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CONTROLLED WETTING TEST OF A SOIL NAILED LOOSE FILL SLOPE: CASE STUDY

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ABSTRACT

For an in-depth understanding of the failure mechanism of loose fill slopes and the strengthening effect by soil nails, a comprehensive field study program has been carried out at a specially built slope. The test slope was constructed by end-tipping of completely decomposed granitic soils almost without any compaction. The controlled failure test program was mainly composed of surcharge at the crest, wetting with surcharge, and wetting without surcharge. This paper is aimed at a case study of the coupled hydro-mechanical response of the test slope during the recharge process with surcharge. Through a comprehensive instrumentation system, abundant monitoring data with regards to the complex performance of the loose fill and soil nails were collected. It was found that redistribution of water content within the loose fill was well consistent with the recharge program, and nonlinear deformation developed significantly in the loose fill by the reduction of shear strength due to increased pore pressure and diminished suction when subjected to large wetting loads. Also a two-dimensional simplified finite element model is established and some preliminary simulation results are presented.

INTRODUCTION

Failures of loose fill slopes have caused severe losses of lives and damage in many countries and regions, which may be attributed to a number of factors, such as geological features, topography, rain-infiltration, or combinations of these factors. Typically in Hong Kong, it was reported that there are about 6000 old fill slopes built before the establishment of Geotechnical Control Office (GEO) in 1977 (Malone and Pun, 1997), which were basically formed by end-tipping method almost without any compaction. Many of these pre-1977 fill slopes are currently considered to be substandard and upgrading is urgently required to improve the safety.

According to current standards, a slope that comprises fills having relative compaction less than 95% can be considered to be loose. Loose fill slopes are particularly prone to landslides during heavy rainfall. Failure mechanism observed in loose fill slopes can be classified into three groups namely washout or surface erosion, shear failure and static liquefaction (Wong et al., 1997), and the third failure mode i.e. static liquefaction is the most vulnerable type, which occurs due to reduced matrix suction and pore pressure increase in the loose soils.

Soil nailing is frequently described as a method of stabilizing slopes (Abramson et al. 1996), which is an efficient and economical in-situ reinforcement technique, and has been applied successfully in stabilizing many marginally stable natural and cut slopes in the last three decades. However, only a few cases of soil nailing in loose fills have been reported for temporary or permanent works. Concerns that soil nails may not work in loose fill mainly includes that the soil may develop high pore pressure and flow around the soil nails, and the soil on either side of the failure zone may be too weak to provide a secure anchorage for the soil nails (HKIE, 1998). Therefore, its applicability in loosely compacted fill still needs further verifications and the field and large-scale laboratory test are recommended.

Although several researches on the soil nailing technique by field tests have been carried out, such as the Bodenvernagelung project undertaken in Germany during 1975-1981, the field studies on the lateral earth support system for deep excavation (Shen et al., 1981), and the Clouterre project on three fully instrumented experimental soil nailed walls (Schlosser et al., 1991; Byrne et al., 1993), there appeared to be no published cases in which field test of soil nailed loose fill slope has been carried out. In this paper, for an in-depth investigation into the failure mechanism of soil nailed loose fill slopes and the feasibility of stabilization by soil nails, a full-scaled field study program has been completed by the authors. Also a simplified plane finite element model developed for simulation for coupled flow and deformation within soil slopes is applied for a numerical study of the controlled failure program. Some preliminary simulation results are shown and quite acceptable agreement can be obtained in comparison with the field monitoring data.

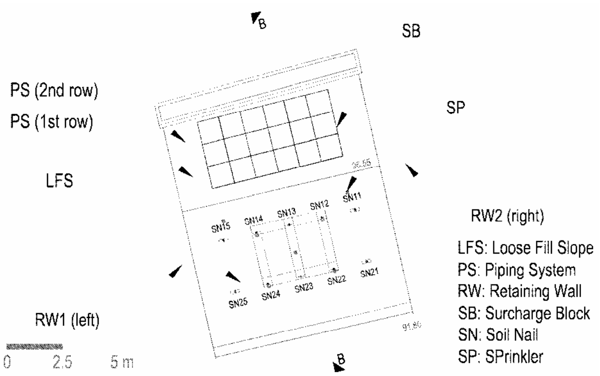
DESCRIPTION OF THE FIELD SLOPE TEST

Design and Construction of the Test Slope

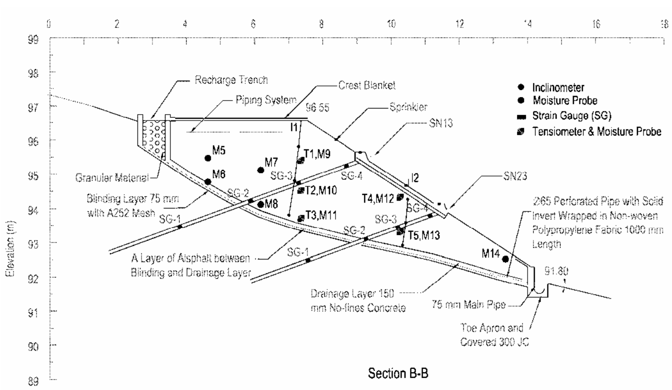
Slope Design and Site Treatment. As it is impossible to carry out the study on existing slopes, a loose fill slope was designed to be constructed by end-tipping method. Similar construction process was adopted as those of existing slopes so that a benchmark analysis can be taken. The built slope was designed to possess a sufficiently low safety such that failure can be induced in a controlled manner by gradually saturating the soil, together with surcharge on the crest.

The selected site was located at the Kadoorie Agricultural Research Centre (KARC) of the University of Hong Kong. The terrain was a moderately gentle sloping ground with an average angle of no more than 20°. The site was mainly composed of a surface layer of bouldery colluvium underlain by completely decomposed granodiorite. No ground water was encountered within the depth of interest.

The arrangement of the test slope is given in Fig. 1. Two robust gravity retaining walls, made up of 1m×1m×0.5m pre-cast concrete blocks, were designed in comply with GEO (1998) at both sides to confine the loose fill slope, which was also presumed to alleviate as far as possible the influence of end effects. Furthermore, asphalt waterproof was applied on the inner side of the walls.



(a) Plan view



(b) Section view

Fig. 1. General arrangement of the field test

As presented in Fig. 1(b), a 75mm blinding layer with A252 steel mesh had been designed for the base to isolate the loose fill slope from the natural ground. Therefore, deep-seated failure beyond the fill was prevented by a durable foundation layer, which could also isolate water from entering into the natural ground. And then a layer of asphalt was applied above the surface of the blinding layer as a further watertight measure. By above treatments, the loose fill slope was designed with an impermeable boundary at both sides and bottom to secure the water only to saturate the fill. Moreover, a layer of 150mm no-fines concrete, a hard durable coarse free-draining material, was placed above the blinding layer forming a drainage layer in comply with GEO (1993). Permeable geotextile was then applied between the no-fines concrete and the soil dumped above to prevent the migration of fine-grained particles of the soil as water flowed across the drainage interface.

For monitoring and influence of the wetting process within the loose fill, the no-fines concrete stopped at 375mm from the slope toe and 65mm perforated PVC pipes of 1 m in length were placed at 500mm interval, which extended into the no-fines concrete and were wrapped with geotextile to prevent blockage of the pipes. All the pipes were connected to a 75mm main pipe through T-junctions to a catch pit, where a valve was prepared at the end of the main pipe for collection of outflow water. Further information on the test slope can be found in (Li, 2003).

Construction of the Test Slope. The test slope was designed at an inclination angle of 33°, the same average slope angle of 1 on 1.5 as most existing loose fill slopes, a height of 4.75m, a width of 9m and a crest of 4m. The slope was constructed by end-tipping loose fill onto the received base up to 3 m thick maximum with hardly any compaction. The soil was obtained from a construction site and was identified to be completely decomposed granite (CDG). The completed field test slope is illustrated in Fig. 2.

A toe apron up to 800 mm high was constructed at the toe of the loose fill slope, which was designed in order that the water table could be raised to a certain level under the controlled wetting program. Since the toe of slope was a very sensitive area of a landslide (Skempton and La Rochelle, 1965; Garret and Wale, 1985), it had been expected that relatively deep-seated failure could be triggered as the fill was saturated in this manner, rather than a shallow failure.

Soil Nailing in the Test Slope. Two rows of 10 grouted nails were designed at a grid of 1.5m×1.5 m at an inclination of 20° horizontally (Fig. 1). The installation procedures and the type of nails were similar to those commonly used in Hong Kong. First, a hole of 100 mm diameter was drilled; secondly, a 25mm diameter steel ribbed bar with appropriately placed centralizers was inserted into each hole; finally, the hole was filled with grout from the bottom up using a plastic hose.

Two types of nail heads were adopted, namely independent head and grillage beams, as shown in Fig. 2. Basically, the independent head was simply a steel plate and nut assembly with a concrete pad, while the embedded grillage beams were a modified version of GEO standard nail head. The beams spanned between the nails and had sufficient structural strength to ensure the grillage behaved in a monolithic manner. It had been considered that the grillage beams seated within and keyed into the upper layers of the slope could produce additional resistance to the soil and hence prevent the occurrence of liquefaction failure (HKIE, 1998).

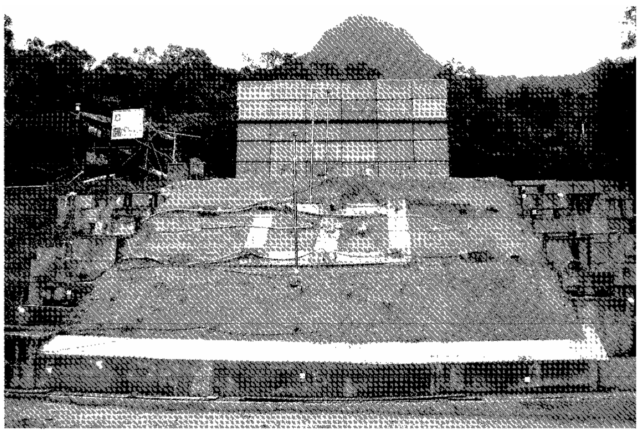


Fig. 2. View of finished loose fill slope (with surcharge)

Four short nails were also designed for pullout tests. They were entirely within the loose fill having length of either 2 m or 3 m. Therefore, the pullout capacity of the nails within the loose fill could be obtained.

Instrumentation of the Test Slope

For investigation into the processes and mechanisms operating as the nailed loose fill slope progressed from a stable to an unstable state and eventually to collapse, a comprehensive instrumentation system was established to measure all the conditions of the soil nailed fill slope both during and after construction and up to the point of failure. Generally, the instruments consisted of 2 in-place inclinometer strings (a total of 6 measuring elements), 7 vibrating wire piezometers, 24 soil moisture probes and 6 tensiometers, 6 vibrating wire earth pressure cells, 2 load cells, and 46 vibrating wire strain gauges, 12 flowmeter/recharge indicators and 2 data loggers. As a whole, the instrumentation system in this study was designed broadly into three categories: pore water pressure/ moisture content, strain/deformation and stress/load related measurement, with layouts close to the central section presented in Fig. 1(b).

Failure Test Program

The failure tests were scheduled in three stages from November 2002 to January 2003, i.e., the dry season in Hong Kong, namely the surcharge process, wetting program with surcharge, and wetting without surcharge. This paper focused on the second stage of failure tests, i.e., wetting program with surcharge, aiming at a deeper investigation into the performance of internal deformation and water content redistribution, as well as the failure mechanism of the test slope.

During the surcharge program, first, a layer of 75mm gravel was placed on the crest of the fill slope to form a blanket to facilitate the surcharge process, and then five layers of concrete blocks of 1m×1m×0.6m were stacked up to 3m, stage by stage on the central area of the crest (Figs. 1 and 2), representing a final surcharge of 70.5KPa.

As shown in Fig. 1, recharge system in the field test comprised of crest recharge trench, buried piping system and surface sprinkling system, which could be operated independently for different watering schemes. The recharge trench directly connected with the drainage layer, through which water could be supplied to the loose fill from bottom. Piping system consisted of 10 sets of perforated bronze pipes laid 300 mm beneath the crest, and the flow rate of each set of pipe could be adjusted separately through a valve. Sprinklers were also installed on the slope surface to simulate artificially rainfall events on the slope. Therefore, in combination with the drainage layer, it had been expected to sufficiently large portion of the fill would be saturated in a controlled manner.

The recharge test program started on November 20, about one week later after the surcharge process. Figure 3 presents the wetting flux as a function of time by different recharge components. It can be seen that during the first two days, only the surface sprinklers were activated discontinuously, and two wetting-drying cycles were implemented; both the sprinklers and buried pipes were turned on during the latter two days, with larger intensity of wetting loads applied particularly during the final day in order that failure of surcharge blocks or flow sliding can be mobilized. In Fig. 3, characteristic transition moments during the recharge process are also marked by A, B, C…, for indication of activation or suspension of the wetting system.

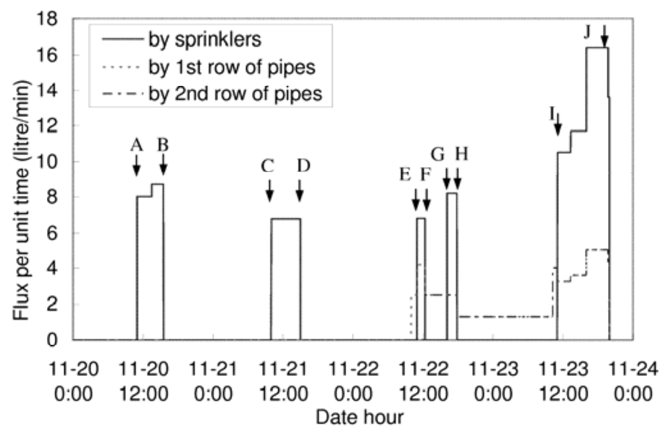


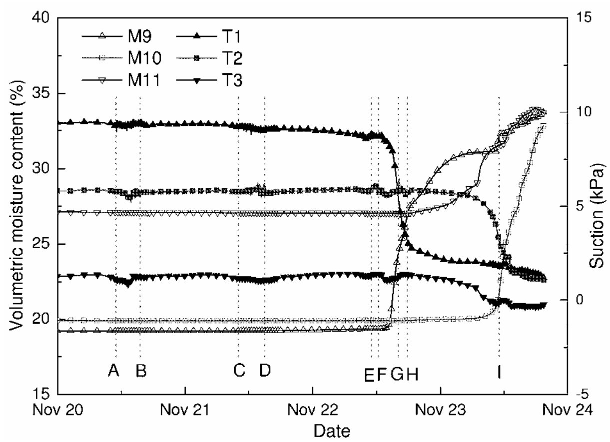
Fig. 3. Time history of infiltration flux per minute

TEST RESULTS AND DISCUSSIONS

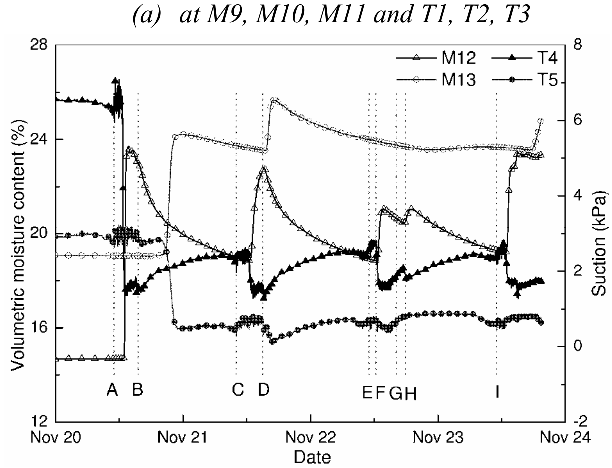
The instrumental records demonstrated that the overburden pressure and deformation within the loose fill, as well as axial forces in soil nails were significantly affected by the former surcharge program, also it was found that pore water contents within the test slope were gradually redistributed to a balanced state. Subsequently, the hydraulic and mechanical responses within the test slope during the following recharge program were also monitored on a real time basis. Some representative test results and discussions are provided below.

Characteristics of Moisture Content Redistribution

Variation of Moisture Content at Typical Points. Through the records of moisture probes, the variations of volumetric moisture content at differently instrument locations within the field slope can be monitored. Typically, as presented in Fig. 1(b), pairs of moisture probes and tensiometers were installed in the middle part of the test slope, such that matrix suction at the same location can be monitored during the recharge program. Time variations of records at these positions are shown in Fig. 4. Correspondingly a summary of the relationship between the volumetric moisture content and matrix suction, i.e., soil-water characteristic curve (SWCC), is given in Fig. 5.



(a) at M9, M10, M11 and T1, T2, T3



(b) at M12, M13 and T4, T5

Fig. 4. The response of moisture probes and corresponding tensiometers during the recharge process

It can be observed that during the first two days of wetting by surface sprinklers, only the moisture probes M12 and M13 beneath the slope surface showed a direct response, which was accompanied by corresponding adjustment of the suction values by tensiometers T4 and T5; whilst M9, M10 and M11 were hardly affected for their departure from the effective range of surface sprinklers. Also it can be seen that some noise-like adjustments of the suction response were monitored by the tensiometers T1, T2 and T3 during the two wetting sessions.

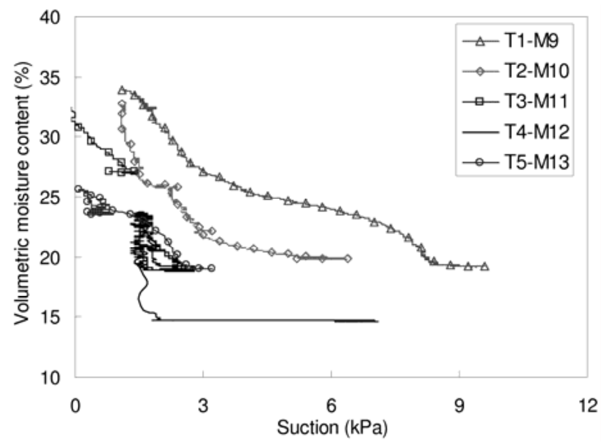


Fig. 5. Monitored soil water characteristic curves

On November 20, when the wetting front came up after nearly two hours of wetting, the volumetric moisture content at M12 increased abruptly up to nearly 24% within 50 minutes. Correspondingly, the matrix suction from T4 dropped synchronously from 6.5kPa to 1.5kPa. However, it can also be seen that the moisture content from M12 began to decrease within the wetting session after it reached the peak at 13:40, accompanied by some "noise" response from T4, which presented a general upward trend. The observation can be mainly attributed to the fact that the coefficient of permeability of the loose fill would increase abruptly when the saturation degree is raised to a certain extent. During the drying period (from B to C), as the pore water flowed from the upper region to lower portion, the moisture content from M12 dropped gradually and the suction was raised. It can be also found that after nearly 19 hours of drying, the water content within the loose fill were still not stabilized and it can not be judged that whether it would return back to the initial state prior to the recharge test.

For M13 and T5 buried at a deeper depth beneath the slope surface, with a larger initial value of moisture content and a smaller suction, similar response was mobilized nearly 8 hours later that at M12 and T4. It can be seen that T5 responded by a noise-like variation within the wetting session, significantly earlier than the prompt of M13 at the same location. Also it was found that the value from M13 increased from 19.1% to a maximum value of 24.2% in about two hours, not so steep at that from M12.

When the surface sprinklers were reactivated again at 10:00 on November 21 and continued for 5 hours, the moisture content from M12 increased again and reached the peak value of 22.8% at 15:15, accompanied by a drop-back of matrix suction at T4. Differently for M13, as influenced by extra wetting loads during the second day while pore water from first day's recharge had not been sufficiently redistributed, its water content increased to a larger magnitude of 25.7% at 17:00, accompanied by a small adjustment of suction from T5. It can also be found that both the records of moisture content and suction basically returned back as those prior to the second wetting period after a sufficiently long drying process.

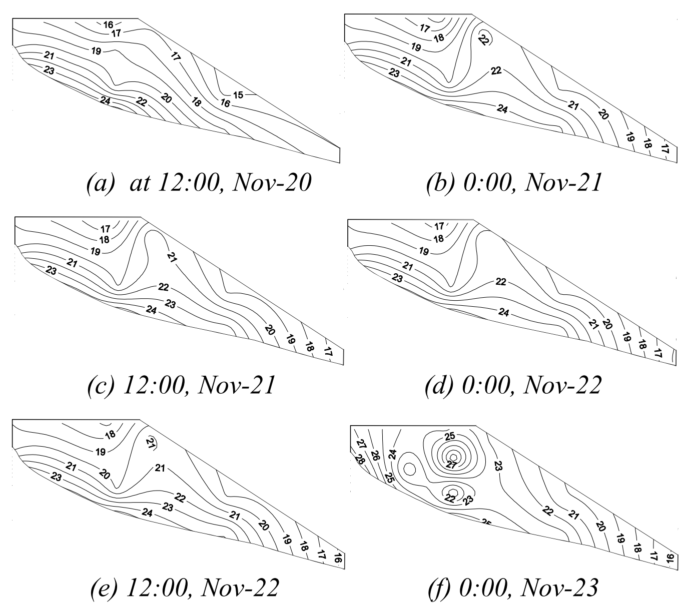
During the latter two days of wetting program, with the introduction of water from the first row of piping system at 10:00 on November 22, it can be seen that the volumetric moisture content in M9 nearby increased continuously from 14:00 until the end of recharge program, and the variation was in accordance with the adjustment of wetting flux from the piping system. As shown in Fig. 1, M10 is 0.9m below M9 and shallower than M11. However, the record from M10 presented hardly any increase within the following 20 hours after the opening of piping system. Moreover, the response of M11 possessed an earlier increase of water content than that of M10. The above observation can be mainly attributed to the fact that M9, M10 and M11 were not directly buried beneath the buried pipes, and water from the second row of buried pipes would prefer seepage towards the adjacent no-fines concrete layer, which resulted in a local zone of larger saturation degree between pipes and the no-fines concrete below. Therefore, a quicker seepage path would be formed as the permeability coefficient would be enlarged with increased saturation degree, and little water from buried pipes would flow through the central zone beneath the crest corner, where M10 was installed. It is also noted that the volumetric moisture contents more or less reached about 33% prior to the collapse of the surcharging blocks at 19:29 in the final day, and the soils surrounding M11 were fully saturated as positive pore pressure were recorded by T4.

For M12 and M13, the reactivation of the surface sprinklers during the sessions of E-F and G-H had different influences. The water content from M12 prompted with prominent increase and decrease due to the direct infiltration effect from slope surface, accompanied by corresponding variation of suction from T4; while only slight adjustment of moisture content and suction can be observed from M13 and T5 by these two wetting-drying cycles. It can be associated to the instrument locations of M13 and T5, which were deeper and closer to the no-fines concrete layer, and would be prone to be affected by the seepage water from upper portion of the fill slope. Also it can be found that the final moisture contents from M12 and M13 were smaller than those of above moisture probes.

From the SWCCs of each instrument pair during the wetting program given in Fig. 5, it can be observed that the SWCCs at different locations varied considerably, although similar in shape and slope, and during the wetting and drying cycles, the response at each point was basically positioned in one line, which indicates that the absorption and exsorption curves are basically coincident for the loose fill. The results also imply that soil suction may play an important role to slope stability at shallow depths, as it is quite susceptible to rainfall events.

Spatial Redistribution of Moisture Content within the Test Slope. Through the discrete instruments installed within the test slope, particularly for the central section with a high degree of instrumentation, a characteristic view of the pore water redistribution along the central section is shown in Fig. 6. Some typical moments during the recharge process are chosen and the evolution of water content redistribution could be basically identified from the results.

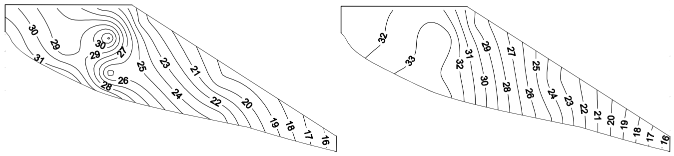
During the first two days, Fig. 6(a)-(d) show that only small portion of loose fill beneath the slope surface and lower parts were affected by sprinklers, and prominent increase of water content was monitored, whilst for the upper portion of the slope, only slight adjustments of redistribution were mobilized. During the latter two days, it can be seen from Fig. 6(e) that shortly after the activation of piping system and surface sprinklers, infiltrated water was limited and the distribution of moisture content in most potions of the slope remained basically the same as in Fig. 6(c). After 12 hours of wetting by the piping system, a local region beneath the two rows of pipes were observed to be more saturated than other parts of the slope (Fig. 6(f)), with maximum magnitude of volumetric moisture content of about 30%. Also noticeable increase could be seen for the middle and lower portions of the test slope. With larger wetting intensity during the final day, as shown in Fig. 6(g) and (h), it can be observed that water content within the whole test slope presented remarkable increase. In particular, from Fig. 6(h), main parts beneath the buried pipes were nearly fully saturated, and most contour lines in the whole slope were basically inclined to be parallel to the vertical direction, which can be associated to fact that the permeability ability of loose fill would be improved observably when getting more saturated.



(a) at 12:00, Nov-20 *(b) 0:00, Nov-21*

(c) 12:00, Nov-21 *(d) 0:00, Nov-22*

(e) 12:00, Nov-22 *(f) 0:00, Nov-23*



(g) 12:00, Nov-23 *(h) 19:00, Nov-23*

Fig. 6. Monitored distribution of volumetric moisture content within the loose fill

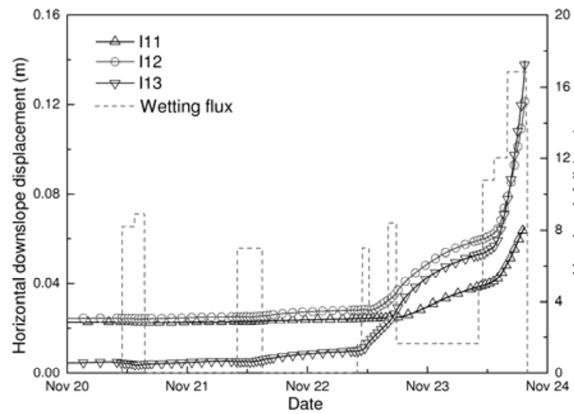
<u>Characteristics of Mechanical Response</u>

<u>Horizontal displacements by inclinometers</u>. Time histories of horizontal downslope displacement for each sensor module of the two inclinometers during the wetting program are shown in Fig. 7, and the variation of hourly "rainfall" intensity is also presented. It should be noted that the readings recorded on November 1, 2002, i.e., prior to the surcharge test, were taken as baseline references.

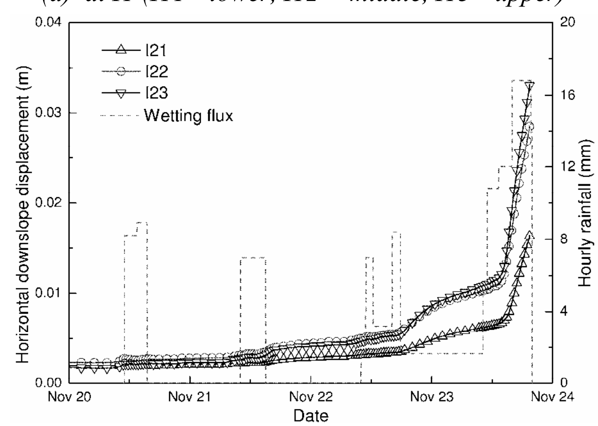
It can be found that horizontal downslope displacements developed substantially in response to the wetting program. During the first day, only slight adjustment of horizontal displacement was mobilized at I1 and I2 by the surface wetting. After the wetting in the second day, both inclinometers, particularly for the upper measuring points, presented gradual increase of horizontal displacement after a sufficiently long water diffusion. During the latter two days, more remarkable displacements were mobilized at the two inclinometers by wetting loads of larger intensity, particularly abrupt increase during the final day prior to the collapse of the first row of blocks. The maximum horizontal displacement from I1 immediately before failure of surcharge blocks was amounted to 139 mm, whilst much smaller displacements were recorded from I2. Therefore it can be concluded that excess displacements were concentrated in the immediate vicinity of the surcharge.

<u>Axial force in soil nails</u>. For investigation into the effect of water infiltration on the strengthening performance of soil nails within the loose fill, variation of axial forces monitored through strain gauges (Fig. 1) along the nailing direction at SN13 and SN23 are presented in Fig. 8. The nail loads were computed with respect to baseline readings on November 1, 2002. Generally, it can be seen that maximum loads in both soil nails were triggered at the strain gauge No. 2 (SG-2), which was located near the fill bottom. Moreover, it can be observed that the first two days wetting only had marginal effects on the nail loads, particularly for SN13, which can be reasonably attributed to the fact that no significant development of horizontal displacement were mobilized during this period, as indicated by the records of inclinometers. However, small enlargements of nail loads were observed from SG-3 in SN23, of which the loading condition was change from tension to compression. Also it can be found that during the wetting and drying sessions, both soil nails would perform some slight adjustments in response to balancing the change of external loads.

During the latter two days, it can be observed that axial force in both soil nails presented upward trends as a whole, which was reasonably consistent with the wetting loads of larger intensity. In particular, from the strain gauge near the head of SN23, abrupt increase of nail loads were mobilized near the grillage beams, which can be related to bending/shearing effect at heads, while records from other strain gauges were hardly affected. Therefore, the observation provided some informative results that the four days of wetting would induce some coupled hydro-mechanical behaviour within the loose fill and soil nails, besides the contribution to the external loads by wetting flux.



(a) at I1 (I11—lower, I12— middle, I13—upper)



(b) at I2 (I21—lower, I22— middle, I23—upper)

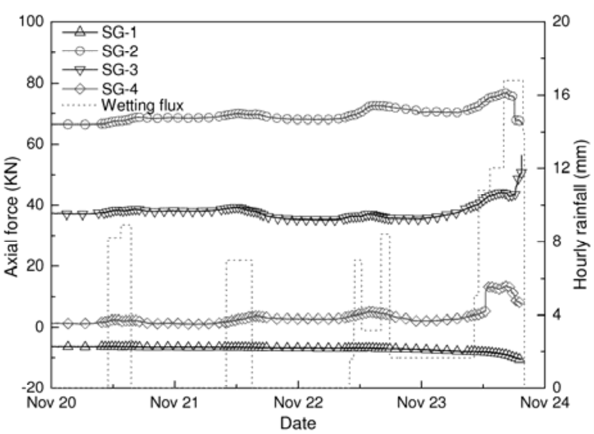
Fig. 7. Development of horizontal downslope displacements by inclinometers during the recharge process

NUMERICAL MODELLING OF THE WETTING PROCESS

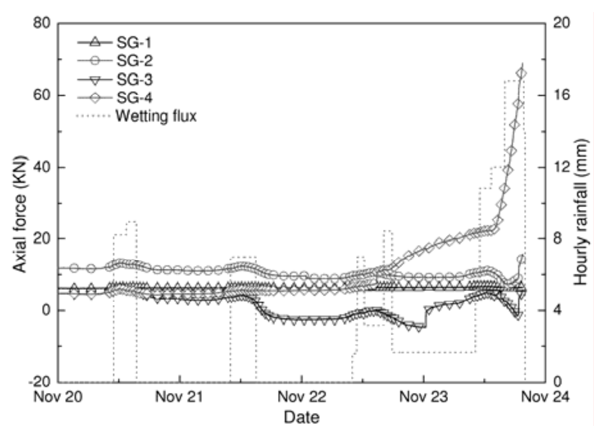
A simplified two-dimensional (2-D) numerical approach has been developed by the authors (Zhou et al., 2007) for simulating the coupled hydro-mechanical response within the field slope. It has been demonstrated that the 2-D simplified model is quite straightforward and effective through comparison of the simulation results and monitored records of the test slope during the surcharge process. In this paper, the same numerical approach is applied and the coupled flow and deformation performance within the slope during the recharge process is studied. Some preliminary results are presented below and corresponding field measurements are also shown for comparison.

Simplified Plane Numerical Approach

The loose fill is taken as a porous medium and was modelled by the conventional approach that considers soils as multiphase materials, and the effective stress principle was adopted to describe its behaviour. As for saturated soils, water flow through unsaturated soils is also governed by Darcy's law (Fredlund and Rahardjo, 1993). The finite element packages ABAQUS (ABAQUS inc., 2006) are taken as a platform for the simulation of the recharge process.



(a) at SN13



(b) at SN23

Fig. 8. Variation of axial forces at different measure points in soil nails during the recharge process (SG—strain gauge))

2-D Finite Element Modelling. The typical central section of the test slope is chosen for a plane simplified simulation. As shown in Fig. 9, the section is modelled by plane strain 4-node bilinear elements, of which coupled degrees of freedom including pore pressure and displacements are incorporated. Taking into account that a layer of asphalt was applied above the natural ground surface as a watertight measure, only deformation is taken as basic variables for the finite elements simulating the ground soils. The no-fines concrete layer, as a hard durable coarse free-draining material, is also modelled as a deforming porous medium by finite elements with coupled nodal variables.

For simplicity, two rows of ten grouted soil nails are transformed into two equivalent strengthening walls extending across the whole slope width, while the nailing length and the total section area are kept the same. The soil nails are then modelled by 2-node truss elements in the plane approach, of which the section area can be determined as that of the equivalent wall in a unit thickness. The embedded element technique is applied for the definition of soil nails reinforcement.

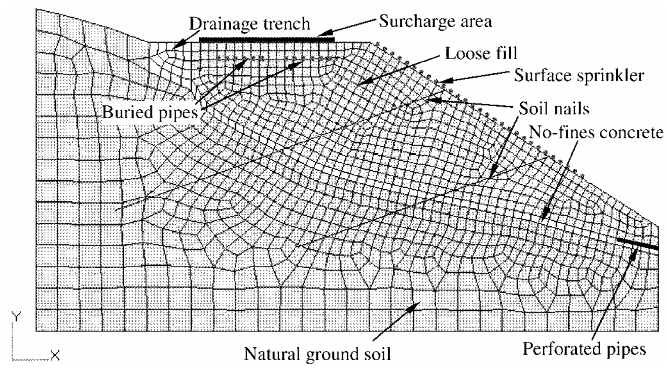


Fig. 9. 2-D finite element modelling

The displacement boundary conditions are modelled as vertical rollers on the left cutting edge, with full fixity at the base and the constrained region enforced by the concrete apron near the slope toe. Moreover, as possible water leakage is allowed from the buried perforated pipes near the toe, drainage-only boundary condition are applied there to account for the effect of water outflow.

The stress distribution within the slope under self-gravity of the loose fill is chosen as the initial state for the simulation of the surcharge process, and the initial conditions of void ratio, saturation degree are respectively taken as corresponding average values of measurements at instrument positions prior to the failure tests. The final state from the surcharge simulation would be taken as the beginning conditions of the following recharge process.

Material models and parameters. Subsequent to the completed surcharge analysis (Zhou et al., 2007), the identical material models are applied in the following simulation. The loose fill is modelled by an elasto-plastic model with a Mohr-Coulomb (M-C) failure criterion and a non-associated flow rule. Furthermore, as in most critical state models, nonlinearity is incorporated into the elastic part of the stress-strain relationship, and the bulk modulus K is assumed to be a function of the effective mean stress p' according to

$$K = \frac{\partial p'}{\partial \varepsilon_{\mathrm{v}}^{\mathrm{e}}} = \frac{\nu}{\kappa} p' \tag{1}$$

where ε_v^e denotes the elastic volumetric strain, v is the specific volume, and κ is the slope of an unloading-reloading line in the $v - \ln p'$ space. The Poisson's ratio μ is assumed to be constant, and the shear modulus G can be written as

$$G = \frac{3(1-2\mu)}{2(1+\mu)} K \tag{2}$$

To incorporate the contribution of matric suction to shear strength, Fredlund et al. (1978) proposed a modified M-C failure criterion based on unsaturated soil mechanics, in which two friction angles, ϕ' and ϕ^b are used to quantify the increased shear strength associated with the net normal stress and the matric suction respectively. It was found that the value of ϕ^b is equal to or less than ϕ' (Gan and Fredlund, 1996), and an equality of these two angles is assumed in this paper such that the original M-C failure criterion follows:

$$\tau = c' + (\sigma - u_w)\tan\phi' \tag{3}$$

where τ is shear strength on the failure plane; c' is the intercept of the "extended" M-C failure envelope on the shear stress axis where the net normal stress and the matric suction at failure are zero; σ is the total normal stress on the failure plane; u_w is the pore water pressure on the failure plane and ϕ' is the angle of internal friction. When u_w is negative, its magnitude is equivalent to matric suction (u_a-u_w) since pore air pressure, u_a, is assumed to be zero.

Only linear elastic properties are defined for the soil nails, natural ground soils and the no-fines concrete layer, as the possibility of nonlinear deformation within these components is considered to be quite low. Table 1 summarizes the parameters adopted in the analysis, which are calibrated from field or laboratory test results.

Table 1 Summary of material parameters

properties	CDG loose fill	Soil nails	Ground soils	No-fines concrete
Basic	γ_d=1.41kg/m^3, κ=8.4E-3, e_0=0.86, M_{c0}=14.9%,			
Elastic	Function (Eqn. (1) and Eqn. (2)), μ=0.05	E=2.57×10^4 MPa, μ=0.2	E=35MPa, μ=0.2	E=1×10^4 MPa, μ=0.2
Shear strength	c'= 2kPa, ϕ' =32°			
Hydraulic	Function (Figs. 10 and 11)			k=1×10^{-4} m/s

Note: E, M_c, γ_d, e, k are Young's modulus, moisture content, dry density, void ratio, permeability coefficient respectively, and the subscript "0" denotes the initial value.

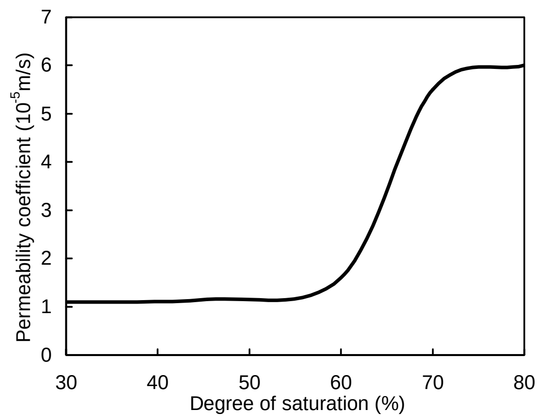


Fig. 10. Permeability coefficient vs. degree of saturation for the loose fill

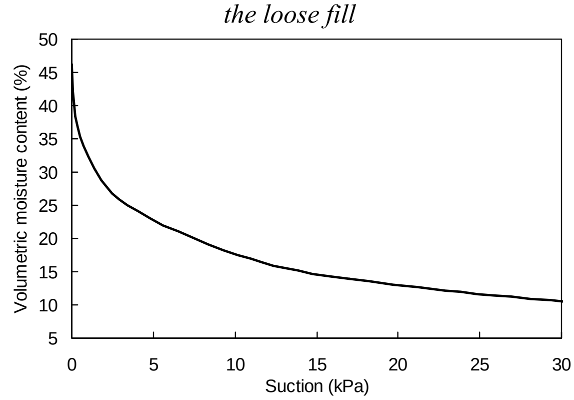


Fig. 11. Water retention curve for the loose fill

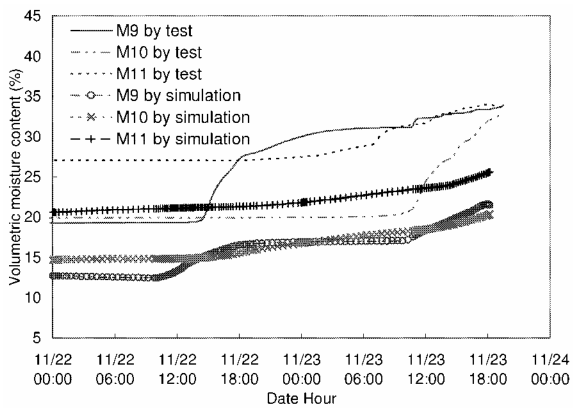
Simulation Results and Discussions

Variation of Moisture Content at Typical Points. Figure 12 presents the variation of volumetric moisture contents at some typical points, corresponding test records by moisture probes are also shown for comparison. It can be seen that the simulation results are basically in agreement with corresponding test records in variation shape and slope, and main differences are attributed to the start point inconsistency. The mismatch of start points is largely due to the uncertainty of initial conditions in the field site, such as the effects of evaporation and rainfalls before and during the slope failure tests, as well as the spatial non-uniformity of wetting loads by the recharge system. Generally, from the simulation results it can be observed that the moisture contents within the affected loose fill increase promptly when the wetting front reaches, and then decrease steadily at the suspension of recharge process. The simulated advancing rate of the water front is about 5×10^{-5}m/s, which is quite close to the test records.

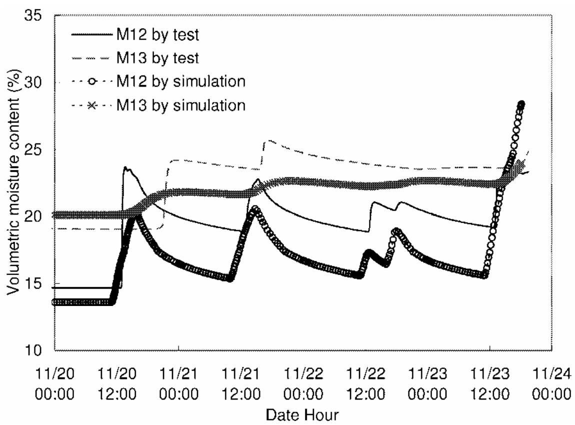
Under the effect of surface sprinkling during the first two days, soils surrounding M12 and M13 present a direct response of adjustment to the infiltration of water. Both simulation results and test records present a similar pattern characterized by quite abrupt increase and drop in accordance with the variation of "rainfall" intensity given in Fig. 3. Meanwhile, no apparent variation is found for M9, M10 and

M11, which are located beyond the effective area by sprinklers.

On the third day of wetting program, as the piping system beneath the crest was also turned on, it is shown that the responses at points, M9~M11, all present more or less adjustments. Particularly, for soils surrounding M9 and close to the buried pipes, the volumetric moisture contents change more promptly than those at M10 and M11, which are located at deeper positions. Similar observation can be also found from the results at M12 and M13.



(a) at M9, M10 and M11



(b) at M12 and M13

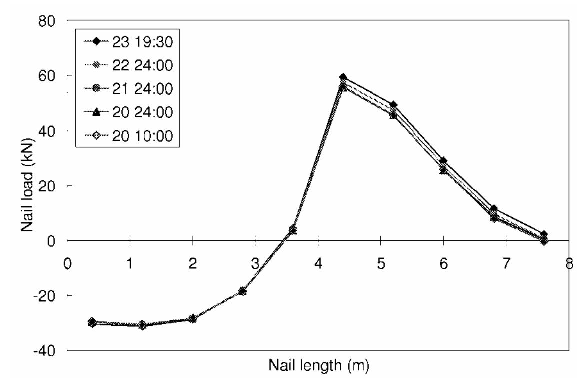
Fig. 12. Variation of volumetric moisture contents during the recharge process

During the final day, it can be observed that although the initial conditions of saturation degree within the fill could not be identified properly, eventually the predicted volumetric moisture contents at most measure points more or less reach about 30% prior to the end of simulation, approximately the same as test results prior to the collapse of surcharge blocks. This agreement also indicates that the hydraulic conductivity ability of the loose fill would increase significantly when saturated to a certain extent.

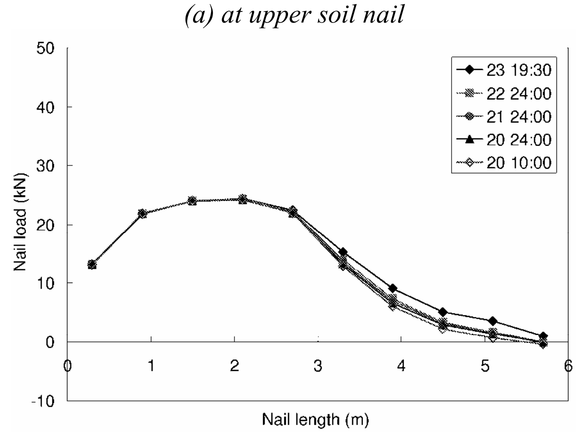
Axial Force in Soil Nails. Figure 13 presents the calculated development of nail loads during the recharge process. It can be observed that the first two days of wetting only have marginal effects on the nail loads, while the variations during the last two days are relatively more prominent, which can be reasonably attributed to the higher recharge intensity during this period. It can be also seen that compression forces are calculated for the lower portion of upper soil nail buried inside the foundation soils, whilst only tensile resistance is provided by the lower soil nail.

Comparison of simulation results and test records at the beginning and the end of the recharge process is also shown in Fig. 14. It can be seen that more apparent increase of axial forces is predicted in the upper portion of two soil nails, which compares favourably with test data and is reasonably consistent with relatively large deformation mobilized within the upper portion of loose fill under simultaneous loadings of surcharge and recharge. Furthermore, the calculated distribution pattern of axial force along the upper and lower soil nails remained basically unchanged, with peak magnitudes mobilized near the middle of the nails, which helps to clarify that the potential global sliding trajectory should be lying near the bottom surface of the slope fill, as approved by the simulation results of deformation field and the plasticity zone development.

As shown by the test results, there existed a sharp increase of axial force near the lower nail head during the last day of wetting, which is explained above that should be related to the local effect of the bending or shearing behaviour caused by the grillage beams connecting the nail heads. However, the grillage beams have not been taken into account within the 2-D numerical model and the observation can not be repeated. Further consideration and investigation into the effect of this frame-like strengthening structure are still needed.

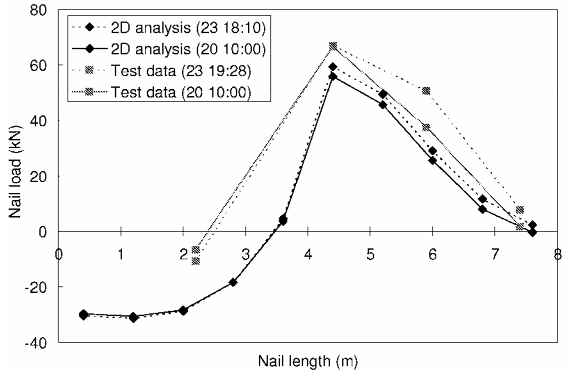


(a) at upper soil nail

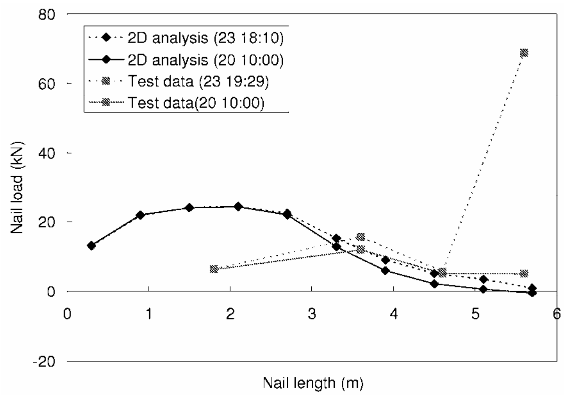


(b) at lower soil nail

Fig. 13. Simulated variation of axial nail loads during the wetting process



(a) at upper soil nail



(b) at lower soil nail

Fig. 14. Comparison of nail loads by simulation and test results

CONCLUSIONS AND REMARKS

Controlled failure tests of a field loose fill slope strengthened by soil nails have been carried out by the authors, and this conference paper is aimed at providing an in-depth investigation into the performance of test slope during the recharge program with surcharge applied on the crest. Representative test results, hydraulic and mechanical, at some typical points during the wetting process are presented and analyzed. Also a simplified numerical approach is applied for simulation of the wetting test.

It has been found that the spatio-temporal variations of the water content and suction from moisture probes and tensiometers are quite consistent with the arrangement and schedule of the wetting program. Wetting loads of large intensity would mobilize significant nonlinear deformation in the vicinity of the slope crest. Both simulation results and test records demonstrated that the soil nails significantly improve the overall stability of the loose fill slope. However, further investigation into the test results and a more comprehensive numerical model for the test slope are strongly demanded.

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