

Chapter 14

Stability Assessment and Stabilizing Approaches for the Majiagou Landslide, Undergoing the Effects of Water Level Fluctuation in the Three Gorges Reservoir Area

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Abstract The Majiagou landslide is located on the left side of the Zhaxi-he River, a tributary of the Yangtze River. There is a large old landslide there originally, on which three secondary landslides were formed. The Majiagou landslide is one of the secondary ones. After the water level of the Three Gorges Reservoir rose from 95 to 135 m in June 2003, the Majiagou landslide occurred including a 20 m-long, 3–5 cm wide, (locally 10 cm-wide) fissure at its back that occurred in a time-frame of 3 months. The implication is that the reservoir impounding reactivated the landslide. The slide mass of the Majiagou landslide is composed of a thin layer of silty clay on the top with low permeability and a thick layer of angular pebbles as the main portion beneath the silt clay which has high permeability. The rate of water level fluctuation is between 0.6 and 4.0 m/d with regard to the altitude between 145 and 175 m in the reservoir during its normal operational state. Under these conditions, the FEM method was applied to simulate the groundwater changes in the slide mass, coinciding with the reservoir water level fluctuation. The results suggest that the groundwater level almost changes with the reservoir water level simultaneously within the slide mass, which means that the groundwater gradient is very gentle. Therefore, the effect of the reservoir water level fluctuation on the landslide stability is mainly by the action of buoyant force. Without considering dam failure, the seepage force is so little as to be negligible. The stability of the landslide in cases of different water levels is then calculated using the Morgenstern-Price method, and the results show that the factors of safety decrease with the water level rise. As the water level increases to 165 m, the factors of safety are at the minimum value, increasing with the water level rise. The minimum value reflects that the landslide is in a critical state, so stabilizing design was applied using stabilizing piles as well as a water drainage system. The project was completed in early 2006, and the water level has risen to 156 m in October 2006. Monitoring data illustrated that there is no further deformation of the landslide and the stabilizing piles, so the stabilizing work is effective.

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Introduction

The Majiagou landslide is located on the left side, 2.1 km up to the outlet of the Zhaxi-he River, a tributary of the Yangtze River. The geographical position is $31^{\circ}01'08''$ – $31^{\circ}01'17''$ north latitude, $110^{\circ}41'48''$ – $110^{\circ}42'10''$ east longitude, and administratively, belongs to Group 8, Penjiapo Village, Guizhou Town, Zigui County, Hubei Province, China (Fig. 14.1). It is one of the secondary landslides which occurred on a large old landslide. A 20 m long, 3–5 cm wide (locally 10 cm wide fissure) developed on the back of the landslide in the 3 months as the water level of the Three Gorges Reservoir rising from 95 to 135 m. After that, the fissure stabilized, with no further opening. This situation indicates that the landslide is sensitive to the water level change. If the water level changes again, there is the possibility of reactivation. There are civilians and farmland as well as an economic forest on the landslide. The census and economic investigation suggested that 47 families including 132 persons are living on the landslide and 320 Mu of farmland and orange forest on it are the critical economic base of the villages. The oranges there are the well-known local products. In 1996, the vice-chairman of the nation at the time, Jintao Hu visited Penjiapo village, located on the landslide to encourage the orange planting (Fig. 14.2). It is estimated that if the landslide fails, it would cause losses of 34,220 thousands Yuan RMB due to direct effects, and 14,390 Yuan RMB in indirect losses, and deaths and injuries could not be prevented. Thus, the deformation of the landslide attracted the attention of the government officials and



Fig. 14.1 Location of Majiagou landslide

Fig. 14.2 The vice-chairman of the nation at the time Mr. Jingtao Hu visited Penjiapo village in 1996



related specialists. They ordered an investigation and the application of control measures, as needed. The front of the landslide is saturated under water as the water level of the Three Gorges Reservoir changes between 145 and 175 m with 30 m in fluctuation. The key problem is the effect of the water level changes on the landslide stability. The worst case scenario for the landslide stability affected by the water level change should be considered and as the base for stabilizing.

The effect of water level change on landslide stability is related to the material components and permeability of the slide mass and the shape of the slide zone as well as the rate and the extent of the water level fluctuation. The authors had carried out the whole process of the landslide investigation and stabilizing design. By field investigation and exploration, the engineering geological features of the landslide were clarified; the physical–mechanical parameters and the permeability of the slip mass, slip zone and the slip bed were tested in lab and in situ. The two-dimensional unstable flow model of FEM method was used to simulate the regularities of the groundwater level change affected by the reservoir water level. The Morgenstern–Price method was applied to calculate the landslide stability in combination with the results of water level simulation to achieve the correlation between the water levels and the factors of safety. The worst case was used to assess the landslide stability and design for the stabilizing plan. The plan was completed in early 2006, and the water level rose to 156 m in October 2006. Monitoring data illustrated that there is no further deformation of the landslide and the stabilizing piles, so the stabilizing work is effective.

Engineering Geologic Characteristics of the Majiagou Landslide

The Geologic Background of the Landslide

Around the landslide outcrops the Suining group, which stratigraphically is Upper Jurassic series composed of grey feldspar quartz sandstone and silty mudstone,

which is interbedded. The bedrock has the attitude of $270\text{--}290^\circ$ in dip direction and of $5\text{--}30^\circ$ in dip angle, close to the major orientation of the landslide movement. The Majiagou landslide is located in the front side of a large old landslide. The large landslide is a bedrock slide slipping along the rock bedding and covers all the left side of the lower part of the Majiagou Brook which is a tributary of the Zhaxi-he River. It is located on the left bank, and the slide mass distributes in the elevation between 130 and 300 m. It covers an area of 5 km^2 and has a size of up to 2 billion m^3 . A large reversed platform, takes up an area of about 1.5 km^2 and is formed on the top of the slide, on which a small dam was built to store water (Figs. 14.3 and 14.4). The slide mass of the large landslide is mostly composed of large sandstone blocks filled with detritus of purplish red mudstone. A 3–5 m thick layer of brown-red clay containing rock blocks covers the top of the slide mass, which is a residual and diluvia deposit and may have been deposited after the landslide formed, and therefore it can be concluded that the old landslide happened at least before the Holocene.

In the front side of the old landslide, under the margin of the top platform, three secondary landslides developed. Two of these secondary landslides on the upper reach of the Majiagou Brook lie above the water level of the Three Gorges Reservoir, and their toe areas are obstructed by the opposite slope of the right side of the Majiagou Brook, with no space for further slippage. As a result, it is empirically recognized that these secondary landslides are of a stable state and have experienced no disturbance. The Majiagou landslide is the one on the outlet of the Majiagou Brook and the front part of it extended into Zhaxi-he River. In 2003, the Three Gorges Reservoir impounded water up to 135 m in elevation, and then a long fissure appeared in the back of the slide. If the slide fails, not only will it bring great damage to local residents and properties but also the wave it creates will run into the Yangtze River along the Zhaxi-he River to threaten the safety of navigation.

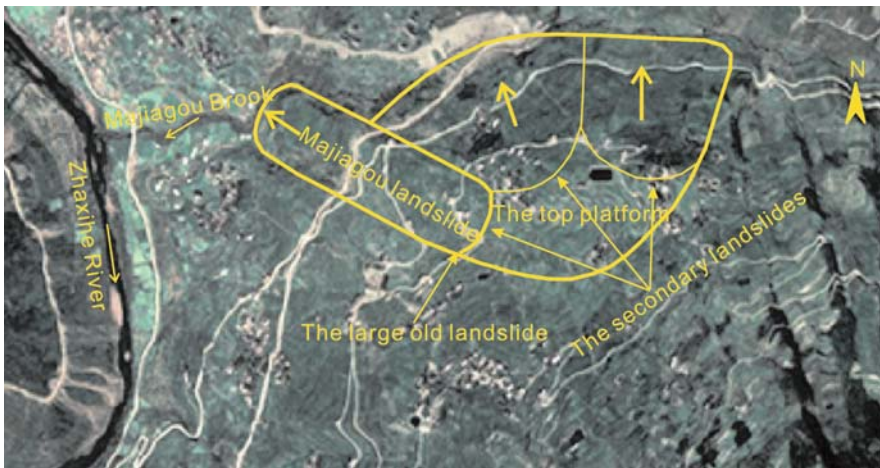


Fig. 14.3 Satellite image of the Majiagou landslide

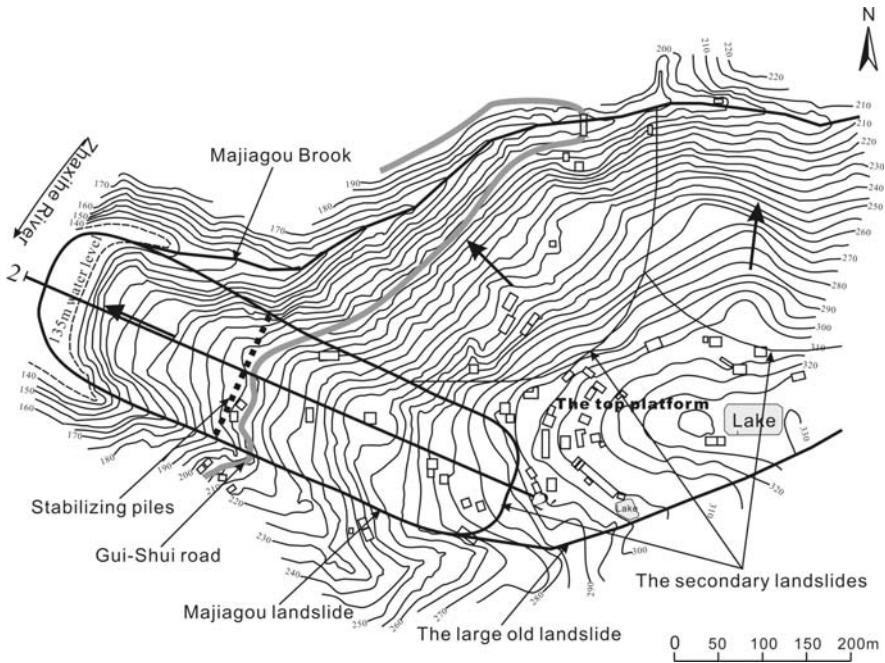


Fig. 14.4 The plan map of the Majiagou landslide

The Geomorphic Form of the Landslide

The Majiagou landslide has a long-narrow tongue-shaped form in plane view, with a movement direction of 290° NWW. The south and north sides of the landslide are bordered by the two brooks and the back is bordered by the new formed fissure. The elevation of the front and the back is 130 and 280 m, respectively, with a height difference of 150 m (Fig. 14.5). The landslide has a gradient of 20–25° on the surface,

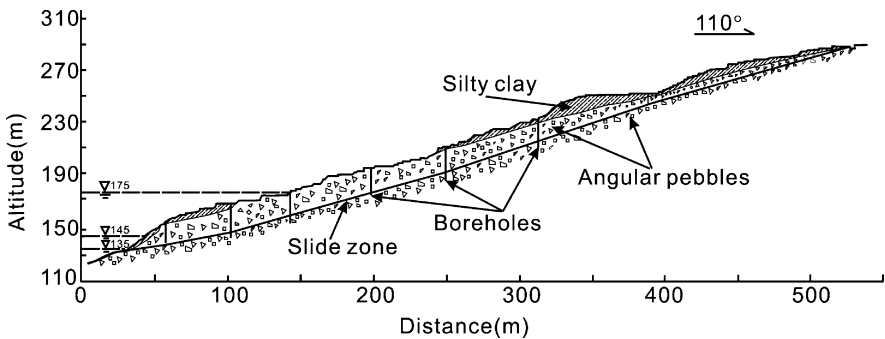


Fig. 14.5 The main section of the Majiagou landslide

a width of 150 m, and a length of 538 m. The front portion has submerged into the Zhaxi-he River whose water level is 135 m at the time. Because of wave erosion, a 30–45 m high scarp has formed above the water table. The mean gradient of the slide surface is 18° . According to the results of the boreholes on the landslide, the slide mass has the thickness of between 17.5 and 8.9 and 13.2 m on an average, has the surface area of $9.68 \times 10^4 \text{ m}^2$ in plan, and has a size of $127.8 \times 10^4 \text{ m}^3$.

The Material Components of the Landslide

The landslide mass is composed of two types of materials. The top 3–5 m is a layer of residual silty clay containing rock blocks. The ratio of rock blocks to soil is between 6:4 and 7:3. The silty clay is brown red, wet, and plastic to hard plastic. The components of the contained rock blocks are composed of feldspar quartz sandstones, shale and siltstones, silty mudstones, etc. The size of the blocks is generally 0.5–5 cm, the maximum ones up to 20 cm in diameter and in angular shape. The main portion of the slide mass lying under the silty clay is full of angular pebbles and composed of large sandstone blocks filled with weathered mudstone detritus. Some of the detritus has broken into a state that is like soil, with a ratio of rock blocks to soil of from 8:2 to 9:1, with a high proportion of rock blocks. This layer has a thickness of 8–11 m, of which the size of rock blocks is generally 1–2 m, with the largest up to 7–8 m. The rock blocks contained are of angular or sub-angular shape, in a filling of soil. The landslide is of a loose structure and high water permeability.

The slide zone revealed by boreholes is composed of muddy shale and crushed sandstone including a small amount of sandstone pieces. The slide zone appears purple and grey in color, saturated and soft plastic in state. It can also be observed in the investigation pits in the southwest front, and the investigation draft in the central part of the slide, there occurs a film of crushed muddy materials containing polished faces and slickensides. The thickness of the slide zone is 0.5–0.8 m. There is a layer of perched water within the slide zone. The perched water can be observed in the draft as overflow, in the boreholes as hanging water, and in the east side as down springs. Because the material of the slide mass and the slide bed are similar, the perched water is an important indicator of the slide zone.

The material of the slide-bed is also angular pebbles, similar to that of the slide mass. The main components are the sandstone rock blocks filled with the weathered mudstone detritus and fine soil. All the boreholes on the landslide did not reveal intact bedrock, which means the old landslide is a large one with a deep slide zone, and the Majiagou landslide is only a little shallow part at its front, reactivated by the reservoir water.

The Hydrogeological Characteristics of the Landslide

The surface water system related to the landslide is that of the Zaxihe River in front, the Majiagou brook in the east side and another little brook in the west side. The two brooks have permanent water flows which cut down below the slip zone of

the landslide in their downstream reach, so the brooks have no essential effect on the landslide stability. However, the Zhaxihe River has a prominent effect on the landslide as its water level changes with the reservoir.

The grounder water in the landslide is mainly the perched pore water on the slide zone which flows out as springs.

Based on the permeability tests on the ground surface and in the boreholes, the silt clay on the top portion of the slip mass has the permeability coefficient of 1.38×10^{-5} cm/s, being a weak permeability; while the angular pebbles below the silt clay has that of 1.52–0.006 cm/s, being of a high permeability. The slip zone has the permeability equivalent to the silt clay which is a relatively water-tight layer. The slip bed has the permeability equivalent to the angular pebbles with high permeability. All the boreholes did not reveal a phreatic level in the landslide.

The landslide is in the Three Gorges rainstorm district where there is frequent rainfall. The monthly mean precipitation by years is 1066.92 mm, and the maximum annual precipitation is 430.6 mm (1963), while the minimum is 733 mm (1966). The rainfall is concentrated from April to October, and monthly mean precipitation is 87.2–147.2 mm in these periods; from May to September it is over 130 mm, which is 67.2% of the total precipitation in a year. The rain season is popular in storm rain and long-term rains, which generally occur in spring and autumn. The groundwater is supplied by penetration of rain water. Because of annual rainfalls, the silt clays of the slide mass and the slip zone are annually saturated, but the angular pebbles have high permeability because of their more and large porosities. With the rainfall occurring, the slip mass and the slip bed usually experience pore water penetrating down instantaneously which is obstructed by the relative water-tight slip zone to form perched water. The phreatic level is deep seated; therefore, the reservoir impounding may have little influence on the water content of the slip zone, but can affect the water dynamic conditions.

Physical and Mechanical Properties of the Soil of the Landslide

The physical and mechanical properties including natural density and saturated density of the slide mass, the shear strength of the slide zone, and the permeability of the slide mass, the slide zone, and the slide bed need to be determined for the landslide stability assessment.

Density of the Slide Mass

As the soil contains too many irregular rock blocks, it cannot be sampled by the ring cut; therefore, the natural density and saturated density of the slide mass are obtained by in situ test.

The process is to excavate a cubic pit of about 50 cm × 50 cm × 50 cm in size and to weigh the soil being cut package by package during excavation. As the pit is completed, its bottom and walls are covered with thin watertight film, as it is filled up with water, the water volume is gauged bucket by bucket. The natural density is

Table 14.1 The test results of the densities of the slide mass

Soil type	Test number	Location	Natural density (kN/m ³)	Saturated density (kN/m ³)
Silty clay	R06	TC02	20.66	21.47
	R03	TC11	23.02	23.59
	R05	TC18	19.40	19.80
	average		21.0	21.6
Angular pebbles	R04	TC01	20.80	21.40
	R02	TC04	21.54	22.29
	R07	TC05	21.54	22.56
	R01	TC15	21.94	22.99
	R08	TC16	21.98	22.85
	average		21.60	22.40

equal to the quotient of the cumulative soil weight to the cumulative water volume. For measuring the saturated density, water is allowed to penetrate to the area tested to make the soil fully saturated. The next operation is the same as that mentioned above. Using this method, the results of the soil densities were achieved and are shown in Table 14.1.

As shown, the difference between the natural density and the saturated density is small, which indicates that the soil of the slide mass is close to a naturally saturated state. The silty clay on the top layer contains irregular rock blocks and fragments, so the deviation of the density is very high. But this layer is relatively thin; as a result, it has little effect on the stability assessment.

Shear Strength of the Slide Zone

Shear strength of the slide zone is based on a laboratory shear test with the soil samples collected in the investigation pits. The laboratory test result statistics are shown in Table 14.2.

Because the samples contain more or less fragments, both c and φ have high variation. We chose the parameter for calculation mainly based on residual values and our experiences regarding the landslides in this area.

Table 14.2 Statistics of the saturated direct shearing test results for the slide zone

Strength indexes	Peak strength		Residual strength	
	$\varphi/^\circ$	c/kPa	$\varphi/^\circ$	c/kPa
Count	6	6	6	6
Range	5.2–30.0	6.8–21.0	2.7–25.0	5.1–18.5
average	14.6	18.0	12.4	13.4
Standard deviation	7.6	8.6	5.7	5.8
Coefficient of variation	0.52	0.48	0.46	0.43

Permeability of the Soils of the Landslide

To determine the permeability of the soils in the landslide, six groups of water injection tests were completed in the boreholes. The material of the landslide can be recognized as two main types of silty clay and angular pebbles, so water injection tests were done, one group in the former and five groups in the latter, respectively. The results are shown in Table 14.3.

It is indicated that the silty clay has a permeability coefficient of 1.38×10^{-5} cm/s, which is a weak permeable soil, while the angular pebbles has that of $1.52\text{--}6.00 \times 10^{-3}$ cm/s, which is a strong permeable soil.

Table 14.3 Water injection test results of the soils on the landslide

Number	Borehole number	Test depth	Soil type	Permeability coefficient	
				(cm/s)	(m/d)
1	ZK2	10.0–10.3	Angular pebbles (Fragments)	0.025	21.60
2	ZK4	0.5–0.67	Silty clay	1.38×10^{-5}	1.19×10^{-2}
3	ZK6	10.4–10.6	Angular pebbles (weathering mudstone)	0.006	5.18
4	ZK10	4.3–4.5	Angular pebbles (Rock blocks)	0.064	55.30
5	ZK1	6.4–6.6	Angular pebbles (Rock blocks)	1.520	1313.28
6	ZK11	6.1–6.3	Angular pebbles (Rock blocks)	0.500	432.00

Effect on the Ground Water by the Reservoir Water Level Fluctuation

The groundwater level in the landslide changes correspondingly with the reservoir water level fluctuation, and therefore movement of the groundwater level impacts the stability of the landslide. Generally the change of groundwater rate falls behind that of the reservoir water level. The dynamic process is related to the soil permeability of the landslide and the rate of the reservoir water level change. Here, we first study the effect of soil permeability and the rate of the water level change on the groundwater. The problem can be generalized as a two-dimensional unsteady flow model. Owing to the high water content of the landslide soil, the unsaturated soil characteristics are not considered, and the permeability is assumed to be homogeneous. The basic equation for seepage flow is expressed as

$$k \left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} \right) = m_w \gamma_w \frac{\partial H}{\partial t} - Q \quad (14.1)$$

where H is the total water head, k is permeability coefficient, Q is flow quantity, m_v is flow-resistant coefficients, and γ_w is water unit weight.

It is difficult to deduce the analytical solution of Eq. (14.1), therefore the finite element method is applied to simulate the groundwater heads for different of reservoir water levels. From the results, the zero water head line is recognized as the phreatic line. The stabilities of the landslide in different reservoir water levels are calculated with the phreatic lines. The meshes of the finite element model on the main section of the Majiagou landslide are shown in Fig. 14.6.

The parameters of the soil permeability and the reservoir water level change need to be selected. In all the cases, it is usually the worst conditions combinations which are taken for simulation. Thus for the permeability, the lowest, and for the rate of water level change, the highest should be available. As shown in Table 14.3, the permeability coefficient is taken as 10^{-5} cm/s.

The water level adjustment schedule of the Three Gorges Reservoir in its normal operational state is as follows (GDGH 2004a). From the beginning to the end of May, the water level in front of the dam drops from 175 to 155 m with the rate of less than 1 m/d, average 0.67 m/d; from 1 to 10 June, water level drops from 155 to 145 m with the rate of 1.0 m/d. The threshold limit for water level is 145 m which is maintained to prevent flooding during the flood season which extends from the middle of June to the end of September. If the flood occurs once per 5, 20, 100 and 1,000 years, the water level would rise to 147.2, 157.5, 166.7 and 175.0 m, respectively. After a flood peak, the reservoir water level in front of the dam drops rapidly to about 145 m, the threshold limit level for preparing for another flood. The rising rate of the water level is up to 3–4 m/d as the 100 and 1,000 years flood occurrence during flood season. The water level rises from 145 to 175 m during the beginning of September to the end of October, and a 175 m level is maintained until the beginning of the next May. The water level in front of the Three Gorges Dam fluctuates between 145 and 175 m, for a range of 30 m (Fig. 14.7). The schedule for the reservoir normal water level management is shown in Fig. 14.6. It is shown that the rate of water level change is between 0.6 and 4.0 m/d, so the highest rate of 4.0 m/d is used for simulation.

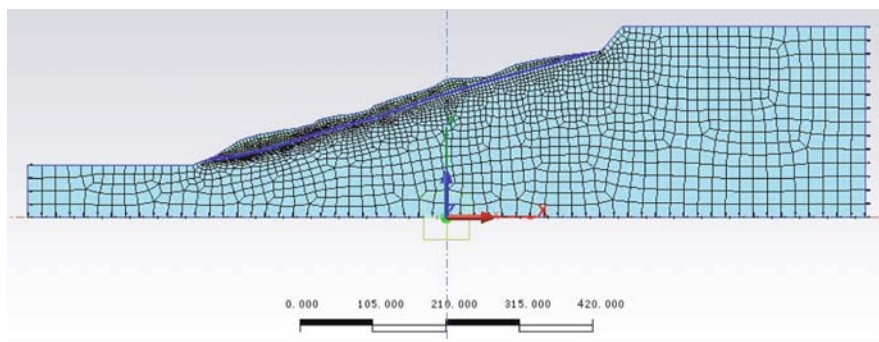


Fig. 14.6 The meshes of the model on the main section for groundwater simulation

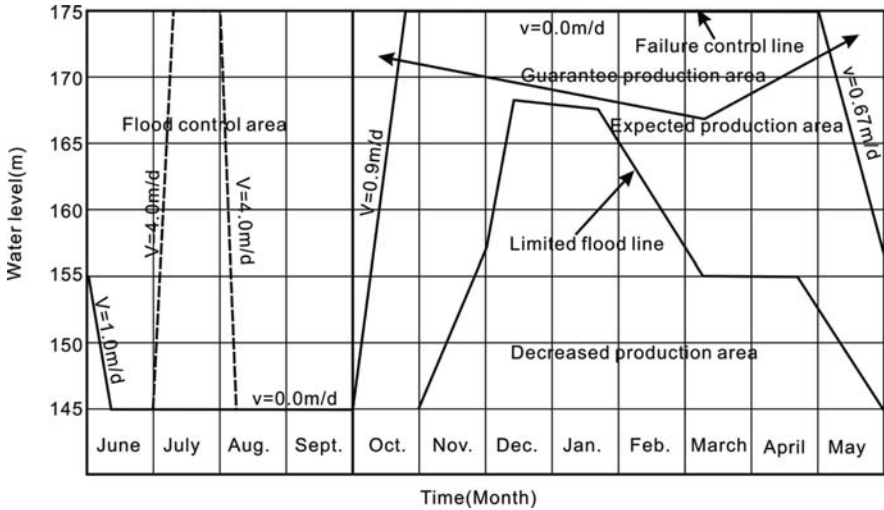
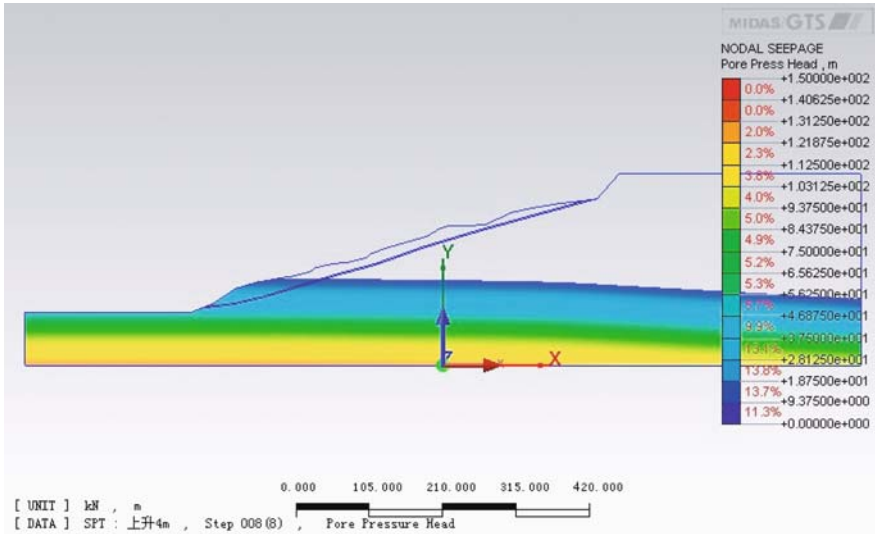


Fig. 14.7 The water level management schedule of the Three Gorges reservoir in a normal operational state

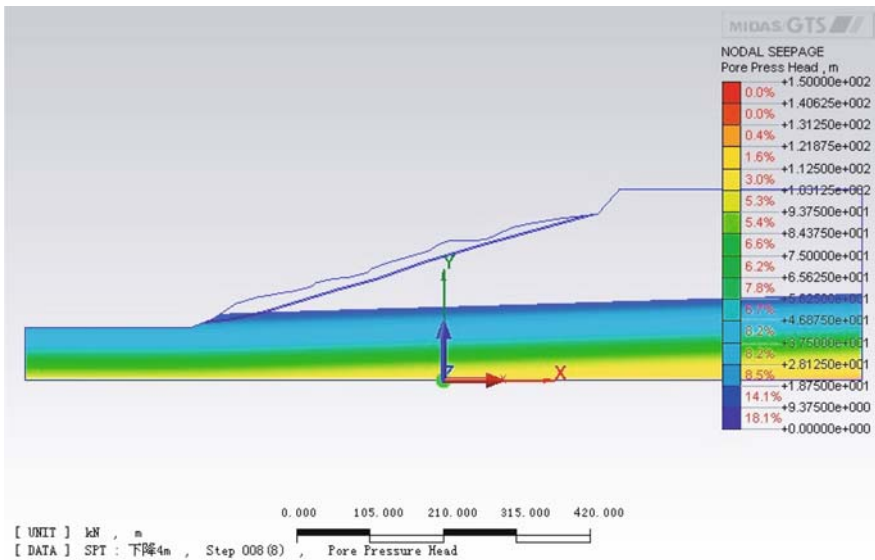
It is now supposed that the model is homogenous in permeability and the permeability coefficient is 10^{-5} cm/s and the rate of water level change is 4 m/d. First, the water level of the reservoir starts from 145 m to simulate the water rising process. At the moment, the groundwater level is horizontal. The water level rising from 145 to 175 m requires 7.5 days, so the simulated phreatic levels are given by the intervals of one day and the last half day which are equivalent to the reservoir water level intervals of 4 m and the last 2 m. Then the initial water level of the reservoir starts from 175 m to simulate the draw-down process. The groundwater level is horizontal at the beginning, too. The simulated phreatic levels are given by the intervals of the first half day and one day, which are equivalent to the reservoir water level intervals of the first 2 and 4 m in the after. Figure 14.8 (a) and (b) shows the simulated results where the reservoir water level just rises to 175 from 145 m and drops down to 145 from 175 m, respectively. Figure 14.9 shows all the simulated phreatic lines up and down.

It is shown that the groundwater level rises with the reservoir water level synchronously, but the phreatic line tends to dip to the inner of the slope, the higher the reservoir water level, the greater the gradient. The highest gradient presents at the reservoir water level occurs at the 175 m level. To each phreatic line, the groundwater level is gentle outside, while it becomes steeper to the inner side, as shown in Fig. 14.8(a). The phreatic line becomes flat when the reservoir water level keeps static at 175 m.

On the contrary, with the reservoir water falling, the groundwater level draws down synchronously either. The phreatic line inclines to the outer of the slope, the lower the reservoir water level, the higher the gradient. The highest gradient appears at the reservoir water level just getting at 145 m. To each phreatic line,



(a) For $k = 10^{-5}$ cm/s, $v = 4$ m/d up rising



(b) For $k = 10^{-5}$ cm/s, $v = 4$ m/d down down

Fig. 14.8 The simulated results of the reservoir water level effect on groundwater

the groundwater level is gentle outside, while becomes steeper to the inner side, as shown in the Fig. 14.8(b). When the reservoir water level stays steadily at 145 m, the phreatic line becomes flat.

Figure 14.8 shows that, in all cases, for both rising and falling of the reservoir water level, the phreatic line in the range of the slide mass is very flat, and even the

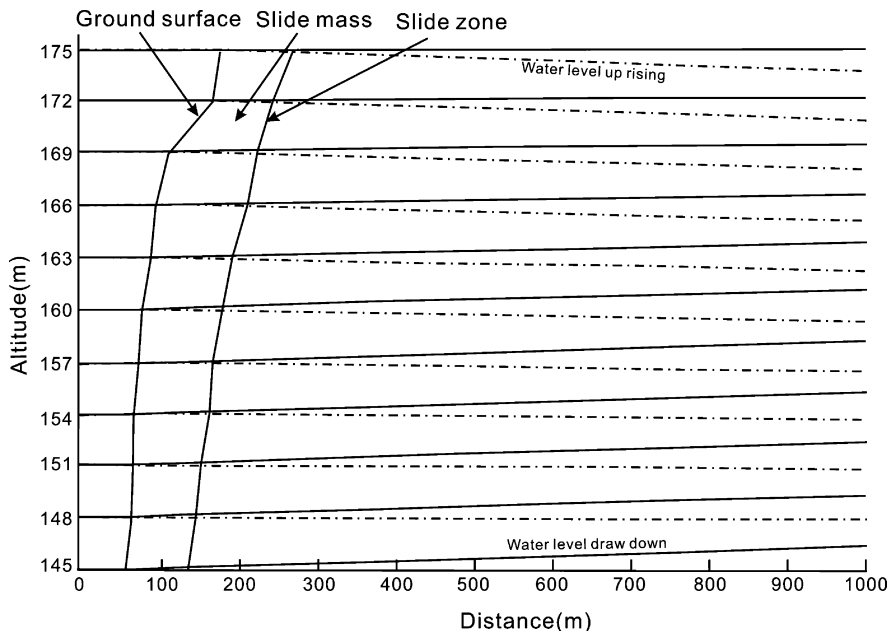


Fig. 14.9 The simulation results of the water level, up and down movement for $k=10^{-5}$ cm/s, $v=4$ m/d.

lines for rising and falling are overlapped. Thus the hydraulic gradient is as little as that of the seepage force and may be negligible.

The simulated cases above are in the worst conditions, for other possible combined conditions, such as larger in permeability coefficient and slower in the water level change, it is easy to recognize that the phreatic line must be gentler than that in the above simulation.

Therefore, the simulated results show that the effect of the reservoir water level on the groundwater of the landslide is mainly due to the fluctuation of the phreatic level, so the action of the groundwater on the slide mass is mainly the buoyant force rather than the seepage force.

Stability Assessment of the Landslide Undergoing the Reservoir Water Level Change

Based on the test results and the experience with the materials of the landslide, the parameters applied in the stability calculation are accepted as follows:

The density of the slide mass:

Natural unit weight:

The silty clay is 21.0 kN/m^3 , the angular pebbles is 21.6 kN/m^3

Saturated unit weight:

The silty clay is 21.6 kN/m^3 , the angular pebbles is 22.4 kN/m^3

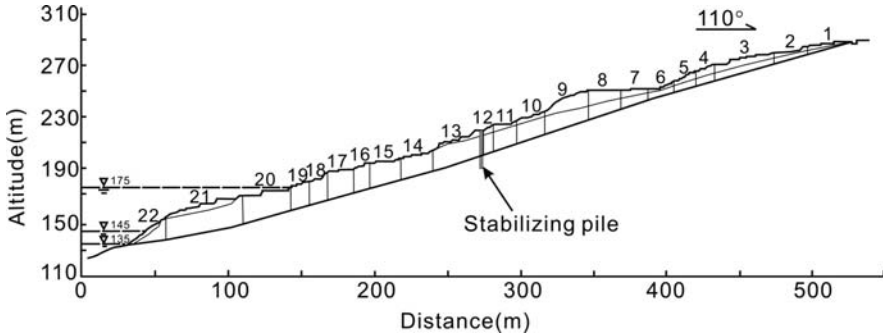


Fig. 14.10 Slices for the stability calculation

The shear strength parameters of the slide zone in effective stress value are

$$c = 10 \text{ kPa}, \varphi = 14^\circ$$

The Morgenstern–Price method (Morgenstern and Price 1965), a more perfect method at present to consider both the force equilibrium and the moment equilibrium, is applied. Under the coordinate system in Fig. 14.10, the force equilibrium and the moment equilibrium are expressed as Eqs. (14.2) and (14.3), respectively.

$$\left(1 - \frac{dy}{dx} \frac{tg\phi'}{F}\right) \frac{dE}{dx} - \left(\frac{dy}{dx} + \frac{tg\phi'}{F}\right) \frac{dX}{dx} = - \left(\frac{dy}{dx} + \frac{tg\phi'}{F}\right) \frac{dW}{dx} - \frac{c' - utg\phi'}{F} \left[1 + \left(\frac{dy}{dx}\right)^2\right] \tag{14.2}$$

$$X = y \frac{dE}{dx} - \frac{d(Ey_E)}{dx} \tag{14.3}$$

where E is the horizontal thrust force between the slices, X is the vertical shear force between the slices, c' is the effective cohesion, ϕ' is the effective friction angle, y is the function of the slide surface, y_E is the function of the horizontal force position, and u is the pore water pressure.

The method assumes

$$X = \lambda f(x)E$$

where λ is a constant and $f(x)$ is a given function.

And let

$$\begin{aligned}y &= Ax + B \\ \frac{dW}{dx} &= px + q \\ f(x) &= kx + m.\end{aligned}$$

Since

$$\begin{aligned}\frac{dy}{dx} &= A \\ X &= \lambda(kx + m)E \\ \frac{dX}{dx} &= \lambda kE + \lambda(kx + m)\frac{dE}{dx}\end{aligned}\tag{14.4}$$

Then, Eq. (14.2) becomes

$$\begin{aligned}\left[-\lambda k \left(A + \frac{tg\phi'}{F}\right)x + 1 - A\frac{tg\phi'}{F} - \lambda m \left(A + \frac{tg\phi'}{F}\right)\right] \frac{dE}{dx} - \lambda k \left(A + \frac{tg\phi'}{F}\right) E \\ = -p \left(A + \frac{tg\phi'}{F}\right)x - (1 + A^2)\frac{c' - utg\phi'}{F} - q \left(A + \frac{tg\phi'}{F}\right)\end{aligned}$$

Let

$$\begin{aligned}K &= -\lambda k \left(A + \frac{tg\phi'}{F}\right) \\ L &= 1 - A\frac{tg\phi'}{F} - \lambda m \left(A + \frac{tg\phi'}{F}\right) \\ N &= -p \left(A + \frac{tg\phi'}{F}\right) \\ P &= -(1 + A^2)\frac{c' - utg\phi'}{F} - q \left(A + \frac{tg\phi'}{F}\right)\end{aligned}$$

Thus

$$\left(Kx + L\right)\frac{dE}{dx} + KE = Nx + P\tag{14.5}$$

This equation is available in the domain $[x_i, x_{i+1}]$, the boundary conditions are

$$\begin{aligned}E(x_i) &= E_i \\ E(x_{i+1}) &= E_{i+1}\end{aligned}$$

The solution of Eq. (14.5) is

$$E(x) = \frac{1}{(Kx + L)} \left(\frac{1}{2}Nx^2 + Px + C \right) \tag{14.6}$$

where C is a constant.

The x in the above equations can be defined as in local coordinate. Taking the left boundary of each slice as the origin of x axis, then $C = E_iL$ when $x = 0$, so

$$E_x = \frac{1}{(Kx + L)} \left(E_{i-1}L + \frac{1}{2}Nx^2 + Px \right). \tag{14.7}$$

The Eq. (14.3) can be changed as the follow:

$$d[E(y_E - y)] = \left(-X - E \frac{dy}{dx} \right) dx.$$

After integrating the above equation, we can get

$$M_x = M_{i-1} - \int_0^x \left(X + E \frac{dz}{dx} \right) dx.$$

Replace X and E in Eqs. (14.4) and (14.6), we have

$$M_x = M_{i-1} - \int_0^x \frac{\lambda x/Kx + (\lambda m + A)/K}{x + L/K} \left(E_iL + \frac{1}{2}Nx^2 + Px \right) dx$$

Let

$$\begin{aligned} G &= -\lambda k/K \\ H &= -(\lambda m + A)/K \\ W &= L/K. \end{aligned}$$

Then

$$M_x = M_{i-1} + \int_0^x \frac{Gx + H}{x + W} \left(E_{i-1}L + \frac{1}{2}Nx^2 + Px \right) dx$$

And

$$\begin{aligned} M_x = M_{i-1} + \int_0^x \frac{1}{x + W} \left[\frac{1}{2}GNx^3 + (GP + \frac{1}{2}HN)x^2 \right. \\ \left. + (GLE_{i-1} + HP)x + HLE_{i-1} \right] dx. \end{aligned}$$

Let

$$\begin{aligned}
 R1 &= HLE_{i-1} \\
 G1 &= \int_0^x \frac{1}{x+W} dx = \ln |(x+W)/W| \\
 R2 &= GLE_{i-1} + HP \\
 G2 &= \int_0^x \frac{x}{x+W} dt = x - WG1 \\
 R3 &= GP + \frac{1}{2}HN \\
 G3 &= \int_0^x \frac{x^2}{x+W} dx = \frac{1}{2}x^2 - WG2 \\
 R4 &= \frac{1}{2}GN \\
 G4 &= \int_0^x \frac{x^3}{x+W} dt = \frac{1}{3}x^3 - WG3
 \end{aligned}$$

Then, we have

$$M_x = M_{i-1} + (R1G1 + R2G2 + R3G3 + R4G4) \quad (14.8)$$

We change Eqs. (14.7) and (14.8) to differential form. Suppose Δx is the width of a slice, the term x in Eqs. (14.7) and (14.8) can be replaced by Δx , thus

$$E_i = \frac{1}{(K\Delta x + L)}(E_{i-1}L + \frac{1}{2}N\Delta x^2 + P\Delta x) \quad (14.9)$$

$$M_i = M_{i-1} + (R1G1 + R2G2 + R3G3 + R4G4) \quad (14.10)$$

Equations (14.9) and (14.10) are the explicit expression of the Morgenstern–Price method, which include two unknown variables F and λ (Guoliang Lei 1988, 1998). The boundary conditions for solution are $E_0 = 0$, $M_0 = 0$; $E_n = 0$ $M_n = 0$. First, to give $E_0 = 0$, $M_0 = 0$, then to calculate $E_i = 0$ and $M_i = 0$ one by one, at last, we get the values of E_n and M_n . Adjust F and λ to make $E_n = 0$ $M_n = 0$. Then, the converged factor of safety F and the corresponding λ are achieved. This procedure can be carried out on Excel Spreadsheet.

According to the simulated results of the phreatic levels, the effect of the reservoir on the landslide is by the buoyant force which is equivalent to the resultant forces of all boundary pore water pressures around the slice, and the seepage force can be neglected. Thus, in order to simplify the calculation, we just use effective density instead of saturated density for the slide mass under water and the pore water pressure in related equation would not be considered yet.

The slices of the main section are shown in Fig. 14.10. The factors of safety are calculated in the cases where the reservoir water levels are 130, 135, 140, 145, 150,

Table 14.4 Results of the stability calculation

Water level (m)	λ	F
130	0.3180	1.03200
135	0.3107	1.03180
140	0.3126	1.02730
145	0.3141	1.02190
150	0.3154	1.01490
155	0.3165	1.00850
160	0.3172	1.00490
165	0.3173	1.00487
170	0.3171	1.00810
175	0.3169	1.01280
Totally submerged	0.3147	1.13480

155, 160, 165, 170, and 175 m, respectively. The whole landslide is above the water level when the reservoir water level is at 130 m. The calculated results are shown in Table 14.4. The correlation between the reservoir water level and the factor of safety is also shown in Fig. 14.11.

As the reservoir water level is 135 m, just a little part of the landslide is submerged in water, so the factor of safety in this case is very close to that in the case where the landslide is above water. As the reservoir water level changes between 135 and 160 m, the factor of safety of the landslide decreases with the water level rising. As the water level rises from 160 to 175 m, the factor of safety increases. The result is contradictory to the general assumption that the stability of a landslide decreases with more sliding mass submerged. As we examine the structure of the slide surface, it may be easy to understand. Generally, the slide surface of a landslide

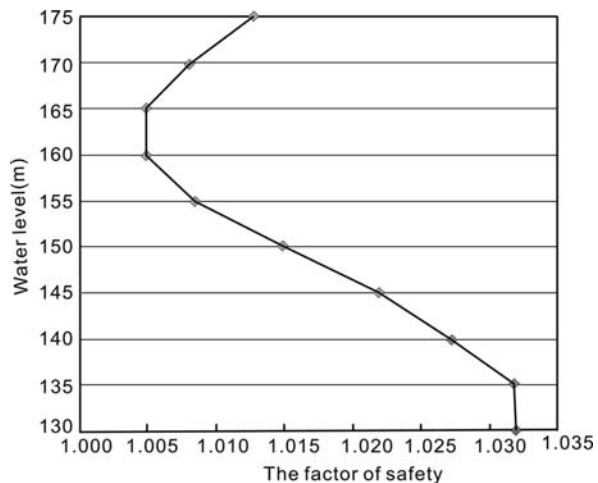


Fig. 14.11 Correlation between the reservoir water level and the factor of safety

has two portions. The lower is a resistant portion which is gentler, while the upper slide portion is steeper. The factor of safety decreases as the resistant portion is submerged because the up resistant force decreases more than the down thrust force, while it increases as the slide portion is submerged because the down thrust force decreases more than the resistant force.

In all the cases, the stability of the landslide is assessed to consider the worst condition, and thus the factor of safety of this landslide should be taken as the lowest value, 1.0046.

Before impounding, the factor of safety is 1.0320. The landslide is close to critical state at this time. As the reservoir water level rose to 135 m, the factor of safety decreased to 1.0318. Though a little reduction, the stability of the landslide deteriorates, causing a tensile fissure on the back of the landslide, this means that the landslide has begun a creep deformation. The landslide experienced a new equilibrium state through deformation. The back tensile crack did not develop as the reservoir water level was kept at 135 m. But the balance may be broken and the landslide may get to the critical state, even slipping, with the rising of the water level. Therefore, the stabilizing work has been done before the reservoir water level rises in the next period.

Stabilizing Work and the Efficiency

The assessment results show that when the reservoir water level rises to 165 m, the factor of safety of the Majiagou landslide would be 1.0046. It is of a critical state. In order to guarantee the safety of the civilians and other facilities on the landslide, the stabilizing work is essential.

All the protected objects on the landslide lie above the Guishui road (Figs. 14.3 and 14.4). The possible approaches such as unloading the top and overloading the foot cannot be accepted according to the situation of the landslide. Anchoring is also not available because the soil of the slide bed is loose. Therefore, the stabilizing piles and necessary drainage system are the few suitable ways that can be considered.

The position of the stabilizing piles should be at the level of the GuiShui road, thus, it can protect the road and the protected objects above the road.

The horizontal forces acting on the piles is the resultant force of the down thrust force on the up side of the piles and the up resistant force on the down side of the piles, which can be calculated by Morgenstern–Prince method. The design factor of safety should be adopted when calculating the down thrust force and the minimal factor of safety should be adopted when calculating the up resistant force. The force acting on the piles is expressed as follow (Li et al. 2006):

$$\Delta E_i = B [E_i(\lambda, F_{\text{des}}) - E_i(\lambda, F_{\text{min}})] \quad (14.11)$$

where ΔE_i is the force acting on the pile plugging in the i th down side of slice, $E_i(\lambda, F_{\text{min}})$ is the down thrust force acting on the pile, $E_i(\lambda, F_{\text{des}})$ is the up resistant force acting on the pile, F_{min} is the minimal factor of safety of the landslide, F_{des} is

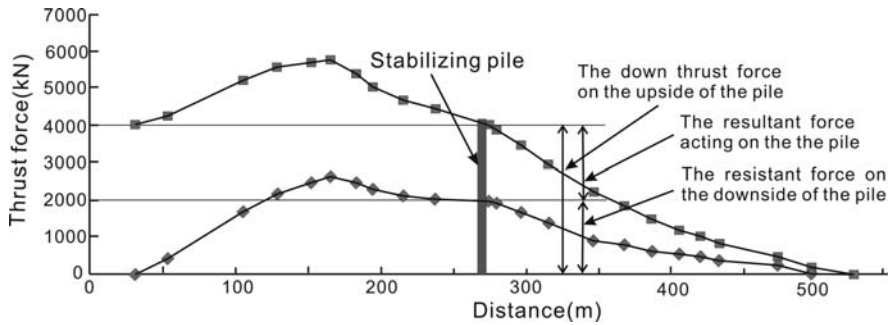


Fig. 14.12 The line of the pushing force of this landslide

the design factor of safety of the landslide, and B is the space between the stabilizing piles.

According to the related mitigation work criterions for the landslides in the Three George Area (GDGH, 2004b), the design factor of safety F_{des} of this landslide is taken as 1.150, the minimal safety factor F_{min} is 1.0046, the space of the piles is taken as 5 m, and then calculated for the line of the thrust forces of this landslide as shown in Fig. 14.12. The stabilizing piles lie in the down side of the 12th slice, and the force acting on the each pile is

$$\Delta E_{12} = 5 \times (3,870 - 1,906) = 9,820 \text{ kN}.$$

Therefore, 20 reinforced concrete stabilizing piles were set in the landslide. The plan section of the pile is 2×3 m, the space between piles is 5 m, the length of all the piles is 22 m, and the length extending in the slide bed is 8 m. The piles were made

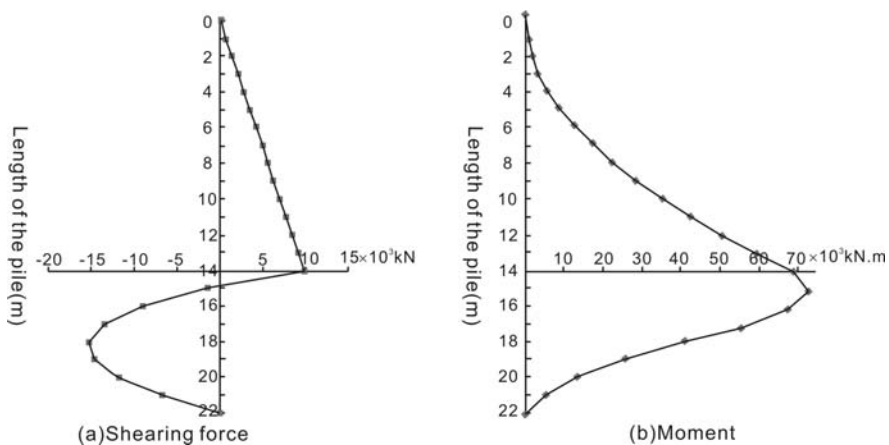


Fig. 14.13 Inner forces of the stabilizing pile

of C30 concrete and $\phi 32$ steel bars. The shearing force and the moment distribution along the pile are shown in Fig. 14.13.

Besides the stabilizing piles, surface drainage systems were designed to decrease penetration of the surface water and to reduce the risk to the piles. After the piles were completed in early 2006, the reservoir water level rose to 156 m in October 2006, and throughout the 2 years of observation up until now, the landslide has not experienced apparent deformation, and it reflects that the stabilizing work is effective.

Conclusions

1. The Majiagou landslide is a secondary slide mass in the front of a large old landslide. It is reactivated by impounding of the Three Gorges Reservoir, which means that the landslide has been close to the natural critical state, and is sensitive to the reservoir water rise. Therefore, investigation and stabilizing are necessary for the safety of the civilians and the facilities on the landslide.
2. The simulated results by the finite element method show that the underground water in the slide mass changes with the reservoir water level simultaneously, in a normal operational state, and the hydraulic gradient is small, so the action of the reservoir water level fluctuation on the landslide is by the buoyant force, and the seepage force may be negligible.
3. The calculated results of the landslide stability by Morgenstern–Price method suggest that the stability of the landslide decreases with rising of the reservoir water level, and the factor of safety goes to the minimum value when the reservoir water level rises to 165 m. After that, the stability increases with the rising of the reservoir water level.
4. The stabilizing piles and the necessary drainage system had been applied to mitigate the landslide. After the project was finished, and throughout 2 years of monitoring, it is proven that the landslide has not experienced apparent deformation and the stabilizing project is effective.

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