Centrifuge Model Test and Numerical Analysis on Failure Behavior of a Loess – Paleosol Landslide Caused by Excavation

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Research Article

Keywords: Loess – paleosol slope, Excavation, Centrifugal model test, Numerical simulation, Deformation process

Posted Date: August 3rd, 2021

DOI: <https://doi.org/10.21203/rs.3.rs-669780/v1>

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Abstract

Many landslides are induced by excavation activities in the loess region. In this article, a loess – paleosol slope model was built and tested under 80 g centrifugal environment. Three certain angle excavations were simulated by manipulator movement. The mini pressure sensor and PIV system were utilized to monitor experimental process respectively. It can be found that the slope from excavation to failure, is liable to form the deep and shallow two sliding surfaces. The distance perpendicular to slope surface was measured as 9.6 cm for the deeper sliding surface, and 4.2 cm for the shallower one. Both of sliding surfaces are caused by the interaction of tensile failure and shear failure, specifically presented as the tensile failure concentrating on the upper part and the shear failure on the lower part. The loess slope can be split into three zones by response of excavation unloading (i.e., the sliding zone, the influenced zone and the uninfluenced zone). The failure pattern belongs to a retrogressive type with the bulging front edge and tension cracking trailing edge. The causes of the fractures on the slope top can be divided into different sections. The fracture near the slope top is induced by tension and shear force. But the fracture away from slope top is only induced by tension. In addition, the plastic zone development distribution of simulation has a good consistency with the centrifugal model deformation zoning diagram. These results can provide guidance for excavation activities in loess – paleosol slopes.

1. Introduction

In the loess region, many loess slopes have to be constructed by the projects for economic development. Many loess landslide disasters are caused by these excavation activities (ie., mining excavation). After landslide occurrence, engineering measures are used to control and monitor the landslide. So It has formed the disaster chain of engineering to disaster to engineering. Beside the geological environment evolution is accelerated after the project construction. It aggravates the occurrence of loess geological disasters and forms the chain of engineering to geological environment deterioration to disaster to engineering. For example, on March 12, 2010, a landslide occurred in Zizhou County, Yulin City, Shaanxi Province. 44 people were buried and 27 died when the slope slid due to the engineering excavation. So Landslides due to excavation cause huge losses. It is very important to evaluate the safety and understand the mechanisms of such landslides.

At present, the limit equilibrium method is usually used to study the excavated slope stability (Potts et al. 1997, Kim et al. 2009, Alison et al. 2020). For example, the limit equilibrium method is used to evaluate the stability excavated slope in engineering construction process (Wei et al. 2012, Yahs et al. 2018). Also, a simplified limit equilibrium analysis is used to correctly predict the instability of excavation before reaching the expected depth, and the effectiveness of the method is verified (E. Botero et al. 2020). In addition, the safety coefficient of the limit equilibrium method in the 3D model is higher than that in the 2D model (Taha et al. 2015). Some scholars have also applied the limit equilibrium method to the stability analysis of loess landslide and obtained that the failure of loess slope occurs due to collapse or excavation (Junjie et al. 2014).

Except that, the laboratory test methods are also studied by many scholars. The methods are usually according to a certain prototype by using a certain scale to scale and carry out excavation physical model tests under the dead weight condition (Mohammad et al. 2011, Techawongsakorn et al. 2015). In addition, there are other factors coupled under conditions of excavation to be studied, for example, rainfalls (Cheng et al. 2014), earthquakes (Ni et al. 2012), supporting methods (Sharma et al. 2019).

Due to the different stress levels between the model and the prototype, some small-scale models cannot fully simulate the excavated soil behavior (Takemura et al. 1999). Therefore, centrifugal model tests can overcome this difficulty by increasing the acceleration field. The test method can reproduce high stress levels in a relatively small model, corresponding to the gravity-induced stress field in the prototype (Schofield 1980). For example, the failure of the slope was predicted by the centrifugal model test (K et al. 2009). The method was used to simulate the excavation slope with high centrifugal acceleration under different inclinations and heights, and the characteristics of the slope strain and slip surface was analyzed (Li, M et al. 2011). At present, the centrifugal tests researching on the excavation loess slopes are less, and these tests are aiming at the stability analysis of the slopes (Xie et al. 2009, Lederer, C et al. 2009). In addition, they mainly focused on the failure of excavation of homogeneous loess slopes (Fan et al. 2016, Zhang et al. 2020). The loess – paleosol slope models in tests are rarely studied.

Besides that, Numerical simulations are also used to study the landslide. Simulation software is developed for physical and mechanical state of slope materials. Finite element model (Summersgill et al. 2018; Zhang et al. 2019), discrete element model (Zhang et al. 2013; Meng et al. 2018) and finite difference model (Guo et al. 2013; Yanhui et al. 2018) are all implemented. Zhu et al. (2019) used the stochastic variable method to establish a finite element model to study layered excavation. Zhen et al. (2020) established a numerical model using PFC2D for the development of cracks and strains during excavation. Fa et al. (2011) analyzed the influence of excavation on the overall stability by using a finite element model for the Shuicaofao landslide. He demonstrated that the excavation causes obvious local failure on the slope, which increases local vertical stress and plastic strain near the excavation surface.

In this paper, the ancient soil layer was added to the loess – paleosol slope model to simulate the actual slope. The failure morphology characteristics and development law of stress and strain of soil in the slope are analyzed. The formation process of sliding surface and the fractures are studied to divide the slope zones. Meanwhile, the numerical simulation method was used for the validation and further analysis.

2. Centrifuge Test Method

2.1 Centrifugal testing machine

To understand the failure process and mechanism of loess-paleosol slope, the centrifugal test was carried out using a TLJ-500 geotechnical centrifugal testing machine as shown in Figure 1. The machine used includes three parts, a power part, a centrifugal part and a control part. The power part contains 900 kW motor, reducer, all-digital speed regulating device, acceleration control and measurement system. The

centrifugal part contains centrifuge host, model box, and manipulator system. The control part contains main console, collecting ring, measuring system, TV monitoring and communication system.

2.2 Slope model design

Different industry standards have different design requirements for the loess cutting slope. According to the Specifications for Design of Highway Subgrades (JTG D30-2015, a national criterion for highway construction project in China) for the Q_3 loess slope with elevation less than 12 m, the slope rate should be controlled between 1:0.3 and 1:0.75. And according to the Chinese code for design on subgrade of railway (TB 1001-2005), the slope rate of cutting slope ranges from 1:1 to 1:1.5 if the height of clay or silty clay slope is less than 20 m. Moreover, as per in the Chinese code for design on special subgrade of railway (TB 10035-2006), for the loess slopes in loess plateau and western Henan Province, the slope rate is 1:0.5-1:0.75 when slope height is less than 10 m, and 1:1-1:1.25 when that ranges from 10 to 20 m. However, for the loess slope belongs to Q_3^{eol} , Q_3^{al} and Q_3^{pl} , the above requirements for slope rate has changed for achieving higher safety factor, that is 1:0.5 for the former, and 1:0.5-1 for the latter.

The medium model box has the dimensions of 1200 mm in length, 1000 mm in width, and 800 mm in depth. Due to the limited operating range of manipulator, the vertical operating range is limited within the length of 600 mm and width of 450 mm (Figure 2a), while in the horizontal direction, the operating range is the rectangle area in the center of model box (i.e., length 600 mm x width 300 mm) as shown in Figure 2(d). The centrifugal model was therefore placed in the middle position of model box with a controlled width and height as 300 mm and 450 mm respectively. The manipulator controls excavation through the hydraulic device which can yield tension (pressure) force up to 14 kN (Figure 2e). By controlling the length and width of a single step, the slope was excavated and shaped into steps with different slope rate so as to ensure that the overall slope satisfies our predetermination.

Considering the boundedness of centrifuge and experimental operability, the centrifugal model tests in the present study focuses on the influence of excavation on the typical loess-paleosol slope, and further, can be used to provide a non-prototype theoretical analysis model. The natural slope in loess region is mainly distributed in the interval of less than 50° . Considering the range of the manipulator operation, the model slope ratio is 1:1, the width 300. the height of prototype (H_p) is 32 m, and geometric similarity ratio (n) is 80. So the model height (H_m) is 40 cm, and the slope of the model is 45° . The model was placed in the middle of the model box. The boundary of both sides was controlled by 5 cm thick plexiglass, and supported by wooden frames, as shown in Figure 2.

Model test box design, (c) Front view Centrifugal model in the cableway bucket, (d) Manipulator operation range, (e) Size of the remake blade of the manipulator, (f) Plan view Centrifugal model in the cableway bucket.

Due to the Specification for highway subgrade design (JTG D30-2015), the rate of loess cut slope within 0 to 12 m height slope is from 1:0.3 to 1:0.75. Because the height of excavation zone is 12 m, and the

excavation height (h_{em}) in the model is 15 cm. The excavation ratio is 1:0.75 and 1:0.4. In order to magnify the influence of excavation on slope stability, a vertical excavation scheme is proposed as listed in Table 1.

Table 1 Excavation stage design

Excavation stage	Excavation length /cm	Slope rate	Excavation height /cm
E0	0	1:1	0
E1	3.75	1:0.75	15
E2	9	1:0.4	15
E3	15	Vertical	15

To realize the slope excavation operation, the manipulator with a blade was operated to realize the excavation for a certain angle without stopping the centrifugal machine.

2.3 Measurement system plan

1. Soil pressure measurement

Total 11 mini sensors were installed to monitor the soil pressure change during the test. According to the simulation result of pressure distribution, the direction of soil pressure could be different for different parts of excavated body. The change of soil pressure is mainly in vertical direction at the lower part of excavated body, while that is mainly in horizontal direction at the back part. The placement method of sensors is adjusted accordingly. The preset positions and specific coordinates for each sensor are listed in Table 2.

Table 2 Locations of earth pressure cells

Code	Stress direction	X /mm	Y/ mm	Z /mm
T1	vertical	750	150	300
T2	vertical	650	150	300
T3	horizontal	500	150	550
T4	horizontal	500	150	450
T5	vertical	500	150	350
T6	vertical	370	150	650
T7	horizontal	370	150	550
T8	horizontal	370	150	450
T9	vertical	370	150	350
T10	vertical	240	150	650
T11	vertical	240	150	550

(2) Deformation measurement

Basic principle of deformation monitoring system

Particle Image Velocimetry (PIV) system divides the images into large amounts of grid windows, and uses the cross-correlation algorithm. The matching area is searched in the two frames of images. And the displacement and direction of the fishing area are matched. It represents the average displacement and direction of the particles in this area. The schematic diagram of PIV principle is shown in the Figure 4.

Displacement marker

The test used the placing the marking points method. According to the particle imaging velocimetry system, the information of displacement variation for marking points was captured. The self-correlation algorithm was used to match the images and to calculate the change of slope position field. Before building the slope model, some landmarks were pre-buried. These marking points are made by the small white golf support, and buried by layers to form a grid (Figure 3).

To sum up, the model size, excavation scheme, layout and measurement system designed above are adopted in the test. The schematic diagram of model preparation is shown in Figure 4. After the slope model preparation, the manipulator and the excavation blade were installed and hoisted into the centrifuge to carry out the test. Before the beginning, the excavation coordinates were calculated to move the blade. There were a moving camera and a digital to observe the experiment process.

2.4 Model materials and loading procedure

The soils used in the experiment are from the Q₂ Loess and the third paleosol layer in the south tableland of Jinghe river, located in Jingyang County, Xi'an. The physical properties are shown in Table 3. Sieve analysis coupled with hydrometer testing was conducted to determine the grain size distribution curve of loess and paleosol according to ASTM D2487, and the results are shown in Figure

Table 3 Physical properties of loess and paleosol

Property	Density g/cm ³	Dry density g/cm ³	Moisture content / %	Liquid limit/%	Plastic limit / %
Loess	1.65	1.42	18.0	33.2	20.1
Paleosol	1.90	1.62	20.0	27.9	12.5

The loess and paleosol are oven-dried, smashed and sieved through 2 mm in the laboratory. Before operating the experiment, we used the drying method to measure the moisture content. The water content is set by the natural state. According to the slope size, ultra-wide filling is carried out in the multiple layers. The amount of soils (m_s) required for each layer is dependent on the actual natural density and water content, which can be calculated as:

$$m_s = \frac{\rho}{1+\omega} w \times l \times h \quad (1)$$

where ρ (kg/m³) is the nature density of the soil, ω (%) is the water content, w , l , and h (m) respectively are the width, length and height of each layer.

Because the similitude ratio is 80, the centrifugal model should be in the state of 80 g for consolidation and excavation operation. As the largest centrifugal acceleration is 100 g, the whole acceleration process will last about 12 minutes. The centrifugal loading process is presented in Figure 6.

3. Results

3.1 Slope failure morphology

Constant centrifugal force, about 80g, was applied on the slope model until failure occurred. The excavation process was simulated in three steps in the case of no downtime. Figure 7 shows the whole process from deformation to failure that the slope model experienced after excavation. In general, it could be categorized into five stages. In the first stage, when excavation operation was finished, stress redistribution occurred inside the slope due to the unloading effect induced by excavation (Figure 7a). This led to the second stage, in which tensile fracture was observed on slope surface with further development of soil deformation, and continuous plastic zone was coming inside (Figure 7b). As the plastic zone and tensile fracture developed to interpenetration, the first slip occurred at the depth of the model, indicating the third stage of deformation process. According to the mechanical characteristics, the

sliding was a retrogressive failure. The back wall showed in an arm-chair shape and the sliding distance of the landslide was short. The sliding body accumulated at slope foot, and the sliding started quickly. During the fourth stage, due to the instability of deep-sliding body, the plastic zone and fracture had a secondary development in the shallow part (Figure 7d). As a result, the second slip occurred in the fifth stage, which exhibits as a shallow sliding. According to the mechanical characteristics, similar to the first one, it also belongs to the retrogressive sliding, which presented an arm-chair shape at back wall. However, shorter sliding distance was observed for the second landslide, as well as faster response.

For a better understanding, the shape and location of the sliding surfaces formed during the overall instability process are shown in Figure 8. As mentioned above, two sliding surfaces were formed in the slope body, where one was deeper and the other was shallower. The distance perpendicular to slope surface was measured as 9.6 cm for the deeper sliding surface, and 4.2 cm for the shallower one. In addition, the deeper sliding surface has a higher inclination angle. Thus, compared to the first collapse that is an overall deformation triggered by the unloading effect, the second one was observed to be more local, resulting in deformation with smaller scale.

There were two fissures with vertical extension direction at the top of slope (Figure 8b). Due to the limited influence range of excavation zone, the closer the fissure is to the slope crest, the longer extension it has. After the first sliding, the slope back wall appears a new lateral face, leading to a small crack close to the back wall by the action of lateral force, which generates a small collapse later.

3.2 Stress field behavior

The variation of soil pressure with time is illustrated in Figure 9, which has obvious multistage characteristics. The developing process of soil pressure can be divided into: the acceleration and consolidation stage (0-1105 s), the first-step excavation stage (1105-2080 s), the second-step excavation stage (2290-3250 s), the third-step excavation stage (3504-4290 s), and the failure stage (4290-5156 s). Prior to excavation operation, it took 12 minutes for acceleration to reach a gravitational acceleration of 80 g, and 5 minutes for consolidation in such a condition. The value of soil pressure monotonically increased, and tended to be stable after consolidation.

During the first-step excavation stage, the sensor readings from T1 and T2 had an obvious variation, mostly because of the excavation starting from slope foot. The soil pressures at T3 and T7 showed a slow decreasing trend with time, while the rest had no significant change. As observed, the soil pressure at T1 decreased rapidly during excavation due to the reduction of upper loads. The soil pressure at T2, meanwhile, increased firstly and then decreased as a result of stress concentration phenomenon induced by the increase of local slope grade.

During the second-step excavation stage, the responses of T1, T2, T3 and T7 sensors were great. As the excavated soils cumulated above the T1 sensor, the measured soil pressure accordingly showed a slow rising trend. The soil pressure at T2 showed a trend of slow decreasing followed by a sharp decrease. This is attributed to the triangular soil distribution with thin top and thick bottom. The excavation at slope

foot has little effect on the pressure distribution until it proceeds to be right above the T2 sensor, which causes a sharp decrease in soil pressure. Moreover, the excavation also affects the horizontal soil pressure since the soil pressures at T3 and T7 both gradually decreased with the excavation operation going on. In the condition of excavation, the confining pressure at T3 and T7 was greatly affected by the unloading effect, while that at T4 and T8 was not. This difference suggests that the pressure redistribution has very limited effect on the position of two sensors.

During the third-step excavation stage, the soil pressures at T2, T3, and T7 were mostly affected. The soil pressure at T2 continued to lower values and the decrease was more significant. It is because the T2 sensor was below the excavation area in the last excavation step. However, the soil pressures at T3 and T7 also decreased, indicating that the effect of unloading spread to the plane at this depth. Considering that these two sensors were installed inside the slope body, the decrease in soil pressure represents an aggravation of global deformation in this period.

The whole process of excavation was finished at 4190 s, and then slope failure took place at 4290 s. The soil pressure at T3 sharply decreased by half. Combined with our laboratory experiment, the results from unloading stress path tests showed that, the soil sample under the same confining pressure failed when the pressure was unloaded to about 0.45-0.56 of the initial. It agrees with the variation of soil pressure at T3. In this case, the soil located around the T3 sensor has broken, which means that the formed sliding surface is across or near this position. At the same time, a steady increase in soil pressure at T2 was observed, because the internal shear zone of the soil forms penetration and the first collapse which leads to the accumulation of the sliding soil around the T2 sensor. The soil pressure at T3 and T7 also decreased during the process of soil breaking and sliding. The second sliding, generated from the first sliding mass, occurred around 4755 s. Therefore, the soil pressures at T1 and T2 further changed, while the rest only had a weak change.

The whole process, from the completion of excavation at 4190 s to the occurrence of overall failure at 5156 s, is a gradual failure process. Not only does the soil pressure at each position have different changes, but also the soil deformation experiences different stages. The evolution of deformation field will be analyzed in detail in the following section.

3.3 Displacement field analysis

3.3.1 Analysis of displacement vectors

As mentioned above, the PIV particle imaging observation system was used to extract the displacement image of the slope throughout the test. By comparing the displacement variation, the displacement vector diagram at each excavation stage was obtained (Figure 10). Before excavation, the deformation of slope body is mainly presented as vertical displacement. With the proceeding of slope excavation, it is observed that the variation in slope position vector gets larger and the affected range of displacement vector increases accordingly. For these three excavation stages, the position where the magnitude and direction of displacement vector has the most significant changes is slope foot, indicating that the unloading

effect mainly induces the displacement increment of slope body in horizontal direction. Meanwhile, the influence of excavation unloading on the horizontal displacement vector component becomes less from the slope surface to the inner. The direction of displacement vectors in the slope is observed to be more vertical and the magnitude of them shows a decreasing trend from the top to the slope base. This leads to the fact that the distribution of displacement vector shows an obvious arc-shape.

3.3.2 Analysis of the slope deformation

Figure 11 illustrates the isoline map of displacement obtained at different time point after excavation. In this figure, H-disp and V-disp represent the horizontal and vertical displacement, respectively. The cumulative value corresponding to each excavation step is shown.

As observed, both H-disp and V-disp varied with the deepening of excavation. The distribution characteristics of the isoline map experienced three stages. In the first excavation stage, the isolines are roughly parallel after the completion of excavation (Figure 11a and b), indicating that the displacement is linearly distributed without obvious local deformation. Thus, the linear deformation stage can be regarded for this period. With the excavation step increasing, the deformation value of slope induced by unloading gradually increased. When developing to the second excavation, it can be observed that the displacement isolines is no longer nearly parallel but close to the slope surface (Figure 11c and d). In the third excavation stage, the slope is at the stage of strain localization. The density of the displacement isolines near the slope surface increases, and gradually decreases into the inner of slope body, presenting a local regional characteristic (Figure 11e and f). Moreover, there are two local isolines concentrated at the top of the slope, which is exactly the location of the cracks that occurred. Both in the horizontal and vertical displacement isoline maps, the local concentration characteristics of the cracks exist near the slope surface, indicating that their formation and propagation are the result of both tension and shear. While those far from slope surface is only shown in the horizontal isoline map, which indicates that the cracks are mainly caused by tension.

3.3.3 Analysis of the slope deformation zoning

To determine the final influenced area after the three steps of excavation, a series of typical timing was selected for analysis, including after the first excavation (2290 s), after the second excavation (3504 s), after the third excavation (4335 s), and after the sliding (5140 s). The results of displacement distribution at the height of 9.49 cm, 15.0 cm, 23.4 cm and 32.1 cm are plotted according to the time point, as shown in Figure 12.

It can be found in Figure 12 that there is always a turning point in the displacement curve. The closer to the slope surface, the greater the displacement is. With the increase of excavation depth and scale, the position of the turning point is constantly approaching the slope body inside due to the uneven deformation of slope body, which means that the influence of excavation unloading extends to the inner. Thus, a boundary where the influence of excavation may reach can be defined according to the connecting line of the turning points at each height, which divides the whole area into two parts: influence

zone for the left (Ⅱ part) and uninfluenced zone for the right (Ⅲ part). The boundary line is generated by the deformation of excavation effect. The boundary caused by deformation at this point is superimposed on the slope position grid graph. The influenced zone (Ⅱ part) during the localization deformation stage, can be further split into three parts. The part i is made up of a flat part and it exists the maximum displacement, but has a relative low gradient. And the part iii consists of the area with a moderate gradient and close to the surface of influence boundary. So the final area concentrates on the area between part i and part ii, which presents a largest displacement gradient. Apart from part i, distributing on the sliding area, all displacement curves exhibit a similar characteristic on different levels. Due to the higher gradient in part ii, the localized deformation concentration is observed, referring to horizontal displacement. On the basis of each turning points, the influence boundaries are lined and presented in Figure 10. The sliding zone experience two parts, which can be observed that the lower zone is in the part ii and the upper part is in the part i, which shows that the formation mechanisms of sliding surface are in different. And further analysis of the different was conducted in the failure process.

In view of the deformation and the stress field variation, further subdivision can be made, as shown in Figure 13(b). The sliding zone is in the leading edge of the influence zone, that is from the sliding surface to the slope body. Within this range, a global sliding failure occurs, which is mainly characterized by horizontal displacement, followed by vertical displacement. The influence zone covers from the sliding surface to the influence boundary, where large soil deformation is observed but no significant sliding failure in this range. However, some cracks are distributed on the slope surface due to the unloading effect and larger deformation. Under the action of unloading, the slope produces different degree of horizontal displacement increment. The soil structure remains in good condition in the influenced zone and hence there is no large deformation and cracks to be found.

3.3.4 Analysis of evolution process of slope body

Based on the morphological characteristics of centrifugal model after failure, each characteristic point is measured to analyze the development process. As shown in Figure 14, the dotted and solid arrows indicate tensile failure and shear failure respectively. Both of sliding surfaces are the result of the interaction of tensile failure and shear failure, specifically presented as the tensile failure concentrated on the upper part and the shear failure concentrated on the lower part. After excavation, the first failure (i.e., deeper sliding) results in stress concentration at the slope foot and shear failure of soil. Under the lateral plane condition, the shear action of the soil at slope foot leads to the tensile action of the soil in the upper layer of the sliding surface, resulting in the observed tensile failure. For the deeper sliding, it takes a total of 4.5 s from formation to penetration; by comparison, the upper sliding has been in the plastic failure state in the early stage, in which case its penetration is relatively fast (in 1.0 s). Besides, two fissures at the slope top are different in location, extension depth, opening degree and cause of formation. The left one is formed by a long time of period, with deep extension and wide opening because of the combined action of tension and shear; while the right generated by the tensile action has a short forming time, showing short extension depth and small opening.

To further analyze the evolution process of slope deformation and the formation of shear zone, 3 pairs of corresponding measuring points (i.e., located on both sides of the sliding surface) are chosen. The change of horizontal and vertical displacement between the two measuring points with time is shown in Figure 15. As observed, each curve experiences a critical point, after which the horizontal and vertical displacement between the two measuring points gradually increases. Considering the increase of relative displacement, the critical point can be regarded as the beginning of shear failure. By comparing the time corresponding to critical point, it is found that the critical point on the I and II curves appears the earliest, around 37s, followed by III and IV curves, and V and VI curves. This explains that the development of shear zone is from the bottom to the top.

The evolution process of slope deformation can be summarized as following. The stress field of slope changes after excavation, resulting in the shear failure at the slope foot and the tensile failure at the slope top due to the stress redistribution. The overall deformation rate is relatively uniform in this period (between 0 s to 31 s) as the horizontal and vertical displacement between the two measuring points keep a uniform growth trend. With the further development of slope deformation, new tensile cracks appear at the slope top because the shear zone extends here, meanwhile, the existing ones also develop to the interior of slope body. When the shear zone meets the tensile crack near the front of the excavation face, the slope deformation increases sharply. In this case, the landslide forms and the crack at the slope top expands rapidly under the influence of overall deformation.

3.3.5 Analysis of the cause of the cracks at slope top

The crack type is analyzed based on the point-point displacement on both sides of crack. For tensile crack, the points displacement changes greatly in perpendicular direction, resulting in obvious variation in horizon and few in vertical displacement, which is opposite to the shear crack. In Figure 16, two main cracks are distributed on the slope top and the displacement changes at different positions of the crack. The first crack close to the slope side is 1 cm between the two points, and the second crack is 0.8 cm.

With respect to the first crack, as the excavation proceeds, the variation of horizontal displacement is much larger than that of vertical displacement (Figure 16a, b and c). It indicates that the crack is generated by tensile action at the beginning. However, further excavation leads to the fact that the crack is located in the influence zone and in the plastic zone. Shear stress therefore acts on the soil around crack due to the movement of sliding body, and the vertical displacement increases as a result. At the same time, a higher shear zone was formed on the side slope's arm-chair back wall. When the shear band is interpenetration, another slide will be formed. It shows that the loess landslide is a kind of backward landslide development process. The formation and propagation of the first crack are caused by the joint action of tension and shear. With respect to the second crack, the horizontal displacement between the two measuring points is significantly higher than the vertical displacement (Figure 16d and e). It can be inferred that the formation of crack is ascribed from the tensile stress caused by the deformation of soil mass in the middle and upper part of slope. Thus, the second crack can be classified as tensile crack.

3.3.6 Failure morphology analysis of slope.

To analyze the displacement field distribution characteristic after failure, three cross sections with respect to different height were selected, as shown in Figure 17. It can be observed that all displacement curves have a very similar developing trend. The displacement curves tend to be straight when far away from the slope surface and the displacement values of all curves are close to 0 cm. The horizontal and vertical deformation in the lower section is generally greater than that in the upper section, because the large uneven deformation is mainly concentrated in the middle of the slope body. Several typical points can be found based on the variation of curve slope. The curve at the point ① left shows a small variation of the displacement, but the right shows a large variation, which means that the point is located on the sliding surface. In addition, the curve at the point ① right keeps a little change and the horizontal displacement is about 0.3 cm, but the left not. The curve at point ② has two turning points, indicating that the slope at both sides of the turning point has a large displacement change. In the actual test results, the soil slides back and forth twice. In the slope body, the large uneven horizontal deformation is most concentrated in the slope body middle part.

4. Discussion

4.1 Numerical modeling method

For further analyzing the test results, numerical modeling based on FLAC3D 6.0 was used to simulate the centrifugal model under the multi-stage excavation condition. Figure 18 shows the numerical slope model with the same scale as the physical model, and there are 10,376 units in this numerical model. The model is stimulated under the semi-infinite boundary condition of limited slope perimeter. The excavation zone is established and a suitable model grid is generated. Table 4 summarized the basic properties of the soil used in the numerical model.

Table 4 Material properties used in numerical model

Layer	$\rho/(\text{kg}\cdot\text{m}^3)$	K/(MPa)	G/(MPa)	σ_t/Pa	c/(kPa)	$\phi/(^{\circ})$
Loess	1.65	20.8	9.6	10	37	22
Paleosol	1.90	45	16.9	12	59	28

4.2 Numerical simulations results

4.2.1 Displacement isoline maps

Figure 19 shows the numerical results of the horizontal and vertical isoline maps in each excavation step. After three times of excavation, the maximum displacement in the x and z direction increase from 0.34 mm to 6.5 mm and decrease from -0.24 mm to -4.0 mm, respectively. The numerical results shows a good consistency with the results from centrifugal test with respect to the displacement isoline map in the x direction. And this numerical simulation may also have good implementation in the prediction of

soil unloading and rebound induced by multi-stage excavation. The reason is that the shear failure caused by unloading is more similar to that caused by reduced pressure.

4.2.2 Slope displacement curve

Comparison between the results of numerical simulation and experimental test shows reasonable agreement in the displacement curves of slope surface. For the first two excavations, slope displacement shows fast response to unloading and then achieves a stable state after a period of time. However, at the third excavation, there exist continuously growing displacement curves with similar increasing trend. It should be noted that the soil in different areas of experimental model has different stress states and different stiffness moduli (i.e., volume modulus K and shear modulus G). Instead, each soil layer is assumed to have the same stiffness modulus value in the numerical model. Therefore, the results of numerical simulation and centrifugal test have some differences.

4.2.3 Slope strain development

The development of plastic zone at different calculation steps is shown in Figure 21, which is highlighted in red color. The numerical model has experienced 6 developing stages of plastic zone, that is unloading rebound stage, shallow plastic development stage, shallow plastic deformation stage, deep plastic development stage, deep plastic deformation stage and deep sliding failure stage. The first stage is characterized by unloading rebound, in which the first plastic zone is located on the side of the excavated body, and the unloading rebound mainly appears in the middle of excavated body (Figure 21a). In the shallow plastic development stage, the influence of lateral deformation develops (Figure 21b). The shallow part under the excavation body has plastic deformation due to unloading. And the plastic zone develops to the upper part of slope due to the effect of tensile stress. During the shallow plastic deformation stage (Figure 21c), the shallow plastic deformation continues to develop, and a large plastic shear failure zone appears inside the slope body. Plastic deformation occurs in the shallow part, for example, cracks on the slope surface layer. Besides, a larger plastic zone appears in the deep part. During the deep plastic development stage (Figure 21d), a larger plastic failure zone appears along the slope foot, which develops towards the slope top at a certain angle. The range of plastic failure in shallow part no longer increases, but continues to enlarge at the top of the slope. During the deep plastic deformation stage (Figure 21e), due to the continuous development of plastic zone at the deep and top of the slope, large deep plastic deformation and deeper cracks occur at the top of the slope. In the deep sliding failure stage (Figure 21f), a sliding surface forms after the penetration of the deep plastic zone along which great sliding failure take place. The distribution of plastic zone in numerical model agrees with the deformation zoning results in experimental test (i.e., matched with the influenced zone).

5. Conclusions

Based on centrifugal results, the stress and deformation fields of slope body under three excavations were analyzed by centrifugal experiment, and the test result was verified further through numerical simulation. The main conclusions are as follows:

(1) After the loess - paleosol slope excavation. the soil at the side and the bottom of the slope body is mainly affected, which is mainly manifested as the stress state change of the soil at the side and the unloading rebound of the soil at the bottom. And in the centrifugal model test, the mainly affected pressure cells are T1, T2, T3, and T7. Because of the limited influence of the excavation body, the stress station of the slope body shows partial changes in the process of the excavation steps.

(2) Based on deformation characteristics of loess-paleosol slope, the slope is under various degrees of deformation in the first and second excavation. And geological body generates sliding failure in the third. According to the research results, the slope state can be divided into four stages: linear deformation stage, the strain incremental stage, the stage of strain localization, and the final failure stage

(3) By sorting out the displacement field changes of the slope body, the deformation field characteristics show that the curved turning points of different height positions are connected to get a boundary line. So the line is the demarcation line of the influenced zone and the uninfluenced zone. And the whole slope deformation can be divided into the excavation body area, the sliding zone, the influence zone and the uninfluenced area.

(4) Two sliding surfaces are generated in the centrifugal. The first and second sliding surfaces are both generated by tension and shear action. The rear edge of the landslide had an obvious armchair-like sliding back wall, reflecting the traction failure characteristics. There are two cracks on the top of the slope, which are caused by tensile shear and tensile crack respectively. The cracks generally show the development process from the front edge to the back edge. It presents the loess slope progressive sliding characteristics.

(5) According to the numerical simulation results, the strain level of the numerical model is similar to the centrifugal test. The numerical simulation also has good implementation in excavation unloading of springback. After excavation, the slope has gone through 6 plastic development stages, namely unloading rebound stage, shallow plastic development stage, shallow plastic deformation stage, deep plastic development stage, deep plastic deformation stage and deep sliding failure stage. The plastic zone development distribution has a good consistency with the centrifugal model deformation zoning diagram. The red part in as the same as the influenced zone. The gray is the uninfluenced zone.

Declarations

Acknowledgements

This research was financially supported by the Open Project of Technology Innovation Center of Land (Cultivated Land) Ecological Monitoring and Restoration Project of the Ministry of Natural Resources (Grant No.2020), Natural Resources Science and Technology Project of Jiangsu Province (Grant No. KJXM2019028), Fundamental Research Funds for the Central Universities (Grant No. B200202013).

Funding: This research was financially supported by the Open Project of Technology Innovation Center of Land (Cultivated Land) Ecological Monitoring and Restoration Project of the Ministry of Natural Resources (Grant No.2020), Natural Resources Science and Technology Project of Jiangsu Province (Grant No. KJXM2019028), Fundamental Research Funds for the Central Universities (Grant No. B200202013).

Conflicts of interest/Competing interests: The authors have no relevant financial or non-financial interests to disclose.

Authors' contributions: Conceptualization, Wen Yue Che, Jin Liu and Jianbing Peng. Methodology, Wen Yue Che, Jin Liu and Zhongjie Fan. Software, Yuxia Bai and Ying Wang. Formal analysis, Wen Yue Che, Jin Liu, and Wei Qian. Investigation, Wen Yue Che, Jin Liu, Zhongjie Fan and Ying Wang. Data curation, Wen Yue Che, Jin Liu and Zhongjie Fan. Writing—original draft preparation, Wen Yue Che and Yuxia Bai. Writing—review and editing, Wen Yue Che, Jin Liu.

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Figures

Figure 1

Schematic drawing of TLJ-500 geotechnical centrifugal testing machine.

Figure 2

Centrifugal slope model and excavation scheme design (a) Centrifugal slope model, (b) Model test box design, (c) Front view Centrifugal model in the cableway bucket, (d) Manipulator operation range, (e) Size of the remake blade of the manipulator, (f) Plan view Centrifugal model in the cableway bucket.

Figure 3

Deformation measurement plans (a) Principle of the earth pressure cell, (b) Photo of an earth pressure cell, (c) Location of each earth pressure cell, (d) PIV system principle, (e) Photo of a marking point, (f) Location of each marking points.

Figure 4

General layout of the centrifugal test model.

Figure 5

Grain size distribution curve.

Figure 6

Centrifugal loading process curve.

Figure 7

Deformation state of the model at each stage of the test: (a) Excavation completed, (b) Deep sliding surface penetrates, (c) Deep sliding begins, (d) Shallow sliding surface penetrates, (e) Shallow sliding begins.

Figure 8

Failure characteristics of the model during sliding test: (a) Deep sliding surface, (b) Shallow sliding surface.

Figure 9

The variation curves of earth pressure and acceleration.

Figure 10

Slope displacement vector diagram at different excavation stages: (a) variations in displacement vector after the first excavation, (b) variations in displacement vector after the second excavation, and (c) variations in displacement vector after the third excavation.

Figure 11

Horizontal and vertical displacement isoline maps at: (a) and (b) the first excavation stage, (c) and (d) the second excavation stage, and (e) and (f) the third excavation stage.

Figure 12

Displacement distribution at different slope elevations and time points: (a) after the first excavation, $T_1 = 2290$ s, (b) after the second excavation, $T_2 = 3504$ s, (c) after the third excavation, $T_3 = 4335$ s, and (d) after the sliding, $T_4 = 5140$ s.

Figure 13

Slope deformation zoning diagram: (a) Image of influence boundary of each stage, (b) Slope partitioned grid diagram.

Figure 14

Development of sliding surfaces and fissures.

Figure 15

Displacement curves of each measuring points.

Figure 16

Analysis of the cause of the cracks at slope top surface: (a) 1-1 section, (b) 2-2 section, (c) 3-3 section, (d) 4-4 section, (e) 5-5 section,

Figure 17

Displacement distribution at three sections.

Figure 18

Numerical slope model diagram.

Figure 19

Horizontal and vertical displacement isoline maps in every step of excavation condition: (a) Horizontal displacement isoline maps after step1, (b) Vertical displacement isoline maps after step1, (c) Horizontal displacement isoline maps after step2, (d) Vertical displacement isoline maps after step2, (e) Horizontal displacement isoline maps after step1, (f) Vertical displacement isoline maps after step1.

Figure 20

Horizontal and vertical displacement curves of points on the slope after every step excavation condition: (a) Horizontal displacement(step1), (b) vertical displacement(step1), (c) Horizontal displacement(step2), (d) vertical displacement(step2), (e)Horizontal displacement(step3), (f) vertical displacement(step3).

Figure 21

Diagram of slope plastic zone development under different time steps (a) unloading rebound stage, (b) shallow plastic development stage, (c) shallow plastic deformation stage, (d) deep plastic development stage, (e) deep plastic deformation stage, (f) deep sliding failure stage.