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Post-evaluation of slope-cutting on loess slopes under long-term rainfall based on model test

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Abstract

Failures of treated slope occurring in China are at a consistently increasing rate, leaving the huge number of treated loess slopes calling for post-evaluation, however, no mature technique is in place. Depended on an loess slope in Shaanxi province treated by slope-cutting, indoor geotechnical and model tests were conducted, revealing the rainwater infiltration characteristics and pressure varying characteristics inside the slope, the results of which were then adopted to perform the post-evaluation of the treated slope. The results showed that the rainwater scouring effect on the loess slope surface attenuates gradually, and enters a steady stage after the first year of rainfall. The rainwater preferentially penetrates the platforms with gradually attenuating rates, however the wetting front can not be deemed as the boundary between the saturated and unsaturated areas, as the most parts of the model slope were indicated unsaturated by the pore water pressure sensors. Caused by the in-situ stress release, the soil pressures don't increase but decrease sharply at the start of the rainfall. The displacements mainly occurs in the first two years of rainfall, following by steady periods. The model test results and investigation results were then used to conduct the post-evaluation of the prototype slope, which formed a post-evaluation frame relevant to other slope post-evaluations.

Introduction

At the end of the 20th century, China initiated the policy of western development, which promoted the construction of the infrastructure in the western China including railways, highways, water conservancy facilities, living houses, etc., inevitably brought on a great number of treated slopes. Shaanxi consists of northern Shaanxi, Central Shaanxi and southern Shaanxi, is included in the western China, thus was deeply influenced by the policy, most evidently in northern Shaanxi. Almost the whole area of northern Shaanxi is covered by Loess which is characterized by the collapsibility and looseness, leading to the insecurity of the treated slopes there after a few decades of operation under rainfall¹. That is the broadly distributed treated loess slopes in northern Shaanxi is calling for remedy to prevent them from collapse. This has posed a new issue of post-evaluation of treated slope, different from safety evaluation in designing stage^{2,4}.

As scholars addressed, the post-evaluation of treated slopes is about the operational situation of the slope including the adaptions to the environment, the cracks and the displacements of the slope, etc.^{5,6} To make the slope post-evaluation more objective, Tian⁷ employed the developed concept of post-evaluation of slopes, and brought in the stress evaluation method, while some other scholars focused on filed investigation method or experience judgement^{5,7}. However, these developed post-evaluation methods all falls into the field of qualitative of semi-quantitative assessment, with no consideration of rainfall. That seems to be the limitation of the developed post-evaluation technology.

In recent years, more and more scholars devoted to the field of slope stability and failure mechanism investigation under rainfall, incorporating modelling test⁸⁻¹¹, numerical simulation¹²⁻¹⁵, and filed monitoring¹⁶⁻¹⁸. As revealed, slope safety is critically influenced by rainfall, leading to the surface erosion to the slopes by rainwater runoff¹⁹⁻²³, degrading the slope soil strength as percolating into the slope²⁴⁻²⁶, and lessening the effective stress in the slope soil while saturating it²⁷⁻³¹. As the loess slope is loose, the rain drops and the runoff water can easily scour the soil particles of the slope surface, forming fall-holes and gullies in the slopes³². To alleviate the scouring effect by rain-water, some scholars proposed the vegetation covering method and geobarrier system, and ascertained the effectiveness of the methods in protecting the soil slopes from rainwater scouring and maintaining the stability of the slopes³³. It is well known that, the suction in the soil contributes significantly to the strength of slope soil, thus is a crucial factor to the slope stability³⁴. As the rainwater permeates into slopes, the water content of the slope soil increases inevitably deducing the decrease of the suction, even causing the lifting of the ground water table, which may eventually cause the collapse of the slopes³⁵. Moreover, as a

positive factor to the frictional strength of the loess, the effective stress favorable to the stability of the loess slope decreases with the increase of the pore water pressure in the rainfall process³⁶. Summarily, the rainfall is a critical factor inducing the instability of slopes. Within the circumstances of engineering design, the water drainage measures should be the obligatory practice to slope treating projects to minimize the hazards induced by rainwater percolating.

As the most convictive way to study the slope stability and failure mechanism, it was taken by many researchers in this field. Schenato et al.³⁷ brought an optical fibre sensing system into slope model test to measure the pore water pressure, the water content and the strain in the model slope during rainfall. The results revealed that the general evolution of the slope during rainfall comprised of 4 stages which validated the effectiveness of the fibre sensing system in slope model test. Lan et al.³⁸ employed the model test to validated the postulation that contraction and expansion in loess slope is related to the thermal and moisture fluctuations, based on which a quantitative relationship between the weather cycle and the thermal deformation of the loess slope was established. Chen et al.³⁹ employed model test to study the effect of vegetation covering on the rainwater scouring of the residual soil slope surface. It was found that the surface vegetation can reduce the scouring effect by lowering the splash erosion and adjusting the rainfall forms. Hung et al.⁴⁰ adopted centrifuge model test to study the influences of gravity, rainfall and earthquake on the stability of the clay slope. The results indicated that earthquake is most important factor influencing the stability of clay slopes. Sun et al.⁴¹ conducted physical model tests to study the failure mechanism of loess slope under rainfall, and found that rainwater infiltration reducing the matric suction in the loess slope in turn leading to the loss of shearing strength of the slope soils, eventually caused the failure of the slopes. From the above, the proceeding researches in the current field mainly focus on the pre-evaluation (serves the designing work), while a limited number of post-evaluation research were conducted without consideration of the influence of rainfall.

Aiming at the repairing works of the treated loess slopes widely distributed in the north of Shaanxi, a loess model slope shrunk by 10 times from the prototype slope treated by slope-cutting was built to study the treating effect. In the model test, the rainwater infiltration process and the variations of pore water pressure, and soil pressure and displacement at the key positions were recorded, which were in turn used to conducted the post-evaluation of the prototype slope. The formed method of post-evaluation can be adopted in the treating effect assessments of other type of slope, while the rainwater infiltrating characteristic and the variations of pressures and displacements in the slope have facilitating effect in the further researching and designing works at this aspect.

76 The Prototype Failure

The slope prototype is situated in Luochuan county, Yan'an city, Shaanxi Province, with the latitude of 35°45'19.46" N and the longitude of 109°25'34.63" E as illustrated in Fig. 1. The average elevation of Luochuan county is about 1100m. It consists of temperate continental monsoon climate, with an annual average temperature of 5~17°C, and an annual utmost maximum temperature of 37.4°C. The average annual precipitation of Luochuan County is approximately 606 mm, briefly concentrated in July, August and September.

Figure 1. Slope prototype location in China. It is situated in Luochuan county, Yan'an city, Shaanxi Province, with the latitude of 35°45'19.46" N and the longitude of 109°25'34.63" E.

Located on the loess plateau, this project is a loess slope treated by slope-cutting with dwelling houses on the crest. That is, the soils of the slope are wholly loess formed in late pleistocene epoch. As Fig. 2 shows, the height of the slope is 17.6 m, cut into three grades with the identical gradient of 56°. The height of the first grade is 5m, the height of the second grade is 4m and the height of the third grade is 5.6m.

90 From field investigation, this project was built around the year 2012, having been run for about 5 years when the filed
 91 investigation. After a long period of operation, affecting by the rainfall, the slope was found collapsed in the right side
 92 (see Fig. 2). To perform the post-evaluation of this project by model test, the simulated time must be 5 years according to
 93 the actual project operation time frame. Depending on the similarity theory, the test time can be shortened by 100 times,
 94 letting this model test be completed in a more tolerable period time.

95
 96 **Figure 2.** The slope prototype section with collapse. The ratio and height of each grade of slope are indicated in black in the sectional
 97 view, while the collapse area is indicated in red in the original location.

98 Methodology and Test Model

99 **Test Device.** The current model test was conducted in the Soil Mechanics Laboratory of Shangluo University, China.
 100 Adopted from the research of Liu⁴², a 1 m wide, 2.5 m long and 1.8 m high model box was employed in the experiment
 101 (see Fig. 3a). The left and right walls of the box were made from organic glass, letting the displacements in the slope and
 102 the wetting front visible. The base and back walls of the model box were made from plank. And the the lower 30cm high
 103 front was blocked by a plank which was drilled holes to vent the rainwater infiltrated into the slope. The upper plank of
 104 the front could be removed to let slope surface unrestrained.

105 A frame made from steel pipes drilled holes of 1mm in diameter on one side was used as rainfall simulator. The valve
 106 connected to the water pipe (see Fig. 3b) was used to controll the rainfall intensity. Prior to the start of rainfall, the rainfall
 107 intensity could be calibrated to the certain values, which were achieved by a beaker and a measuring cylinder.

108
 109 **Figure 3.** Model test box and rainfall simulator. **a** Model test box; **b** Rainfall simulator. The left and right walls of the model box are
 110 made from transparent organic glass, while the front, back and bottom walls are made from plank. The upper 1.5 m high front plank
 111 can be removed to make the model slope surface free.
 112

113 **Test Model.** The test model corresponded fully to the prototype characteristics of the slope in Luochuan County,
 114 Yan'an city, Shaanxi Province, as depicts in Fig. 1. The scale of the model was 1:10, thus the total model height was 1.76
 115 m, the total length of the model was 2.5 m identical with the length of the model box, while the 0.377 m long in the front
 116 was just 30cm thick and horizontal. By slope-cutting, the three gradients of the three grades of slope were the same value
 117 of 56° identical with the prototype, while the heights of the first, second and third grades of slope were 0.5 m, 0.4 m and
 118 0.56 m respectively. The widths of the second and third platform were identically 0.38 m (Fig. 4).

Figure 4. Slope model. The width of the slope model is 1 m, consistent with the width of the model box.

Similar to the research of Liu⁴², the slope prototype was firstly sampled as undisturbed samples in the field investigation. The density, water content, grain size distribution, permeability and shear strength of the samples were determined by indoor experiments. Accordingly, the slope model material was the same soils procured from the project site. The model was constructed by stratified compaction. To control the mode slope soil density and water content, the soil was prepared to a certain water content in advance, and the mass of each layer of 10 cm thick was weighed before fill in. When the model box was filled to the required height, the upper front plank was removed, then the model was cut to the shape corresponding to the prototype.

As Fig. 5 illustrates, the model construction process was considerably complex, which can be delineated as follows:

(1) The procured loess was prepared with a precise water content as the prototype ($\omega=17.4\%$), and packed into the model box with a certain quantity as a layer, then each layer was compacted to a thickness of 10 cm to control the density of the slope model.

(2) In the model construction process, the pore-water pressure sensors and soil pressure sensors were buried in the designed position.

(3) Synchronously, little colored sand particles were buried at the desired position close to the left and right walls of the model box as the inner displacement marks.

(4) As the model box was rammed and filled to the desired height, it was then excavated to the shape corresponding the prototype

(5) Lastly, the surface displacement markers was set at the three shoulders of the slope model by nails.

Figure 5. Slope model construction process. The construction process strictly follow the sequence denoted by the arrows in blue. The words in red illustrate the operations in each step.

Model Materials and Similarity Relation. The loess of the prototype slope was used to construct the slope model. Therefore, the water content, density, permeability coefficient and other mechanical properties of the model material were consistent with the prototype slope. All the relevant parameters are in Table 1.

Parameters	Value	Parameters	Value
Water content (%)	1.42	Compression modulus (MP _a)	5.58
Permeability coefficient (cm/s)	17.4	Internal friction angle (°)	27.0
Density (g/cm ³)	5.4×10 ⁻⁴	Cohesion (kP _a)	15.0

Table 1. Key properties of model soil material.

It is noteworthy that the soil properties were obtained from indoor tests according to Liu⁴². The water content of the model soil was measured by the drying method: a certain mass of the loess was weighed before and after being dried in an oven for 8 hours under 105~110°C. The water content was derived as the ratio of the loess mass lost to the loess mass after dried. The permeability coefficient was tested by the standard variable head permeability test: firstly, installed the loess specimen into the penetration container and sealed. Then, the the specimen was saturated in the container. Thereafter, filled the head pipe with water to the proper water head. Opened the inlet of the penetration container, and

153 recorded the the water heads in the head pipe and the corresponding time. The following equation was used to calculate
 154 the permeability coefficient.

$$155 \quad k_T = 2.3 \frac{aL}{A(t_2 - t_1)} \log \frac{H_1}{H_2} \quad (1)$$

156 Here, L is the specimen height, a is the section area of the head pipe, t_1 and t_2 are the starting and end timings of the water
 157 head variation respectively, H_1 and H_2 are the starting and end water heads corresponding to t_1 and t_2 . The density of the
 158 soil sample was determined by the standard ring knife test: the prototype loess was sampled as ring specimens and
 159 weighed by the scale. The density of the sample was the ratio of the specimen mass to the specimen volume in the ring
 160 knife. The compression modulus of the loess was determined by the consolidation test: the standard specimens were
 161 installed into the osmotic pressure container firstly. Then the different specimen heights at corresponding pressures were
 162 recorded while the compressing settlement was stable. The varying void ratios derived from the gravity of soil particles,
 163 initial density of the specimen, the initial water content and the specimen heights at different pressures were used to
 164 derive the compression modulus of the specimen. The internal friction angle and the cohesion were measured by the
 165 standard direct shear test: the ring specimens were installed into the direct shearing apparatus, then applied normal
 166 pressure and quick shearing process. The shear stresses at shear failure were measured as the shear strengths, which can
 167 be fitted by the shear strength line in the coordinate system of $\tau - \sigma$. The intercept of the line at the τ axis was the
 168 cohesion value, while the angle between the line and the σ axis was the internal friction angle value. To be persuasive, the
 169 indoor test process is illustrated in Fig. 6.

170 **Figure 6.** Photos of indoor geotechnical tests. **a** Density test; **b** Direct shear test; **c** Compression test; **d** Water content test. The soil
 171 samples were procured from the prototype slope site in Fig. 1. The red words illustrate the main operation or status in each step.
 172

173 According to the π theorem^{43,44}, the similarity criterion of the slope model was derived by dimensional analysis. For the
 174 complexity of the slope model, it was impossible to satisfy the similarity of each parameter. Therefore, we chose the
 175 primary parameters geometric length, density and gravity acceleration as fundamental dimensions according to the model
 176 characteristics and the test purpose. All the similar constants in this test are listed in Table 2.

Parameters	Similarity relation	Similarity constant
Geometric dimension (L)	C_L	10
Density (ρ)	C_ρ	1
Gravity acceleration (g)	C_g	1
Stress (σ)	$C_\sigma = C_\rho C_g C_L$	10
Strain (ε)	$C_\varepsilon = 1$	1
Displacement (s)	$C_s = C_L$	10
Cohesion (C)	$C_C = C_\rho C_g C_L$	10
Internal friction angle (ϕ)	$C_\phi = 1$	1
Rainfall intensity (C_q)	$C_q = C_\rho^0 C_g^{0.5} C_L^{0.5}$	$\sqrt{10}$

177 **Table 2.** Similarity ratios of the test model.

178 It should be noted that the similarity constant of rainfall duration was not derived from π theorem, but from the
 179 calculation process according to Terzaghi's consolidation theorem⁴⁵. This method was validated by Li⁴⁶ and was
 180 employed by Zhao⁴⁷ and Tang⁴⁸ in the study of slope stability under rainfall. Though the geometric dimensions of the
 181 model slope were different from the prototype, in the experimental process the consolidation degrees of the model slope
 182 and the prototype should be the same. Based on Terzaghi's consolidation theorem, the consolidation degree of the slope
 183 model and the prototype can be delineated as:
 184

$$185 \quad U = 1 - \beta e^{-\lambda T_v} \quad (2)$$

186 where β and λ are two invariable constants, T_v is the time factor varying with time. Thus, the consolidation degree U is
 187 only affected by the consolidation time t .

Reasonably, β and λ values of the model and the prototype are the same. Thus, upon the same consolidation degree their time factors are identical, i.e.:

$$T_{vp} = T_{vm} \quad (3)$$

Here, T_{vm} is the time factor of the model slope, and T_{vp} is the time factor of the prototype. Eq. (4) and (5). delineate the relations between them and the consolidation time.

$$t_p = (H_p^2 / C_v) T_{vp} \quad (4)$$

$$t_m = (H_m^2 / C_v) T_{vm} \quad (5)$$

Here, t_m and t_p are the consolidation time of the model slope and the prototype respectively; H_m and H_p are the dimensions of the model slope and the prototype respectively; C_v is the identical consolidation constant of the prototype and the model slope. Thus, the similarity constant of rainfall duration can be calculated as :

$$C_t = \frac{t_p}{t_m} = \frac{H_p^2}{H_m^2} = C_L^2 \quad (6)$$

In the current study, C_L equaled to 10, deriving C_t equaled to 100. That is, the test duration was shortened by 100 times, letting the simulation finished in an acceptable time. From the filed investigation, the project had run for about 5 years, thus leading to an experiment duration of 0.05 years.

Measuring System and Rainfall Scheme. A total of 7 pore water pressure sensors and 7 soil pressure sensors were employed in the test. The pore water pressure sensor U4 was buried at a depth of 10 cm below the shoulder of the second grade of slope, while the buried depth of U3, U2, U1 increasing 20 cm sequentially in a vertical line with U4. The pore water pressure sensors U5, U6 and U7 were buried 10 cm below the toes of the first, second and third grade of slope respectively. To be meaningful, the soil pressure sensors P1, P2, P3, P4, P5, P6, P7 were buried at identical depths as U1, U2, U3, U4, U5, U6, U7 respectively with corresponding positions. The pressure sensors were connected a strainometer, which converted the pressures into digital signals and sent them to the computer to store. Though the experiment corresponded to a planar issue, the pore water pressure sensors and soil pressure sensors were set near the axial plane of the model delivering more reliable data⁴².

To measure the displacements of the three slope shoulders, three key points were marked by nails inserted into the corresponding positions, which are illustrated by Fig. 7 as S5, S4, S6 sequentially. S5 was located on the shoulder of the first grade of slope, while S4 and S6 were located on the shoulders of the second and third grades of slope respectively. To measure the displacements of the inner positions, three points in the slope close to the right wall of the model box was marked by colored sand particles which are S1, S2, S3 at the identical depth of P1, P2, P3 respectively as Fig. 7 shows. Meanwhile, three points in the slope close to the left wall of the model box at the same depths corresponding to S1, S2, S3 were marked as S1', S2', S3' respectively in the same manner. The displacements of the key points were derived as the differences of distances before and during rainfall, which were measured by a laser rangefinder seated a fixed position. The accuracy of this laser rangefinder is up to 0.01mm which could satisfy measuring requirement without any doubt. Conveniently, the laser rangefinder was also connected to the computer to display the distance values. It needs to be emphasize that, the real displacement of S1 is the average value of S1 and S1', the same goes for S2 and S3.

223 **Figure 7.** Full dimensions of model slope and layout of measurement points. P1, P2, P3, P4, P5, P6, P7 represent soil pressure sensors;
 224 U1, U2, U3, U4, U5, U6, U7 represent pore water pressure sensors; S1, S1', S2, S2', S3, S3', S4, S5, S6 represent displacement
 225 markers.

226 To capture the deformation process of the slope and the rainwater percolating process in the slope, a camera was used
 227 to take photos from the front and the sides of the model slope at constant intervals.

228 As mentioned above, the annual precipitation of Luochuan county is about 606 mm, mainly concentrated in July,
 229 August and September. Here, we assumed that the total annual precipitation distributed in three months, and every month
 230 was with just one time of rainfall lasting for 2 hours. After the rainfall, the slope model was placed as undisturbed for the
 231 remaining time of the month. Consequently, the rainfall intensity was constantly 101 mm/h. The simulating rainfall
 232 intensity in the test could be derived from dividing 101 mm/h by the similarity constant $\sqrt{10}$, deducing the value of 31.94
 233 mm/h. Similarly, the total test duration could be derived from dividing 5 years by the similarity constant 100 (see Eq. (5))
 234 rendering the value of 0.05 years, i.e. 18 days, while the intervals between the 3 simulating rainfall could be calculated
 235 from dividing 30 days by the similarity constant 100 then subtracting 2 hours, rendering a duration of 5.2 hours. The
 236 detailed experimental scheme of one year is listed in Table 3, which could be repeated 5 times to simulate the 5 years of
 237 rainfall.

Simulating time	Actual time	Rainfall start	Rainfall end	Read data
0h	0h	Yes		Yes
2h	2h		Yes	Yes
7.2h	30d	Yes		Yes
7.2h+2h	30d+2h		Yes	Yes
14.4h	60d	Yes		Yes
14.4h+2h	60d+2h		Yes	Yes
21.6h	90d			Yes
28.8h	120d			Yes
36h	150d			Yes
43.2h	180d			Yes
50.4h	210d			Yes
57.6h	240d			Yes
64.8h	270d			Yes
72h	300d			Yes
79.2h	330d			Yes
86.4h	360d			Yes

238 **Table 3.** The rainfall scheme. The process in this table should be repeated 5 times to simulate 5 years of rainfall.
 239

240 Generally, the experimental process followed the steps below:

241 (1) The initial values of the soil and water pressure sensors were measured before they were buried in the constructing
 242 of the model slope.

243 (2) When the built of the slope model was accomplished, the sensor cables were connected to the strainometer which
 244 was connected to the computer to record the pressure data. Also, a laser rangefinder was fixed at a position in front of the
 245 slope model and was connected to the computer to get the distances between the position and the displacement points (S1,
 246 S1', S2, S2', S3, S3', S4, S5, S6), thus to derive the horizontal displacements of the 6 points.

247 (3) The rainfall intensity of the simulator was calibrated to 31.94 mm/h, and acted on the model slope when the
 248 computer program started.

249 (4) The steps in Table 3 was repeated 5 times to simulate the 5 years of operation while the computer program
 250 recorded the soil pressures, pore water pressures and the horizontal displacements.

251 Results and discussion

252 **Slope Scour Failure Process.** By the camera fixed in front of the slope model, the slope scour failure process in the
 253 rainfall was captured at certain timings. Depending on the field investigation, the simulating period was 5 years,
 254 responding to the long-term rainfall effect on the slope. However, the scouring effect of rainfall after the third year was
 255 negligible, thus is not presented in this section.

256 Fig. 8 shows the scour failure process of the model slope during the first year of rainfall. It is visible that the scouring
 257 effect during this year is very vibrant. Within the 25 minutes of the start of the rainfall, the scouring was visibly splash
 258 erosion, with the raindrops hitting the slope surface and forming shallow pits. Later, within the duration of 25 minutes to
 259 60 minutes, the scouring pattern could be categorized as surface erosion. At the timing of 25 minutes, the shallow pits
 260 formed by the the raindrops started to connect, forming shallow gullies. At the timing of 45 minutes, the left part of the
 261 third grade of slope showed a shallow sliding in a small range, and the gullies in the slope surface were completely
 262 formed. Lastly, within the duration of 60 minutes to 120 minutes, the scouring pattern could be categorized as gully
 263 erosion and sliding. At the timing of 60 minutes, the left part of the first grade of slope showed the deepened gully with

264 7~8 cm in depth and 5 cm in width, while the second grade of slope showed a gully with 4~5 cm in depth and 3~4cm in
 265 width. Meanwhile, the middle part of the third grade of slope showed a bigger range of slide, which destroyed the
 266 displacement marker S6. Clearly, the rainwater accumulated upon the left part of the second grade platform softened the
 267 soils there, facilitated the formation of the gully in the first grade of slope induced by the flowing down of accumulated
 268 rainwater. Also, it can be inferred that the bigger range of slide on the third grade of slope was caused by the rainwater
 269 accumulated upon the slope crest, degrading the shear strength of the soil there while infiltrating. Thereafter, in the later
 270 duration of the first rainfall, the formed gullies were deepened and widened by the running down of rainwater. When this
 271 rainfall was ended, the main gully in the left part of the first grade of slope was about 30 cm in depth and 10 cm in width.

272 Fig. 8b depicts the scour failure process of the model slope during the second rainfall of the first year. Different from
 273 the first rainfall, this process just consisted of two stages, which were the undisturbed stage and the gully erosion and
 274 sliding stage. The duration from the rainfall start to the timing of 20 minutes belonged to the undisturbed stage, which
 275 composed no visible erosion to the slope surface, as the running-down rainwater being limpid. It can be inferred that,
 276 during the interval between the first and the second rainfall the wetted soil of the shallow layer consolidated, becoming
 277 hard for the rainfall to erode, thus leading to the undisturbed stage of this process. As the rainfall advanced, the slope soils
 278 of the slope were softened again, thus rousing the gully erosion and sliding stage. The gully erosion and sliding stage was
 279 composed of the duration from 20 minutes to the end of this rainfall, within which the gullies were expanded and shallow
 280 slides occurred occasionally. The second grade of slope showed small ranges of shallow slides at the timings of 30
 281 minutes, 40 minutes, 56 minutes and 78 minutes respectively, which destroyed the displacement marker S4 completely.
 282 Meanwhile, the third grade of slope showed larger slides at the timings of 80 minutes and 100 minutes respectively,
 283 cutting the total gradient of the third grade of slope to declined. Also, the number and dimension of gully were developed
 284 in this stage. At the end of the second rainfall in this year, the width of the main gully in the first grade of slope was
 285 approximately 13 cm.

292 **Figure 8.** Scour failure process during the first year of rainfall. **a** Scour failure process during the first rainfall of the first year; **b** Scour
 293 failure process during the second rainfall of the first year; **c** Scour failure process during the second rainfall of the first year. The arrows
 294 in blue denote the sequence of scour development.

295 Fig. 8c presents the scour failure process of the model slope during the third rainfall of the first year. Comparing with
 296 the second rainfall, the scouring effect in this rainfall process was more weak, with the third grade of slope showed a little
 297 range of shallow slide at the timing of 85 minutes while the gullies gradually developed. Visibly, the main gully at the left
 298 of the first grade of slope developed to 15 cm of width at the end of this rainfall.

Figure 9. Scour failure process during the second year of rainfall. The arrows in blue denote the sequence of scour development. The red dotted lines denote the main gully at the first grade of slope.

Figure 10. Scour failure process during the third year of rainfall. The arrows in blue denote the sequence of scour development. The red dotted lines denote the main gully at the first grade of slope.

Fig. 9 and Fig. 10 present the scour failure process of the model slope in the second and third year of rainfall respectively. It was found that, during these two years of rainfall the model slope was at the relative steady stage, with no slide occurred while limpid rainwater flowing down. However, the main gully at the left of the first grade of slope developed insistently. At the end of the rainfall of the third year, the width of the main gully at the left of the first grade of slope was approximately 30 cm with the depth of approximately 45 cm. On one hand, the erosion effect of the first year of rainfall formed the outlet of the rainwater upon the slope surface, which could alleviate the scouring effect of the rainwater on the slope. On the other hand, in the long interval between the first year of rainfall and second year of rainfall, the slope soils were strongly consolidated thus impeding the further scouring by the rainwater. Therefore, we could conclude that the slope was relatively steady after the first year of rainfall, with the gully dimensions insistently developing.

Rainwater Infiltration Characteristics. Rainwater infiltration can saturate slope soils, thus increasing the pore water pressure, which significantly affects the stability of the slope. For slope-cutting treated slopes, the infiltrated rainwater would reduce the internal friction angle of the slope soil, which is adverse to the treating project. Additionally, the infiltrated rainwater causing the large water pressure in the slope body is also adverse to stability of the project. Some researchers studied the influence of rainfall on the slope stability and found that the rainfall was the most significant factor affecting the stability of slopes, the rainwater penetration dominates the deforming process of slopes^{49,50}. In this study, the two side walls of the model box were of transparent glass. Thus, the advancement of the wetting front could be captured by cameras from the sides.

Fig. 11 presents the wetting front advancement with the rainfall process in the first year. It is clear that the rainwater preferentially penetrated the platforms vertically, which could be due to the water accumulated upon the platforms. The rainwater accumulated upon the platforms produced water heads there, actuating the rainwater into the soils vertically in a larger rate. When the first rainfall lasted for 10 minutes, the penetration depths under the three platforms were identically 8 cm. When the first rainfall lasted for 20 minutes, the penetration depths under the first grade of platform, the second grade of platform and the third grade of platform were 9 cm, 11 cm and 10 cm respectively. When the first rainfall lasted for 40 minutes, the penetration depths under the first grade of platform, the second grade of platform and the third grade of platform were 11.3 cm, 11.5 cm and 12 cm respectively. Thus we could derive that the vertical penetration rate of the rainwater decreased with the proceeding of the rainfall. The average penetration rate was about 0.8 cm/min in the first 10 minutes of the rainfall, which decreased to about 0.2 cm/min when the first rainfall lasted for 20 minutes, then decreased to about 0.075 cm/min when the first rainfall lasted for 40 minutes. It is reasonable that, the pore air in the shallow layer of the slope soils is easier to be discharged by the infiltration of rainwater, thus giving more unblocked paths to the penetration of the rainwater, which caused the higher penetration rate at the beginning of the rainfall. Eventually, when the first rainfall lasted for 120 minutes, this rainfall was ended, showing an average vertical penetration rate of about 0.090 cm/min. Summarily, the wet front advancing rate decreased gradually in the first rainfall of the first year, eventually stabilizing at the value of 0.090 cm/min.

339 Similarly, we could derive the wetting front advancing rate after the rainfall. 30 minutes after the end of the first
340 rainfall, the average wetting front advancing rate under the platforms was approximately 0.07 cm/min. 5.2 hours after the
341 first rainfall, the vertical penetration depths under the first grade of platform, the second grade of platform and the third
342 grade of platform were 25 cm, 36 cm and 38 cm respectively, delivering the average wetting front advancing rate of about
343 0.026 cm/min. From the above, we can conclude that the wet front advancing rate after the rainfall is less than that during
344 the rainfall, with a decreasing trend over time. Further more, we could found from Fig. 11c that the penetration depths
345 under the slope shoulders were greater than that under the slope faces and platforms. It should be attributed to the dual
346 effects of penetration from the platform and the slope face, which could be deemed as the superposition of the two effects.

Figure 11. Rainwater infiltration within the first year. **a** First rainfall lasted for 10 minutes; **b** First rainfall lasted for 30 minutes; **c** First rainfall lasted for 120 minutes; **d** 5.2 hours after the first rainfall; **e** Second rainfall lasted for 120 minutes; **f** 5.2 hours after the second rainfall; **g** Third rainfall lasted for 120 minutes; **h** 70 hours after the third rainfall. The black line in subfigure **h** denotes the wetting front.

During the second and third rainfall of the first year, the wet front insistently advanced downward with a rate less than that in the first rainfall, which was gradually decreasing. At the end of the third rainfall, the average advancing rate of the wet front was about 0.05 cm/ min. At the ends of the rainfall, the wet front was clear, but became obscure 5.2 hours later (see Fig. 11e, f). 5.2 hours after the second rainfall, the wet front showed a circular arc, seemingly corresponding to the classical Swedish arc method in solving the slope safety. Eventually, 70 hours after the third rainfall of the first year, most of the slope body was wetted, with only the triangle area in the lower right corner being dry. At that time, the horizontal edge of the triangle area was about 1.3 m.

374 **Figure 12.** Rainwater infiltration within the second year. **a** First rainfall lasted for 120 minutes; **b** Second rainfall lasted for 120
 375 minutes; **c** Third rainfall lasted for 120 minutes; **d** 70 hours after the third rainfall. The black lines denote the wetting front.

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381 **Figure 13.** Rainwater infiltration within the third year. **a** First rainfall lasted for 120 minutes; **b** Second rainfall lasted for 120 minutes;
 382 **c** 5.2 hours after the second rainfall. The red lines denote the wetting front. When 5.2 hours after the second rainfall in this year, the
 383 whole slope model was wetted, thus the wetting front was not visible.

384 Fig. 12 and 13 present the wetting front advancing processes in the second and third year of rainfall respectively.
 385 Because the entire slope body was wetted at 5.2 hours after the second rainfall of the third year, the followed rainwater
 386 migration was not visible thus not presented here. As can be seen, in the second and third year the lower right triangle dry
 387 area shrunk gradually, and vanished 5.2 hours after the second rainfall of the third year. However, the triangle dry area
 388 shrinking rate was very mini, even imperceptible during the rainfall processes for the relatively short duration of rainfall.
 389 Approximately, the shrinking rate of the dry triangle was about 0.003 cm/min 5.2 hours after the second rainfall of the
 390 third year, almost 10 times less than that of the end of the first year.

391 **Pressure Variations of the Model Slope.** The slope stability is directly correlated with the pressures inside, inclusive
 392 of the soil pressure and the pore water pressure. Theoretically, in the rainfall process the soil pressure and the pore water
 393 pressure to some extent increase, thus raising the shear stress and lowering the shear strength in the slope. In a case when
 394 the increment of the shear stress exceeds the limit, the slope may collapse⁵¹. In this study, before the start of test, different
 395 pressure coefficients were set in the capturing program. According to this, the computer program can automatically
 396 convert the signals sent by the strainometer into pressures, and the actual pressures can be derived by subtracting the
 397 initial pressure from that.

398 **Pore Water Pressure Variations.** The variations of the pore water pressures of the 7 points (U1, U2, U3, U4, U5, U6,
 399 U7) with time are shown in Fig. 14. As stated earlier, the annual precipitation is assumed to concentrate in three months
 400 while the remaining period of the year are without precipitation. The rainfall of each month was concentrated in 2 hours
 401 with a constant intensity. Generally, during the 5 years of rainfall the pore water pressures of the 7 points varied
 402 considerably, with a general trend of decreasing except for the point U5, seemingly inconsistent with the general theory of
 403 soil mechanics⁵². Only for the point U5 the pore water pressure showed positive values in the rainfall process, indicating
 404 the saturated condition nearby. As can be seen, the pore water pressure value of U5 increased from nearly 0 kPa to about
 405 1.5 kPa in the rainfall of the first year, then gradually increased to about 2 kPa at the end of the fifth year, implying the
 406 concentration of infiltrated water at the toe of the first grade of slope. That was consistent with the findings of
 407 Chueasamat⁵⁰. For other key points, the most obvious is that the pore water pressure value of U1 decreased insistently
 408 from -10.35 kPa to about -12 kPa, and the pore water pressure value of U4 declined insistently from -6.05 kPa to about
 409 -12 kPa with a more drastic decreasing rate in the early years. As U1 was situated at the deepest location, the infiltrated

rainwater was unable to recharge it while the soils under it absorbing the water nearby, causing the saturation degree of U1 declined. According to Fredlund's unsaturated soil theorem⁵², the negative pore water pressures in the soil implies suctions, while lower saturation corresponding to higher suctions. Causally, the pore water pressure value of U1 decreased in the rainfall process. As U4 was situated near the shoulder of the second grade of the slope, it was exposed in the air after the first rainfall of the first year when the second grade of shoulder was ruined by the rainwater. That caused the abnormal increasing of suction of U4 meaningless to this research. However, U2 and U3 situated at the mediate depth of the model slope, the rainwater recharging effect from the above soils and the absorbing effect from the soils below kept in balance, leading to no perceptive change of the pore water pressure variation there.

Moreover, from Fig. 13 we could find that the model slope was completely wetted by the rainwater, but the most part of the model slope except U5 showed negative pore water pressures from Fig. 14, implying the unsaturated condition. Thus, we can conclude that the wetting front can't be deemed as the boundary between the saturated and unsaturated areas in rainfall processes. Accordingly, the developed Green-Ampt infiltration model⁵³ deemed that after rainfall the rainwater continually migrates inducing a unsaturated area behind the wetting front. Thus, this developed Green-Ampt model is supportive to the findings above.

Figure 14. Pore water pressure variations within the test. **a** First year; **b** Second year; **c** Third year; **d** Fourth year; **e** Fifth year. The rainfall period of each year was comprised of 3 times of rainfall, with each time lasting for 2 hours.

Soil Pressure Variations. The variations of the soil pressures of the 7 representing points (P1, P2, P3, P4, P5, P6, P7) with time are presented in Fig.15. In the rainfall duration of the first year, it is adverse to the classical soil mechanics that

the soil pressures of all the representing points except P1 declined drastically, which could be attributed to the in-situ stress release caused by the rainwater infiltration. That was most prominent for P6, with a declination from 14.3 kPa to 1.0 kPa in the first rainfall of the first year. In the duration after rainfall of the first year, all the seven representing points showed no regular variation of soil pressure except some mini fluctuations, indicating a relative steady status of the model slope. However, for the point P1 the soil pressure started to decrease in the rainfall of the second year, which is later than other points. These could be attributed to the deepest location of P1, which needed longer time for the rainwater infiltration to influence. As the second deepest point, P2 showed insistent declining soil pressure in the second year, indicating that the in-situ stress was still releasing in the deeper place within this period. In the years later, the soil pressure of P2 insistent decreased with a lower rate, while the soil pressure of P1 fluctuated down. At the end of the fifth year, the soil pressures of P1 and P2 were about 3.5 kPa and 2.0 kPa respectively. Conversely, the soil pressures of other points remained steady since the start of the second year, implying the steady status of the shallower layer of the model slope. Summarily, inducing by the in-situ stress release, the soil pressures in the model slope decreased but not increased, implying the effect of in-situ stress release overweighted the effect of self-weight increase by rainwater infiltration, seemingly inconsistent with the classical soil mechanics⁵⁴.

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Figure 15. Variations of soil pressure inside of the model slope. **a** First year; **b** Second year; **c** Third year; **d** Fourth year; **e** Fifth year. The rainfall period of each year was comprised of 3 times of rainfall, with each time lasting for 2 hours.

Displacements of the Key Points. The distances between the 6 key points (S1, S2, S3, S4, S5, S6) and a fixed point were measured by the laser rangefinder, the differences of which before and after the start of rainfall were the horizontal

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displacements for the corresponding points. Fig. 16 shows the horizontal displacement variations of the 6 key points with time. In the first year, all the 6 key points except S1 showed displacements increasing from 0 mm, especially during rainfall. With the progress of the rainfall, the soils of the slope were wetted, lessened the internal frictional angel and the cohesion of the slope soils thus induced the yielding of some of the slope soils, which ultimately caused the deformation of the slope. At the end of the first year, the displacements of S2, S3, S5 increased to 15.1 mm, 9.2 mm and 12.9 mm respectively, while the displacement of S1 fluctuated around 0 mm. Reasonably, the fluctuations of the displacements were caused by the measuring errors of the apparatus. It is noteworthy that, the displacement of S6 increased to about 35 mm during the first rainfall of the first year, and the displacement of S4 increased to about 20 mm during the second rainfall of the first year. Combining with the rainwater scouring results addressed above, the displacement marker S4 and S6 were ruined in the second rainfall and first rainfall of the first year respectively, thus leading to the peculiar large increments of displacement. Therefore, the figures of the second to the fifth year do not show the displacements of S4 and S6.

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Figure 16. Variations of horizontal displacements of the six key points with time. **a** First year; **b** Second year; **c** Third year; **d** Fourth year; **e** Fifth year. The rainfall period of each year was comprised of 3 times of rainfall, with each time lasting for 2 hours.

In the second year, the displacement of S5 fluctuated up to about 14 mm and kept fluctuating around this value in the following years. And the displacements of S2, S3 fluctuated around 2.5 mm and 9.2 mm respectively, while the displacement of S1 still fluctuated around 0 mm. Thus, we could conclude that the point S1 kept stationary in all the five

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years of rainfall. For the points in a vertical line with S1, the larger upward distance from the point S1 in the slope model, the larger displacement with it. That indicated the potential sliding surface went between S1 and S2. This finding is consistent with that addressed by Cui et al.⁵⁵ while presenting the case of the landsliding in five stages: steady deformation, slow deformation, intense deformation, steady deformation and intense deformation. However, we could also find that the displacements of the remaining 4 points fluctuated with no increasing trend after the second year, likely indicating the stable state of the model slope ultimately.

Post-Evaluation of the Treating Project

Post-evaluation was brought into slope treating project in China by Zheng⁴, proposed the definition of post-evaluation of slope treating. In pre-evaluation, engineers just concern about the safety of the project after the completion of construction. Whereas, in post-evaluation they usually focus on the slope safety after a long term of operation and the running situation. As Zheng addressed, the displacement rates of key points of the slope not greater than 0.1 mm/day could used as an indicator of the slope safety. In addition, corresponding to the safety factor, a compound safety factor of slope was adopted to judge the treating effect of the slope, which is detailed in Table 4. Based on the deforming and failure degree, some qualitative criteria⁴ were also introduced into the post-evaluation of slope treating effect. In accordance with the above criteria, this section carries on the post-evaluation of the prototype project adopting the filed investigation data and the model test data.

Indicator value range	Treating effect
$K > 1.20$	Very good
$1.10 < K \leq 1.20$	Good
$1.0 < K \leq 1.10$	Not bad
$K \leq 1.0$	Conservative or failed

Table 4. Compound safety factors with treating effect.

Post-evaluation based on the Deformation and Failure Degree. From Fig. 16, at the end of the fifth year the maximum horizontal displacement of the model slope was about 14mm, rendering the maximum horizontal displacement of the prototype slope of 140mm. Also, the main gully width in the first grade of the model slope was about 30 cm at the end of the fifth year, the whole area of the third grade of the model slope slid. From that, the ratio of the maximum displacement to the slope height was approximately 0.08, while the ratio of the collapse area to the slope surface was approximately 0.3. Consequently, the maximum collapse ratio could be deemed as 30.0%, seemingly an overall destruction which had much influence on the operation of the slope. Also, that was much consistent with the results of the filed investigation (see Fig. 2). Thence, the treating effect of the project could be preliminarily judged as failed according to Zheng's qualitative criteria.

Post-evaluation based on the Displacement Rate. From Fig. 16, the maximum displacement was about 14 mm for the point S5 at the end of the fifth year, from which the maximum prototype displacement was 140 mm at the end of the fifth year. Thus, the displacement rate of the prototype slope in the five years was approximately 0.08 mm/day, indicating the safety of the treating project. Consequently, the treating effect of the current project could be preliminarily judged as good or very good according to the qualitative criteria proposed by Zheng.

Post-evaluation based on the compound safety factor. The Morgenstern-Price method has been widely accepted as an effect way to solve the safety factor of soil slopes, as it can generate the sliding surface with consideration of all the inter-slice forces and satisfies all the equilibriums of forces and moments^{56,57}. This section employed the Morgenstern-Price method in the software Geo-studio to generate the sliding surface, which was consistent with the results from the inner displacements presented in Fig. 16. And utilized the tested soil pressures in the model slope to generate the sliding force and anti-sliding force thus deriving the safety factor of the prototype slope. Combined with Table 4, the treating effect of this slope was evaluated.

Fig. 17 shows the critical sliding surface at the end of the fifth year generated by Morgenstern-Price method in Geo-studio. Clearly, the sliding surface went between S1 and S2, which is strictly consistent with the model test results.

In the field of geotechnical engineering, the safety factor of slope is defined as the ratio of the total resisting moment to the total sliding moment, as Eq. (7).

Figure 17. The critical sliding surface generated by Geo-studio. S1, S2, S3, S4 denote displacement markers identical with Fig. 7.

$$K = \frac{\sum M_{resisting}}{\sum M_{sliding}} \quad (7)$$

Here, $M_{resisting}$ is the resisting moment of the sliding slice defined as the resisting force multiplied by the arm, and $M_{sliding}$ is the sliding moment of the sliding slice defined as the sliding force multiplied by the arm. In the Ordinary method of the limited equilibrium method, the arm of the moments is identically the sliding arc radius, while the inter-slice forces are completely neglected. Adopted this concept, the safety factor of the slope can be derived as:

$$K = \frac{\sum F_{resisting}}{\sum F_{sliding}} \quad (8)$$

Here, $F_{resisting}$ is the resisting force of the sliding slice, and $F_{sliding}$ is the sliding force of the sliding slice. Considering the suction upon the sliding surface, Wang et al.⁵⁸ derived the resisting force and the sliding force of the sliding slice as:

$$F_{resisting} = W_i \cos \alpha_i \tan \varphi + \frac{(u_s \tan \varphi^b + c)B}{\cos \alpha_i} \quad (9)$$

$$F_{sliding} = W_i \sin \alpha_i \quad (10)$$

Here, W_i is the gravity of the sliding slice i ; α_i is the bottom inclination angle of the sliding slice i ; c , φ and u_s are the cohesion, internal friction angle and matric suction at the bottom of sliding slices respectively; φ^b is the suction internal friction angle, which is valuated as $\varphi/2$ according to the research of Fredlund⁵²; and B is the width of the sliding slice. Incorporating the vertical soil pressures derived from the model test, W_i can be derived as:

$$W_i = \sigma_y \cdot B \quad (11)$$

Here, σ_y is the vertical soil pressure at the bottom of the sliding slice, which is adopted from the model test. Substituting Eq. (11) into Eq. (9) and (10), the results of which were then substituted into Eq. (8), we ultimately derived the formula of the safety factor as:

$$K = \frac{\sum \left[\sigma_y B \cos \alpha_i \tan \varphi + \frac{(u_s \tan \varphi^b + c)B}{\cos \alpha_i} \right]}{\sum [\sigma_y B \sin \alpha_i]} \quad (12)$$

To calculate the safety factor of the current slope, the sliding mass must be divided in to slices firstly. Simply, the sliding mass of this project was divided into 6 slices, as illustrated in Fig. 18. To get the soil pressures and the suctions at the bottoms of the slices, we assumed that the positions with the same burying depth have the identical soil pressure and suction in this slope. Then, the soil pressures and suctions at the bottoms of slices could be derived by interpolating the values of the adjacent depths. However, the soil pressures and pore water pressures at the toes of the three grades of slope were different. Thus, the measured data of the points beneath the slope toes were used to derived the soil pressures or suctions of the slices nearby, if it could be. By this way, the soil pressures and suctions at the bottoms of the six sliding slices were derived as Table 5 shows.

Figure 18. Slice division profile of the sliding mass. The numbers 1 to 6 denote the slice numbers. The bottom of each slice is simplified as straight line.

Slice number	1	2	3	4	5	6
σ_v (kPa)	19.45	9.46	6.63	5.68	12.30	19.90
u_s (kPa)	96.45	52.66	37.60	32.58	67.72	85.42
B (m)	2.6	3.8	3.7	2.8	3.7	3.5
α_i (°)	61	47.1	37.1	25.7	18	8.4
$F_{resisting}$ (kN)	219.01	166.92	121.12	78.29	143.77	160.67
$F_{sliding}$ (kN)	43.9959	26.24204	14.7186	6.83872	14.1081	10.4475

Table 5. Sliding slice information. The information in this table is for each slice bottom.

According to Eq. (8), the safety factor of the should be:

$$\begin{aligned}
 K &= \frac{\sum F_{resisting}}{\sum F_{sliding}} \\
 &= \frac{889.79 \text{ (kN)}}{116.35 \text{ (kN)}} \\
 &= 7.65
 \end{aligned} \tag{13}$$

It is clear that the safety factor derived from the proposed method combining with the measured data of model test was largely greater than the critical value 1.2, indicating the conservative designing method of the loess slope treating project. By inferring, it should be due to the underestimation of the soil strength in the slope designing method of China, which didn't incorporate the matric suctions in the unsaturated condition. However, the derived safety factor here shows a consistent result with that of the displacements of the model slope. Referring to Table 4, the treating effect of the prototype project should be very good preliminarily.

Evaluating Results and Post-evaluation Frame. Synthesizing the above evaluating results, the slope was globally stable having no further sliding trend, with relatively huge destruction caused by rainwater scouring. From that, the collapse of the prototype slope was a local destruction. As a result, the treating effect of the project should be not bad, reasonably.

Lastly, it is noteworthy that the post-evaluation frame formed in this paper are valuable to other slope treating projects incorporating slope-cutting. Thus, this post-evaluation frame is detailed in Fig. 19.

573
574 **Figure 19.** Post-evaluation frame for slopes treated by slope-cutting.

575 Conclusions

576 To reveal the influence of long-term rainfall on the loess slopes in Yan'an city of Shaanxi province treated by
577 slope-cutting, filed investigations and indoor model tests were conducted, the results of which were employed to perform
578 the post-evaluation of the loess slope treated by slope-cutting. The following conclusions were obtained:

579 (1) The rainwater runoff induces serious scouring to the slope surface with the main patterns of gullies and shallow
580 sliding. For the critical reason of consolidation of the shallow layers in the intervals of rainfall, the scouring effect of
581 rainwater runoff becomes weaker and weaker with time, leading to the relative steady stage after the first year of rainfall.

582 (2) The rainwater preferentially penetrates the platforms with large rates for the rainwater accumulated there. During
583 the simulating duration, the wetting front advancing rate decreased gradually. 5.2 hours after the second rainfall of the
584 third year, the whole model slope was wetted with a wetting front advancing rate of about 0.003 cm/min.

585 (3) Though the whole model slope was wetted, the pore water pressure values of the prescribed points in the model test
586 were negative over the test duration, indicating the unsaturated condition. That is, the wetting front can not be deemed as
587 the boundary of the saturated and unsaturated area in the rainfall process. And the infiltrated rainwater preferentially
588 accumulates at the toe of the first grade of slope.

589 (4) Induced by the in-situ stress release, the soil pressures in the model slope drastically decreased in the first years of
590 rainfall, especially for the deeper points in the model slope.

591 (5) The horizontal displacements of the prescribed points in the model slope increased significantly in the first year of
592 rainfall, with a decreasing rate. After the second year of rainfall, the horizontal displacements had no regular increment,
593 implying the stable state of the slope.

594 (6) The above results were employed to perform the post-evaluation of the slope-cutting treating project of the loess
595 slope, with a new method proposed to calculate the safety factor of the slope. Moreover, a frame for the post-evaluation of
596 slope-cutting treating loess slope under long-term rainfall (see Fig. 19) was structured. This evaluation frame can be
597 valuable to other slope treating projects.

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732 Author contributions

733 G.L. conceived, designed and coordinated the research and revised the original manuscript. S.X. conducted the indoor experiments and
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740 **Competing interests**

741 The authors declare no competing interests.

742 **Additional information**

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