Evaluation of the dynamical modeling of the Valette landslide.

L.P.H. van Beek,
Department of Physical Geography, Utrecht University

ABSTRACT

In this paper the results of two empirical dynamic models based on the concept of rate dependent shear strength are evaluated for the mass movement complex of la Valette.
The mobility characteristics as defined by the models were obtained from ringshear tests and applied in a one-dimensional stability model, linking the rate of movement to the inverse safety factor for the middle part of the complex. Daily pore pressure fluctuations have been calculated by means of a simple linear store model, using effective rainfall as input.
The predicted displacements of the different dynamic models are compared to the observed values and their validity is discussed with respect to the mobility of the Valette landslide.

INTRODUCTION

The mass movement complex of la Valette consists of series of interrelated mass movement events which occurred since March 1982, when excessive pore pressures along the contact between more permeable Flysch deposits and the underlying Terres Noires (black marls) triggered a large scale rotational slide (RTM[1] Digne, 1982). The disturbance of the drainage of the upper slope led consequently to the mobilization of the regolith consisting of weathered Terres Noires and argillaceous moraine deposits in the lower zone. Here, on the *amont* a total of 3,000,000 m^3 is displaced (RTM Digne, 1982).
Catastrophic movement occurred in January 1988, after an abrupt thawing period. Excess pore water forced 55,000 m^3 of morainic material to flow in the lower course of the Serre ravine where it reached a barrier some 1000 m from its source area in less than one day. The subsequent stress release in the source area and similar thawing conditions in March of that year initiated a second, more rigid earth flow with a far lower run-out distance. As a result, the morainic deposits in the middle part were mobilized, both movements amounting to 600,000 m^3.
To prevent further catastrophic development remedial measures were taken by the RTM, including the drainage of surface- and ground water.
In addition, the superficial displacements in the middle part of the landslide complex were monitored along a survey line, roughly perpendicular to the direction of movement, from 9 September 1988 till 10 September 1991 (Figure 1). Because of the volumes of material entering the Serre ravine the middle part was assumed to be sliding *en bloc* over a base of Terres Noires. Volumetric displacements were calculated by multiplying the observed displacements of pickets with the corresponding area under the profile which was determined by the CETE[2] using seismic refraction methods (total area = 4252 m^2; Figure 2).

[1]: branch *Restauration des Terrains en Montagne* of the *Office National des Forêts*
[2]: *Centre des Études Techniques de l' Equipement*

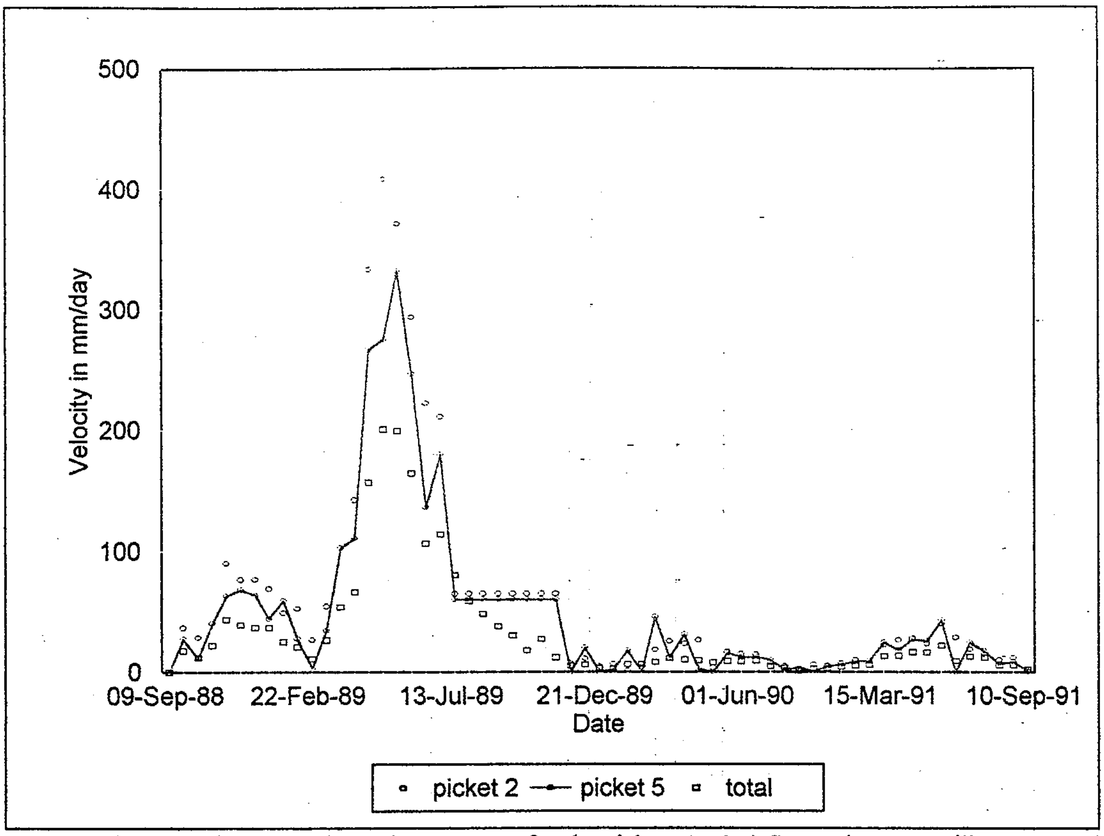

Figure 1: Velocities for picket 2 and 5 and on average for the pickets 1-13; 9 September 1988 till 10 September 1991

From 9 September 1988 to 10 September 1991 the maximum total displacement amounted to 52.2 meter for picket 2 of the survey line. From the observed displacements mean daily velocities have been calculated by dividing them over the time between two observations. The maximum velocity for the area was observed from the 27th of April to the 3rd of May 1989 when 881 $m^3.d^{-1}$ passed the survey line, what corresponds with an average velocity of $0.2 m.d^{-1}$. The highest velocity in this period ($0.4 m.d^{-1}$) was observed at picket 2, while at the cascade velocities of 1 $m.d^{-1}$ were found. There is an obvious relation between rainfall through the resulting pore pressure fluctuations and movement for the area (Figure 2), although a break in the trend can be discerned due to the extensive drainage works which were accomplished in the same period (RTM, October 1990).

However, to explain the observed limiting strain rates a regulating mechanism is needed in addition to the concept of plasticity alone for this predicts indefinite acceleration after failure (Keefer and Johnson, 1983). With this design numerous dynamic models have been proposed based on the concept of rate dependent shear strength (Minzuno, 1989; Van Asch, in prep.). According to this concept the strain under the excess shear stress is controlled by the viscous drag in a shear zone above the sliding plane. With increased shear stress additional strength is mobilised in the shear zone and the overlying material is displaced as a rigid mass with a uniform velocity.

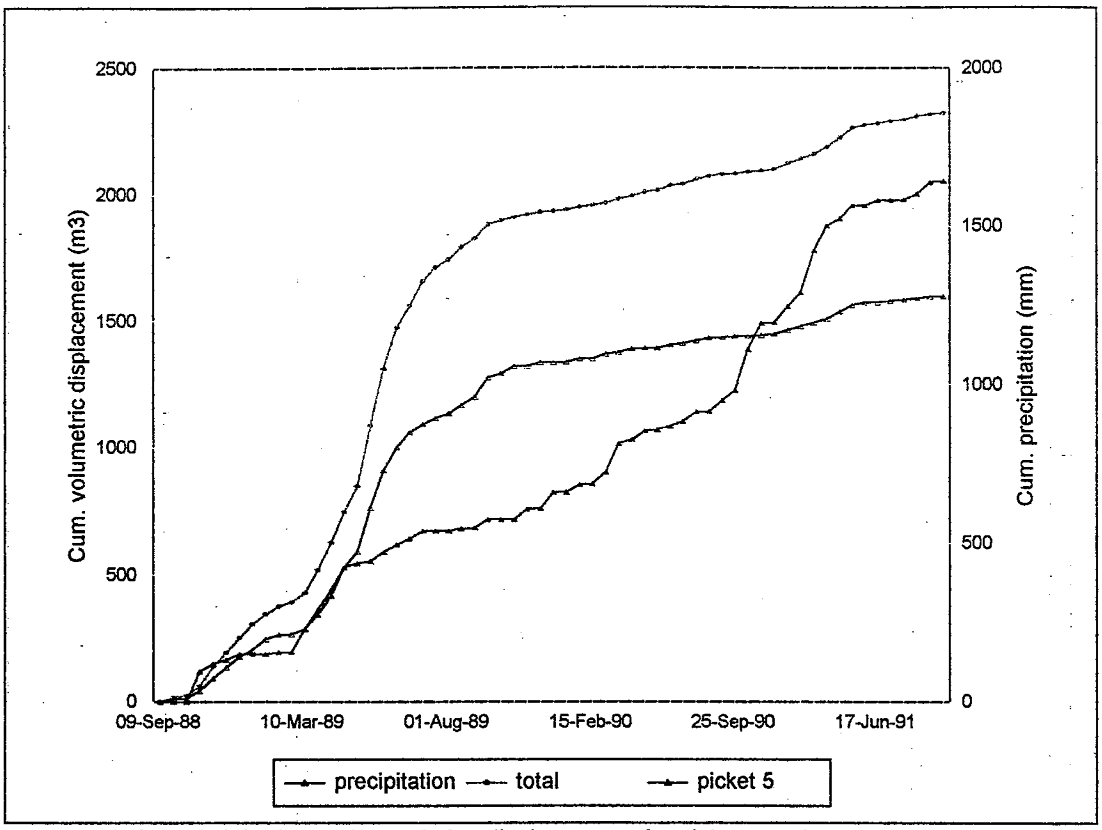

Figure 2: Cumulative precipitation and cumulative displacements for picket 2 and 5 and on average for the pickets 1-13; 9 September 1988 till 10 September 1991

Two empirical rate dependent shear strength models are evaluated here for the Valette landslide. The first, presented by Nieuwenhuis (1991), describes the strain rate as lineary dependent on the natural logarithm of the shear stress ratio τ/τ_0 :

$$\ln\left(\frac{v}{v_0}\right) = a.\ln\left(\frac{\tau}{\tau_0}\right) \quad \textbf{(1)}$$

where : a is a factor, intrinsic to the material,
v_0 is the strain rate associated with the yield strength t_0 ($\tau < \tau_0$, $v = 0$),
τ is the actual shear stress and v is the corresponding strain rate.

The second, as defined by Van Asch (in prep.), describes the mobility characteristics of the slip surface as a tangent hyperbolic function of the form :

$$y = 1 - a(\tanh(b) - \tanh(x + b)) \quad \textbf{(2)}$$

where : y is the shear stress ratio τ/τ_0
x is ln (v) in $mm.d^{-1}$ (hence y = 1 for x = 0);

Unlike Equation 1 that describes the indefinite increase of the strain rate with an increasing shear stress ratio, the shear stress ratio y of Equation 2 varies between the two asymptotic values which are given by :

$$y = 1 - a(\tanh(b) \pm 1) \quad for\ x = \pm\infty \quad \mathbf{(3)}$$

and which delimit the mobile range, the interval over which the material behaves in conformity with the concept of rate dependent shear strength. The lower limit $\tau/\tau_{0\ min}$ denotes the yield strength whereas the upper asymptotic value $\tau/\tau_{0\ max}$ can be considered as the stress ratio limit for plastic strength at which the velocity increases infinitely (Van Asch, in prep.).
For both dynamic models the excess shear stress, expressed as the inverse safety factor, has been calculated by means of a limit equilibrium model. Since the midle part of the Valette complex can be considered as an non restricted planar slide an infinite slope model is presumed to be appropriate. Daily pore pressures are calculated through a linear store model, using effective rainfall as input.

1. MODEL DESCRIPTION

The required mobility characteristics have been determined in the laboratory by means of ring shear tests in an apparatus based on Bishop's design (Nieuwenhuis, 1991). The effects of rate dependent shear strength have been determined by performing strain-controlled, consolidated tests on a samplecomposed from morainic material the middle part of the Valette landslide. During all tests the applied effective normal stress was 86.1 kPa. The resulting shear stress was recorded at strain rates of 0.2, 2, 4, 5 and 10 $°.h^{-1}$ (corresponding with 5, 52,105, 131 and 262 $mm.d^{-1}$). For the morainic material within the mass movement complex of la Valette a gain of 3% in shear strength was observed for an increase in speed from 0.2 to 10 $°.h^{-1}$. For the model of Equation 2, he small difference $\Delta\ \tau/\tau_0$ -or mobile range- of 0.03 influences also the viscosity index, which is defined as the maximum slope within this interval (Van Asch, in prep.).
In a regression of the ringshear tests (R^2= 99%) the constants a and b were found to amount to respectively a= 1.447 10^{-2} and b= -4.368. The plastic strength $\tau/\tau_{0\ max}$ for the material is nearly 1.03. Due to the assumption that the initial velocity at failure (1/F= 1) is 1 $mm.d^{-1}$, the yield strength $\tau/\tau_{0\ min}$ is slightly below 1. This assumption for the initial velocity, however, is justified by the minimal field displacements along the survey line, which were close to 1 $mm.d^{-1}$. The same strain rate was taken as the minimal velocity n_0 in the model of Nieuwenhuis (1991). The value of the factor a, based on a linear regression on the same ringshear tests, was 136.162 (R^2= 91%). A considerable source of error is here the intercept of the Y-axis of 0.437 but which is of minor importance for the assessment of larger displacements.
The peak and residual shear strength were determined by triaxial compression tests. Peak strength for the undisturbed morainic material amounts to $c'_p = 0$ kPa, $\phi'_p = 26.8°$ (n = 10, R^2 = 98%). With a strain of 16 % the incompletely developed residual strength for 4 samples -R^2 = 95%- has a ϕ'_r of 24.5 ° (c'_r is assumed to be zero).
With the mobility characteristics the displacements have been calculated, using the inverse safety factor for the shear stress ratio. The stability model used is an infinite slope model for which seepage parallel to the surface is assumed. The pore pressure was estimated by the ground water level that was obtained by a linear store model. The parametrization of this model, for which the Terres Noires were considered as an impermeable boundary, is given in Table 1. The geometry is based on the values for picket 5 of the survey line, the permeability is defined as the depth of the aquifer divided by the saturated hydraulic conductivity (k_{sat}= 8.6 * 10^{-3} $m.d^{-1}$, determined by inverse auger hole method).

For the input precipitation data of the station "le Verger" of the Office National des Forêts have been used. The initial ground water level at 09.09.1988 is set to 11.66 m, the value for limiting equilibrium.
To represent the fractured nature of the moraine ccover the recharge parameter of the model has been set to 1000%. Before 1 October 1989, the date arbitrarily set for the drainage to come in effect, the area of discharge is taken to be 2, the ratio between the upper slope and the middle part of the landslide complex (Figure 2). After the effectuation of the drainage this parameter is set to 1.

2. MODEL RESULTS

The observed and predicted strain rates against the safety factor of the one-dimensional model are given in Figure 3. As is apparent in Figure 3, the observed velocities are in general overestimated by the model proposed by Van Asch (in prep.). The calculated ground water levels (Figures 3 & 4) are in a small range, resulting in a safety factor close to unity. From 27.04.1989 till 24.10.1989, the perio in which the highest velocities were observed, the inverse safety factor exceeds the plastic limit τ/τ_{0max} of 1.03. In this period the plastic limit was exceeded continuously but no catastrophic development occurred. This is probably the consequence as the model underestimates the contribution of the rate dependent shear strength to the stability, either because of the importance of some other strain regulating mechanism[3] or because of the nature of the hyperbolic tangent that in combination with the scarce data gives too low a treshold.

A better correlation -R^2= 33%, n= 61- is obtained by the model presented by Nieuwenhuis (1991). In the light of Figure 4 in which the model results and the observed velocities are plotted its performance can be explained in more detail; the model is able to predict the magnitude of the strain rates adequately but lags by some weeks with the observed values. This is probably due to the parametrization of the hydrological model, especially to an underestimation of the hydraulic conductivity of the material. Calibration of this model -if it were possible- would certainly improve its predictions.

In consideration of the past catastrophic events the analysis of the dynamic stability models give an estimation for a worst case scenario for the complex under the present drained conditions. The large displacements of the spring and summer of 1989 were only achieved after a cumulative rainfall in the fall and winter and was maintaned by subsequent high rainfall in spring and summer. On the same assumption for the effectiveness of the drainage system it will require the double amount of precipitation in the same period, amounting roughly to 1200 mm. With regard to the annual rainfall of 763 mm it seems unlikely that these critical levels can be reached under the present, drained conditions.

[3]: see for discussion on this subject: Van Beek & Van Asch, 1996

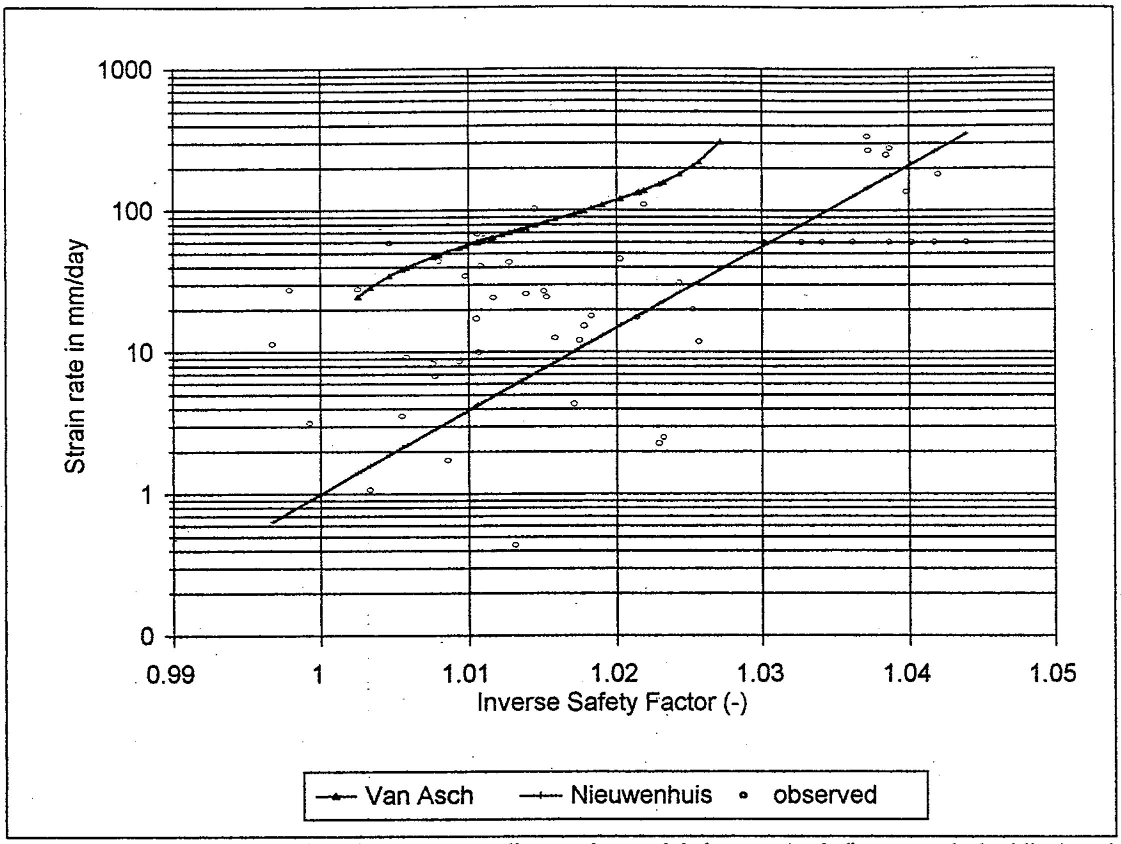

Figure 3: Observed and predicted strain rates according to the models by van Asch (in prep.; dashed line) and Nieuwenhuis (1991; drawn line) versus the calculated inverse safety factor τ/τ_0

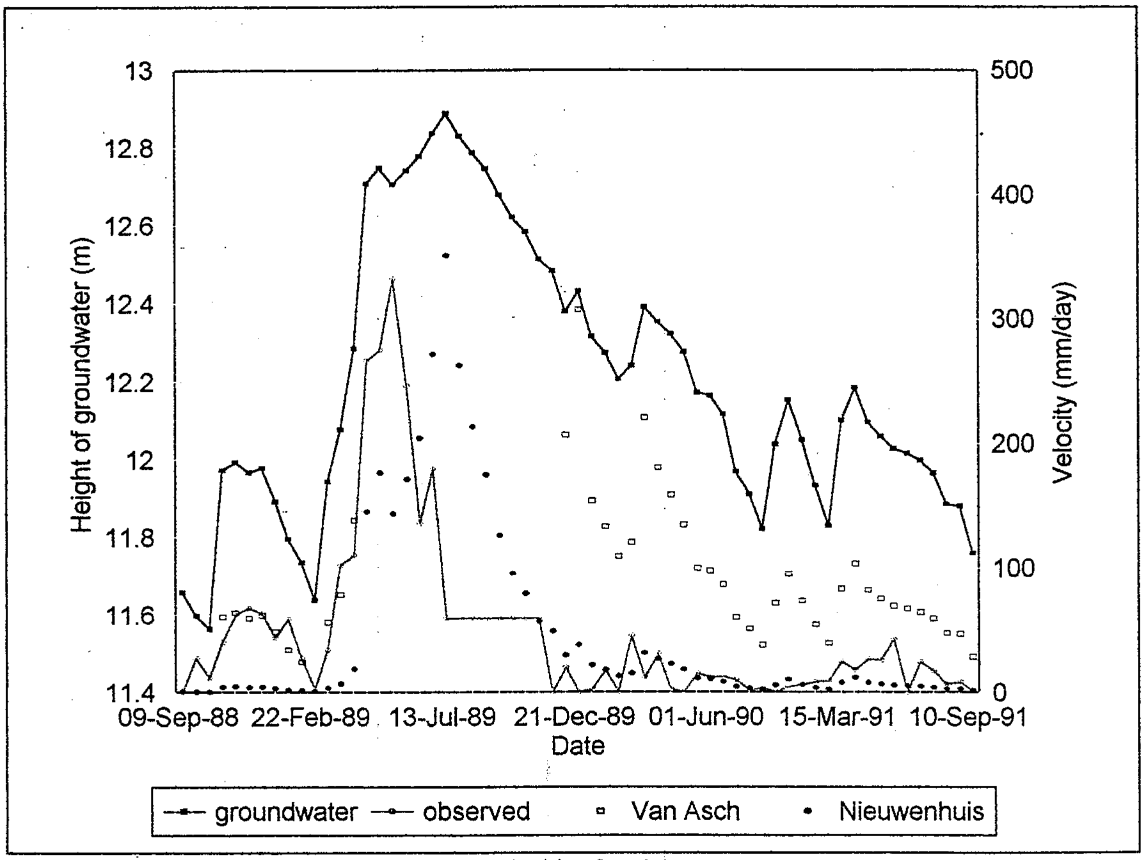

Figure 4: Model results: predicted and observed velocities for picket 5

CONCLUSIONS

Although the correlation between observed and predicted strain rate values is significant for the approach suggested by Nieuwenhuis (1991) and the actual values are approximated by it, the predicted values tend to lag with the observations. Better predictions should therefore be possible if a data-set for the calibration of the hydrological model were available. The model presented by Van Asch (in prep.) underestimates the contribution of viscous drag to the rate dependent shear strength. However, in the evaluation of the two dynamic models for the Valette landslide, the material, as reflected in the mobility characteristics, is highly sensitive to an increase in the shear stress ratio τ/τ_0. The total range of the inverse safety factor based on the linear store model ranges from 1 to only 1.04, what corresponds with a 1.3 m rise in groundwater level from 11.6 m at limiting equilibrium for residual strength conditions to 12.9 m. In this range the predicted and observed strain rates vary from total rest to 0.35 $m.d^{-1}$.
Under the present drained conditions catastrophic movement is unlikely to reoccur. However, given the sensitivity of the material to the shear stress ratio and the possibility of strain-softening, high strain rates can be achieved if more water can infiltrate. Therefore, in addition of the careful maintenance of the drainage network, continued monitoring of displacements and piezometric levels is required.

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LIST OF FIGURES AND TABLES

Table 1: Model parameters of the linear store tank model and of the infinite slope stability model

Parameters of the linear store tank model	
recharge parameter	100 %
multiplication factor for discharge area	2; after 1.10.89 : 1
depth of aquifer	20 m
k_{linear} = depth/k_{Darcy}	2325 days
porosity	n = 38%
Parameters of the infinite slope stability model	
residual shear strength	c_r' = 0 kPa; ϕ_r' = 24.5°
topography	inclination: 18° depth: 20 m