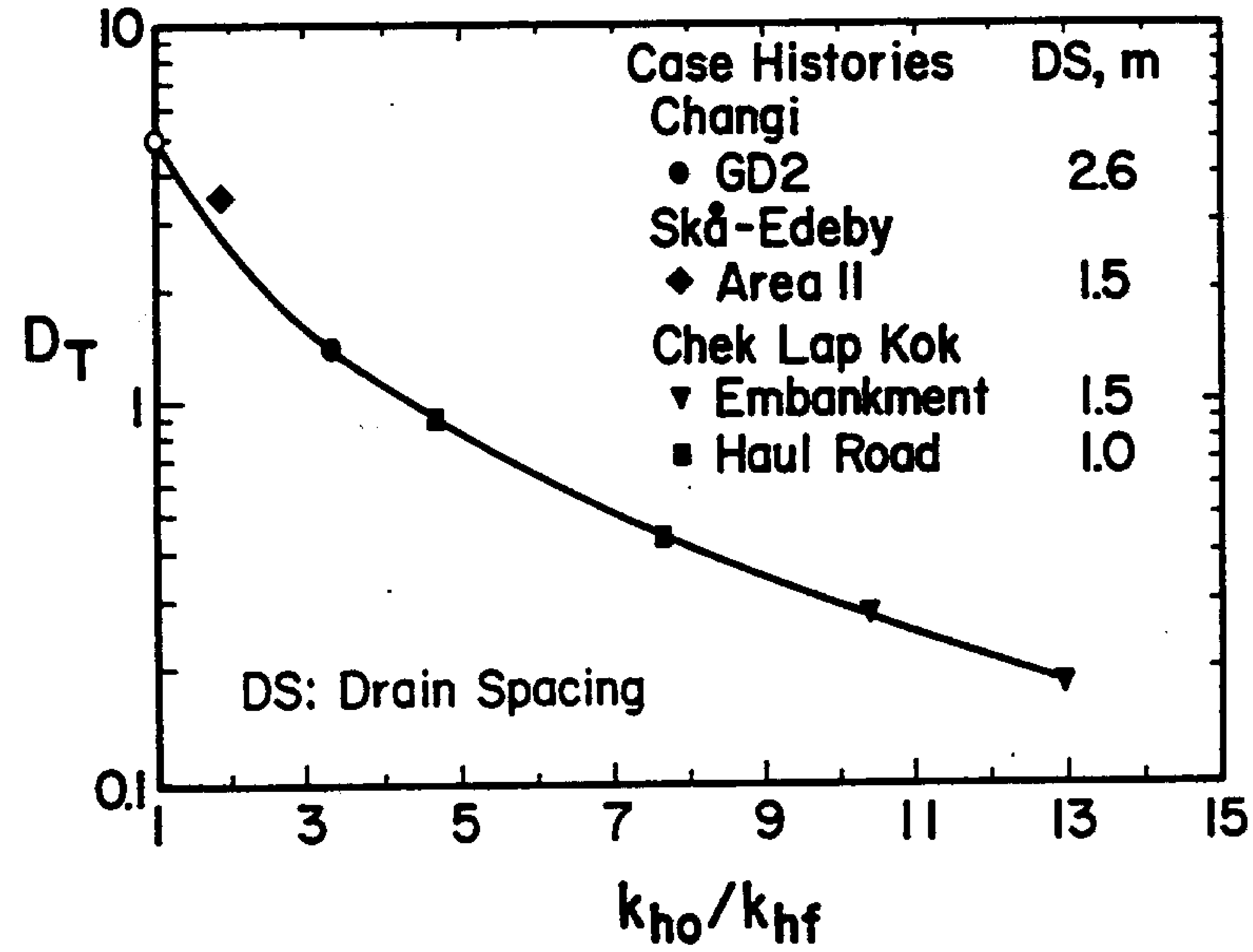

Fig. 3 Decrease in threshold discharge factor, D_T, as a result of decrease in horizontal permeability during consolidation (after Mesri and Lo 1994)

charge factor, D_T, was determined to be equal to 5 for negligible well resistance, i.e., the minimum discharge capacity of drain, q_w(min), required for negligible well resistance is equal to 5 $k_h \cdot l_m^2$. This is illustrated in Fig. 1, indicating that for long drains installed even in relatively pervious silts, large discharge capacities are required for negligible well resistance and for installations in most soft clays, the required discharge capacity ranges from 2 to 80 m^3/yr.

Values of mobilized discharge capacity, q_w(mob), were computed by comparing their analytical predictions of surface and subsurface settlements and porewater pressures with those measured in the field for the embankment loadings at four major construction sites (Mesri and Lo, 1991). A comparison of q_w(mob) with q_w(min), Fig. 2, shows that the PD's employed at three sites performed with q_w(mob) values which were at least as large as q_w(min). At Chek Lap Kok, q_w(mob) was less than q_w(min) computed using the coefficient of horizontal permeability at the in-situ void ratio, k_{ho}.

As shown in Fig. 3, the value of D_T is a function of decrease in horizontal permeability during the primary consolidation stage, expressed in terms of k_{ho}/k_{hf}, where k_{hf} is horizontal permeability at the end-of-primary consolidation. In other words, D_T and the corresponding q_w(min) decrease during the process of consolidation. As k_h decreases, less water enters the drain at a given time and therefore, a smaller q_w(min) is required to discharge water with negligible hydraulic resistance. Thus, in general, Fig. 1 specifies a conservative upper limit for q_w(min) which may be reduced with the help of Fig. 3 and Equation 7. It is concluded, therefore, that the PD's used at Chek Lap Kok functioned quite adequately (Lo and Mesri 1994).

It has been pointed out by quite a few researchers (e.g., Mesri, et al, 1994) that a number of important factors related to installation procedure, filter action, creep, chemical deterioration of filter fabric, clogging of filter and/or core channels, and bending of PD's, may operate in field situations, and may result in values of q_w(mob) significantly less than those measured by laboratory longitudinal flow tests.

3.3 Behavior of prefabricated band-shaped drains (PD) in soft clay samples

It has been pointed out repeatedly that the PD may be deformed in soft consolidating clay so badly its discharge capacity may be drastically reduced. Various possible configurations for a drain to accommodate soil settlement were considered and the effect of kinking on actual flow rates through a deformed PD was studied by Lawrence and Koerner (1988). Based on these earlier studies, the deformation of PD's was

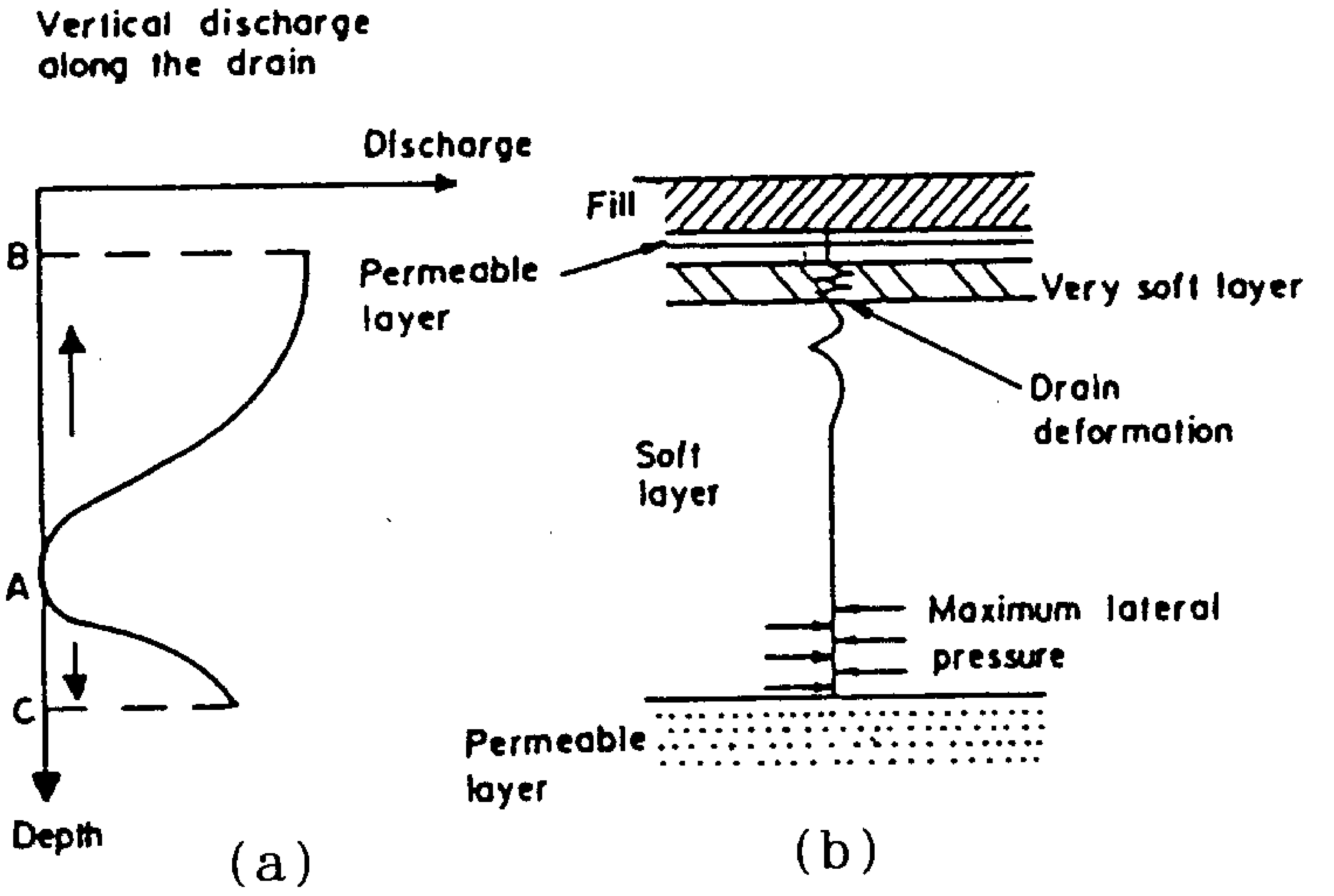

Fig. 4 Discharge mode and deformation of a PD installed in a soft clay layer (after Ali 1991)

divided into two types; folds(gentle bends) and buckle(sharp kinks) whose occurrence is attributed to the stiffness of the drain core and the type of soil surrounding the PD (Ali 1991). As shown in Fig. 4(a) and (b), he visualizes vertical discharge through a PD to the top and bottom permeable layers as in (a) and severe deformations it may experience as the soft clay consolidates more at shallow depths as in (b).

Ali(1991) presented the results of a laboratory investigation on the behavior of 5 types of PD's denoted K, L, M, N and P, briefly described in Table 1. The test apparatus consisted of a large consolidation cell, 50 cm in diameter and 120 cm in height, schematically shown in Fig. 5. A PD specimen was placed in the center of a soft Kaolin clay sample, 50 cm thick, which was sandwiched by a top and a bottom drainage layers of sand.

Water was supplied from a tank that could be held at a desired elevation and was led to the base plate so that the flow through a PD specimen was upward. Vertical pressures were applied in increments to consolidate the clay sample up to 300 kPa in approximately two weeks. Settlements and discharge capacities were measured at various stages of consolidation during the two-week test period. At the end of each test the consolidated clay sample was cut carefully to examine the condition of each PD specimen.

Fig. 6 gives the test results showing

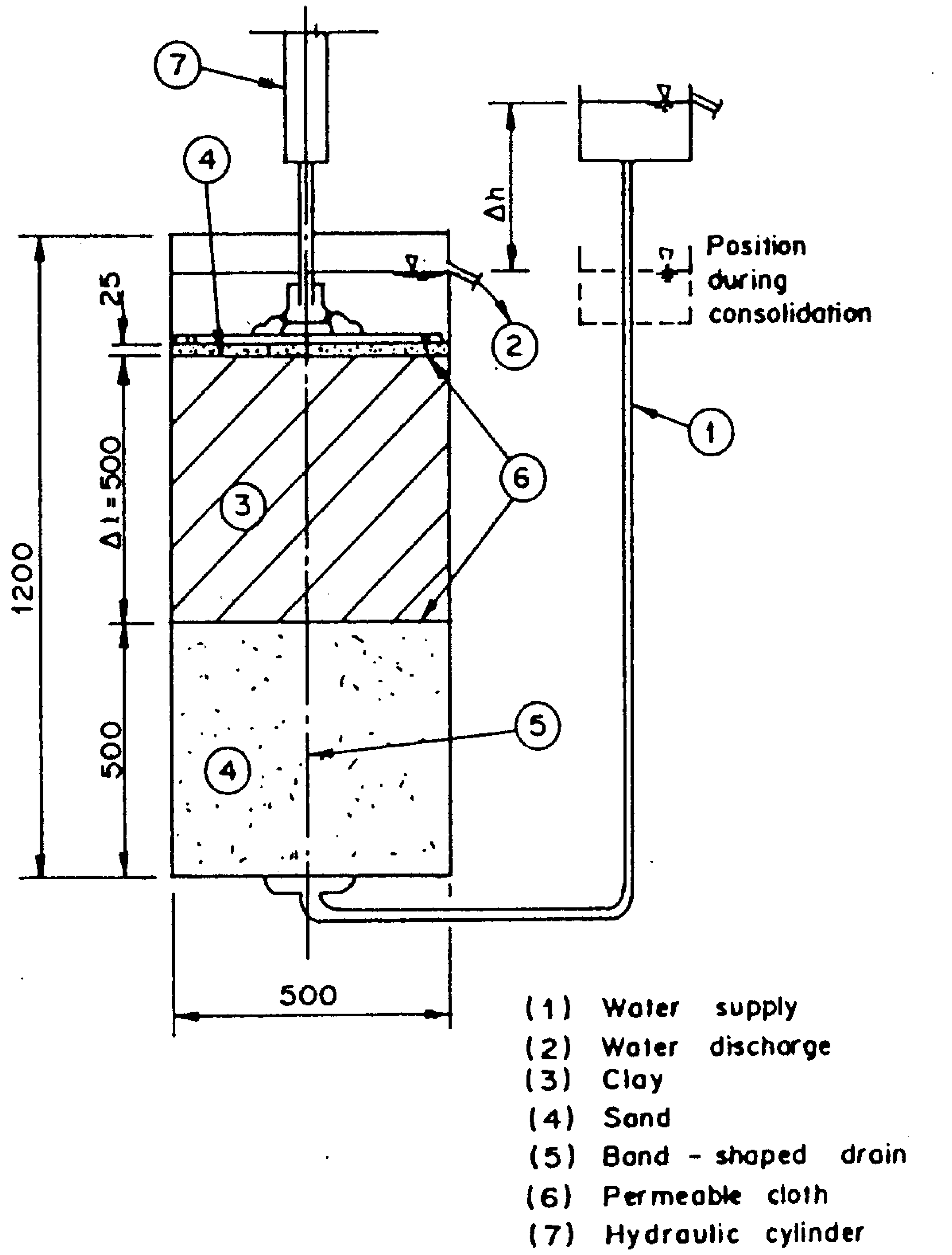

Fig. 5 Schematic diagram of test apparatus (after Ali 1991)

Table 1 Description of vertical drain cores (after Ali 1991)

Drain	Properties			Description
	Type	Material	Dimension	
K	Monolith	Polyolefine	95 × 2 mm	A strip consisting of 24 parallel rows of channels with tiny perforations on the wall.
L	Composite	Polyester (core) Polyester (fil. jacket)	100 × 5 mm	Core is 3-dimensional, open mat-structure. Non-woven filter jacket.
M	Composite	Polypropylene (core) Polypropylene (fil. jacket)	100 × 3 mm	Core consists of 38 parallel rows of rectangular castle-shaped groove. Filter jacket is non-woven continuous filaments
N	Composite	Polyethylene (core) Polyethylene/ Polypropylene (fil. jacket)	100 × 4 mm	Core consists of sharp studs. N1 – core is wrapped with Terram 1000 N2 – core is wrapped with Typar 3407
P	Composite	Polyethylene (core) Polypropylene (fil. jacket)	110 × 5 mm	Core consists of very flexible mat-structure The filter jacket is very flexible non-woven geotextile

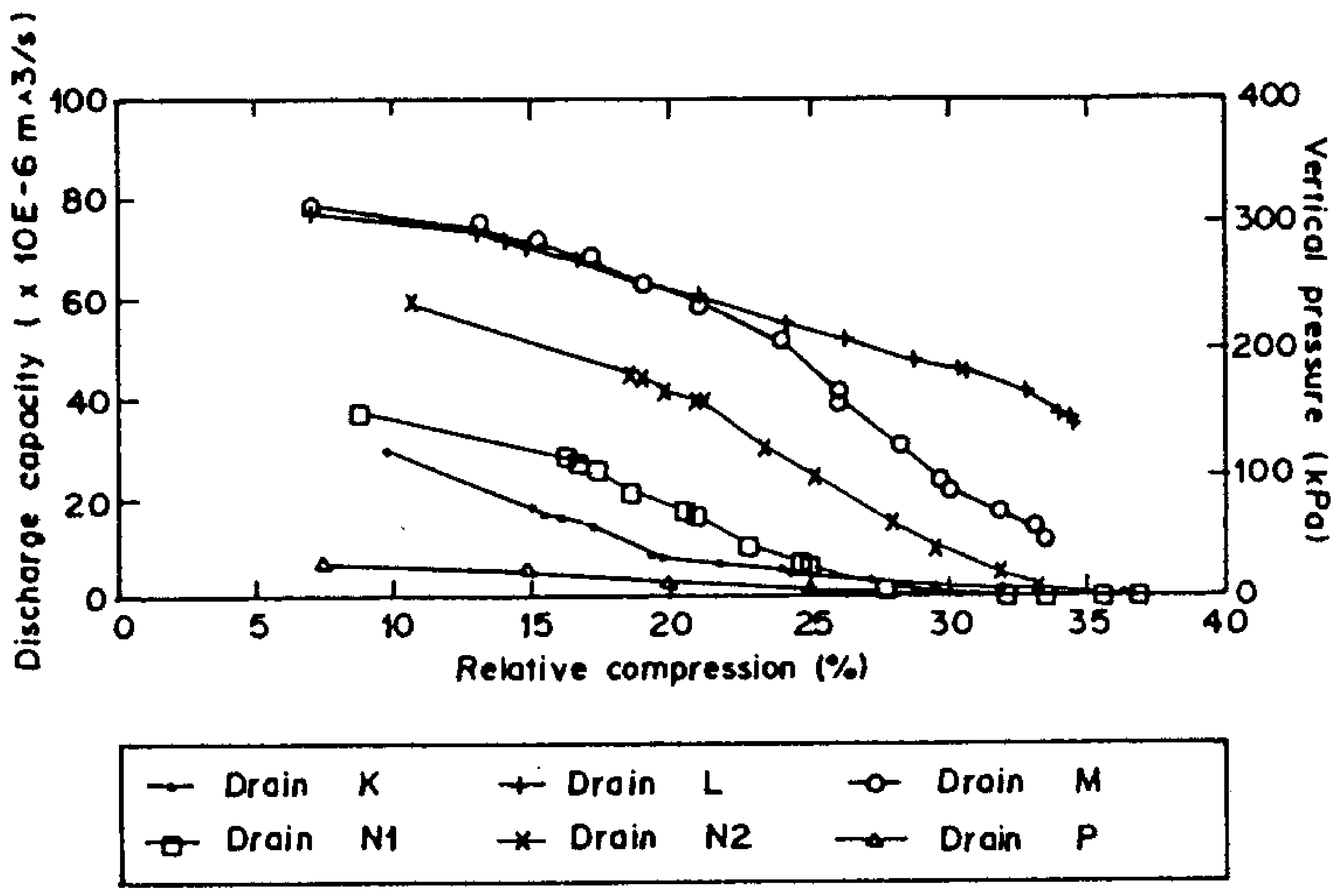

Fig. 6 Discharge capacity versus relative compression (after Ali 1991)

the variation of the discharge capacity q_W against the relative compression that is the ratio of vertical compression to the initial thickness of the clay sample. At the end of the test when the vertical pressure increased to 300 kPa and the relative compression reached 35%, Drain L gave the highest discharge capacity followed by Drain M. Drains N2, N1, K and P all showed nearly no discharge at the end of the tests.

After the test it was found that Drain M had a few sharp kinks while Drain L showed several more gentle folds. Folds were in greater number and closer spacings in Drains N1, N2 and P, while Drain K produced sharper kinks. No significant clogging was noted in the cores of all the drains except Drain K.

Table 2 Comparison of discharge capacities of vertical drains tested in straight and deformed conditions (after Ali 1991)

Drain type	*Rigidity (core)*	*Discharge capacity* ($\times 10^{-6}$ m^3/s)			
		24 h after load application		*48 h after load application*	
		straight[a]	*deformed*[b]	*straight*[a]	*deformed*[b]
K (monolith)	Rigid	66·7	2·6	66·7	2·5
L (comp.)	Semi-rigid	98·0	51·9	96·2	51·7
M (comp.)	Semi-rigid	161·3	41·4	155·0	38·8
N1 (comp.)	Flexible	78·8	1·1	75·5	1·0
N2 (comp.)	Flexible (the filter jacket is less flexible than N1)	74·9	10·5	74·9	10·5
P (comp.)	Very flexible core and filter jacket	not measured	0·25	not measured	0·25

[a]Lateral pressure = 10 psi (69 kPa), i = 0·5.
[b]Vertical pressure = 120 kPa (lateral pressure = 66 kPa assuming K_o = 0·55), i = 0·5, relative compression = 30%.

Ali(1991) concluded: a) The decrease in q_W due to deformation of PD is related to the bending rigidity and geometrical structure of the drain core, and the stiffness of the filter jacket. b) Slightly flexible drain cores with 3-dimensional, inter-connected, open flow channels as in the case of Drain L should resist reduction in q_W better. c) A core with sharp studs, corners or any points of high stress concentration which can give a punching effect to the filter jacket is not suitable, and d) Stiffer filter jackets perform better because they cannot be easily squeezed into the flow channels.

Table 3 Physical properties of PD's tested (after Miura, et al 1993)

		GL	MW, MB	CS, CS_2	TS	TF
Size (mm)	Thickness	3.4 ± 0.5	3.0 ± 0.5	2.6 ± 0.5	4.6 ± 0.3	7.5 ± 1.0
	Width	95.8 ± 2.0	100 ± 20	94 ± 2	100 ± 3	100 ± 5
Unit weight (g/m)		100	75	90	100	80
Structural type		Free	Free	Fixed	Free	One body
Material	Filter	Synthetic fiber of cellulose and polyester	Non woven fabric made from polypropylene	Spun bonded of polyester	Non woven fabric made from polypropylene	Spun bonded non woven fablic made from polyethylene
	Core	Polyolefin	Polypropylene	Polyethylene	Polyethylene	
Section diagram						

MB and CS_2 are improved ones of MW and CS respectively.

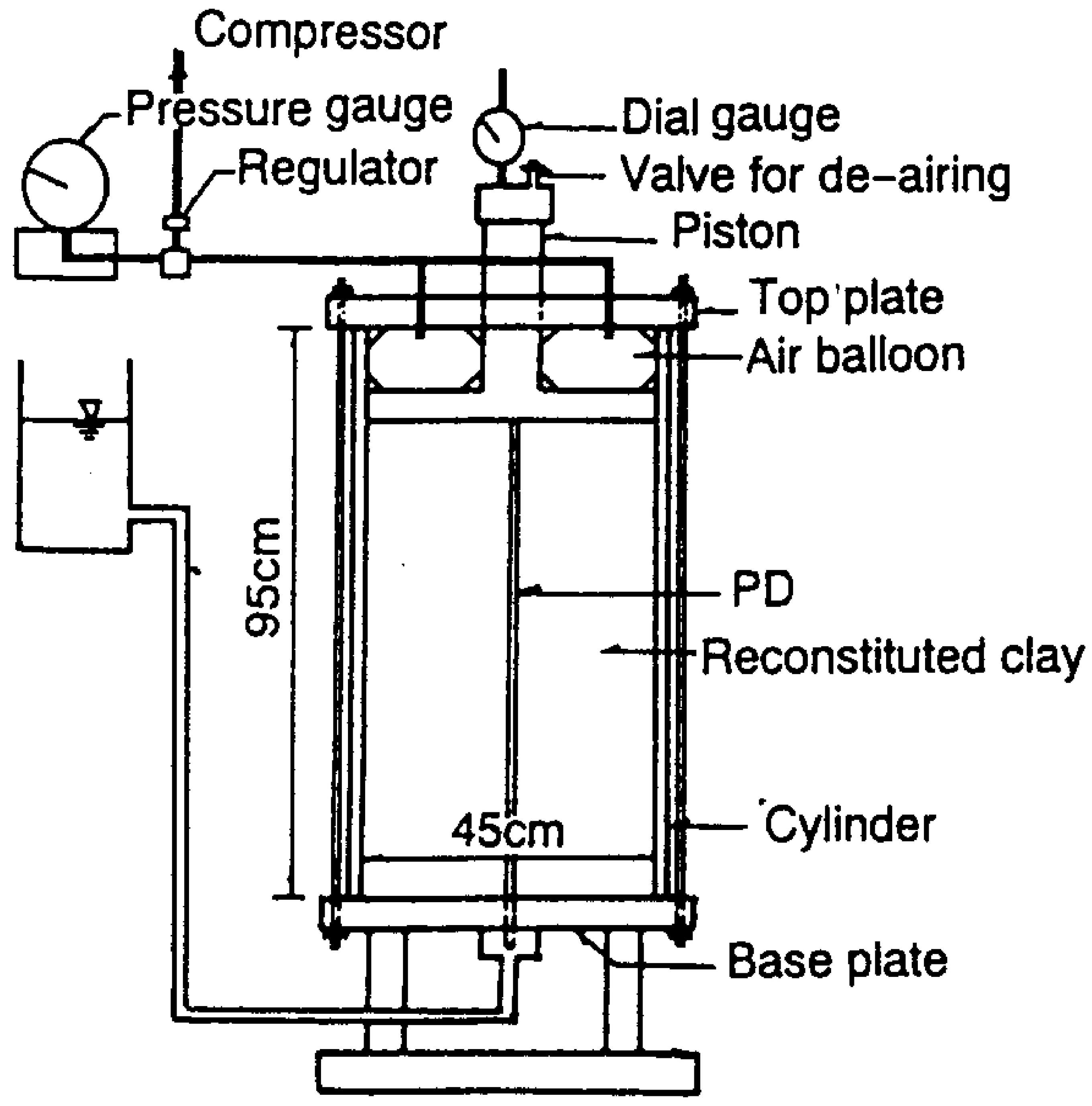

Fig. 7 Schematic diagram of large-scale consolidation apparatus (after Miura, et al 1993)

In Table 2, Ali(1991) also gives the results of tests in which these PD specimens were subjected directly to a lateral pressure of 69 kPa when they were straight, and compares them with the test results of the same PD specimens which were deformed in a consolidating clay being subjected to an estimated lateral pressure of 66 kPa. The latter showed much smaller values of q_W. He concludes, therefore, that the q_W value of a PD should not be determined by the tests on straight PD specimens especially when a large settlement is expected because the performance of buckled PD's is not related to that of straight ones.

Miura, et al (1993) also conducted a series of laboratory tests on five types of PD designated as GL, CS, MB, TS and TF as briefly described in Table 3. The PD specimens were all 5 cm in width (Full size PD's were split into two halves) and a model sand drain designated as SD was also tested for comparison consisting of sand packed in the same nonwoven PP filter as that of TS, also 5 cm in width.

These specimens were inserted in the center of a remolded, very soft, highly plastic clay sample, 45 cm in diameter and 85 cm in height in a large oedometer, which was consolidated under a pressure of 49 kPa, Fig. 7. Fig. 8 shows sketches of the side views of the deformed PD and SD specimens when the vertical compressions exceeded 20% in the consolidating clay. Fig. 9 gives a

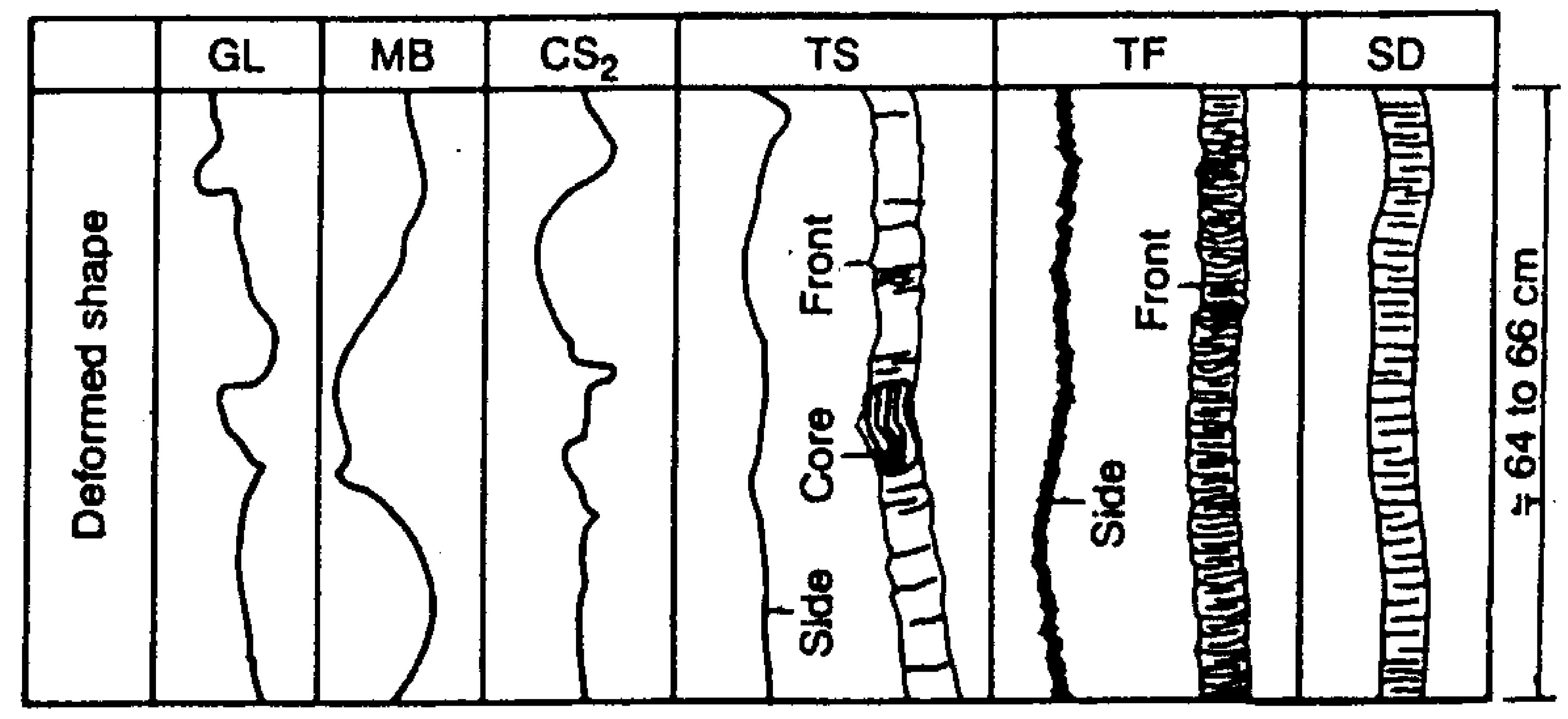

Fig. 8 Deformed shapes of PD's and an SD after consolidation with vertical compressions greater than 20% (after Miura, et al 1993)

photograph of one such example.

The time-settlement curves obtained for the composite clay samples were approximately the same for all the PD and SD installations and the measured discharges out of these model vertical drains were also about the same with the passage of time. The above results indicated that the equivalent diameter of a PD ranged between $2(a+b)/\pi$ and $(a+b)/2$ in which a and b are the width and thickness of a PD, respectively.

Based on the results of a large scale field tests (see 3.7.2 and 3.7.3) Crawford, et al (1992) assumed an equivalent diameter of 63 mm for their PD installation and Oikawa, et al (1989) reported that the equivalent diameter was more like 60 mm which was very close to the value of $2(a+b)/\pi$.

As has been proposed by Kjellman (1948) and verified by Hansbo (1979), a band-shaped drain would cause the same radial consolidation in the clay around it as cylindrical vertical drain having a circular cross-sectional area equal to that of a band-shaped drain.

It appears to be still a common practice, however, to assume the equivalent diameter of a typical PD having a cross-section, 100 mm by a few mm, to be 50 mm. It is considered that the common assumption of 50 mm for a typical 100 mm wide PD would generally be acceptable with a reasonable degree of conservatism.

On the basis of their extensive experimental study, Miura, et al (1993) reported the following results;

a) The distributions of water contents of the clay around the PD specimens

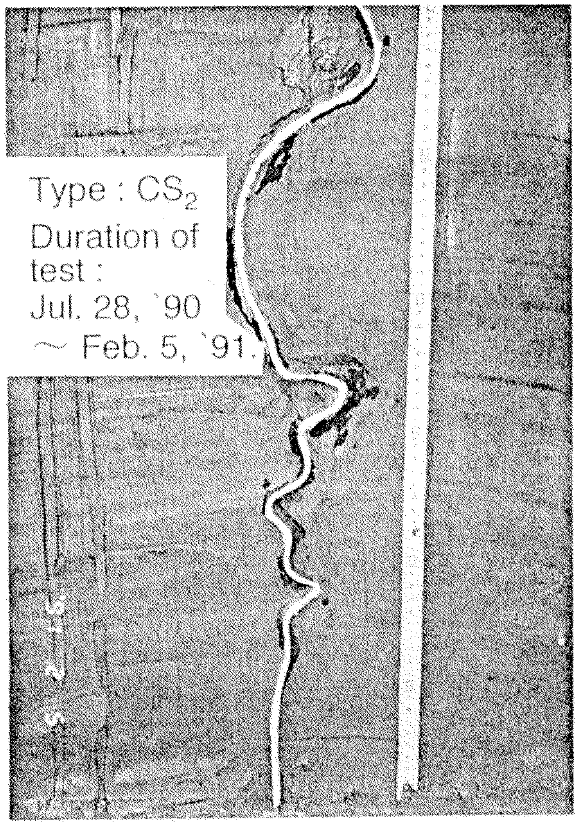

Fig. 9 Deformed shape of a PD (CS_2) when consolidation of the clay reached 22% (after Miura, et al 1993)

revealed that generally a radial flow pattern prevailed around a PD leading water into it almost as if it were a cylinder unless it is sealed at the both edges.

b) A significant reduction was noted in fine particles smaller than 0.01 mm in the clay in the immediate vicinity of the filter surface of a PD according to the grain size distributions determined of clay taken systematically on a 1 cm spacing being away from the PD surface.

c) This reduction of fines contributed to clogging of the PD filter and infiltration of fines. For example, across the PD filter with a total area of 855 cm^2 the fine particles which stayed in the filter (clogging) amounted to 22.5g, while the fines found inside the core channels was 7.8 g, totaling approximately 30 g. This total amount was in reasonable agreement with the loss of fines in the clay within a 4 cm zone in contact with the filter, determined to be approximately 24 g, which must have migrated into the PD with the flow of porewater.

The experimental results such as the foregoing led to a concept that a "filter zone" was formed several cm (3 to 8 cm) away from the PD surface and in between a "bridging zone," a few cm (3 to 4 cm) in thickness was developed. Further elucidation appears due since the possible formation of such zones in the close proximity of driven PD's may have a significant bearing on the permeability of the clay around them.

d) Constant-head permeability tests run on the PD filters before and after the tests revealed no significant changes in permeability in spite of clogging noted in the 0.25 mm thick filter.

3.4 Filter criteria for prefabricated band-shaped drains (PD)

The general filter criteria for geosynthetics have been discussed in 2.2 and in general apply to the filter jacket of a PD. The design criteria for PD filter should, however, take into account the fact that the soil in direct contact with the filter is almost always soft clay.

Very few studies have been made specifically on the requirements for PD filters. Chen and Chen (1986) presented empirical criteria for PD filters as follows:

$$O_{90}/d_{85} < 1.3 \text{ to } 1.8 \qquad (8)$$

$$O_{50}/d_{50} < 10 \text{ to } 12 \qquad (9)$$

Bergado, et al (1993b) compared the grain size data of the soft Bangkok clay with pore size data of several different types of PD's and proposed 'preliminary' criteria specifically for PD filters installed in the Bangkok clay as follows:

$$O_{90}/d_{85} < 2 \text{ to } 3 \qquad (10)$$

$$O_{50}/d_{50} < 18 \text{ to } 24 \qquad (11)$$

Based on the European experience, Rathmayer(1994) recommends:

$$O_{90} < 0.15 \text{ mm} \qquad (12)$$

$$O_{90} < d_{85} \qquad (13)$$

$$O_{90} < (1.1 - 2.8)\, d_{50} \qquad (14)$$

Apparently the criteria seem to depend upon the type of a PD and the clay in contact with it as well as various other factors yet to be identified.

3.5 Discharge capacity of prefabricated band-shaped drains (PD) determined in triaxial cells

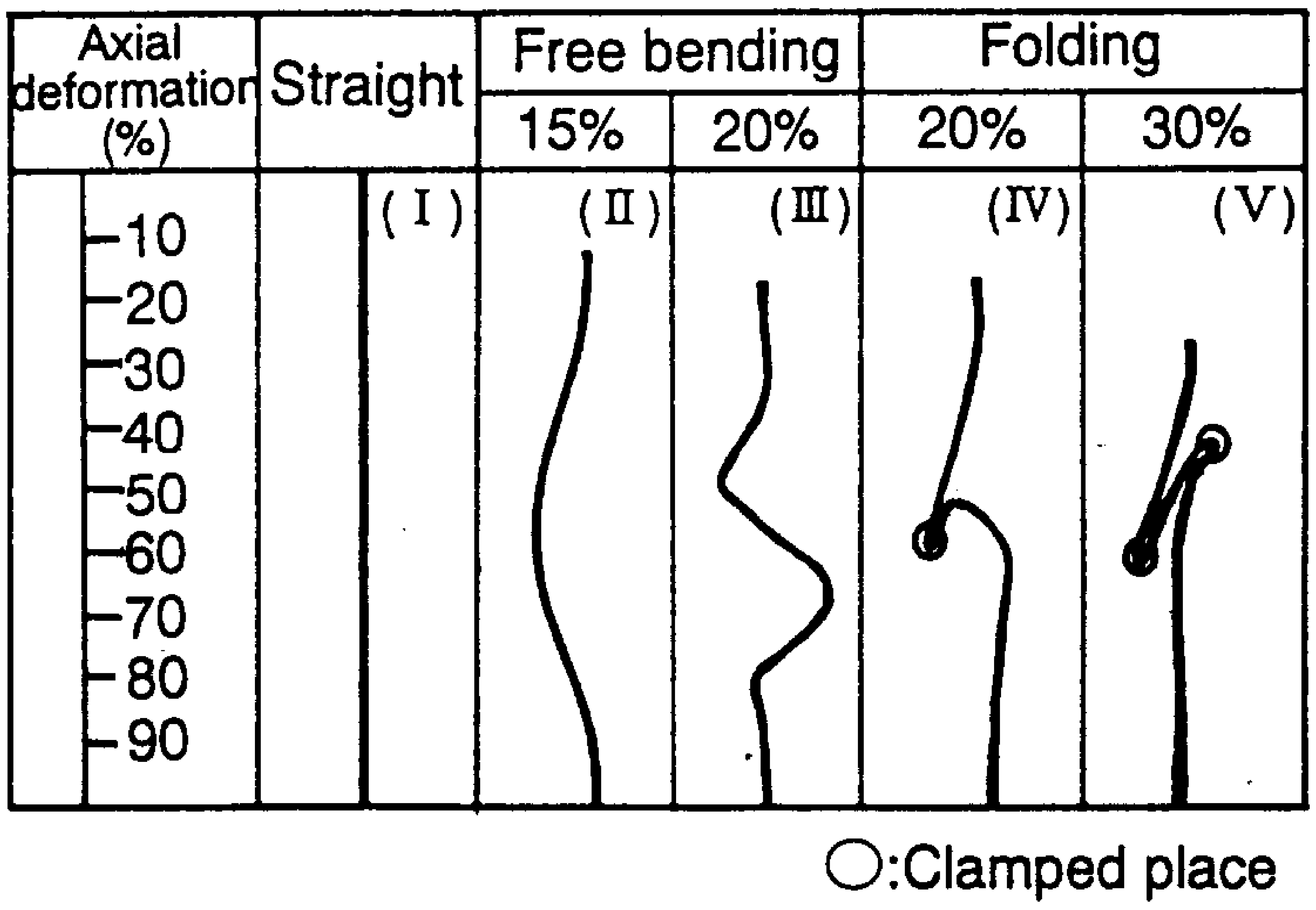

Fig. 10 Deformed shapes of PD's for the longitudinal permeability tests (after Miura, et al 1993)

Miura, et al (1993) tested five brands of PD's shown in Table 3 for their discharge capacities when they were subjected to confining pressures ranging from 49 and 392 kPa, under hydraulic gradients varying from 0.1 to 0.9 and also when they were deformed in 5 different shapes, Cases I through V as illustrated in Fig. 10.

Each PD specimen was 10 cm in width and 40 cm in length, sealed by a 0.8 mm thick rubber membrane and placed in a pressure cell filled with water, 20 cm in diameter and 50 cm in height, Fig. 11. Flow of water through a PD specimen was upwards from its lower end to its top. While free vertical deformations of 0, 15 and 20% were given to the PD specimens in Cases I, II and III, sharp bends were made and held firmly by clips over the full width in Cases IV and V.

A general trend of decreasing discharge capacity with increasing confining pressures is seen in Fig. 12 which shows the test results of Case I (when i = 0.9 and where the PD specimens stood straight with no axial deformation) as well as similar data obtained by others (Hansbo 1986 and Holtz, et al 1989).

Miura, et al (1993) also gives part of their test results of Cases I through V in Fig. 13, together with the data obtained by others (Holtz, et al, 1989 and Pradhan, et al 1991). The sharp bends given to the PD specimens resulted in a drastic reduction of q_w; in the most extreme case it decreased to only 26% of the initial value of the initially straight specimen.

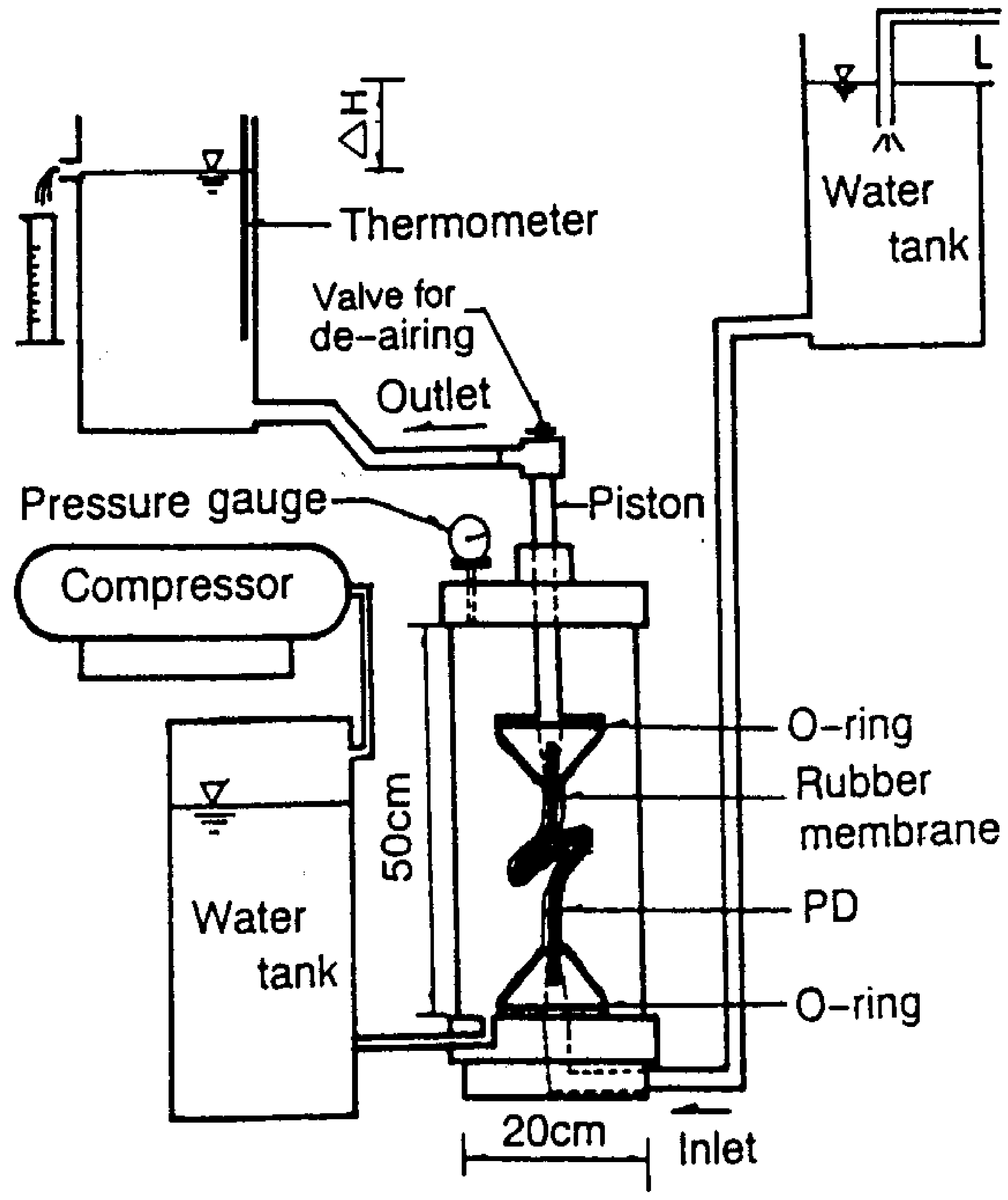

Fig. 11 Test apparatus for longitudinal permeability of a PD (after Miura, et al 1993)

Kamon, et al (1994) also conducted a laboratory investigation on the discharge capacity of straight and deformed PD's placed in a triaxial cell, illustrated in Fig. 14. Water is supplied from the bottom and flows upwards through a PD specimen sealed by rubber membrane and subjected to increasing cell pressures up to 320 kPa. The

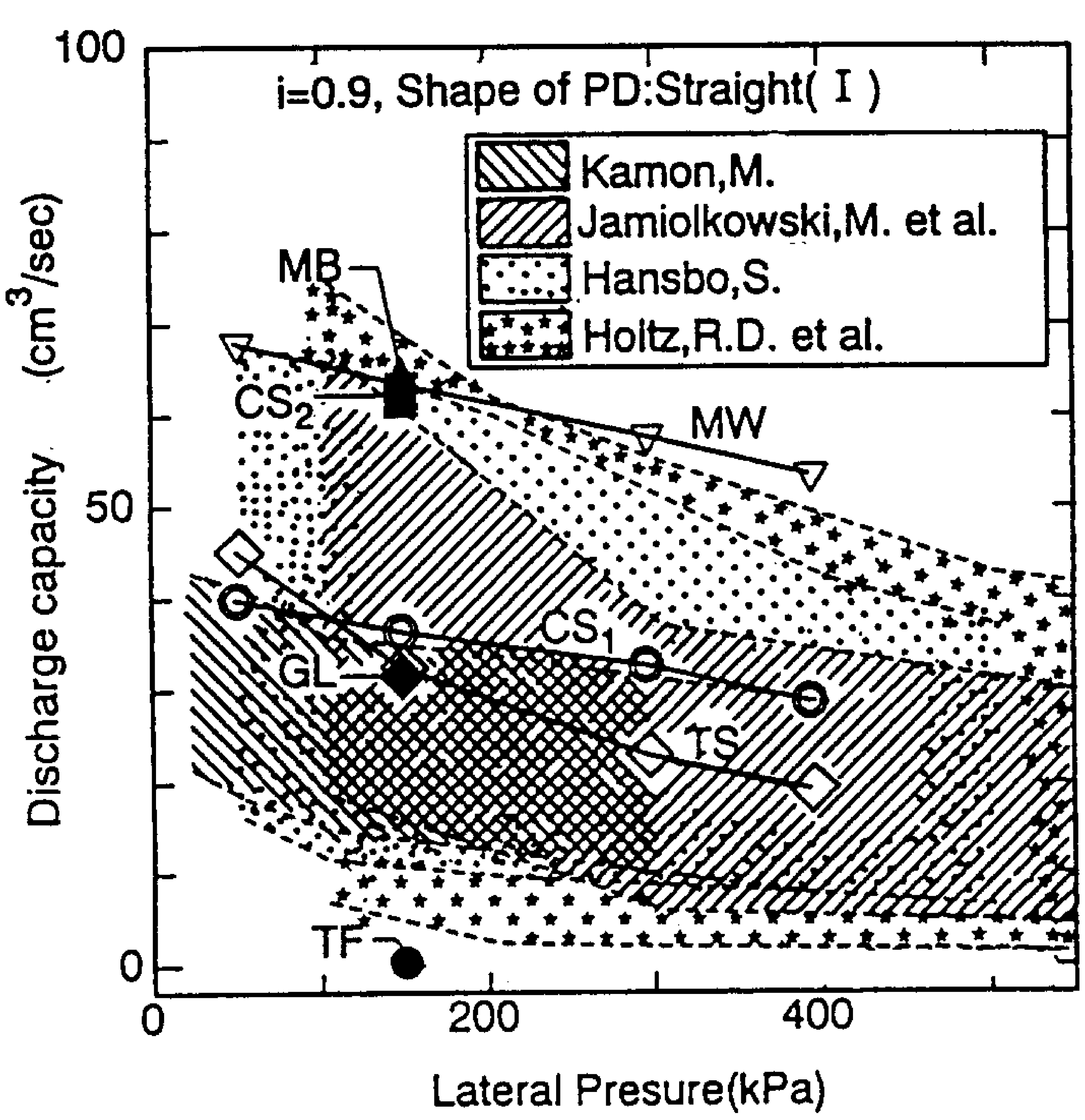

Fig. 12 Variation of discharge capacity versus lateral pressure (after Miura, et al 1993)

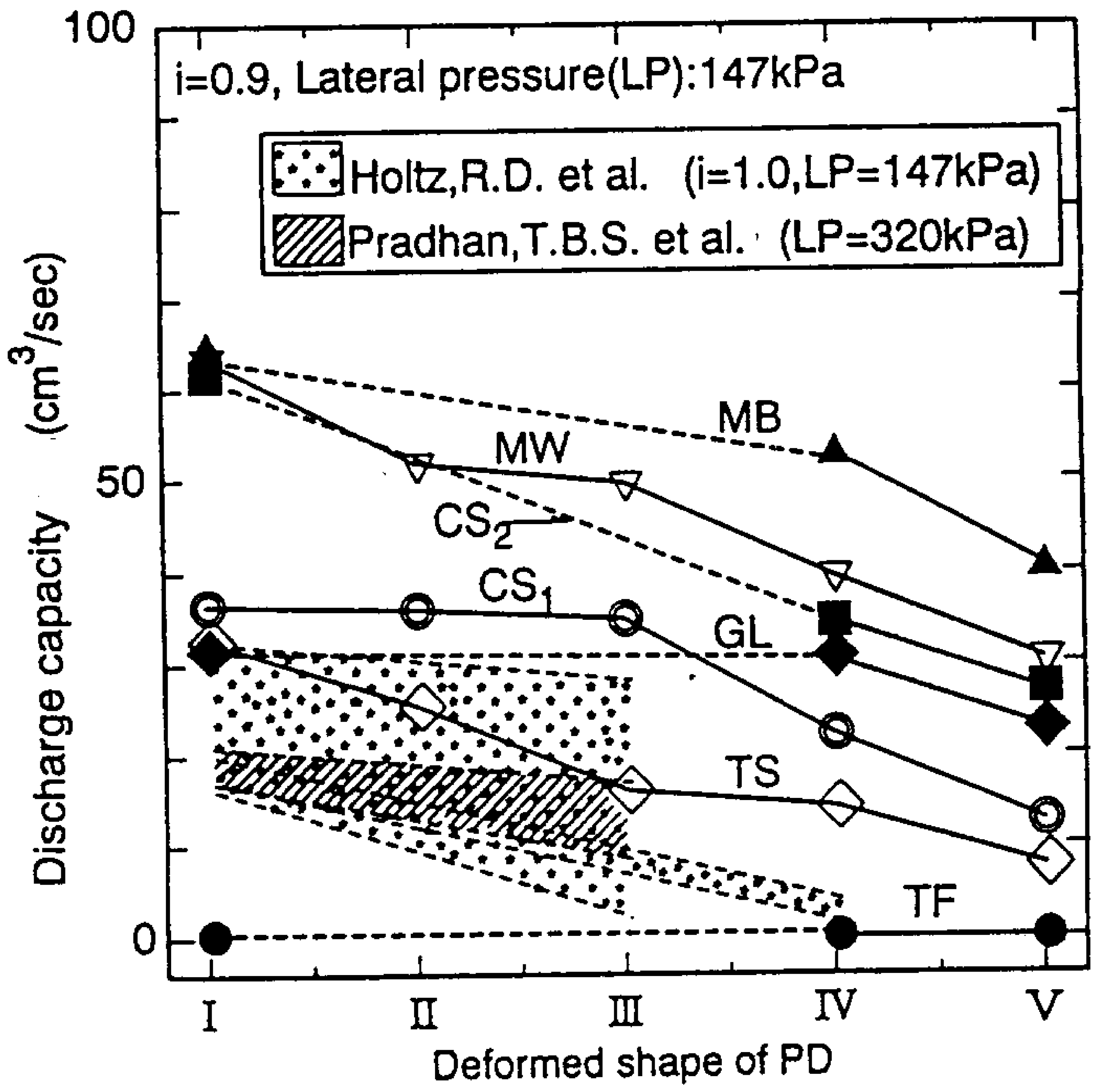

Fig. 13 Variation of discharge capacity of deformed PD's (after Miura, et al 1993)

properties of 9 different PD's tested are tabulated in Table 4. The PD specimens tested were 5 cm wide and 10 cm long.

When tested with no axial deformation given, the discharge capacity decreased with increasing confining pressures, Fig. 15. Some of them lost their capacity more markedly when the cell pressure increased from 120 to 320 kPa which falls still within a range of lateral pressures expected when PD's are driven to great depths. It is to be noted that the effect of confining pressure is significant but also the discharge capacity varies over a wide range depending on the type of products.

Table 4 Properties of various PD's (after Kamon, et al 1994)

Symbol	*Average thickness (mm)*	*Porosity (%)*	*Type*	*Actual width (mm)*
A1	2.80	49.8	1	94
A2	2.60	48.7	1	94
B	3.70	59.6	2	97
C	3.43	65.0	2	96
D	5.10	72.3	2	98
E	6.10	62.3	3	103
F	3.71	—	1	97
G	4.75	—	2	99
H	5.21	—	2	101
I	3.70	—	2	97

Type 1 : Composite drain with a filter sleeve attached to the profiled core
Type 2 : Composite drain with a filter sleeve surrounding the profiled core
Type 3 : One-piece, non-woven fabric drain

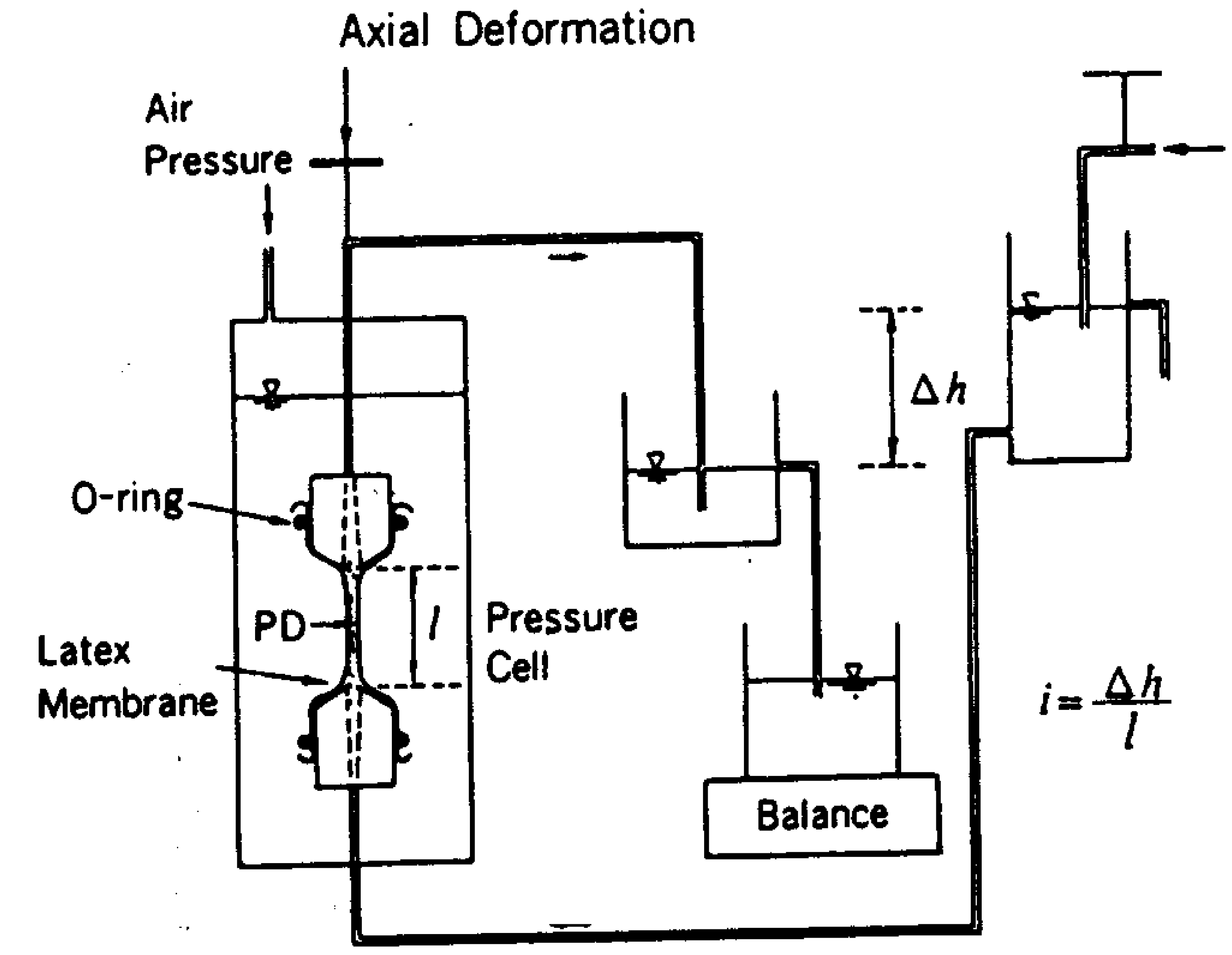

Fig. 14 Schematic diagram of the test apparatus (after Kamon, et al 1994)

In Fig. 15 the PD specimen 'I' (shown by star marks) was cut out of the PD (the same PD as Drain B) excavated from a shallow depth about 9 months after installation in a dredged clay fill which had been compressed under an estimated overburden pressure of the order of 130 kPa. As compared with the discharge capacities of Drain B, it appeared that those of Drain I were reduced to 40 to 50% of its initial values reflecting the effect of severe field conditions. It was considered, however, that Drain I retained a sufficient capacity to drain the consolidating clay around it.

When the PD specimens were subjected to increasing axial strains, their discharge capacities decrease correspondingly. Fig. 16 shows the experimental

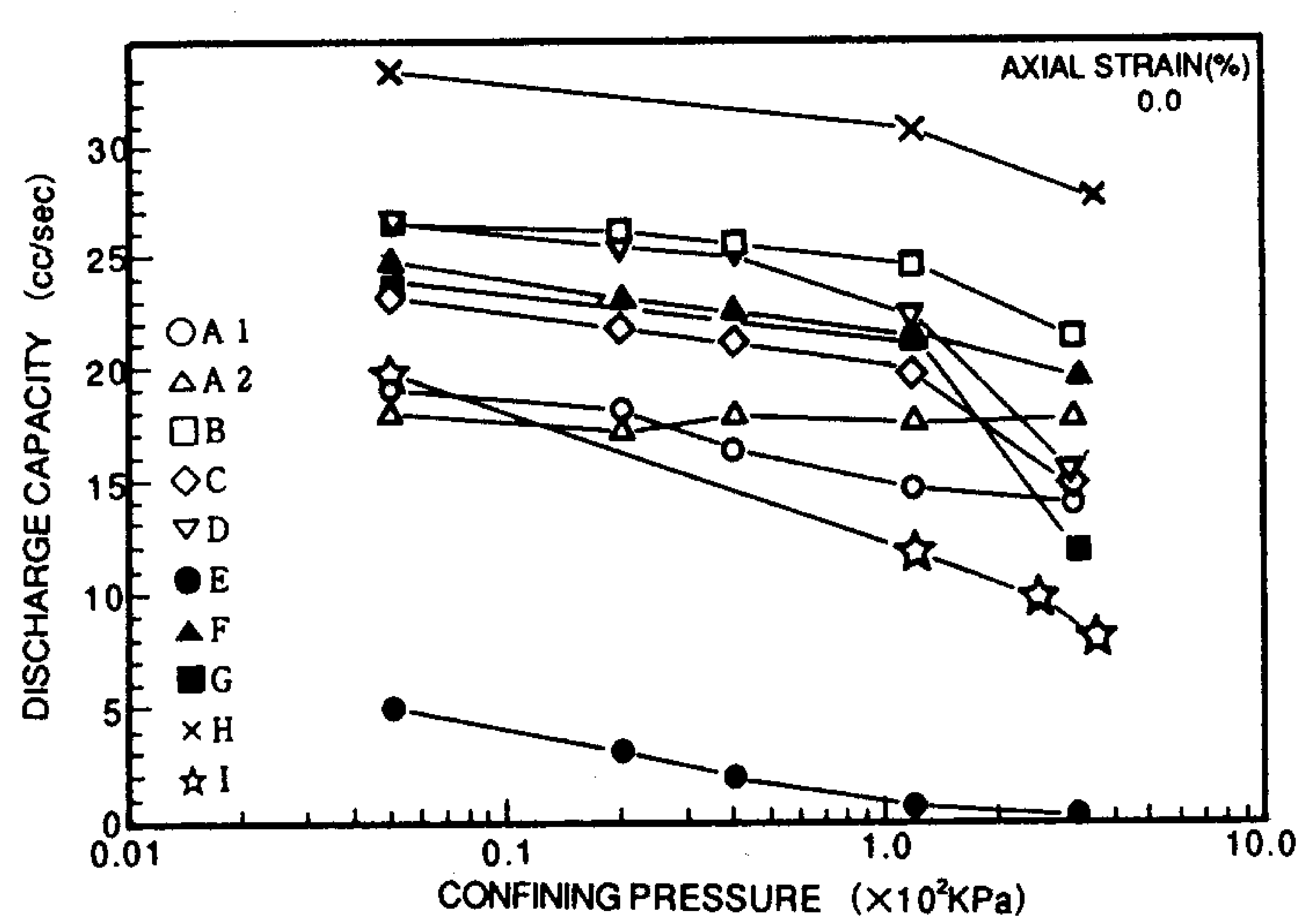

Fig. 15 Effect of confining pressure on discharge capacity for no axial strain (after Kamon, et al 1994)

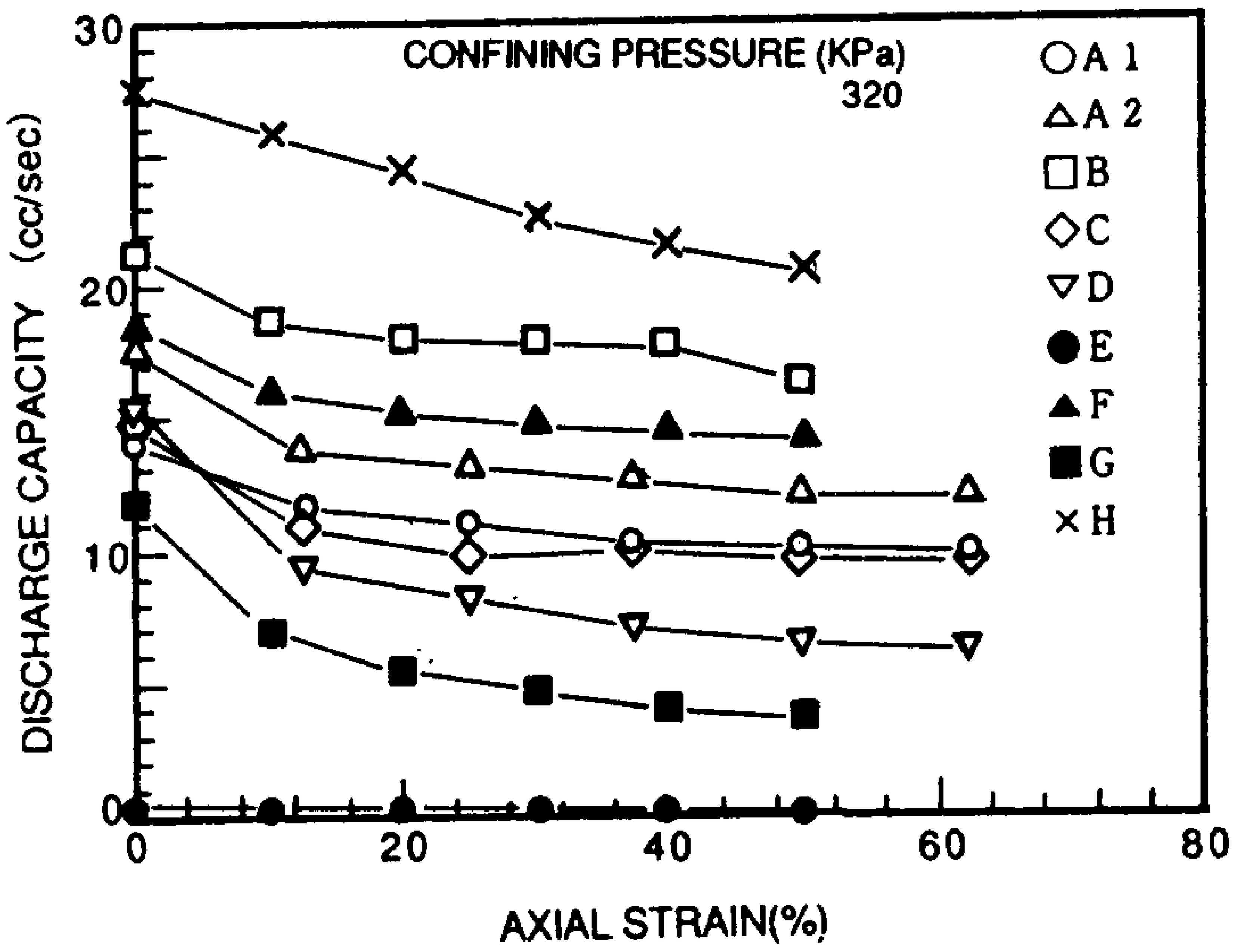

Fig. 16 Effect of axial strain on the discharge capacity for a confining pressure of 320 kPa (after Kamon, et al 1994)

results when the cell pressure was maintained at 320 kPa, indicating that the discharge capacity reduces to 35 to 70% when the axial strain reaches 50%.

Kamon, et al (1994) showed that the cross-sectional area of a PD when confined at a cell pressure of 320 kPa could reduce to 55 to 90% of that measured at a cell pressure of 5 kPa. Reduction rates of 0.35 to 0.75 were indicated also by Miura, et al(1993).

As shown in Fig. 17, Kamon, et al (1994) shows an increasing trend of the discharge capacity, q_w, of PD specimens having lengths of 10, 15 and 20 cm under confining pressures of 0.05 and 2.5 kgf/cm^2 with the hydraulic gradient, i, increasing from 0.5 to 2.5. On the other hand, as shown in Fig. 18, Park, et al (1994) gives the data of Drains CS_1 and MW (refer to Table 3) showing a decreasing trend of the q_w value with the value of i increasing from 0.2 to 0.9 under a cell pressure of 147 kPa. These specimens tested were 10 cm wide and 40 to 50 cm long.

While the type of PD's, the range of i values, lateral pressures applied and specimen sizes are all different in the foregoing tests (Figs. 17 and 18), laboratory determination of the discharge capacity is known to be affected significantly by such factors as head loss in the test device, entrapped air in the device and the PD specimen, etc.

The flow through longitudinal channels inside a PD is likely to be laminar when the i value is relatively small, most likely when it is less than 0.5. When it is greater than 1.0, it is probable that the flow could be turbulent. Park, et al (1994) reports a case study in which the i values estimated in the field ranged from 0.03 to 0.8, and concludes that water coming out of the consolidating clay and seeping into a PD should constitute a laminar flow when flowing through channels in the PD core and therefore, the value of q_w should be measured for an i value ranging from 0.2 to 0.5 in the laboratory.

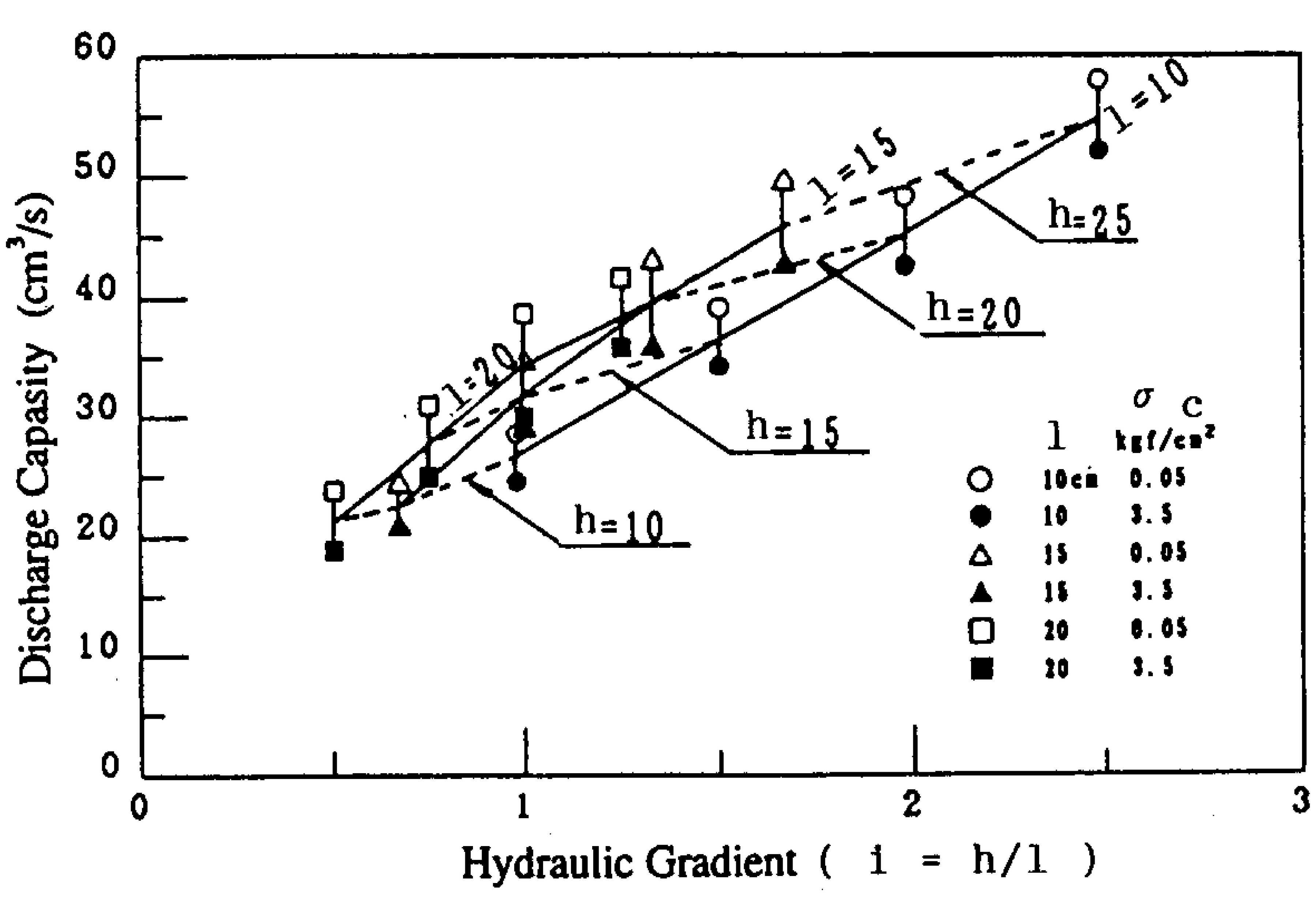

Fig. 17 Effect of hydraulic gradient on discharge capacity (after Kamon, et al 1994)

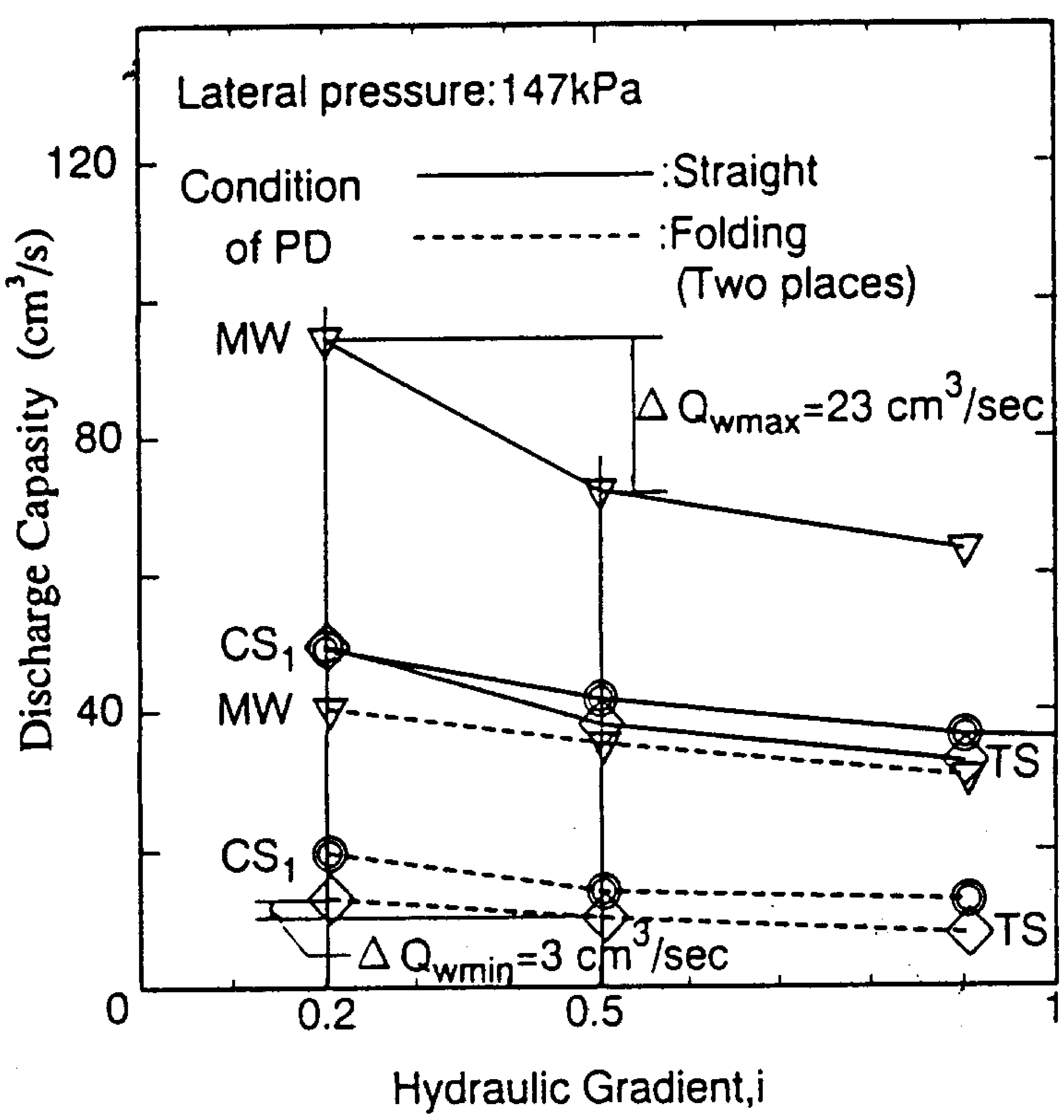

Fig. 18 Hydraulic gradient versus discharge capacity (after Park, et al 1994)

It has been noted that the q_w value decreases with time during laboratory tests. It appears to take a few days to more than a month before the measurement attains a constant value depending upon the type of a PD and test conditions.

3.6 Well resistance of and smear around prefabricated band-shaped drains (PD)

3.6.1 Analysis on smear and well resistance

Installation of displacement type vertical drains inevitably disturbs the clay around each drain reducing its permeability considerably, and this is known as the smear effect. The resistance against the flow or the head loss that occurs in the flow through the vertical drain is called well resistance.

Well resistance and smear effect were both considered when Barron (1948) first formulated his solution for radial consolidation around a vertical drain. Hansbo (1979) modified Barron's equations and presented simple formulas taking into consideration combined effects of well resistance and smear.

Simple design charts were developed on the basis of Hansbo's approach, indicating that the smear effect tends to increase with the value of k_h/k_s, while the effect of well resistance tends to approach a certain finite value as the drainage length increases (Bergado, et al 1993b).

Also on the basis of Barron's approach, Yoshikuni (1979) gave an approximate solution for the average degree of consolidation to be attained by installation of vertical drains with the well resistance taken into account. Miura, et al (1993) reports, however, that the effect of the coefficient of well resistance defined by Yoshikuni is in general relatively small on the computed average degree of consolidation when they deal with PD's in the highly plastic Ariake clay even when H is as great as 50 m and therefore, concludes that it can be disregarded from a practical standpoint at least when H does not exceed 50 m.

3.6.2 Effect of well resistance

In theoretical treatment of the effect of well resistance, a finite drain permeability is imposed on the continuity equation of flow toward the drain on the assumption that the flow rate in the considered section of the drain is equal to the maximum flow rate discharged through the drain (Hansbo 1979 and 1981). As has been noted, the discharge capacity of a PD varies considerably depending upon a number of factors.

Miura, et al (1993) summarizes factors affecting the well resistance of PD's; a) lateral pressures causing reduction of the cross-sectional area, A_f, of flow channels in the core (reducing the value of q_w to 0.3-0.85), b) large consolidation settlement of the surrounding clay causing significant deformation of PD which reduces A_f (0.25-0.6), c) temperature difference between laboratory and field conditions causing changes in the viscosity of water (0.9), d) intrusion of fine particles causing clogging of filter and core (perhaps negligible), e) entrapped air which could cause very considerable decrease in q_w if the test apparatus allows presence of air bubbles in it and in the PD specimen being tested, and also if the hydrostatic pressure in groundwater decreases releasing air bubbles in flow through clay and PD's (very considerable), f) changes in hydraulic gradient causing the flow through channels in the core to be laminar or turbulent (0.6-0.85), g) creep deformation of core and filter materials resulting in reduction of A_f (0.55-0.7), and h) length of PD increasing the head loss of flow through PD's. They point out a necessity that the design value of q_w should be reduced to a fraction of the initial value.

In connection with the foregoing consideration of the actual value of q_w, the allowable flow rate to be used in a PD design should be evaluated on the basis of the ultimate flow rate as determined by a standard in-plane flow rate test (e.g., ASTM D4716) which should be reduced by dividing it by the combined factors of safety; a) for elastic deformation of the adjacent geotextile into the drainage core space (1.5-2.5), b) for creep deformation of the drainage core itself and/or of the adjacent geotextile into the drainage core space (1.0-2.5), c) for chemical clogging and/or precipitation of chemicals onto the geotextile or within the drainage core space (1.0-1.2), d) for biological clogging of the geotextile or within the drainage core space (1.0-1.2) and e) for kinking of the PD (1.0-4.0). The numbers in parentheses indicate the recommended preliminary values of the factors of safety (Koerner, 1994).

As a case history on such a decrease of q_w, Koda, et al (1989) reported, for instance, that the PD's installed in an organic clay layer were excavated 250 to 1000 days after installation and tested for q_w, which had been reduced by 50 to 70 % under confining pressures of the order of 250 kPa, but still exceeded 350 m^3/year.

3.6.3 Effect of smear

Among different installation methods for PD's any of which influences the drain performance considerably, the displacement type is most popular employing a steel mandrel which penetrates into soft soil by static or vibratory force applied to it. The mandrel encases a PD to protect it during installation and creates the space for the PD by displacing the soil. This forced displacement very considerably disturbs the soil in the immediate vicinity of the PD installed creating a smear zone. The degree of disturbance and the extent of the smear zone depend upon the size and shape of the mandrel with respect to the those of the PD as well as the soil properties, the installation method, etc.

While obviously the cross-section of the mandrel has to be minimized, the adequate stiffness must be given and a sufficient space must be provided for a PD not to be structurally damaged nor to be subjected to excessive friction when the mandrel is being withdrawn.

It is interesting to note that the displacement type installation of a PD displaces once (even though very briefly) a much greater volume of clay than the volume of the PD to be installed, while displacement type sand drains are installed displacing roughly the same volume of clay as that of the mandrel. This difference gives a different impact on the disturbance of the clay in terms of the permeability and the stress history which is yet to be investigated.

Among the studies made on the diameter of a smear zone, d_s, with respect to the diameter of the mandrel, d_m, on the basis of some case records, Hansbo (1981) recommends the use of a relationship:

$$d_s = 2\ d_m \tag{15}$$

He also points out that the influence of smear increases with increasing diameter of the mandrel employed for installation of PD's.

Miura, et al (1993) also made an experimental study in the laboratory to investigate a smear zone created by installation of a driven PD and concluded that:

$$d_s = (2 - 3)d_m \tag{16}$$

Their theoretical analysis in which the permeability reduced by smear is taken into account indicates a very considerable delay in reaching any degree of consolidation. Mesri, et al (1994) also reports that:

$$d_s = (2 - 4)d_m \tag{17}$$

3.7 Some case records of installations of prefabricated band-shaped drains (PD)

3.7.1 PD installation in Bangkok, Thailand

The effectiveness of PD in soft Bangkok clay has been investigated by extensively instrumented field experiments conducted on the AIT campus north of Bangkok, Thailand (Bergado, et al 1990, 1993a and 1993b).

Full size PD's having a cross section of 3 x 95 mm were installed in a triangular pattern on 1.5 m spacings down to a depth of 8 m over a test area 14.6 x 16.6 m (Bergado, et al, 1990). They were driven into clay by means of a special mandrel with the inner dimensions of 28 x 133 mm and the outer dimensions of 45 x 150 mm, which was designed to minimize the smear effect. Disposable shoes were attached to the bottom of each drain for anchorage. A 4 m high test fill was constructed over the test area exerting a vertical stress of 65 kPa on the ground surface. The measured settlements indicated that 90% consolidation was achieved in 430 days after placement of the fill.

These PD's were designed on the assumption that d_s/d_w = 2.5 and k_h/k_s = 10, where d_s and d_w are the diameter of the smear zone and the equivalent diameter of a drain, respectively, and k_h and k_s are the coefficient of permeability of the in-situ clay in the horizontal direction and the disturbed clay in the smear zone, respectively. It was reported that a reasonably good agreement was obtained between the observed time-settlement data with those predicted by an analysis which considered vertical and horizontal consolidation with both smear and well resistance effects taken into account.

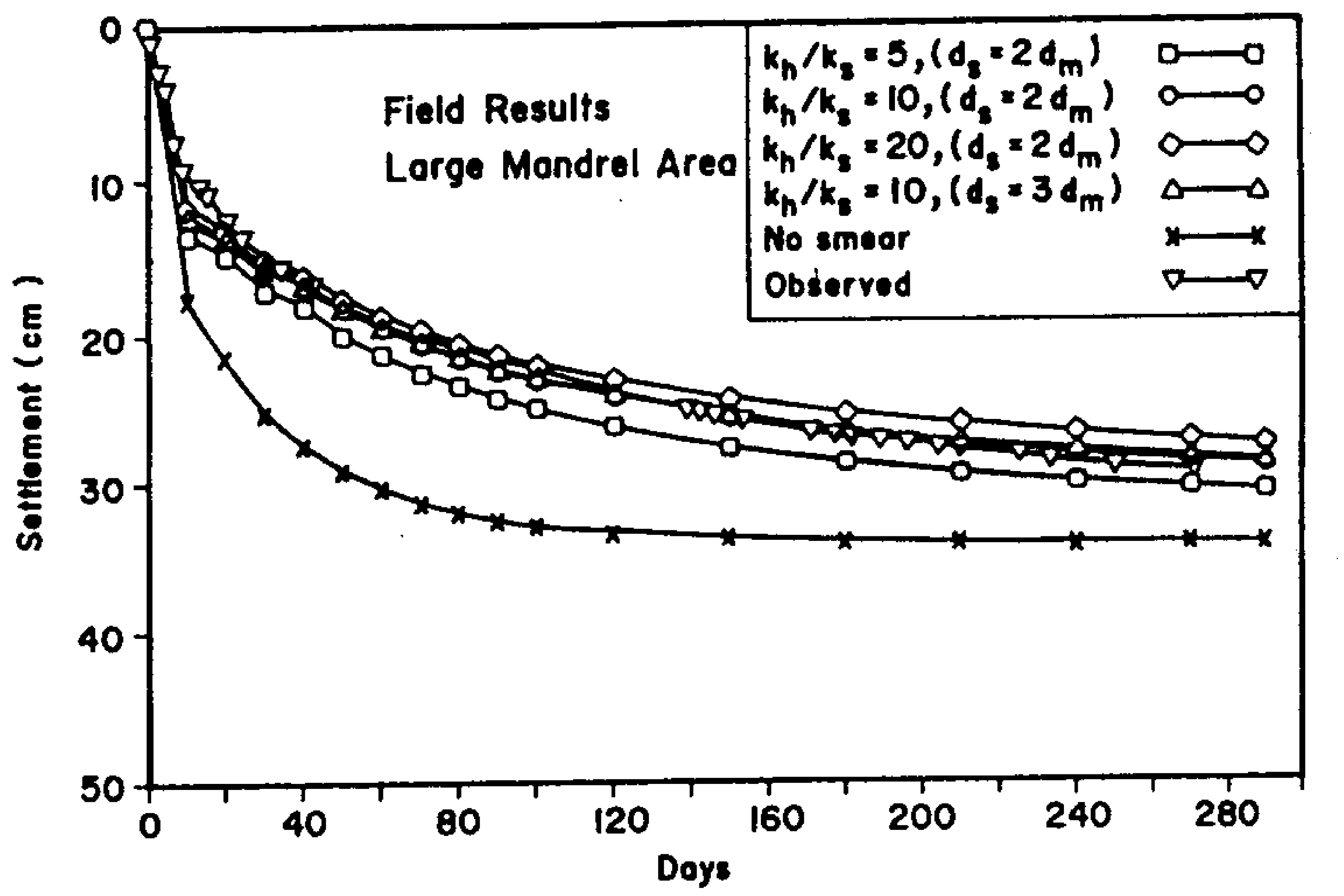

Fig. 19 Observed and predicted time-settlement relationships in the large mandrel area, AIT campus (after Bergado, et al 1993a)

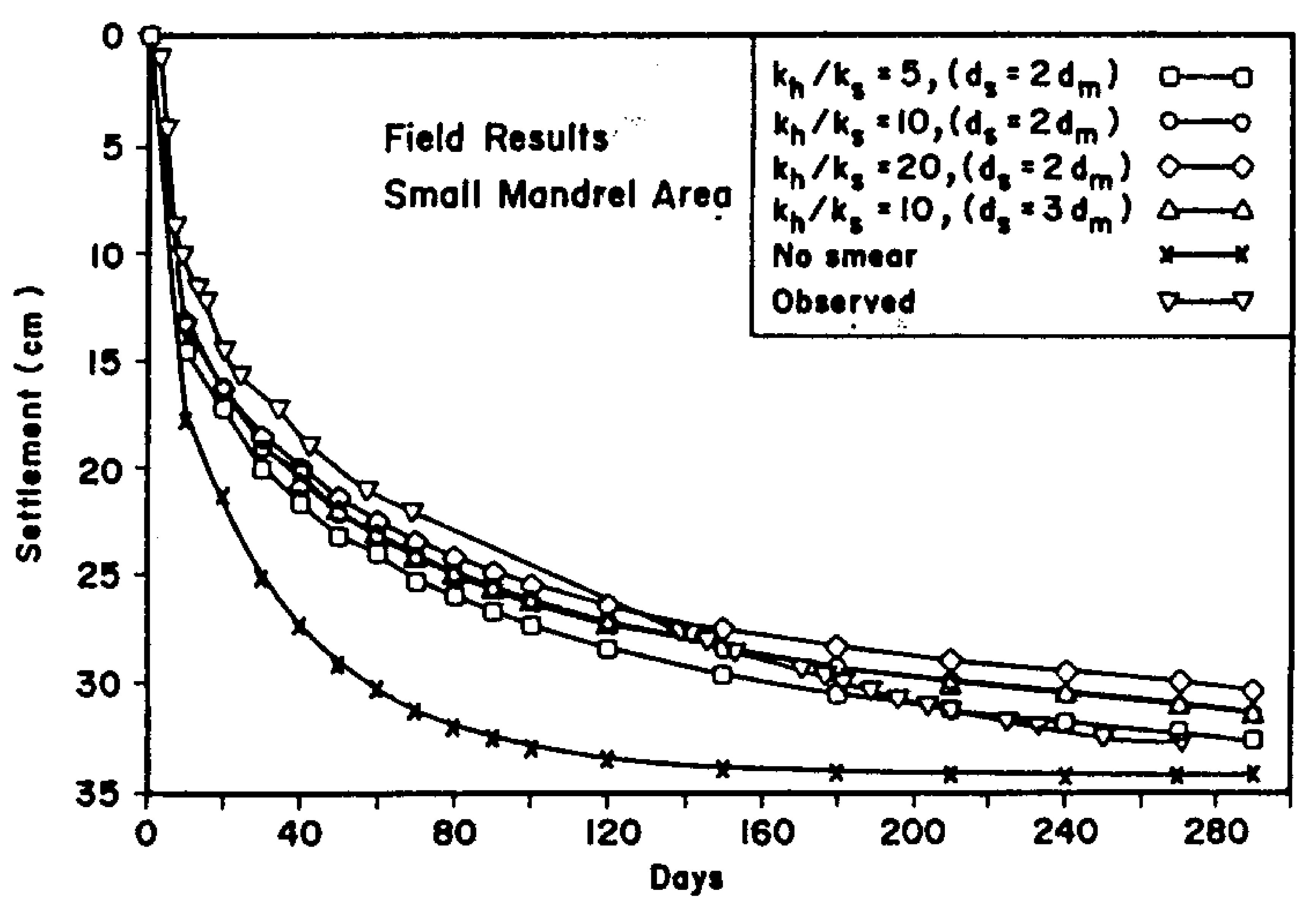

Fig. 20 Observed and predicted time-settlement relationships in the small mandrel area, AIT campus (after Bergado, et al 1993a)

The subsurface conditions are quite complex at this site, consisting of the following; 1) 0-2 m: heavily overconsolidated clay, 2) 2-8 m: soft clay with fine sand, silt seams and 3) 8-9 m: top of stiff sandy clay, about 6 m thick, with the groundwater table fluctuating from 0.5 to 2.5 m below the ground surface. In addition, the oedometer test results indicated the c_h value being 3 to 4 times as large as the c_v value and the back analysis of the field data showed the field value of c_h being 4 times as large as that determined in the laboratory.

A further attempt was made to investigate the effect of the size of the mandrel by conducting a full-scale field test on the AIT campus (Bergado, et al, 1993a). Prototype PD's having a cross-section 6 x 100 mm, a type different from the preceding case, were installed in a square pattern on 1.2 m spacings to a depth of 8 m over a test area, 19 x 15 m. The test site was divided into two halves, each area being 9.5 x 15 m. The PD installation was carried out by driving a small mandrel with outer dimensions of 45 x 150 mm in one half of the site, and by a larger mandrel, 150 x 150 mm, in the other half.

A 1 m thick drainage blanket of clean sand and a 4 m high test embankment were placed over the test area. Figs. 19 and 20 show the observed and the predicted time-settlement curves in the large mandrel area and in the small mandrel area, respectively. The predicted curves were obtained by means of a finite element analysis with the smear effect taken into account using variously assumed values of k_h/k_s (=5, 10 and 20) and d_s/d_m (= 2 and 3).

It was concluded that the predicted settlements agreed well with the field measurements when it was assumed that k_h/k_s = 10 and d_s/d_m = 2. Furthermore, it was concluded that a faster settlement rate and slightly higher compressions were observed in the small mandrel

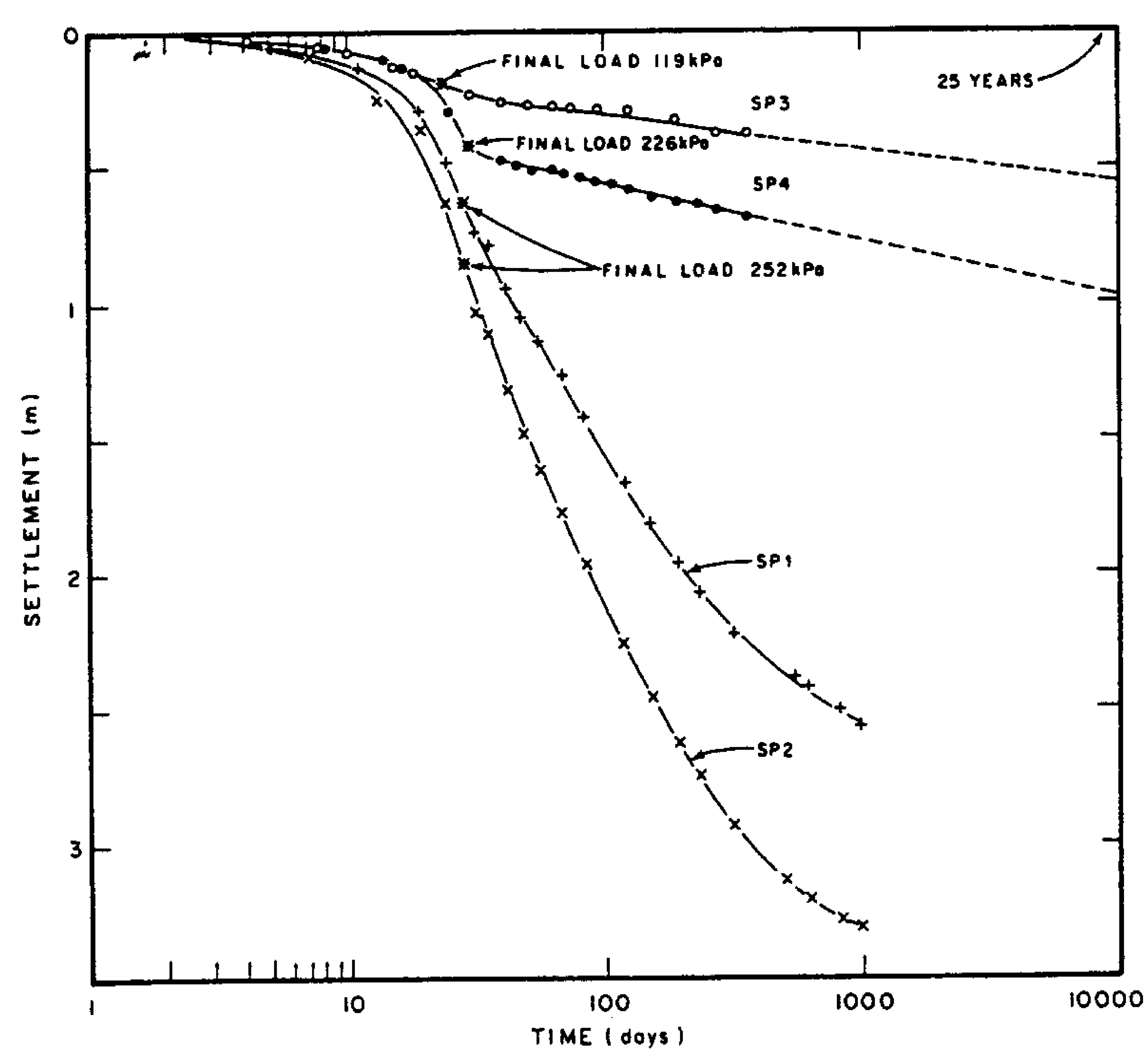

Fig. 21 Observed time-settlement curves in the PD drained area and the undrained area, British Colombia (after Crawford, et al 1992)

area than those in the large mandrel area, suggesting that the former area had a smaller smeared zone than the latter (Bergado, et al, 1993a).

3.7.2 PD installation in British Colombia, Canada

Crawford, et al (1992) presented a case study on two well-instrumented test fills constructed on soft clay; one with PD's and the other without, in Venon, B. C., Canada. The former test site is roughly 84 x 95 m, while the latter covers an area 40 x 84 m. PD's were driven to a depth of 24 m on a triangular pattern with 1.2 m spacings.

The subsurface conditions are quite complex consisting generally of the following; 1) 0-4 m: alluvial silt, sand, and clay, all soft to loose, 2) 4-8 m: clay, stiff to very stiff, 3) 8-33 m: clay, soft to firm, 4) 33-53 m: clay, stiff to very stiff, and 5) 53-64 m: surface of dense till. The groundwater table is at a depth of 1-2 m.

Fig. 21 shows the time-settlement curves recorded by four settlement plates placed at the base of the two test embankments; SP1 and SP2 are the settlement plates installed in the PD drained site, while SP3 and SP4 are those placed in the undrained area. Since the embankment loads as indicated in Fig. 21 are comparable for SP1, SP2 and SP4, it is apparent that PD's are effective in accelerating consolidation of the subsoil.

These field data as well as porewater pressure measurements were carefully analyzed. It was concluded, however, that prediction of consolidation settlements did not give realistic results. The equivalent diameter of the PD having a cross-section 3 x 100 mm was assumed to be 63 mm for the sensitivity analysis. This analysis indicated that the equivalent diameter did not affect the back analysis of the consolidation settlement as significantly as the permeabilities of the disturbed and undisturbed soil, or their ratio. It was concluded that the coefficient of consolidation in radial drainage, c_h, is the most important parameter for any realistic prediction of the settlement behavior of PD-installed embankments.

3.7.3 PD installation in Osaka Bay, Japan

For construction of a large man-made island for Kansai International Airport

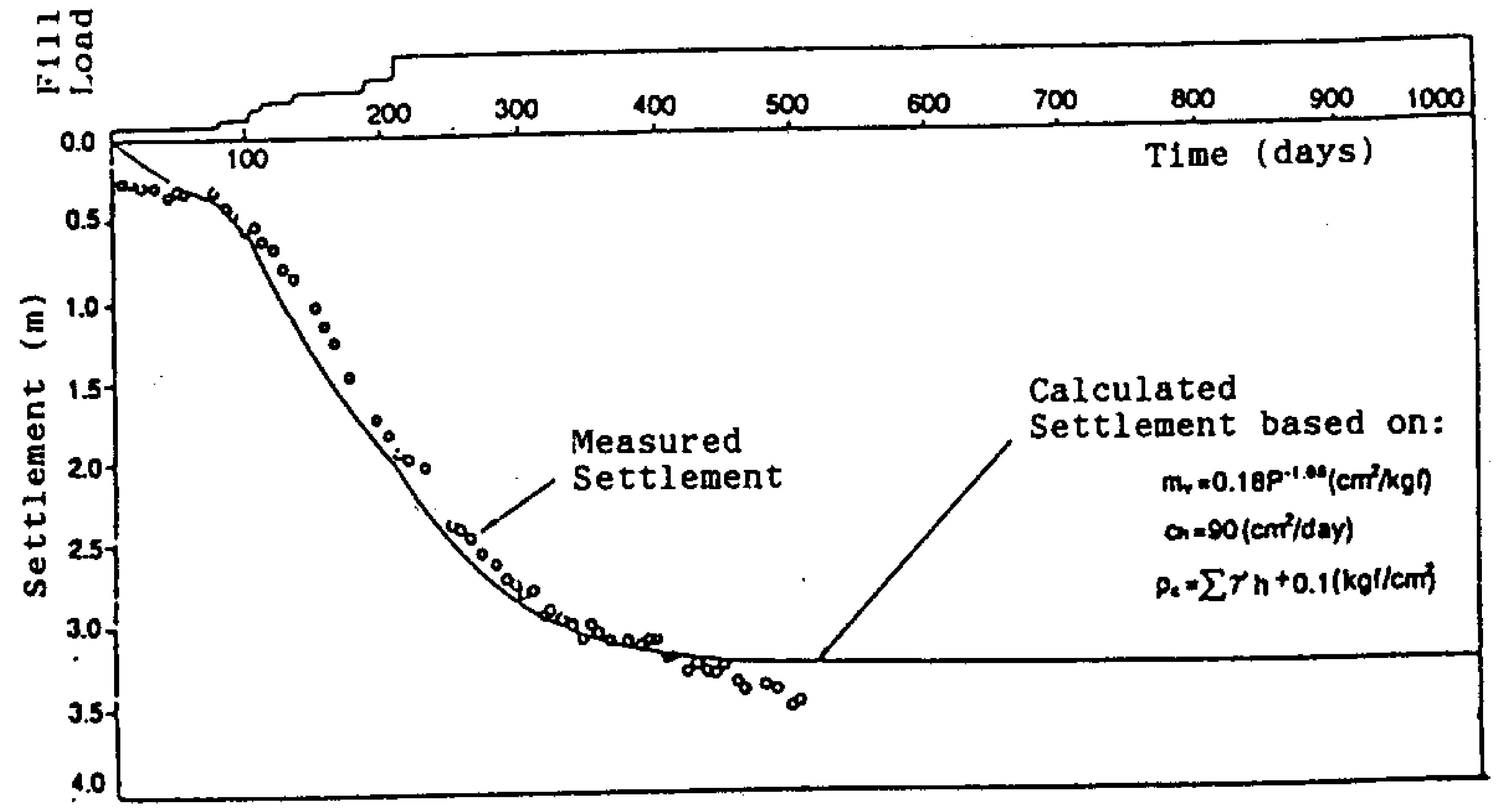

Fig. 22 Measured and calculated time-settlement relationships in the PD drained area, KIA island (after Oikawa, et al 1989)

located 5km offshore in Osaka Bay, Japan, approximately one million sand drains were installed over the site of the 511 hectare island. In this area the mean sea depth is 18 m and the seabed consists of a very soft clay layer having an average thickness of 20 m underlain by alternating layers of compact sand and gravel, and stiff clay to an undetermined great depth.

In addition to 40 cm diameter sand drains, a total of 768,800 m of PD's was driven in two areas; Area A, roughly 118 x 118 m at the south-west corner of the artificial island and Area B 198 x465 m located not too far away from the former. PD's were installed by a barge-mounted driving equipment capable of installing 8 PD's simultaneously.

The mandrels, each 165 mm in diameter, were lowered through 17 m of sea water on the average in these areas and pushed through a 1.5 m thick sand mat and then into very soft clay until the plate anchor reached the underlying sand stratum. The load applied to the mandrels were monitored throughout the installation to see when the tip reached it. When the mandrels were pulled up out of the sand mat, the PD's were automatically cut off underwater.

In Area A, 6,500 PD's were pushed into clay in square patterns on 1.3 m spacings, having a mean length of 14.2 m. In Area B the average length of 32,100 PD's installed was 21.1 m and they were installed also in square patterns on 1.7 m spacings.

In Fig. 22, small circles show the time-settlement records of a settlement plate placed on the sand mat in Area A where subsequently a sand fill and stones were dumped from barges. Based principally on Barron's method (the equivalent diameter of a PD was assumed to be 6.0 cm), the measured settlements were constantly reviewed and back-analysed. The solid curve in Fig. 22 shows the best fit obtained using the values of c_h and m_v which were determined from the back-analysis of the observed time-settlement data. Such a curve served to estimate the future settlement.

It was concluded that the PD installations at this site gave approximately the same results in terms of accelerated settlements and increased undrained strengths as those observed in the sand drained areas and therefore, were effective in spite of the great depths involved and difficulties in offshore operations (Oikawa, et al 1989 and Hashizume, et al 1992).

4 CONCLUDING REMARKS

In the field of geotechnical engineering the subject of filtration and drainage is a difficult one to deal with in an analytical way due principally to the fact that soil is not homogeneous nor hydraulically isotropic. In addition, the validity of a basic principle like Darcy's law depends upon so many factors geotechnical engineers have to be satisfied with determination of the coefficient of permeability of soil which is expressed normally as something of the order of magnitude rather than a range of numbers of the same order.

The availability of geosynthetic products are rapidly replacing the traditional filters and drainage layers consisting of granular soils. It is only those replaced portions, however, that are now relatively easy to handle more logically. It is to be noted that the soil, natural or artificially placed, is there to be filtered and/or to be drained by the man-made material called a geosynthetic which has been considerably improved and better understood than its predecessor, although neither perfectly well defined nor 100% under our control all the time.

As we have just discussed in some detail, prefabricated band-shaped drains (PD's) have far superior engineering properties than those of the old fashioned sand drains in terms of hydraulic properties, strength characteristics, durability, reliability, ease of placement and even economy. But PD's are still installed essentially in the same way, disturbing the soil around them and being influenced by the surrounding soil probably to a somewhat less extent but suffering from the same old problems.

The smear effect, well resistance, intrusion of clay into sand or clogging, large deformations of sand columns resulting in objectionable necking and even complete severing, etc., have been problems of many sand drain installations and PD's are immune from none of these if not called by exactly the same terms.

The main purpose of vertical drains, whether they are sand drains or PD's, is to accelerate significantly drainage from a thick clay stratum, i.e., to increase the time rates of settlement. This can truly be verified only by comparing the field settlement records taken of the drained area and an adjacent undrained area with the same subsurface and loading conditions, not merely by an agreement between the observed data and the theoretical prediction. There have been a number of sand drain installations which did not accelerate settlements significantly enough or even totally failed to achieve the purpose. Indeed, there were quite a few cases of sand drain installations where sand drains were not needed at all (e.g., Akagi 1989).

In this connection, the following quotation appears to be appropriate to conclude this keynote paper: "Since the rate of reported failures of geotextile filters is low, a reasonable conclusion might be either that the design rules are conservative or, taking a jaundiced view, that geotextiles always work when they are not needed (Ingold 1993)."

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