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Evaluation of landslide triggering mechanisms in model fill slopes

Abstract Hong Kong is particularly susceptible to landslide risk due to the steep natural topography and prolonged periods of high intensity rainfall. Compounding the risk of slope failure is the existence of loose fill slopes which were constructed prior to the 1970's by end-tipping. A clear understanding of the underlying triggering mechanisms of fast landslides in fill slopes is required to analyse landslide risk and to optimise slope stabilisation strategies. The work described here had the objective of evaluating two candidate triggering mechanisms—static liquefaction and the transition from slide to flow due to localised transient pore water pressures—against observations of slope behaviour obtained from highly instrumented centrifuge model tests. These results indicate that static liquefaction is unlikely to occur if the model fill is unsaturated and the depth to bedrock large, as the high compressibility and mobility of air in the unsaturated void spaces allows the model fill slope to accommodate wetting collapse without initiating undrained failure. In contrast, high-speed failures with low-angle run-outs are shown to be easily triggered in model fill slopes from initially slow moving slips driven by localised transient pore water pressures arising from constricted seepage and material layering.

Keywords Triggering · Slope stability · Rainfall · Centrifuge tests · Layered soils

Introduction

Hong Kong is particularly susceptible to landslide risk due to the steep natural topography and the ability of the wet season to produce storms with a daily rainfall commonly exceeding 200 mm. The development of Hong Kong in the first half of the 20th century led to the cutting of benches and the end-tipping of spoil to create loose fill slopes. These fill slopes often failed, sometimes with severe consequences. On 18 June 1972, heavy rainfall triggered a small slip, and 30 minutes later a catastrophic slide-flow, in completely decomposed granite (CDG) fill at Sau Mau Ping in Kowloon, resulting in 71 deaths and 60 other injuries. On 25 August 1976, another large-scale slope failure occurred at Sau Mau Ping. Although the slide mass was fairly shallow, the debris buried the ground floor of an apartment block at the toe of the slope killing 18 people. An investigation after the 1976 slide concluded that the failure was caused by the infiltration of rain water into the slope, leading to a loss of strength in the fill, envisioning the instantaneous conversion of the slope into a mud avalanche (Knill et al., 1976). However, eye-witness accounts speak of the emergence of localised seepage prior to the initial land slip in the first case: Chen et al (2004). Furthermore, many slips have occurred previously at Sau Mau Ping and other similar sites, without triggering a flowslide. Two questions arise. What triggers a slip? What triggers a flowslide? Two contrasting hypotheses will be explored.

Static liquefaction hypothesis

The mechanism most often mentioned in relation to the fast landslides witnessed in fill slopes in Hong Kong is soil liquefaction. Static liquefaction is a phenomenon seen in triaxial tests on very loose, saturated, collapsible soils which are unable to drain when their volume attempts to reduce due to shearing. When subject to an initial increase of shear stress τ (equivalent to deviator stress q in a triaxial test) or a reduction of effective normal stress σ' (or mean effective stress p' in a triaxial test), the soil state approaches a region of instability in which the shear strength can suddenly reduce to zero. Typical undrained test paths to static liquefaction for very loose sands are shown as effective stress paths 1, 2 and 3 in Fig. 1a. Once liquefied, the granular skeleton loses almost all its effective stress and the soil flows rapidly if the applied shear stress is maintained.

For very loose saturated soils, instability is triggered on the line IL which lies below the critical state line CSL. The critical stress ratio ϕ_{CSL} in a direct shear test is defined as the ultimate (residual) angle of friction of sands irrespective of their initial density. Ordinarily, soil can not fail if it mobilises $\phi \leq \phi_{CSL}$ and can temporarily mobilise $\phi > \phi_{CSL}$ if its relative density exceeds 20%. The problem on the Instability Line is not loss of ultimate friction ϕ_{CSL} but rather the sudden creation of excess pore pres-

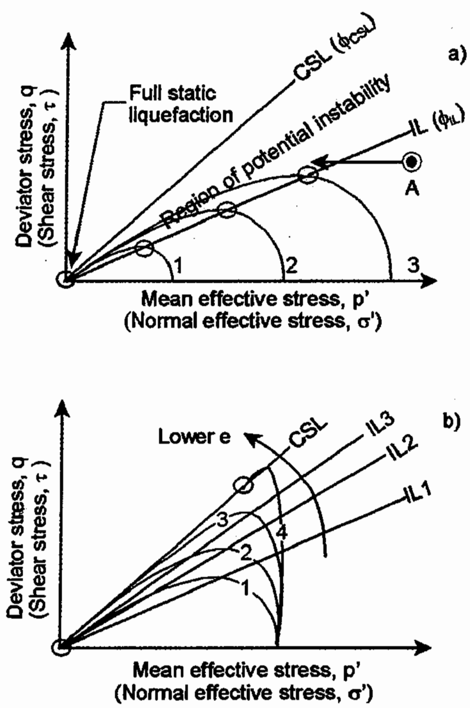


Fig. 1 Static liquefaction landslide triggering mechanism

sure when very loose saturated soil suffers a collapse of structure when it was mobilising $\phi_{IL} \leq \phi_{CSL}$. Lade (1993) indicates that the instability line may be estimated experimentally by joining the points of highest shear stress from multiple undrained test paths. These points are shown in Fig. 1a, as open circles, one for each of the three effective stress paths beginning at numbers 1,2, and 3 and ending with full static liquefaction (zero shear stress). As the field stress path of a slope during wetting is approximately horizontal (stress path A), it has been conjectured that the slope will experience instability if the magnitude of the pore pressure increase due to infiltration causes the stress path to cross the current instability line, triggering a further burst of pore pressure.

Further exploration of this phenomenon has indicated that the position of the instability line is not unique but, rather, is a strong function of voids ratio (Chu et al, 2003) and therefore stress path (Ng et al, 2004). Following the work of Chu et al (2003), the variation of the position of the instability line in stress space (q-p', or τ-σ') is summarised in Fig. 1b for very loose (stress path 1) to loose (stress path 4) voids ratios. As the voids ratio decreases (i.e. the soil becomes a little denser) the slope of the instability line (defined as M_{IL} in q-p' space, or ϕ_{IL} in τ-σ' space) approaches the critical state friction angle, ϕ_{CSL}. At the lowest voids ratio (stress path 4), the soil no longer experiences full static liquefaction, but rather a small reduction in the available undrained shear strength. Such behaviour has been called temporary liquefaction by Yamamuro and Lade (1997), but this term is misleading since most soils soften to a certain extent after achieving their peak strength, and the large residual shear strength cannot properly be regarded as that of a liquid.

The static liquefaction hypothesis is therefore that certain slopes in very loose material transform from stable quasi-elastic mounds of earth, directly to unstable flowslides. Other slightly denser slopes simply slip. This draws attention to the looseness of the slopes as the key element in determining whether life-threatening flowslides occur or not.

Slide-to-flow transformation due to groundwater ponding

An alternative hypothesis (Bolton et al, 2003) is that the rare but damaging Hong Kong flowslides begin with a local slip triggered by transient pore water pressures arising from seepage flow restrictions or from soil layering. The geometry of the slip and the nature of the slowly moving debris may then cause a fast landslide. One such slide-to-flow scenario is presented in Fig. 2. Here, a fill slope has been constructed by end tipping spoil down a pre-existing slope of lower permeability material. The hydrologic equilibrium of such a slope is considered in Fig. 2 as comprising four interdependent processes:

a) *Unsaturated infiltration.* Slopes are typically unsaturated. If rain falls on the exposed slope surface at a rate less than the permeability of the slope, the water will percolate vertically downwards into the fill material without causing large positive pore water pressures.

b) *Saturated transmission.* If the permeability of the underlying material is much lower than the fill material, the infiltrating water will form a thin seepage layer which will follow the contours of the low permeability material. As the flow quantity must increase with distance along the slope catchment area due to the integration of the rainfall infiltration applied over this area, the depth of the seepage layer will correspondingly increase in the down-slope direction.

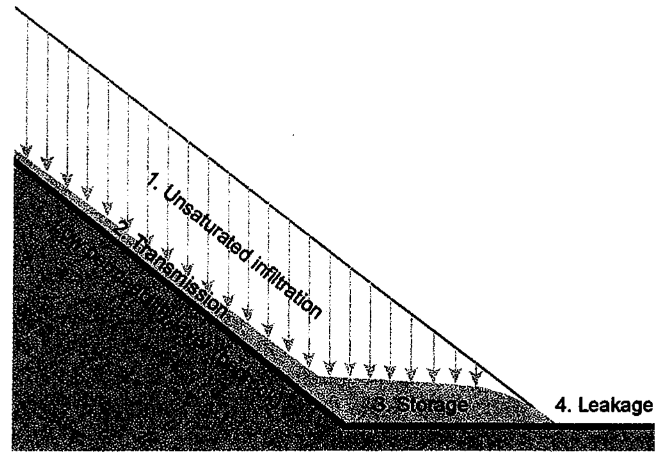


Fig. 2 Slide-to-flow landside triggering mechanism

c) *Storage.* Any significant change in the inclination of the lo permeability layer will result in a localised change in the hydrauli gradient. Darcy's law states that

$$\frac{Q}{A} = v = ki$$

where, Q is the flow rate in m^3/s, A is the area of flow in m^2, v i the flow velocity in m/s, k is Darcy's permeability in m/s, and i i the hydraulic gradient, herein defined as the difference i piezometric height between two points as measured in Earth' gravity divided by the distance along the flow path separatin them.

Since the mass of water through a control volume must b conserved, a consideration of Darcy's law at each hydraulic gradient dictates that the area of flow must correspondingly change If, as in our hypothetical triggering mechanism, the slope of th low permeability layer decreases, a localised zone of high transient pore water pressure must be created within the fill material By definition, one area of the slope in which the gradient mus change significantly is the toe of the slope. This is also arguabl the most critical region of the slope from a stress perspective.

d) *Leakage.* Using similar logical arguments to that describe above, it can be shown that the exit gradient of the seepage flov will also influence the transient pore water pressures within th fill.

The elevated pore water pressures from such a scenario will b temporary, localised, and a function of the relative rates of each o the interdependent processes, in particular the rainfall intensity duration, and antecedent conditions.

Transient pore water pressures in fill slopes may also be th consequence of soil layering. It is widely recognised that fil materials placed by end tipping have the propensity to form layered deposits, depending on the heterogeneity of the source materials. Upon reaching a permeable layer, infiltrating rain wate will be preferentially transmitted downslope within the permeabl layer. If the inclined layer is close-ended, the rate of transmissior of seepage water to the end of the layer can exceed that of leakage from the layer, thereby ensuring a local transient build up of pore water pressures.

Experimental Methodology

Testing Programme

A good understanding of the underlying triggering mechanisms of fast landslides in fill slopes is required to analyse landslide risk and to optimise slope stabilisation strategies. The identification of these mechanisms of failure from instrumented model tests has been a recent research theme at Cambridge University, an overview of which being presented by Bolton et al (2003). In this paper, the experimental results are analysed in greater detail with the objective of evaluating the two candidate triggering mechanisms introduced in the previous section against observations of slope behaviour in centrifuge model tests. The first test aims to reproduce static liquefaction, whereas the second and third tests investigate the transition process from slide to flow.

Principles of centrifuge modelling

The physical modelling of landslide processes in the laboratory can potentially create well documented, highly-instrumented case-studies of slope behaviour in which the material properties, initial state, and boundary conditions are all well defined. However, practical constraints such as laboratory space, personnel, and large material quantities make full-scale (or even slightly reduced scale) modelling prohibitively costly. However, the reproduction of the process at a greatly reduced scale is widely acknowledged to be not fully representative of full-scale behaviour due to the difference in stress levels between model and prototype. In particular, the capillary suction of damp silty sand will be out of proportion with its self-weight stress, allowing the model slope to remain steeper than would be possible at higher effective stress levels. In other words, a 200 mm high model fill slope (Fig. 3), will be more stable than a fill slope of 10 m in height. However, if our 200 mm high slope is subjected to an acceleration field N times greater than Earth's gravity, g, each soil particle or water droplet in the model fill slope will weight N times greater. As a result, the gradient of body stresses within our reduced scale model will now be similar to a fill slope N times larger, ensuring similarity of effective stresses and groundwater pressures at equivalent depths in model and prototype.

The model fill

Decomposed granite is the typical material found in Hong Kong for fill slopes. Bulk samples of this soil were obtained from one such fill slope at Beacon Hill in Hong Kong, and transported to Cambridge for use in the model fill experiments. Index tests performed in Hong Kong by the Public Works Central Laboratory on this material indicate that the soil is a slightly clayey, silty, very gravely sand (GEO, 1999) with a mean grain size of 1.1 to 1.4 mm and approximately 16% of relatively low-plasticity fines (plasticity index of 11–14%). To reduce the unrealistic influence of large particles on the behaviour of the small scale fill slope, the fill material was first sieved to remove all particles in excess of 5 mm in diameter. Proctor compaction tests indicate that the maximum dry density of the soil is 1,920 kg/m^3 and is achieved at an optimum water content of approximately 11–12%. In drained triaxial compression tests, the fill material was observed to have a critical state angle of friction of 34 degrees. When placed very loose, similar decomposed granite fill materials have a well documented susceptibility to static liquefaction in saturated triaxial tests (e.g. Ng et al, 2004) and a history of failure in the field (Knill et al, 1976). Undrained triaxial tests performed on very loose samples of the Beacon Hill decomposed granite (relative compaction about 70% of Proctor optimum compacted density, circa 0% relative density) indicate that the model fill material is similarly collapsible with an instability line inclined at 22 degrees. Thus, if it is maintained at this large void ratio of about unity while it becomes saturated, and if collapse is so fast that excess pore pressures can not dissipate, static liquefaction could occur at a stress ratio much lower than the critical state.

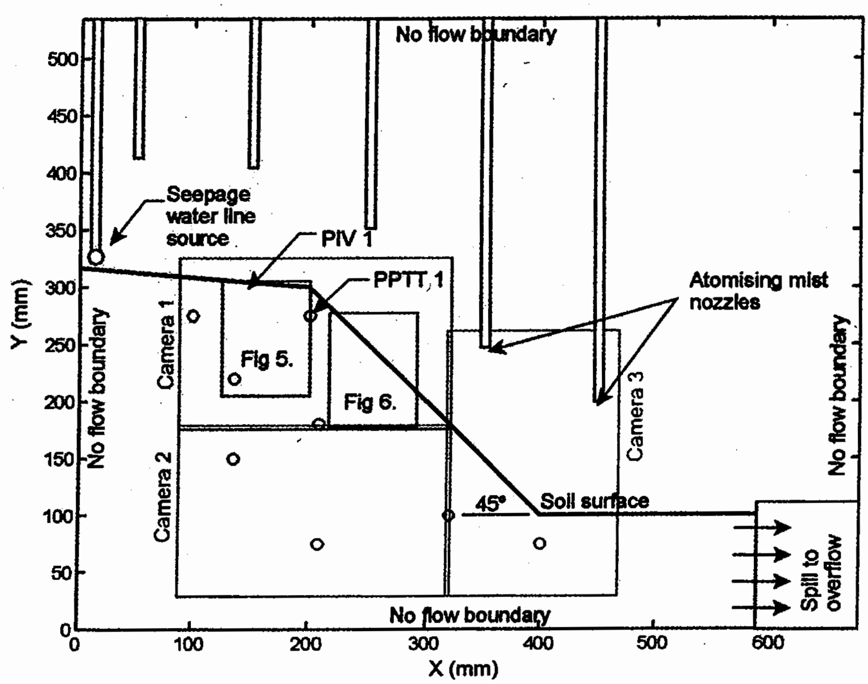


Fig. 3 Static liquefaction model test geometry

Boundary conditions

Both of the candidate triggering mechanisms for instability arise from rainfall infiltration. This boundary condition was reproduced directly in the model test investigating the mechanism of static liquefaction by uniformly applying atomised mist in two rows of five nozzles placed just above the soil surface (Fig. 3). The size of the nozzles was chosen to provide small droplet sizes and to reproduce a continuous rainfall intensity representative of a severe weekly rainfall in Hong Kong (4.2 mm/hr).

If the grain size distribution of the fill material is not reduced by the scale factor N, Darcy's permeability, k, will be identical between model and prototype. Since stresses (or water pressures) in the prototype are the same as those in the model, and mindful that in the centrifuge all prototype lengths are reduced by a factor of N, hydraulic gradients in the model will be N times greater. Therefore velocities of seepage, or rainfall infiltration, must be equally scaled up by a factor of N in the centrifuge model. However, for Darcy's Law to remain valid, this increased velocity of internal seepage must not compromise the assumption of laminar flow. Following the work of Goodings (1979), it can be shown that the seepage flow regime will remain laminar for centrifuge models of decomposed granite fill slopes tested at sixty gravities. Therefore, an applied mist intensity of 250 mm/hr provided uniformly to the surface of the model fill slope will be equivalent to the target rainfall infiltration of 4.2 mm/hr. The recent work of Rezzoug et al (2004) has confirmed that these scaling relationships equally apply to unsaturated flow.

Rather than directly model rainfall infiltration in the second series of model tests on slide-to flow transformation, seepage water was simply introduced at the crest as a line source to permit the visual observation of the progress of the seepage front.

Finally, to avoid unintentional drying of the model during centrifuge testing, the experiments were conducted in a sealed atmospheric chamber (Take and Bolton, 2002). As shown in Fig. 3, a no-flow boundary condition is provided on all boundaries of the model fill slope, with the exception of the bottom right-hand corner in which seepage water is allowed to spill to overflow once the water level has reached the elevation of the toe of the slope.

Instrumentation

The model fill slope is heavily instrumented to quantitatively capture the pore water pressures and deformations at triggering. Pore water pressures are measured using a network of new miniature (7 mm in diameter) pore pressure and tension transducers (PPTT) buried within the model fill at each of locations indicated by open circles in Fig. 3. After a conscientiously applied programme of saturation, these devices are capable of reliably measuring negative water pressures as low as −140 kPa, when fitted with a nominal 1 bar ceramic filter (Take and Bolton, 2003).

The deformations of the model fill slopes have been measured by a new image-based system of deformation measurement which combines the technologies of digital imaging, the image processing technique of particle image velocimetry (PIV), and close-range photogrammetry (White et al, 2003). Using this new technique, digital images captured through the transparent window of the atmospheric chamber are compared through time, allowing the determination of displacements at potentially thousands of points in each camera view of the model by tracking the soil texture (the unique way the grains are orientated at every location in the model) without resorting to embedded target markers.

Evaluating the static liquefaction mechanism

Model geometry

The first candidate landslide triggering mechanism to be investigated by physical modelling is static liquefaction. For this model fill slope experiment, the geometry and boundary conditions were designed to encourage static liquefaction to occur. The model fill was constructed of CDG, moist-tamped to form a fill slope with only a minimal compaction effort, and inclined at an over-steepened slope angle of 45 degrees (Fig. 3). Also, the large depth of the fill material represents a relatively limitless depth to bedrock, allowing the assessment of possible static liquefaction triggered in an unsaturated material solely by rainfall infiltration.

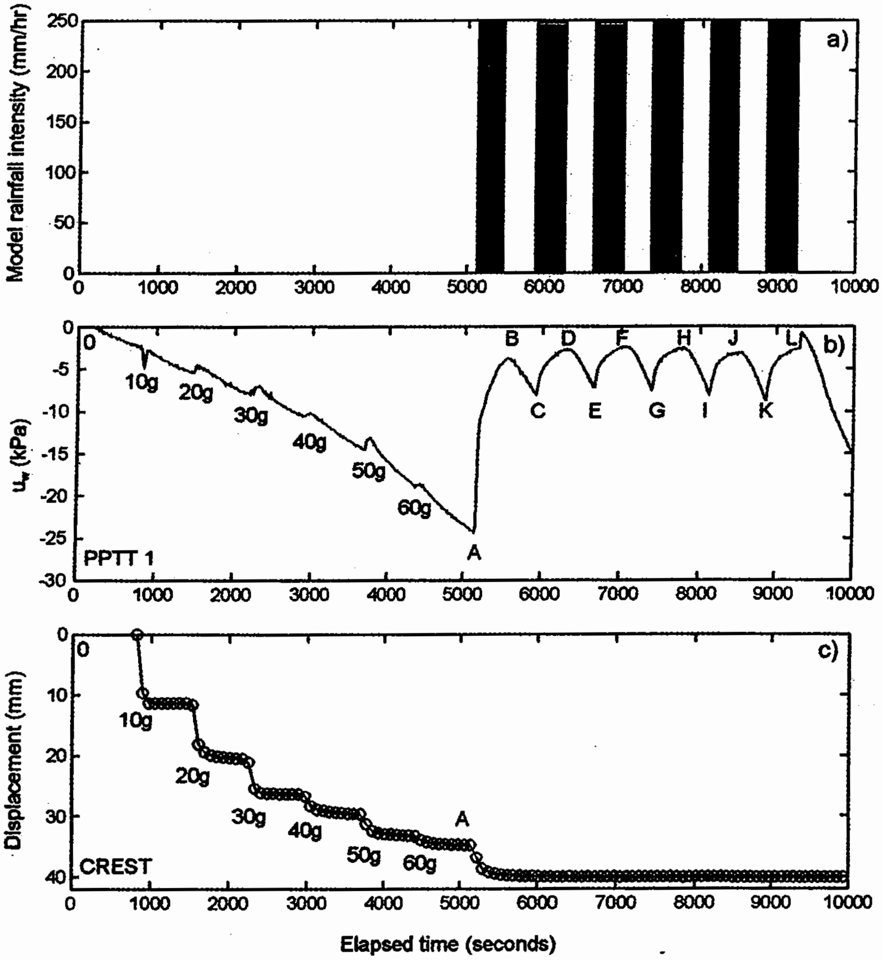
Initial conditions

Upon finishing the model fabrication process, the model fill slope was installed in the atmospheric chamber, sealed from the external environment, and allowed to come into moisture equilibrium with chamber air. This moisture transfer was observed to result in an initial suction throughout the model of approximately 2.5 kPa. The atmospheric chamber was then installed on the 10-m diameter beam centrifuge at Cambridge University, and slowly brought to the testing acceleration of 60 g in 10 g increments. The response of the very loose fill slope to the stepwise increase in effective stress is summarised in terms of observed pore pressure and displacements of the crest region (tensiometer PPTT1 and 32×32 pixel patch PIV1) in Figs. 4a and 4b, respectively.

As the loose fill material becomes incrementally heavier, a cumulatively larger percentage of the loose fill can no longer support this increase in total stress and rapidly decreases its void ratio. The reason for this very compressible behaviour becomes abundantly clear when the initial fabric of the soil is visually inspected. Photographs of the initial moist-tamped structure of the model fill at the crest and slope (see locations in Fig. 3) are included as Figs. 5 and 6, respectively. In part "a" of these figures, the very loose soil is observed to have an initially very open structure which consists of large voids supported by capillary suction. One such macro-void is highlighted in Fig. 5a. By the time the self-weight of the individual grains has increased sixty times, many of these macro-voids have been observed to collapse (Figs. 5b, 6b). However, not all voids have collapsed. In particular, voids at low stress levels (i.e. shallow depths) such as the highlighted void of Fig. 5a have simply settled along with the fill.

At each increment of effective stress, the rapid reduction in voids ratio is observed to cause a small increases in pore water pressure on the order of 1–2 kPa (Fig. 4b) despite experiencing large volumetric strains. Considering the low degree of saturation of the fill, it is not surprising that the void spaces are very compressible. As increments of "gravity" are progressively turned on, the height of the fill above the phreatic surface effectively becomes higher, requiring progressively higher capillary forces to be developed if the soil moisture is to be retained. As a result, the biggest pores shed their pore water vertically downwards into the fill, creating an initial suction distribution which increases with

Fig. 4 Observed behaviour of static liquefaction model



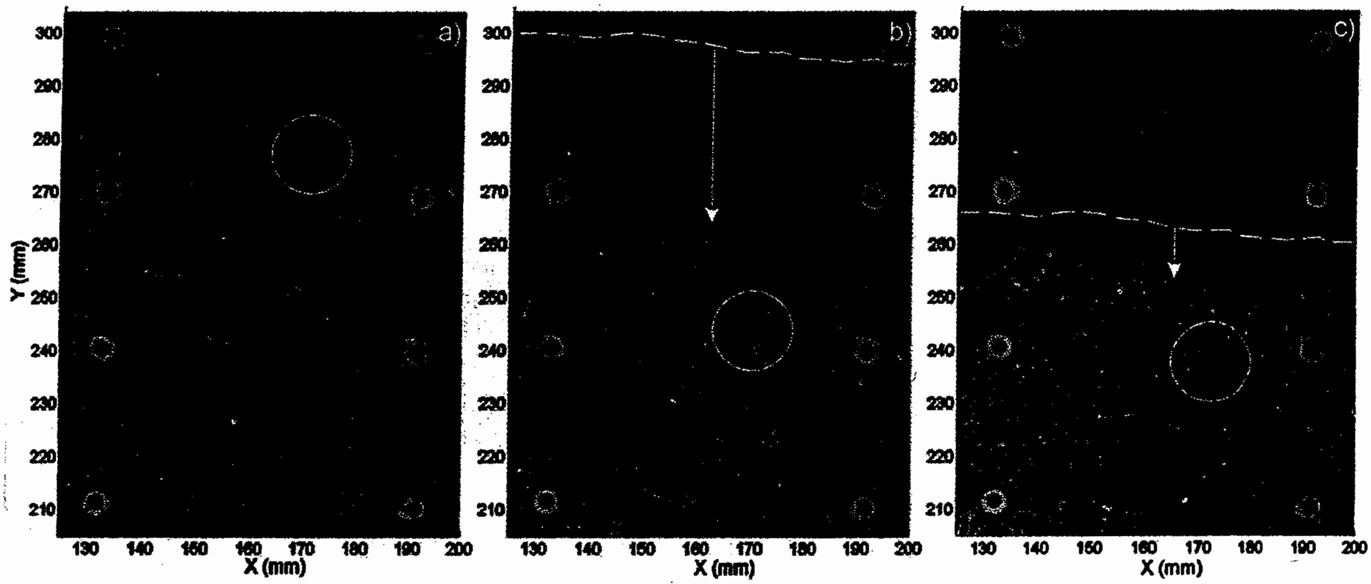


Fig. 5 Soil structure of crest region

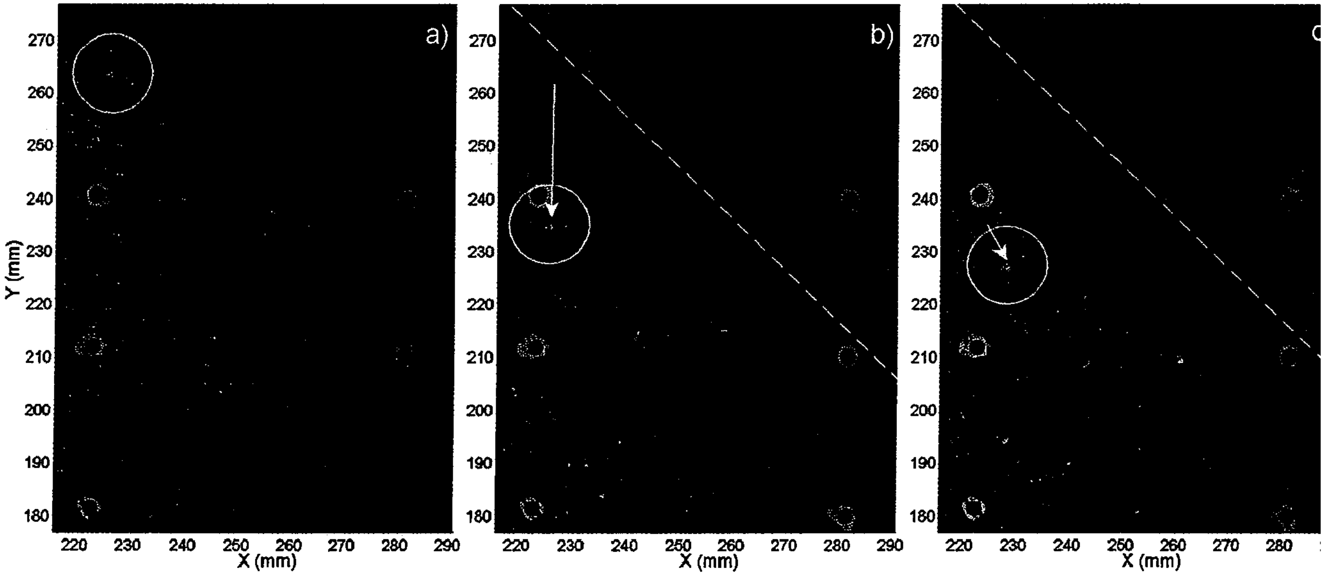
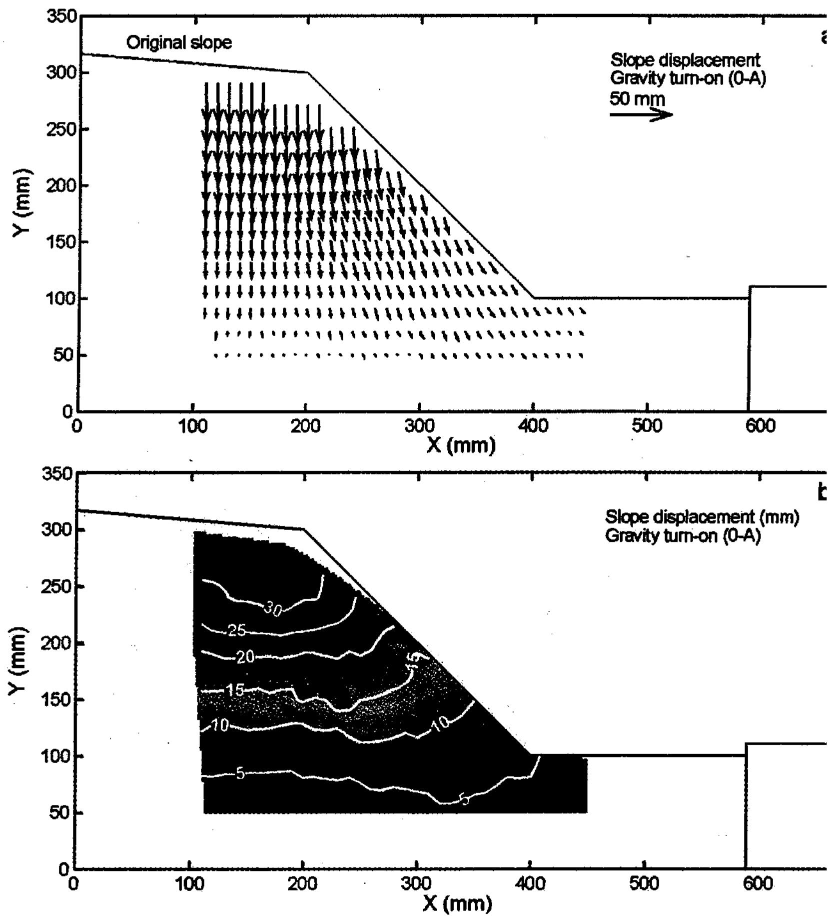


Fig. 6 Soil structure of slope region

Fig. 7 Self-weight settlement of loose model fill slope



elevation from a value of approximately zero at the toe to −25 kPa at the crest (PPTT1 in Fig. 4b).

The total settlement of the loose fill embankment analysed from the three camera views of the model fill during the process of gravity turn-on is shown in Fig. 7a with no vector magnification. Displacements are predominantly vertical, with a maximu vector magnitude of 34 mm at the crest. These large settlemen correspond to a settlement of over 2 m at prototype scale ar represent a vertical strain of 10% when averaged over the enti fill height. The high compressibility observed in these loadi increments indicates that the entire fill slope is on its normal compression line. That is, it is impossible to have a looser slope at the current effective stress levels.

Response to rainfall infiltration

After the initial self-weight consolidation phase of the model test, the fill slope was subjected to the equivalent of six weekly periods of rainfall infiltration (Fig. 4a). As shown in Fig. 4b, the arrival of rainfall on the slope surface at time A destroys a significant portion of the soil suction very rapidly at the shallow location of PPTT1. The loose model fill responds immediately to this loss of surface tension, by collapsing the macro-voids which had survived self-weight consolidation (Fig. 5c). The resulting settlement of the bench above the fill slope is purely vertically downwards. Similarly, the loss of suction in the region of static shear stress (i.e. beneath the sloping portion of the embankment) also causes macro-voids to collapse, now with a significant down-slope displacement (Fig. 6c).

As rainfall infiltration continues, the rate of suction loss decreases, as does the rate of settlement. Further rainfall infiltration results in the development of a water table at the toe of the slope—the prescribed elevation of the overflow control. Above this elevation, the slope is experiencing vertical percolation at roughly zero pore water pressure.

By the time the model has been subjected to rainfall for an equivalent duration of 1 week (Time B in Fig. 4b), the rate of pore water pressure increase has diminished to almost zero, becoming asymptotic to a small negative value. At this point, the mist nozzles were turned off and the fill slope was allowed to experience gravity drainage for an equivalent period of one week.

Despite being subjected to an additional five infiltration events of identical severity, the model slope was observed neither to achieve positive pore water pressures nor to experience any significant additional deformation.

The spatial distribution of settlement vectors presented in Fig. 8 clearly defines the region of instability observed during the first rainfall event. This region is coincident with the region of lowest total stresses (i.e. near the slope surface). Despite the vast majority of the infiltration settlement having occurred at low stress levels, a 0.2 mm contour interval has been included in Fig. 8 to provide a total depth of influence of the wetting collapse.

Discussion of observed behaviour

Although the fill has experienced instability, a flow failure has not been triggered by the mechanism of static liquefaction. In triaxial tests on loose, saturated, samples of the same fill material, static liquefaction was observed to occur when the model fill was unable to drain when its volume attempted to reduce. The fill material in the model test remained very loose near the surface despite being subjected to an increased self-weight. It is likely, therefore, that the difference in observed behaviour relates to the model fill slope's ability to accommodate the wetting collapse through the compressibility and ease of migration of the pore air. Thus, the unsaturated nature of the fill material, and the lack of groundwater ponding, has eliminated the possibility of creating static liquefaction in this model.

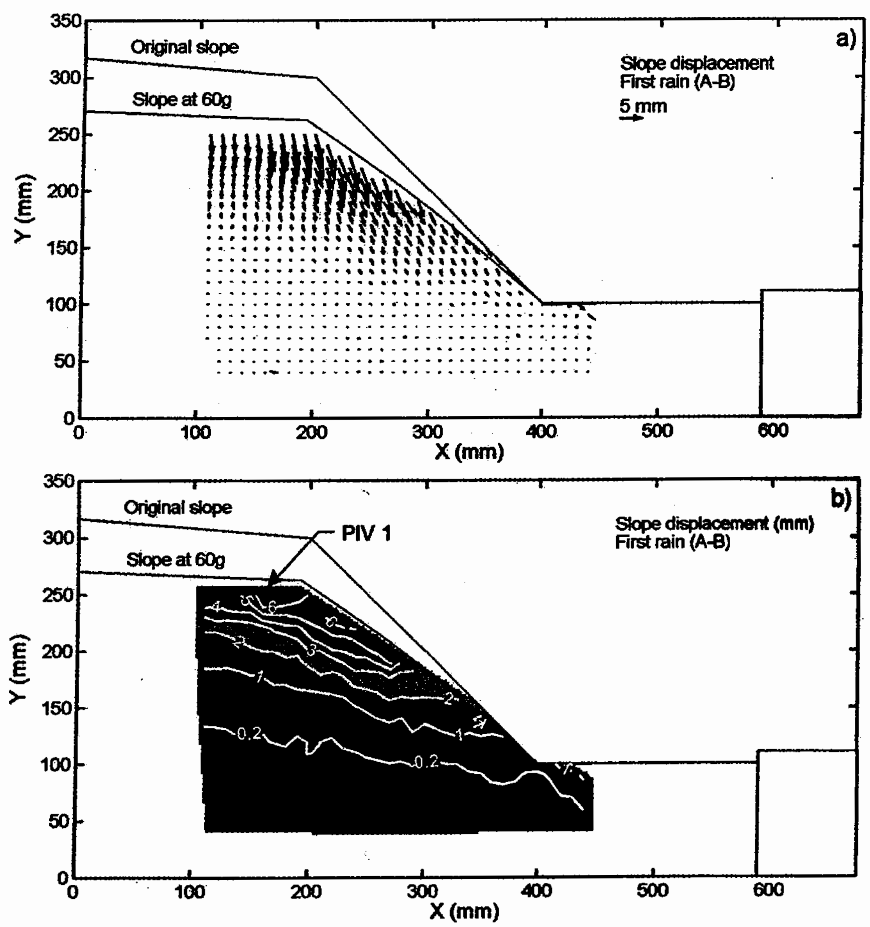
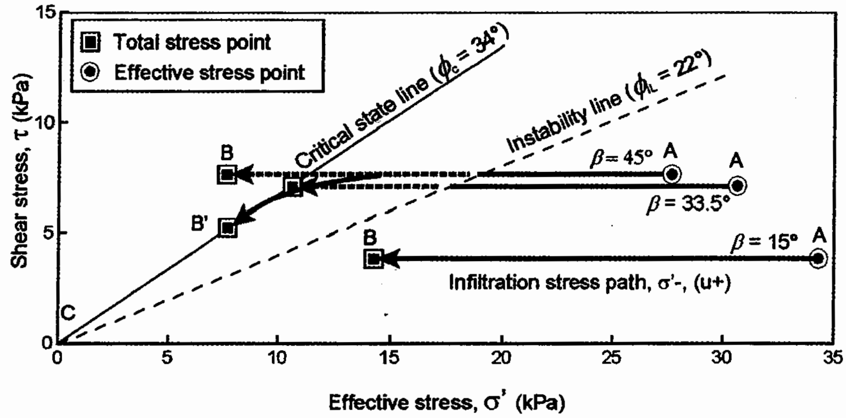


Fig. 8 Observed wetting collapse of loose model fill slope

Fig. 9 Theoretical stress paths for infinite slopes of varying inclinations



It is perhaps useful to compare the behaviour observed in this model test with the stress paths which would exist if we were to subject an infinite slope of our fill material to the same boundary conditions. Since the stresses acting on such a failure surface are well defined, we can readily assess the stress paths which would be followed on wetting. The results of this analysis are presented in Fig. 9, for an arbitrary depth of 1 m (the actual depth not influencing the failure mode as we will assume pore water pressures will go to 0 kPa upon wetting). On this figure we have also drawn the critical state and instability lines for this material.

We will first consider the case of a 15 degree slope initially subject to an arbitrary suction. Since the pore water pressure will be negative in this case, the effective stress point (circle) will lie some distance to the right of the total stress point. Upon rainfall infiltration, the stress point tracks horizontally towards the total stress point as suction is lost. Assuming that the material is as equally free draining as was observed in the model test, the effective stress path will not travel past the total stress point into the region of instability. Such a slope will be stable.

The stress path for a 45 degree infinite slope is also plotted. This time, as rainfall infiltration decreases the matric suction in the soil, the stress path intersects the instability line. As the stress path moves through this zone of instability, the loose soil will try to collapse. If, as was the case of our model test, the contents of the void space are sufficiently compressible or mobile, a drained instability will result without a large pore pressure increase, and the stress path will continue to push towards the total stress point (B). As the slope is steeper than the critical state friction angle, the wetting stress path for an infinite slope must intersect the critical state line at some negative pore water pressure. Since the fill material is looser than critical, the critical state line will also be the failure line. Thus, if the hydrology dictates that the pore water pressure will rise to zero, the stress path must follow the critical state line to a point B', and by so doing, the shear resistance must fall. If this drop in shear stress cannot be accounted for by the instability settlement reducing the slope angle, a slip failure will result. Such a failure was reproduced in the centrifuge for a 70 degree slope (Yeung, 2002).

The infinite slope analysis present above predicts that a 45 degree slope (such as our model fill slope) is potentially in danger of failure. However, due to the high compressibility of the fill material, the model fill slope achieved an average angle of inclination of 33.5 degrees prior to rainfall infiltration. The infiltration stress path for an infinite slope of this slope angle is also presented in Fig. 9. Here, as was observed in the model fill experiment, the unsaturated infinite slope may experience instability but, if the voids can collapse, undrained failure will be averted and the stress path remains inside the failure line.

Evaluating the slide-to-flow mechanism

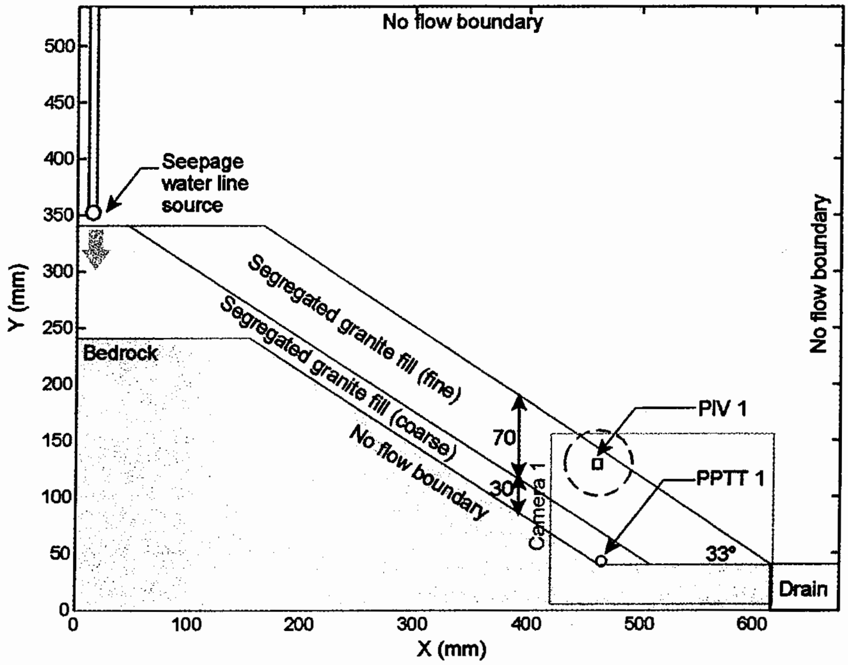
Model geometry

In addition to the triggering mechanism of static liquefaction, it is proposed that fast landslide events in Hong Kong fill slopes may also be triggered from initially slow-moving slips driven by localised transient pore water pressures. Two model tests have been performed to investigate this transformation in both loose and dense fill slopes. The geometry adopted for both fill slopes is presented in Fig. 10. The slope angle for the new model tests has been reduced to 33 degrees for the sake of similarity to the average inclination of the first model test after self-weight settlement. At a new test acceleration of thirty gravities, the model corresponds to a fill slope of 9 m in height, with a vertical depth of fill of 3 m.

The intended triggering mechanism combines the two closely-related pore pressure generation scenarios of flow constriction and heterogeneity thereby compounding the landslide risk. Firstly, elevated transient pore water pressures will be generated at the toe as the inclination change of the impermeable bedrock will constrict seepage flow. As described at the outset of the paper, water must be stored at this location, and will be forced to seep through the fill material at the toe of the slope. In this experiment, the impermeable bedrock layer is modelled by a solid wooden block, the top surface of which has been coated in varnished coarse decomposed granite to ensure a high interface friction angle.

The chosen soil profile for the model fill also represents an idealised case of layering in which the decomposed granite fill material has been sieved and separated into its coarse and fine fractions and placed one on top of the other to form a layered backfill. In this scenario, the layer ends blindly at the toe of the slope, which is also notably the location of the flow constriction. This ensures that the rate of arrival of seepage water at the toe greatly exceeds that of leakage, thereby ensuring a more rapid local transient build up of pore water pressures in this region than would have existed in the absence of layering. In other words, the presence of the layering in this model will reduce the intensity and duration of rainfall infiltration required for failure.

Fig. 10 Slide-to-flow landslide triggering mechanism model



As before, the behaviour of the model fill will be quantitatively observed in terms of pore water pressure and slope displacements. In the new model, the pore water pressure will be measured at the base of the bedrock. Since the model fill material is air dry at placement, a PPTT placed in the model would loose moisture until the negative pore water pressure within the device was sufficient to cause air-entry through the ceramic filter. To account for this behaviour, the miniature tensiometer was fitted with a sintered bronze filter element which has a large pore size and, therefore, a correspondingly negligible air entry value. Thus, although the sensor is placed completely dry, the arrival of seepage water can be accurately measured.

In addition to digital still cameras, the local behaviour of the toe of the slope was also measured at the higher frame rate of 15 frames per second using a modified web cam (Camera 1 in Fig. 10). As the grain size of the fine fraction is too small to be visible in the field of view of the camera, high contrast plastic inclusions were mixed with the fill to generate a soil texture which can be tracked by image analysis.

Loose Layered Fill Slope

The density of the fill material in the first layered slope model was intended to be representative of tipped fill and was placed with only a minimal compaction effort. The resulting layered slope is very loose, with an approximate relative compaction of 77%. As before, the model fill slope was installed on the centrifuge and slowly brought to the testing acceleration, this time 30 g. As expected, the loose model fill material experienced significant settlement but, because much of the model cross section has now been replaced by stiff bedrock, the magnitude of settlement is an order of magnitude smaller.

The arrival of the transient pore water at the toe of the embankment is shown in Fig. 11. Once the line source of seepage water was activated, the high transmissivity of the coarse layer quickly delivered water to the toe of the fill slope. In this figure we have plotted the observed output versus time after the first arrival of the seepage water. As intended, the rate of water transfer into the toe region has exceeded the seepage velocity through the model fill, causing a transient increase in pore water pressure at the toe. The local pore water pressure was observed to increase at a nearly constant rate reaching a maximum value of 16 kPa at point B in Fig. 11a. Since the slope material is dry, the current position of the wetting front can be visually observed. As shown in Fig. 12b, the wetting front has progressed well into the fill material by this point, and at time B (frame b in Fig. 12) it is nearing the slope surface. As this seepage front has been progressing towards the toe, the slope has been slowly creeping (Fig. 11b).

After time B, the slope mass is observed to accelerate (points B-C on Fig. 11b). Digital images at these two instants in time are presented as Fig. 12b, and c, respectively. Image analysis performed on this image pair indicates that at this time, the onset of more rapid failure, the toe is accelerating horizontally with an average velocity over this time period of approximately 6 mm/s (Fig. 13). The observed displacement field over this time interval indicates that the surface of the model fill is moving down-slope at a slower velocity. The subsequent behaviour of the model fill slope is laid out pictorially in the remainder of Fig. 12. As the toe continues to accelerate horizontally, the surface of the model fill accelerates towards the toe (Fig. 12 d, e), with the velocity increasing to such a point that it exceeds the shutter speed of the camera (Fig. 12 f, g). When the fill material has finally come to rest, it has formed a low-angle run-out.

Dense layered fill slope

Unlike the static liquefaction mechanism, the slide-to-flow triggering mechanism does not require the fill material to be saturated, or potentially even very loose. The experiment was therefore repeated with a fill compacted to 95% maximum Proctor density whilst keeping all other factors constant.

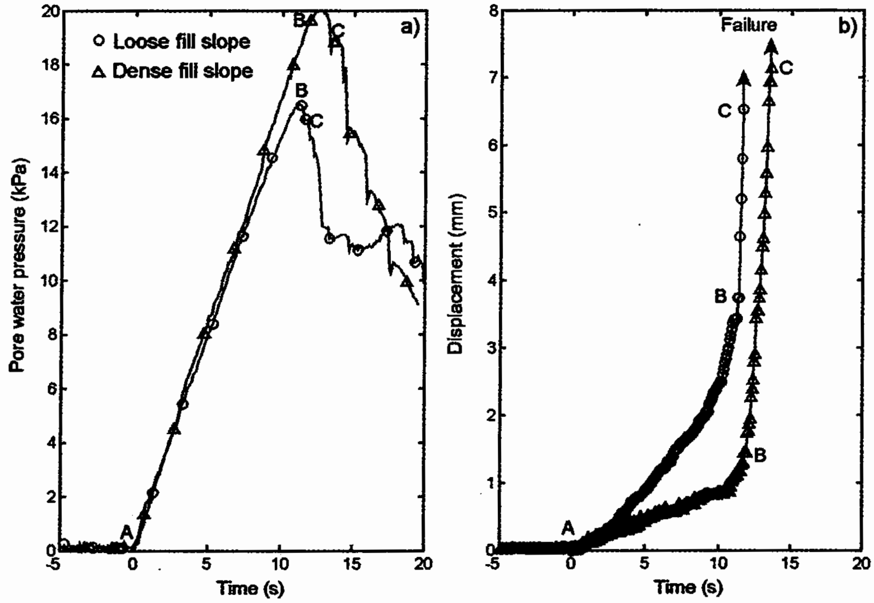


Fig. 11 Observed behaviour of slide-to-flow models

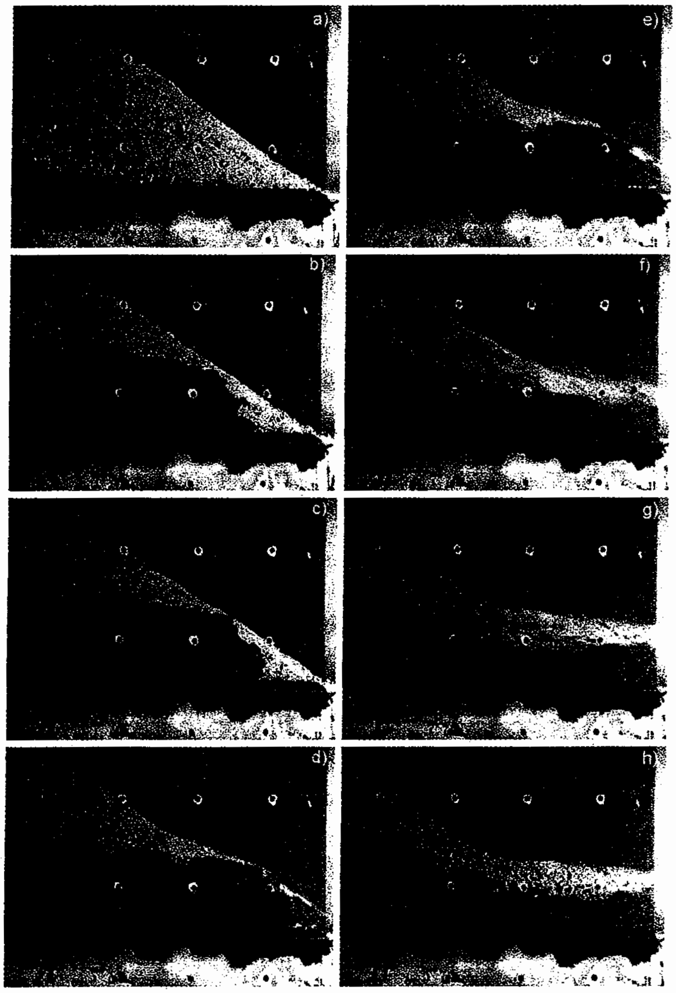


Fig. 12 Transition from slide to flow in loose fill model

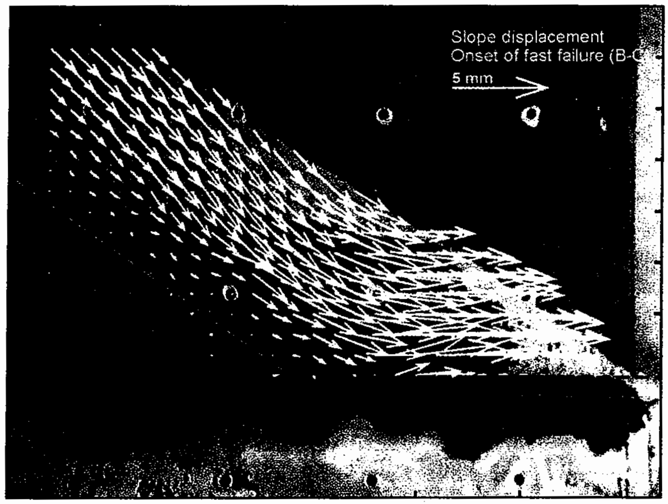


Fig. 13 Displacement field prior to final acceleration of loose fill model

As before, seepage water was introduced to the crest of the model fill slope where it was quickly transmitted to the toe of the slope, building up localised transient pore water pressures at an identical rate to the loose fill model (Fig. 11a). The dense slope exhibits a much stiffer response to the build up of pore water pressures, with less than one half of the pre-failure displacements warning of the onset of failure. Just before reaching the failure pore water pressure, the brittle fill material cracked and water rapidly entered the fill. As high pressure water entered this crack, the acceleration of the slide increased. The extent to which this crack has injected water into the fill material at time B is shown in Fig. 14b. After time B, the slope mass is observed to accelerate, albeit at a slower slide velocity than observed in the loose fill slope (points B-C on Fig. 11b). Digital images at these two instants in time are presented as Fig. 14b, and c, respectively. The subsequent behaviour of the model fill slope is laid out pictorially in the remainder of Fig. 14. As the toe continues to accelerate hor-

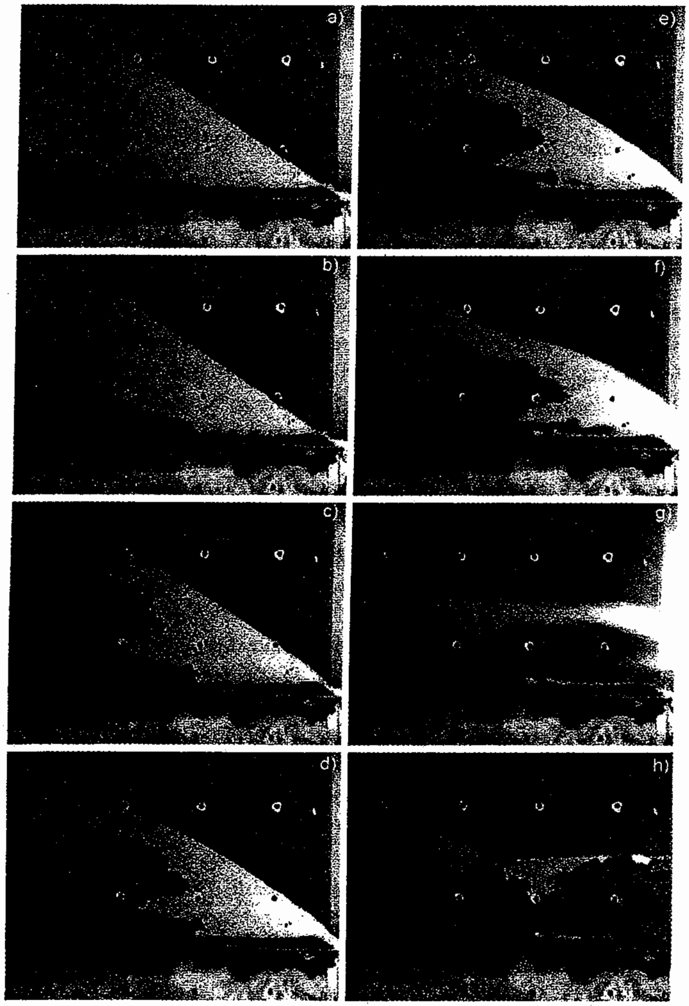


Fig. 14 Transition from slide to flow in dense fill model

izontally, the surface of the model fill accelerates towards the toe (Fig. 14 d, e), with the velocity increasing to such a point that it exceeds the shutter speed of the camera (Fig. 14 f, g). Similarly to the loose model fill, the landslide event triggered from localised transient pore water pressures has formed a low-angle run-out. The densification of the fill slope has slightly increased the pore water pressure required to initiate failure, but has made the failure more brittle.

Discussion and conclusions

A series of physical model tests have been performed to evaluate two candidate triggering mechanisms of fast landslides in decomposed granite fill slopes against observations of slope behaviour in centrifuge model tests. It has been demonstrated that the mechanism of static liquefaction, although easy to reproduce in saturated triaxial specimens, is much more elusive in physical models. Particularly, the unsaturated nature of the fill material, and the lack of groundwater ponding, has eliminated the possibility of creating static liquefaction in this model.

On the other hand, it has been easy to create the slide-to-flow mechanism of landslide triggering by creating local groundwater mounds where seepage was restricted. The initial slide has been observed to create rapid flow events with low run-out angles apparently very similar to those occasionally observed in Hong Kong fill slopes. These model flowslides were just as striking in dry, dense fill as in wet, loose fill. If these model events are realistic, the priority in hazard reduction should be in preventing triggering of a slip. Interception of groundwater percolation would be more useful than densification as a remedial measure, although the removal, mixing, and compaction of loose fill would have the coincidental benefit of eliminating permeable layers. Attention should particularly be focussed on regions of slopes where springs of seepage are observed after rainstorms. Shallow horizontal drains should be particularly effective in suppressing slip triggering in such locations.

The non-appearance of an event in certain circumstances can not be taken to mean that the event will never occur under any circumstance. Therefore, the conventional understanding of static liquefaction in loose CDG fill slopes can not be dismissed on the evidence of one centrifuge test. Furthermore, there is a factor arising out of centrifuge testing that may indeed have militated against liquefaction. Inertial events such as the duration of a pressure pulse due to soil collapse will take the form $t_{pulse} \propto \sqrt{(h/g)}$. In centrifuge tests, fall distances h are reduced by factor N whereas accelerations are increased by factor N. Accordingly, $t_{pulse,\ model} = t_{pulse,\ field}/N$. Diffusion processes, such as the release of air or water from a collapsing void, have durations of the form $t_{dissipation} \propto h/v$ where the flowrate v may be deduced from Darcy's Law. As explained before, v is increased by factor N and h reduced by factor N in a centrifuge test, so $t_{dissipation,\ model} = t_{dissipation,\ field}/N^2$. This means that the release of trapped pressure in a centrifuge model, during a collapse event, will be greater in proportion than that found at full scale. This problem is well-known in centrifuge earthquake liquefaction studies, Taylor (1995). Two corrective strategies to slow down rate of dissipation by factor N are: to increase fluid viscosities by factor N, or to reduce pore sizes (i.e. grain sizes) by factor $\sqrt{N}$. These further studies will be valuable in the clarification and classification of fast landslides. High-speed video coupled with PIV will be an essential tool.

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