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ABSTRACT

Loose cohesionless saturated materials have proved responsible for a number of serious or catastrophic flowslides. Liquefaction failures with no obvious triggering mechanism have also been recorded. This phenomenon of sudden liquefaction without a presence of cyclic shear stresses is often referred to as spontaneous or static liquefaction. Results from previously published studies suggest that liquefaction is triggered not by the undrained loading and generation of pore pressures but by the collapse of the metastable sand structure which in turn generates the driving pore pressures in a saturated material. Hence, the collapse is a characteristic response of a material to certain stress states rather than a result of some enforced undrained loading. This theory is evaluated on very loose dry Ottawa sand. It is shown that the very loose dry sand when subjected to a constant deviatoric stress path significantly changes its behavior at a certain discreet stress state, increases compressibility and becomes increasingly unstable. This results in collapse - vigorous contraction of the specimen. This structural collapse appears to be equivalent to the pore pressure generation in collapsing very loose saturated sand.

Key words: dry sand, collapse, liquefaction, stress path, triaxial cell

COLLAPSE OF DRY SAND

INTRODUCTION

Liquefaction is an important phenomenon. Dramatic effects of earthquake triggered liquefaction failures in 1964 in Japan and Alaska marked the onset of a concentrated research effort to understand the triggering mechanisms of such events. The result of this endeavor is a fairly solid understanding of the behavior of sands under cyclic loading conditions.

Nevertheless, failures have also been observed for which no source of cyclic loading has been identified. Terzaghi (1956) describes several cases of submarine slope failures where only minor triggering mechanisms could be detected. He refers to this phenomenon of sudden liquefaction as **spontaneous liquefaction**. Such failures of natural or man made slopes, commonly called **flowslides**, are believed to be initiated by minor changes in stresses such as ground water table fluctuations or toe erosion, hence essentially by static loading. Therefore, today the term spontaneous liquefaction is often used interchangeably with the term **static liquefaction**.

Casagrande (1936) described an experiment in which a weight had been placed on the surface of a very loose saturated sand and a stick was driven into the sand adjacent to the weight. The slight disturbance produced by the penetration of the stick initiated a collapse of the essentially metastable sand structure which quickly propagated throughout the entire mass with the simultaneous generation of pore pressures. The pore pressures decreased the frictional resistance so that the weight could no longer be supported and it sank below the surface. The stick penetration essentially simulates a minor change in stresses in a metastable sand structure resulting in statically triggered liquefaction.

Further studies on liquefaction behavior of sands under static loading (e.g. Castro 1969, Castro and Poulos 1977, Poulos et al. 1985, Kramer and Seed 1988, and Alarcon-Guzman et al. 1988) concluded that for liquefaction to occur the material must be loose, i.e., contractive during shearing at large strains, and the loading path must not allow for excessive volume changes prior to the collapse. Also the material must be saturated and have a sufficiently low permeability to sustain the generated pore pressures. The ultimate shear strength must be less than the in situ driving shear stress, i.e., there must be a potential for weakening and associated stress redistribution possibly leading to full scale liquefaction. The underlying assumption of these criteria is that the deformation during liquefaction is undrained. Based on this a number of evaluation methods and parameters have been developed for design against static liquefaction (Poulos et al. 1985, Casagrande 1976, Bishop 1967, and Kramer and Seed 1988).

Recently the concept of the collapse surface, which will be discussed later in this paper, has been introduced to explain the 1983 submarine flowslides at Nerlerk in the Beaufort Sea (Sladen et al. 1985a). A total of six liquefaction slides occurred during the construction of a subsea berm. These slides were apparently triggered by static loading arising from the hydraulic placement of sand. The void ratio of the sand within the berm was approximately 0.8 with a permeability in the order of 10^{-5} m/s. The initial slopes and height of the berm were in the range between 10° to 12° and 25 to 30 meters, respectively. The flowslides came to rest at very flat postfailure slopes of 1° to 2° after a runout of up to 100 meters.

Similar events were observed in coking coal stockpiles at northern Australian coal export terminals. The stockpiles were about 15 meters high and failed within 10 to 20 seconds reaching runouts of about 60 meters. A laboratory testing program at James Cook University (Eckersley 1990) investigated these events by conducting a series of 19 model tests to induce flowslides in an instrumented test tank. The stockpiles were modeled by placement of the moist coal in a tank 1 meter high, sloping at 36°, and at various average void ratios according to the placement method used. Flowslides were then induced by slowly raising the water table in the model from behind the slope. The failure of the loose coal berms occurred in a period from 2 to 10 seconds at movement velocities of about 1 m/s and reached flat postfailure slopes of approximately 5°. Furthermore, the instrumentation supplied information on pore pressure generation in the collapse zone. Hence, it could be established that the excess pore pressures were generated after the first movements and were consequent to the collapse.

The occurrence of similar collapse liquefaction (collapse followed by liquefaction) phenomena in coal mine waste dumps is more enigmatic. A study of Rocky Mountain coal mine waste dumps (Golder Associates - private communication) shows that there have been a number of slope failures with long runouts up to 2.5 kilometers, typical of flowslides. The common belief prevails that there is insufficient saturation inside the waste dumps because of the natural particle segregation during dumping resulting in a free draining toe. A study by Dawson et al. (1992) shows that when somewhat finer material is deposited from the dump crest a blanket of a considerable thickness can be created and is subsequently covered by successive dumping. This finer layer inside the dump can be very loose, can retain water and its permeability can be sufficiently low to sustain pore pressures necessary for the development of liquefaction flowslides.

In all cases presented above, calculations based on limit equilibrium along a distinct failure plane yielded Factors of Safety well above unity. In the case of the Nerlerk failures the calculated Factors of Safety were as high as 3 (Lade 1993). In the case of Rocky Mountain coal mine waste dumps the Factor of Safety was usually calculated around 1.2 (Dawson et al. 1994). Hence, the concepts embraced in the classical equilibrium approach were insufficient to properly assess the behavior of these loose materials. A Rocky Mountain waste dump failure and Eckersley's flowslide model test were analyzed by Dawson et al. (1992) using the finite element liquefaction model developed by Gu (1992) and Gu et al. (1993). The calculations demonstrated the progressive propagation of pore pressures due to weakening of the material in undrained loading during collapse and confirmed Casagrande's opinion that there is no need for the liquefaction of the entire failing body, since "*the overlying mass can slide out as if suddenly placed on roller bearings*" (Casagrande 1936).

The common factor of the above events are dramatic failures without warning and without any obvious trigger followed by long runouts and very flat postfailure slopes suggesting significant loss of internal friction resistance. Classical equilibrium methods of geotechnical engineering consistently overestimate the Factor of Safety of these structures, because they are based on ultimate friction values and pre-failure pore pressures. The catastrophic nature of these events, i.e., the collapse of the metastable material followed by very mobile flow liquefaction, makes this problem very real and its understanding is crucial to certain geotechnical designs.

Throughout this paper some generic terms will often be used to describe certain events or conditions of sand behavior. Therefore, in this paper the term **collapse** refers to the specific response of the very loose structure which can be exhibited either as rapid contractive strains under fully drained conditions with no pore pressure generation or as vigorous pore pressure generation under undrained conditions. The term **liquefaction** or **flow liquefaction** (Robertson 1994) refers to the behavior of a saturated material immediately after the collapse when the generated pore pressures significantly decrease the effective stresses and the mass strains in a manner resembling the flow of a thick viscous liquid. The combination of the above two terms yields the term **collapse liquefaction** to describe the behavior when the material collapse results in flow liquefaction. The term **very loose** is used in this paper to describe the metastable sand structure which can collapse at certain stress states.

COLLAPSE SURFACE AND STATE BOUNDARY

The collapse surface concept adopted to explain the Nerlerk subsea berm flowslide was first published by Sladen et al. (1985b). They observed that the peaks of the undrained effective stress path for Nerlerk sand samples at the same void ratio but at various initial stress states fell on a straight line in the p' - q (mean principal stress - deviatoric stress) plane and that these stress paths eventually reached the same state, i.e., the steady state. The line joining the peaks of the stress paths to the ultimate state was called the **collapse line**. The same material tested at different void ratios yielded collapse lines of the same slope but ending at different steady state points. The global picture could then be obtained in p' - q - e (mean principal stress - deviatoric stress - void ratio) space where the collapse lines compose the collapse surface and the ultimate state points compose the **steady state** or **critical state line**. The collapse surface can be imagined as the locus of state conditions (combination of void ratio and stress state) at which the collapse of a very loose soil structure is initiated during undrained loading. Hence, the collapse surface constitutes a triggering criterion for collapse in undrained loading.

This concept was further examined by Alarcon-Guzman et al. (1988) and Ishihara et al. (1991). They demonstrated that a **state boundary surface** can be defined in p' - q - e space above which a state condition for a particular material is not admissible. The state boundary surface is positioned above the collapse surface and is an envelope composed of the post peak portions of the undrained stress paths (Fig. 1). Sasitharan et al. (1993) demonstrated that any attempt to cross this boundary for a very loose saturated sand by a q = constant stress path resulted in collapse and flow liquefaction. Such a stress path essentially simulates the natural condition of a rising water table investigated in Eckersley's flowslide modeling. Furthermore, Sasitharan et al. (1994) showed that when a sample was brought to a stress state on the state boundary surface during undrained strain controlled loading and the loading was changed to triaxial compression or to p' = constant stress path, the state condition remained on and traveled along the same state boundary surface. These tests demonstrate that the state boundary surface controls the direction of a stress path by forcing it to deflect and follow along the state boundary surface.

OBJECTIVE OF THE STUDY

The objective of this study is to investigate the phenomenon of collapse of very loose sand by exploring the behavior of very loose dry sand. From the work by Eckersley (1990) and Sasitharan et al. (1993) it appears that the mechanism of flow liquefaction is not controlled by drainage conditions. Both studies reported triggering of flow liquefaction during fully drained, i.e., without any excess pore pressures, q = constant loading. This suggests that the liquefaction phenomenon is driven by the material response rather than by specific drainage conditions and that the generated pore pressures are the consequence and not the cause of the collapse. The absence of the pore water in a dry sand predicates fully drained conditions by definition and ensures that the behavior is controlled exclusively by the material. If the collapse is found to be a material property and shown to be related to pore pressure generation during flow liquefaction of saturated sand, a theory of liquefaction in terms of effective stresses can be later advanced.

TESTING PROGRAM

In this study samples of essentially dry Ottawa sand were subjected to triaxial $q \cong$ constant stress paths. Tests were conducted using a Wykeham Farrance loading frame modified to accommodate the application of dead load to the top of the sample. The triaxial cell was also modified accordingly. During the test, readings of axial load, cell pressure, axial deformation and sample circumference changes were recorded on a 80286 AT computer through a DAS16 interface board. A high speed data acquisition system (10 readings/second) was utilized, since monitoring of time development of volume changes was essential.

The novel technical challenge was the need for accurate volume change measurements. This is particularly difficult, since there is no pore fluid in the sample. For this purpose a special transducer for circumference change measurement was developed (Skopek and Cyre 1994). A high resistance wire, 0.063 mm diameter, 390 Ω/m was wrapped once around the sample, tensioned by a 20 g load and excited by 6V. The voltage was then measured in the wire between the two fixed points on the sample circumference. As the sample deforms the length of the wire between the two fixed points changes and this is reflected in the voltage differential signal output. The signal is amplified 50 times and recorded by the data acquisition system. This wire transducer has a resolution of 0.01

mm/division and excellent output stability. It should be noted that, due to the conductive character of the wire transducer, compressed air instead of water or oil was used as a cell pressure medium which required a little more care in cell pressure control, since the high compressibility of air hampers the control of pressure change. Sample volume was calculated after the deformed crossection was established using the circumference change readings recorded at ¼, ½ and ¾ of the height of the sample along with the axial deformation reading. The volume was calculated by integration of the deformed crossection. Once the volume was calculated, the void ratio calculation was straightforward.

Ottawa sand (C109) from Ottawa, Illinois was used for this study. This sand is a uniform, medium sand composed primarily from round to subrounded quartz grains with a specific gravity of 2.67, mean grain size D_{50} = 0.34 mm, e_{min} = 0.5, e_{max} = 0.82 according to ASTM D2049. Samples were prepared in the laboratory using the moist tamping method with about 2.5% moisture. Void ratio uniformity of moist tamped samples prepared using the same equipment and procedure was evaluated by Sasitharan et al. (1994). The samples were 64 mm in diameter and about 118 mm high. Prepared void ratios ranged between 0.75 - 0.85.

Only samples prepared by the moist tamping preparation technique were sufficiently loose for the purposes of the study. Dry pluviation produced samples too dense to exhibit collapse. It is believed that the low moisture content of about 2.5% did not excessively affect the test results and the estimated suction in the samples of less than 5 kPa was much smaller than the applied stress levels during the test.

The tests were conducted in three stages. The samples in the triaxial cell were first subjected to an all round pressure and then the deviatoric load was applied in steps up to the desired magnitude. The third stage of the test was conducted at approximately q = constant conditions. The cell pressure was slowly decreased manually at an overall average rate of 1 kPa/min. The resulting stress path is only approximately q = constant, since the vertically transmitted load is affected by the decreasing cell pressure due to the fixed connection between the top cap and the loading ram. Thus, q is slowly increased as the cell pressure decreases.

TEST RESULTS

Only three tests from the total of sixteen tests are presented in detail in this paper. Characteristics of each test are summarized in Table 1.

Typical sand behavior during tests

The typical course of a test is shown in Figures 2 and 3.

1. In the first stage (Fig. 3, from a to b) the isotropic loading of the sample produced only minor axial deformation (less than 0.3%) and negligible lateral deformation. The associated void ratio change was a contraction of about -0.01. This stage usually lasted for about 10 to 20 minutes before all changes had stabilized.

2. During the second stage (Fig. 3, stretch from b to c) the application of the deviatoric stress, i.e., dead load, produced only a small effect on the lateral deformation of the sample. Axial deformations of about 0.5% were recorded. The void ratio changes resulted in a contraction of less than -0.02. The dead load was applied in steps for approximately 5-10 minutes each to completion of the volume changes.
3. In the following third stage (Fig. 3, from c to d) a gradual cell pressure decrease produced almost negligible volume changes and the sand behaved as an essentially elastic material. This behavior continued up to a certain distinct stress level when the material started to contract and both the axial and lateral strains accelerated slightly. This is shown in p' - e plane as a deflection in the direction of the stress path (Fig. 3, point d). Shortly after, a new compressibility $\Delta e/\Delta p'$ was established. At this stage more pronounced time dependent deformation could be observed but all movements stabilized within several minutes.
4. Further decreases in the confining pressure resulted in discreet contractive response during which the axial strain 'jumped' by about 2% with an associated void ratio contraction of -0.05. These discreet jumps accelerated in terms of seconds (20s) and decelerated over a minute (Fig. 4). Usually 2 or 3 contractive "jumps" could be recorded during each test.

EXPLANATION OF OBSERVED PHENOMENON

All tests have shown that very loose essentially dry Ottawa sand when subjected to triaxial $q \cong$ constant stress path first behaves essentially elastically, then at a certain point the deformation response abruptly changes. The material contracts and exhibits a significant increase in compressibility. Thus the initial direction of a stress path in the $p' - e$ plane is notably deflected. After a short transition period a new and significantly higher compressibility is observed (Fig. 3). As the sample is further loaded two different straining patterns can be detected. The first pattern is associated with moderate slips at grain contacts which have both a stress dependent and time dependent component (Fig. 2). This response will be referred to as **structural contraction**. The second straining pattern is characterized by rapid contractive strains of primarily time dependent nature (Fig. 2 and Fig. 4). These strains accelerate rapidly within 10 to 20 seconds and then slowly decelerate over a minute. It is believed that this second straining pattern is peculiar to the collapse of very loose saturated sand and it will be further referred to as the **structural collapse**.

The structural collapse of very loose dry sand is a result of progressive destabilization of the grain structure. This process of structural destabilization commences as small strains/slips at grain contacts. The grains try to reorient to accommodate the developing instability. This accounts for the first straining pattern. Later the ability of the material to compensate is insufficient and the entire structure collapses, i.e., contracts. The contractive strains densify (harden) the dry sand which results in self-stabilization of the collapse and all movements eventually cease. The new state condition of the material is still far from the ultimate state condition and so the process can repeat itself under further loading. The hardening of saturated sand during a structural collapse is inhibited by the pore water and so pore pressures are generated. This in turn leads to material weakening and progressive propagation of collapse leading to full scale flow liquefaction can be established.

The mobilized friction angles at the points of observed structural collapse range from 20.4° to 30.1° which is below the ultimate mobilized friction angle (Figure 5, Table. 1). This observation is consistent with the observations made for collapse of a very loose saturated sand and supports the fact that geotechnical design cannot always rely on the ultimate mobilized friction angle as a guarantee of safety. The undrained collapse leading to flow liquefaction of a saturated sand, which appears to be equivalent to the structural collapse of a very loose dry sand, can occur at mobilized friction angles much below the ultimate mobilized friction angle.

The relationship between the mobilized friction angle ϕ'_m and the vertical displacement dH during presented $q \cong$ constant tests is shown in Fig. 6. This diagram shows that the behavior of the sand is consistent with the behavior of the same sand tested by Sasitharan et al. (1994) and also that the tests are mutually consistent. The mobilized friction angle approaches the ultimate value of about 30° which is expected for Ottawa sand.

CONCLUSIONS

The deflection of the original direction of the stress paths in p' - q - e space is shown as an increased compressibility. This breakpoint in material response and onset of structural contraction resulting in establishment of a new path direction in the p' - e plane appears to be conceptually similar to the behavior of a very loose saturated sand attaining the state boundary surface. Figure 3 and table 1 show that the mobilized friction angles at the observed breakpoints are significantly smaller than the ultimate friction angle for Ottawa sand.

The contractive character of very loose dry Ottawa sand collapse is conceptually consistent with the behavior of very loose saturated sand. In very loose saturated sand the structural collapse of the sand matrix is exhibited by pore pressure generation and, if the pore pressures cannot dissipate sufficiently rapidly, a consequent decrease in effective stress and significant loss of shear strength will occur. This can result in flow liquefaction. In the dry sand there is no incompressible pore medium, therefore the collapse reflects itself in contractive volume change. The collapse is confined by self stabilization due to hardening as a result of densification.

The pore pressure generation in very loose saturated sand has already been shown to be a consequence and not a cause of collapse (Eckersley 1990, Sasitharan et al. 1993). This observation is supported in this study by means of structural collapse of dry sand, which did not have any triggering pore pressure levels available but collapse still occurred.

Conclusions of the presented study can be summarized as follows:

1. Very loose essentially dry Ottawa sand during q ≅ constant loading attains a discreet state condition where the stress path in p' - q - e space is deflected and a new significantly higher compressibility associated with structural contraction is established. This change occurs at a mobilized friction angle considerably smaller than the ultimate mobilized friction angle (Fig. 3) and agrees with the state boundary surface concept established for saturated sand (Sasitharan et al. 1994).
2. Further loading of the element renders the structure increasingly unstable, which eventually results in discreet contractive time dependent "jumps", which have been referred to as structural collapses. It is our opinion that these structural collapses are responsible for the pore pressure generation during the collapse of very loose saturated sand resulting in flow liquefaction. The structural collapses occur at mobilized frictional angles significantly smaller than the ultimate mobilized friction angle.

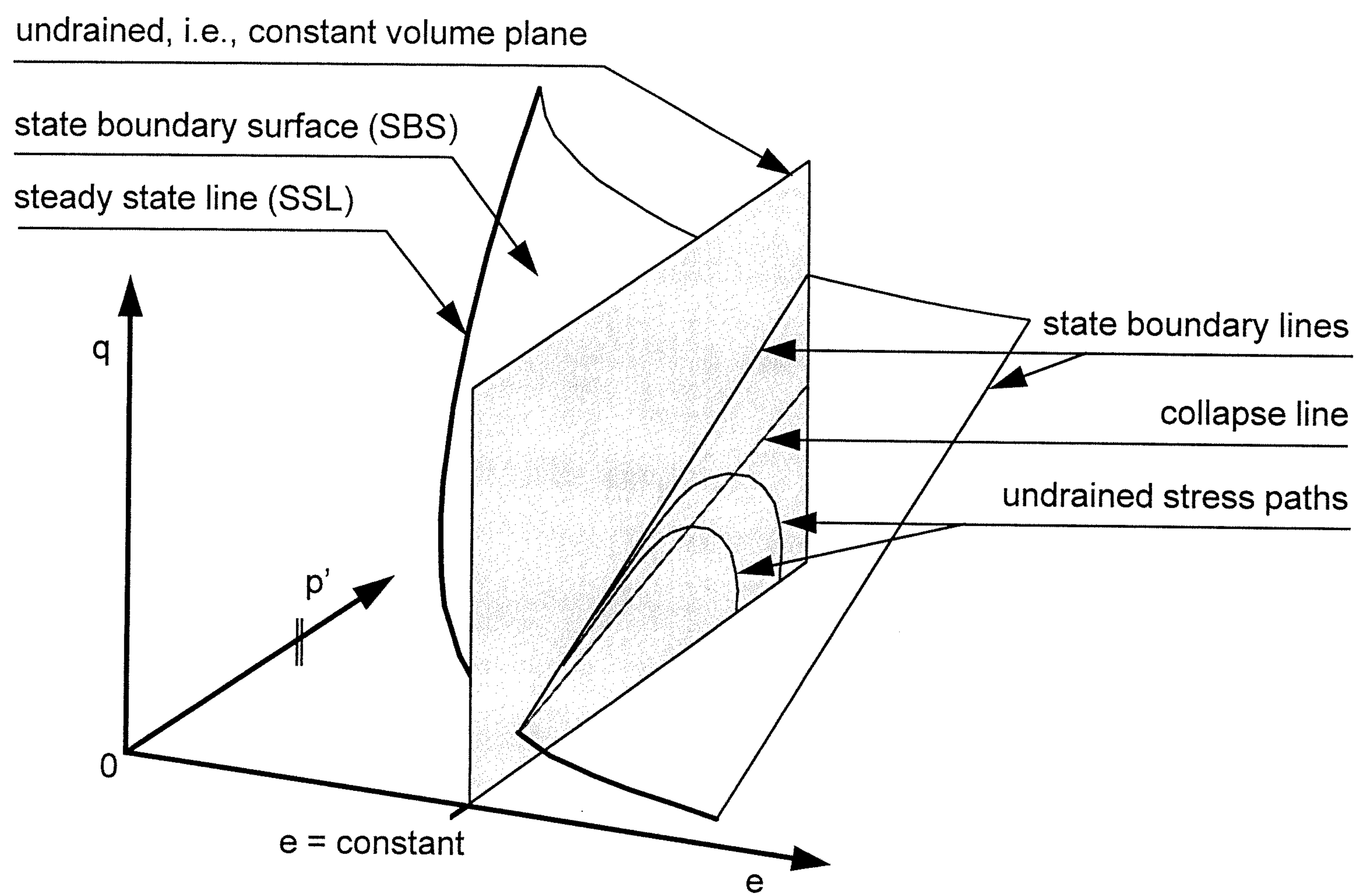
Acknowledgment

The authors would like to acknowledge the assistance of the University of Alberta technical staff, particularly S. Gamble and G. P. Cyre

Test identification	1	2	3
Initial parameters			
void ratio e	0.825	0.833	0.813
moisture content (%)	2.73	2.54	2.51
p' / q (kPa)	300 / 98	240 / 77	246 / 130
Onset of structural contraction			
p' / q (kPa)	182 / 108	123 / 81	191 / 131
ϕ'_m (°)	15.7	17.3	17.9
Characteristics of structural	σ_3 / e / ϕ'_m	σ_3 / e / ϕ'_m	σ_3 / e / ϕ'_m
collapse 1st	70 / 0.780 / 24.6	77 / 0.810 / 20.4	75 / 0.762 / 27.0
2nd	64 / 0.768 / 26.5	54 / 0.786 / 26.2	68 / 0.754 / 27.9
3rd	(units are kPa and degrees)	49 / 0.783 / 27.2	58 / 0.744 / 30.1

Table 1 Summary of test initial parameters, parameters at the onset of structural contraction and characteristics of structural collapses

LIST OF FIGURES



Note: collapse line is BELOW the state boundary line in the shown undrained plane; collapse surface pertaining to this collapse line is NOT shown on this figure

Fig. 1 Definition of the elements of state boundary surface

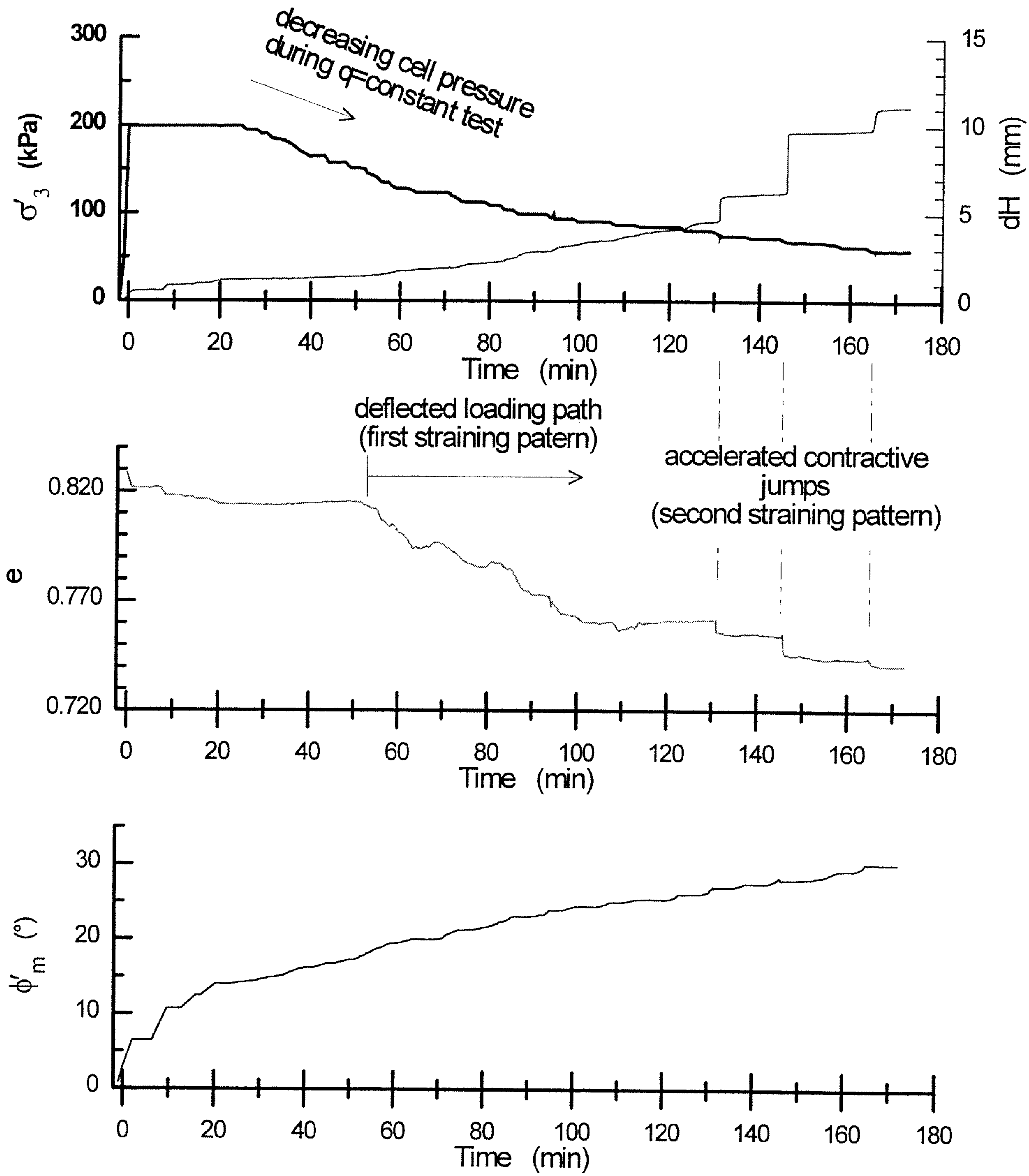


Fig. 2 Typical strain development during the q = constant test (test #3)

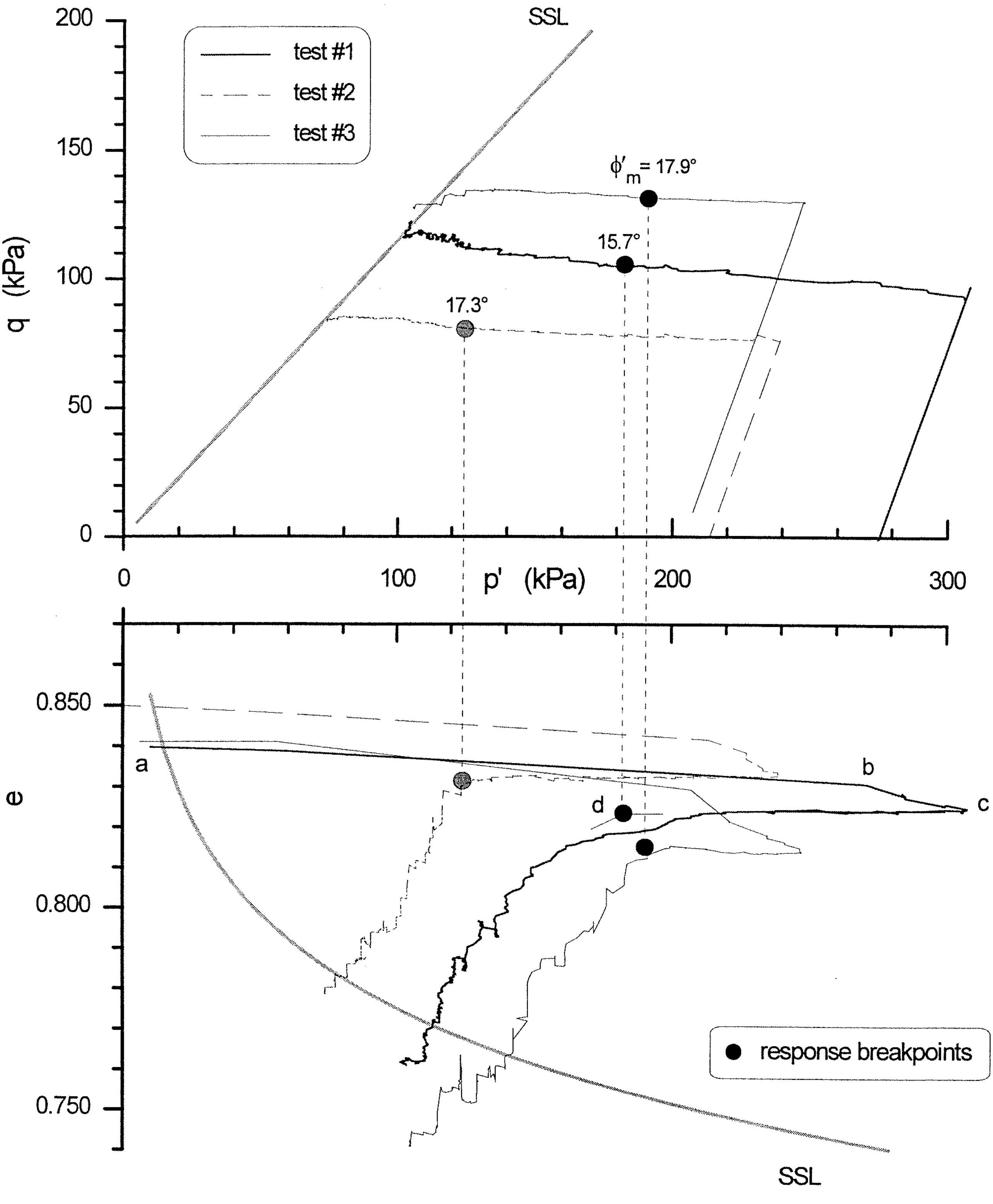


Fig. 3 Stress path and contraction during the q = constant tests

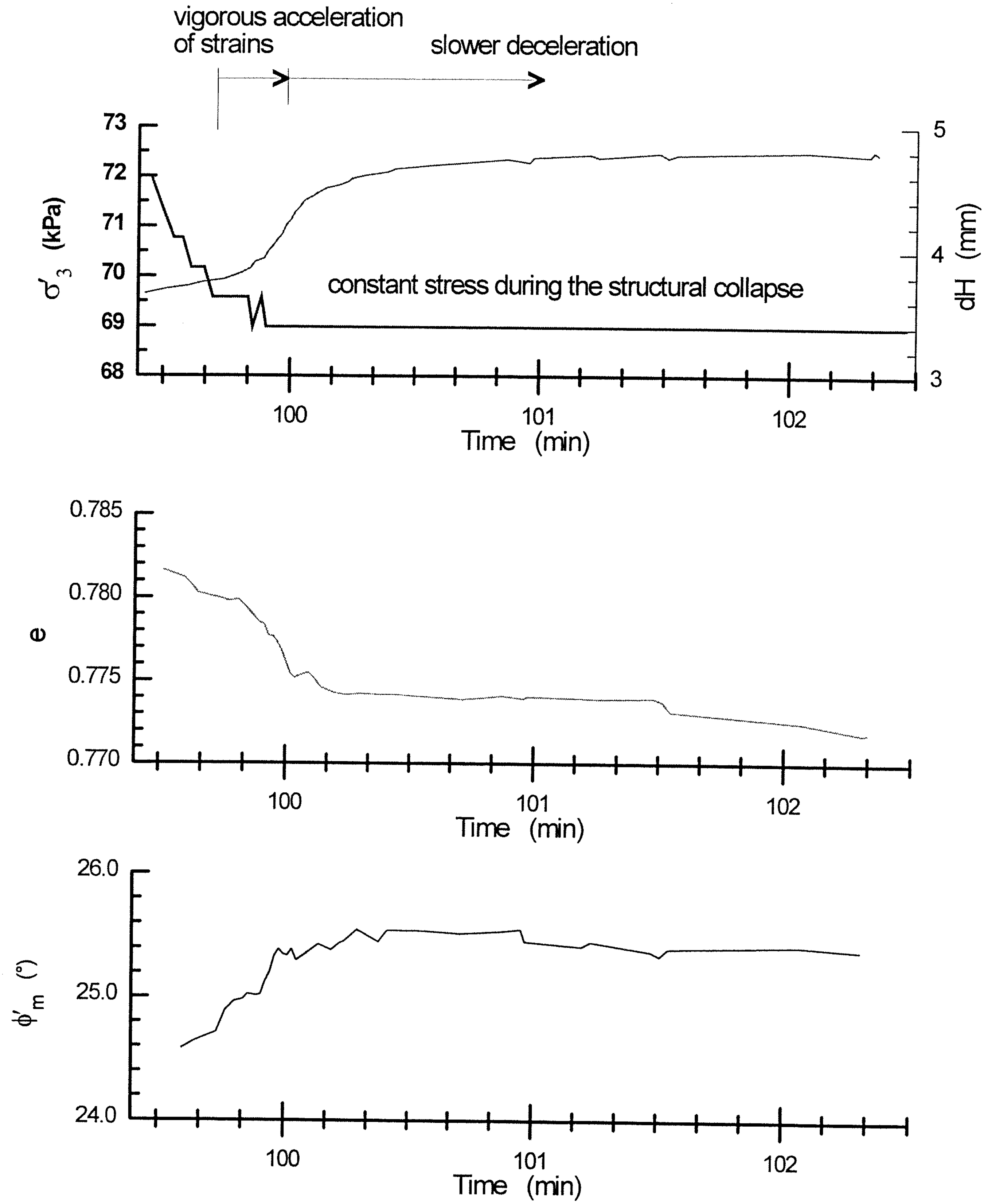


Fig. 4 Typical strain development during structural collapse (test #1, 1st jump)

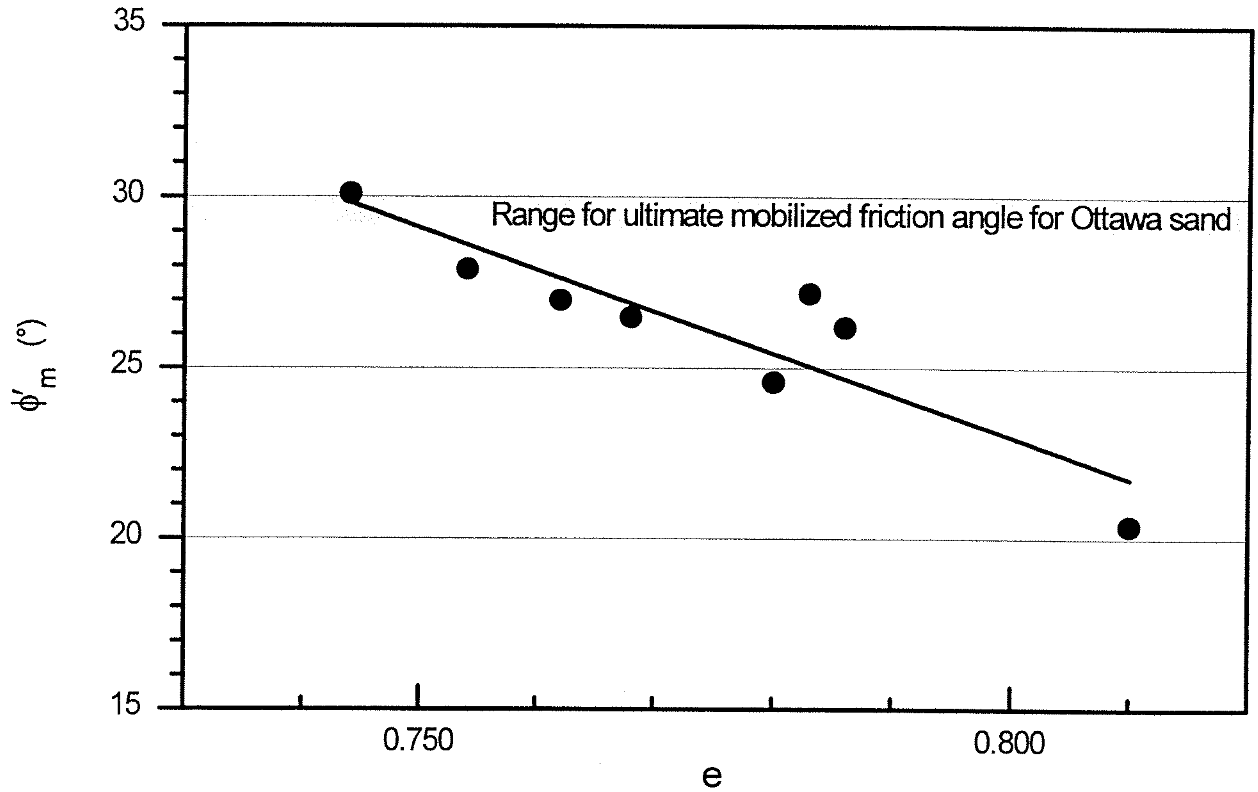


Fig. 5 Mobilized friction angle at points of observed structural collapse for all q = constant tests on very loose dry Ottawa sand

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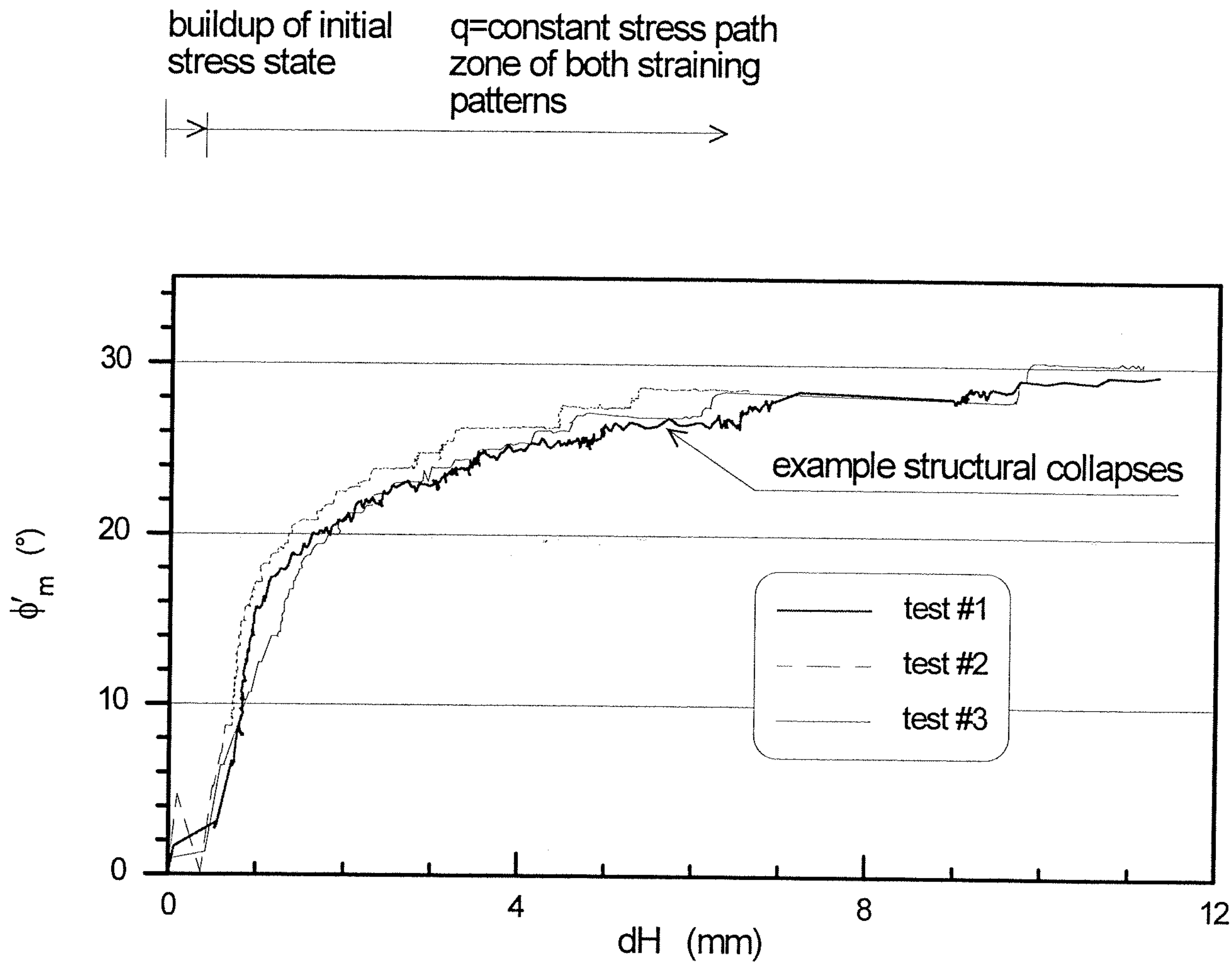

Fig. 6 Mobilized friction angle during q = constant tests on very loose dry Ottawa sand

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LIST OF SYMBOLS

e	void ratio
p'	effective mean principal stress (kPa); $= 1/3 \cdot (\sigma_1' + \sigma_2' + \sigma_3')$
q	deviatoric stress (kPa); $= (\sigma_1' - \sigma_2')$
ϕ'_m	effective mobilized friction angle (°)
σ_3	minor principal stress, cell pressure in triaxial cell (kPa)