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Displacements of Slopes Under Earthquake Loading

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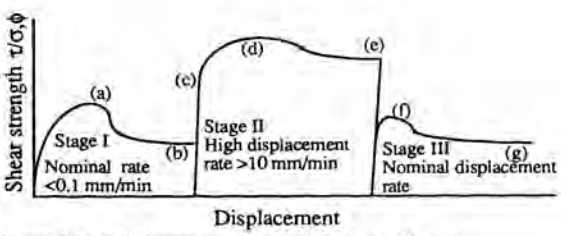
YNOPSIS: The displacements and rates of displacement of slopes post-rupture are influenced by the dynamic response of soils. re-existing shear surfaces at or close to residual strength are frequently present in slopes of clay and weak mudstone, due to revious slope movement or tectonic disturbance. A knowledge of the strength of such surfaces under rapid loading is necessary f stability during and after an earthquake is to be examined. To obtain such data, high speed displacement controlled ring shear ests have been performed, on samples after being pre-sheared to residual conditions.

Vith the results from those tests, suitable constitutive laws were derived for the soils and numerical analysis were made of an old andslide submited to dynamic earthquake loading. Analysis were also performed for first landslides, on plastic clays, in order to alculate the maximum velocity, displacement and duration of sliding. The calculation is based on the assunption that the whole noving mass is displaced as a single rigid body with resistance mobilised along the sliding surface. Newmark's sliding block vas used, sliding on a non linear surface involving a resistance dependent on factors such as displacement and rate of slip.

1 INTRODUCTION

A program of testing, on the ring shear apparatus, was carried out on various soils to investigate the influence of the rate of displacement on residual strength, Lemos, Skempton and Vaughan (1985), Lemos (1986), Tika (1989), Lemos (1991). Rates of displacement ranging from 0.001 to 6000 mm/min were used. Residual conditions, for a given normal stress were established at the nominal displacement rate, which is calculated in order to ensure 90% dissipation of the pore water pressures in 1 to 2 mm of displacement. After the residual conditions were achived (b) on figure 1, fast rates were applied followed by slow probes at the nominal rate. In the fast rates it was measured the threshold shear strength (immediate increase in strength) (c), the maximum shear resistance (d) and the minimum shear resistance (e) (fast residual). Slow probes, following the fast rates and consolidation, were used to measure the disturbance of the shear zone induced by fast shearing (f-g).



-Stage I Establishement of the residual conditions
(a) peak shear strength (b) residual shear strength
-Stage II Fast stage (c) immediate increase in strength (threshold) (d) maximum shear resistance (e) fast residual strength (minimum)
-Stage III Slow stage to determine the disturbance of the shear zone in the previous fast stage (f) peak shear strength
(g)=(b) residual shear strength [(f)-(g)] indicates the disturbance of the zhear zone.

Figure 1 Typical stages of a ring shear test to investigate the soil response to fast shear, after the residual conditions have been established.

In fast shearing three types of behaviour were observed, Lemos, Skempton and Vaughan (1985), Lemos (1986), Lemos (1991), which are exemplified on figure 2, with the results of some soils. The soils and their classification are given on table 1. The types of behaviour identified were: Type I showing a positive rate effect, Type II showing a neutral rate effect and Type III showing a negative rate effect.

TABLE 1 SOIL CLASSIFICATION

Soil Type		w_L	I_P	%<2μm	*	**
Lower Cromer Till	A	22	10	14	Tu	Neg
Cowden Till	B	34	16	28	Tu/Tr	Pos
Buoninvetre	C	53	26	39	S	Pos
London Clay	D	82	49	60	S	Pos
Kaolin	E	72	36	82	S	Pos
Kalabagh (Pakistan)	G1	62	36	52	S	Pos
	G2	39	18	20	Tr	Neg
	G3	56	31	40	S	Pos
	G4	44	20	18	Tr	Neg
	G5	45	21	15	Tr	Neg
	G6	49	27	27	Tr	Neg
	G7	39	17	10	Tu/Tr	Neg
	G8	NP		3	Tu	Neu
	G9	NP		2	Tu	Neu
	G10	55	27	39	S	Pos
Fiji	H	90	37	40	S	Neg
Magnus Till	I	32	15	37	Tu	Pos

* Type of behaviour in slow shear Tu-Turbulent, Tr-Transitional, S-Sliding
** Type of behaviour in fast shear Neg-Negative rate effect Pos-Positive, Neu-Neutral

Soils showing a positive rate effect, figure 2a, have a clay fraction over 50% and the behaviour is caracterised by a viscous immediate increase in strength (c), followed by an increase in strength with displacement up to a maximum (d) and then a moderate decrease up to a minimum (e), which is higher than the slow residual (b). The fast stage will disrupt the shear zone and a peak (f) is obtained when the slow stage is resumed.

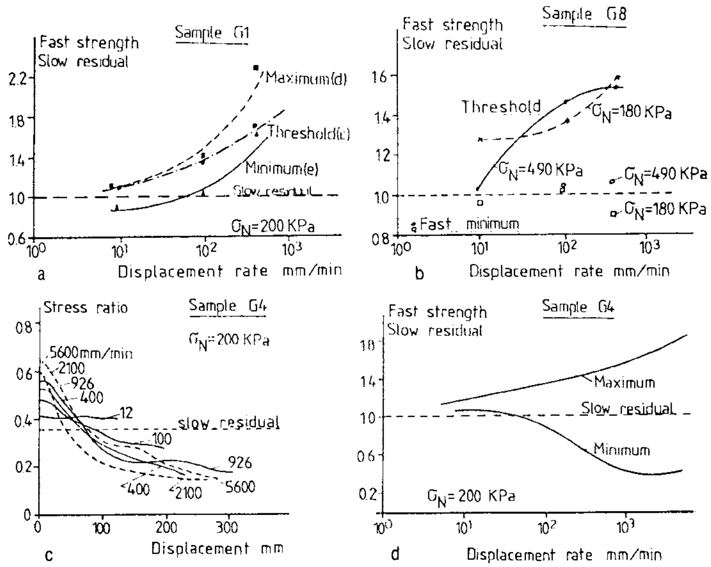


Figure 2. Test results of fast ring shear tests. (a) positive rate effect, (b) neutral rate effect, (c) and (d) negative rate effect.

The disturbance of the shear zone is measured by the diference between (f) and (b=g). Soils with a neutral rate effect are pure granular soils with less than 5% of clay fraction. Their typical behaviour is shown on figure 2b. There is an immediate increase in strength which is as well the maximum (c=d) and then a drop in resistance with displacement up to a minimum value equal to the slow residual strength (e=b=g). The immediate increase in strength is believed to result of interlocking and temporary build up of negative pore water pressures. The void ratio tends to increase in faster shearing, Hungr and Morgenstern (1984), which develops interlocking when the rate is increased and in saturated undrained conditions there will be build up of negative pore water pressures. The fast residual shear strength is independent of the displacement rate and it is equal to the slow residual strength.

Soils with clay fraction between 5 and 50% can show a behaviour of the type shown on figures 2c and 2d with a negative rate effect. This type of behaviour is identical to the one with positive rate effect, but the fast residual shear resistance is much smaller than the slow residual value, for displacement rates above 100 mm/min. The drop in shear resistance in fast shear appear not to be due to heat or build up of pore water pressure, it is an intrinsec property of the soil, Lemos (1991).

It is of interest to analyse the behaviour post-rupture of slopes and the displacement of slopes under earthquake loading, considering the influence of rate on the shear strength.

2 NUMERICAL CONSTITUTIVE MODELS

2.1 Threshold Strength

An immediate increase in strength was observed when the rate of displacement was increased. This immediate increase in strength increases with increasing displacement rate. The test results are plotted on figure 3, using logarithmic axes.
It can be seen that there is an almost linear relationship, which can be formulated with the following expression:

$$\frac{\mu_T-\mu}{\mu} = a\,\dot{d}^b \quad (1)$$

In which:

μ_T represents the dynamic resistance at the displacement rate of $\dot{d}$
μ represents the static resistance at the nominal dispalacement rate.
a and b are constants, and
$\dot{d}$ displacement rate

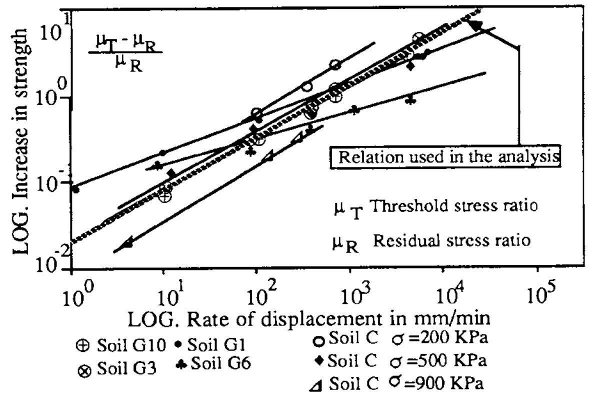


Figure 3 Immediate increase in resistance in relation to the residual strength plotted against rate of displacement.

The immediate increase in strength (threshold) seem to be of viscous nature in the high plastic clays and on the low plastic clay it depends as well as on the build up of negative pore water pressures. The viscous shear resistance is dependent on the rate of shear strain and so on the thickness of the shear zone. The thickness of the shear zone differs in the laboratory and in the field and so the constants derived from the laboratory tests may be different from those in the field.

$$\tau = \mu\,\frac{\dot{d}}{h}$$

$$R_L = \frac{\mu_T-\mu}{\mu} = a_1\,\dot{d}^b\left(\frac{1}{h_L}\right)^b \quad (2) \qquad R_F = \frac{\mu_T-\mu}{\mu} = a_1\,\dot{d}^b\left(\frac{1}{h_F}\right)^b \quad (3)$$

Dividing 3 by 2 we obtain:

$$\frac{R_L}{R_F} = \frac{(1/h_L)^b}{(1/h_F)^b} = \left(\frac{h_F}{h_L}\right)^b \quad (4)$$

$$R_F = R_L\left(\frac{h_L}{h_F}\right)^b \quad (5)$$

Changing R_L by the right hand side of equation (2) we get

$$R_F = a_1\left(\frac{1}{h_L}\right)^b \dot{d}^b \left(\frac{h_L}{h_F}\right)^b \quad (6)$$

By putting $a = a_1(1/h_L)^b$ we get

$$R_F = a\left(\frac{h_L}{h_F}\right)^b \dot{d}^b \quad (7)$$

Comparing equations 7 and 1 we see that in order to consider the viscous strength component for different thicknesses of the shear zone we have only to multiply the constant "a" obtained from the laboratory by the ratio of the shear zones to the power "b". The constant values used in the analysis were a=0.9, b=0.6 and the ratio of the shear zones 10.

2.2 Disturbance of the Shear Zone

When a slow probe was applied after a fast rate of displacement a peak strength was observed which reduced to residual conditions with relatively small further displacements (typically 2 to 3 cm). The peak shear stress ratio refered to above increases with the increase of the displacement rate of the previous fast shearing stage. The test results are shown on the figure 4.

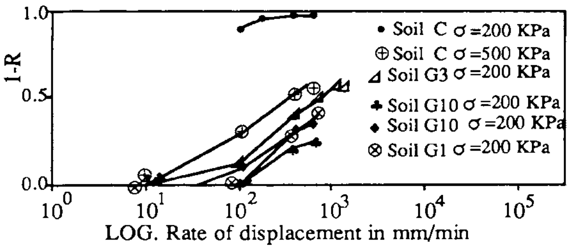


Figure 4 Ring shear test results showing the disturbance of the shear zone as a function of the displacement rate.

It can be seen that for the lowest previous rates of displacement the disturbance of the shear zone is smallest. As the rate of displacement increases the disturbance of the shear zone increases. It is suggested that, the disturbance increases most drastically over a limited interval of displacement rates and at higher rates there is little further disturbance tending asymptotically to a maximum value of disturbance with increasing displacement rate. The disturbance of the shear zone may be explained by a change in the shear mechanism from sliding to turbulent, Lupini, Skinner and Vaughan, (1981) due to an increase in the particle friction caused by the viscous component and by the increase in the mobility of the granular particles in the faster rates disrupting the alignement of the clay particles.

The results shown on figure 4 can be with a good approximation modelled with a sectioned linear approach shown on figure 5. The values (1-R) represented in the figure 5 represent the increase in static strength, that would be observed in a slow test on a surface over which large displacements had taken place at a fast rate. By using the approach developed in the last section it can be assumed that the rates for the field, with thicker shear zones and the same degree of disturbance of the shear zone, can be obtained by multiplying the laboratory values by $(h_F/h_L)^b$, so that;

$$\dot{d}_F = \left(\frac{h_F}{h_L}\right)^b \dot{d}_L$$

F and L signify field and laboratory respectively
h is the thickness of the shear zone
$\dot{d}$ is the rate of displacement

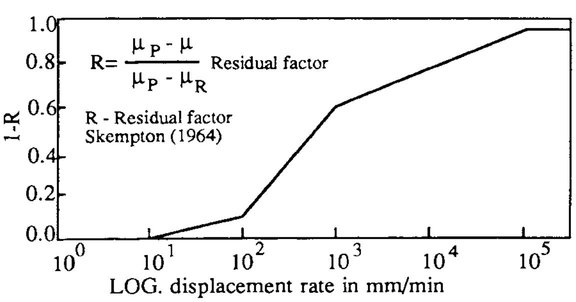


Figure 5 Model for the disturbance of the shear zone as a function of the displacement rate.

The increase or decrease in strength, depending on the current static strength and rate of displacement, was modelled using an exponential function.

2.3 Fast Minimum Strength

By looking at figure 2 c) and d), we observe that;
1 There is a threshold displacement rate, above which the minimum fast strength drops to values below the slow residual.
2 The minimum fast strength attains the minimum value for displacement rates of about 1000 mm/min, being then constant for higher rates.
3. The minimum fast value is obtained within displacements of the order of 200 mm. After this minimum is obtained it remains constant during prolonged fast shear and even when the displacement rate is immediatly reduced to values lower than 10 mm/min (without consolidation).

Thus, this behaviour can be modeled as shown on figure 6. The values given on the figure 6 were taken as the average of the samples that did show this type of behaviour.

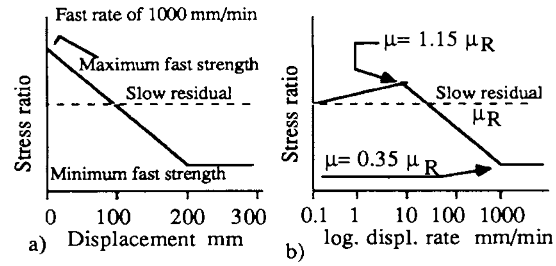


Figure 6 a) Variation of the stress ratio in fast shear with displacement b) Variation of the fast residual strength with displacement rate.

2.4 Decrease in Strength with Displacement

The relationship describing the decrease in static strength with displacement from peak to residual was established in tests carried out with slow rates of displacement, then an assumption has to be made concerning the shape of this curve when different rates are considered. It is assumed that the general shape will be similar to the one obtained from the slow tests but the curves will differ in their ultimate strengths (i.e. the residual strength and the degree of particle orientation will be dependent on the rate of displacement).

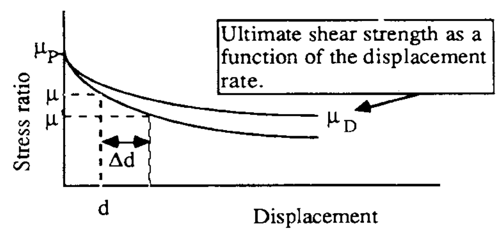


Figure 8 Model for the decrease in the static shear strength with displacement from peak (μ_P) to residual (μ_D) function of the displacement rate and degree of particle orientation.

This assumption is illustrated in figure 8. If different displacement rates were used from the same initial conditions, the two curves representing the decrease in the static strength with displacement will initially be coincident but will tend to different ultimate strengths, which depends on the degree of particle orientation.

An exponential function was found to fit the experimental data, as shown below

$$\mu' = \mu_D - (\mu_P - \mu_D)\, e^{-b(d+\Delta d)}$$

Where $d=-(1/b)\ \ln[(\mu-\mu_D)\div(\mu_P-\mu_D)$ displacement correspondly to the strength ratio μ, at a given time.
Δd displacement increment
μ_P Peak shear strength
μ_D ultimate (residual) strength for a given rate of displacement
μ e μ' static strength of the soil before and after the displacement increnent

The value of the constant "b" used in the analysis was equal to 0.3.

2.5 Increase in Strength with Displacement

It was found that when a fast rate was applied to a shear surface at residual shear strength, followed by a slow one, a peak strength, in the later, was obtained and the residual conditions were reestablished in relatively small displacements, see fig 1.

The actual increase in strength with displacement in a fast rate can only be guessed, as from the ring shear work, it is only known the static shear resistance before and after a fast stage. The increase in static strength at fast rates is due to the disturbance of the orientated particles in the shear zone. This disturbance will be more accentuated in the early displacements, and an equilibrium will be reached after some displacement, which can be approached with an exponential form as shown on figure 9.

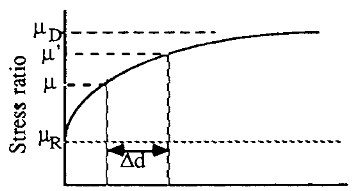


Figure 9 Model representing the increase in static shear strength with displacement.

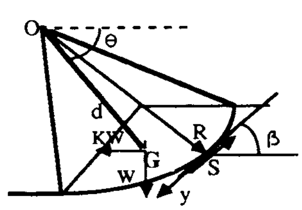
$$\mu' = \mu_D - (\mu_D - \mu_R)\, e^{-a(d+\Delta d)}$$

Where:
$d = -(1/a) \ln [(\mu_D - \mu)/(\mu_D - \mu_R)$ displacement corresponding to the stress ratio for a given displacement.
Δd displacement increment
μ_D static strength which depends on the rate of displacement, if enough displacement was allowed for it to develop fully.
μ_R stress ratio value for the residual strength
μ and μ' stress ratio at displacement d and $d+\Delta d$.

The value of the constant "a" used in the analysis was equal to 0.4.

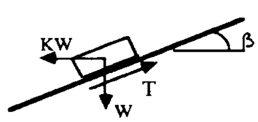
2.6 Model of Analysis

The sliding block model developed by Newmark (1965), was used. The slope angle was varied in order to simulate a circular slope, as shown below. In this simple method it is assumed that slope failure would be initiated and movements would begin to develop if the seismic forces on a potential slide mass were large enough to overcome the yield resistance and that movements would stop when the seismic forces were removed or reversed. Thus, by computing the acceleration at which yielding begins and summing up the displacements during the periods of instability, the final cumulative displacement of the slide mass can be evaluated. The calculation is based on the assumption that the whole moving mass is displaced as a rigid body with resistance mobilised along the sliding surface. Newmark's sliding block method is based on the simple equation of rectilinear motion under the action of a time dependent force involving a resistance dependent on displacement, rate of slip and pore water pressure.



Moment of disturbance

$M_d = W d\cos\theta + KW d\sin\theta$



Force of disturbance

$F_d = W\sin\beta + KW\cos\beta$

Resistance moment

$$M_r = \Sigma RS = \int_{\theta_1}^{\theta_2} R\, dS\, d\theta$$

Resistance force

$$F_r = T$$

Then by the second law of Newton

$$I\ddot{\theta} = W d\cos\theta + K W d\sin\theta - \Sigma RS$$

$$\ddot{\theta} = \frac{Wd}{I}(\cos\theta + K\sin\theta) - \frac{1}{I}\Sigma RS$$

$$\frac{\delta\ddot{\theta}}{\delta\theta} = \frac{Wd}{I}(-\sin\theta + K\cos\theta) - 0$$

$$\delta\ddot{\theta} = \frac{Wd}{I}(-\sin\theta + K\cos\theta)\,\delta\theta$$

$$\delta y = \delta\theta\, R$$

$$\delta\ddot{y} = \delta\ddot{\theta}\, R$$

$$\delta\ddot{\theta} = \frac{RWd}{I}(-\sin\theta + K\cos\theta)\,\delta\theta$$

But $R\,\delta\theta = \partial y$

$$\partial\ddot{y} = \frac{RWd}{I}(-\sin\theta + K\cos\theta)\,\partial y \quad (a)$$

$$M\ddot{y} = W\sin\beta + K W\cos\beta - T$$

Considering that β is a function of y (displacement), and that the angle of the sliding surface varies with displacement, and that the value of T is constant in a small interval ∂y, we get

$$\frac{\partial\ddot{y}}{\partial y} = \frac{W}{M}(\cos\beta - K\sin\beta)\frac{\delta\beta}{\partial y} - 0$$

$$\partial\ddot{y} = \frac{W}{M}(\cos\beta - K\sin\beta)\,\delta\beta \quad (b)$$

Equating (a) and (b) and putting $I=Mr^2$, $(W/M)=g$, we get:

$$\delta\beta = \frac{d}{r^2}\cdot\frac{-\sin\theta + K\cos\theta}{\cos\beta - K\sin\beta}\,\delta y$$

Dividing and multiplying the right hand side of the equation by R, we get;

$$\delta\beta = \frac{dR}{r^2}\cdot\frac{-\sin\theta + K\cos\theta}{\cos\beta - K\sin\beta}\cdot\frac{\delta y}{R}$$

If we put $C=(dR/r^2)$, for which Sarma (1980) demonstrated to be nearly equal to 1 and puting $\beta = 90-\theta$, then we obtain;

$$\delta\beta = -C\cdot\frac{\cos\beta - K\sin\beta}{\cos\beta - K\sin\beta}\cdot\frac{\delta y}{R}$$

$$\delta\beta = -\frac{C}{R}\cdot\delta y$$ and puting C=1 Sarma, (1980), we get;

$$\delta\beta = -\frac{1}{R}\cdot\delta y$$

Thus in order to have the same acceleration on a circular slope and a sliding block, the initial conditions being equal, the slope angle in the latter has to be changed linearly with displacement with the constant of proporcionality approximatly equal to (1/R).

3 DISCUSSION AND PRESENTATION OF THE RESULTS

3.1 First Time Landslides

A sliding block was analysed in which the slope angle decreased with displacement in order to simulate a rotational slope as described above. The peak and residual stress ratio τ / σ' was respectively equal to 0.4 and 0.176. The slide was considered to be initiated by a fall of the resisting forces, so that equilibrium could not be maintained.

The influence of the rate of displacement was considered in the analysis using the models described in section 2, and for comparision the slope was analysed without the rate effects.

In the analysis with soils with a positive rate effect, it was observed that, the maximum acceleration was reached immediatly after the start of the sliding at $t=0^+$, and as the velocity increased the viscous component of shear strength increased, which caused a decrease in the acceleration. The velocity increased until the dynamic shear strength was nearly equal to the disturbing forces, after which the slope continued to move, but with a slowly decreasing velocity. At any time the velocity will be that required to mobilise sufficient "viscous" strength, so as to balance the disturbing forces, as illustrated in figure 10. The slope will reach a stable position when the slope angle was such to be in balance with the residual shear strength, which agrees with field observations.

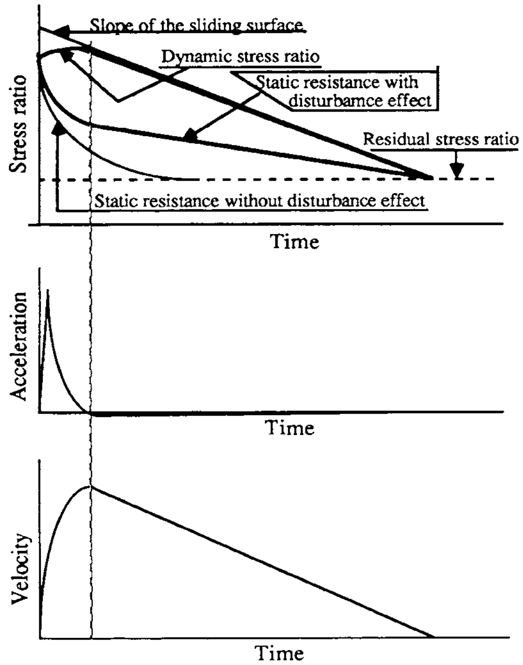


Figure 10. First time sliding of a slope considering the rate effects. It shows the variation of the shear resistance, acceleration and velocity with time.

In the analysis without rate effects, the shear resistance decreases quickly from peak to residual generating high unbalance forces and very fast movements. The slope overshoots and reaches a position where the slope angle is much lower than the one needed to balance the residual shear resistance.

The effect of including the rate dependent model can be summarised in the following two main points;

a) A change in the static strength displacement curves was observed, in the modelling of the slide. The curve showed a less steep reduction in strength with displacement than the one observed in the laboratory in a slow rate of displacement test. The shape of the curve in fast shearing is influenced by the velocity, disturbing the alignement of the clay particles in the shear zone.

b) The model gives a satisfatory qualitative description of the field behaviour of a first time failure in a clay slope. The post rupture movements are fully developed over a period of hours and reaches a stable position which is in equilibrium with the residual shear strength of the clay. The comparison of the analysis with and without rate effects is detailed below in table 2.

TABLE 2

	With rate effect	No rate effect
Maximum velocity	1.2 cm/sec	208 cm/sec
Maximum displacement	210 cm	419 cm
Time of sliding	±4 hours	3.5 sec
Factor of safety at the end of slide	1	>>1

The same analysis carried out on soils showing a negative rate effect showed that, if the rate of displacement above which there is high loss of shear resistance, is not exceeded, the behaviour is similar to the one described above, but if that displacement rate is exceeded then very fast movements will take place.

3.2 Displacements Induced by Earthquakes

A sliding block moving on a curved surface, with initial sloping angles of 8, 6 and 4 degrees and with the shear resistance at residual ($\mu=0.176$), was analysed under a triangular pulse and earthquake loading. In the triangular pulse the area was maintained constant varying the peak acceleration and the duration of the pulse. The peak acceleration varied between 1.8g and 0.15g. The earthquake records used in the analysis are described in table 3.

TABLE 3
LIST OF EARTHQUAKE STRONG MOTION CHARACTERISTICS

No	EARTHQUAKE	STATION	COMP HORIZ	A_{max} g	M_S	R Km	D sec	P_P sec
E1	Imperial Valley	El Centro	S00E	0.344	7.2	12	24.4	0.318
E2	Imperial Valley	El Centro	S90W	0.217	7.2	12	24.5	0.340
E3	Kern County	Taft Lincoln	N21E	0.156	7.7	42	28.7	0.302
E4	Kern County	Taft Lincoln	S69E	0.189	7.7	42	30.4	0.278
E5	Niigata	Kawagishi	N-S	0.140	7.5	38	37.5	-------
E6	Niigata	Kawagishi	E-W	0.161	7.5	38	37.8	-------
E7	Leukas	Leukas	N-S	0.513	5.7	20	5.1	0.412
E8	Leukas	Leukas	W-E	0.245	5.7	20	8.0	0.385
E9	Gazli	Gazli	N-S	0.613	7.1	5	6.5	0.091
E10	Gazli	Gazli	E-W	0.734	7.1	5	6.8	0.082
E11	San Salvador	Geogr. Inst.	270	0.536	5.4	8	20.3	0.038
E12	San Salvador	Geogr. Inst.	180	0.403	5.4	8	20.3	0.039

The final displacements were calculated with and without the influence of the rate of displacement on the shear resistance properties of the soil, and are plotted against the critical acceleration ratio (K_c/K_m) in figure 11. The parameter $K_c g$ is the minimum ground acceleration required to bring about incipient failure of a slope a parameter controlled by yield resistance, and the parameter $K_m g$ is the maximum acceleration of the ground motion time-history. Ambraseys and Menu (1988) investigated the effects of magnitude M_s, source distance R, predominant period P, duration of shaking D and peak acceleration A on the prediction of the permanent displacements. They concluded that, the magnitude and duration of shaking play an insignificant role in the prediction of the permanent displacement and that the critical acceleration ratio (K_c/K_m) is the most significant parameter in predicting displacements for large ratios. On figure 11 are presented two relationships presented by Ambrasyes and Menu (1988).

The final displacements with rate effects were 10 times smaller than without rate effects (one log cicle), for values of (K_c/K_m) smaller than 0.5. This difference seems to decrease then with the increase of that ratio. The relationships given by Ambraseys and Menu (1988), shown in figure 11, predicte the final displacements with a probability of not being exceeded higher than 80%, for the analysis without rate effects.

Identical relationships fit the earthquake induced displacements calculated with rate effects with a probability of not being exceeded higher than 80%. In figure 11 is shown the relationship "A" with the following equation;

$$u = (1 - (K_c/K_m))^{2.53} * (K_c/K_m)^{-1.09}$$

and the relationship "B" with the following equation;

$$\log(u) = 1.12 - 3.2\,(K_c/K_m).$$

An increase in the static strength of 40% was observed at the end of the earthquake, which means that a slope could be safer at the end of an earthquake discarding the generation of positive pore water pressures. This value should not be regarded as an absolute prediction as its computation is dependent on the model used to describe the increase in strength with displacement and disturbance of the shear zone and many assumptions have been made in the derivation of the model.

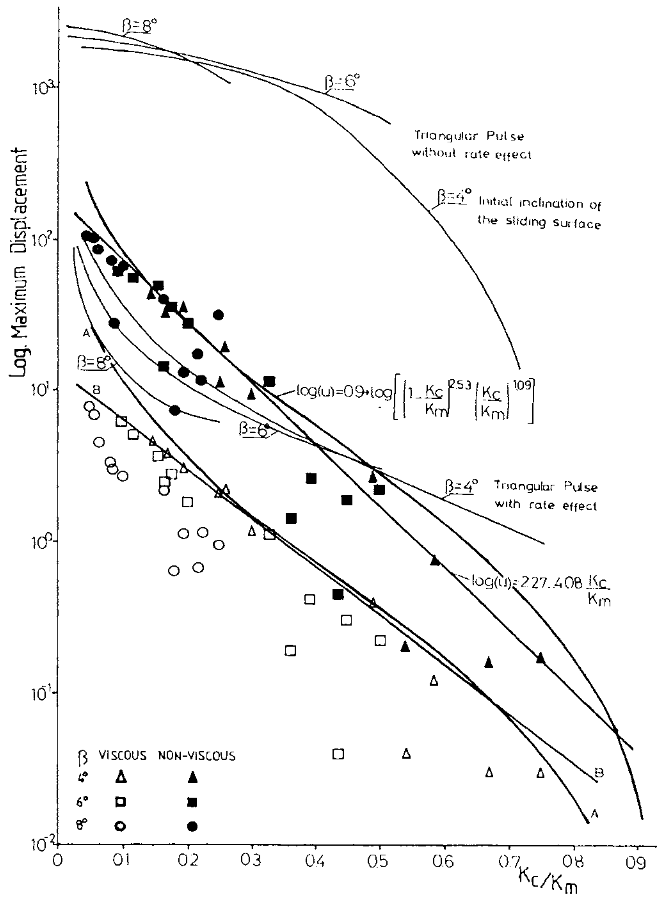


Figure 11. Maximum displacements induced by earthquake ground motions and triangular pulses with constant area.

However, the new model provides a qualitative means of evaluating the significance of the ring shear data in actual landslides dynamics.

4 CONCLUSIONS

The post-rupture movements of first time landslides were closely modelled using the dynamic properties of plastic soils, where the followings points should be pointed out;

a) the static strength-displacement curves were influenced by the displacement rate and were less steep than the ones given by the slow ring shear test.

b) the static strength at the end of movement was at residual and in balance with the disturbing forces. The factor of safety was equal to one.

It is important to consider the influence of the rate of displacement for predicting the displacements induced by earthquake. The displacements calculated with the rate effects were 10 times smaller than the ones calculated without their effect. The following expressions fit resonable well the data with a probability higher than 80% of not being exceeded.

$$u = (1 - (K_c/K_m))^{2.53} * (K_c/K_m)^{-1.09}$$

and;

$$\log(u) = 1.12 - 3.2\,(K_c/K_m).$$

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