SOILS AND FOUNDATIONS Vol. 23, No. 1, Mar. 1983
Japanese Society of Soil Mechanics and Foundation Engineering

THE INFLUENCE OF STRAIN RATE ON PORE PRESSURES IN CONSOLIDATED UNDRAINED TRIAXIAL TESTS ON COHESIVE SOILS

Tsutomu Kimura* and Kunio Saitoh**

ABSTRACT

The influence of strain rates on pore pressures of cohesive soils in consolidated-undrained triaxial tests was experimentally studied. Three types of cohesive soils were used; Kawasaki clay with plasticity index of 30 and two artificial mixtures of Kawasaki clay and Toyoura sand with plasticity index of 20 and 10. Pore pressure measurements were carried out by small transducers at the centre, bottom and periphery of triaxial specimens. It was found that at low strains the pore pressure at the centre was larger than that at the bottom and periphery, while at high strains the difference was less significant. The difference in pore pressures at the three different locations was smaller for lower strain rates and for soils with lower plasticity index. Pore pressures in undrained triaxial specimens of soil with plasticity index up to 30 were considered to be uniform at the strain rate of 4%/h. It was concluded that the test duration given by Blight was satisfactory from the standpoint of pore pressure equalization.

Key words: consolidated undrained shear, pore pressure, triaxial compression test, test procedure, shear strength, measurement (**IGC**: D 6)

INTRODUCTION

It is well-known that cohesive soils show different pore pressure response to different strain rates even in undrained triaxial tests. It has been shown by several research workers that pore pressure distribution is not uniform in triaxial specimens in undrained tests.

Crawford (1959) conducted comprehensive studies on Leda clay. Samples in his experiments were taken from the depth of approximately 5 m. The plasticity index was 39. He observed that the magnitude of pore pressure at failure in undrained triaxial tests was different for different strain rates. In order to look into this problem, he carried out undrained triaxial tests with two different strain rates, 2%/h and 0.4%/h. Crawford stopped increasing the axial stress in each test when the deviator stress reached one-third of the maximum value and held it constant for several hours. For the first five hours the increase of pore pressure was

* Professor, Civil Engineering Department, Tokyo Institute of Technology, 2-12-1 O-okayama, Meguro-ku, Tokyo.
** Research Associate, ditto.
Manuscript was received for review on July 20, 1982.
Written discussions on this paper should be submitted before January 1, 1984.

larger in the test with the strain rate of 2%/h than in the other. This trend can be more clearly seen in his plot of the pore pressure change versus the change in axial strain. The rate of increase of pore pressure with the increase of axial strain was constant for his slow tests, whereas it was larger for the quick test. The rate became identical at the axial strain of 0.03%, five hours after he stopped increasing the axial stress. This led him to infer that considerable pore pressure gradient arose in the quick test. After the tests he measured the profile of water content of the specimens by slicing them into five segments. He found that the water content at the centre was about 2% lower than that at the end. This might imply that the flow of water took place from the centre towards the ends of the specimen.

Whitman (1960) observed the similar profile of water content in a triaxial specimen. He concluded that non uniformity in strain in the triaxial specimen, larger in the middle and smaller at the ends, was responsible for the local variation in water content. He also obtained interesting relationships between pore pressures and axial strains in undrained triaxial tests. The pore pressure at the centre was higher than that at the ends and the pore pressure at the ends was higher in slow tests than in quick tests. Bishop et al. (1960) measured the pore pressure in the specimens of compacted shale. The strain rates they employed were 2.5%/h and 0.17%/h. In both tests they observed the higher pore pressure at the base of the specimens. The difference in pore pressure at the centre and at the ends of the specimen was larger for the test with the former strain rate. They pointed out that the pore pressure gradient might arise due to a tendency to narrow zone failure in some samples.

Henkel (1960) also referred to narrow failure zone. He indicated that in undisturbed samples failure was likely to take place along a thin band and stressed the importance of employing low strain rates in order to achieve reasonable equalization of pore pressure in triaxial specimens. Crawford (1963) attempted to measure the pore pressure inside the triaxial specimen of Leda clay by using hypodermic needles. He conducted two types of triaxial tests; one with constant strain rate of 0.5%/h, and the other changing the rate from 0.5%/h to 2.5%/h at 1.0% axial strain. In the former test there was a tendency for the pore pressure at the centre to lag slightly behind the pore pressure in the lower quarter. The difference was of greater significance at low strains than at failure. The other test showed that the more rapid loading increased the pore pressure gradient from centre to base. As Crawford himself admitted, these observations cannot explain the higher water content at the ends of the triaxial specimens measured by Crawford (1959).

A research work similar to that of Crawford (1963) was conducted by Akai (1965). He measured the average pore pressure in a triaxial specimen by using a thin needle with many holes and compared it with the pore pressure at the base. The pore pressure at the base was considerably higher than the average pore pressure and the difference became negligible at high strains. He observed the highest pore pressure at the midpoint between the centre and the end of the specimen.

Blight (1963 a) presented an interesting discussion on the pore pressure distribution in triaxial specimens. Combining Filon's solution on an elastic cylinder with assumed variations of pore pressure coefficient A, he demonstrated that practically all sorts of distributions could be produced. He confirmed this half-theoretical deduction by conducting pore pressure measurements in triaxial specimens. He showed that at low strains the pore pressure at the centre was greater than at the ends, while at high strains this profile was reversed.

It seems to be almost certain from research works performed so far that nonuniform pore pressures arise in triaxial specimens even in undrained conditions. Blight (1963 b) emphasized the importance of measuring the accurate pore pressure in a triaxial specimen and proposed the use of a suitable test duration calculated from a desired degree of equalization of pore pressure in the specimen. He gave an equation for calculating the test duration for achieving 95% equalization as a function of the height of the specimen and the coefficient of consolidation. Sekiguchi et al. (1981) conducted finite element analysis of partially-drained and undrained triaxial tests. As their undrained analysis was for perfectly smooth platens, nonuniform pore pressure was not derived.

The soil mechanics group of Tokyo Institute of Technology has been engaged in comprehensive researches on intermediate soils these couple of years (Nakase et al., 1978; Kimura et al., 1982). In the course of various experiments it was strongly felt that the suitable test durations must be determined for each cohesive soil with different plasticity index. In the present paper the authors attempt to establish suitable strain rates for undrained triaxial tests on various cohesive soils normally consolidated in K_0 condition on the basis of pore pressure measured by small transducers at the centre, at the periphery and the base of triaxial specimens.

EXPERIMENTS AND THE RESULTS

Three types of cohesive soils were used in experiments. One was Kawasaki clay and the other two were artificial mixtures called T. I. T. mixture (Nakase et al., 1978; Kimura et al., 1982; Nakase et al., 1983). Physical properties of the three cohesive soils are given in Table 1, where they are named M 30, M 20 and M 10 refering to their plasticity indices (I_p). Their grading curves are shown in Fig. 1. Soil was mixed with water into slurry at the water content of 70% for M30, 60% for M20 and 40% for M10 and consolidated under the preconsolidation pressure of 69 kN/m^2 in a consolidometer with the diameter of 200 mm and the height of 270 mm. A triaxial specimen with 150 mm in height and 75 mm in diameter was trimmed from the preconsolidated cake. The mixture

Table 1. Physical properties of soils tested

Properties \ Soil		M 30	M 20	M 10
Specific Gravity of Soil Particles		2.69	2.68	2.67
Liquid Limit (LL)		55.3	42.7	27.6
Plastic Limit (PL)		25.9	23.3	16.9
Plasticity Index (I_p)		29.4	19.4	10.7
Fraction of Soil Components (%)	Sand	16.1	34.2	60.4
	Silt	61.6	48.7	29.9
	Clay	22.3	17.2	9.7
Amount of Toyoura Sand added (weight %)		0	0.23	0.57
Fraction of Particles finer than 2μm (%)		14.8	11.4	6.4
Coefficient of Consolidation c_v (cm^2/min) determined at $\sigma_{3c}'=196$kN/m^2; all around drainage		1.13×10^{-2}	1.69×10^{-2}	3.38×10^{-2}

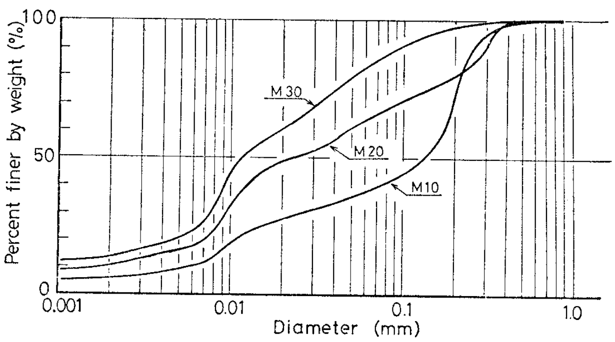


Fig. 1. Particle size distribution of Kawasaki clay, M 30 and T. I. T. mixtures, M 20 and M 10

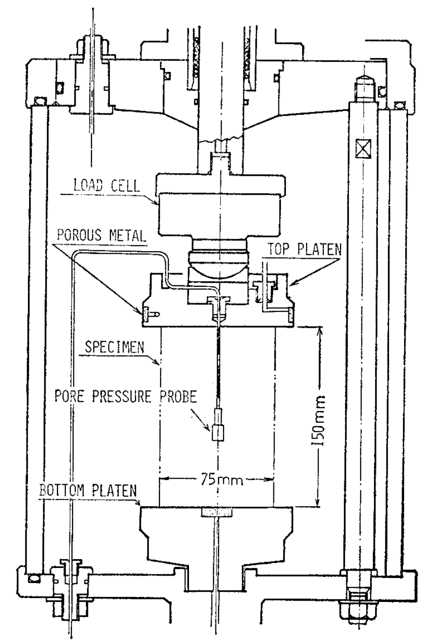


Fig. 2. Triaxial specimen and pore pressure transducer

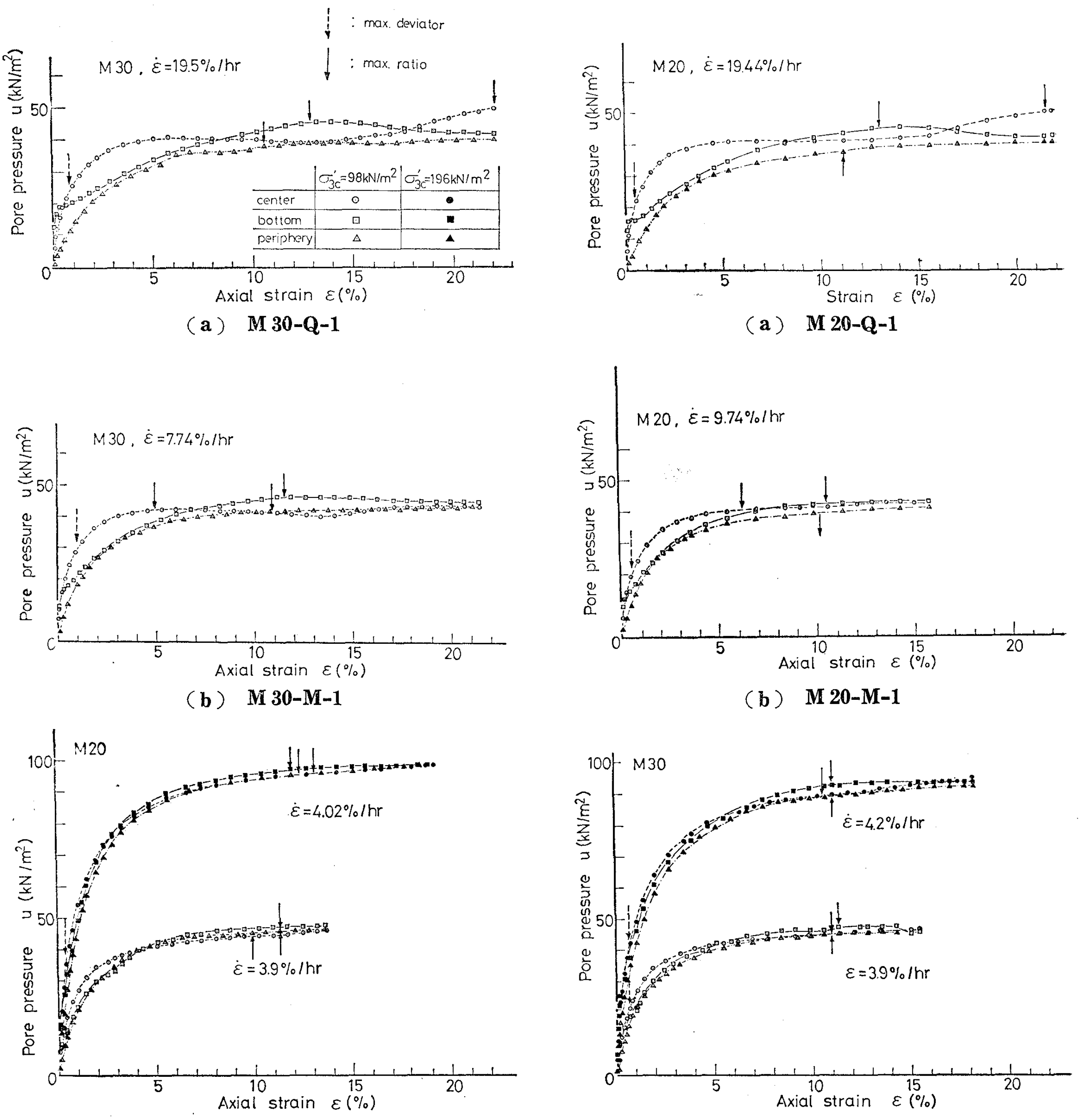
of silicon greese with silicon oil was smeared to the top and bottom platens of a triaxial apparatus and a latex disc was placed on each platen in order to lubricate the surface of contact between soil and metal. After placing the specimen on the bottom platen side drains consisting of six strips of filter paper strengthened with paper towel were attached to the surface of the specimen. The top end of each strip was extended to the side of the top platen so that the pore pressure at the periphery of the specimen could be measured with a pore pressure transducer connected to a porous metal in the top platen (Fig. 2). Subsequently a vertical hole was bored with a small auger along the axis of the specimen and a pore pressure transducer of silicone chip type, 6.35 mm in diameter and 13 mm in length, was installed at the centre as in Fig. 2. The detail of installation technique was described elsewhere (Kimura et al., 1982). The pore pressure measurement was also carried out at the bottom of the specimen. The pore pressures at the periphery and the bottom were monitored with pressure transducers of strain gauge type.

After introducing the back pressure of 196 kN/m^2 into the specimen and the triaxial cell the cell pressure was increased to the predetermined level, either 98 or 196 kN/m^2, and at the same time the axial load was applied to achieve K_0 consolidation. The loading rate was adjusted so as not to cause the lateral strains at the periphery of the midpoint of the specimen. The lateral strains were monitored with the lateral strain indicator of Bishop type.

Undrained shear tests were conducted on these specimens under four different strain rates. The strain rates were selected so as to cover the range of the strain rates recommended by the Japanese Society of Soil Mechanics and Foundation Engineering (Japanese Society of Soil Mechanics and Foundation Engineering, 1979). The tests are tentatively classified into four categories depending upon the strain rates; very quick, quick, moderate and slow. The test conditions are shown in Table 2. The approximate strain rate for each type of test is 37, 19, 8 and 4%/h. The middle symbol in the name of specimens as VQ in M 30-VQ-1 stands for the class of the test. The observed relationships between the pore pressures and the axial strains for each test are given in Fig. 3. Points of failure determined from the maximum deviator stress $[(\sigma_1-\sigma_3)_{max}]$ and from the maximum effective stress ratio $[(\sigma_1'/\sigma_3')_{max}]$ are marked in the figures. Strains at these particular points are termed strains at failure (ε_f) for the sake of con-

Table 2. Conditions and results of triaxial tests

Specimens		Consolidation pressure and K_0 $\sigma_{3c}'/\sigma_{1c}'=K_0$	Strain rate $\dot{\varepsilon}$ (%/h)	Strain at failure and angle of shearing resistance $[\varepsilon_f(\%)]$ $[\phi'(°)]$							Ratio of undrained strength to the vertical consolidation pressure c_u/σ_{1c}'
				ε_f : max deviator stress $[(\sigma_1-\sigma_3)_{max}]$	ε_f : max stress ratio$[(\sigma_1'/\sigma_3')_{max}]$						
					Centre		Bottom		Periphery		
					ε_f	ϕ'	ε_f	ϕ'	ε_f	ϕ'	
M 30	M 30-Q-1	1.000/2.333=0.429	19.5	0.86	22.05	38.02	12.82	39.20	10.61	36.58	0.404
	M 30-M-1	1.000/2.335=0.428	7.74	1.01	4.94	39.24	11.55	40.41	10.89	37.95	0.392
	M 30-M-2	1.000/2.270=0.440	7.62	0.82	15.87	39.72	11.43	39.52	11.07	37.57	0.381
	M 30-S-1	1.000/2.189=0.457	3.90	0.66	11.03	36.73	11.36	37.64	11.03	36.30	0.377
	M 30-S-2	2.000/4.511=0.443	4.20	0.76	10.69	36.39	10.69	37.47	11.08	36.16	0.385
M 20	M 20-Q-1	1.000/2.388=0.419	19.4	0.59	21.48	39.42	13.02	39.94	12.65	37.16	0.404
	M 20-M-1	1.000/2.463=0.406	7.74	0.51	6.24	38.39	10.65	39.05	10.28	37.59	0.390
	M 20-S-1	1.000/2.344=0.427	3.90	0.44	9.89	36.97	11.36	37.87	11.36	36.40	0.374
	M 20-S-2	2.000/4.761=0.420	4.02	0.45	13.15	37.30	12.00	37.88	12.38	36.83	0.366
M 10	M 10-VQ-1	1.000/2.350=0.426	37.4	0.29	21.02	38.02	12.53	40.42	17.48	37.88	0.382
	M 10-VQ-2	1.000/2.296=0.436	37.1	0.21	18.81	36.02	13.54	38.35	11.79	36.09	0.375
	M 10-Q-1	1.000/2.412=0.415	18.8	0.22	21.13	37.20	12.24	38.54	13.66	37.14	0.382
	M 10-S-1	1.000/2.385=0.419	3.72	0.18	13.71	37.46	11.59	38.17	11.94	36.93	0.356
	M 10-S-2	2.000/4.838=0.413	3.90	0.20	12.16	37.09	12.16	37.29	14.73	36.58	0.355



(a) M 30-Q-1 (a) M 20-Q-1

(b) M 30-M-1 (b) M 20-M-1

(c) M 30-S-1 and S-2 (c) M 20-S-1 and S-2

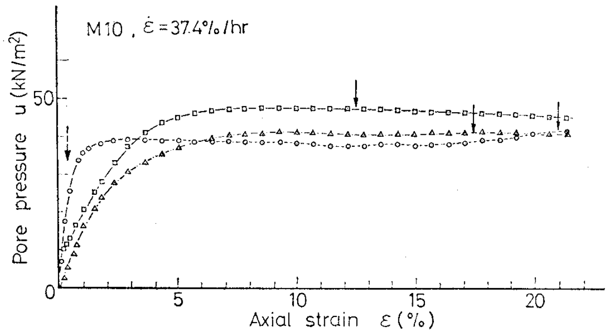
I M 30 II M 20

Fig. 3. Relationship between pore pressure and axial

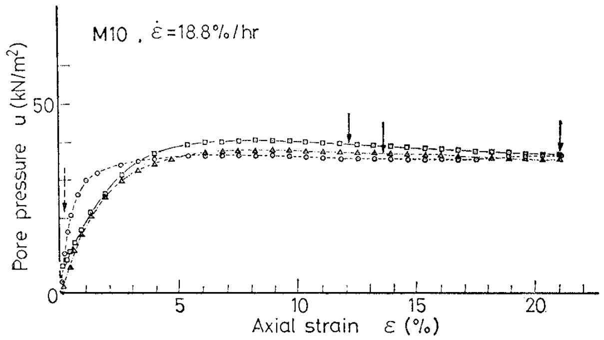
venience. Fig. 4 shows stress paths during the tests. They were derived from pore pressures measured at the three locations of the specimen.

DISCUSSION OF THE RESULTS

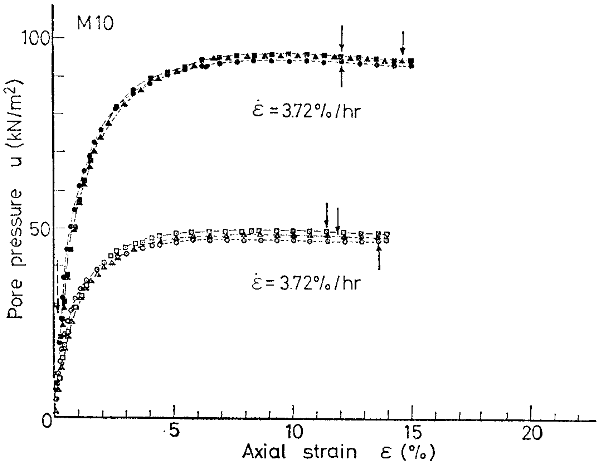
As is shown in Table 2, the magnitude of strain at the maximum deviator stress is approximately 0.2~1.0% for the soils tested, which is by far smaller than that at the maximum effective stress ratio. As the plasticity index of the soils decreases, the strain at the maximum deviator stress de-



(a) **M 10-VQ-1**



(b) **M 10-Q-1**



(c) **M 10-S-1 and S-2**

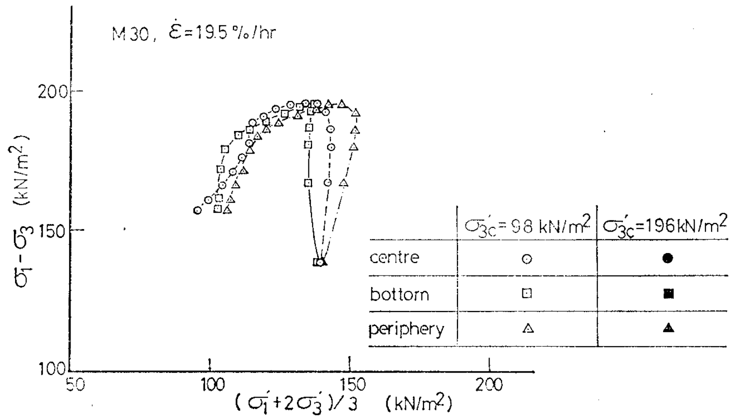
III M 10

strain

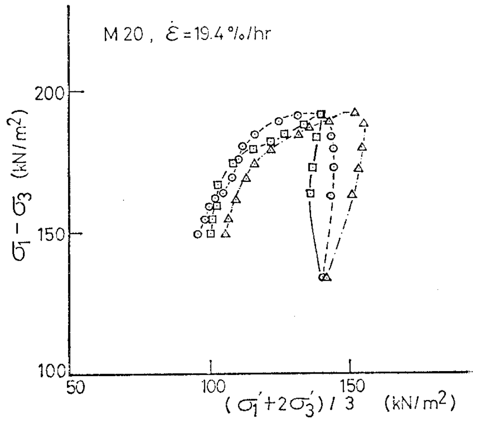
creases. This tendency can be explained by the fact that as the plasticity index of soils reduces their properties approach those of sand. The strains at the maximum effective stress ratio at the bottom and periphery of the specimen are practically identical in the quick tests. The strain at the centre is much larger. It can be seen, comparing Figs. 3 I(c), II(c) and III(c), that the difference in strain at the maximum effective stress ratio in the slow tests is greater for the soils with lower plasticity index. The difference is very small for M 30.

The relationships between pore pressures and axial strains given in Fig. 3 show that as the strain rate reduces the difference in pore pressures at three locations becomes smaller. In the quick tests the pore pressure at the cente of the specimen is considerably higher than that at the bottom and periphery at low strains, while at high strains the difference tends to become less significant. In M 10 the difference is only marginal. The pore pressure difference in the slow tests is negligible for M 10 for the whole range of axial strains employed in present experiments, whereas in M 20 as well as M 30 the slight difference as in the quick tests appears at low strains. It may be concluded that the pore pressure is quite uniform in M 10 under the slow tests. This can be accounted for by the fact that the consolidation coefficient c_v is larger for the soils with lower plasticity index. It has to be pointed out, however, that uniformity in pore pressure does not necessarily mean uniformity in total stress in the specimen. Although there is practically no difference in pore pressures at the different locations of the specimen of M 10 in the slow tests, the difference in strain at the maximum effective stress ratio is greater than that for M 20 and M 30. This implies that, although pore pressures appear to be uniform throughout the specimen, the stress may not be uniform. At present there seems to be no suitable method available to measure accurately the total stress inside triaxial specimens.

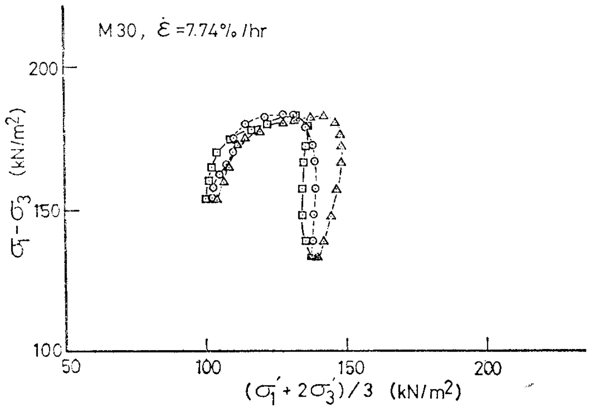
The relationships between the pore pressures and the axial strains for M 20 and M 30 are strikingly similar. This may lead to the conclusion that M 20 and M 30 have very similar mechanical properties, which has been confirmed in other series of the tests by the soil mechanics group of Tokyo Institute of Technology (Nakase et al., 1983). The



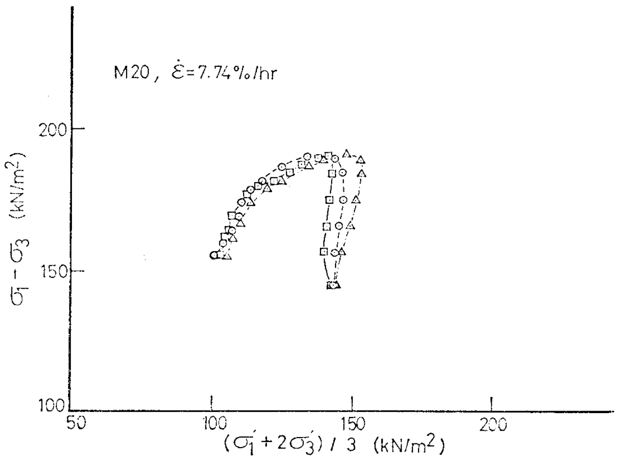
(a) M 30-Q-1



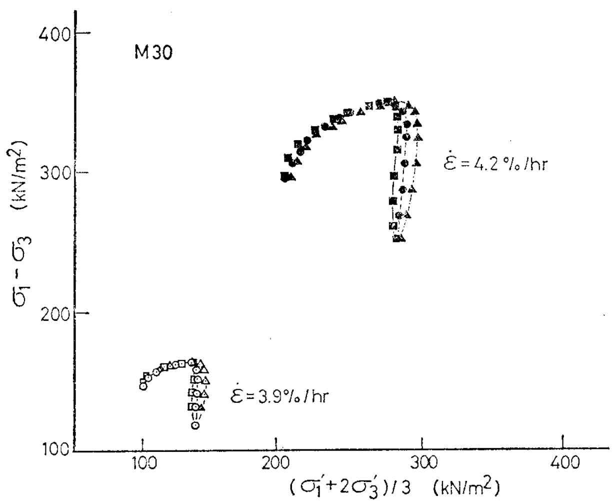
(a) M 20-Q-1



(b) M 30-M-1

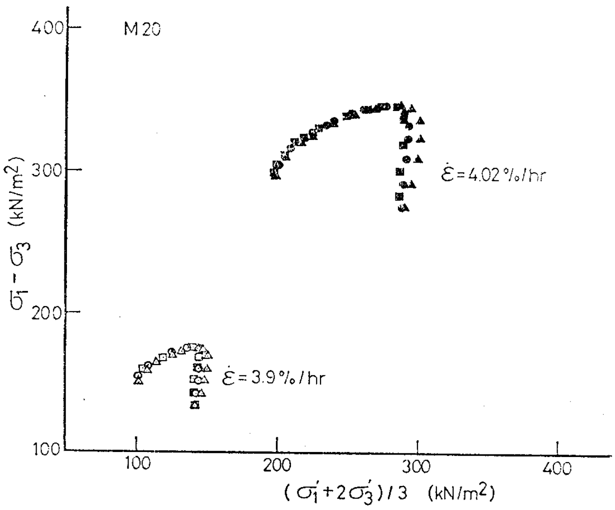


(b) M 20-M-1



(c) M 30-S-1 and S-2

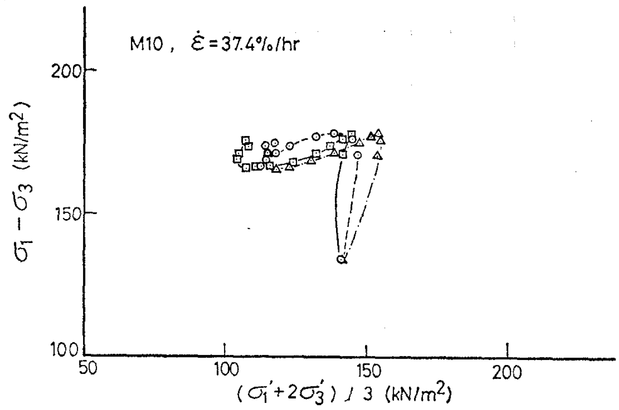
I M 30



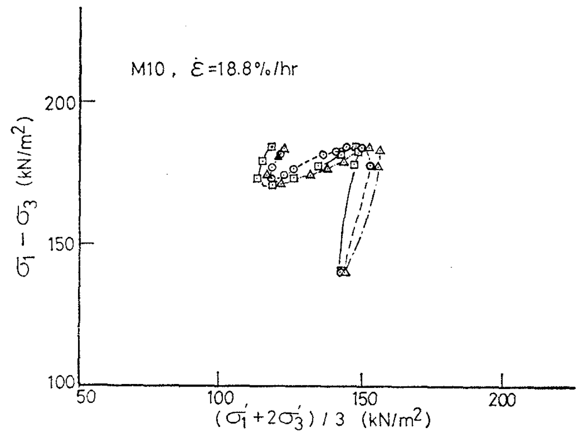
(c) M 20-S-1 and S-2

II M 20

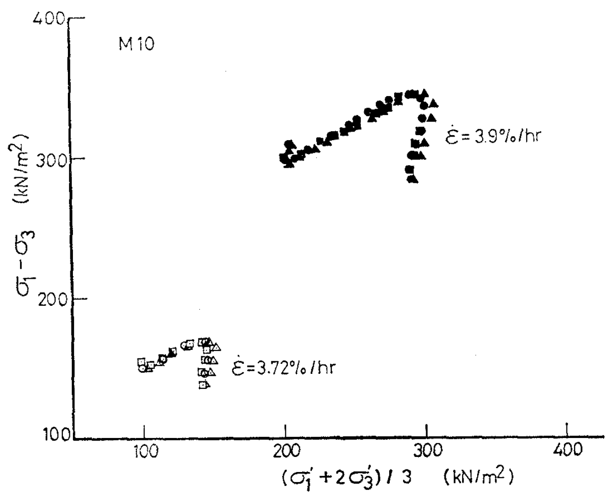
Fig. 4. Effective stress path



(a) M 10-VQ-1



(b) M 10-Q-1



(c) M 10-S-1 and S-2

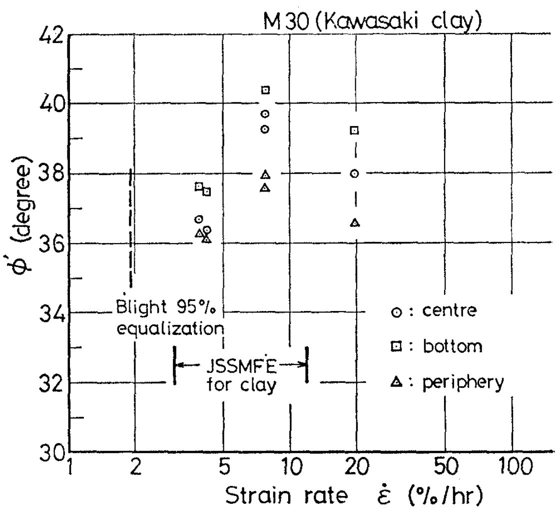
III M 10

fact that the coefficient of consolidation for the two types of soil is very similar is considered to be responsible.

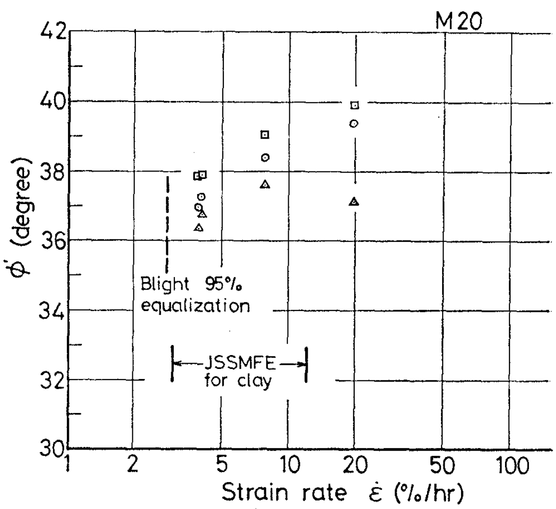
The same applies to the effective stress paths shown in Fig. 4. The stress paths for M 20 and M 30 are alike and they are very different from those for M 10. The former paths have only one inflection point with gentle curvature, while the latter have two with rather large curvature.

The difference in the stress paths at the three different locations in the slow tests is marked at the initial stage but it is less conspicuous at the later stage. The difference is not significant for M 10.

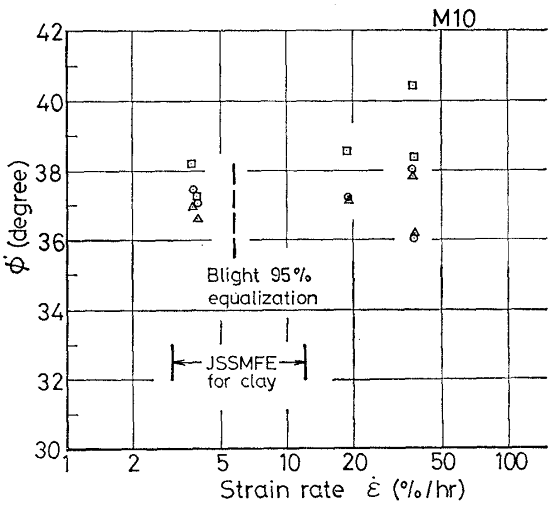
The relationships between the angle of shearing resistance in terms of effective



(a) M 30 (Kawasaki clay)



(b) M 20



(c) M 10

Fig. 5. Relationship between angle of shearing resistance in terms of effective stress (ϕ') and strain rate ($\dot{\varepsilon}$)

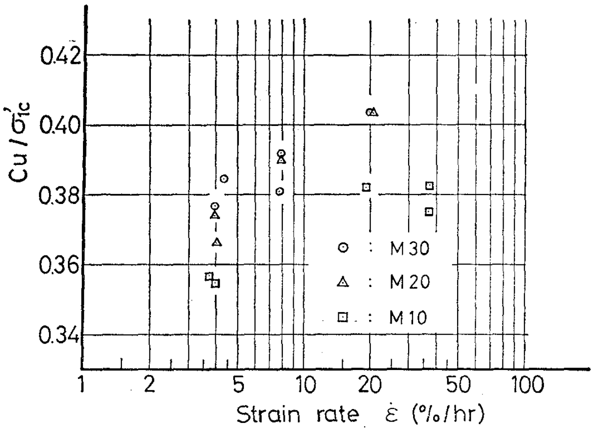


Fig. 6. Relationship between the ratio of undrained strength to vertical consolidation pressure (c_u/σ_{1c}') and strain rate ($\dot{\varepsilon}$)

stress ϕ' and strain rates are shown in Fig. 5. The values of ϕ' were calculated from the maximum effective stress ratio. It can be seen that the difference in ϕ' values calculated from the effective stress at the three different locations becomes smaller as the strain rate decreases. In the very quick and quick tests the difference is rather significant, whereas the difference in the slow tests is considered to be not very important from the engineering viewpoint. In Fig. 5 the recommendation by the Japanese Society of Soil Mechanics and Foundation Engineering is shown. Although this looks generally acceptable, the use of the strain rates in the lower limit of the recommended band is encouraged.

Blight (1963 b) produced a convenient chart for evaluating the degree of equalization of pore pressure in triaxial specimens in undrained tests. He gave an equation for calculating the time to reach failure (t_f) for pore pressure equalization of 95% in undrained tests as

$$t_f = 0.07\frac{H^2}{c_v} \qquad (1)$$

where H is half the specimen height. The strain rates corresponding to t_f were calculated assuming that the strain at failure was approximately 11%. They are shown in Fig. 5 by broken lines. The strain rate for the slow tests in the present experiments is quite close to that for 95% equalization. The lower limit of recommendation by the Japanese Society is also tantamount to the Blights' strain rates. The strain rate by Blight for M 20 and M 30 is slightly smaller than that in the slow tests in the present experiments. The tendency in Fig. 5(a) and (b) implies that the use of Blight's strain rate results in more uniform pore pressure in the specimen. It is concluded that the Eq. (1) gives very reasonable strain rates for undrained triaxial tests.

Fig. 6 shows a relationship between the averaged ratios of undrained strength to the vertical consolidation pressure c_u/σ_{1c}' and the strain rates. The undrained strength is for maximum deviator stress. The ratio c_u/σ_{1c}' increases with the increase of the strain rate, which agrees with the results of experiments by other research workers (Berre and Bjerrm, 1973). The relationships shown in the figure may lead to a conclusion that the rate of increase of the c_u/σ_{1c}' ratio with the strain rate is less for soils with lower plasticity index. In Fig. 6 again the tendency that M 30 and M 20 are very similar in their mechanical properties can be seen.

In the present research the pore pressure

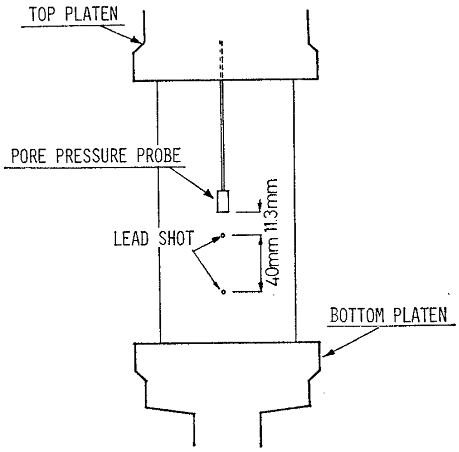


Fig. 7. Location of embedded lead shot for radiograph and transducer

Table 3. Comparison of strains: overall and local

Overall strains of specimen (%)	Local strains beneath transducer : X-ray measurement (%)
0	0
0.636	0
2.019	2.507
4.094	4.776
6.237	6.746
8.312	8.746
10.317	11.520
12.392	14.000
14.466*	16.269
16.576	18.746
18.612	22.000

* Failure strain 15.466%

at the centre of the triaxial specimen was measured by the small transducer embedded in soil. The authors have received some criticism on this method; the strain of soil just beneath the transducer might become very large with the transducer being pushed into soil and as the result uniformity of the strains in the specimen might be hampered. In order to investigate this the authors carried out additional undrained triaxial tests. After completing trimming the specimen lead shot was pushed into the specimen by using a thin pusher. The final arrangement of the embedded lead shot is shown in Fig. 7. Radiographs were taken at different stages of the tests. The coordinates of the image of the lead shot on the radiographs were read off by an X-Y analyzer and they were converted into strains by comparing them with the initial coordinates. Although this series of experiments have not been satisfactory, one of the results is shown in Table 3. The local and overall strains agree farelly well up to the strain of about 9%. The difference between the two becomes larger as the strain increases.

CONCLUSION

The following conclusions are drawn from the present research work on consolidated undrained triaxial tests of cohesive soils with different plasticity index under different strain rates.

(1) Concerning the strain at the maximum effective stress ratio, the magnitude at the centre of a specimen is much larger than that at the bottom and periphery.

(2) At low strains the pore pressure at the centre is larger than that at the bottom and periphery, while at high strains the difference is less significant.

(3) As the strain rate reduces the difference in pore pressures at the three different locations becomes smaller. In the tests with the strain rate of about 4%/h the difference in pore pressures at the three different locations is negligible for M 10, whereas for M 20 and M 30 some difference appears at low strains.

(4) The effective stress paths for M 10 are different from those for M 20 and M 30. The latter have one inflection point with gentle curvature, while the former have two inflection points with large curvature. The difference in stress paths for the three different locations is more marked at the initial stage of the tests than the final stage.

(5) The difference in angle of shearing resistance ϕ' calculated from the maximum effective stress ratio at the three different locations becomes smaller as the strain rate decreases. The difference can be neglected in the tests with the strain rate of about 4%/h.

(6) In order to achieve reasonable equalization in pore pressures in undrained tests,

the strain rates in the lower limit of the recommendation by the Japanese Society of Soil Mechanics and Foundation Engineering shoud be employed.

7) The time to failure proposed by Blight is considered to be more reasonable to determine the test duration in undrained tests.

(8) The rate of increase of the ratio of the undrained strength to vertical consolidation pressure with the strain rates is less for soils with lower plasticity index.

REFERENCES

1) Akai, K. (1965) : "Problems of pore pressure measurement in triaxial tests," Procs. 10th Soil Mechanics Symposium, Japanese Society of Soil Mechanics and Foundation Engineering, pp. 125-131 (in Japanese).
2) Berre, T. and Bjerrum, L. (1973) : "Shear strength of normally consolidated clays," Procs. 8th International Conference SMFE (Moscow), Vol. 1-1, pp. 39-49.
3) Bishop, A. W., Alpan, I., Blight, G. E. and Donald, I. B. (1960) : "Factors controlling the strength of partly saturated cohesive soils," ASCE Res. Conf. Shear Strength of Cohesive Soils, Colorado, pp. 503-532.
4) Blight, G. E. (1963 a) : Discussion on "Pore pressures within soil specimens in triaxial compression," by Crawford, C. B., Symposium on Laboratory Shear Testing of Soil, ASTM Special Technical Publication No. 361, pp. 199-204.
5) Blight, G. E. (1963 b) : "The effect of nonuniform pore pressures on laboratory measurements of the shear strength of soils," Symposium on Laboratory Shear Testing of Soil, ASTM Special Technical Publication No. 361, pp. 173-184.
6) Crawford, C. B. (1959) : "The influence of rate of strain of effective stresses in sensitive clay," Symposium on Time Rates of Loading in Soil Testing, ASTM Special Technical Publication No. 254, pp. 36-48.
7) Crawford, C. B. (1963) : "Pore pressures within soil specimens in triaxial compression," Symposium on Laboratory shear Testing of Soils, ASTM Special Technical Publication No. 361, pp. 377-386.
8) Henkel, D. J. (1960) : "The shear strength of saturated remoulded clays," ASCE Res. Conf. Shear Strength of Cohesive Soils, Colorado, pp. 533-545.
9) The Japanese Society of Soil Mechanics and Foundation Engineering (1979) : Soil Testing Manual, p. 525.
10) Kimura, T. and Saitoh, K. (1982) : "The influence of disturbance due to sample preparation on the undrained strength of saturated cohesive soil," Soils and Foundations, Vol. 22, No. 4, pp. 109-120.
11) Nakase, A., Nakanodo, H. and Kusakabe, O. (1978) : "Influence of soil type on pore pressure response to cyclic loading," Proceedings of the 5th Japan Earthquake Engineering Symposium, pp. 593-600.
12) Nakase, A. and Kamei, T. (1983) : "Undrained shear strength anisotropy of normally consolidated cohesive soils," Soils and Foundations, Vol. 23, No. 1, pp. 91-101.
13) Sekiguchi, H., Nishida, Y. and Kanai, F. (1981) : "Analysis of partially-drained triaxial testing of clay," Soils and Foundations, Vol. 21, No. 3, pp. 53-66.
14) Whitman, R. V. (1960) : "Some considerations and data regarding the shear strength of clays," ASCE Res. Conf. Strength of Cohesive Soils, Colorado, pp. 581-614.