Deformation of Walls Retaining Soft Clay Backfills

By

G. K. Spencer, Ph.D., M.I.E.Aust.

(Engineer, Coffey and Hollingsworth, Melbourne)

AND

P. J. Moore, Sc.D., M.I.E.Aust.

(Senior Lecturer in Civil Engineering, University of Melbourne)

SUMMARY.- The inter-relationship between the deformation and the lateral restraint of a retaining wall has been investigated both experimentally and theoretically. A rigid model wall retaining a soft clay backfill was allowed to yield by reducing the lateral support on the wall. Observations were made of the amount and rate of lateral wall deformation. These observations indicated that the rate of wall movement was inversely proportional to the lateral restraining force and for the very soft clays the rate of wall movement finally became constant. Predictions of the wall deformation as a function of time were made by means of the rate process theory supported by data from a series of torsion shear tests. These theoretical predictions were in reasonable agreement with the observations.

I.- INTRODUCTION

The Civil Engineering Code of Practice (Ref.1) expressions for active earth pressure have been used extensively during the past twenty years for the design of rigid retaining structures supporting clay masses. In this conventional analysis it is assumed that the minimum or active earth pressures which are used for design exist after a finite outward wall movement. Following development of this active pressure distribution the wall movements were assumed to cease.

In situations in which yielding of the retaining wall is prevented it has been proposed (Refs. 2, 3,4) that the wall should be designed to resist the "at rest" lateral earth pressure. With a cohesive backfill, Taylor (Ref.3) concluded that it was reasonable to design a wall on the basis of active pressure if continuous movements can occur without serious consequences. He considered that the shearing stresses within a clay backfill undergo slow relaxation so that as the shear stress decreases the lateral pressure must gradually increase to maintain equilibrium. Tschebotarioff (Ref.4) and Vidmar (Ref. 5) have conducted model retaining wall tests which have produced results that confirm the shear stress relaxation concept presented by Taylor.

In order to examine quantitatively the inter-relationship between wall movement and the lateral force exerted by a cohesive backfill, a rigid model retaining wall was designed and built.

II.- NOTATION

P lateral restraining force applied to retaining wall
c_u undrained cohesion
τ_o relaxed shear strength
τ shear stress
c_w wall adhesion
t time
t_1 reference time
c_r residual cohesion
D creep deviator stress
$\dot{\epsilon}$ strain rate
$\dot{\delta}$ deformation rate
$\dot{\delta}_w$ wall displacement rate
A strain rate at time t_1
m negative slope of the plot between log of strain rate and log of time
α coefficient in rate process theory
θ inclination of failure plane to horizontal

III.- EXPERIMENTAL APPARATUS

The model retaining wall is shown in Figure 1. The bin containing the clay backfill was 18 inches wide, 20 inches long and 15 inches high. The kaolin soil in the bin was supported on one side by a moveable metal plate ¼ inch thick, 18 inches wide and 12 inches high. The retaining plate was permitted to move horizontally by a system of three roller bearings which roll in tracks between metal guides. During placement and consolidation of the backfill a horizontal proving ring connected between the retaining wall and a rigid frame provided lateral support and also a measure of the total earth pressure.

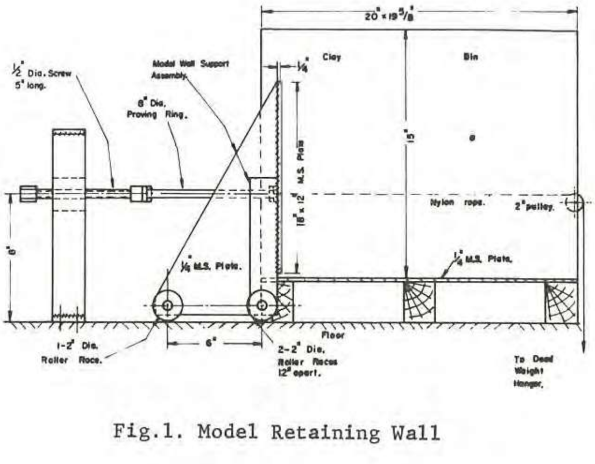

Fig.1. Model Retaining Wall

Prior to the wall moving, the proving ring restraint was replaced by dead weights via a pulley system from the retaining plate to the rear of the bin. Once this transfer of lateral support was achieved the proving ring was removed; wall movements were induced by removing a portion of the dead weights. A calibration test carried out on the wall system indicated that a frictional force of approximately ½ lb. exists as the wall is moved away from the backfill.

The horizontal displacements of the retaining wall were measured with two dial guages, which were independently fixed onto the rigid base of the bin having no contact with the proving ring. The backfill surface was surcharged with a uniformly distributed load. The settlements of the soil surface were measured with two vertical dial guages.

Determination of the shear stress-deformation characteristics of the clay backfill was obtained from torsion shear tests on remoulded Kaolin samples. The torsion shear apparatus was basically the same as that described by Hvorslev (Ref.6) with a few modifications made to simplify the construction. The apparatus permitted progressive shearing to occur during the test and the shearing resistance characteristics after initial failure, including the residual shearing resistance, to be fully explored.

IV.- EXPERIMENTAL TEST RESULTS

Two stress controlled tests involving the small retaining wall were conducted. In Test No.1 for which the post consolidation water content and unit weight were respectively 78% and 98 lb./ft.^{3}, the total lateral force in the restraining proving ring after consolidation of the backfill was 102 lb. To produce wall displacements, loads were removed from the dead weight hanger such that the restraining force on the wall was less than the equilibrium value of 102 lb. To achieve an appreciable lateral wall movement within a period of about 5 days, lateral loads of less than 60 lb. were used. The load required to resist the hydrostatic water pressure alone was calculated to be 44 lb. The wall displacement variations with time corresponding to loads of 56,54,52 and 48 lb. are shown in Figure 2. The rates of wall movement extracted from these plots and which are listed in Table I reveal two important features:

(a) after a period of approximately 2 days the wall movements for all levels of restraining force appear to be linear with time, and

(b) the rate of wall movement is inversely proportional to the lateral restraining force.

The aim of Test No.2 for which the post consolidation water content and unit weight were respectively 52% and 106 lb./ft.^{3}, was to verify the above conclusions for a clay backfill placed at a consistency which resembles prototype conditions. The total lateral force in the restraining proving ring after consolidation was 125 lb. The external load required to resist the hydrostatic water pressure alone was found to be 44 lb.

TABLE I - WALL DEFORMATION BEHAVIOUR

TEST NO.1 (backfill surcharge 0.41 lb/in.^{2})	
Lateral Restraining Force Applied to Retaining Wall P,(lb.)	Horizontal Rate of Wall Movement (in./day)
56	0.0035
54	0.008
52	0.012
48	0.020

TEST NO.2 (backfill surcharge 0.53 lb/in.^{2})					
Lateral Restraining Force Applied to Retaining Wall P,(lb.)	Horizontal Rate of Wall Movement (in./min.) x 10^{-6}				
	Day:	3	4	5	6
60		4.2	3.6	3.1	2.1
45		12.6	10.4	8.3	6.3

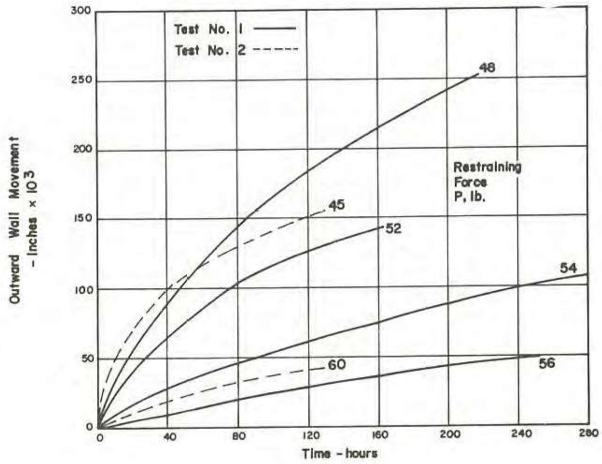

Fig.2. Retaining Wall Creep Tests

It was again found necessary to reduce the external loads such that an appreciable lateral wall movement was obtained within a 5 day period. The wall displacement variations with time corresponding to loads of 60 lb. and 45 lb. are shown in Figure 2. The plots reveal that for all levels of restraining force the rate of horizontal wall movement decreases with time. This result is contrary to that of Test No.1 in which a constant rate of wall creep was observed. The results of Test No.2 are tabulated in Table I.

A comparison of the test results suggests that the horizontal creep behaviour of the wall is a function of the stiffness of the backfill material, since all other variables are the same in both tests. In Test No.1, where a constant rate of wall movement was observed, the moisture content of the clay backfill was 78% which is just above liquid limit of the soil. In Test No.2, the moisture content was 52% and produced a decreasing rate of wall movement. This apparent discrepancy may be explained if in Test No.1 the wall movement is considered to be decreasing at an extremely slow rate. Therefore, it can be postulated that as the natural moisture content of a clay backfill decreases, the decrease in the rate of outward wall movement becomes more apparent.

In all retaining wall tests the undrained shear strength of the clay backfill after consolidation was determined by vane apparatus. These insitu values of the undrained cohesion, c_u were determined at various depths throughout the clay profile. The main advantage of this apparatus is that it permits values of shear strength to be obtained in extremely soft soils. The vane shear strength variation with depth corresponding to Test Nos.1 and 2 are shown in Table II.

In addition to the vane tests, strain-controlled torsion shear stage tests were conducted on kaolin samples duplicating the vertical effective stress and moisture content of the clay backfills. The values of the undrained shear strength, c_u and the residual shear strength, c_r from the torsion shear tests are also shown in Table II. During the tests it was observed that the undrained shear strength was compatible with a finite strain rate, since the shear stress decreased immediately deformation was terminated. It was noted that after all shearing stages, the shear strength of the sample relaxed and fell to a fixed level which was compatible with zero displacement. The shear stress corresponding to this level may be termed the "relaxed" shear strength, τ_o of the soil, the values of which are tabulated in Table II correspondi g to Tests Nos. 1 and 2.

In addition to the strain-controlled torsion shear tests, creep tests were performed on Kaolin samples corresponding to the stress conditions at the mid-depths of the backfills in Test Nos. 1 and 2. These were essentially stress-controlled tests in which a constant shear stress was applied to the samples and the deformation characteristics observed. The magnitude of the constant shear stress, τ used in the creep tests ranged from 50% to 88% of the drained peak shear strength. It was observed that creep did not occur if the applied shear stress was below the relaxed shear stress, τ_o . Typical results of the creep tests are provided in Figure 3. As expected, the creep behaviour is similar to that observed with the retaining wall. A tabulation of results for both series of creep tests is given in Table III.

TABLE II - SHEAR STRENGTH OF CLAY BACKFILL AFTER CONSOLIDATION

Test No.	Vane Tests	Torsion Shear Stage Tests		
	c_u (psi)	c_u (psi)	c_r (psi)	τ_o (psi)
1, 8" above base	0.19	-	-	-
6" above base	-	0.22	0.22	0.08
2" above base	0.23	-	-	-
2, 8" above base	0.23	-	-	-
6" above base	-	0.25	0.25	0.10
2" above base	0.27	-	-	-

TABLE III - TORSION SHEAR CREEP TEST

Creep Test No.	Applied Shear Stress (psi)	Observed Deformation Rate (in./min.)				
C.T.1	0.27	2.3×10^{-4}	Constant Creep Rate			
	0.22	5.6×10^{-6}				
	0.17	7.6×10^{-8}				
	0.15	7.6×10^{-9}				
		Day	2	5	7	Factor
C.T.2	0.30		34	20	18	$\times 10^{-6}$
	0.25		3.33	2.00	1.85	$\times 10^{-6}$
	0.20		0.30	0.18	0.17	$\times 10^{-6}$
	0.17		0.07	0.05	0.04	$\times 10^{-6}$

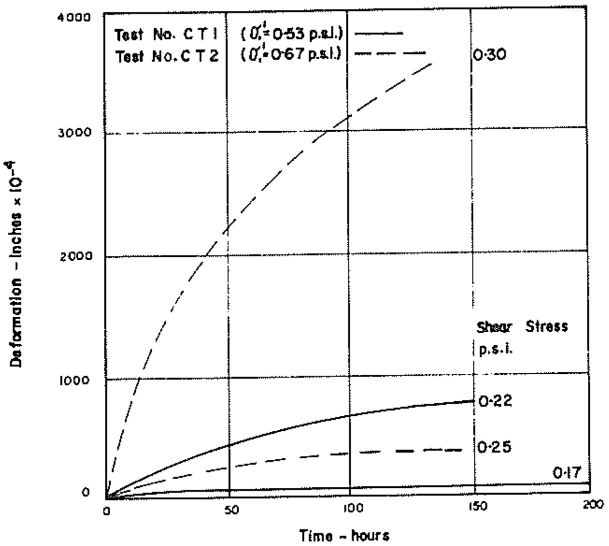

Fig.3. Torsion Shear Creep Tests

These results tend to confirm the earlier statement that the creep behaviour of a soil is dependant upon the stiffness of the material. It seems that an almost constant rate of creep can be expected for very soft soils, whereas for stiffer clays the creep rate has been observed to decrease with time.

V.- THEORETICAL ANALYSIS

To predict the deformation behaviour of retaining walls the theory of absolute reaction rates, commonly known as rate process theory (Ref.7) has been used to describe the creep behaviour of clay under stress. In particular Mitchell, Campanella and Singh (Ref.8) have adopted the principles of the rate process theory to determine a simple three parameter expression for the creep rate, under conditions of constant temperature and over a stress range of engineering interest. The expression has been derived using experimental test results and is given below:

$$\dot{\epsilon} = A.(t_1/t)^m . e^{\alpha.D} \qquad ...(1)$$

In its present form equation (1) is unsuitable as a means of predicting retaining wall behaviour from torsion shear creep tests. If the strain rate $\dot{\epsilon}$ is replaced by the deformation rate, $\dot{\delta}$ and the shear stress increment $(\tau-\tau_o)$ substituted for the deviator stress (D), then the creep equation becomes:

$$\dot{\delta} = A_1.(t_1/t)^{m_1} . e^{\alpha_1.(\tau-\tau_o)} \qquad ...(2)$$

Equation (2) can now be used to predict deformation rates from the results of torsion shear creep tests allowing values of the constant A_1, m_1 and α_1 to be found.

The creep equation derived above only gives the displacement rate along a failure plane within the soil mass which, according to the conventional analysis, passes through the base of the wall and slopes at an angle θ to the horizontal. Therefore the horizontal wall displacement rate, $\dot{\delta}_W$ can be determined from:

$$\dot{\delta}_W = \dot{\delta}.\cos\theta$$

where from the conventional analysis:

$$\theta = \arctan(1/\sqrt{1 + C_W/C_u})$$

If a typical value of C_W is assumed such that C_W/C_u is equal to 0.33, then it can be shown that:

$$\dot{\delta}_W = 0{\cdot}76\,\dot{\delta} \qquad ...(3)$$

VI.- DISCUSSION OF EXPERIMENTAL RESULTS

As derived earlier the expression for the rate of displacement along an assumed failure plane by the theory of rate processes is given by equation (2). For the conditions corresponding to Test No.1, the observed results from the torsion shear and retaining wall tests revealed that after a period of transient creep, deformation occured at a constant rate so that m_1 is zero in equation (2). Also, since the observed behaviour corresponds to the fully active condition (P = 54 lb.) in which the undrained shear strength, C_u is mobilized then equation (2) becomes:

$$\dot{\delta} = A_1 . e^{\alpha_1.(C_u-\tau_o)} \qquad ...(4)$$

To determine the constants A_1 and α_1 it is necessary to plot the shear stress increment, $(\tau-\tau_o)$ against the logarithm of the deformation rate, $\dot{\delta}$ from the torsion shear creep tests. This plot is shown in Figure 4. The slope of the line of best fit passing through the experimental points gives the value of α_1, which in this case is 88.0. The value of A_1 is 3.1×10^{-11}. obtained from the intercept where $(\tau-\tau_o)$ is zero. Therefore the deformation rate along the assumed failure plane within the clay backfill is given by:

$$\dot{\delta} = 3{\cdot}1 \times 10^{-11}\, e^{88{\cdot}0(C_u-\tau_o)} \qquad ...(5)$$

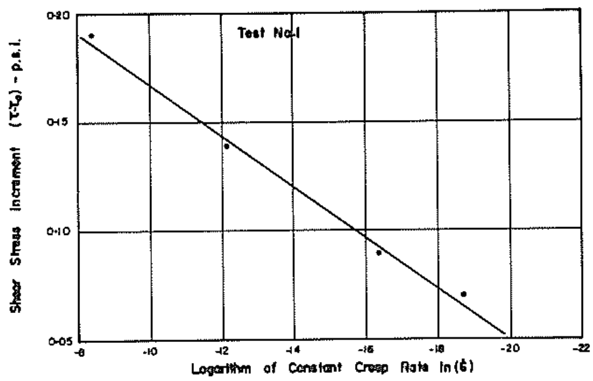

Fig.4. Plot of $(\tau-\tau_o)$ Versus LN($\dot{\delta}$)

If 0.14 psi, which is the shear stress increment corresponding to the 56 lb. lateral restraining force, is substituted for $(c_u-\tau_o)$, then the deformation rate, $\dot{\delta}$ is equal to 0.009 in./day, which in terms of the rate of outward wall movement, $\dot{\delta}_w$ is 0.0068 in./day. The comparison between the predicted rate of 0.0068 in./day and the observed rate of 0.0080 in./day is extremely good.

In Test No.2 the deformation behaviour corresponding to lateral restraining forces of 60 lb. and 45 lb. were observed. From considerations of equilibrium of the failure wedge the shear stress developed along the plane failure surface passing through the toe of the wall and corresponding to the above lateral forces are 0.26 psi and 0.29 psi respectively. Since the relaxed shear strength τ_o is equal to 0.10 psi, the shear stress increments, $(\tau-\tau_o)$ corresponding to lateral forces of 60 lb. and 45 lb. are 0.16 psi and 0.19 psi, respectively.

The results of the torsion shear creep tests have been analysed by plotting the logarithm of deformation rate, $\dot{\delta}$ versus the logarithm of time, t as shown in Figure 5. This plot produces four nearly parallel straight lines, each corresponding to a level of applied shear stress, τ. The slope of these lines ranged from 0.56 to 0.51, having an average value of 0.53 which is the magnitude of m_1 in the rate process theory equation.

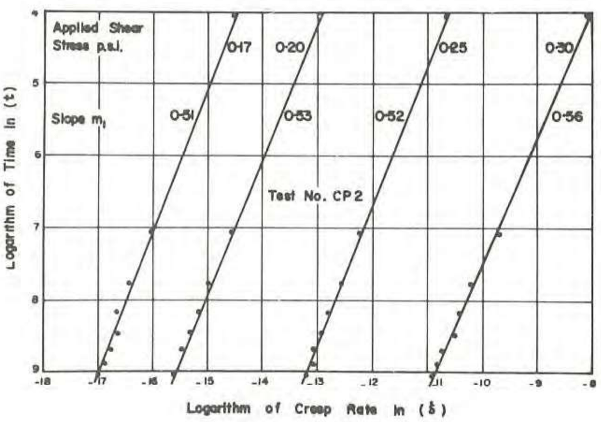

Fig.5. Plot of LN(t) Versus LN($\dot{\delta}$)

The values of A_1 and α_1 in this equation were obtained by plotting the deformation rate at various times after commencement of the creep test versus the shear stress increment $(\tau-\tau_o)$. This plot of LN $(\dot{\delta})$ versus $(\tau-\tau_o)$ is shown in Figure 6 for the four creep stresses at times corresponding to 1,3 and 6 days after each test was begun. This plot produced three parallel straight lines which correspond to the above times. For calculation purposes the unit time, t_1 was chosen to be 1 day. From the 1 day line in Figure 6, the slope α_1 was found to be equal to 49.0 and the intercept where $(\tau-\tau_o)$ is zero, A_1 equal to 3.2×10^{-9}. If these values are substituted into the equation derived from the rate

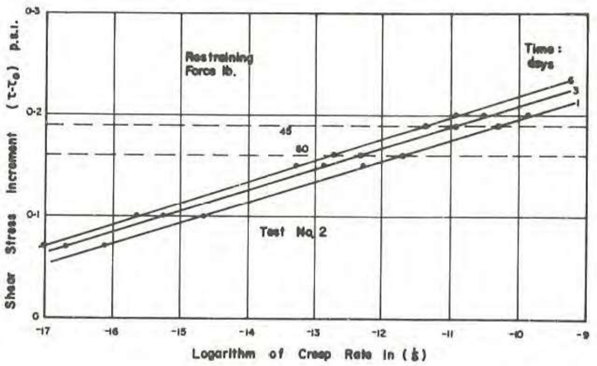

Fig.6. Plot of $(\tau-\tau_o)$ Versus LN($\dot{\delta}$)

process theory, then the deformation rate along the assumed failure plane within the clay backfill is given by:

$$\dot{\delta} = 3.2\times10^{-9}\left(\frac{1}{t}\right)^{0.53} e^{49.0(\tau-\tau_o)} \qquad \ldots(6)$$

From the above expression the rate of displacement along the failure plane can be determined at any time, t (days) and at any value of the mobilized shear stress, τ along the failure surface. The horizontal wall movement, $\dot{\delta}_w$ can be found from equation (3). The observed rates of outward wall movement variation with time for restraining forces of 60 lb. and 45 lb. have been plotted in Figure 7. Superimposed on this figure is the theoretical behaviour predicted from the rate process theory. The average difference between the observed and predicted behaviour is approximately 15%, thus indicating that the adopted theory provides a reasonable method of estimating rates of wall movement.

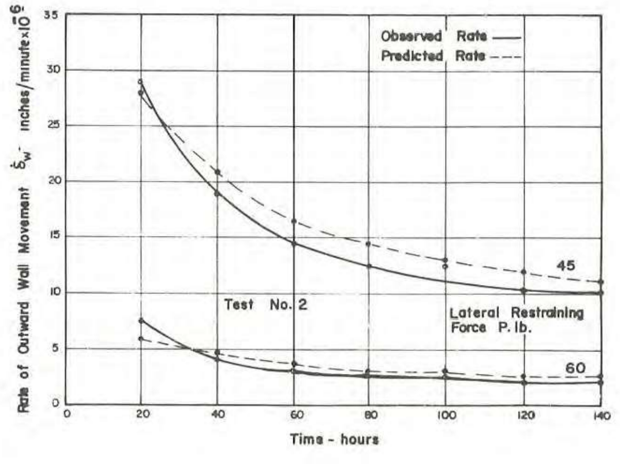

Fig.7. Comparison of Observed & Predicted Rates of Outward Wall Movement

VII.- CONCLUSIONS

The results of torsion shear tests on soft clays have indicated that in order to keep the undrained shear strength mobilized, a definite rate of deformation must be applied to a shear surface. Similarly, to maintain a mobilized shear stress less than the failure stress along a failure plane within a clay backfill, the retaining structure must continually creep to achieve equilibrium. The relaxed shear strength is the shear stress which is compatible with zero deformation along a shear surface. If a retaining wall is designed on the basis that the relaxed shear strength is the maximum stress acting across a failure plane, then outward wall displacements will be zero.

The results of measurements of the relaxed shear strength τ_o in the torsion shear stage tests suggests that continual creep of walls will occur if shear stresses in a backfill are larger than τ_o. For prototype backfill materials, the results from the model retaining wall and torsion shear creep tests reveal that deformations occur at a decreasing rate. The rate process theory was used to derive an expression for the rate of deformation along an assumed plane failure surface. The constants in the expression were calculated from the results of a series of torsion shear creep tests and the deformation rate converted into an equivalent wall movement. The observed and predicted outward wall displacement rates disagreed by about 15%. Thus it seems that deformation of a retaining structure can be predicted within reasonable limits by considering the rate process theory equation applied to the failure surface within a soil mass.

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