### 3.3.3 Pore water pressure changes in the saturated zone

A number of attempts have been made to relate rainfall to water table fluctuations, assuming saturated conditions (Jacob, 1943, Maasland (1959). They generally assume that a change in piezometric head $\Delta h$ following a rainfall event is given by

$$\Delta \mathbf{h} = (R/S) - Q$$

Where R is net infiltration (allowing for evapotranspiration), S is the specific storage (the volume of water taken into the soil structure in response to a unit change in pore water pressure) and Q is a term expressing groundwater recession. A recent example, (Sangrey et al 1984) uses the one dimensional Boussinesq continuity equation for a sloping, unconfined aquifer beneath a hillslope, ie where

$$S\ \partial h/\partial t = R + Q$$

$$\text{and}\quad Q = -\,k.h_a \times \partial^2 H/\partial x^2$$

Terms are as before except k is the permeability of the aquifer, $h_a$ is the depth through which flow occurs ($k \times h_a$ = transmissivity) H is the piezometric height, t is time, and x is the horizontal distance from the crest of the slope. The value of Q is difficult to estimate and Headworth (1972) has shown that the recession rate (decline of water level with time) in a piezometer can be used to obtain an appropriate value in practice. An estimate of S can then be included in the equation for calculating h. In this method the decline in water level, h is measured with time (dh/dt) to obtain a minimum value $h_m$. When time t = 0, $h = h_o$, and Headworth shows

$$(h - h_m) = (h_o - h_m)e^{-kt}$$

where k is a recession constant obtained from field readings. If the water level changes to $h_w$ due to a combination of recharge and recession then he also shows that

$$h_w = R/S + 2h_o - (h_o - h_m)e^{-kt} - h_m$$

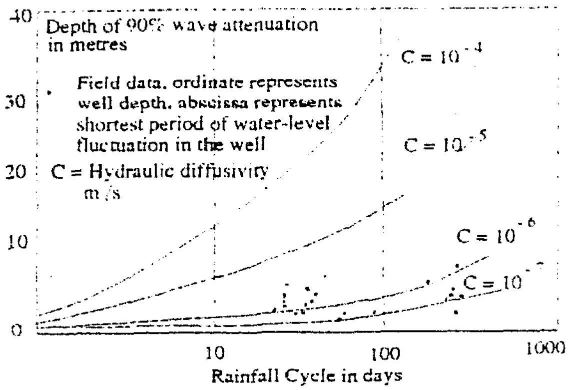


Figure 16. Rate of pore pressure decrease with depth as a function of rainfall cycle using field data from California (After Iverson and Major,1987)

Harper (1975) and Barton and Thompson (1986) have used this method as a basis for predicting groundwater levels in rock slopes and landslides.

The rate at which a change in piezometric head at the upper surface of the saturated zone travels down, through a landslide is considered by Iverson and Major (1987). They assume a transient head, h, at the ground surface varies sinusoidally with a given amplitude and frequency (to simulate a rainfall input). The solution to the one dimensional consolidation equation with this boundary condition suggests that the duration of rainfall cycles and the hydraulic diffusivity control the amplitude, speed and phase delay of pore pressures at depth. Figure 16 from their paper illustrates the theoretical depths at which pressure waves attenuate to 10% of their surface amplitude as a function of rainfall cycle. The figure indicates the important control exerted by the hydraulic diffusivity. Their calculations (with an appropriate value for diffusivity) show that weekly, monthly and yearly rainfall cycles take 2.5, 22, and 130 days to reach shallow, intermediate and deep wells in the landslide respectively (with a 90% pressure attenuation). Field measurements indicated that piezometric levels at depths between 0-3m, 3-6m, and 6-9m, fluctuated by 3m, 3m, and 0.5m, although the amounts varied in different parts of the slide. The results suggest that some pressure attenuation occurred beyond the depth of sliding, which was at 6m. The times of high piezometric head (delayed by a few weeks) corresponded with rapid landslide motion and they suggest that the critical water levels are a better predictor of landslide movement than rainfall.

Another example of the link between rainfall, groundwater levels and landslide movements in the saturated zone can be found in Skempton et al (1989). They have adopted an approach used in Hong Kong (Lerner, 1986) where piezometer readings are classified according to whether rainfall response is seasonal or due to storms. Figure 17 from their paper shows that groundwater levels in typical UK landslide sites (in clays) are at their lowest in October following summer recession, rising to a maximum in January and February after the winter recharge. The records suggest average seasonal fluctuations range from between 1.1 and 1.5m in slipped ground. However, long-term climatic trends may affect the seasonal rise from year to year. For example piezometric records published by Hutchinson and Gostelow (1976) in a landslipped London Clay slope at Hadleigh, UK illustrated that a negligible winter response was recorded following the dry summer of 1973. An increase of 1.3m occurred however in 1974/1975, following increased rainfall and water surplus.

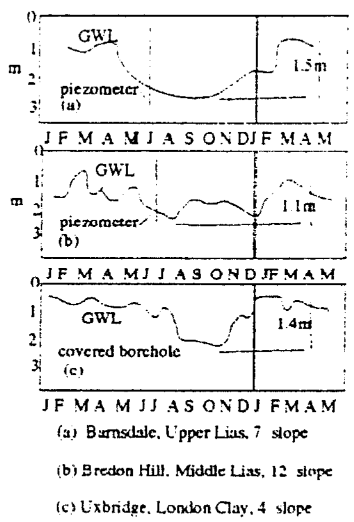


Figure 17. Typical seasonal variations in piezometric levels from landslides on clay slopes, UK (After Skempton et al 1989)

These seasonal fluctuations are not untypical of other areas. Ogawa et al(1989) quote 2-4m for slides in Japan, and Iverson and Major (1987) measured fluctuations of up to 3m in California. However, greater seasonal fluctuations are not unknown, for example, Skempton et al (1989) quote a figure of nearly 9m near the centre of the Mam Tor slide.

Landslide reactivation is usually caused by exceptional storm rainfall which raises the groundwater table (and porewater pressures) above the seasonal fluctuations. Skempton et al (1989) have introduced a simple parameter called the winter storm response ratio h/R where R is rainfall in mm and h is piezometric head increase in metres. They suggest, following Lerner (1986) that in saturated ground there is a linear relationship between the two parameters and this may be characteristic of any particular site. At Mam Tor they suggest h/R = 5 and that the transient rise of the water table following a winter rainfall event capable of initiating movement is about 0.5m.

Earlier discussions suggest that greater increases in pore pressure (head) are theoretically possible in the unsaturated zone (with this level of rainfall), but comparable seasonal or storm measurements are not yet available.

### 3.3.4 Discussion

Because of the difficulties of modelling porewater pressure increases it seems likely that simple field relationships between rainfall, piezometric head, and mass movement processes will be continue to be used to provide the basic 'stability rules' for regional hazard zoning in landslide susceptibility studies.

## 4. Failure Mechanisms

### 4.1 First Time slides and Post-Failure Displacement

#### 4.1.1 General

The increase in pore water ($u_w$) or air pressure ($u_a$) decreases the effective stress and reduces the average frictional strength of the soil and this ultimately leads to shear failure. Landslide morphology, mechanisms, and methods of analysis, which consider such factors as progressive failure, are described in engineering geology and geotechnical engineering texts, and have been dealt with in the accompanying lectures to this course. They will not be considered again, here.

### 4.1.2 Post Failure Displacement

Large post failure dispacements are commonly associated with first-time rainfall induced failures. A simple material parameter which predicts post failure behaviour is the brittleness index, $I_B$ (Bishop, 1973) where

$$I_B = S_f - S_r/S_f$$

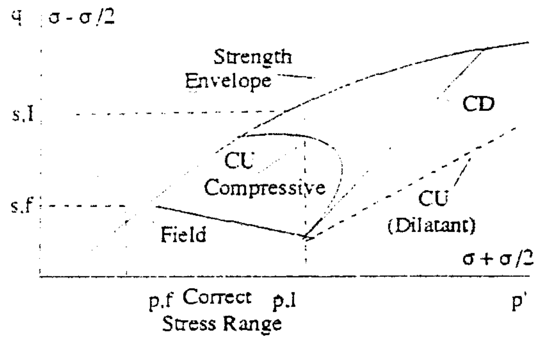


Figure 18. Stress path for field failure in comparison to those followed in conventional triaxial tests. I=the initial condition, CU=consolidated undrained, CD=drained tests (After Brand 1985)

$S_f$ and $S_r$ are the peak and residual shear strengths in terms of effective or total stresses.

High drained $I_B$ values are associated with high plasticity clays with a low residual frictional strength. Most of these are overconslidated and tend to dilate during shear, but they are, nevertheless susuusceptible to large post-failure displacements.

Brand (1985) has suggested that the soil structure and stress paths which are followed during infiltration may be important controlling factors in rainfall induced failure in lower plasticity materials. Figure 18 illustrates that failure takes place under constant total stress, unlike the conditions normally experienced in a conventional undrained or drained triaxial test.

Some low plasticity materials have a low relative density, high in situ void ratio, and a high undrained brittleness (see Gostelow, 1990, this volume). Stress controlled triaxial tests have shown that an almost complete transfer of the total normal stress to the pore water takes place if the soil structure is able to collapse. This could arise following rainfall infiltration and be responsible for the large post-failure displacements which are characteristic of the debris slides described earlier. Typical materials which have a high undrained brittleness are the sensitive 'quickclays' in Scandinavia and some residual or weathered soils. The latter are commonly associated with shallow, rainfall induced slides and Vaughan

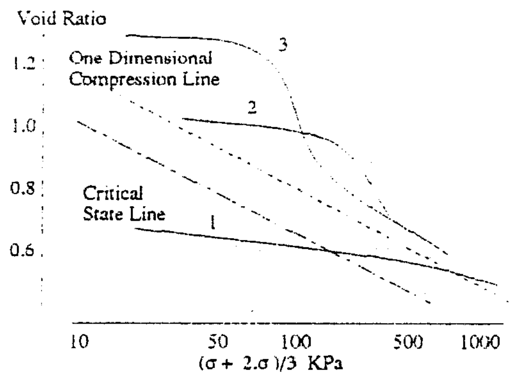


Figure 19. Compression curves for soils with different initial structure in relation to 'critical state' and one dimensional compression line (After Vaughan, 1989)

(1989), has suggested that three structural states, related to overall porosity and the critical state void ratio (the void ratio reached in granular materials at large strains, for a given confining pressure) can be recognised (fig 19):

1. Stable dilatant - Soil expands towards the critical state line, but can exist in a destructured state at the same stress level.

2. Stable contractive - Soil contracts during shear

towards the critical state line. but can exist in a destructured state at the same stress level.

3. Metastable - Soil exists at a void ratio which is impossible for the same soil in a destructured state or at the same stress level.

The metastable state has the highest void ratio and porosity, is also contractive, and Vaughan suggests that soil profiles which develop an open structure by weathering and the formation of grain to grain bonds are most prone to first time debris slides. Fig 19 summarises the three states for soils which have the same grading and bond strength, but different void ratios. The plot shows the rapid void ratio decrease in soil state three as the constant stress ratio in the triaxial test increases. Vaughan (1989) has proposed a simple index parameter. known as the relative void ratio, $e_r$, to help distinguish metastable soils prone to large post failure displacements. ie

$$e_r = (e - e_{opt})/(e_L - e_{opt})$$

where e is the in situ void ratio. $e_{opt}$, is the void ratio at optimum density and $e_L$ is the void ratio at the liquid limit. High values of $e_r$ are associated with metastable soils.

### 4.2 Other possible first-time failure mechanisms

Natural liquefaction by piping is a process which may also be triggered by rainfall. Hack and Goodlett (1960) use the term 'water blowout' to describe a number of circular holes up to 15m in diameter which they observed on slopes in the Appalachians after a heavy storm in 1949. Their positions were closely related to the underlying structure, ie impermeable rock bands, and groundwater conduits. Iverson and Major (1986) have analysed the conditions which are necessary to induce piping failures. which might arise from shallow groundwater flow. and show that they are theoretically possible only if groundwater flow lines are deflected upslope. This can theoretically occur in the vicinity of local groundwater discharge where high hydraulic graFdients are set up, for example from the underlying bedrock. This could be a cause of failure, although there are no field measurements to confirm the process. However, the hydrogeological role of macropores or subsurface channels in the soil mantle are still poorly understood, and these may provide the physical conditions for piping, or even hydraulic fracture in some soil types.

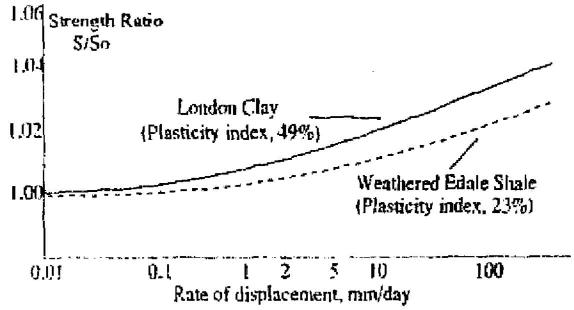


Figure 20. The effects of displacement rate on the residual strength of London Clay and Edale Shale (After Skempton, 1985)

### 4.2 Slides on pre-existing shear surfaces

Most slides of this type are close to a factor of safety of 1.0 and small increases in piezometric head (following rainfall) raise the pore water pressures on the shear surface and induce movement. Skempton et al (1989) call this a storm response movement. However. Fig 20 shows that the shear strength increases with the rate of displacement in clays of medium to high plasticity (examples are shown for the London Clay and Edale Shale at MamTor) where $S_o$ is the static strength corresponding to a factor of safety of 1.0. When the strength drops below 1.0 as a result of a piezometric head increase the displacement rate increases according to the relationship in the figure, but is limited by the associated increase in strength (S). Table 2 summarises the displacements for different storm rainfall assumptions and approximate monthly rainfall. It shows the head response and displacement assuming the initial water table is (i) at the level where the factor of safety is equal to 1.0 (ii) 0.1m below that level, and iii) 0.2m below that level. Skempton et al (1989) suggest that on a long time-scale. slides of this type become more stable because a greater piezometric rise is required

| Storm rainfall | Response | | Displacement (m) | | | Monthly |
|---|---|---|---|---|---|---|
| R mm | h/R | h/m | 0m | 0.1m | 0.2m | rainfall mm |
| 80 | 4.5 | 0.36 | 0.13 | 0.04 | 0.01 | 150-200 |
| 100 | 4.65 | 0.46 | 0.29 | 0.12 | 0.04 | 190-240 |
| 120 | 4.8 | 0.60 | 0.60 | 0.28 | 0.10 | 230-280 |
| 140 | 4.9 | 0.69 | 1.2 | 0.60 | 0.25 | 270-320 |

Table 2. Figures from Mam Tor Derbyshire how storm and monthly rainfalls have been related to piezometric levels and landslide displacements (After Skempton et al. 1989)

for a given displacement. Eventually a stage is reached when the slide is stable under all but the most extreme conditionst

Discrete shear surfaces do not form in materials of low plasticity (Lupini et al. 1981) and the same rainfall control does not apply. The strengths of most debris flows at the time of failure are controlled by strain-induced pore pressures and, as suggested above, the frictional strengths at failure are probably less than those corresponding to a drained residual friction angle.

## 5. Identification of Susceptible Slopes - Engineering geology mapping

The previous sections have shown that it is difficult to consider the role of rainfall as a landslide trigger without taking into account the nature of the ground conditions. Two broad groups can be identified. 1. Those slopes greater than about 25°, which consist of weak, brittle (in the soil mechanics sense) materials ansd exhibit the greatest water pressure increases can be defined as susceptible to rainfall-induced, first-time failure. 2. Any slopes, close to a factor of safety of 1.0 which are underlain by pre-existiing slides with high plasticity shear surfaces are also susceptible to periodic deformation. A technique for identifying and zoning critical areas (from most to least susceptible), which is able to take into account a number of relatred factors, is thus required. The most important of these can be listed as:

1. Local Climate

2. Frequency of 24 hour rainfalls greater than 100 mm

3. Topography/slope angles and geometry

4. Soil classification/structure

5. Depth of water table (saturated zone) and regional hydrogeology

6. Geotechnical properties, including stress-strain characteristics and shear strength (see further discussion in Gostelow, 1990, this volume)

7. Potential change in pore-water pressure in i) the unsaturated zone, and ii) saturated zone

These factors are spatially variable and can ideally be presented in map form. There are a number of publications which provide guidelines for the mapping of landslips, and areas susceptible to mass movement. However, the UNESCO review by Varnes (1984) considers examples from several countries and provides the best overall summary of the techniques currently available. Two additional approaches, which have been developed recently, in Hong Kong and California are briefly described here.

Hong Kong:

Regional terrain classification and susceptibility studies have been carried out in the dominantly urban area of Hong Kong (Brand, 1988) at different scales, as an input to strategic planning. For example, at 1:2500 scale maps are presented as a series of overlays and include i) terrain classification, ii) surface hydrology, iii) vegetation, iv) engineering data, v) engineering geology, vi) and geotechnical land use. They have also experimented with a computer based geotechnical terrain classification system.

An overall geotechnical assessment is made in which stability considerations form a part. The information is summarised on an interpretative map of four classes based on a graded system of five characteristics: i) geotechnical limitations, ii) suitability for development, iii) engineering development costs, iv) extent of site investigation required, and v) terrain characteristics.

California:

A terrain evaluation system specifically for first time slides has recently been developed in California by Ellen et al (1988) for planning purposes. They make a basic distinction between 'hard' and 'soft' terrains. The former are underlain by hard intact rock and form steep slopes. The latter are more rounded and consist of weak, sheared, and broken materials with lower overall slope angles. Intermediate forms are also recognised. These divisions are useful, because each unit represents a combination of material strength and topographic form.

The terrain units are sub-divided into three basic slope habitats, i) amphitheatres, or hollows, ii) slopes adjacent to non-alluvial channels, but without amphitheatres, and iii) slopes which adjoin alluvial channels. These habitats are very similar to those originally recognised by Hack and Goodlett (1960) in the Appalachians, although they also used two further categories, ie, iv) channelways,- a continuation of the hollow, and v) footslopes, which form a link between the channelway and habitat (ii) above. Evidence collected so far in California suggests most first-time slides are associated with habitat (i) environments.

These layered, hierarchical approaches to mapping, ie using map overlays, and identifying different slope types or habitats likely to contain soils of different soil moisture characteristics within larger terrain groupings is potentially useful for relating to the environmental factors reviewed here. A number of pathways can be established which contain data sets at different levels. These can be used in appropriate stability models for establishing the most and least, susceptible areas to rainfall-induced sliding. However, this approach is difficult and time consuming to carry out with conventional maps, where data (usually from several sources) must be extracted manually, and entered, or re-entered into the models. It is therefore anticipated that future susceptibility mapping will increasingly use computer based Geographic Information System (GIS) technology. This is particularly suited for interfacing large spatially referenced datasets with physical models. The data sets, modelling and susceptibility zonation can also be continually updated as new developments are made and local requirements change.

## 6. Conclusions

This review has shown that a number of interelated factors control the susceptibility of slopes to rainfall induced mass movement. The most important of these have been discussed above and must be considered in engineering investigations and regional planning studies. They can be listed under broad headings as:

1. Rainfall statistics

2. Geology and geomorphology

3. Brittleness of slope forming materials (first time slides)

4. Presence of pre-existing slides consisting of medium to high plasticity materials.

5. Regional hydrogeology

The relationship between rainfall and pore-water pressure change is difficult to predict from theoretical considerations, althiough two separate effects can be identified

1. A comparatively rapid change of pressure in the unsaturated zone (where present) which reduces suctions to either zero or, craetes a positive pressure. A rainfall intensity in excess of the surface hydraulic conductivity and sufficient to induce ponding favours loss of suction. In a homogeneous soil the antecedent soil moisture condition affects the rate at which suction is lost (to a zero value and saturated/ nearly saturated conditions).

2. A delayed increase in the water table level at depth, which results in a positive (hydrostatic) rise in the pore-water pressure. Shallow water tables favour the greatest and most rapid increase in level following rainfall.

Rainfall data from a number of countries suggest that there is no general threshold for landsliding, although high intensity storms within wet periods which result in 24 hour totals of 100mm or more seem to be most critical. If the models are correct then this may reflect the figure which is generally necessary to induce infiltration and ponding in weathered soil profiles. However, there are few examples of pore-water pressure measurements in the unsaturated zones of both landslips and slopes susceptible to landsliding. Further research is required before the theoretical models can be used confidently for prediction.

Many of the worlds most destructive landslides have been triggered by rainfall, and it is suggeated that advances in the understanding of their spatial and temporal distribution will follow climatic and terrain analysis of past and potential slides within susceptible areas using Geographic Information System databases (GIS).

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Course on

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# 4. LANDSLIDE ANALYSIS AND LAND USE LEGISLATION