THE EFFECT OF PARTICLE GRADATION ON THE UNDRAINED BEHAVIOUR OF SAND

by

JENNIFER M. FISHER

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Department of Civil Engineering

The University of British Columbia
1956 Main Mall
Vancouver, Canada
V6T 1Y3

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ABSTRACT

The effect of particle gradation on the undrained monotonic and cyclic loading behaviour is presented. Straight line gradations of Earls Creek sand with varying coefficients of uniformity and identical mineralogy and D_{50} were tested, using the triaxial test. Improved sample preparation techniques were used to ensure sample uniformity. The data indicates that, under monotonic loading, the relative shear-induced compressibilities due to a variation in the coefficient of uniformity are a function of the type of loading. Cyclic loading tests on isotropically consolidated samples showed that the effect of particle gradation depends on the relative density. At low relative densities, (less than about 45%), the well graded sand had greater cyclic strength than the uniform sand. At high relative densities, (greater than about 60%), this trend was reversed.

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NOTATION

B	Skempton's pore pressure parameter.
CSR	Critical effective stress ratio.
C_u	Coefficient of uniformity.
D_r	Relative density.
D_{rc}	Relative density after consolidation.
D_{ri}	Relative density prior to consolidation.
D_{50}	Mean particle diameter or effective grain size of soil sample; 50 % by dry weight of sample is smaller than this grain size.
e	Void ratio.
e_c	Void ratio after consolidation.
e_{min}, e_{max}	Minimum and maximum void ratios as determined.
N	Number of loading cycles.
p'	$1/2(\sigma_1' + \sigma_3')$.
PT	Phase transformation.
s	$1/2(\sigma_1' + \sigma_3')$.
t	$1/2(\sigma_1' - \sigma_3')$.
u	Porewater pressure.
Δu	Excess porewater pressure.
ϵ_a, ϵ_v	Axial and volumetric strain.
ϕ'	Angle of internal friction.
ϕ_{cv}	Constant volume friction angle.
σ_d	Deviator stress.
σ_{dcy}	Cyclic deviator stress.

σ_1', σ_3'	Major and minor effective principal stresses.
σ_{3c}'	Effective consolidation pressure in the triaxial test.
τ_{cy}	Cyclic shear stress = $\sigma_{dcy}/2$.
τ_{cy}/σ_{3c}'	Cyclic stress ratio.

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1. INTRODUCTION

During rapid shearing, the development of large deformations in saturated cohesionless soils may occur. These deformations may be the result of the loss of shear resistance or progressive stiffness degradation during cyclic loading, called liquefaction and cyclic mobility respectively. (Castro 1969 & 1975, Seed 1979). Rapid shearing may be the consequence of cyclic earthquake loading or monotonic increases in shear stress. The rapid nature of the loading limits pore pressure dissipation, hence the behaviour is considered undrained.

Liquefaction is a strain softening response, in which highly contractive (loose) sand loses a large percentage of its shear resistance and deforms continuously in a state of constant normal effective and shear stresses, constant volume and constant velocity termed steady state. (Casagrande 1976, Castro 1969 & 1975, Seed 1979, Poulos 1981). Equilibrium is restored only after enormous displacements or settlement. (National Research Council 1985).

With cyclic mobility, the deformation is accumulated when cyclic loading momentarily reduces the effective stress to zero at the instant when the cyclic shear stress passes through zero. Following this, deformation accumulates with each cycle of loading. Deformations are limited and the earth mass remains stable following shaking without great changes in geometry.

Liquefaction is associated with cyclic or static loading whereas cyclic mobility is associated with cyclic loading only. (Vaid & Chern 1985, Castro 1975, Casagrande 1976, Seed 1979). The concern with liquefaction is stability, while with cyclic mobility, it is the accumulation of undesirable deformation.

Many investigations have been carried out to study the effect of various parameters on the undrained monotonic and cyclic response of saturated sands. These include parameters such as void ratio, effective confining stress, static shear stress, particle angularity, stress path, and prestrain history. (Castro et al 1982, Chern 1981, Chung 1985, Ishihara et al 1975, Seed 1979, Tumi 1983, Vaid & Chern 1985). However no investigation has been done which isolates and clearly defines the effect of coefficient of uniformity on the undrained monotonic and cyclic loading behaviour of saturated sand. The clarification of its effect will improve the understanding of the undrained behaviour of saturated sands. Also, knowledge of those soil characteristics which preclude liquefaction is important in identifying in-situ conditions where liquefaction may not be a concern. Substantial benefits will be derived from a better understanding of the limits on gradation outside which dynamic loss of soil strength and liquefaction instability need not be considered. (National Research Council 1985).

In this study, the effect of the coefficient of uniformity on the undrained behaviour of sand is investigated using undrained monotonic and cyclic triaxial tests, on isotropically consolidated samples. Three medium sands with straight-line gradations, identical mineralogy and identical D_{50} are tested. Although idealized straight-line gradations are not found in nature, their use, herein, is justified for the same reason as the use of remolded clay in fundamental studies of clay behaviour. Straight-line gradations, combined with identical mineralogy and constant D_{50} ensure isolation of the effect of gradation for a given medium sand. Constant D_{50} ensures that membrane penetration is not a cause of variation between the results of the different gradations. (Frydman et al 1973). Since well graded sand tends to segregate during conventional water pluviation, improved sample preparation techniques were developed to ensure sample uniformity. Testing of uniform specimens is a prerequisite for fundamental studies of material behaviour.

2. REVIEW OF PREVIOUS INVESTIGATIONS

2.1. GENERAL ASPECTS OF THE UNDRAINED BEHAVIOUR OF SANDS

The undrained response of saturated sand is traditionly investigated separately under monotonic and cyclic loading conditions. The desire to study each of these loading conditions is stimulated from quite different concerns. Flow slides, caused by undrained failure, have generated the interest in monotonic loading. The concern with cyclic loading of sand is mainly with the accumulation of undesirable deformation during earthquake loading.

2.2. MONOTONIC LOADING BEHAVIOUR

The three types of responses of an isotropically consolidated saturated sand, subject to undrained triaxial compression under moderate confining pressures, are shown in Figure 2.1. The stress-strain curves 1 through 3 represent increasing relative density. Similar behaviour manifests under initial anisotropic consolidation.

Types 1 and 2 represent the strain softening or contractive response which is a behaviour associated with a loss of shear resistance after a peak. The strain softening response

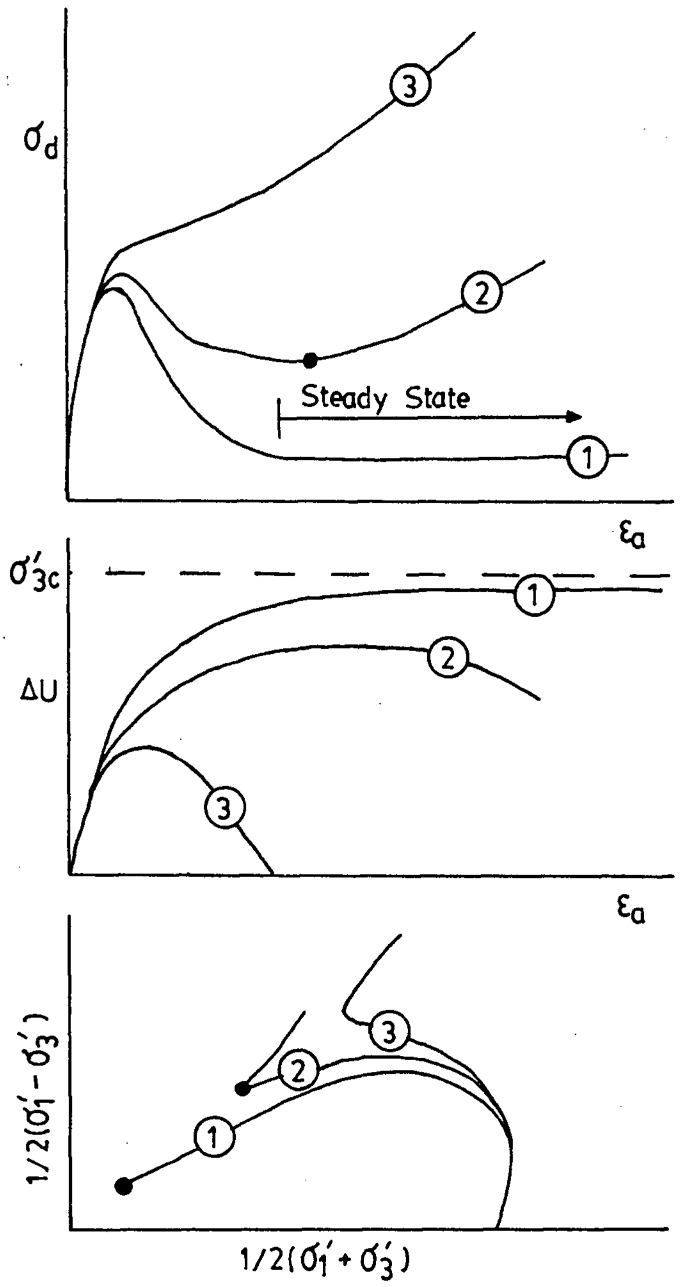


Figure 2.1: Characteristic behaviour of saturated sand under monotonic loading. (Adapted from Vaid & Chern 1985).

is initiated after attainment of a peak deviator stress.

Type 1 response is liquefaction as defined by Castro (1969), Casagrande (1976), and Seed (1979). After an initial peak strength, continuous deformation occurs at constant confining and shear stress and constant volume, in a state termed steady state. In this steady state, the sand mass flows as a frictional liquid and hence the association of the term 'flow failure'. (National Research Council 1985).

Limited liquefaction is represented by the type 2 response, in which temporary strain softening, similar to that of liquefaction, is initiated. This transitory loss of shear resistance is regained with further straining. The effective stress ratio (σ_1'/σ_3') at which the strain softening response is initiated is defined as the Critical Stress Ratio (CSR) (Vaid & Chern 1983). The CSR has been shown by a wide body of experimental data to be a constant for a given sand irrespective of the void ratio and stress state prior to the commencement of undrained deformation. (Chern 1985).

The arrest of the strain softening behaviour in limited liquefaction is at the minimum deviator stress. This minimum deviator stress is higher than the deviator stress at steady state in the liquefaction case. At the minimum deviator stress in limited liquefaction, dilation occurs, the pore pressure begins to decrease, and the effective stress path takes a sudden turnaround. The state at this point of change has been designated as the Phase Transformation by Ishihara et al (1975). The effective stress ratio at phase transformation is a material constant for a specific sand. (Ishihara et al 1975, Vaid & Chern 1985). After the phase transformation, the effective stress path approaches the undrained failure envelope with further straining. At high deviatoric stresses, the sample may eventually reach steady state. (Castro 1982).

With liquefaction, the effective stress path terminates at steady state. Failure occurs as a result of large deformations prior to and at steady state, without the stress path reaching the undrained failure envelope. The friction angle at phase transformation equals the friction angle at steady state. (Vaid & Chern 1985).

Type 3 response depicts the strain hardening or dilative response. No loss of shear resistance is experienced. The phase transformation state exists for dilative sand also, and corresponds to the point at which dilation commences and the pore pressure starts to drop. The phase transformation state for dilative and contractive behaviour is at the same effective stress ratio. (Vaid & Chern 1985). At the phase transformation, in effective stress space, a turnaround occurs in the stress path although it may not be discernible for highly dilative states.

Under monotonic loading, the steady state line has been proposed by Castro et al (1977 & 1982) as a boundary in two dimensional (void ratio, e_c, versus effective confining pressure, σ_3', at steady state) space between initial states prior to undrained loading that are liquefiable and those that are not. States substantially to the right of the steady state line lead to liquefaction, while those below are nonliquefiable.

Sladen, D'Hollander and Krahn (1958) propose the existence of a collapse surface in p'-q-e space. They combine the concepts of steady state with the critical state concept put forward by Roscoe et al (1958). For a fixed e_c, the peak points on the stress paths in p'-q space form a straight line that passes through the steady state point, not through the origin as proposed by Vaid & Chern (1985). A possible reason for this difference is that Sladen's model was based on a small number of test results. Sladen also prepared his samples using moist tamping. This technique produces samples with nonhomogenieties. (Castro 1969, Castro et al 1982).

Pluviation, the technique used by Vaid & Chern, forms more homogeneous samples, therefore their results are more reliable. The position of this collapse line is shown by Sladen et al to shift with changes in void ratio, while its slope remains constant. Drawn in p'-q-e space, these lines combine to form a surface, termed a 'collapse surface'. The behaviour of a sand is stated in terms of its stress state relative to the collapse surface. For this collapse surface concept to be used, contactive behaviour is necessary for the definition of a peak in the stress path, and of the steady state line.

2.3. CYCLIC LOADING BEHAVIOUR

Sand liquefaction was reported as long ago as 1783 (Hobbs 1907), but it wasn't until it caused severe damage in the form of building settlement and tilting and slope failures during earthquakes in Niigata, Japan and Alaska in 1964 that the phenomenon was begun to be investigated.

Initial research dealt purely with strain development during cyclic loading. This was attributed to the development of states of zero effective stress during some stages of loading. (Seed & Lee 1966). Later it was recognised that there were two distinctly different mechanisms of strain development : liquefaction, and cyclic mobility. (Castro 1969).

True liquefaction, due to cyclic loading, is illustrated in Figure 2.2. At some stage during cyclic loading, liquefaction is triggered and the sample undergoes unlimited deformation. (Castro 1969). This occurs in a manner similar to that observed under monotonic loading. (See Figure 2.1).

Limited liquefaction, shown in Figure 2.3, occurs with cyclic loading as well as with monotonic loading also. (cf Figure 2.1). (Vaid & Chern 1985, Chern 1985). With limited liquefaction, the sand develops a strain softening response in a manner similar to liquefaction but over a limited strain range. The CSR, the phase transformation state associated with limited liquefaction and the steady state associated with liquefaction are the same for monotonic and cyclic loading for a given sand. (Vaid & Chern 1985).

Cyclic mobility due to cyclic loading is shown in Figure 2.4. Cyclic mobility is caused by the progressive build up of pore pressure with cycles of loading. If the cyclic shear stresses are higher than the static shear stresses, ie. a state of stress reversal occurs, and there are sufficient loading cycles, the cyclic loading can momentarily reduce

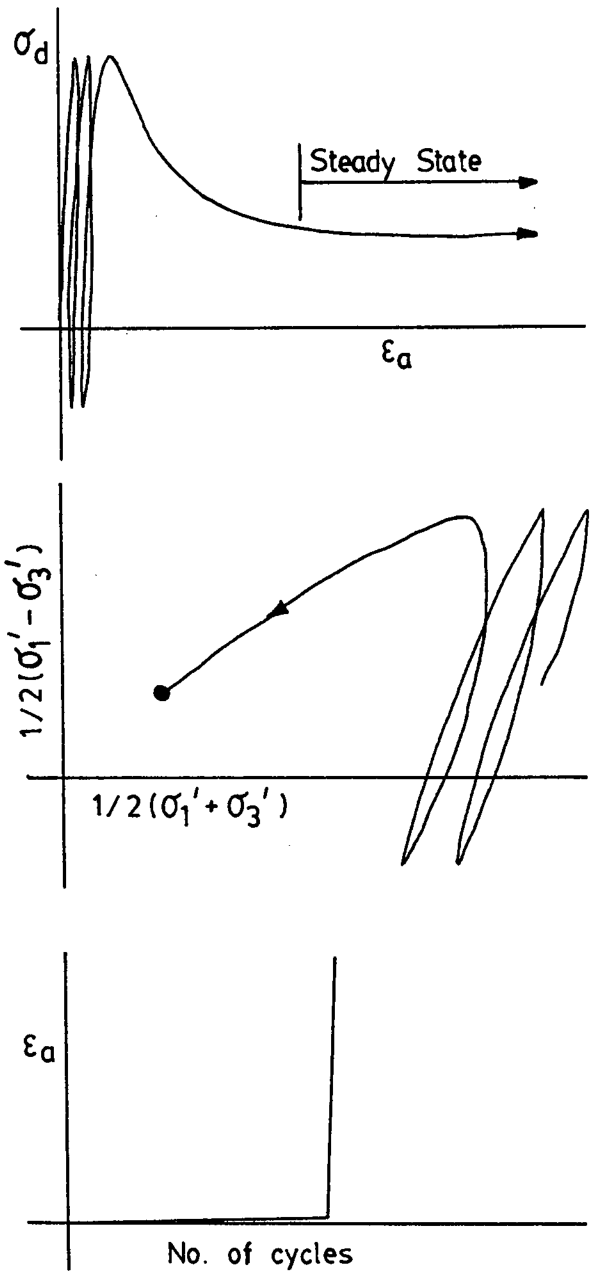


Figure 2.2: Liquefaction due to cyclic loading.
(After Vaid & Chern 1985).

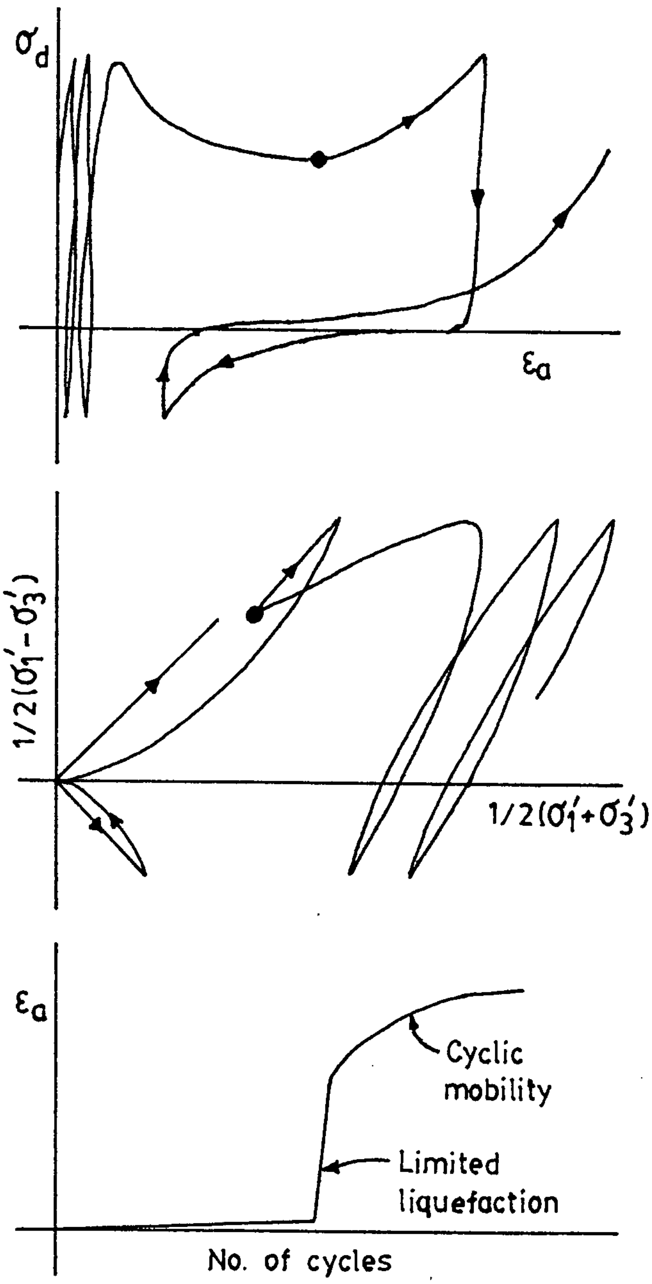


Figure 2.3: Limited liquefaction due to cyclic loading. (After Vaid & Chern 1985).

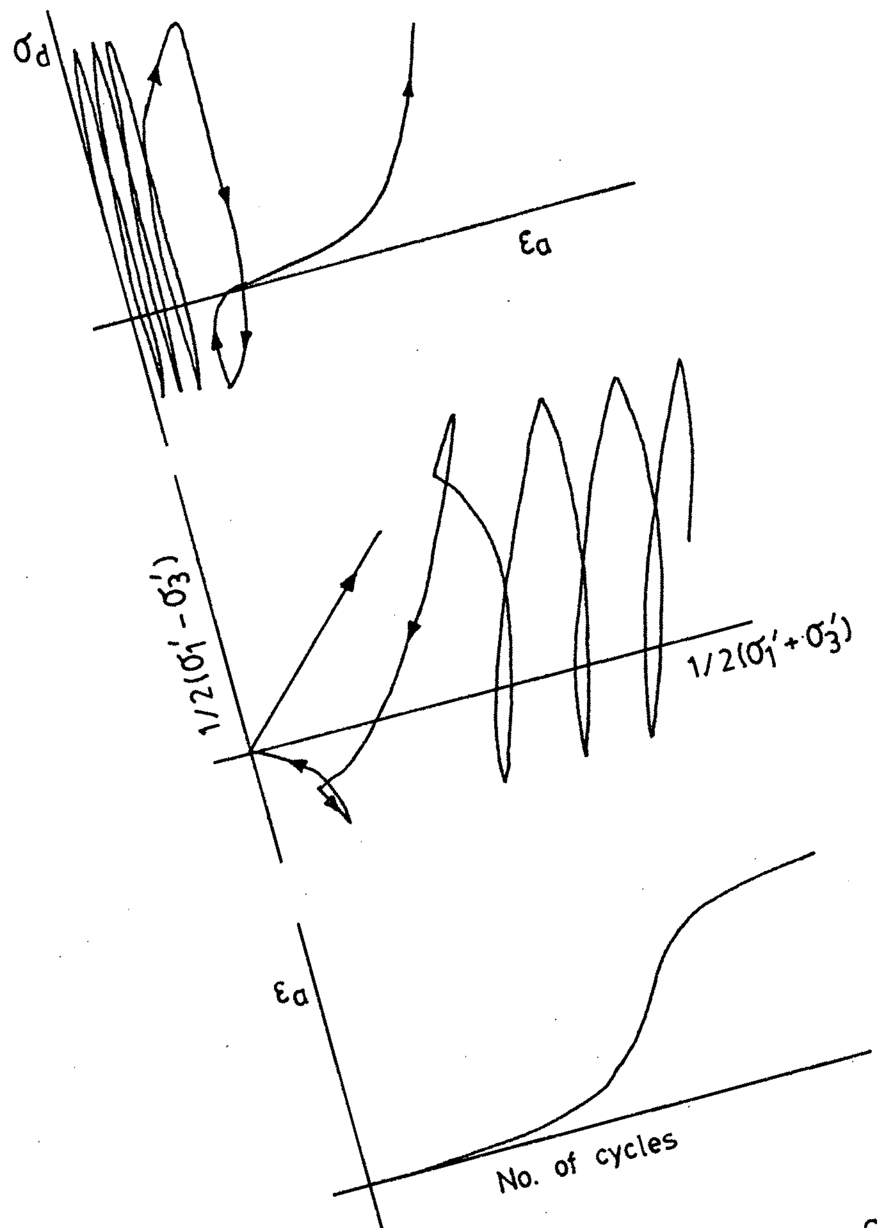


Figure 2.4: Cyclic Mobility due to cyclic loading.
(After Vaid & Chern 1985).

the effective stresses to zero when the cyclic shear stress passes through zero. Deformation then accumulates with each cycle of loading, and attains a finite magnitude with the completion of loading.

If during cyclic loading a specified level of deformation occurs, it could be due to limited liquefaction, cyclic mobility or a combination of the two. It should be noted that deformation due to limited liquefaction always occurs before the realization of momentary states of zero effective stress. (Vaid & Chern 1985). If liquefaction or limited liquefaction occurs, the concern is with stability because the associated deformations will be very large and unacceptable. If cyclic mobility occurs, the concern is with the accumulation of undesirable deformation.

The resistance to cyclic loading is defined as the cyclic stress ratio required to cause contractive deformation or to develop a specified amount of axial strain due to cyclic mobility in a fixed number of cycles. (Castro et al 1982, Vaid & Chern 1985). Development of liquefaction is always associated with the accumulation of a large strain.

2.4. RELATIONSHIP BETWEEN MONOTONIC AND CYCLIC LOADING

There is a close link between the type of monotonic response and the mechanism by which strain development occurs under cyclic loading. (Castro 1969, Castro et al 1982, Vaid & Chern 1985). For a sand to develop contractive behaviour under cyclic loading (liquefaction or limited liquefaction), there are 3 requirements:

1. The initial state of the sand must lie in the region where contractive deformation would occur under monotonic loading, ie. well above the steady state line.
2. The maximum shear stress must exceed the shear strength at phase transformation for limited liquefaction or at steady state for liquefaction.
3. There must be a sufficient number of loading cycles to move the effective stress state of the sand to the CSR state. (Vaid & Chern 1985).

It has been shown by Vaid & Chern (1985) that the contractive response leading to liquefaction or limited liquefaction is initiated at a constant value of the critical stress ratio, CSR, which is independent of the type of loading (monotonic or cyclic). It is also independent of the initial state of the sand (void ratio, effective confining stress, and shear stress) and of the amplitude of cyclic stress.

The mobilised friction angle at phase transformation, ϕ_{PT}', for contractive or dilative behaviour is equivalent to the constant volume friction angle, ϕ_{cv}, at steady state. The mobilized friction angles at phase transformation and steady state are independent of confining pressure, initial packing density, particle size and particle shape. They are dependent only on the mineral constituency of the material and are consequently unique for a granular material. (Negussey et al 1986, Wijewickreme 1986).

2.5. THE EFFECT OF PARTICLE GRADATION

Most research on the undrained response of sands to cyclic and monotonic loading has been done on uniform and clean sands. Uniform clean sands are rarely encountered in natural soil deposits and it is now known that liquefaction can occur in a variety of situations with soils of different properties. (National Research Council 1985). Therefore, it is necessary to determine the effect of other factors, such as particle gradation, on the response of sand.

Lee & Fitton (1969) did a study on the effect of grain size, grain size distribution, and grain shape on the strength of soils under simulated earthquake loading conditions. They performed cyclic triaxial tests on an alluvial sand and gravel deposit from El Monte, California. They concluded that there was no significant difference in strength between the well-graded and the uniformly graded sand. They ensured that the mineral types were the same for all size ranges, although they did not isolate the effect of coefficient of uniformity. The comparison of samples for the effect of grain size distribution was done with samples of varying mean diameter, (D_{50}).

A report submitted to the National Science Foundation, Washington DC, by Geotechnical Engineers Inc. (1982) looked at the effect of particle gradation on the steady state line. They concluded that relatively small differences in grain size distribution significantly affected the position, but not the shape and slope, of the steady state line. They used five gradations of Banding sand which had varying D_{50}, so the effect of coefficient of uniformity was not isolated.

Chang, Yeh and Kaufman (1982) studied the effect of gradation and silt content on liquefaction potential of sand. They did undrained cyclic triaxial tests on a Denver sand. They concluded that for coarse sand, D_{50} greater than 0.37 mm, the resistance to liquefaction decreases with an increase in the coefficient of uniformity. For fine sands, D_{50} smaller than 0.23 mm, the resistance increases with the coefficient of uniformity. The effect of the coefficient of uniformity vanishes at C_u greater than about 8. The sample preparation technique used was moist tamping, with saturation under back pressure. Moist tamping, however, gives rise to samples with densities that are considerably more non-uniform than other methods such as air and water pluviation. (Castro 1969, Castro et al 1982).

Wong et al (1974) performed cyclic triaxial tests to investigate the behaviour of "gravelly soils". They tested soils of constant D_{50} at a relative density of 60% and found that the well graded gravelly material required a smaller cyclic deviator stress than the uniform material to develop 2.5 % axial strain in 10 cycles. They proposed that this result could be due to membrane compliance and the fact that the well graded material has some tendency to densify as the finer particles move into the spaces between the larger particles. They compared samples at a constant D_{50}, however the mineralogy varied through the grain sizes. Thus, some of the observed difference may be due to this aspect. Their tests were also restricted to one relative density.

The National Research Council Report 'Liquefaction of Soils during Earthquakes', Nov. 1985, identified the effect of grain size distribution on dynamic loss of soil strength and liquefaction as one of the areas which required research. It was this report which initiated this study.

The literature review, herein, suggests that no fundamental study has been performed in which the effect of the coefficient of uniformity has been isolated from other factors which influence the undrained behaviour of sand. In this study, sands of varying C_u, but identical mineralogy, straight-line gradations and identical D_{50} were tested such that the effect of gradation on the undrained behaviour could be identified.

3. EXPERIMENTATION

3.1. TESTING PROGRAM

In order to determine the effect of particle gradation, both monotonic and cyclic loading tests were performed on the 3 medium sands of varying coefficients of uniformity. All tests were undrained and were performed on isotropically consolidated samples.

Strain controlled monotonic loading tests were conducted to determine the variation due to gradation of the undrained response of the sand to monotonic load. The monotonic loading tests also give an indication of the mechanism of strain response during cyclic loading. These tests were performed on the 3 gradations at a constant relative density of 38.5 ± 1.5 %. This density was the minimum density obtainable for Gradation 3 under an initial confining pressure of 500 kPa. The lowest possible density states were selected in order to provide the most favourable conditions for the occurence of contractive deformation. Tests in the compression mode were carried out for initial confining pressures of 50, 200 and 500 kPa. Tests in the extension mode were performed for an initial confining pressure of 200 kPa only.

Stress controlled cyclic loading tests were performed to determine the resistance curves to cyclic loading for the 3 gradations, so that the behaviour could be compared. All cyclic tests were carried out for an initial confining pressure of 200 kPa. The cyclic stress ratio, $(\sigma_{dcy}/2\sigma_{3c}')$, was varied between 0.123 and 0.23. The relative density was varied between 22 and 73 % depending on the cyclic stress ratio and the gradation of the sand being tested.

3.2. TESTING APPARATUS

A schematic layout of the testing apparatus for the stress controlled cyclic loading tests is given in Figure 3.1.

The cyclic axial load was applied using a double-acting air piston. Initially the pressures in the two chambers of the piston are equal and the loading ram is at rest. The pressure in the bottom chamber of the piston is controlled by a pressure regulator. Volume boosters are connected to the top and bottom chambers of the piston to ensure that there is no degredation of the load pulse when large deformations occur. The cyclic load is applied through the top chamber of the piston. It is applied by an electro-pneumatic transducer which is driven by the function generator. The maximum output of the electo-pneumatic

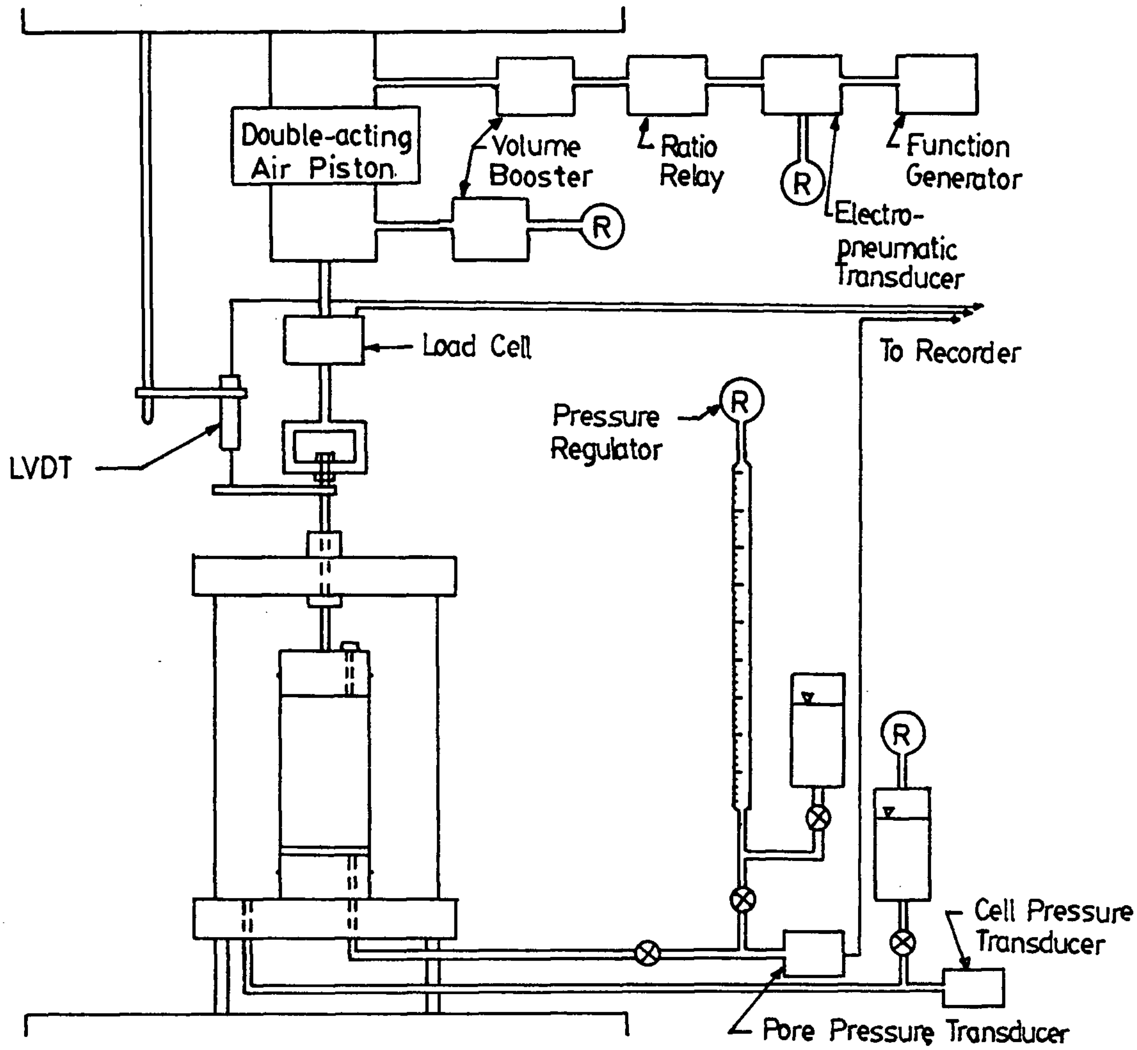


Figure 3.1: Schematic layout of testing apparatus.

transducer is 103 kPa, so the ratio relay was included to amplify the pressure provided to the piston.

The monotonic loading tests were performed using a layout similar to the cyclic loading system, in which the air piston was replaced by a strain drive. An adjustable speed DC motor was used to provide the strain controlled loading required for the tests.

The load, deformation, and pore pressure in the sample were recorded during the test.

3.3. MATERIAL TESTED

The sand used in this study is a natural river deposit from Earls Creek, British Columbia, obtained from the Vancouver Municipal yards having been transported from Earls Creek by barge. The sand is used locally for backfilling trenches and in the production of asphalt mix. Earls Creek sand is sub-angular, with particle sizes ranging from 0.06 mm to 5 mm.

The sand fraction which passed through the #8 sieve was divided into 12 grain size ranges by sieves ranging from #10 to #200. These grain sizes were then combined to form 3 linear gradations with a constant D_{50} of 0.42 mm. The gradations 1, 2, and 3 have coefficients of uniformity of 1.5, 3, and 6 respectively. The linear grain size distribution curves of these gradations are shown in Figure 3.2, along with that of the original Earls Creek sand.

The mineral composition of the sand is approximately 50 % quartz and 30 % feldspar with the remaining being hornblende, clinopyroxene, biotite mica, and sphene. The composition is uniform over the entire range of grain sizes which allows for the isolation of the effect of the coefficient of uniformity. Maintaining a constant D_{50} was also chosen to fulfill this requirement. The membrane penetration into the voids of a specimen of granular soil, due to application of a particular ambient pressure, is a function of the D_{50} of the soil, and not its gradation. (Frydman et al,1973). Consequently, any differences in behaviour between the 3 gradations is not caused by a variation in the membrane penetration.

The minimum and maximum void ratios, coefficients of uniformity, and particle size range for the 3 gradations are given in Table 3.1. The minimum and maximum void ratios, (e_{min} and e_{max}), were obtained in accordance with the standard test method, ASTM D2049. There is a large variation

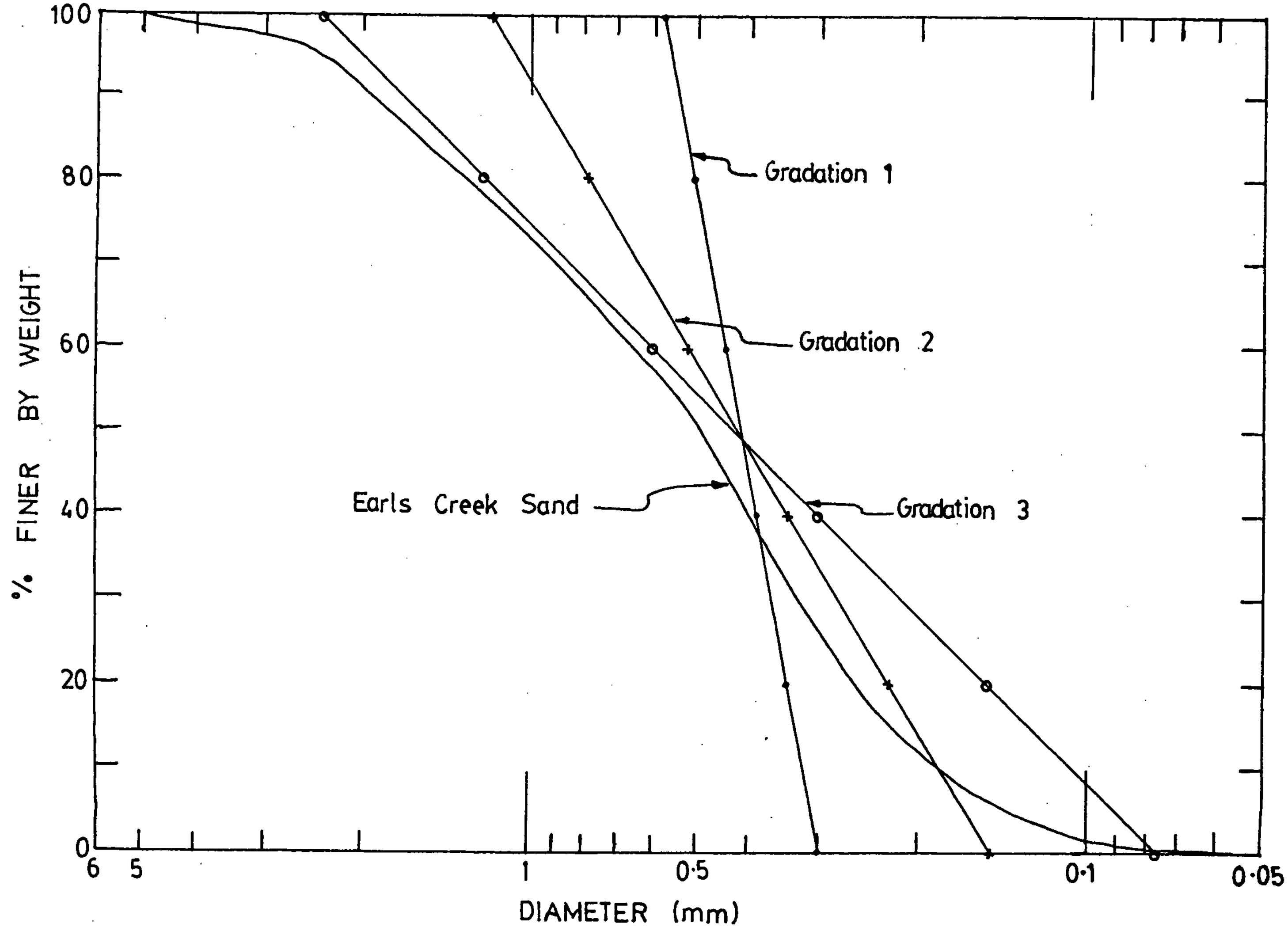


Figure 3.2: Grain Size Distribution Curves.

Gradation	Cu	e_{max}	e_{min}	Particle Size Range (mm)
1	1.5	0.94	0.63	0.3-0.59
2	3	0.77	0.51	0.15-1.2
3	6	0.61	0.37	0.074-2.4

Table 3.1: Material properties.

in e_{max} and e_{min} between the gradations, however, there is not much variation in $(e_{max}-e_{min})$.

The specific gravity was obtained using the method recommended by Lambe (1951) and is constant at 2.72 for the 3 gradations.

The hydrostatic consolidation characteristics of the 3 gradations, for a relative density of 38.5% at 500 kPa confining pressure, are shown in Figure 3.3. The consolidation characteristics are given in terms of the relationship between volumetric strain and mean normal stress during consolidation. The well graded sand is more compressible than the uniform sand, shown by the well graded sand developing higher volumetric strains than the uniform sand at constant mean normal stress.

3.4. SAMPLE PREPARATION AND TESTING TECHNIQUES

Sample homogeneity, uniformity of density and saturation were the prime requirements for sample reconstitution. It was found that pluviation through air or water caused segregation, therefore a new method of sample preparation, called 'the slurry method', was developed by Ralph Keurbis (1987). This method resulted in homogeneous and uniform

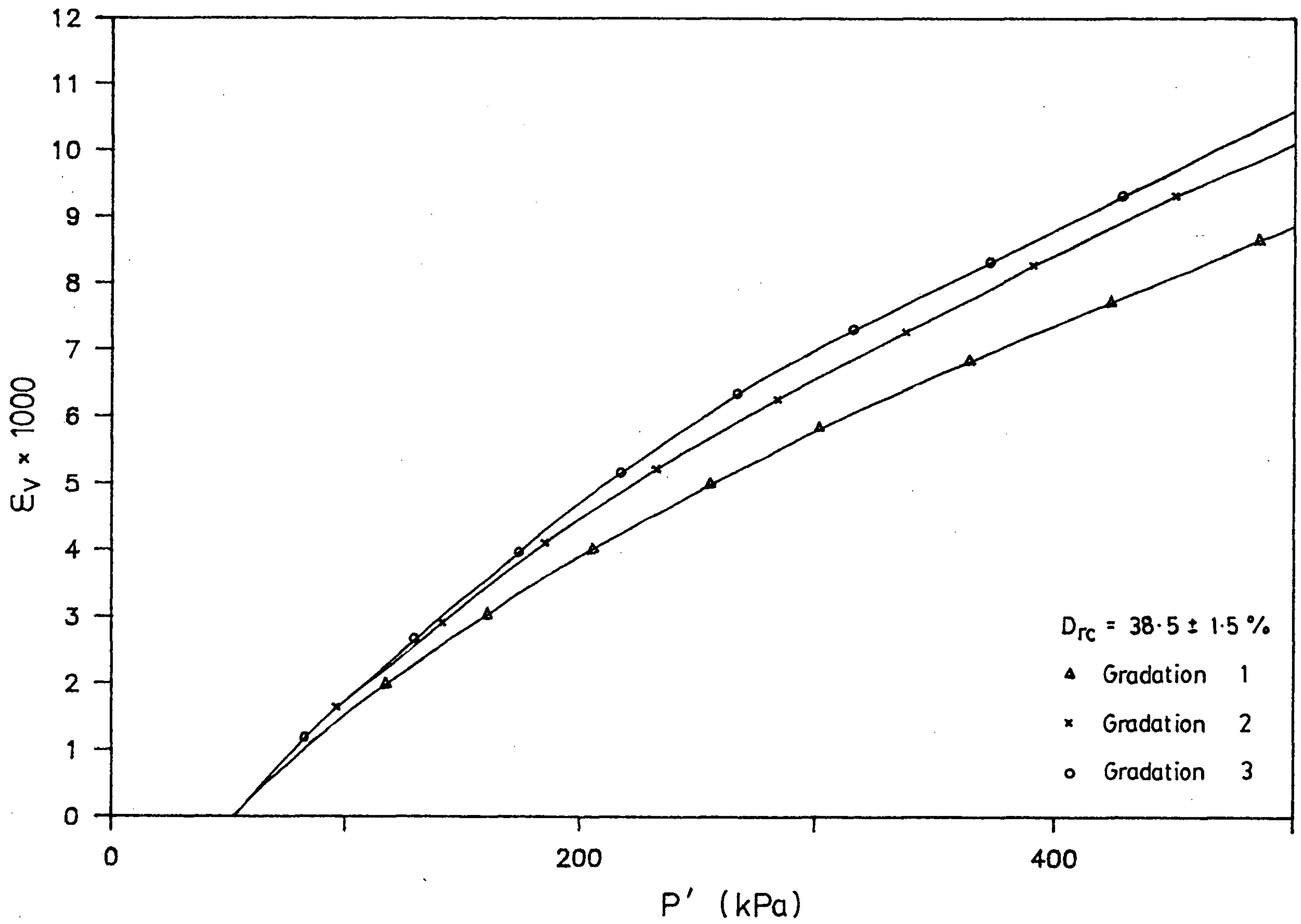


Figure 3.3: Relationship between volumetric strain and mean normal stress during consolidation.

specimens. Uniformity was verified by an analysis of the grain size distribution and void ratio throughout the sample. A sample was formed using gel mixed with the porewater. When the sample had solidified, it was cut into 4 sections and the void ratio and grain size distribution were calculated for each section. The variation between the void ratios of the slices was 1.5 %, while for the grain size distributions, the variation in the percent finer by weight was 2 %. (Keurbis 1987).

The samples had a diameter of 637 mm and an average height of 123 mm.

Initially, the sand and the porous disks were boiled for a period of 10 minutes to insure saturation. (see Figure 3.4a). The sand was then transferred by water pluviation into deaired water in a cylinder which was plugged at one end. (see Figure 3.4b).

The cylinder has an outside diameter of 60 mm and thus can fit inside the sample former. The length of the cylinder was such that there was sufficient water present to allow mixing of the sand-water slurry, but not too much such that segregation did not occur on inversion.

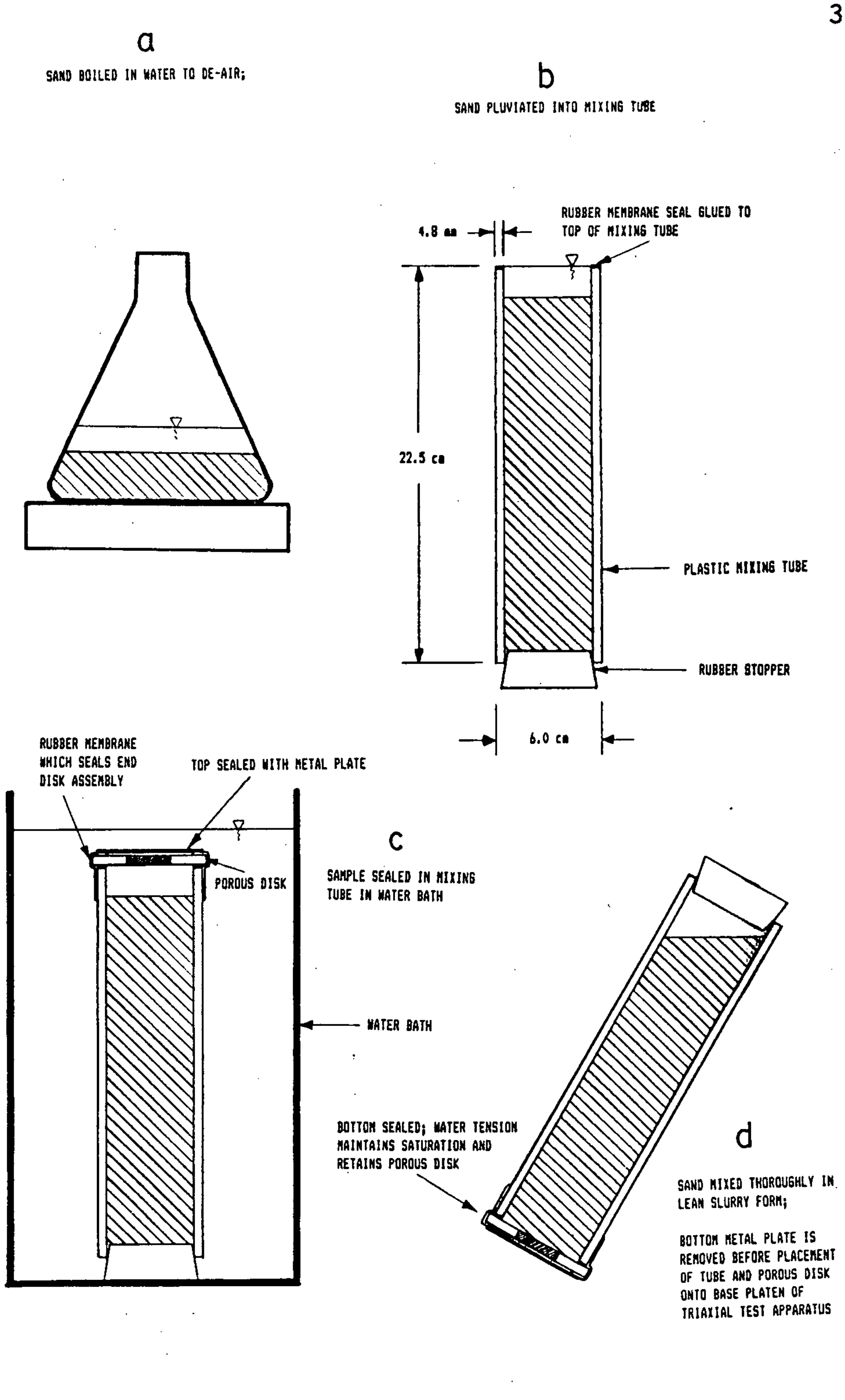
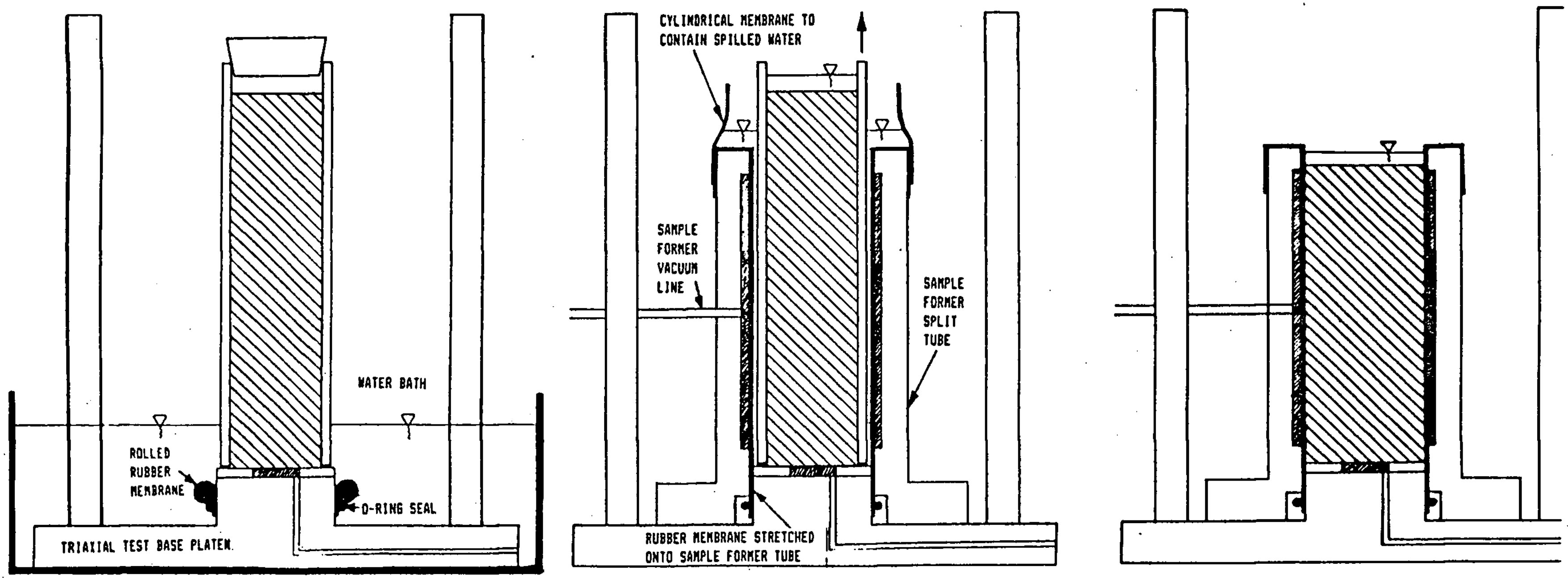


Figure 3.4A: Sample preparation by 'The Slurry Method'. (Adapted from Keurbis 1987).



e

MIXING TUBE PLACED UPON TRIAXIAL TEST APPARATUS BASE PLATEN IN WATER BATH; RUBBER MEMBRANE IS ROLLED UP SIDES OF MIXING TUBE

f

SAMPLE FORMER TUBE ASSEMBLED AROUND MIXING TUBE; SAMPLE MEMBRANE STRETCHED OVER SAMPLE FORMER TUBE; APPLICATION OF VACUUM TO FORMER TUBE EXPANDS MEMBRANE AND DRAWS IN RESERVOIR WATER FROM ABOVE, MAINTAINING SATURATION

g

MIXING TUBE WITHDRAWN LEAVING LOOSE SATURATED UNIFORMLY MIXED SAMPLE IN FORMER TUBE; TOP OF SAMPLE CAREFULLY FLATTENED

Figure 3.4B: Sample preparation by 'The Slurry Method'. (Adapted from Keurbis 1987).

The sand-filled cylinder was then immersed in a water bath. The porous base disk, 637 mm in diameter and 4.7 mm thick, was transferred under water to the top of the cylinder and held in place by a section of membrane. It was insured that there was no air within the cylinder or trapped between the membrane, stone, and cylinder such that water tension would hold the stone in place when the cylinder was inverted. There was a rubber seal on the end of the cylinder so a good seal between the porous disk and the cylinder was maintained. (see Figure 3.4c).

The cylinder was removed from the water bath. An aluminium disk was then placed on top of the porous disk to prevent its desaturation during mixing. The sample was then mixed thoroughly. (see Figure 3.4d).

Preparation of the base of the cell was done in a water bath. The 0.012 inch thick membrane was fixed to the base pedestal with an O ring, and the air removed from between the pedestal and the membrane. The membrane was then rolled down such that it did not protrude above the top of the base pedestal.

When the sand was mixed to an homogeneous state, the cylinder was inverted with care to prevent segregation, porous disk down, the aluminium disk removed and the connective membrane carefully pulled away from the face of the porous disk. The cylinder was then placed on the base pedestal, having removed any air bubbles from the face of the porous disk under water. (see Figure 3.4e). A firm downwards pressure was then maintained on the cylinder, the connective membrane removed, and the test membrane rolled up the outside of the cylinder.

The cell base was then removed from the water bath and the sample former put in place. The membrane was pulled away from the cylinder, and held in place within the former with a vacuum of about 7.5 cm of Hg. This was done with the base drainage open to a reservoir and a supply of deaired water to the top of the base disk to prevent desaturation of the porous disk. (see Figure 3.4f).

The plug was removed from the top of the cylinder, and the excess water on top of the sand eviscerated to prevent overflow on removal of the cylinder. The cylinder was then removed with one slow continuous movement. Continuity is required to prevent segregation of the grainsizes. (see Figure 3.4g).

The top of the sample was leveled, then the top cap placed and leveled. Next the sample was densified by vibration maintaining double drainage until the initial placement density required. A gentle pressure was maintained on the top cap during densification to prevent the possible formation of a loose surface layer which would lead to an underestimation of the liquefaction resistance. The top drainage line was then closed and the membrane sealed to the top cap with an O ring.

The sample was applied a suction of about 12 cm of Hg (17 kPa), the sample former removed, and the cell put together. The cell was then filled with deaired water and placed and centred on the loading frame. A small cell pressure was applied to overcome suction, and the pore pressure lines were then connected. The sample was now checked for saturation. Samples were only accepted if a Skempton's pore pressure parameter, B, of at least 0.99 was obtained.

Consolidation of the sample to the required stress with basal drainage only was then performed. This was done by increasing the cell pressure, step-wise, such that the consolidation characteristics of the sample could be monitored. At the final consolidation stress, the equilibrium readings were taken after the secondary consolidation, if any, had taken place. This was to ensure that pore pressure build up due to secondary consolidation did not effect the results during the shearing phase. For the sands tested, the secondary consolidation phase took approximately 10 minutes.

Tests were performed to determine the membrane penetration for the 3 gradations and the correction for each was applied to the volume changes in the consolidation phase. The corrections for the 3 gradations were found to be approximately equal. This is in accordance with Frydman et al (1973) who concluded that membrane penetration is a function of D_{50} only and not of gradation.

After consolidation the loading ram was connected to the loading piston. An eyed connecting ring was used to minimize the disturbance to the sample during connection and to prevent its premature loading.

The sample was now ready to be loaded.

The mean grain size is 0.42 mm and thus the permeability of the sand is high. Consequently, the rate of testing in the monotonic loading tests has no effect on the results on account of possible end restraint. A constant rate of strain of 0.4 % axial strain per minute was used for convenience.

During the monotonic tests, the axial load, porewater pressure, and axial deformation were monitored by electronic transducers and recorded using a data acquisition system, coupled to a computer.

Constant shear stress amplitude cyclic tests were performed. A sinesoidal load pulse with a frequency of 0.1 Hz was used. The low frequency loading was used to be compatible with the low frequency response capabilities of strip chart recorders. During each test, the cyclic axial load, porewater pressure, and axial deformation were continuously monitored by electronic transducers and recorded on a strip chart recorder. Corrections were made to the data for rod friction, and membrane strength. (Bishop & Henkel 1962).

4. TEST RESULTS

In this chapter, the undrained monotonic loading behaviour of the isotropically consolidated samples is discussed first, subdivided into the compression mode, the extension mode, and a discussion of isotropy. The cyclic results are then discussed.

The basis of comparison of the tests is identical relative density which is the approach taken by other researchers.

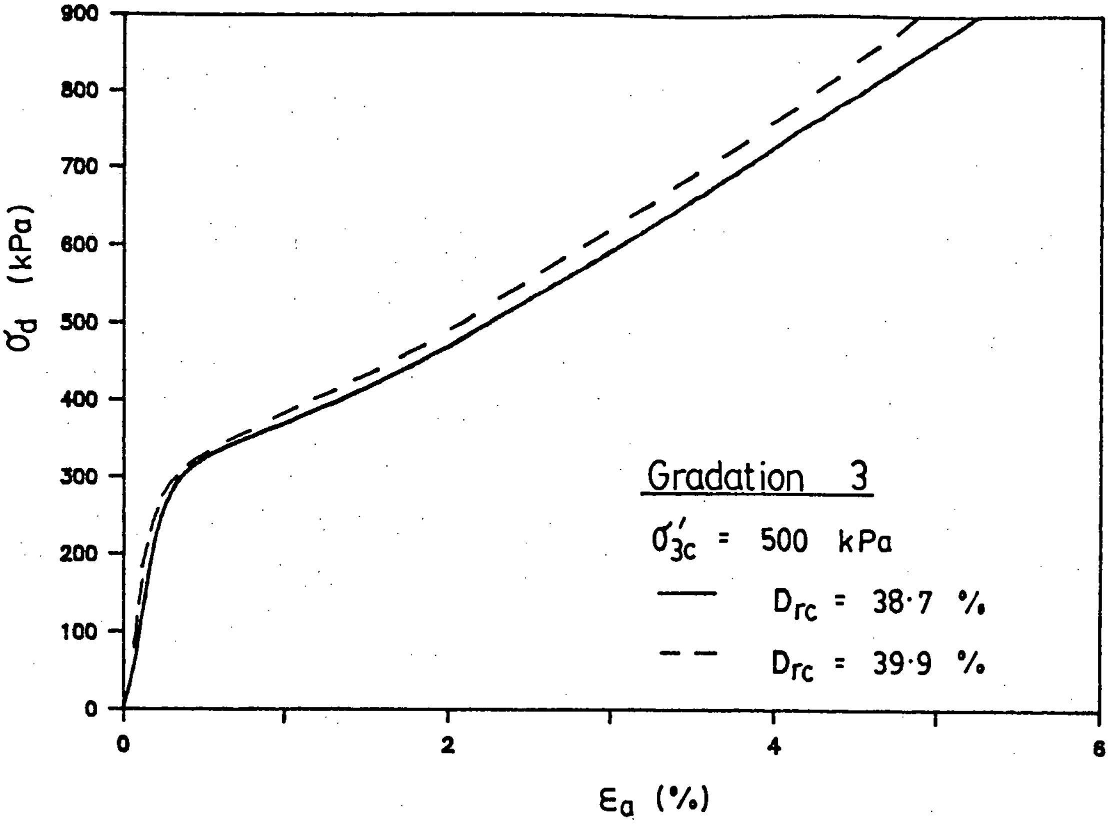
4.1. MONOTONIC LOADING BEHAVIOUR

Undrained monotonic loading tests were performed on isotropically consolidated samples at a constant relative density of 38.5 ± 1.5 %. Tests were performed in both the compression and extension modes. The compression modes will be discussed first.

4.1.1. Monotonic Compression Results

Monotonic compression tests were performed for initial confining pressures of 50, 200 and 500 kPa, for a relative density of 38.5 ± 1.5 %. Additional tests were carried out at each confining pressure for every gradation in order to ensure the repeatability of the tests. For Gradation 3, for an initial confining pressure of 500 kPa, Figure 4.1 presents the results of 2 tests at essentially identical relative density, given in terms of the deviatoric stress and excess porewater pressure as a function of axial strain. These results show excellent repeatability of the tests, reflected in the similarity in porewater pressure and deviatoric stress developed. This confirms the uniformity and consistency of the samples prepared by 'the slurry method.' The test performed at a relative density of 39.9% is slightly more dilative than the test at 38.7% relative density, probably due to its slightly higher density. (Seed & Lee 1966, Castro 1969, Casagrande 1976, Vaid & Chern 1985). The increased dilativeness is shown by the slightly higher deviatoric stresses and lower porewater pressures induced.

Results of the deviatoric stress and the excess porewater pressure as a function of axial strain, for initial confining pressures of 50, 200, and 500 kPa, are given in Figures 4.2, 4.3, and 4.4 respectively. All samples tested in monotonic compression, regardless of the coefficient of uniformity, exhibited strain hardening or dilative behaviour. The response of all the samples was that of type 3, shown in Figure 2.1, in that the samples at no time



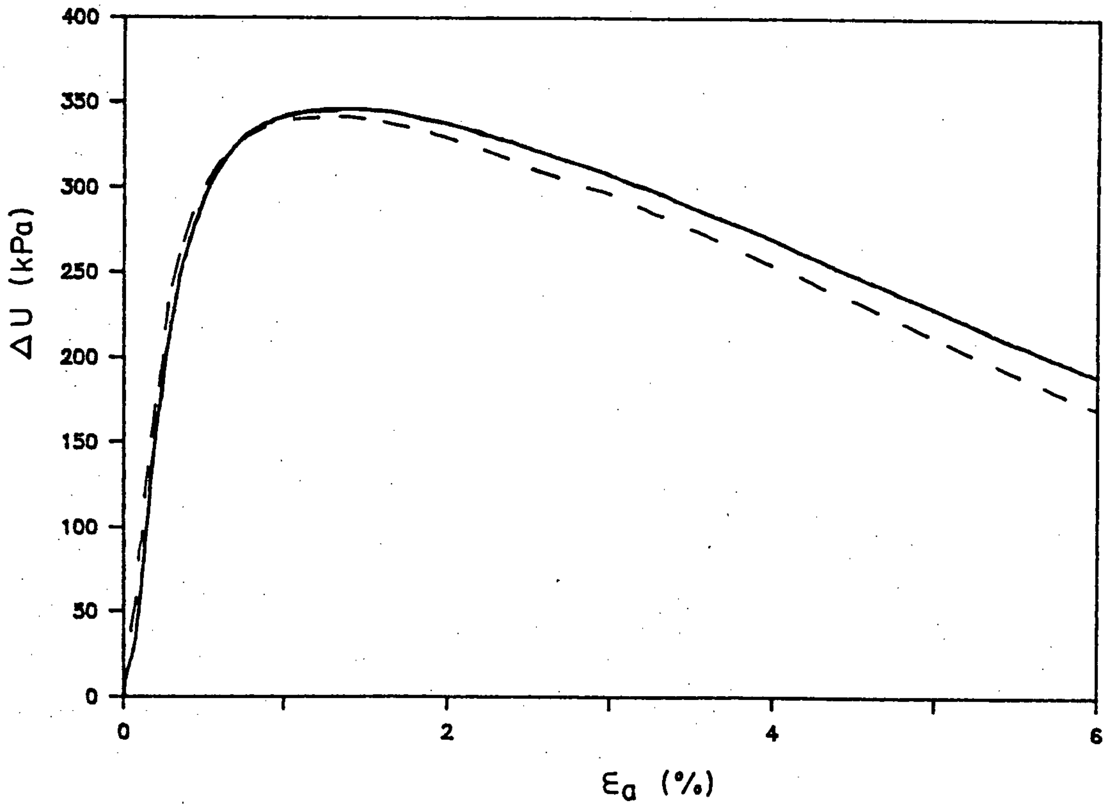
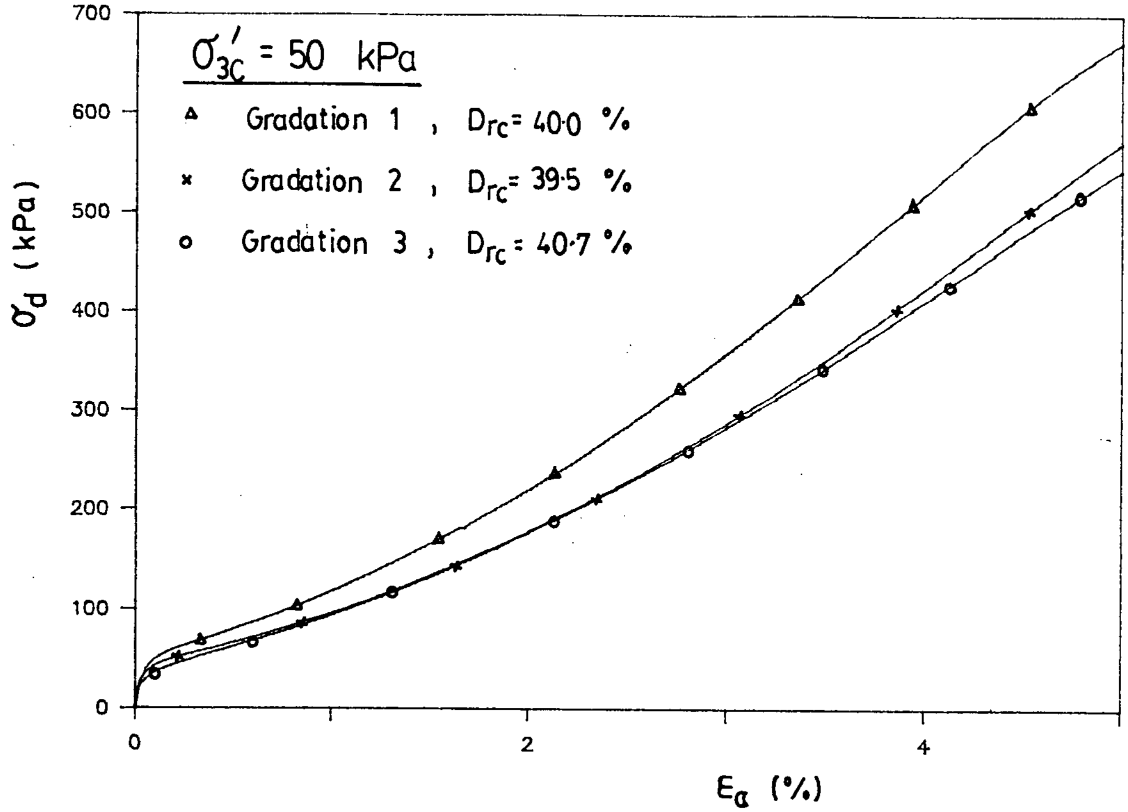


Figure 4.1: Undrained monotonic compression results for Gradation 1.



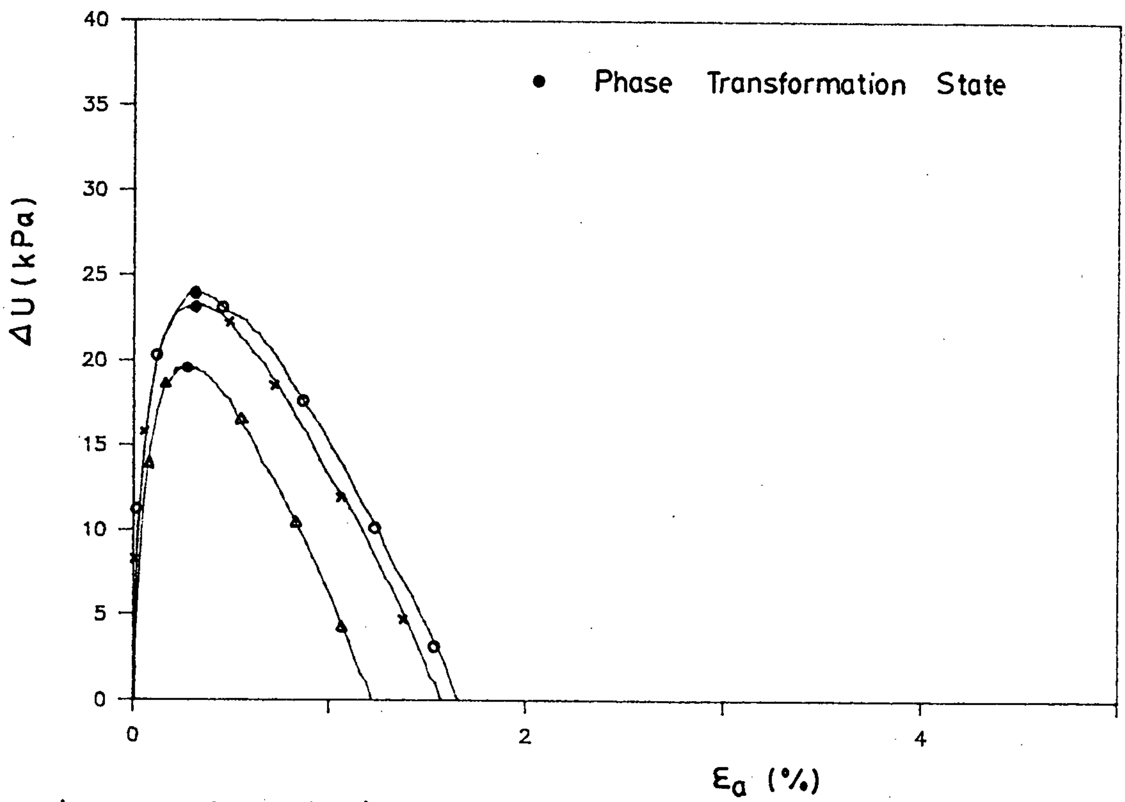


Figure 4.2: Undrained monotonic compression results for 50 kPa initial confining stress.

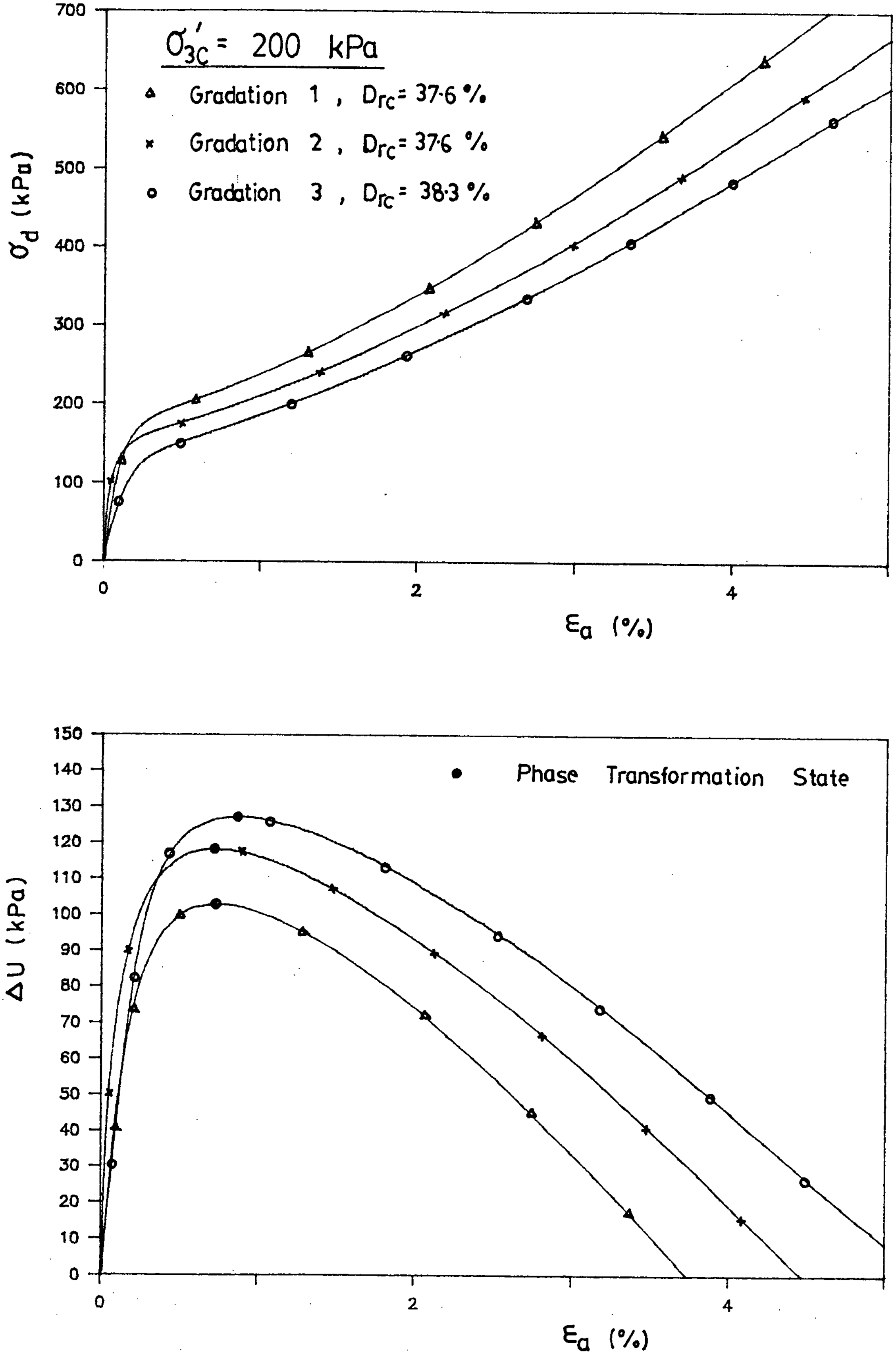


Figure 4.3: Undrained monotonic compression results for 200 kPa initial confining stress.

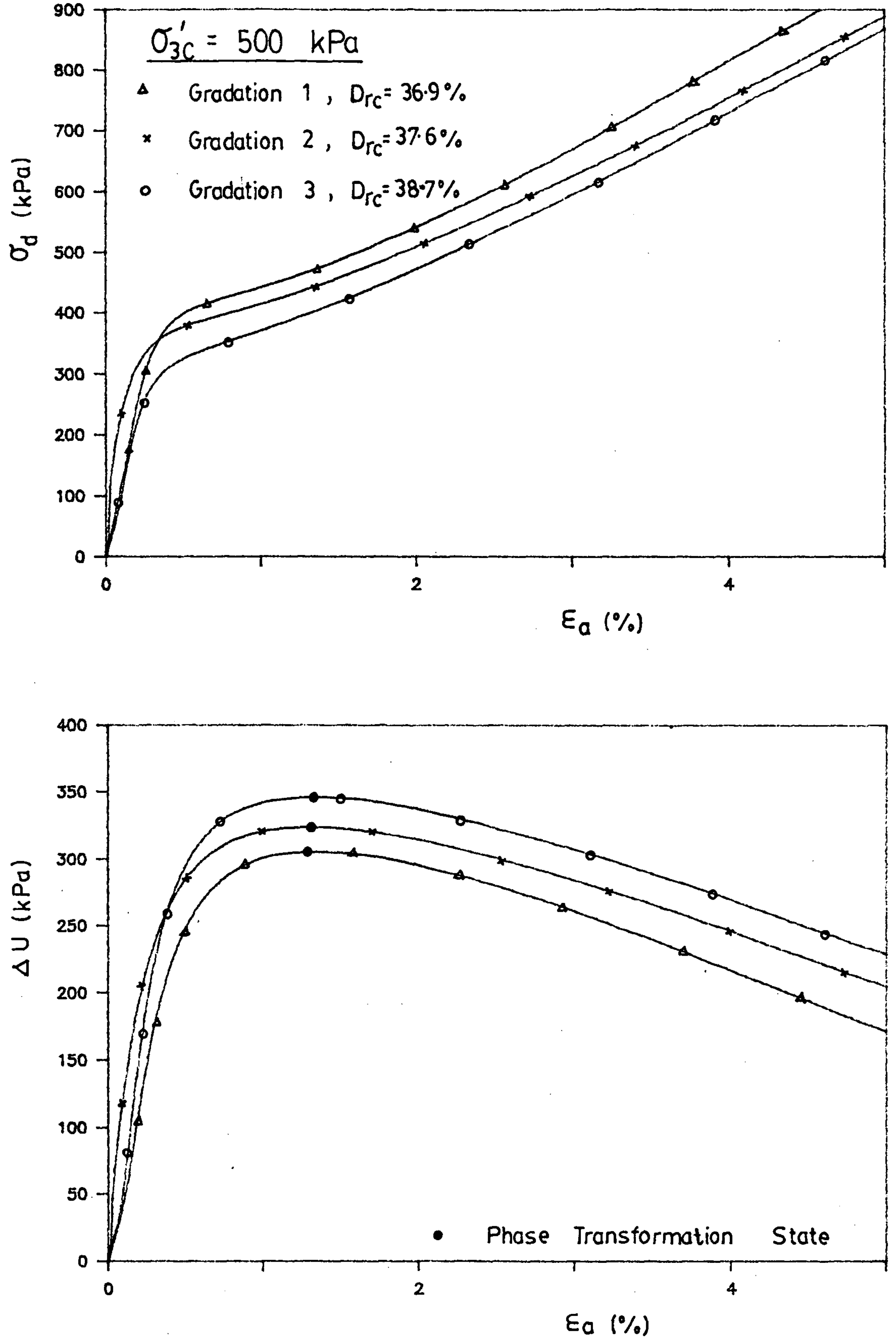


Figure 4.4: Undrained monotonic compression results for 500 kPa initial confining stress.

experienced a loss of shear resistance. It was not possible to explore the region of contractive behaviour at other relative densities over the range of confining pressures used, since specimens looser than a relative density of about 38 % could not be prepared by the slurry deposition technique.

The deviatoric stress and the excess porewater pressure as a function of axial strain for the compression test for an initial confining pressure of 50 kPa is given in Figure 4.2. The results show that an increase in the coefficient of uniformity causes the behaviour to be less dilative. This is reflected by the fact that the well graded sample sustained lower deviatoric stresses and developed higher porewater pressures than the uniform sample. This relationship can also be observed for the compression tests at 200 and 500 kPa initial confining pressure, Figures 4.3 and 4.4 respectively.

The stress paths for the monotonic compression loading for 50 kPa initial confining pressure, plotted on the Modified Mohr diagram are given in Figure 4.5. As evidenced in the deviatoric stress and excess porewater pressure vs axial strain plot, Figure 4.2, the stress paths indicate the strain hardening response, type 3, as shown in Figure 2.1.

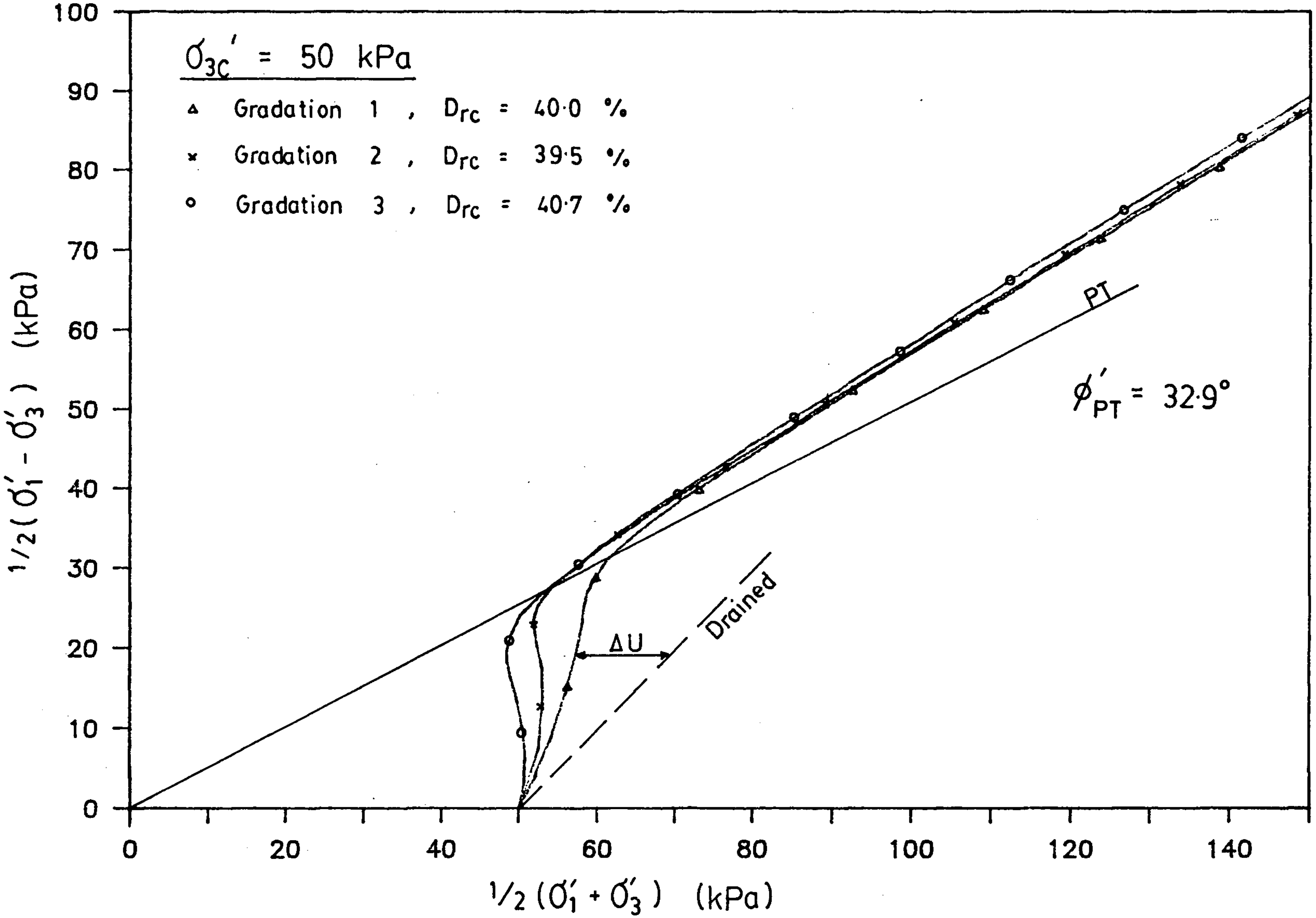


Figure 4.5: Modified Mohr diagram for undrained monotonic compression at 50 kPa initial confining stress.

The effect of the increasing coefficient of uniformity, causing a less dilative tendency, can also be seen in this Figure. This is reflected by the well graded sand developing greater porewater pressure than the uniform sand. This is shown by the relative horizontal shifts of the effective stress paths from the drained loading condition. The Modified Mohr diagrams for the monotonic compression tests at initial confining pressures of 200 and 500 kPa, Figures 4.6 and 4.7 respectively, show that this trend is continued at higher confining pressures, ie. an increase in the coefficient of uniformity causes the behaviour to become less dilative.

The undrained friction angle at maximum obliquity is found to be constant at 37.2 ± 0.7 degrees regardless of the gradation as shown in Figures 4.8, 4.9 and 4.10. The undrained friction angle at maximum obliquity is a constant for a given sand. (Seed & Lee 1967, Chern 1985). The drained friction angle, however, is affected by the dilatancy of the sand at failure, which is controlled by the level of confining pressure and relative density. (Lambe & Whitman 1969).

The stress state at phase transformation for all monotonic compression tests performed, regardless of relative density,

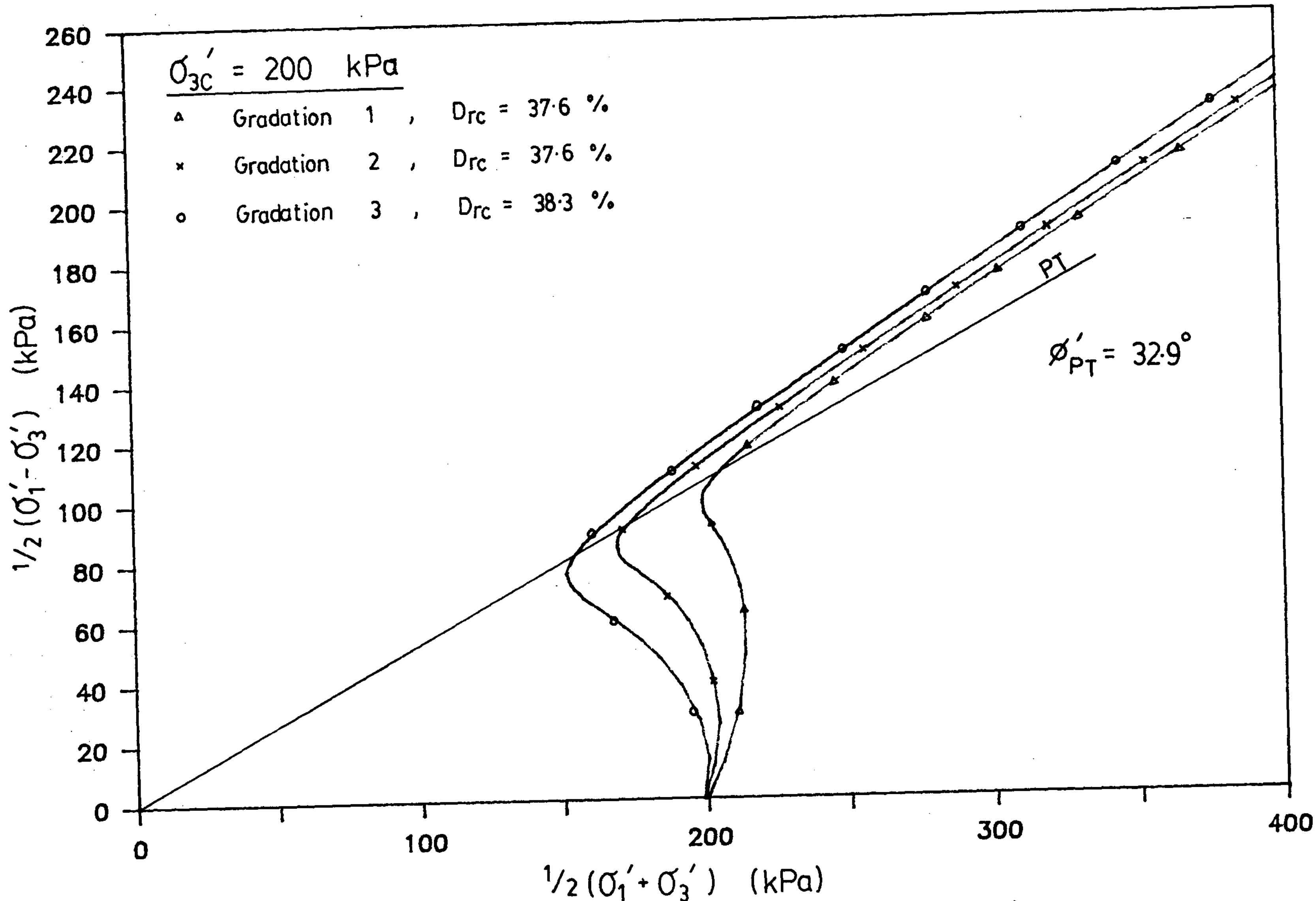


Figure 4.6: Modified Mohr diagram for undrained monotonic compression at 200 kPa initial confining stress.

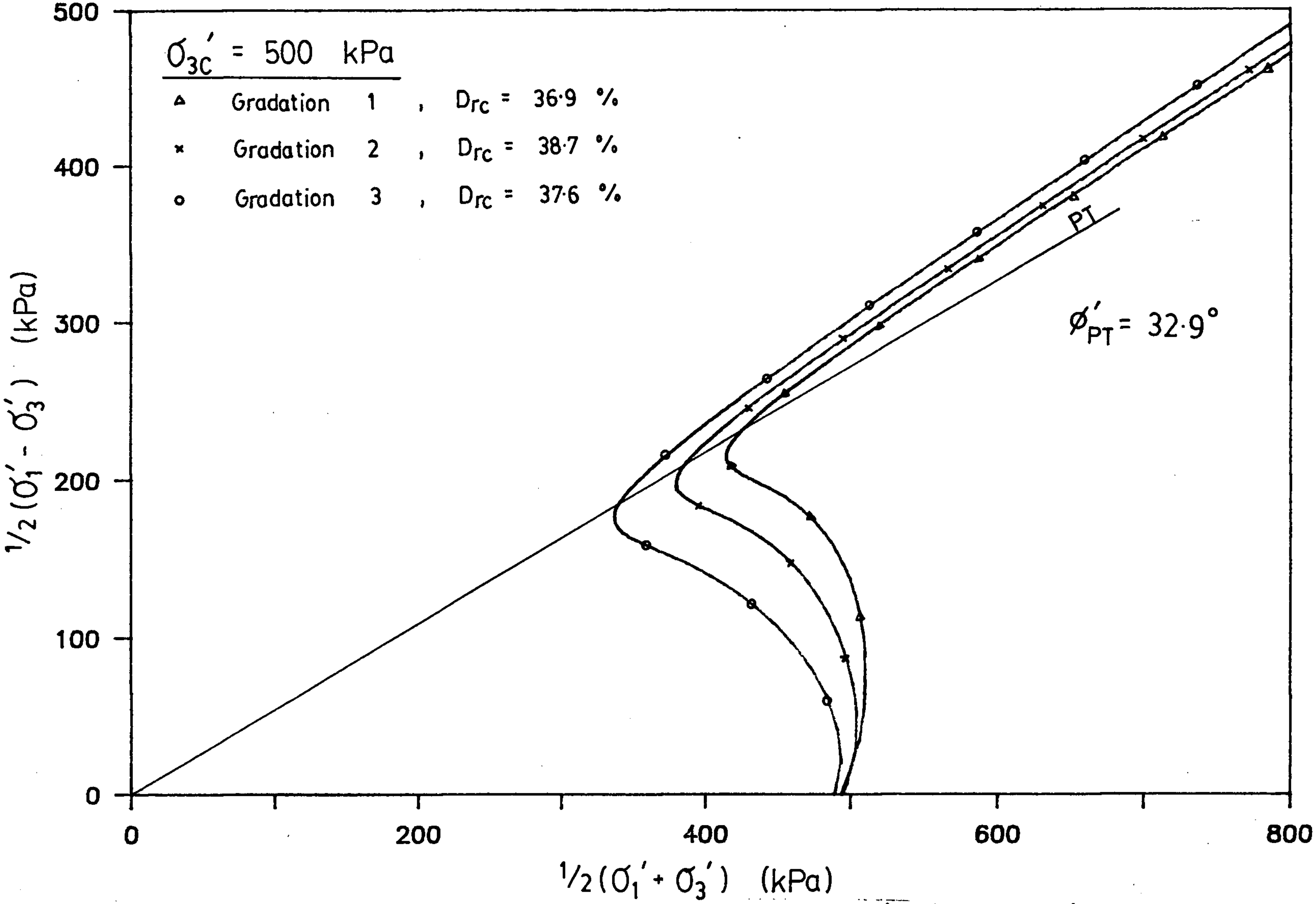


Figure 4.7: Modified Mohr diagram for undrained monotonic compression at 500 kPa initial confining stress.

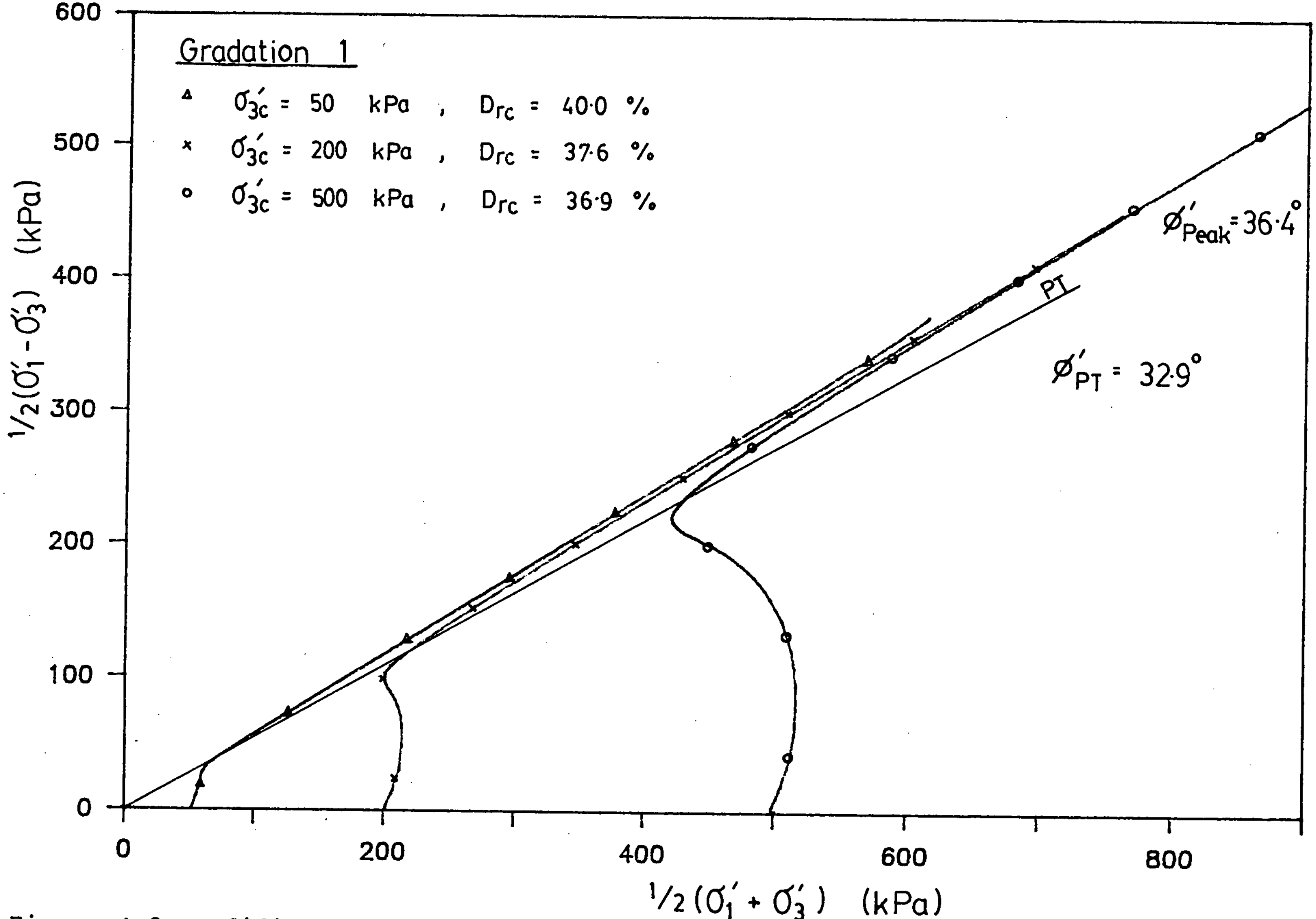


Figure 4.8: Modified Mohr diagram for undrained monotonic compression for Gradation 1.

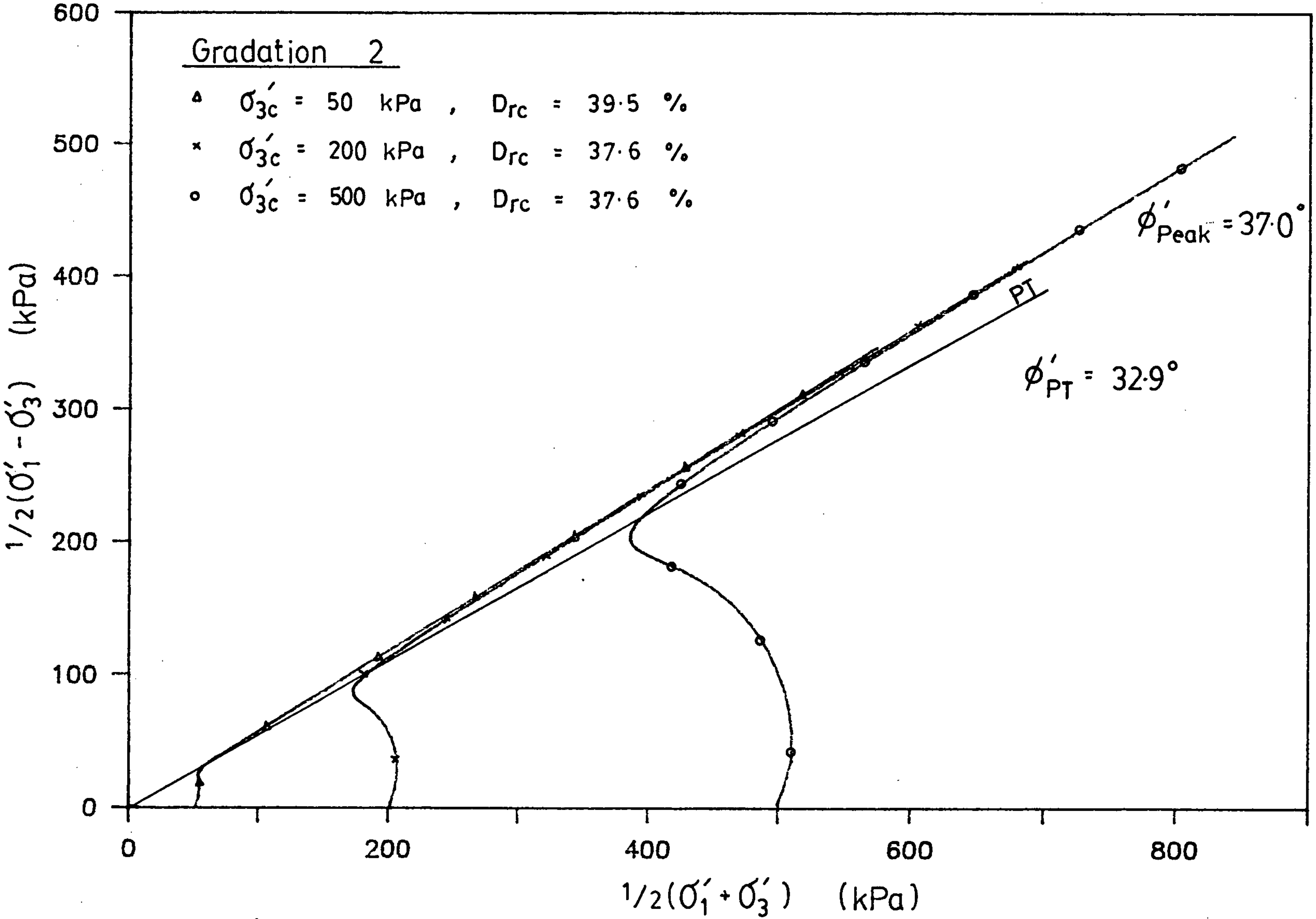


Figure 4.9: Modified Mohr diagram for undrained monotonic compression for Gradation 2.

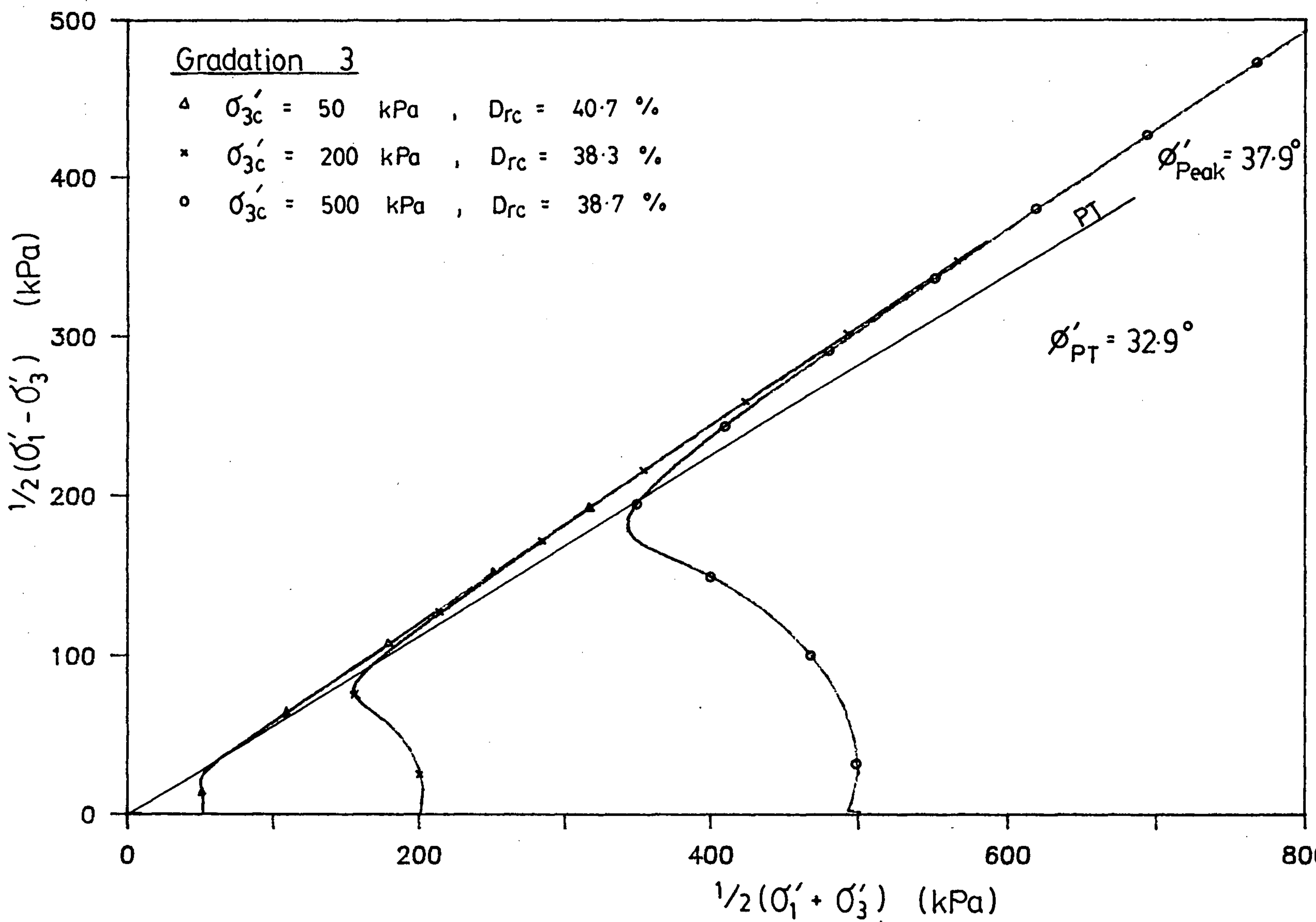


Figure 4.10: Modified Mohr diagram for undrained monotonic compression for Gradation 3.

is shown in Figure 4.11. The data points may be seen to lie on a straight line passing through the origin. Hence the friction angle mobilized at phase transformation is a constant at 32.9 degrees regardless of gradation. This would have been expected since the mineralogy is constant over the full particle size range. Negussey et al (1986) determined that, for a given mineralogy, the friction angle mobilized at phase transformation was independent of the particle size, confining pressure, porewater pressure, and density. This observation can now be extended to include gradation.

4.1.2. Monotonic Extension Results

Monotonic loading tests in the extension mode on the 3 gradations were performed for a constant initial confining stress of 200 kPa. Comparative results at identical relative density in terms of the deviatoric stress and excess porewater pressure vs axial strain and the stress paths on the Modified Mohr diagram are given in Figures 4.12, and 4.13 respectively. The results indicate that as the coefficient of uniformity increases, the behaviour becomes more dilative under extension loading. This is reflected by the fact that as the coefficient of uniformity increases, the sand sustains higher deviatoric stresses as shown in Figure 4.12, and it develops lower porewater pressures as

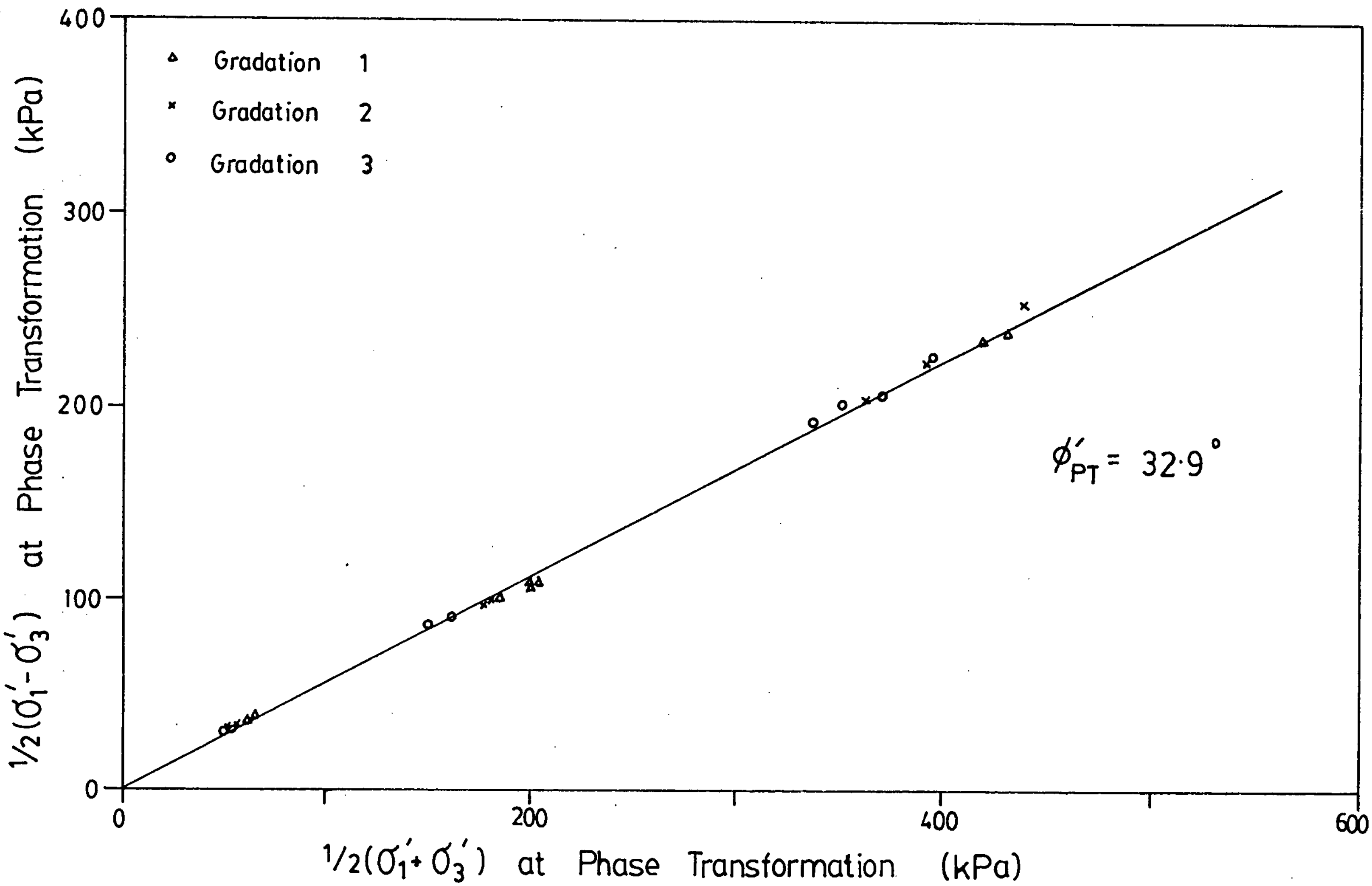


Figure 4.11: Modified Mohr diagram showing the phase transformation state.

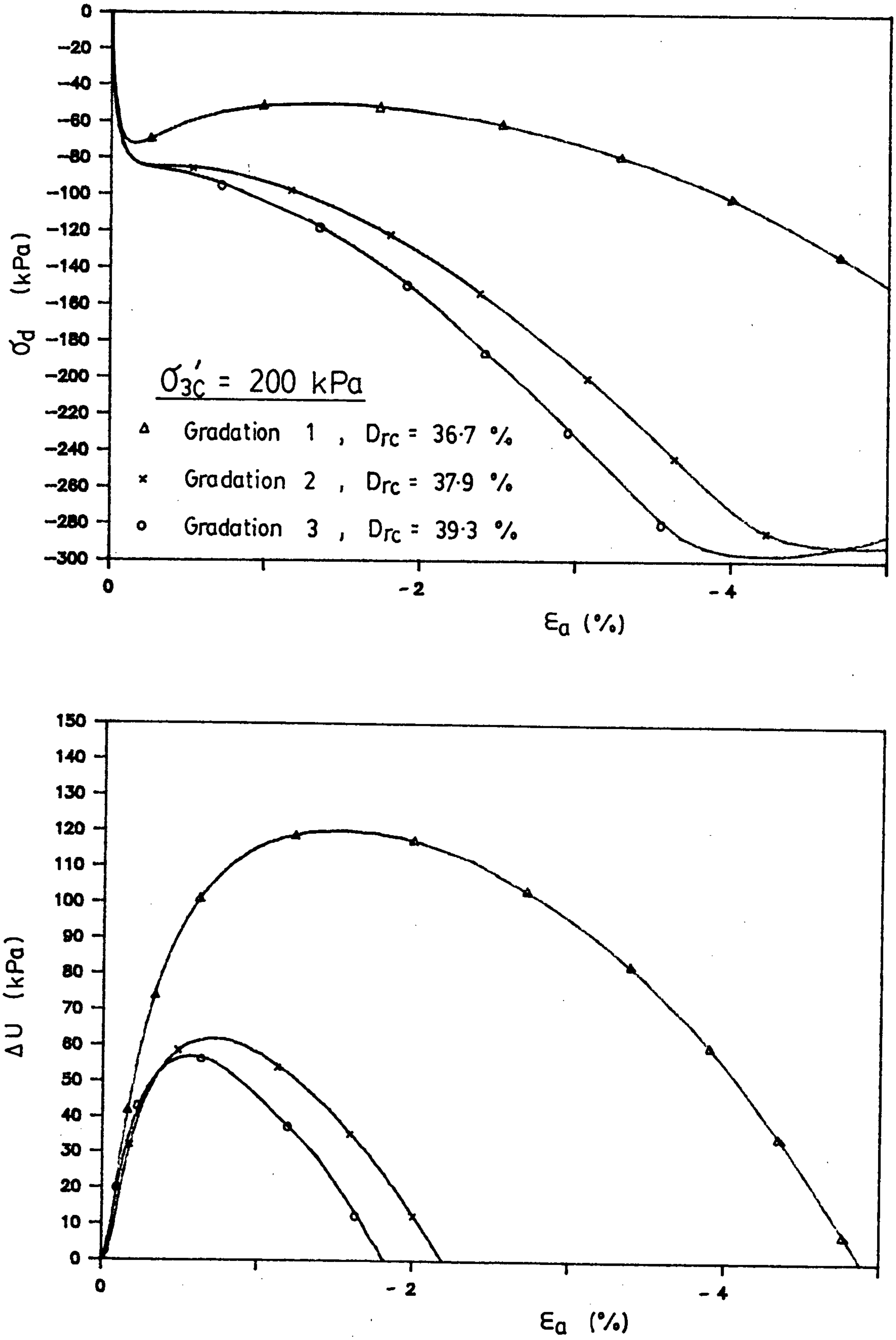


Figure 4.12: Undrained monotonic extension results for 200 kPa initial confining stress.

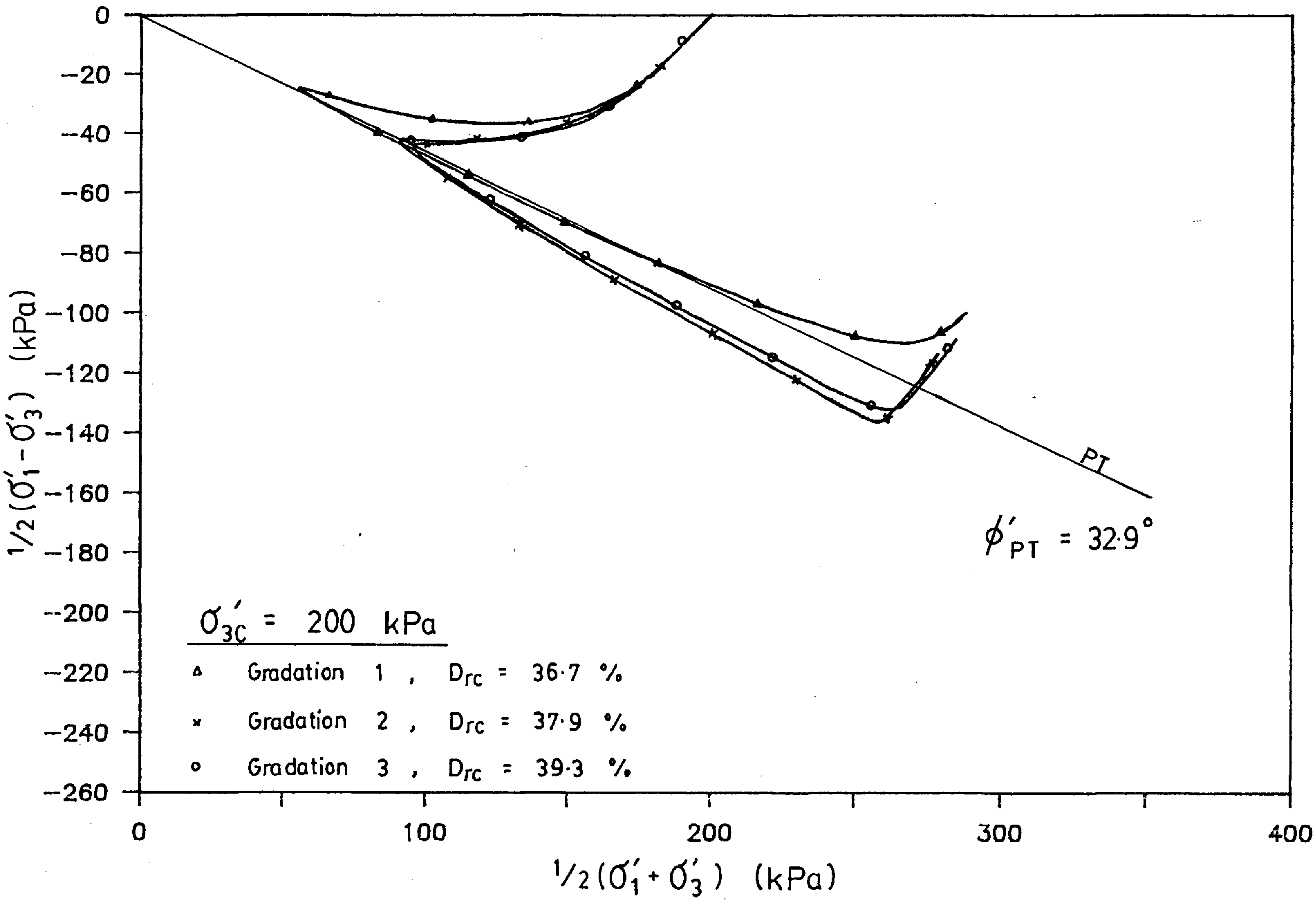


Figure 4.13: Modified Mohr diagram for undrained monotonic extension for 200 kPa initial confining stress.

shown in Figures 4.12 and 4.13. This is the reverse to the trend identified when similar samples are subject to compression loading.

Gradation 1, the more uniform sample, exhibits contractive behaviour. The response is that of type 2, Figure 2.1, limited liquefaction. The sample suffers a temporary loss of shear resistance which is regained with further straining.

Gradations 2 and 3, with coefficients of uniformity of 3 and 6 respectively, on the other hand, show a strain hardening response, type 3, Figure 2.1. The plateau of deviatoric stress observed, in Figure 4.12, in Gradation 3 at 3.8 % axial strain, and in Gradation 2 at 4.2 % axial strain is caused by necking of the sample. This necking also causes the sharp turn around in the stress paths in Figure 4.13. Necking in Gradation 1 occured gradually, and is shown by the gradual change of the slope of the stress path after the phase transformation state in Figure 4.13.

The friction angle mobilized at phase transformation, under extension loading is 32.9 degrees. Thus, this angle is the same under compression and extension loading. This was also observed by Chern (1985) and Chung (1985) for other sands.

Initial conditions which give rise to a dilative response under monotonic loading, can develop only cyclic mobility under cyclic loading. If a contractive response is obtained under monotonic loading, cyclic loading can give rise to liquefaction, limited liquefaction, or cyclic mobility. Contractive response was obtained only with Gradation 1 under extension loading at relative densities of less than about 48 %. for the selected initial confining pressure of 200 kPa.

In extension, provided contractive response ensues, the strength at phase transformation, S_{PT}, is a function of initial relative density, D_{ri}, as well as the relative density after consolidation, D_{rc}. (Chung 1985). Consequently, further tests were performed on Gradation 1 under monotonic extension loading to determine the relationship between D_{ri}, D_{rc}, and S_{PT} to assist in a rational interpretation of the cyclic loading results in respect of the mechanism of strain development. Extension tests were performed on samples for a constant initial confining stress of 200 kPa, with the relative density before consolidation, D_{ri}, varying between 16.3 and 47.8 %.

Figure 4.14 shows the stress paths on a Modified Mohr diagram for 2 of these tests, for a constant initial confining stress of of 200 kPa. The angle of internal friction mobilized at phase transformation can be seen to be a constant of 32.9 degrees regardless of relative density. The relative density, however, governs the strength at phase transformation.

The angle of internal friction mobilized at the Critical Stress Ratio is also a constant at 17.8 degrees, regardless of the relative density, as shown in Figure 4.14.

The relationship between S_{PT}, D_{ri}, and D_{rc} is given in Figure 4.15. The relationship between D_{rc} and S_{pt} was determined at D_{ri} = 34.5 %. For additional D_{ri}'s, one test was carried out and the relationships were assumed parallel. As D_{rc} and/or D_{ri} increases, the strength at phase transformation, S_{PT}, increases.

4.1.3. Review of Monotonic Test Results

The effective stress paths on a Modified Mohr diagram for the monotonic compression and extension tests at D_{rc} = 38.5 ± 1.5 % and for 200 kPa initial confining pressure are shown in Figure 4.16. Under compression loading, the well graded sand is more contractive than the uniform sand, as it develops higher porewater pressures. This appears to be in

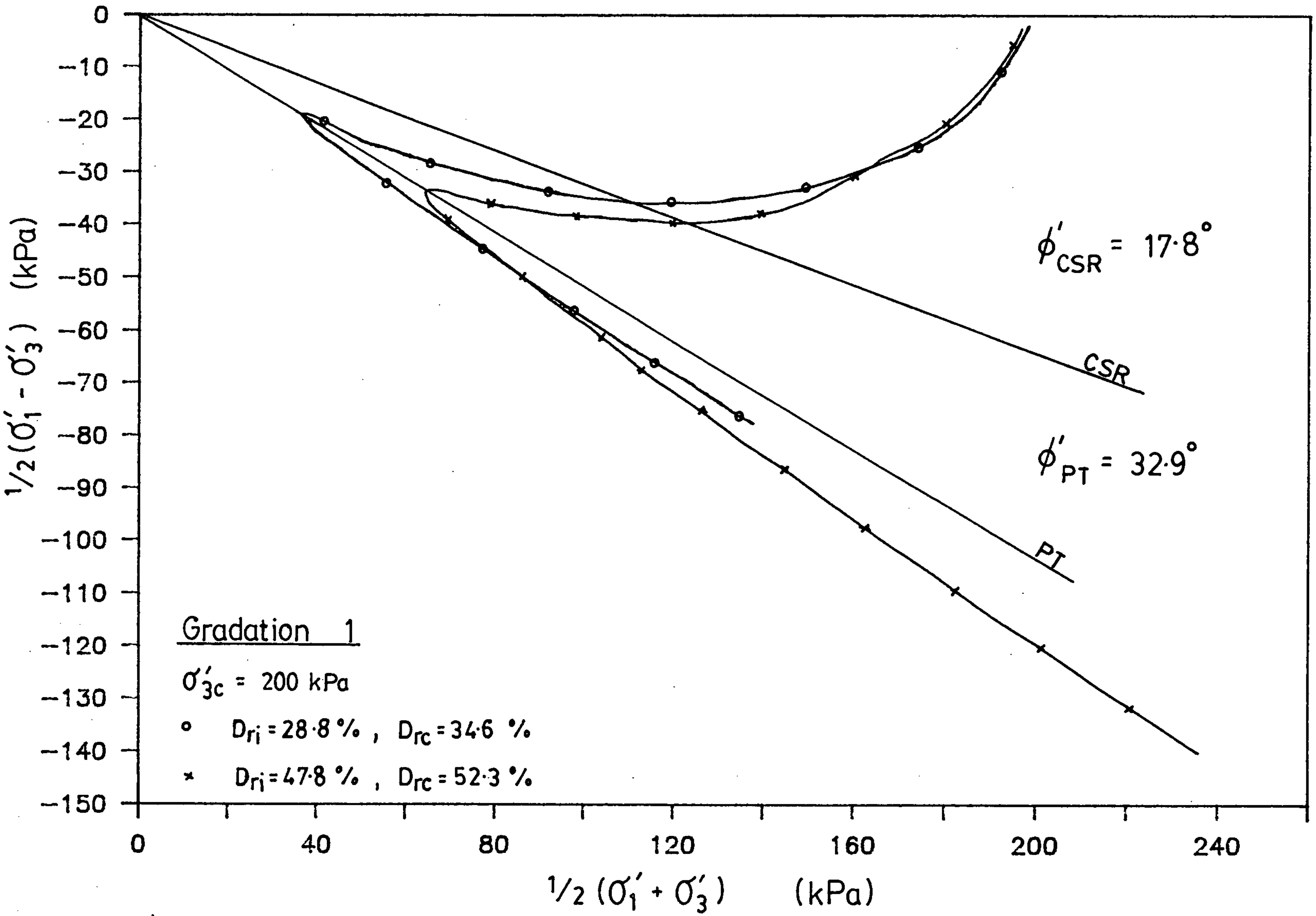


Figure 4.14: Modified Mohr diagram for undrained monotonic extension for Gradation 1 at 200 kPa initial confining stress.

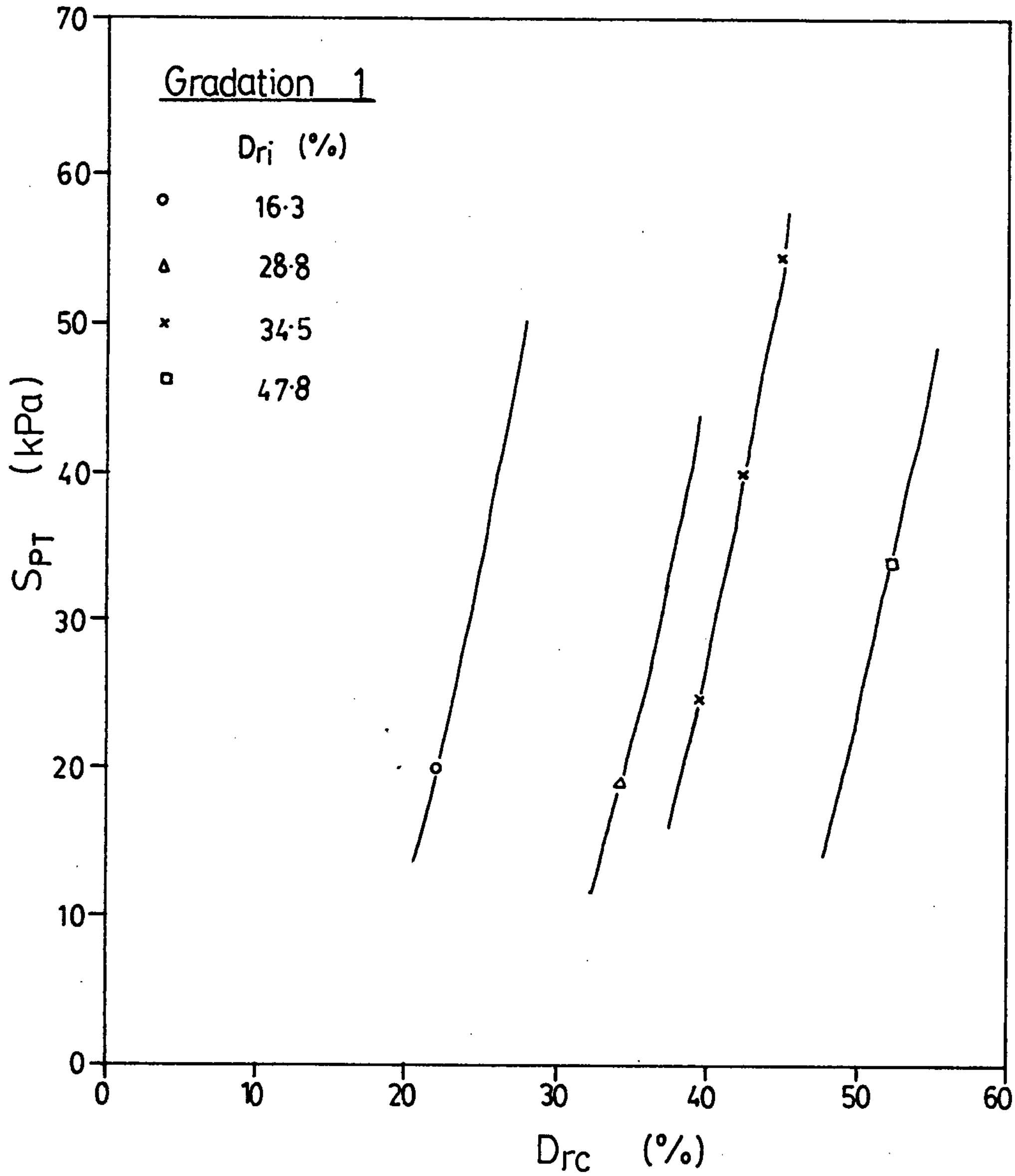


Figure 4.15: Relationship between initial relative density, relative density after consolidation and the strength at phase transformation.

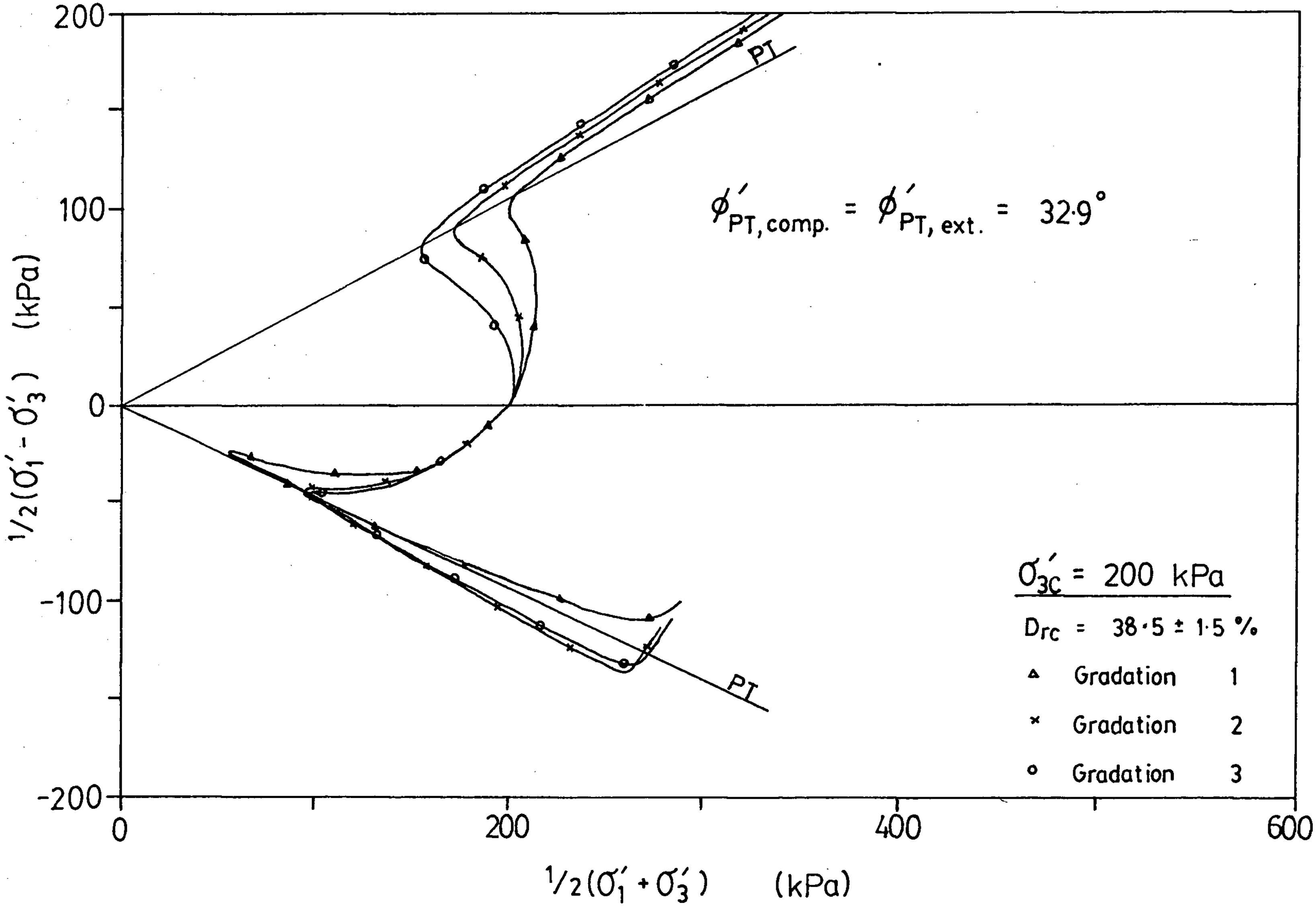


Figure 4.16: Modified Mohr diagram for monotonic extension and compression loading for 200 kPa initial confining stress.

conformity with consolidation test results, Figure 3.3, that show that the well graded sand is more compressible than the uniform sand under hydrostatic load. The effective stress paths in Figure 4.16 show that the relative porewater pressure development of the 3 gradations under compression loading is reversed under extension loading. Thus, relative shear-induced compressibilities under monotonic loading are not constant for a given sand but a function of the type of loading, ie. compression or extension.

4.2. CYCLIC LOADING BEHAVIOUR

Cyclic loading tests were performed on isotropically consolidated samples at a constant initial confining stress, σ_{3c}', of 200 kPa. The cyclic stress ratio (τ_{cy}/σ_{3c}'), and relative density were the variables. The results are presented in the plots of relative density against the no. of cycles to liquefaction (or limited liquefaction) or 2.5 % axial strain for constant cyclic stress ratios in Figures 4.17, 4.18, and 4.19, Gradations 1, 2, and 3 respectively.

For gradations 2 and 3, all samples achieved 2.5 % axial strain through cyclic mobility, following first realization of a state of transient zero effective stress. For the same initial density and stress states, the behaviour of

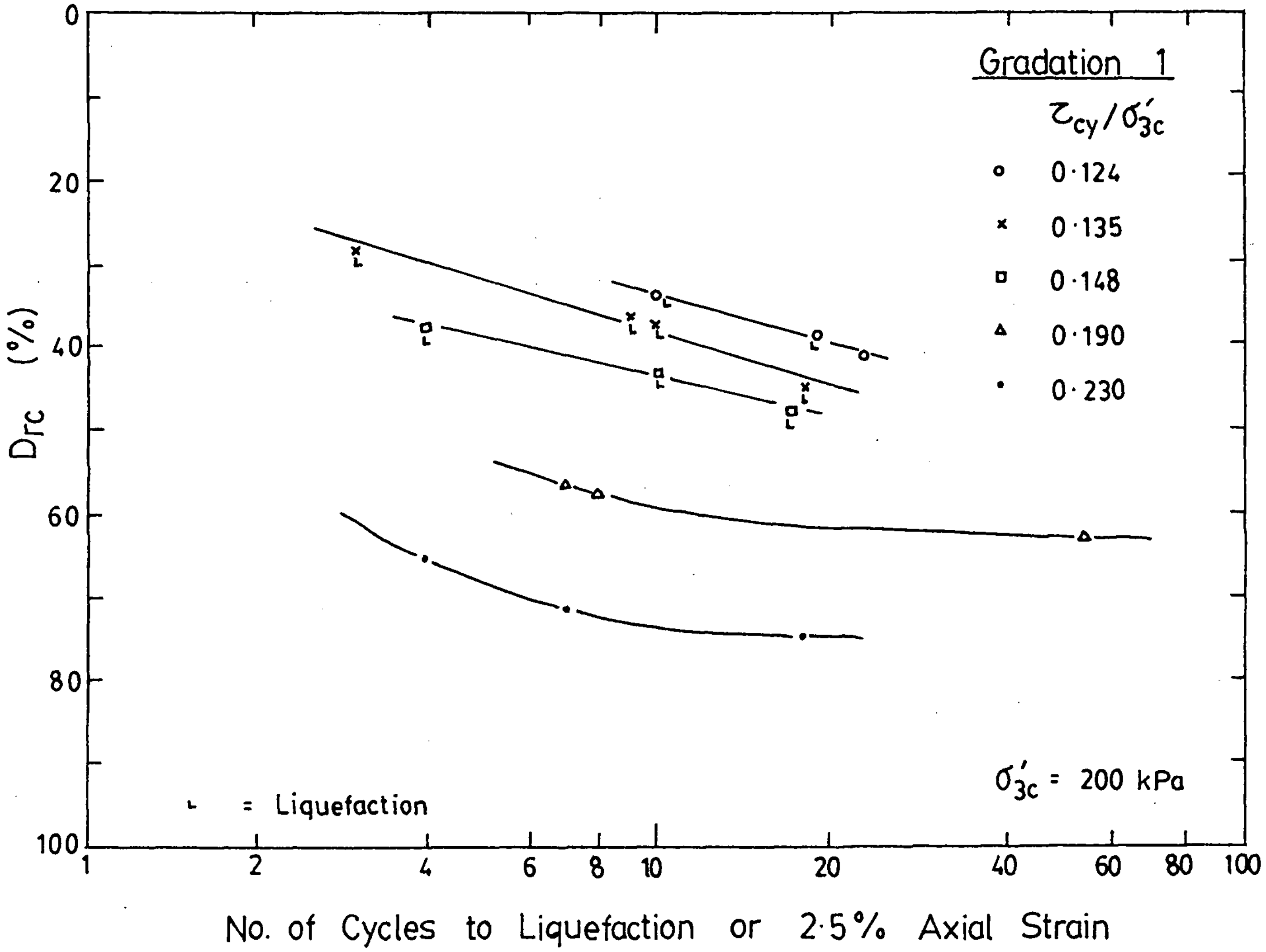


Figure 4.17: Relationship between relative density and no. of cycles to liquefaction or 2.5 % axial strain at constant cyclic stress ratios for Gradation 1.

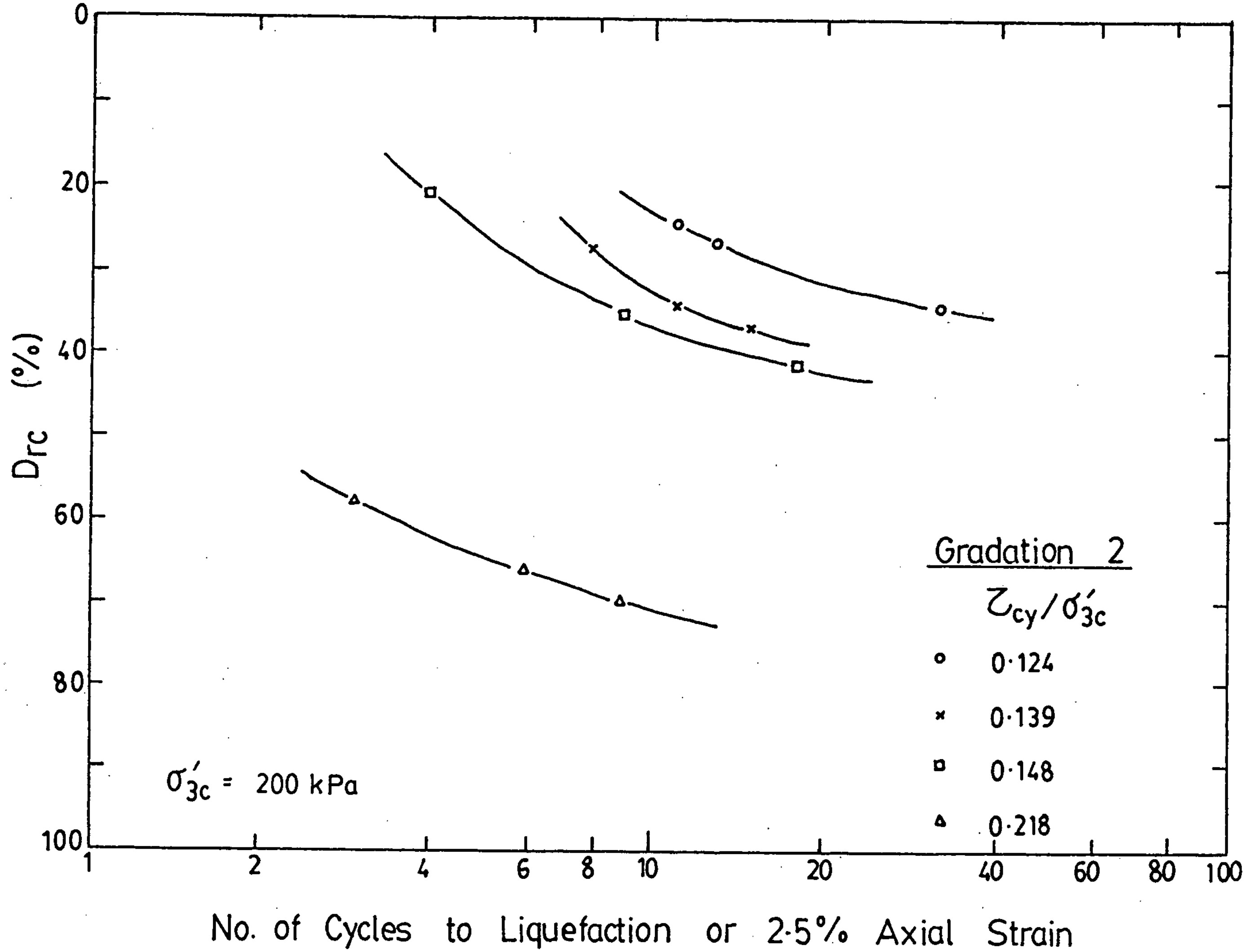


Figure 4.18: Relationship between relative density and no. of cycles to liquefaction or 2.5 % axial strain at constant cyclic stress ratios for Gradation 2.

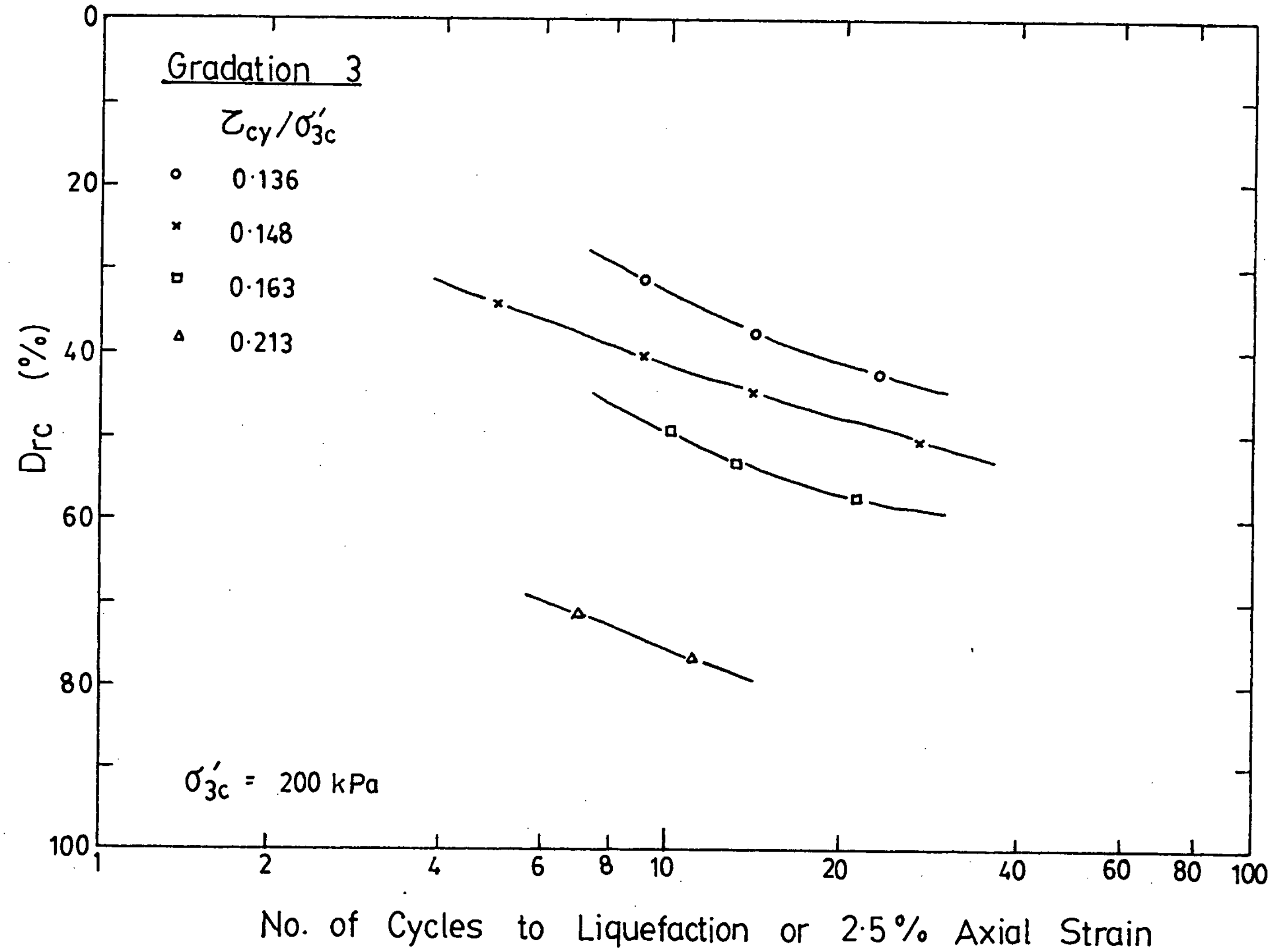


Figure 4.19: Relationship between relative density and no. of cycles to liquefaction or 2.5 & axial strain at constant cyclic stress ratios for Gradation 3.

Gradations 2 and 3 under monotonic loading was dilative in extension and compression, therefore liquefaction (or limited liquefaction) under cyclic loading in this stress range could not occur.

Under monotonic extension loading, Gradation 1 was contractive over a range of relative densities for the selected 200 kPa initial confining pressure. Thus under cyclic loading, Gradation 1 can develop dilative or contractive behaviour. Contractive behaviour under cyclic loading was developed if the 3 requirements listed in Section 2.4 were met, ie. the initial state would lead to contractive behaviour under monotonic loading, the shear stress was greater than the strength at phase transformation, and there were sufficient cycles. Those samples that met these requirements, ie. developed liquefaction or limited liquefaction, are marked by an 'L' in Figure 4.17. If liquefaction occured, the relationship in D_r vs log N space, for a fixed cyclic stress ratio, is approximately linear. This relationship is exhibited by tests at cyclic stress ratios of 0.124, 0.135 and 0.148, in Figure 4.17. This behaviour is similar to observations made by Castro (1982) and Vaid & Chern (1983).

As the relative density increases, the shear strength at phase transformation, S_{PT}, increases. Consequently, the cyclic shear stress required to initiate contractive deformation also increases. Therefore as the relative density increased, the range of initial stress states for dilative behaviour increased, and the response of the sand tested changed from liquefaction to cyclic mobility. When liquefaction occured, it was initiated in the extension phase in all cases. Gradation 1 portrayed contractive behaviour under monotonic extension only. Consequently, the potential for liquefaction under cyclic loading in this stress range was in the extension phase.

A comparison of the resistance of the 3 gradations to cyclic loading is given in Figure 4.20. The data is presented in terms of the cyclic stress ratio required to induce liquefaction or 2.5 % axial strain in 10 stress cycles for a range of relative densities. From Figure 4.20, certain trends can be identified.

At low relative densities, (less than about 45 %), the uniform sand, Gradation 1, has the least resistance to cyclic loading. This is the range of densities over which Gradation 1 was contractive under extension loading. The resistance increases as the sand becomes more well graded. The increase may be related to both improved gradation, as

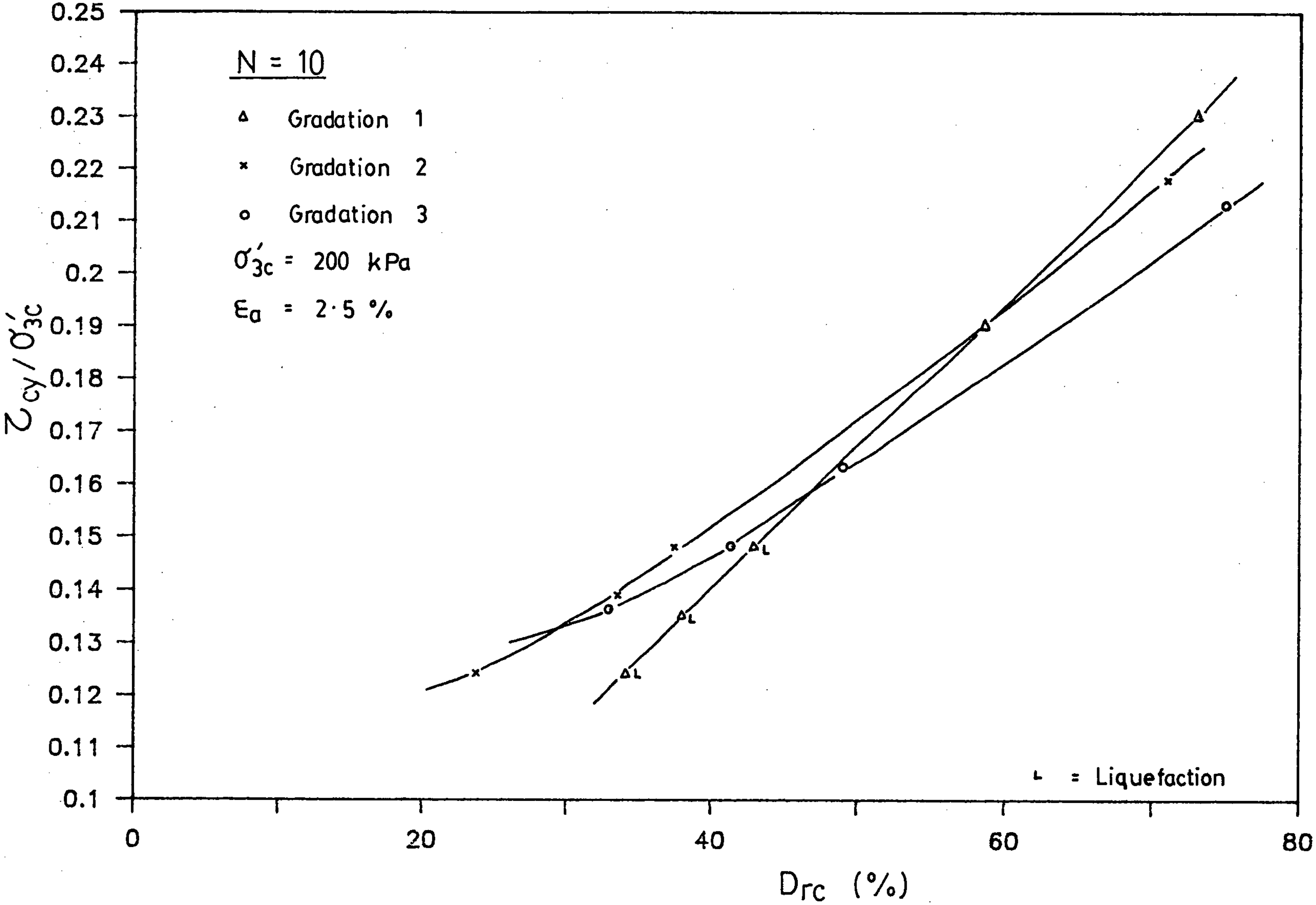


Figure 4.20: Liquefaction resistance curves for N=10.

well as the change in the mechanism of deformation from liquefaction to cyclic mobility.

The cyclic strength of every sand increases with increasing relative density. As the coefficient of uniformity increases, less benefit appears to be derived from increasing the relative density, ie. less strength is gained. This is evident from the slopes of the lines in cyclic stress ratio-relative density space. (Figure 4.20). This trend causes the relative strengths of the gradations to be reversed at high relative densities, (greater than about 60 %), with the well graded sand sustaining a lower cyclic stress ratio than the uniform sand at the same relative density.

These results are generally supported by those of Wong et al (1974), who showed that at a relative density of 60 %, a well graded sand required a smaller cyclic deviator stress than a uniform material to develop 2.5 % axial strain in 10 cycles.

The basis of the comparison of the undrained behaviour of the 3 gradations of Earls Creek sand is identical relative density. This basis, however, may not be completely satisfactory due to the large range in absolute density. The stress-deformation and shear strength are not only affected by the relative density, but also by the absolute density, ie. the grain size distribution. (de Beer 1965). For the 3 gradations of Earls Creek sand tested, the large variation in absolute density between the gradations preclude its use as a basis of comparison. On examination of Figure 4.20, it can be seen that at high D_r, (greater than about 60 %), an attempt to compare the liquefaction resistance curves at an identical absolute density, however, would push the curves further apart. This is due to the fact that the uniform sand, Gradation 1, has the highest cyclic shear strength but the lowest absolute density. Consequently, normalization of the liquefaction resistance curves with respect to absolute density would exaggerate the trend already shown at high relative densities.

The state parameter has been proposed as an alternative initial parameter to relative density. The state parameter defines the state of the sand as a function of its position relative to the steady state line in e-log p' space. (Been & Jefferies 1985). However the state parameter is not unique in extension and compression. (Chern 1985). Also the state parameter is only defined for sands which exhibit contractive behaviour, as for dilative behaviour, the steady state line does not exist. The state parameter cannot be used here as, for the stress range considered, of the 3 gradations that were tested, only Gradation 1, the more uniform sand, tested in extension, exhibited contractive behaviour. The stress range considered here is relevant to most applications.

5. CONCLUSION

In order to determine the effect of the coefficient of uniformity on the undrained behaviour of sand, undrained monotonic and cyclic triaxial tests were performed on 3 sands of varying straight line gradations, with identical mineralogy and identical D_{50}. Monotonic tests in compression and extension were performed at constant relative density, D_{rc}, and with initial confining stresses varying up to 500 kPa. Undrained cyclic tests were performed from a constant isotropic effective confining stress of 200 kPa, with varying relative density and cyclic stress ratios. All the samples that were tested, were isotropically consolidated. Based on the test results, several conclusions can be drawn.

Under monotonic compression loading, the sand becomes less dilative as the coefficient of uniformity increases, ie. as the sample becomes more well graded. The compressibility of the sand increasing with gradation is also exhibited under hydrostatic loading during consolidation. Under monotonic extension , the opposite trend is observed, with the sand becoming more dilative as the gradation increases. Thus, the relative shear-induced compressibilities are a function of the undrained stress path.

Under cyclic loading, at low relative densities, (less than about 45 %), increased cyclic strength is obtained by increasing the coefficient of uniformity. Cyclic loading induced liquefaction or limited liquefaction in the uniform sample, while the deformation in the more well graded samples accumulated by cyclic mobility.

Increasing the relative density causes a greater cyclic strength increase in the more uniform samples. Thus at high relative densities, (greater than about 60 %), the well graded samples show greater propensity towards deformation accumulation. At these high relative densities the deformation was caused by cyclic mobility.

At low relative densities, the uniformly graded sand was found to have a much lower cyclic resistance than the well graded sand. When compared at low cyclic strength levels, equivalent relative density states for the uniform sand can be 15 to 20 % greater than for the well graded sand.

For uniform sand, liquefaction was experienced over a range of relative densities, from the loosest state at 33 % to 43 % relative density. The more well graded sands, even at their loosest relative density states, (approximately 23 %), experienced cyclic mobility. This implies that, at low relative densities, gradation might control the occurrence of liquefaction.

The effectiveness of field densification is dependent on the gradation of the sand at low relative densities. The cyclic shear strength of a uniform sand is greatly improved by an increase in relative density. For a well graded sand, similar increases in relative density will cause much smaller cyclic shear strength increases.

At high relative densities, there is not much improvement in cyclic shear strength with gradation. Consequently, the effect of gradation on the undrained response may not be significant at high relative densities.

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