Experiments on the failure of slopes made of loose sand due to the rise in water level

Reza Imam
Assistant Professor, Amirkabir University of Technology and member, Center for Seismic Upgrade and Optimization of Structures, Sites and Lifelines
Vahid Masumi Fard
Graduate Student, Amirkabir University of Technology, Tehran, Iran

ABSTRACT
A rise in pore water pressure in a soil mass leads to a decrease in the mean effective normal stress, while the vertical gravitational loads may change only slightly. Very loose sand may contract substantially under such loading as failure is approached and, under poor drainage, this can lead to the increase in pore pressure, loss of strength and, consequently, failure of the soil mass. Using results of experiments carried out in a test tank, this paper shows that slope instability resulting from rises in water level is controlled by sand density, pore pressure distribution, and rate of rise in water level.

RÉSUMÉ
Une hausse de la pression interstitielle dans une masse de sol conduit à une diminution de l'efficacité moyenne des efforts, alors que des charges verticales de gravité peut varier légèrement. Très sable meuble peut contracter sensiblement sous chargement tels que la défaillance est abordé et, sous un mauvais drainage, ce qui peut entraîner une augmentation des pressions interstitielles, perte de force et, par conséquent, l'échec de la masse du sol. Utilisation des résultats des expériences réalisées dans un réservoir d'essai, ce document montre que l'instabilité des pentes résultant de l'élévation du niveau d'eau est contrôlée par la densité du sable, de la distribution de la pression interstitielle, et le taux d'élévation du niveau de l'eau.

1 INTRODUCTION

Failures of slopes made of loose granular soils due to rises in water level may be attributed to one or a combination of mechanisms. One mechanism is the decrease in soil shear strength due to saturation, and the loss of suction in the unsaturated soil, leading to the instability at the current slope angle, and the subsequent stabilization at a milder slope.

Another possible cause is related to the tendency of very loose granular soils for contraction, when such soils are subjected to certain stress paths involving the decrease in the mean effective normal stress (see e.g. Anderson and Sitar 1995, Anderson and Riemer 1995). Very loose granular soils have been observed to experience substantial contractions when subjected to loading with constant, or near constant shear (deviatoric) stress, but with decreasing mean effective normal stress. Previous laboratory tests (by e.g. Skopek 1994, Anderson and Riemer 1995, Azizi and Imam 2008) have shown that in such cases, substantial volume contractions may initiate at stress states corresponding to mobilized friction angles well below failure. Such volume contractions observed in drained loading can result in the development of excess pore pressures in undrained or semi-drained loadings, if the drainage path is long enough, or the soil permeability is low enough (Sasitharan et al. 1994), resulting in the loss of the shear strength of the soil and failure of the slope.

If the rate of rise in water level is high, seepage forces may also initiate slope instability. In natural slopes made of loose granular soils, other mechanisms such as the dissolution of cementations among the soil grains may also result in the loss of shear strength and the failure of the slope, when water level within the slope rises.

In the current paper, model tests on the failure of slopes made of loose and very loose granular soils due to the rise in water level are described. The tests were carried out on slopes made of local, very loose, fine sand placed using a method similar to that used for preparing moist-tamped samples used in triaxial testing. Since some of the failures observed in the model tests may be related to the contractions that occur in very loose sands in stress paths similar to those experienced during the constant deviatoric stress (CDS) loading mentioned earlier, behavior of the sand used in the model tests under CDS loading is described based on results of a previous testing program.

Results of model tests showed that slopes made of loose and very loose sand fail as a result of the rise in water level, and stabilize at slope angles that are sometimes substantially lower than those at which the original slope was made. It was also noticed that the mechanism of failure and the final slope at which the sand stabilizes depend on a number of factors such as the sand relative density, original slope angle, method of introduction of water into the slope, time rate of the rise in water level, etc. Possible causes of slope failure in each test are discussed based on the observations made during testing, including the type of slope failure.

Some aspects of the design of the tests such as the introduction of water into the test tank, and then into the soil slope, separation of the soil from the coarser grain filter material, effects of boundary conditions, rate of rise of water level, etc. are also discussed.

2 THE SAND TESTED

The sand tested is a local, predominantly quartzic, fine grained, angular sand called the Firoozkooh No. 151 sand. For this sand, maximum and minimum void ratios are e_{max}=0.87 and e_{min}=0.55, respectively, mean grain size is D_{50}=0.25 mm, and specific gravity is G_s=2.65. Grain size distribution of this sand is shown in Figure 1.

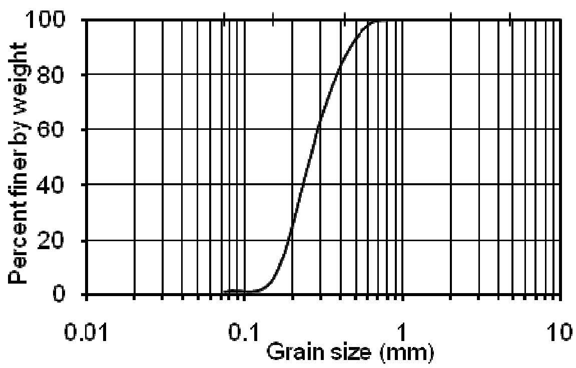

Figure 1 – Grain size distribution of Firoozkooh No. 151 sand

For this sand, angle of repose of the air-dried samples was measured at 34 degrees. The same angle of repose was also measured below water, when sand was gently poured underwater.

3 SAND BEHAVIOR IN CDS LOADING

A number of constant deviatoric stress (CDS) tests were carried out in the triaxial apparatus by Azizi and Imam (2008) on moist-tamped samples of Firoozkooh sand. Since some of the failures observed in the test tank due to the rise in water level may have resulted from the sand following a stress path similar to that of the CDS loading, behavior of this sand in such loading is described in this section.

In preparing the samples for CDS testing, in order to achieve sufficiently low relative densities, the moist tamping procedure was used. A 2.5 percent moisture was added to the soil, and the soil was then poured in seven to ten layers of approximately equal thicknesses, and each layer was tamped with a plastic tamper having a diameter equal to that of the triaxial mould. In order to achieve sample density that is as uniform as possible, the energy used in tamping subsequent layers was gradually increased. This method led to the preparation of samples with mostly negative relative densities. Details of sample preparation and testing are described by Azizi and Imam (2008). This method of sample preparation has also used by others to achieve very loose sand samples (see e.g. Ishihara 1993).

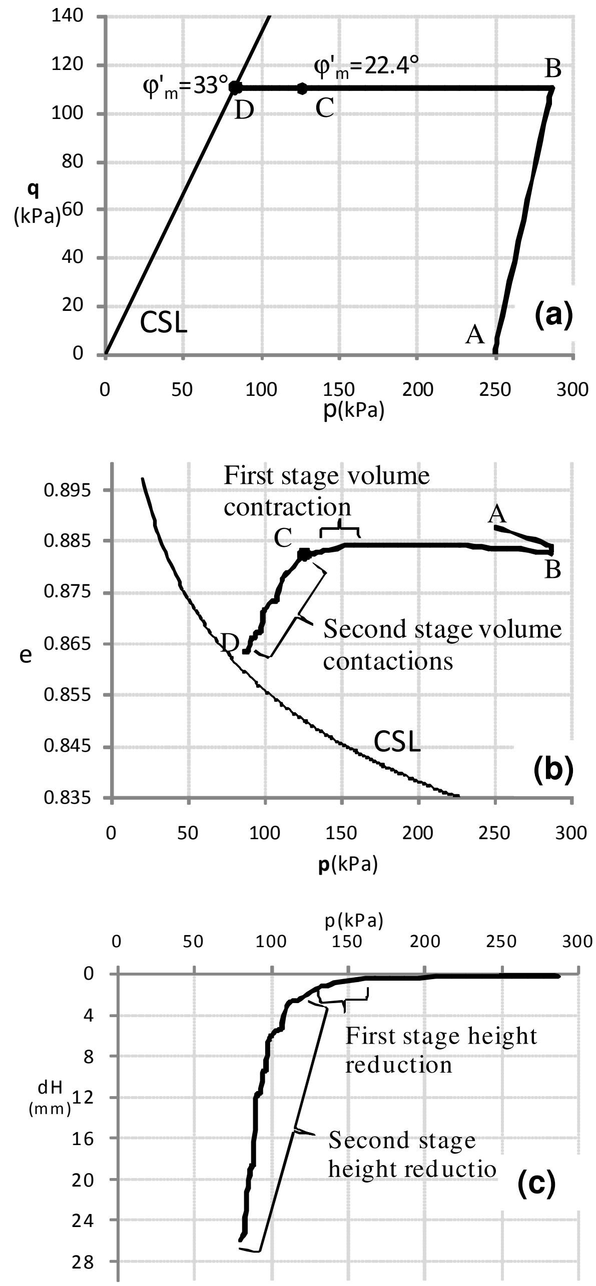

Figure 2. Behavior of Firoozkooh No. 151 sand in CDS loading (modified after Azizi and Imam 2008)

Figure 2 shows results of a CDS loading test on a sample of moist tamped Firoozkooh No. 151 sand. In this test, the sample was first consolidated to a void ratio of 0.888 (relative density of -5%) and mean effective normal stress of 250 kPa (point A), and then sheared to a deviatoric stress of 110 kPa (point B). The shear (deviatoric) stress was applied using a dead load placed on a plate screwed to the upper triaxial platen and was therefore kept constant after it was applied. The mean effective normal stress was then gradually decreased. At a certain stage during the decrease in mean effective normal stress (point C), substantial volume contraction initiated, and this continued until the steady state condition was reached at point D, where no more volume contraction occurred. Prior to the start of substantial volume contraction, a stage of smaller-rate volume contraction was reached. Sample height also experienced similar changes, such that during the CDS stage of loading, it initially remained almost unchanged, but then started to decrease at a small rate, and finally reached a stage of high-rate height reduction before reaching the critical state condition.

In an undrained or semi-drained condition, such tendency for volume contraction may result in the increase in pore water pressure and the decrease in shear strength since, for such loose sand, the undrained effective stress path exhibits strain softening associated with a substantial loss of strength. Under CDS loading, this reduction in shear strength may lead to the inability of the sample to sustain the applied constant shear stress, and its vigorous collapse (Sasitharan et al. 1994).

Anderson and Riemer (1995) showed that the volume contractions observed in CDS loading occur only in loose and very loose sand. As sand density increases, the amount of volume contraction decreases and at high enough relative densities (such as in medium dense to dense sands), the possibility of contraction is completely eliminated (see also Azizi and Imam 2008).

Many failures observed in slopes made of very loose saturated or near saturated granular materials have been attributed to “flow Liquefaction,” and may result from loads that are static or dynamic in nature (see e. g. Morgenstern 1994, Ishihara 1993). These failures also result from the tendency for volume contraction and strain softening of such soils and the resulting loss of strength, which may occur due to a small disturbance or straining. Imam et al. (2002) showed that certain stress paths, such as that applied in CDS loading, lead to such behaviour in loose sands.

4 THE TEST TANK

Figure 3 shows various parts of the test tank in which the model experiments were carried out.

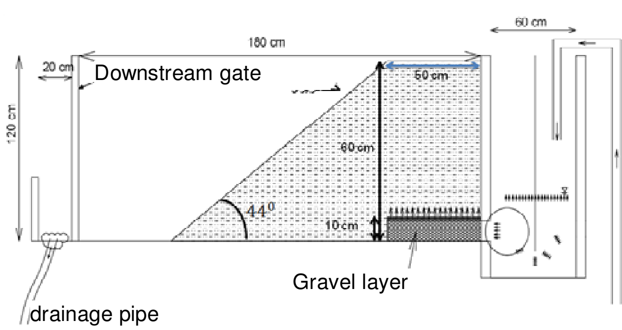

Figure 3. Schematic figure of the tank in one of the tests

The tank is 1800 mm long, 750 mm wide and 1200 mm high. Sheets of 15 mm thick glass on one side and 15 mm thick plexiglass on the other side constitute the tank side walls. The plexiglass is used in anticipation for the possible future need for connecting instrumentation to the side walls. The end walls are made of 10 mm thick Teflon plates that can slide up and down to allow changes in the height of the openings for the entry of water into the tank on the upstream side, and removal of the end wall to allow entry of a person for pouring the sand slope on the downstream side.

Water is pumped into a first tank, and after losing its energy and most of its turbulence, it enters a second tank through an opening that connects the bottom of the two tanks. Laminar flow then enters a 100 mm opening under the upstream end wall and then into a 100 mm thick layer of gravel, from which it then seeps into the sand slope constructed in the tank. In the initial tests, the gravel layer covered the full area of the tank base; however, in later tests, its length was limited and varied depending on slope geometry. A layer of geosynthetic was used to cover the gravel layer such that the sand placed on this layer will not enter the voids between the gravel grains. For tests in which the gravel layer did not cover the full area of the tank base, a 100 mm high, 10 mm thick piece of plexiglass was glued to the bottom and sidewalls of the test tank across the front edge of the gravel layer. This was done since some tests showed a tendency of the gravel layer to move downstream during seepage of water through the gravel.

Figure 4. Overall view of the test tank

For the construction of the sand slope, the downstream gate (wall) of the tank was removed in order to allow a person to enter the test tank and place and tamp the sand in layers. Prior to pouring each layer, weight of the sand needed for placing the layer was estimated, and 2.5% water was added to the sand and thoroughly and uniformly mixed with it. The sand was then carefully placed into a layer with uniform thickness over the surface of the previous layer, after slightly scratching the surface of the previous layer to ensure better contact between the layers. The surface of the new layer was then lightly pressed with a piece of flat 250 mm long, 150 mm wide and 40 mm thick wood. The pressure used in pressing the layer was increased gradually in subsequent layers, such that the lower layers will not be over-compacted by the energy used for compacting the upper layers, and the compaction energy transferred to all layers at the end of construction of the slope remains as uniform as possible. For the construction of the 500 mm high slopes, a total of seven layers, each having approximately 70 mm thickness, were used.

In the tests discussed here, length of the slope crest was 500 to 550 mm, and height of the slope above the gravel layer was 500 to 510 mm. The downstream slope of the fill was 44 degrees in some tests and 34 degrees, which is equal to the angle of repose of the sand, in others.

The rate of inflow of water was controlled using an electronic "dimmer" which was used to change the speed of the inflow water pump. This rate was kept constant during each test. However, because of the presence of the sand slope in the tank and the change in sand volume at different elevations, the rate of rise in water level slightly changed as the water level rose. Figure 5 compares the changes in the elevation of the water level with time for the case in which the tank was empty, with that in which the slope was built in the tank. These data were measured from a test in which the gravel layer covered the full area of the tank base. As can be seen from this figure, due to the volume occupied by the soil, the rate of rise in water level with time slightly increased and became nonlinear when the sand slope was present in the tank.

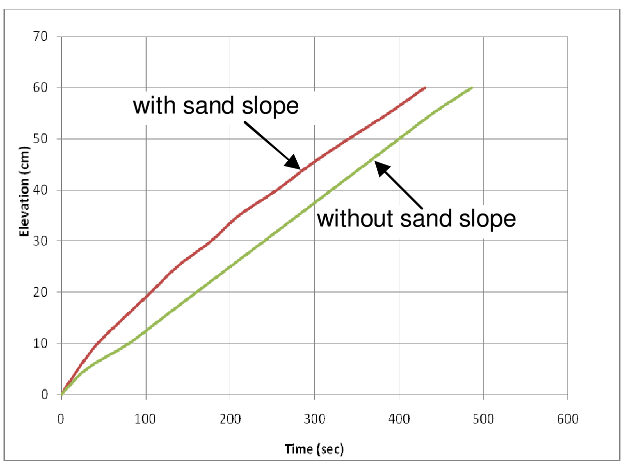

Figure 5. Changes in the water level elevation with time in the tank with and without the sand slope

5 DESCRIPTION OF THE TESTS

5.1 Test No. 1

Figure 6 shows a schematic view of the setup for Test No. 1. In this test, the 10 mm thick gravel layer covered the full area of the tank base and therefore, water level was raised uniformly in the areas inside and outside (in front of) the fill. The fill was placed at a 44 degree slope angle with the horizontal, and the average void ratio after placing the fill was estimated at 0.78 (Dr=28%). Total time for water to reach an elevation of 600 mm above the tank base was 9 minutes; therefore, the average rate of rise of water level was approximately 67 mm/min.

As the water level reached an elevation of 100 mm above the gravel layer, the turbidity of the water in front of the fill started to increase, and the slope of the underwater portion of the fill decreased (slope became milder). This continued while the water level was rising; and, since the slope of the portion of the fill below water level was decreasing, an overhang was being created in the portion of the fill above water level. In the meantime, crest of the fill was also settling. Settlement of the crest reached 10 mm when water level was at 200 mm above the gravel layer and 30 mm when the water level was at 410 mm above the gravel layer.

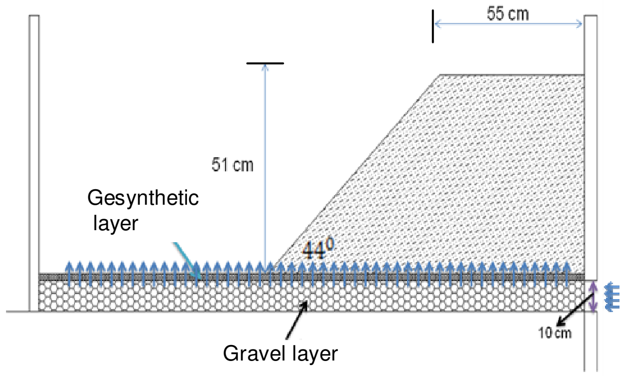

Figure 6. Geometry and slope for Test No. 1

As the water level was rising, the overhang length was increasing, and vertical cracks in the slope crest were appearing. When the water level reached 450 mm above the gravel layer, a toppling-type failure in the part of soil above the water level at the crest of the fill occurred, and it progressed from the front to the back of the crest until all the soil above the water level completely failed. The toppling-type failures were due to the creation of soil blocks between the cracks that were being formed at the slope crest.

Following failure of its crest, the slope was completely submerged and stabilized at an angle of 30.4 degree with the horizontal after failure. This slope angle is 3.6 degrees smaller than the angle of repose measured for this sand in its submerged condition.

In order to examine the effects of the rate of water level rise on the slope failure, at the end of the aforementioned first-stage test, and while the 30.4 degree failed slope was still in the tank, water was first drained from the downstream end of the tank and a second-stage test was initiated, in which the water level was raised again at a higher rate of 100 mm/min, which is about 50% more than that used in the first-stage test.

The higher rate of water level rise in the second-stage test resulted in a decrease in the slope angle of the soil below water level as it reached any elevation. When the water level finally reached 500 mm above the gravel layer, the whole slope was submerged and stabilized at a 23 degree angle to the horizontal. Figure 7 shows the location of the sand slope before raising the water level, and after the first and second stage tests.

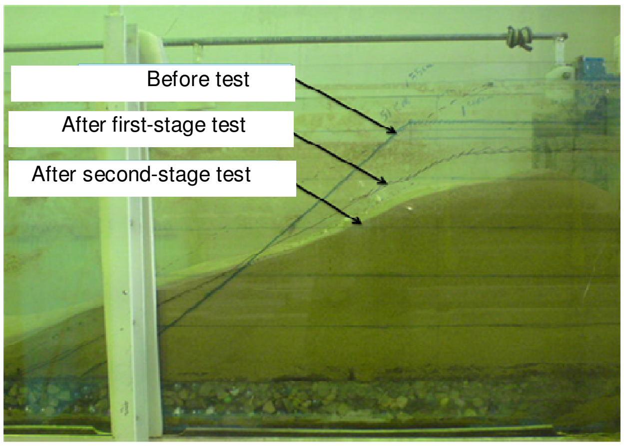

Figure 7 Locations of the slopes before the test and at the end of the first and second stages of Test No. 1

5.2 Test No. 2

Setup and slope dimensions for the second test are shown in Figure 3 discussed in Section 4. In this test, water was introduced into the sand slope from part of the tank base and, therefore, during raising the water level, it remained inside the slope and, as a result, the hydrostatic pressure that existed in front of the slope in test No. 1 was eliminated. In order to minimize the possible effects of seepage forces, the rate of rise in water level inside the fill was decreased to 50 mm/min, which is the lowest rate that could be achieved by the electronic pump speed regulator. Compaction energy used in placing the sand layers in this tests was kept to a minimum and, as a result, average void ratio of the soil placed in this test was back-calculated at 0.96 (Dr=-28%).

As the water level in the soil was rising, settlement in the crest of the fill was observed and gradually increased. In the fifth minute of the test, when water level rose to approximately 250 mm above the gravel layer, a sudden collapse of the fill occurred, and the slope decreased from 44 to 25 degrees with the horizontal. This collapse occurred when water level was still inside the slope and at a relatively large distance (approximately halfway) from its outer perimeters (Figure 8).

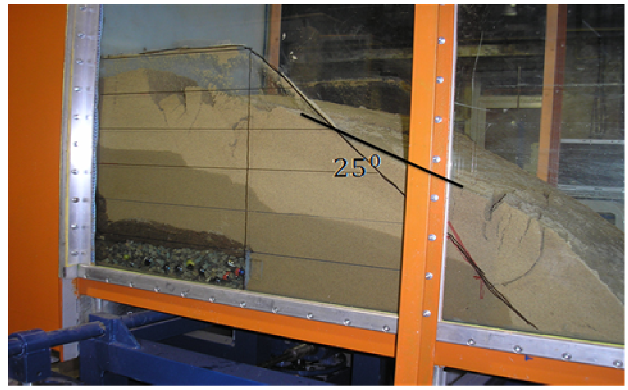

Figure 8. Slope after first collapse in Test No. 2

Two minutes later, when the water level had reached an elevation of 350 mm above the gravel layer (about 100 mm from the collapsed slope crest), a second collapse occurred, after which slope of the fill reached a 13 degree angle with the horizontal, and water overtopped the fill, leading to the submergence of the whole slope (Fig. 9)

Figure 9. Final slope at the end of Test No. 2

5.3 Test No. 3

In the third test, slope of the fill was chosen to be 34 degrees, which is equal to the angle of repose of the sand tested in its dry and submerged states. The length of the gravel layer was also increased accordingly to 700 mm such that during seepage of water from the gravel layer into the slope, the distances of the saturation front from the slope outer perimeters remain approximately the same. Setup and slope dimensions for this test are shown in Figure 10.

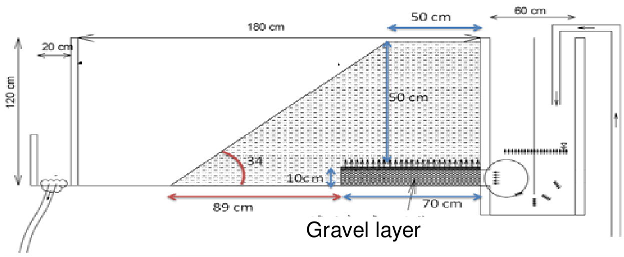

Figure 10. Geometry and slope for Test No. 3

The test procedures such as the rate of rise in water level, the method of placing, and the compaction energy used for constructing the fill, and the height and the crest length of the slope were the same as those used in Test No. 2. At the end of placing the fill, the average void ratio of the soil was back calculated at 0.94 (Dr=-22%).

During the first 6 minutes of introducing water into the slope, the crest of the slope experienced slight gradual settling as water level was rising inside the slope, but no failure occurred (Figure 11). However in the sixth minute, when water level within the slope had reached an average of approximately 300 mm above the gravel layer, a sudden collapse took place, during which the slope of the fill changed from 34 to an average 24 degrees with the horizontal. The collapse occurred when the water level was still inside the soil, and the water front was at an approximate distance of 200 mm from the crest, and an average of approximately 100 mm from the face of the slope.

Figure 11. Condition of slope prior to the sixth minute in Test No. 3

As the water level continued to rise, the water front was moving closer to the outer perimeters of the slope, and one minute later, when the water front was approximately 100 mm from the collapsed crest and at about the same distance from the face of the slope, another collapse occurred, after which the slope angle decreased to an approximately 15 degree angle with the horizontal. The slope then continued to flatten slowly, and about 15 seconds later, water emerged outside of the slope crest and overtopping of the slope followed, after which the slope was fully submerged and stabilized at an approximate angle of 10 degrees with the horizontal (Fig. 12).

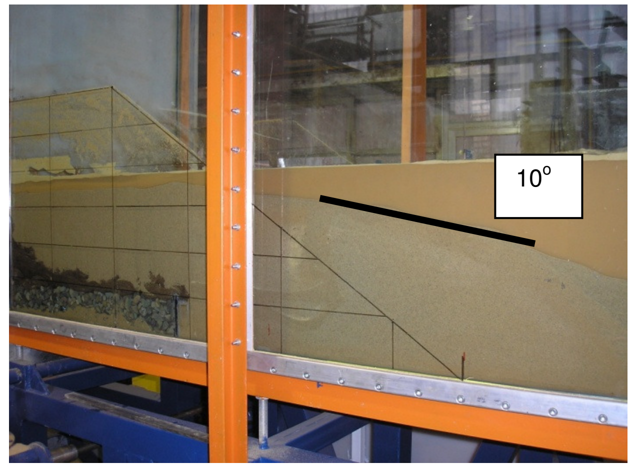

Figure 12. Final condition of the slope in Test No. 3. Outline of the slope initial condition is also shown.

6 DISCUSSION OF THE RESULTS

Based on the observations described for the first-stage of Test No. 1, such as the gradual change in slope of the fill as water level was rising, and the post-failure slope angle, which was slightly lower than the angle of repose of the submerged sand, it is likely that the slope failure in this test was the result of saturation and loss of strength of the unsaturated soil. The slight decrease in the slope of stabilization of the saturated soil compared to the soil angle of repose may have resulted from the relatively small seepage forces resulting from the relatively small rate of rise in water level.

The second stage of test No. 1 demonstrated the effects of the higher seepage forces applied to the soil due to the higher rate of rise in water level in this stage. The slope stabilized at 23 degrees with the horizontal, which is 7.4 degrees smaller than the angle of stabilization of the slope when subjected to the slower rate of rise in water level, and 11 degrees smaller than the underwater angle of repose of the sand.

In many ways, Tests No. 2 and No. 3 were different from Test No. 1. In Tests No. 2 and 3, the hydrostatic pressure in front of the slope was eliminated, and water seeped into the slope from an area that covered nearly half of the slope base. This resulted in the water front expanding such that it was at approximately similar distances from the slope outer perimeters (i.e. its crest and face). In these tests, failures occurred while the water front was still inside the slope, and the post-failure overtopping of the slopes, and surface erosion of its crest and face had relatively small effects on altering the final geometry of the failed slope. In tests No. 2 and 3, void ratios of the sand were also significantly higher than that for Test No. 1.

In Test No. 2, the first collapse occurred suddenly, and the slope angle after collapse was 25 degrees, 9 degrees smaller than the angle of repose of the sand. The sand was also very loose (Dr=-28%) and prone to contractions similar to those observed in the CDS loading described in Section 3. However, the first collapse in this test occurred while the slope was not yet fully submerged; therefore, the contracted portion of the slope soil did not include the full slope volume. The subsequent rises in water level and the ensuing failures led to milder slope angles such that the slope finally stabilized at 13 degrees after all the soil was submerged.

It is likely that more than one mechanism may have been responsible for the slope failures observed in Test No. 2. A more likely mechanism is the contraction of the very loose sand as a result of the decrease in mean effective normal stress, similar to what is observed in the CDS tests on this sand. Other possible mechanisms include the effects of soil saturation and the sliding along the tank base. It is noted, however, that full saturation of the soil during the rise in water level could not be verified and, it is likely that it was not achieved. However, the contractions might have been large enough to cause failure even in the nearly saturated soil.

In test No. 2, since the initial slope angle was 44 degrees, which is higher than the angle of repose of the dry sand, loss of strength due to saturation may also have played a role in causing instability.

However, in Test No. 3, the slope angle of the fill was taken to be equal to the angle of repose, decreasing the possibility that the loss of strength due to saturation could drive instability. In this test, the relative density of the soil and the instability that occurred were similar to those of Test No. 2. Therefore, there is a greater possibility that the slope failure in this test has been a result of the contractions of the very loose sand similar to those observed in the CDS loading tests. Eckersley (1990) observed similar failures in slopes made of very loose coal mining tailings materials in his laboratory model tests.

Although the ranges of stresses used in the presumably “single element” CDS tests described in Section 3 are much larger than those applied to the soil in the “multi element” model tests in the tank, it is curious to note the approximately similar mobilized friction angle at the onset of volume contraction in the CDS test (22.4 degrees) to the angle of stabilization of the slope after collapse (24 degrees) in the model test. If, in a drained CDS test on saturated sand, the sample cannot drain fast enough during the volume contractions, and fails under a nearly undrained condition when it reaches the mobilized friction angle at the onset of the large volume contractions, considering the geometry of the undrained effective stress path of loose sands, the sand shear strength is expected to be consistent with a friction angle near this mobilized value during failure. A slope made of such saturated sand is therefore expected to stabilize at an angle close to, or slightly higher than this mobilized friction angle, due to some drainage that is expected to occur during the slope failure.

7 SUMMARY AND CONCLUSIONS

Failures of slopes made of loose granular soils due to the rise in water level may be attributed to one or a combination of different mechanisms. One mechanism is the decrease in the soil shear strength due to saturation. This may occur due to the loss of suction in the unsaturated soil, and results in instability at the current slope angle, leading to failure and subsequent stabilization at a milder slope angle. Another cause may be related to the tendency of very loose granular soils for contraction, once such soils are subjected to loading with constant shear (deviatoric) stress, but with decreasing mean effective normal stress. Previous laboratory tests have shown that under such conditions, substantial volume contractions may initiate at stress states corresponding to mobilized friction angles well below failure. These volume contractions in drained loading can lead to the development of excess pore pressures in undrained or semi-drained loading, if drainage path is long enough or soil permeability is low enough, resulting in the loss of the soil shear strength and the failure of the slope.

Another mechanism can occur in cases where the rate of rise in water level and the resulting upward seepage forces are high enough to cause slope failure.

In the current paper, an experimental program for testing of slope failures resulting from the rise in water level is described. Tests carried out on slopes made of a local, very loose, fine grained sand in a tank built for this purpose are explained. The sand comprising the slopes was placed using a procedure similar to that used in preparing moist-tamped triaxial samples. Model test results showed that such slopes may fail due to the rise in water level, and stabilize at slope angles that can be substantially lower than those at which the original slope was made. It was noticed that the final slope angle at which the sand stabilizes depends on the mechanism controlling failure. Failures due to saturation result in milder slope angles and failures due to the volumetric collapse of the very loose sand lead to milder slope angles after failure. The rate of rise in water level also affects the final slope angle even in cases in which saturation is the main cause of the slope failure.

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