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by
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SOME CHARACTERISTICS OF WINNIPEG CLAY

CARL B. CRAWFORD*

ABSTRACT

Compressibility and strength characteristics of specimens cut from a fairly uniform chunk sample of highly plastic Winnipeg clay are reported. A high cohesion intercept and a low friction angle in terms of effective stresses are confirmed by both undrained and special drained tests. The influence of swelling is shown to be an important factor in interpreting the natural strength in the usual working range of effective stresses.

SOMMAIRE

La compressibilité et la résistance d'échantillons, obtenus d'un bloc intact d'argile passablement uniforme et très plastique de Winnipeg, sont exposées. Des essais non drainés et des essais spéciaux drainés ont confirmé, en termes de tensions efficaces, une cohésion élevée et un petit angle de frottement. L'influence du gonflement est vue comme facteur important dans l'interprétation de la résistance naturelle dans le domaine habituel des tensions efficaces.

The opportunity for obtaining good undisturbed block samples of Winnipeg clay arose with the excavation of a test trench along the proposed right of way of the Red River Floodway. The results of extensive tests carried out on some of these samples by the PFRA were summarized by J. Mishtak (1964). Of the several block samples provided by the Water Resources Branch of the Manitoba Provincial Government to the Division of Building Research, NRC, one particularly uniform block was selected for special studies of the influence of variations in triaxial test procedure. This block of clay, from a depth of 30 ft., was obtained at the same location as those on which Mishtak reported. In reality the variability of the soil, both vertically and laterally, is sufficient that these results cannot be assumed to be generally representative of the area.

The tests reported here are intended to supplement those described by Mishtak and to promote discussion of the interpretation of shear strength parameters of a swelling clay. The block sample had a range of water contents from 51.4 to 57.8 per cent (average 53.9 per cent). The liquid limit averaged 94 per cent, the plastic limit 34 per cent, the specific gravity 2.76, and the grain size distribution was 83 per cent clay size (less than 0.02 mm.) and 17 per cent silt.

COMPRESSIBILITY

Three one-dimensional consolidation tests were performed on specimens enclosed in teflon-coated rings, 20 sq. cm. in area and 2 cm. high. Under an initial pressure of approximately 0.5 kg/sq. cm. the specimens tended to swell when water was added so the pressure was increased immediately to at least 0.75 kg/sq. cm. and this was sufficient to prevent swelling. The results of the three tests are given in Table I; Figure 1 shows a typical pressure-void ratio curve. The most probable preconsolidation pressure is 4.4 kg/sq. cm. (4.5 tons/sq. ft.).

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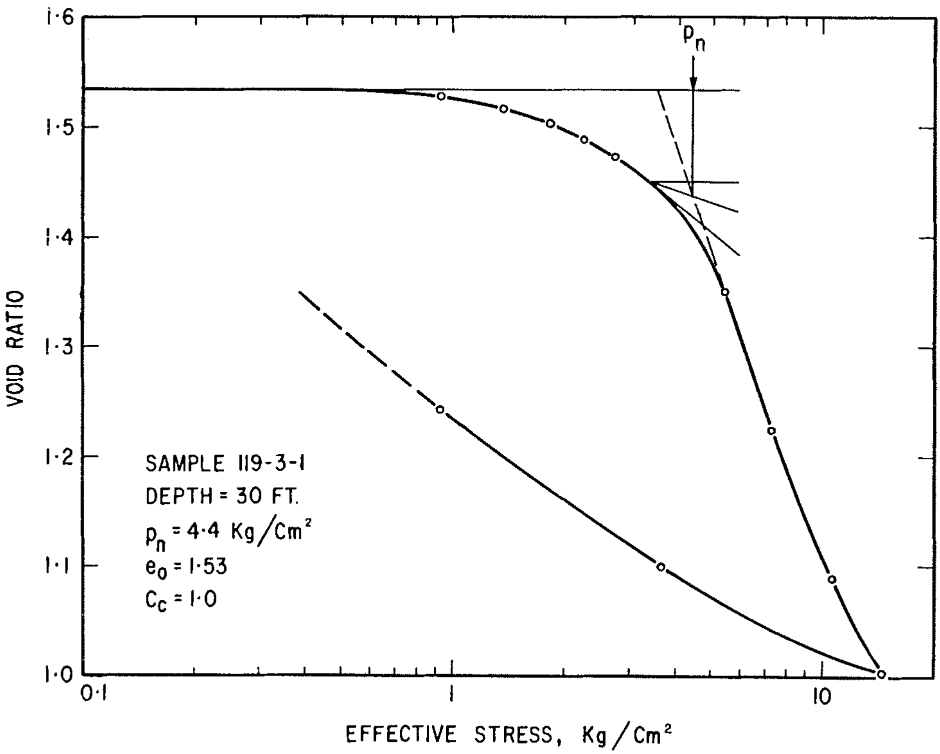


FIGURE 1. Typical pressure–void ratio curve

TABLE I
Consolidation Test Results

Specimen	Water content, %		Initial void ratio	Compression index	Most probable preconsolidation pressure, kg/sq. cm.
	Initial	Final			
119–3–1	54.9	54.2	1.53	1.0	4.4
119–3–2	57.2	56.0	1.58	1.1	4.4
119–3–3	57.8	54.1	1.61	1.0	4.0

STRENGTH TESTS

All strength tests were performed on cylindrical specimens approximately 8 cm. in height and 3.6 cm. in diameter.

Unconfined Compression and Unconsolidated Undrained Triaxial Tests

These tests, carried out at a rate of strain of 1%/min., gave consistent results as shown in Table II. The average of six tests shows the compressive strength to be 1.7 kg/sq. cm., the undrained shear strength 0.85 kg/sq. cm. The soil specimens had an average water content of 54.1 per cent. The failure strain in every case was just under 2 per cent.

TABLE II

Compression Test Results

Specimen	Water content, % Initial	Water content, % Final	Compressive strength $(\sigma_1 - \sigma_3)f$ kg/sq. cm	Pore pressure at failure $\Delta\mu f$ kg/sq. cm	Cell pressure σ^1_c kg/sq. cm	Strain at failure $\epsilon f\%$	Strain rate, %/hr.
			Unconfined compression				
119-3-4	53.5		1.70			1.9	60
119-3-5	54.6		1.69			1.9	60
119-3-6	53.9		1.76			1.9	60
			Unconsolidated-undrained triaxial				
119-3-7	54.5		1.63		3	1.9	60
119-3-8	55.5		1.59		5	1.7	60
119-3-9	52.7		1.73		7	1.9	60
			Consolidated-undrained triaxial				
119-3-10	53.0	50.6	2.01	1.35	3	2.5	2
119-3-11	51.4	46.1	2.43	2.21	5	4.0	2
119-3-12	51.9	42.1	2.95	2.85	7	5.0	2

Consolidated Undrained Tests

Three consolidated undrained tests were carried out at a rate of strain of 2%/hr. The results are listed in Table II and the stress-strain and pore water pressure relationships are shown in Figure 2. The Mohr stress circles at failure for these tests are shown in Figure 3. The failure envelope indicates a cohesion intercept in terms of effective stresses, $c' = 0.6$ kg/sq. cm. and a $\phi' = 9°$.

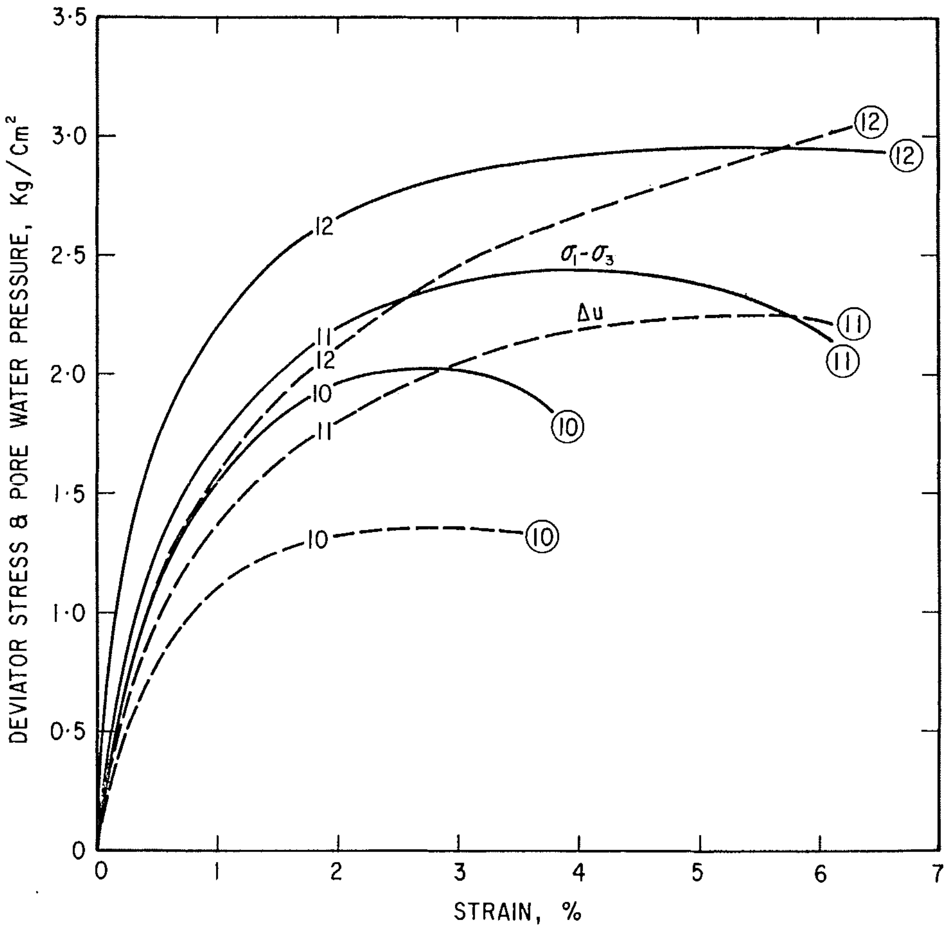


FIGURE 2. Relation between stress, pore water pressure, and strain for consolidated-undrained tests

Consolidated Drained Tests with Axial Stress Increasing

Consolidated-drained tests on four specimens strained at rates varying from ¼% to 2%/hr. are listed in Table III. Stress-strain curves for the four specimens are shown in Figure 4 up to a strain of 15 per cent. None of these tests was strained past 20 per cent and specimen 17 was strained only to 11 per cent. Some difficulty was experienced with specimen 17 and the result is therefore not averaged with the other three tests. On Figure 3, the average Mohr stress

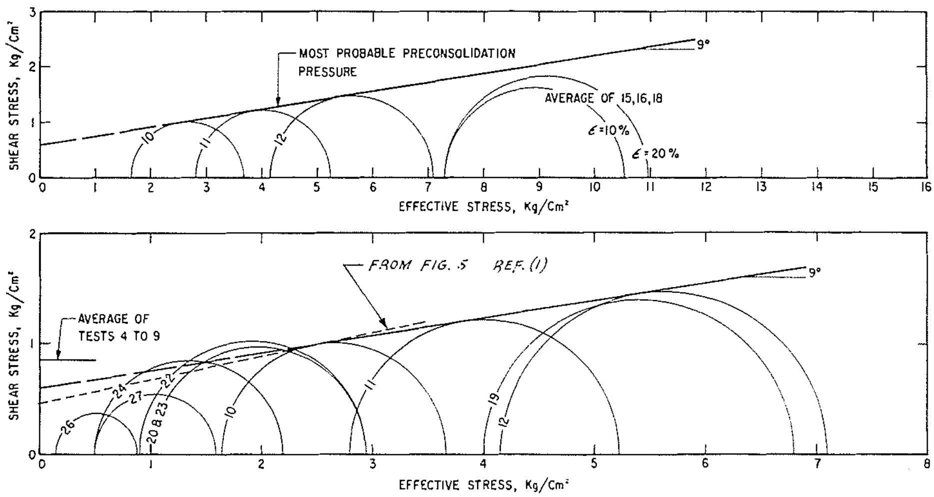


FIGURE 3. Stress circles at failure

circles at strains of 10 and 20 per cent are shown. The stress circle, even at 20 per cent strain, falls below the envelope for the consolidated undrained tests. This stress circle might have expanded to the envelope under further strain but it was noticed that at 20 per cent the specimen was severely distorted.

Speciments 26 and 27 (Table III) were completely immersed in distilled water for 4 hr. before testing. The ends were then retrimmed and each was weighed and measured before they were installed in the triaxial cell. Specimen 26 was then consolidated overnight under a cell pressure of 1 kg/sq. cm. and a back pressure on the pore water of 0.85 kg/sq. cm. Specimen 27 was consolidated for 3 days under a cell pressure of 1.4 kg/sq. cm. and a back pressure of 0.9 kg/sq. cm. Each specimen was then strained axially under drained conditions at 0.4 per cent per hr.

Consolidated Drained Tests with Lateral Stress Decreasing

The results of five of these controlled stress tests are shown in Table III. Test specimen 19 was consolidated under a cell pressure of 7.3 kg/sq. cm. The lateral stress was then decreased in decrements to 6, 5, 4.5 and 4 kg/sq. cm. while the vertical stress was maintained relatively constant except during the last decrement when it was dropped to 6.8 kg/sq. cm. This was not intentional but was the final computed axial stress taking into account area correction. The time-deflection curves for the specimen under each decrement are shown in Figure 5; the Mohr stress circles for each loading decrement are also shown inset in this figure in relation to the failure envelope from Figure 3. The failure stress circle falls slightly below the failure envelope.

Specimens 20, 22 and 23 were each consolidated under an effective stress of 3 kg/sq. cm. After consolidation, the lateral stress was decreased suddenly to 1 kg/sq. cm. while the vertical stress was held more or less constant. Sample 20 failed during the night at more than 6 hr. and less than 20 hr. after the deviator stress was applied. Specimen 23 failed $13\frac{1}{2}$ hr. after the deviator stress

TABLE III

Drained Compression Test Results

Specimen	Water content, % Initial	Water content, % End of consol.	Water content, % Final	$(\sigma_1 - \sigma_3)f$ kg/sq. cm.	Consolidation pressure kg/sq. cm.	$\sigma^1{}_{3f}$ kg/sq. cm.	$\sigma^1{}_{1f}$ kg/sq. cm.	Remarks
Consolidated–drained triaxial with axial stress increasing								
119–3–16	54.8	42.2	37.8	4.16	7.3	7.3	11.46	strain at ¼%/hr
119–3–18	54.1	43.5	40.3	3.27	7.3	7.3	10.57	strain at ½%/hr
119–3–15	53.8	43.3	41.5	3.57	7.3	7.3	10.87	strain at 1%/hr
119–3–17	54.0	42.7	41.7	3.02	7.3	7.3	10.32	strain at 2%/hr
119–3–26	53.3	—	57.3	0.71	0.15	0.15	0.86	3.3% strain at failure
119–3–27	53.3	56.6	54.8	1.08	0.5	0.5	1.58	3.9% strain at failure
Consolidated–drained triaxial with lateral stress decreasing								
119–3–19	53.5	42.9	42.5	2.8	7.3	4.0	6.8	
119–3–20	57.5	54.5	55.3	1.95	3.0	1.0	2.95	
119–3–22	52.8	53.0	51.2	2.05	3.0	0.9	2.95	
119–3–23	53.7	53.5	52.2	1.93	3.0	1.0	2.93	
119–3–24	54.8	54.5	54.3	1.69	1.5	0.5	2.19	

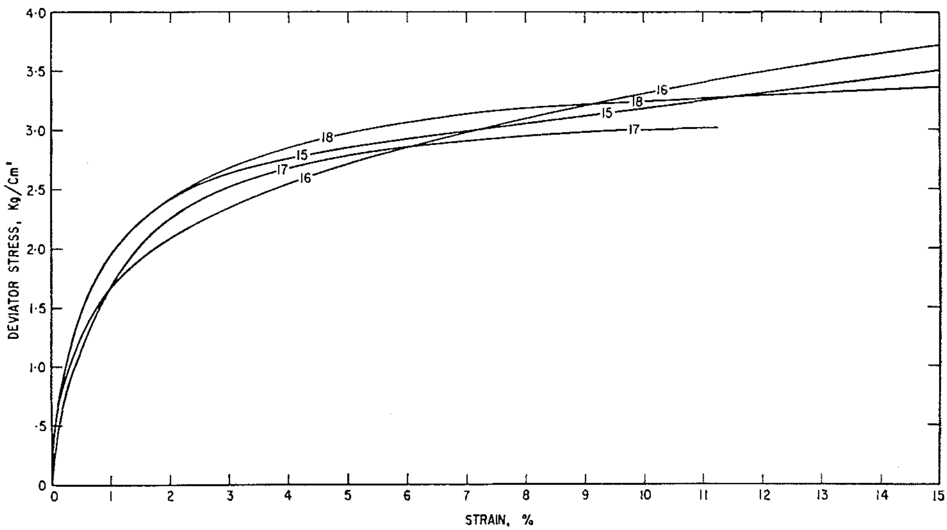
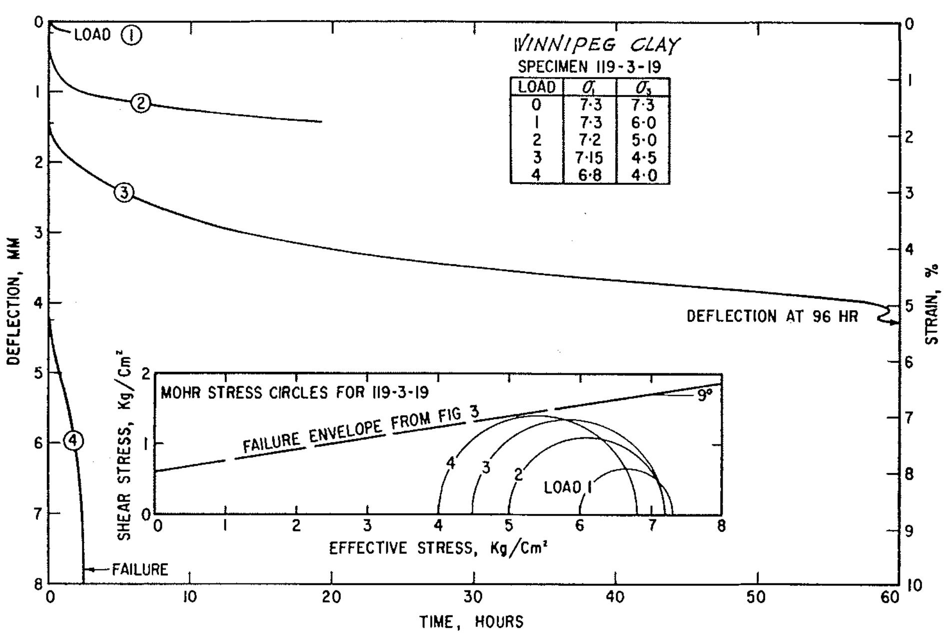


FIGURE 4. Stress-strain curves for drained tests—axial stress increasing



LOAD	σ_1	σ_3
0	7·3	7·3
1	7·3	6·0
2	7·2	5·0
3	7·15	4·5
4	6·8	4·0

FIGURE 5. Strain-time curves during drained test with σ_3 decreased in stages

was applied. Specimen 22 had not failed after $22\frac{1}{2}$ hr. and so the lateral stress was decreased to 0.8 kg/sq. cm. It then failed in 5 minutes. Each of these three specimens failed rapidly after about 2 per cent of axial strain.

Specimen 24 was consolidated under an effective stress of 1.5 kg/sq. cm. After consolidation, the axial stress was increased to 2 kg/sq. cm. and the lateral stress was decreased to 0.5 kg/sq. cm. Less than 1 per cent deflection occurred under this loading in 2 hr. The axial stress was then increased to 2.2 kg/sq. cm., the specimen deflected a further 1 per cent in 25 minutes and then failed suddenly.

Discussion of Results

All of the tests reported were made on specimens obtained from a depth of 30 ft. Most of the triaxial tests were made with cell pressures less than the preconsolidation pressure. This was done because the working effective stress range for most stability problems will be below 1.5 kg/sq. cm.

It is of interest that the Mohr failure envelope for consolidated-undrained tests ($c' = 0.6$ kg/sq. cm., $\phi' = 9°$) is confirmed to a reasonable degree by consolidated-drained tests in which failure is induced by decreasing lateral stress. In the low stress range the drained strength is slightly greater than the *CU* envelope but this can be attributed to precompression. The failure envelope is compared in Figure 3 with that reported by Mishtak ($c' = 6.5$ lb./sq. in. and $\phi' = 12°$, Figure 5).

One of the most important factors to be investigated is the influence of time or strain rate on strength. In an attempt to accelerate possible time-dependent softening, two specimens (26 and 27) were immersed for several hours before testing. As illustrated in Figure 3, this pretreatment caused a considerable reduction in strength under low stresses. These particular tests should be considered only to indicate the effect of stress release with unlimited water supply. Because the supply of soil from the block was exhausted, no further tests of this nature were possible.

Tests on another block of soil from the same depth but from the opposite side of the test trench had an average compressive strength of 1.4 kg/sq. cm. Specimens immersed in de-aired distilled water for 20 hr. increased quite uniformly in water content from 65 to 74 per cent. In similar specimens, tested after immersion, the compressive strength had decreased to about 0.4 kg/sq. cm.

Conclusions

The tests reported here indicate a slightly greater shear strength in the working stress range (both total and effective stresses) than those described by Mishtak. At higher effective stresses the opposite is true. The difference is probably due to variation in rates of loading.

If taken at face value these tests show an even greater discrepancy between laboratory values and the strength required for stable slopes as reported by Mishtak. His values were approximately double the required values for test trench stability and several times the required values for the long term stability of river banks.

A few tests have shown a substantial reduction in strength when the soil is allowed to swell. This feature imposes a restriction on the acceptability of tests carried out at effective stress levels greater than those expected in a practical problem and in which the soil element is not permitted to swell as it could in nature. Further tests of this nature are greatly needed.

Acknowledgment

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Reference

Mishtak, J., 1964. "Soil Mechanics Aspects of the Red River Floodway." This Journal *1*, no. 3: 133–146.