Performance of a large-scale slope model subjected to rising and lowering water levels

G.W. Jia, Tony L.T. Zhan, Y.M. Chen, D.G. Fredlund

A R T I C L E   I N F O

Article history:
Received 22 June 2008
Received in revised form 2 March 2009
Accepted 15 March 2009
Available online 25 March 2009

Keywords:
Large-scale model test
Water level changes
Sandy silt
Slope stability
Failure mode

A B S T R A C T

Many slope failures have been observed to occur during times of water level fluctuations. This paper presents a large-scale model test simulating the performance of a sandy silty soil slope subjected to water level rise and drawdown. The model slope is 15 m long, 6 m high and 5 m wide, and the sloping part has an inclination of 1:1 and a height of 4 m. A water level control system was developed to allow the simulation of the rise and sudden drawdown of the water level. Pore-water pressures (negative and positive), total earth pressures, the slip surface and the landslide process were recorded during the simulation process. Data gathered under controlled laboratory conditions was useful for the verification of analytical and numerical modelling methods. Pore-water pressure inside the slope showed a significant delay relative to the drawdown of the water level outside the slope. The failure mode that developed during rapid drawdown was of the multiple retrogressive rotational type. The results provide an improved understanding of the physical behaviour and failure mode of a saturated–unsaturated soil slope subjected to water level fluctuations.

© 2009 Elsevier B.V. All rights reserved.

1. Introduction

The focus of most slope instability studies is directed at an evaluation of the shear strength of the soil and the internal pore-water pressures at the time of failure. However, there is a class of slope instability problems where failure is induced by changes in the slope boundary conditions. A situation of particular interest involves the raising or lowering of the water level immediately adjacent to a slope. This class of slope instability problems often involves the modelling of the unsaturated soil portion near to ground surface as well as infiltration associated with rainfall.

There are numerous approaches that can be followed when studying this class of slope instability problems. Numerical simulation of changes in the boundary conditions as well as the pore-water pressure changes in the saturated and unsaturated soils can contribute to an understanding of the trigger mechanisms leading to failure. Physical modelling using a 1 g physical model or a multi-gravity model in a centrifuge can also assist in understanding conditions leading to slope instability.

The primary objective of this paper is to describe a large-scale, 1 g physical model and observe the slope behaviour when the water level adjacent to the slope is changed. This paper presents details of the 1 g model test as well as an analysis of the test results. The pore-water pressures (negative and positive), total earth pressures and the landslide process were recorded during the raising of the water level as well as during the drawdown process. The results provide an improved understanding of the behaviour and failure mechanism of an unsaturated soil slope subjected to external water level changes. This paper also provides background on a number of case histories that fall under the category of slope instability induced by fluctuations of the water level adjacent to the slope.

2. Example slope instability case histories associated with changing boundary conditions

The slopes adjacent to reservoirs and rivers often experience instability as a result of fluctuations in water levels adjacent to the slope. There have been numerous reports of slope failure associated with water level fluctuations (Morgenstern, 1963; Lane, 1967; Nakamura, 1990; Liao et al., 2005). The rapid drawdown of the water level adjacent to a slope may occur when the river level drops following a flood stage. The water level in a reservoir or a canal can drop suddenly due to a breach in a dyke, a sea level rise or a storm tide. Jones et al. (1961) investigated landslides that occurred in the vicinity of Roosevelt Lake in United States from 1941 to 1953. It was found that about 30% of the landslides occurred as a result of drawing down the water level in the reservoir. Nakamura (1990) reported that about 60% of the landslides around reservoirs in Japan occurred under drawdown conditions.

Raising the water level in rivers, lakes, reservoirs, and canals can also trigger landslides. The initial filling of the Panama Canal was cited by Lane (1967) as an example where landslides on cut slopes were triggered by the initial raising of the water level. There have been
numerous slope failures associated with water level fluctuations during the operation of the Three Gorges Reservoir in China. These landslides have also resulted in fatalities and great economic losses. One example of a major landslide along the Three Gorges Reservoir is the Qianjiangping landslide which involved a mass movement of $2.4 \times 10^7$ m$^3$ (Liao et al., 2005). The behaviour of slopes subjected to the water level fluctuations has increasingly become a subject of scientific attention and research.

2.1. Numerical modelling of changing boundary conditions

Numerical modelling using the finite element method has been extensively used to study complex problem such as changing water levels adjacent to a slope. Most studies have investigated the influence of a sudden drawdown on the stability of slopes (Griffiths and Lane, 1999; Lane and Griffiths, 2000; Rinaldi et al., 2004; Zhang et al., 2005; Luo et al., 2005, Zhan et al., 2006). Numerical studies have investigated the influence of such factors as slope geometry, soil properties, the rate of water level rise and water level drawdown. Numerical modelling studies are not overly costly to undertake. These studies have provided improved insight into slope performance and the failure mechanisms associated with the water level fluctuations. It is difficult, however, for these numerical studies to reliably simulate all details of the failure mode and mechanism.

2.2. Physical model tests for changing boundary conditions

Physical model experiments provide an improved insight into failure modes and mechanisms associated with changes in water level adjacent to a slope. These studies are costly to perform and few such studies have been undertaken. Small-scale 1 g tests have been conducted by Zhang et al. (2004), and Hu et al. (2005). Zhang et al. (2004) investigated the behaviour of a layered soil slope under water level fluctuations. It was observed that the soil mass developed tension cracks on the surface of the slope during water level fluctuations. These cracks became wider and led to the development of a main sliding block. Hu et al. (2005) conducted three small-scale 1 g physical model experiments on a model table to investigate the potential deformation features and failure mechanism associated with the Zhaooshuling landslide along the Three Gorges Reservoir (China). The study was undertaken for conditions of reservoir impoundment and water level fluctuation. The experimental results indicated that for a landslide to occur, the failure mechanism must be of the “pull-type” where the lowering the water level tends to pull on the lower portion of the slope.

Small-scale 1 g model tests have provided a general indication of slope behaviour related to water level lowering adjacent to a slope. The reduced scale models may not provide a complete understanding of the response that might occur for similar larger scale systems. It is possible to install more instrumentation internal to a large-scale model than a small-scale model, thereby providing an improved understanding of changes in pore-water pressures and related movements. There are also significant differences in stress level between small-scale and large-scale tests. Concern over stress level differences has led to the use of centrifuge model tests where the stress conditions can be scaled.

A number of centrifuge model tests have been conducted by other researchers. Xu et al. (2005) used centrifuge model tests to investigate instability associated with levees placed on soft foundation soils during rapid drawdown. A well-defined overall slip surface was observed to pass through the toe of the slope. Several tension cracks appeared at the crest of the slope but no slide block formed. Naoki et al. (2004) conducted centrifuge model tests with soils of differing coefficients of permeability. The rate of water reservoir level lowering was used to study seepage flow during drawdown. Sliding failures on the upstream slope were observed. A near-circular type slip surface developed when failure occurred in response to a sudden drawdown of the water level adjacent to the slope. Test results showed that the residual pore-water pressures during rapid drawdown directly led to mass movements of the upstream slope. Centrifuge model tests have a deficiency related to grain size effects and side boundary effects. Large-scale and also full-scale model tests have an advantage when studying the performance of slopes during water level fluctuations.

Several full-scale studies have also been reported in the literature. Rinaldi et al. (2004) installed a series of tensiometers and piezometers in a bank along the Sieve River, in Tuscany, Italy. The riverbank was monitored for 4 years to investigate pore-water pressure changes in the soil in response to flow events. It was concluded that relatively small changes in positive pore-water pressures reduced the effective stresses and annulled the shear strength contribution from matric suction. A sudden loss in confining pressure provided by the river during the initial drawdown was responsible for triggering mass movement. Luo et al. (2005) investigated the deformation characteristics of reservoir-triggered landslides. The Shilushubao landslide along the Three Gorges Reservoir was simulated using a large-scale test system.

3. Scale effects and significance of large-scale model tests

There are significant scale effects between laboratory models and natural slopes. The scale effects occur primarily as a result of differences in stress level between the model slopes and the natural slopes. Large-scale model tests are advantageous when investigating the mechanisms of slope failure. The larger the size of the model, the smaller the difference in stress level between the slope model and the natural slope. The large-scale model tests conducted as part of this study had geometric dimensions (15 m by 6 m by 5 m) quite similar to some small natural slope failures adjacent to a river or an embankment. The slope model used a large quantity of soil making it easy to install instrumentation to observe soil behaviour. The material properties, initial state, and boundary conditions were well controlled. The large-scale model tests are costly to perform but are useful in meeting special needs required for the verification of soil behaviour and the validation of numerical models. The results of the large-scale experiments in this study can be compared with the calculated factors of safety for slope stability. Positive and negative pore-water pressures can be measured and confirmed through numerical simulation. The large-scale experiments provide insight into failure mechanisms and failure modes. These studies also enable the verification of constitutive models for soils. In particular, unsaturated soil constitutive models require further validation as they are put into engineering practice.

4. Test facility for 1 g physical modelling

The test facility and general arrangement of the slope model box are illustrated in Fig. 1. The main element of the test facility is the massive steel model box. The model box has inside dimensions of 15.0 m × 5.0 m in plan view and is 6 m deep. The side walls of the model box consist of 5 mm thick steel plate which is braced to provide lateral rigidity to the sidewalls. One sidewalk has 5 rows of 20 mm thick acrylic windows to enable visual observation of deformations and movement along the failure surface (Figs. 12 and 17). The two longer sidewalls of the model box are smooth and rigid to simulate plane strain conditions. Teflon lubricant was used as an interface along the sides of the model box to reduce friction to a minimum. A concrete base slab provided a rigid foundation for the soil model box.

A water level control system was designed to regulate the rise and drawdown of water in the slope model. The water level control system is illustrated in Fig. 2. The water level control system consisted of a movable water storage tank, a main supply tube and a network of branch tubes with water exit points (Fig. 2). The water storage tank
was equipped with a constant-head control device. The water storage tank can be elevated or lowered to any level through the use of a manual hoist. A network of branch tubes was designed to distribute water throughout the bottom of the slope model. The spacing of the water exit points along each branch tube was 200 mm. The network of branch tubes was embedded in a layer of coarse sand with a thickness of about 200 mm to provide uniformity of the water pressure distribution. A layer of geotextile material was placed on the top surface of the coarse sand layer. Changes in water level inside the model slope were controlled by injecting water into the sand layer or drawing water out of the sand layer through a control valve. A valve was placed on the sidewall adjacent to the toe of the slope at an elevation of 2 m and was used to draw water from the storage tank during the simulation of drawdown conditions. The valve could also be used to control the rate of the drawdown process.

4.1. Properties of the soil used in the 1 g slope model

The soil used in this slope model experiment is sandy silt which was obtained from a deep excavation near the Qiantang River in Hangzhou, China. The particle size distribution was measured in a hydrometer test and the results are shown in Fig. 3. The soil consists of 12% sand, 80% silt and less than 5% clay size particles. The specific gravity of the soil is 2.69. The maximum dry density from a standard compaction test was 1570 kg/m³ and the corresponding optimum water content was 18%.

The drying and wetting soil–water characteristic curves for the sandy silt were obtained using a volumetric pressure plate extractor and the results are shown in Fig. 4. The air-entry value of the sandy silt (on the desorption curve) is approximately 20 kPa.

The saturated hydraulic conductivity, \( k_s \) of the sandy silt ranged from \( 4.2 \times 10^{-6} \) m/s to \( 6.4 \times 10^{-6} \) m/s, with an average value of \( 5.3 \times 10^{-6} \) m/s. The shear strength of the silt was measured using conventional isotropic consolidated, undrained triaxial compression tests. The tests show that the cohesion and internal friction angle of the sandy silt are 1 kPa and 30°, respectively. The stress-strain curves for the sandy silt are shown in Fig. 5.

5. Preparation of the 1 g model slope

The large-scale slope model was prepared in the steel model box. The geometry of the model slope is shown in Fig. 1. The slope model consisted of a homogeneous soil slope that was 6 m thick on the upslope portion and 2 m thick on the down-slope portion. The slope model angle was 45° and the net model slope height was 4 m. The distance back of the crest was 5 m. The distance outside the toe of the slope was 6 m. The overview of the slope model after construction is shown in Fig. 6. Approximately 300 m³ of soils was required to construct the slope model.

The “raining” method appeared to be impractical for the placement of the slope soil in this situation and therefore a belt-type conveyor and a clamshell bucket were used to transport the soil to the slope model box. The sandy silt was dropped from the clamshell bucket at a constant height of about 3 m above the soil surface. A tape 3 m long was suspended from the bucket hinge. The operator could open the...
bucket when the tape just touched the soil surface. The clamshell bucket was attached to a movable overhead crane. The height of the bucket was controlled by the crane operator. The maximum scooping capacity of the clamshell bucket is about 0.75 m³. After placement, the soil slope was manually trimmed to form the desired geometry.

The (gravimetric) water content of the sandy silt ranged from 10% to 26% (i.e., close to optimum water content 18%). No additional compaction effort was applied to the soil after it was dropped from the bucket. The matric suction of the soil is known to primarily be a function of placement water content (Fredlund and Rahardjo, 1993) and therefore compaction should not significantly affect the initial matric suction value. The initial matric suctions were measured in the range of 20 to 40 kPa. The fill material at the completion of constructing the slope model was in a relatively loose state, particularly the soil layers near the slope surface. Fig. 7 shows a profile of the density for each soil lift. The density was measured through borehole sampling after completing the construction of the slope model. The dry density ranged from 1.26 to 1.43 g/cm³, which is lower than the maximum dry density corresponding to standard compaction test (1.57 g/cm³). These results indicate that the fill material was relatively loose and somewhat non-uniform.

A cone penetrometer was used to perform a site investigation on the soil slope. A constant penetration rate of 20 mm/s was used and the results are shown in Fig. 8. The diameter of the cone was 35.7 mm,

and therefore compaction should not significantly affect the initial matric suction value. The initial matric suctions were measured in the range of 20 to 40 kPa. The fill material at the completion of constructing the slope model was in a relatively loose state, particularly the soil layers near the slope surface. Fig. 7 shows a profile of the density for each soil lift. The density was measured through borehole sampling after completing the construction of the slope model. The dry density ranged from 1.26 to 1.43 g/cm³, which is lower than the maximum dry density corresponding to standard compaction test (1.57 g/cm³). These results indicate that the fill material was relatively loose and somewhat non-uniform.

A cone penetrometer was used to perform a site investigation on the soil slope. A constant penetration rate of 20 mm/s was used and the results are shown in Fig. 8. The diameter of the cone was 35.7 mm,
6. Instrumentation in the 1 g soil model

Several types of instrumentation were installed to monitor the behaviour of the slope model. The instruments included jet-fill tensiometers, vibrating earth pressure cells, vibrating-wire piezometers, tiltmeters, and LVDTs. The layout and locations of the instruments are schematically shown in Fig. 9. Details of the instrumentation program are summarized in Table 1.

Tensiometer tips were installed along the rear sidewall of the tank to measure changes in matric suction. A total of five jet-fill tensiometers were installed along one section at elevations of 1.35 m, 2.85 m, 3.35 m, 4.85 m, and 5.85 m. The arrangement of the tensiometers along the slope model is shown in Fig. 9.

Two pairs of earth pressure cells were installed at the 2 m elevation of the slope model. The intent was to measure the vertical and horizontal total stresses in two orthogonal directions and the changes generated during water level rise and drawdown. Each Geokon earth pressure cell consisted of two circular stainless steel plates welded together along the perimeter, creating a narrow cavity that was filled with a low compressibility fluid (glycol). A length of steel tubing connected the cavity to the transducer housing which contained a semiconductor strain gauge pressure transducer. The cells had a diameter of 230 mm and an aspect ratio of 18 (i.e., ratio of diameter to thickness). The cells were custom built to ensure adequate sensitivity under low earth pressure conditions. An installation procedure similar to that used by Zhan et al. (2007) was adopted. When the slope model was constructed to 2 m elevation, 4 earth pressure cells were installed. For the vertically placed cells which measure the horizontal stress, a 250 mm deep slot was excavated for the vertical flat plate of the pressure cell. After the cell was installed, sandy silt soil was used to fill the narrow clearance between the sidewall of the slot and the pressure cell to ensure satisfactory contact. For the horizontally placed cells which measure the vertical stress, a 50 mm deep pit was excavated for the horizontal flat plate of the pressure cell. After the cell was installed, sandy silt soil was used to refill the slot. After placement of each cell, sandy silt was hand tamped to ensure satisfactory soil-cell contact.

Piezometers were placed in pairs at different elevations providing intentional redundancy to the measured pore-water pressure under low earth pressure conditions. An installation procedure similar to that used by Zhan et al. (2007) was adopted. When the slope model was constructed to 2 m elevation, 4 earth pressure cells were installed. For the vertically placed cells which measure the horizontal stress, a 250 mm deep slot was excavated for the vertical flat plate of the pressure cell. After the cell was installed, sandy silt soil was used to fill the narrow clearance between the sidewall of the slot and the pressure cell to ensure satisfactory contact. For the horizontally placed cells which measure the vertical stress, a 50 mm deep pit was excavated for the horizontal flat plate of the pressure cell. After the cell was installed, sandy silt soil was used to refill the slot. After placement of each cell, sandy silt was hand tamped to ensure satisfactory soil-cell contact.

Piezometers were placed in pairs at different elevations providing intentional redundancy to the measured pore-water pressure

### Table 1

<table>
<thead>
<tr>
<th>Measurement Type of instruments</th>
<th>Quantity</th>
<th>Measuring range</th>
<th>Source/reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil suction Jet fill tensiometer</td>
<td>5</td>
<td>&lt;90 kPa</td>
<td>Soilmoisture Equipment Corp, Santa Barbara, USA</td>
</tr>
<tr>
<td>Pore-water pressure Vibrating-wire piezometer</td>
<td>6</td>
<td>0–170 kPa</td>
<td>Geokon Inc, USA</td>
</tr>
<tr>
<td>Horizontal stress Vibrating-wire earth pressure cell</td>
<td>2</td>
<td>0–350 kPa</td>
<td>Geokon Inc, USA</td>
</tr>
<tr>
<td>Vertical stress Vibrating-wire earth pressure cell</td>
<td>2</td>
<td>0–700 kPa</td>
<td>Geokon Inc, USA</td>
</tr>
<tr>
<td>Slope inclination Tiltmeter</td>
<td>2</td>
<td>± 5°</td>
<td>Applied mechanics Corporation, Canada</td>
</tr>
<tr>
<td>Vertical displacement LVDT</td>
<td>4</td>
<td>6 in.</td>
<td>Schaeftz, USA</td>
</tr>
<tr>
<td>Displacement Flux Pin marker</td>
<td>40</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Flux Flowmeter</td>
<td>1</td>
<td>N/A</td>
<td></td>
</tr>
</tbody>
</table>

![Fig. 9. Layout of instruments in the slope model (dimensions are in meters).]
results. There was also the possibility that a particular piezometer might fail during the experiment. It was also rationalized that more than one pore-water pressure measurement at the same elevation would provide information on the non-homogeneity of the soil.

Buried pin markers were placed on the side of the slope model adjacent to the acrylic glass window. These pins were used to measure soil movement within the soil mass and to identify the location of the internal slip surfaces. Movement within the soil mass of the slope model was monitored using the 10 mm diameter acrylic polyvinyl (PVC) marker rods. The rods were 150 mm long and were placed in the soil mass with the long axis in the plane strain direction.

Each instrument was calibrated prior to installation. All the instruments located inside the slope model were installed during soil placement. Apart from the tensiometers and the pin markers, all other electrical instruments were connected to a data-logger for automatic recording of data. Each electronic instrument was logged at 10 min intervals. However, during the raising and lowering of the water level the logging interval was 10 s.

7. Test program for the large-scale model test

The experimental program focused mainly on the performance of the slope model when the water level was raised and lowered. The primary purpose of the experiment was to investigate the effect of the rate at which the water was raised and the rate at which the water was drawn down. The failure mode associated with the water drawdown rate was of particular relevance.

The first stage of the test program evaluated the ability of the constant-head water storage tank to simulate the rising of the underground water level. Fig. 10 shows the staged elevation history of the water level in the constant-head water storage tank (0–192 h). The water level program consisted of 6 elevation steps with each elevation increment being 1 m. Each water level elevation was held for a period of 24 h. The water levels inside and outside the large-scale slope model were monitored during this time.

The second stage involved opening the water level control valve to drain water from the tank. In other words, the water level outside the slope model was drawn down from 5.6 m to 3 m in elevation within 152 min. The sudden drawdown stage was started after the final water level step had been sustained for 72 h. In the second stage, the valve of the water storage tank was closed. The water level outside the slope model is shown in Fig. 10. The drawdown rate was about 1 m/h.

8. Presentation and discussion of test results and observations

Initial readings were taken on the instrumentation after the slope model was constructed. The water level in the constant water storage tank was then raised.

8.1. Response of slope model to a rise in water level

The water level in the constant water storage tank was raised in 1 m steps. The hydraulic head in the tank was greater than the hydraulic head in the slope model. Consequently, water flowed into the slope model and the wetting front produced an increase of the water level in the model slope. This process was meant to simulate a rise in the groundwater level.

Two phenomena were observed while raising the water level. First, the crest of the slope model settled. Fig. 11 shows the surface settlements monitored using 4 linear variable displacement transducers (LVDTs) installed along the crest of the slope. Settlement along the crest ranged from 150 to about 350 mm. The total height of the slope was 6 m, therefore the volumetric strain of the soil mass ranged from 2.5 to 5.8%. The observed settlements are likely attributable to wetting-induced collapse of the unsaturated loose silt. Others have observed similar wetting-induced collapse phenomena. For example, Tadepalli et al. (1992) performed wetting tests on Mississippi silt having a dry density of 1.32 g/cm³ and observed a volumetric strain of about 1.1%.

Second, the loose fill under the sloping surface collapsed gradually as the water level rose (Fig. 12). The wetting-induced collapse resulted in a reduction in the shear strength of the loose sandy silt. The reduction in shear strength was related to a reduction in the initial matric suctions (i.e., in the order of 5.8 to 29 kPa) to values of zero corresponding to saturated conditions (see Fig. 13). The initial slope angle of 45° was reduced to approximately 33° as the water level rose to near the crest of the slope model. A final slope angle of 33° was observed through the perspex windows along the side of the slope model. A comparison of the slope geometry before and after the water level rising is shown in Fig. 19. Fig. 12(d) shows an overview of the slope model when the water level rose to near the crest of the slope (i.e., 5.6 m).

8.2. Matric suction response to the rising water level

Fig. 13 shows the matric suction values measured in five tensiometers in response to raising the water level. The five
Fig. 12. Appearance of the model slope when raising the water level.
tensiometers (T1–T5) were located at the following elevations, 1.35 m, 2.85 m, 3.35 m, 4.85 m, and 5.85 m. The line joining the solid circular symbols in Fig. 13 represents the water level measured inside the slope. The water level is derived from tensiometers. The measurements of matric suction on the tensiometers that went to zero indicate that water level reached the ceramic tip of the tensiometer according to the adsorption curve of the soil–water characteristic curve. The water level measurements show the time required for saturation to occur. Prior to raising the water level, the measured initial matric suctions ranged from 5.8 to 29 kPa (Fig. 13). The higher matric suction values were near the top of the slope.

All five tensiometers showed similar behaviour as the water level was raised. Each tensiometer showed that the initial matric suction was maintained until water rose to the soil near the tensiometer. Once water reached the tensiometer, the matric suction decreased quite rapidly and eventually reduced to 0 kPa. It appears that the tensiometers at locations 1.35 m, 2.85 m, 3.35 m, 4.85 m, and 5.85 m required about 8 h, 56 h, 78 h, 120 h, and 190 h, respectively, to reach saturated conditions. All five tensiometers showed similar behaviour as the water level was raised. Each tensiometer showed that the initial matric suction was maintained until water rose to the soil near the tensiometer. Once water reached the tensiometer, the matric suction decreased quite rapidly and eventually reduced to 0 kPa. It appears that the tensiometers at locations 1.35 m, 2.85 m, 3.35 m, 4.85 m, and 5.85 m required about 8 h, 56 h, 78 h, 120 h, and 190 h, respectively, to reach saturated conditions.

Fig. 14 illustrates the variation in water level and wetting front with time within the model slope. The wetting front was observed as the point at which the tensiometer reading started to change. The distance between the water level and the wetting front is called the “height of wetting front”. The height of the wetting front ranged from 0.35 m to 0.75 m and appeared to be related to the matric suction in the soil. The higher the initial matric suction, the lower the height of the wetting front. It can be observed that the initial water content significantly influenced the height of wetting front.

8.3. Pore-water pressures in response to raising the water level

Pore-water pressures measured from the 5 piezometers are shown in Fig. 15 (Note: one of the installed piezometers failed). The pore-water pressures measured in the piezometers at different elevations increased slowly with time in response to changes in water level in the constant-head water storage tank.

Piezometers P-1, P-2, P-3, P-4, and P-5 were installed at elevations 0 m, 0 m, 2 m, 2.0 m, and 2.5 m, respectively. Once the valve was opened to allow water to flow into the slope, piezometer P-1 started to respond. When the hydraulic head difference between the water storage tank and piezometers in the slope became low, the rate of pore-water pressure change also became low, and vice versa. Piezometer P-4 located at an elevation of 2 m began to respond to the water level differential 24 h after the commencement of the experiment. The water level inside the slope was then at 2 m. These results are consistent with the pore-water pressure measurements obtained from P-4.

As compared with the water level change in the water storage tank, a delay was observed for the pore-water pressure response measured by all the piezometers, even for piezometers P-1 and P-2 located at the top of the coarse sand layer (Fig. 15). The delay in the pore-water pressure measurement is likely due to entrapped air in the cavity around the piezometers. Fig. 15 also shows that there is a difference between the water level in the storage tank and the water level interpreted from the measured pore-water pressures. The difference was up to 0.6 m when 192 h elapsed after the commencement of the experiment. When the water level in the storage tank is at the elevation of 6.1 m, the water level interpreted from the measurements of pore-water pressures is at the elevation of 5.5 m, and the water level interpreted from the responses of tensiometers is at the elevation of 5.8 m.

8.4. Earth pressures measured in response to raising the water level

Fig. 16 shows the total stresses measured on the 4 vibrating-wire earth pressure cells (EPCs) installed in the slope model. Changes in total stress are in response to raising the water levels. All earth pressure cells were installed at an elevation of 2 m.

Pressure cells EP-1(H) and EP-2(H) measured total stress changes acting in the horizontal direction whereas EP-1(V) and EP-2(V) recorded total stress pressures acting in the vertical direction. The two...
vertical pressure cells showed a similar general trend during the raising of the water level. Before the water level inside the slope reached the locations of earth pressure cells, EP-1(V) and EP-2(V), the vertical total stress increased significantly and reached a peak value. Then the vertical total stress decreased gradually towards a new constant value.

The vertical total stresses recorded by EP-1(V) increased from an initial total stress of 72 kPa to 112 kPa. The incremental change of 40 kPa occurred within 26 h after the start of the experiment. The recorded total vertical stress gradually decreased with time as the water level passed the pressure cells and approached a final value of 70 kPa. The vertical total stress recorded by EP-2(V) increased from an initial total stress of 52 kPa to 70 kPa. The incremental change of 18 kPa occurred within 26 h after the start of the experiment. The vertical total stress gradually decreased as the water level passed the pressure cells and approached a final value of 57 kPa.

The horizontal total stresses measured in pressure cells EP-1(H) and EP-2(H) shows similar results in the last 144 h. (Note: Data for EP-1(H) was lost prior to that time). The horizontal total stress recorded by EP-1(H) showed a gradual increase from an initial value of 15 kPa to a final value of 38 kPa as the water level rose.

The incremental change in total stress prior to the water level having reached the locations of earth pressure cells may be due to air entrapment. The differential settlement inside the slope may also have induced a stress concentration. If the horizontal total stress increment was entirely caused by entrapped air, it is anticipated that the magnitude of the increment should have been about 10 kPa according to the EP-1(H) value at 26 h. Since air pressure is isotropic, the magnitude of the vertical total stress increment should also have been about 10 kPa. The vertical total stress increment for earth pressure cell, EP-1(V) was 40 kPa within 26 h. EP-1(V) was located close to EP-1(H). It is anticipated that an increment of about 10 kPa was due to entrapped air and an increment of 30 kPa was caused by differential settlement. The incremental change caused by differential settlement for EP-2(V) appears to be about 8 kPa.

The slope appears to have become more uniform as the water level in the slope was raised. Stress concentrations associated with differential settlement appear to have become smaller with time. The total vertical stresses decreased gradually to a constant value after the cells were immersed in water. The decrease in total stress may also have been due to the dissipation of entrapped air. The horizontal total pressure appears to have been primarily influenced by water pressure and the horizontal effective stress is rather small. As the water level became higher, the horizontal total pressure also increased.

8.5. Response of slope model during the drawdown process

The sudden drawdown of the water level outside the slope began after a water level of 6 m was maintained in the water storage tank for 72 h. At that time, there appeared to be no further change in the slope geometry. Then the slope surface was subjected to a rapid drawdown condition.

8.5.1. Visual observations made during drawdown

The drawdown of water level was implemented by opening the control valve located above the toe of the slope model at an elevation of 2 m. The discharge flow rate depended upon the head above the discharge valve and ranged from 70 to 35 m³/h. The change of water level with time during the drawdown process is shown in Fig. 10. The rate of drawdown was approximately 1 m/h. A total of 2.6 h was required to lower the water level from 5.6 m to 3.0 m.

A video camera was set up in front of the slope face to record the drawdown process. The intent was to also capture failure initiation and subsequent movements. Slope profiles before and after failure, along with displacements associated with each slide block were recorded. The video is available on the website http://www.ssgeo.zju.edu.cn/show.aspx?id=151&cid=144.

The failure of the slope model was systematically shown in Fig. 17a, b, c, and d. The following phenomena were observed during the water level drawdown process. The drawdown of the water level outside the slope initiated the formation of a lateral crack across the crest of the slope. The crack formed about 15 min after the commencement of drawdown and is most likely due to the reduction in lateral restraint. The tension crack transverse the slope at a location 0.4 m back of the crest of the slope. The length and width of the tension cracks increased with the lowering of the water level. The number of tension cracks increased as the water level reached the lower portion of the slope. When the water level outside the slope was drawn down to 0.7 m, one part of the soil mass began to form a large slide block (i.e., named Block 1). This slide block moved downward rapidly at a rate of approximately 0.015 m/min. The slip surface of Block 1 was relatively shallow ranging from 0.5 m to 1.0 m in depth. The movement of Block 1 took on the form of a rotational slip.

The slope material behind Block 1 appeared to remain stable but many thin tension cracks formed. When the water level was drawn down to 1 m, the crest of the first slide block had displaced about 0.5 m. There was a substantial transverse crack pattern across the crest of the slope but there no further sliding blocks appeared to be developing. At the same time, the soil mass adjacent to the first slide started to pull downward. With a further lowering of the water level, the transverse cracks became wider and wider, and further settlement took place.

When the water level was drawn down to 1.7 m, a second sliding block (i.e., Block 2) developed. Sliding Block 2 took the form of a rotational type slip. The depth of the slip surface was deeper than the first block, being about 0.7 to 1.5 m deep. The rate of movement of Block 2 was at a moderate rate of approximately 0.005 m/min. The maximum displacement of Block 2 was about 1.5 m. A seepage exit point was observed about 1 m above the water table following the failure of Block 2. The seepage exit point indicated that there was outward seepage from the slope.

A third sliding block (i.e., Block 3) appeared to start at the same time as the second sliding block. As more sliding occurred and a soil mass accumulated at the foot of the slope, the sliding mass moved slower and slower. The sliding mass finally stopped and it was not possible for Block 3 to fully develop. The observational evidence indicated that all three sliding blocks took on a two-dimensional character. Observations of movement would indicate that the lubrication treatment along the lateral sides of the model box were effective.
8.5.2. Observed failure mode

The observations of the failure mechanism of the slope subjected to sudden drawdown revealed a system of multiple retrogressive, rotational sliding blocks (Fig. 18). Slide Block 1 moved faster than the other blocks. Movement in Block 1 reduced the horizontal restraint on Block 2 and Block 3. Failure of the later two slide blocks was accentuated by outward seepage from the slope. The slope geometries before and after soil movements are shown in Fig. 19.

There are few large-scale experiments reported in the research literature where the failure modes associated with sudden drawdown of water submerging a slope has been fully documented. In particular, there have been few documented cases of the processes associated with incipient failure as a result of drawdown.

Several centrifuge model tests have been conducted where the failure mode associated with the sudden drawdown on a landslide were investigated. Xu et al. (2005) observed a well-defined sliding surface passing through the toe of the slope. Naoki et al. (2004) found that a near-circular slip surface developed when failure occurred. Small-scale tests by Zhang et al. (2004) observed that tension cracks became wider and led to a main sliding block.

The failure mechanism observed and documented as part of this large-scale slope model test was shown to be quite complex. The study also identified three stages in the development of failure conditions. The soil mass failed as three retrogressive rotational sliding blocks. Numerous tension cracks were observed at the crest of the slope.

8.5.3. Comparison of water levels and pore-water pressures during drawdown

Fig. 20 shows the pore-water pressure responses recorded by the piezometers at an elevation of 2 m. The change of water level outside
the slope with time is also shown in Fig. 20. The water level outside the slope was drawdown for 2.6 m and the pore-water pressure inside the slope (P-4) only decreases 10 kPa (converted to 1 m equivalent depth of water). There was a significant delay in the pore-water pressure response relative to the drawdown water levels outside the model slope. In other words, the piezometric water levels inside the slope model were always higher than that outside the slope model. Seepage water was clearly observed to exit at points on the slope surface. The drawdown rate of the water level submerging the slope was significantly higher than the drawdown rate of the phreatic line inside the slope. When the water level outside the slope was drawn down to 1.7 m, the seepage exit point was observed about 1 m about the water table. The water head difference will cause outward seepage from the inside of the slope. The movement of water towards the surface of the failed slope appears to have significantly affected the stability of the slope.

Based on the phenomenon observed in the experiment, it would appear that any measures that can lower the water levels inside the slope should help to stabilize the slope. The use of horizontal drains would appear to be an effective means of improving stability in engineering practice. Horizontal drains are holes drilled into a slope and cased with a perforated metal or slotted plastic liner. The drains release the high pore-water pressures near the slip surface and increase the stability of the slope.

8.5.4. Earth pressure and pore-water pressure response to the drawdown of the water level

Fig. 21 shows the results of pore-water pressures measured by piezometer P-4 and the total stresses measured by 4 earth pressure cells in response to the drawdown of the water level. Piezometers P-5 and P-6 were displaced considerable distances by ground movements, resulting in erratic readings. Consequently, the sensor readings were terminated and no further results are shown in Fig. 21. Pore-water pressures recorded by piezometer, P-4 located 2.5 m towards the rear of the slope decreased from an initial value of 38 kPa to 28 kPa.

The vertical total stress recorded by earth pressure cell, EP-1(V) located 3 m in a horizontal direction towards the rear of the slope decreased from an initial value of 70 kPa to 54 kPa. The horizontal total stress recorded by earth pressure cell, EP-1(H) located 3 m in a horizontal direction towards the rear of the slope decreased from an initial value of 38 kPa to 25 kPa. The total stress decreases appear to be related to a pore-water pressure decrease of approximately 10 kPa. The decrease in total stress associated with a reduction in the thickness of the overlying soil should be about 3 kPa. The measurements show that the decrease generated by changes in pore-water pressure plays a significant role in reducing total horizontal stresses during the drawdown process.

Total vertical stresses recorded by earth pressure cell, EP-2(V) located 5 m in horizontal direction towards the rear of the slope decreased from an initial value of 57 kPa to 38 kPa. The horizontal stresses recorded by earth pressure cell, EP-1(H) located at a similar location decreased from an initial value of 38 kPa to 22 kPa. The total stress decrease associated with the pore-water pressure decrease is about 10 kPa. The decrease in total stress associated with the reduction in overlying soil should be about 6 kPa. The measurements again show that the decreases in pore-water pressure play a significant role in reducing total horizontal stresses during the drawdown process.

9. Concluding remarks

A large-scale 1 g model test simulating the performance a loose, sandy silty soil slope was subjected to external water level fluctuations. The model test revealed multiple retrogressive and rotational type failure modes associated with the sudden drawdown of water adjacent to a slope. Based on the results of the slope model study, the following conclusions can be drawn:

1.) Large settlement occurred at the crest during the raising of the water level. The observed settlements appear to be associated with a soil structure collapse induced by wetting.
2.) During the raising of the water level there was differential settlement due to the water-induced collapse of the soil. This process resulted in an increase in total stresses. It also appears that there may have been excess air pressures built up in the soil.
3.) Pore-water pressures inside the slope showed a significant delay relative to the drawdown of water level outside the slope. The delay resulted in an outward movement of water towards the surface of the slope. The seepage forces adversely affected the stability of the slope.
4.) The failure mode that developed during rapid drawdown resulted in three retrogressive rotational slides. The three sliding blocks shared the same basal slip surface.
5.) Based on the observations made during the experimental model slope, it is recommended that wherever possible, measures be used to lower the water levels inside a slope. The use of perforated horizontal drains can provide an effective means of reducing pore-water pressures and increasing the stability of a slope. Their usage is recommended in engineering practice.
Acknowledgements

The authors would like to acknowledge the financial support from the National Key Technology R&D Program funded by the Ministry of Science and Technology of China (2006BAJ06B02) and research grants (50538080 and 50878194) provided by the National Natural Science Foundation of China (NSFC).

References