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Dealing with Uncertainty in Engineering Design for Large-Scale Gravel Soil Slopes in the Three Gorges Reservoir Zone

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Abstract

The objective of this paper is to first present a general formulation for analysis of uncertainties and evaluation of risks associated with large-scale slopes. The risks may be expressed in terms of the reliability of the system and the consequence. Preliminary studies regarding the mitigation of landslides in the Three-Gorge reservoir zone (TGRZ) of the Yangtze River is presented next. At the normal water level of 175 m, the Three-Gorge reservoir stretches for 665 km along the Yangtze River and extends into many tributaries. Some 684 colluvial deposits, ancient slides and hanging rock blocks, which are larger than 100,000 m^3 individually, and numerous smaller landslides have been identified. Since 2001, over 650 landslides of varies sizes and 2300 cut slopes at low elevations in the Chongqing section of the Yangtze River have been or are being stabilized. This paper deals with three subjects related to the landslides in the TGRZ. The general characteristics of the landslides and the consequences of these landslides are described first. The main causes of activation are considered to be rainfall infiltration, reservoir level fluctuations, and human activities. An attempt is then made to summarize the uncertainties in the design of the slope stabilization works against slope failure due to rainfall infiltration and reservoir level changes. The uncertainties discussed include those involved in the selection of design soil parameters and design-loading combinations, determination of pore-water pressures and potential slip surfaces, and use of analysis models. Finally a design scenario tree is developed to evaluate the landslide risk and to assist risk-investment decisions.

Introduction

The objective of engineering design is to produce a system that will deliver a desired level of performance. Satisfactory performance depends on how well that the design variables in the model and the corresponding model performance can be predicted. Unfortunately, uncertainties do occur in these variables. For instance, in engineering design to mitigate geohazards, severe storms, earthquakes and groundwater changes can impose threatening loading conditions. Clearly, the frequency and magnitudes of these loads are subject to uncertainties. On the capacity side, the strengths of structural systems and soils are also subject to uncertainties. Moreover, the analytical model used for engineering analysis and design are often far from perfect. As a result, the designed system cannot guarantee to deliver the desired performance. At best, the achievement of satisfactory performance could only be expressed in terms of a reliability level. In this case, the engineers themselves are the best persons to evaluate the associated reliability, because they are most familiar with the uncertainties involved in each of the variables and the validity of the design model. They also possess judgmental knowledge learned from experiences, as well as having access to pertinent statistical data. In the evaluation process, they can immediately examine how modification of design could, or if additional sources of information would be needed to, improve the optimality of the ultimate design or hazard mitigation decisions.

The objective of this paper is to first present a general formulation for modeling and analysis of uncertainties and evaluation of the corresponding risk. Preliminary studies regarding the mitigation of landslides in the Three-Gorge reservoir zone (TGRZ) of the Yangtze River is presented next, including a brief introduction to the large-scale slopes in the TGRZ, uncertainties in the design of the slope stabilization works against slope failure due to rainfall infiltration and reservoir level changes, and a design-scenario-tree approach to evaluating the landslide risk and assisting risk-investment decisions.

Landslides in the Three-Gorge Reservoir Zone

The construction of the Three-Gorges Project consists of three phases. The preparatory or first phase spanned five years from 1993 to 1997, ended in November 1997 when half of the Yangtze River was closed. The second phase run from 1998 to 2003. This phase was completed when the first power generation unit in the left-bank power plant went on line and the reservoir level in front of the dam rose to 135 m in June 2003. The third phase is planned for 2004-2009. The reservoir water level will rise to 156 m in late 2006 and finally the normal water level 175 m in 2009. This final phase includes the installation of all turbines and power generators.

At the normal water level of 175 m, the Three-Gorge reservoir will stretch for 665 km along the Yangtze River and extend into many tributaries. The length of the reservoir bank is approximately 1300 km along the Yangtze River and 3680 km along tributaries. The Yangtze River Water Conservancy Commission (1997) has identified some 684 colluvium deposits, ancient slides and hanging rock blocks, which are larger than 100,000 m^3 individually with an estimated total volume of

3035 million m^3. Table 1 shows statistics of these slopes and Table 2 lists some large landslides. In accordance with the developments in phases 2 and 3, two slope stabilization programs were implemented to mitigate landslide risks associated with unstable slopes affected by the reservoir levels at each construction phase. In 2001, over 150 slopes at low elevations in the Chongqing section of the Yangtze river were identified for urgent stabilization and these slopes had been stabilized before the reservoir was filled to elevation 135 m. Recently, more slopes in the Chongqing section, including approximately 500 landslides of various sizes and 2300 cut slopes have been identified for stabilization and over 1600 landslides have been identified for safety monitoring.

Table 1. Statistics of large-scale slopes along the Three-Gorge reservoir (after The Yangtze River Water Conservancy Commission 1997)

		Slides			Collapsed rocks	Cliffs
		Deposits	Rock slides along bedding	Rock slides against bedding		
Main channel	Number	22	101	40	48	4
	Volume (x10^6 m^3)	114.9	1335.6	142.2	130.3	5.5
Tributary	Number	203	92	94	78	2
	Volume (x10^6 m^3)	314.9	555.1	316.3	117.9	2.4

The activation mechanisms of the landslides are primarily rainfall infiltration, reservoir water filling and water level changes, and human activities. For example, rainfall and reservoir filling activated the movements of the Xietan and Maoping slopes in the past few years (Deng et al. 2005). The Qianjiangping landslide failed on 13 July 2003 under the combination of prolonged rainfall and reservoir impounding to 135 m (Dai et al. 2004). Located on the left bank of the Qinggan River, a tributary of the Yangtze River in Zigui County, the landslide had a volume of 24 million m^3, with a maximum width of 800 m and maximum length of 1100 m. The maximum failure depth is approximately 30 m. Figure 1 shows the landslide. The displaced block of weathered sandstone and shale slipped down along the bedding plane. A brick fabricating factory, which was located on the sliding body, slipped down with the debris for approximately 500 m (see Fig. 1b). The debris blocked the Qinggan River completely and formed a landslide dam (see Fig. 1b). The landslide destroyed the home of 129 families and four factories; 14 peopled were confirmed dead.

Social and Economic Impacts

At the normal reservoir level of 175 m in front of the dam, the reservoir will inundate homes for approximately 846,000 people, together with the farmland, the work places and the infrastructure the people rely on (The Yangtze River Water Conservancy Commission 1997). Approximately 1.13 million residents affected must be resettled, most reallocated to new towns at higher elevations. Coping with the geohazards faced by people living on existing natural slopes affected by the reservoir is a major challenge. Figure 2 shows a county town, Yunyang, once the home of approximately 60000 residents. The front part of the town will be inundated

Table 2. Cases of landslides that blocked a river or resulted in dangerous shoals in the Three-Gorges Reservoir Zone (based on Huang et al. 2001)

Name	Location	River	Time	Volume $\times 10^6$ m^3	Trigger	Consequences
Qianjiangping	Zigui	Qinggan	13 July 2003	24.0	Storm, reservoir filling	Complete blockage of river, 14 casualties
Zhongyangcun	Wuqi	Daning	10 June 1988	7.65	Storm, excavation	26 casualties, formation of a 600 m long, 150 m wide and 30 m high debris dam
Huanguancao	Yunyang	Yangtze	July-Aug. 1986	10.0	Storm	Formation of the Xinglong Shoal
Majiaba	Zigui	Tonggudong	16 July 1986	28.8	Storm, excavation	Direct damage RMB 4.5 ml
Xingtan	Zhgui	Yangtze	12 June 1985	30.0	Storm and surcharge	Partial blockage of the river, burial of Xingtan Town, surf height 59 m
Huanglianxia	Yunyang	Changtan	19 July 1982	-	Storm	Formation of a debris dam, villages downstream flooded at collapse of the dam
Shankou	Zhongxian	Ganjing	18 July 1982	18.0	Storm	Formation of a lake
Niujiaodong	Yunyang	Changtan	18 July 1982	6.0	Storm (284 mm/day)	Complete blockage, formation of a 40 m high debris dam
Jipazi	Yunyang	Yangtze	17 July 1982	15.0	Storm (211 mm/day)	Partial blockage of the river, direct damage RMB5.6 ml, slope stabilization and navigation channel remediation RMB80 ml
Tianbao	Yunyang	Ganziping	17 July 1982	7.0	Storm (158 mm/day)	Complete blockage, destroying 487 houses
Yanchihe	Yuannan	Yanchihe	10 Jan. 1980	1.0	Storm, blasting, mining	281 causalities
Xixiangkou	Badong	Guandu	July 1950	5.6	Long rain	Partial blockage
Baota	Yunyang	Yangtze	3500-4000 ys. ago	105.0	Storm	Blockage of main channel

at the normal water level. The town has to be relocated nearly completely, however, due to the Wufengshan landslide behind the town. The landslide occurred on 17 January 2001. It is 170-230 m long along the slope and 80-150 m wide. On 18 June 2001, the landslide debris turned into mudflow amidst rain, destroying several buildings.

The Yangtze River is an important navigation channel. In the past, landslides in the TGRZ zone had either blocked the navigation channel or formed dangerous shoals that affected navigation. Table 2 summarizes some of these landslide cases.

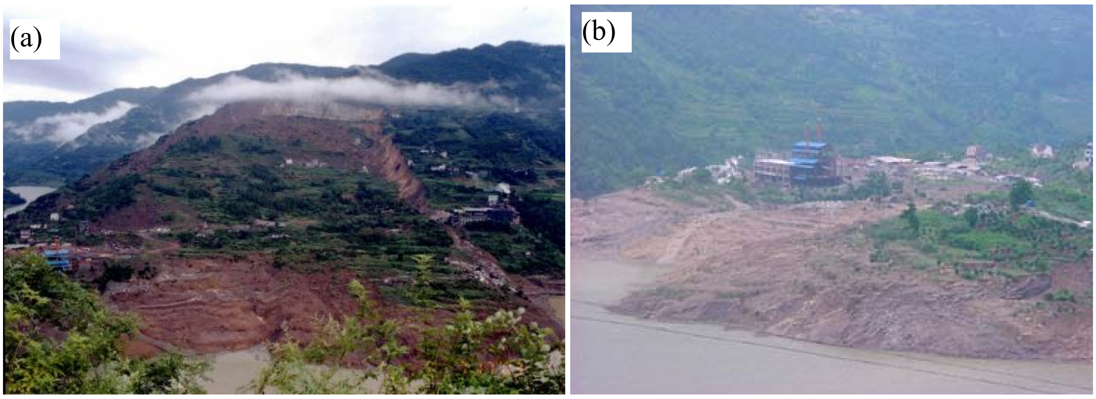


Figure 1. The Qianjiangping landslide (24 million m^3) which blocked the Qingan River (Photos by Y.F. Shi and R. Cheng, http://www.people.com.cn, 17 July 2003)

Figure 2. Yunyang, a county town, is threatened by a landslide in the rear part and to be inundated in the front part. By 2005, much of the town was demolished

Classification of Landslides in Terms of Risk Level

For the purposes of stabilization design for specific landslides, the landslides in the TGRZ are classified in terms of the failure consequences, i.e., the number of people at risk and direct or expected economic losses. Table 3 shows the classification adopted in the Guidelines for Investigation and Stabilization of Landslides in Chongqing (Chongqing Quality and Technical Control Bureau 2003).

Landslides Mitigation Design Scenarios (Load Combinations)

After the completion of the Three-Gorge project, the reservoir will be operated based on a proposed operation scheme (Lin 1992). The water level will be at 175 m from November to middle February. Then it will be lowered gradually to 145 m before the flood season, with an average rate of 0.29 m/day. During the summer (June to September), the water level will be at the flood control reservoir level of approximately 145 m. In the case of floods, the water level may rise and drop quickly. For example, in an event equivalent to the 1954 flood the water level may drop for

Table 3. Engineering classification of landslides and dangerous rock slopes (Adapted from Chongqing Quality and Technical Control Bureau 2003)

Class		I	II	III
Objects in risk		New county towns or larger	Important new towns, large enterprises, important roadways	Ordinary new towns, large enterprises, important roadways
Consequence of failure	People at risk	>300	50 - 300	< 50
	Economic element at risk (RMB in millions)	100	50 - 100	< 50
Suggested safety factor	Normal combinations	1.25	1.15	1.05
	Earthquake	1.05	1.03	1.01

17 m at an average rate of 1.2 m/day. The water level will rise again in October at an average rate of about 1 m/day.

The process of transient seepage and changes in pore-water pressure are of great interest to the safety of slopes. The pore fluid pressures in the slopes above the ground level are negative, which contribute to a significant part of the shear strength of soils and rock masses. As the reservoir water level rises, parts of these slopes will be immersed in water. The infiltration of water into the slopes will be accompanied by a dramatic increase in the pore fluid pressure. Accordingly, the shear strength of slope soils or rock masses, thus the stability of the slopes will decrease. When the reservoir level draws down quickly, a hydrodynamic pressure in a slope will be present, which adds to the driving force as well as the positive pore pressure that reduces the shear strength of the slope materials along the potential slip plane. Due to the complicated nature of transient water flows, it is necessary to identify the key flow conditions that will adversely affect the stability of the slopes. Having done that, it is possible to use appropriate engineering measures to reinforce the slopes, if necessary, and to design and implement a suitable reservoir operation plan to lessen the critical ground water conditions.

Given the above considerations, nine design loading scenarios may be considered (Li et al. 2001):

(1) Natural conditions (pre-reservoir filling, normal rainfall, no surface loading);
(2) Design storm (20-year return storm), drainage system functioning, no surface loading, reservoir level at 135 m;
(3) Design storm, drainage system not functioning, no surface loading, reservoir level at 135 m;
(4) Design storm, drainage system functioning, surface loading (reactivation of slides behind the soil mass being considered), reservoir level at 135 m;
(5) Design storm, drainage system not functioning, surface loading (reactivation of slides behind the soil mass being considered), reservoir level at 135 m;
(6) Normal rainfall, no surface loading, erosion of waterfront, reservoir level at 175 m;
(7) Normal rainfall, water level drawdown from 175 m to 145 m or from 256 m to 135 m, no surface loading, erosion of waterfront;

(8) Design storm, water level drawdown from 175 m to 145 m or from 256 m to 135 m, no surface loading, erosion of waterfront, no surface loading;
(9) Design storm, water level drawdown from 175 m to 145 m or from 256 m to 135 m, no surface loading, erosion of waterfront, drainage system not functioning, surface loading (reactivation of slides behind the soil mass being considered).

Uncertainties in Design Analysis

The primary uncertainties encountered in the analysis of slope stability in the reservoir zone may be summarized as follows and will be discussed separately:
(1) Uncertainties in pore-water pressures caused by rainfall and reservoir level changes;
(2) Uncertainties in the determination of shear strength parameters;
(3) Uncertainties in the search for critical slip surfaces;
(4) Uncertainties in water infiltration and slope stabilisation models.

Uncertainty in pore-water pressures

Pore-water pressure changes due to rainfall infiltration or reservoir level changes are the main drive that activate landslides in the reservoir zone. Analysis of water infiltration is subject to a large number of uncertainties (Chong et al. 2000; Zhang et al. 2003; Zhang et al. 2004):
(1) Geological formations of soils;
(2) Discontinuities or preferential flow channels such as surface cracks;
(3) Spatial variability of soils;
(4) Uncertainties in boundary conditions including boundary flux rate, vegetation, and surface drainage measures;
(5) Uncertainties in initial conditions, especially when evaporation and transpiration effects are significant;
(6) In-situ measurement errors;
(7) Sampling and laboratory measurement errors;
(8) Transformation and curve-fitting errors;
(9) Limited information and correlation among model parameters;
(10) Analysis model errors.

In a design scenario, the calculation of pore-water pressures is usually based on assumed ground water tables. A number of assumptions on pore-water pressure distributions have been made (Zhang et al. 2003). Simplified analytical solutions have also been suggested (Chongqing Quality and Technical Control Bureau 2003, Shi 2004). Yet, variations of pore-water pressures among different design assumptions are large and the pore-water pressure response in the unsaturated zone has not been properly considered in design. Obviously, more rigorous infiltration analyses should be conducted.

Variability in shear strength parameters

Shear strength parameters for slopes at a particular site in the TGRZ are obtained either by large-scale filed direct shear tests or triaxial tests in the laboratory. Field direct shear tests are recommended for investigating the shear strength of soils in historic landslides, particularly those materials in slip zones and the interfaces between soil and bedrock and between soils and concrete.

Based on the specifications of the Chongqing Quality and Technical Control Bureau (2003), the size of soil samples shall not be smaller than 500x500x500 mm. For each landside site, tests at not fewer than 3 locations should be conducted, and not fewer than 3 tests should be conducted at each test location so that the shear strength parameters can be reasonably determined through Mohr circles. The moisture conditions and compositions of the sample must be carefully described before testing. When no existing sliding surfaces are identified or expected, soil samples should be taken and laboratory tests should be conducted to determine the shear strength of the soil deposit.

Shear strength parameters for 67 soil slopes in the TGRZ have been collected in this study. The soils were materials taken from historic slip zones, mostly mixtures of gravelly clay and silt. Statistics show that the plastic index *Ip* of the materials has a mean of 13 and a standard deviation of 3. Figure 3 shows the distributions of the cohesion and friction angle of slip zone materials obtained by laboratory tests. Occasionally, the average shear strength parameters for soil in a sliding zone are back-analyzed assuming a safety factor in the range 0.97-1.05 (Chongqing Quality and Technical Control Bureau 2003). The analysis assumes that the slope was at limit equilibrium when historic landslides occurred amid heavy rains and that the back-analyzed parameters are residual shear strength parameters. The final shear strength parameters for design are usually selected based on a comprehensive evaluation of results from field large-scale direct shear tests, laboratory tests and back analyses. If back-analyzed parameters were used for design, then the role of slope stabilization, either improved drainage or retaining structures, would be to increase the safety factor of the slope from about 1.0 to the selected design safety factor under most unfavourable design conditions.

It is noted that the soil parameters from field tests, laboratory tests and back analyses differ significantly. In general, the uncertainty with soil parameters may be divided into statistical uncertainty and test uncertainty:

$$X_I = N_o N X_A \tag{1}$$

where X_I = in situ soil property; N_0 = correction factor for statistical uncertainty due to insufficient test samples; N = correction factor for test discrepancy; and X_A = spatial average soil property. Assuming that the shear strength of the slopes in a large region is statistically homogeneous; then probability distributions of shear strength parameters can be estimated by pooling data from different sites. Also assume all the three random variables (N_0, N and X_A) follow the normal distribution. Based on field and laboratory test results and taking advantage of the back analyzed soil parameter values from landslides, the statistics of the model factors N_o and N can be updated as shown in Table 4, for various assumed priors. The updated correction factors are sensitive to the assumed variability of the priors. When a COV of N_0 and N of 0.1 are assumed, the mean reduction factor will be approximately 0.85 also with a COV of 0.10.

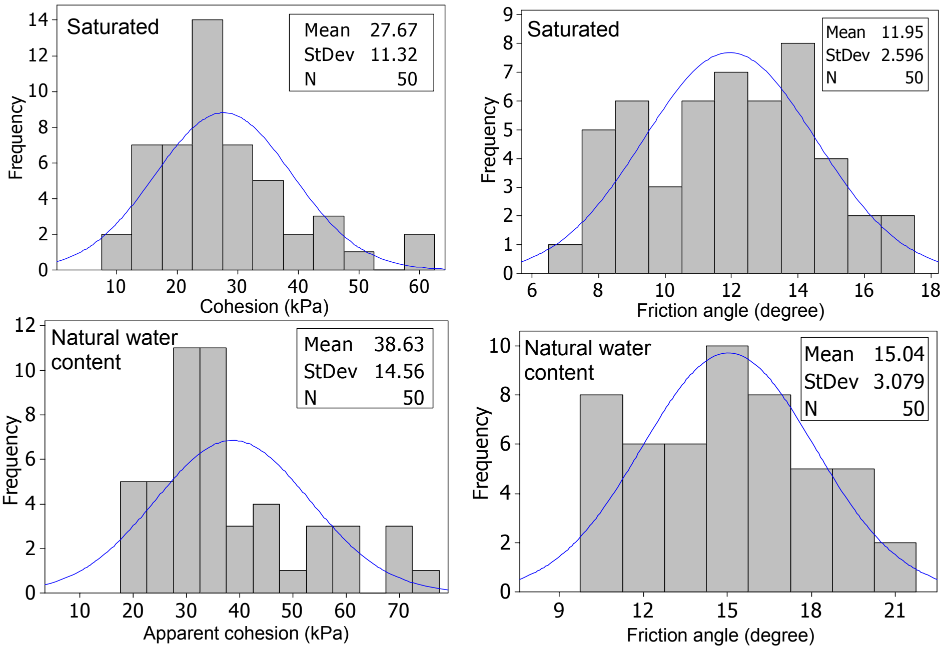


Figure 3. Frequency diagrams of peak apparent cohesion and friction angle from laboratory testing on slip zone soils at 67 sites in the TGRZ

Uncertainty in potential slip surfaces

Many slopes are historic landslides. Effort in site exploration has accordingly been paid to the characterization of the historic slip zones and the slip-zone material properties through geologic explorations. Only a single slip zone is normally recommended from such explorations. This method may however fail to find potential failure mechanisms triggered by recent activities, e.g., reservoir impounding, rainfall infiltration, and human activities. Figure 4 shows a rainfall-induced shallow seated failure underlain by a historic landslide. In a storm event, the rain water infiltrates into the slope soil, destroys the soil suction and reduces the shear strength of the soil. Depending on rainfall and soil characteristics, the historic slip zone may not be wetted before the failure occurs along a new shear failure zone.

Table 4. Statistics and correction factors from multi-variable Bayesian analysis.

		COV of N_0		**Mean of N**		**COV of N**	
		Mean	St.dev	Mean	St.dev	Mean	St.dev
1	Prior	0.10	0.05	1.00	0.05	0.05	0.05
	Posterior	0.13	0.03	0.88	0.03	0.06	0.05
2	Prior	0.10	0.05	1.00	0.03	0.03	0.03
	Posterior	0.15	0.02	0.93	0.03	0.03	0.03
3	Prior	0.10	0.05	1.00	0.10	0.10	0.05
	Posterior	0.10	0.04	0.85	0.03	0.10	0.04
4	Prior	0.10	0.05	1.00	0.10	0.10	0.10
	Posterior	0.10	0.05	0.85	0.03	0.10	0.05

Note: Mean of N_0 = 1.0 (unbiased)

The development of multiple slip surfaces can be simulated by a finite element or finite difference method. Finite element analysis of slope stability does not provide an explicit factor of safety but utilizes the so-called shear strength reduction technique (e.g. Griffith and Lane 1999). The safety factor of a slope is defined as the number by which the shear strength parameters must be factored down to bring the slope to failure. Zheng and Zhao (2004) extended this approach for analyzing the development of multiple slip surfaces in rock and soil slopes.

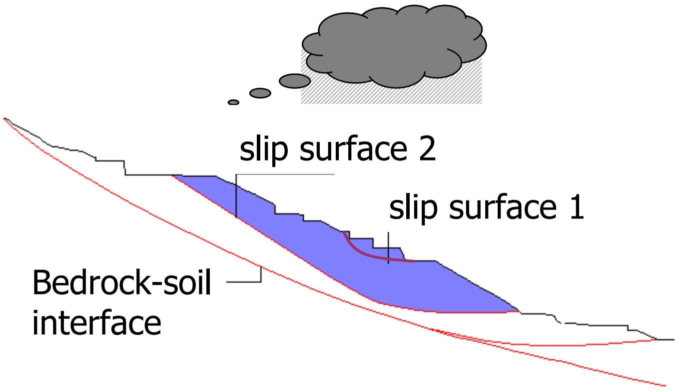


Figure 4. Development of shallow-seated failure or multiple failure zones in a slope

Uncertainties in water infiltration and slope stability analysis models

Errors can arise due to insufficient representation of physical problems in concern, particularly geological formations, boundary conditions, initial conditions, and constitutive models of materials. As such, the calculated reliability of a particular slope is only notional. Tang and Cheung (2004) and Cheung and Tang (2005) developed methods for calibrating the model errors associated with slope stability analysis based on observed performance of a large number of slopes in Hong Kong.

Risk Assessment and Design Strategy

Due to the presence of various uncertainties described in the previous sections, failure of the large-scale slopes in the TGRZ is likely. The risk posed by a specific landslide may be estimated by

$$R = \sum_{i=1}^{n} H_i V_i C_i \tag{2}$$

where R = risk; n = possible scenarios of slope failure; H_i = probability of hazard i; V_i = vulnerability of the element at risk; C_i = element at risk given the occurrence of hazard i, including both people and economical elements. The risk depends on human or economical elements affected by the slope, required design safety levels against possible failure mechanisms, load cases, effectiveness of warning and evacuation systems, and preparedness of the elements at risk.

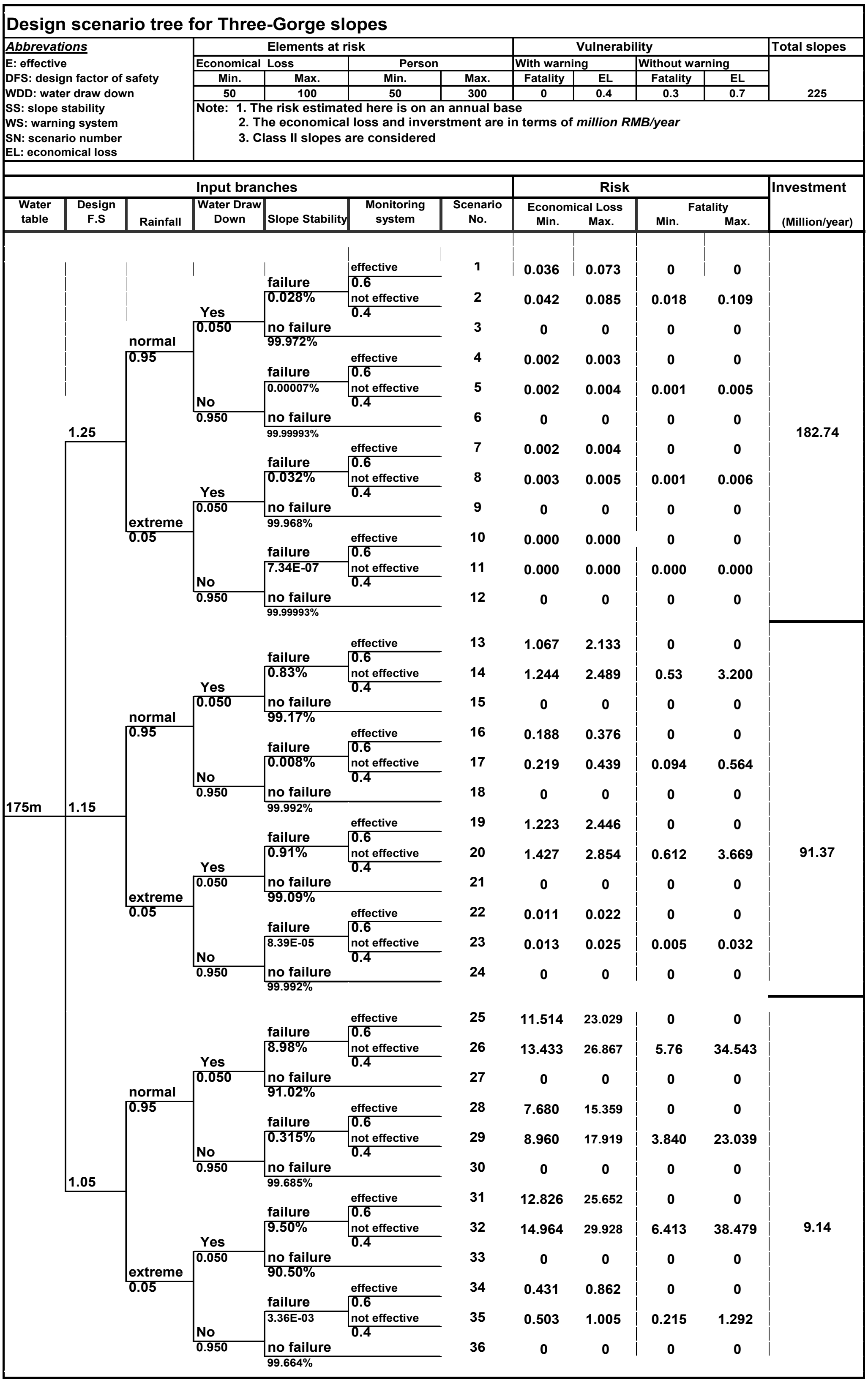
Hazard probability, vulnerability and elements at risk can be integrated into a systematic layout through a design scenario tree. The effect of design and measures for risk reduction can be efficiently analyzed using the scenario tree. Figure 5 shows a scenario tree for 225 landslides in the TGRZ. Assume the slopes are all Class II slopes (see Table 3), with the number of people at risk by each slope in the range of 50-300 and the worth of economic elements at risk in the range of RMB 50-100 millions. The probability of failure of a specific slope can be calculated using a first order reliability method based on the designated design safety factor, load combination (in this case design rainfall and reservoir level fluctuation), and sensitivity of safety factor with respect to various influence factors. More specifically in Figure 5, p_f was calculated for the Baota landslide based on field information described by Li et al. (2001). The designated safety factor has a great influence on the calculated p_f, so three different values of safety factor, 1.25, 1.15 and 1.05, are included in Figure 5. The extreme load combination included reservoir level drawdown from 175 m to 145 m within 14 days plus a concurrent extreme rainfall lasting for 14 days. The vulnerability may be assumed based on the effectiveness of monitoring and evacuation systems. Given the assumed values of occurrence probability of loads, effectiveness of monitoring system, assumed vulnerability and elements at risk, the risk for each scenario can be calculated for these 225 slopes. Table 5 summarizes the risks of the 225 slopes corresponding to the three designated design safety factors. Both the fatality and economic loss of these slopes increase significantly with decreasing design safety factor.

Decision on investment for landslide mitigation measures can also be assisted by the scenario tree. Table 5 compares the investment associated with the three assumed levels of safety. In the second phase landslide mitigation, the average annual cost for increasing the safety factor of one slope by 0.1 is approximately RMB0.406 millions (assuming a life span of 50 years). When a safety factor of 1.25 is adopted, the design strategy would be to increase the safety factor from 1.05 to 1.25 under unfavorable design conditions. The average annual landslide mitigation investment for one slope would be 0.2/0.1x0.406 = RMB0.812 millions and the approximate annual investment for stabilizing the 225 slopes considered would be RMB182.7 millions. The relatively high cost is paid off by the resulting low risk to both human lives and economic losses. When a low safety factor is used, but coupled with a monitoring system, the annual investment cost could be minimal. However, fatality (16-97) and economic loss appear to be unacceptable. A suitable safety factor can be selected based on an investment-risk analysis like this in a more realistic way.

The TGRZ landslide design scenario tree can be extended to include other uncertainties such as pore-water pressures, slip surfaces, and uncertainties in vulnerability and analysis models. The risk confidence interval can be obtained by simulation method. The tree can be also used to compare effectiveness of structural/non-structural options.

Table 5. Comparison of risk and investment for selecting acceptable design strategy

Design safety factor	Ecnomical loss		Fatality		Investment
	Minimum	Maximum	Minimum	Maximum	(Million)
1.25	0.09	0.17	0.02	0.12	182.74
1.15	5.39	10.78	1.24	7.47	91.37
1.05	70.31	140.62	16.23	97.35	9.14



Design scenario tree for Three-Gorge slopes

Abbrevations	Elements at risk				Vulnerability				Total slopes
E: effective	Economical Loss		Person		With warning		Without warning		
DFS: design factor of safety	Min.	Max.	Min.	Max.	Fatality	EL	Fatality	EL	
WDD: water draw down	50	100	50	300	0	0.4	0.3	0.7	225

SS: slope stability
WS: warning system
SN: scenario number
EL: economical loss

Note: 1. The risk estimated here is on an annual base
2. The economical loss and inverstment are in terms of *million RMB/year*
3. Class II slopes are considered

Input branches							Risk				Investment
Water table	Design F.S	Rainfall	Water Draw Down	Slope Stability	Monitoring system	Scenario No.	Economical Loss Min.	Economical Loss Max.	Fatality Min.	Fatality Max.	(Million/year)
175m	1.25	normal 0.95	Yes 0.050	failure 0.028%	effective 0.6	1	0.036	0.073	0	0	182.74
					not effective 0.4	2	0.042	0.085	0.018	0.109	
				no failure 99.972%		3	0	0	0	0	
			No 0.950	failure 0.00007%	effective 0.6	4	0.002	0.003	0	0	
					not effective 0.4	5	0.002	0.004	0.001	0.005	
				no failure 99.99993%		6	0	0	0	0	
		extreme 0.05	Yes 0.050	failure 0.032%	effective 0.6	7	0.002	0.004	0	0	
					not effective 0.4	8	0.003	0.005	0.001	0.006	
				no failure 99.968%		9	0	0	0	0	
			No 0.950	failure 7.34E-07	effective 0.6	10	0.000	0.000	0	0	
					not effective 0.4	11	0.000	0.000	0.000	0.000	
				no failure 99.99993%		12	0	0	0	0	
	1.15	normal 0.95	Yes 0.050	failure 0.83%	effective 0.6	13	1.067	2.133	0	0	91.37
					not effective 0.4	14	1.244	2.489	0.53	3.200	
				no failure 99.17%		15	0	0	0	0	
			No 0.950	failure 0.008%	effective 0.6	16	0.188	0.376	0	0	
					not effective 0.4	17	0.219	0.439	0.094	0.564	
				no failure 99.992%		18	0	0	0	0	
		extreme 0.05	Yes 0.050	failure 0.91%	effective 0.6	19	1.223	2.446	0	0	
					not effective 0.4	20	1.427	2.854	0.612	3.669	
				no failure 99.09%		21	0	0	0	0	
			No 0.950	failure 8.39E-05	effective 0.6	22	0.011	0.022	0	0	
					not effective 0.4	23	0.013	0.025	0.005	0.032	
				no failure 99.992%		24	0	0	0	0	
	1.05	normal 0.95	Yes 0.050	failure 8.98%	effective 0.6	25	11.514	23.029	0	0	9.14
					not effective 0.4	26	13.433	26.867	5.76	34.543	
				no failure 91.02%		27	0	0	0	0	
			No 0.950	failure 0.315%	effective 0.6	28	7.680	15.359	0	0	
					not effective 0.4	29	8.960	17.919	3.840	23.039	
				no failure 99.685%		30	0	0	0	0	
		extreme 0.05	Yes 0.050	failure 9.50%	effective 0.6	31	12.826	25.652	0	0	
					not effective 0.4	32	14.964	29.928	6.413	38.479	
				no failure 90.50%		33	0	0	0	0	
			No 0.950	failure 3.36E-03	effective 0.6	34	0.431	0.862	0	0	
					not effective 0.4	35	0.503	1.005	0.215	1.292	
				no failure 99.664%		36	0	0	0	0	

Figure 5. A scenario tree for landslide risk assessment

Issues for Future Study

The following issues regarding stability of slopes in the TGRZ would be addressed in the next phase of the study:

(1) More reliable analysis models
  - Transient flows in unsaturated soil/rock slopes due to reservoir level changes: how far and how fast the ground water will be affected?
  - Identification of critical water level and storm combinations for design;
  - Strength of infill materials in weak planes in response to water infiltration;
  - Stability of slopes under the various groundwater scenarios (mechanism study; numerical and centrifuge modelling);
  - Engineering measures particular to the scenarios;

(2) Risk analysis and management
  - Critical reservoir water changes that should be avoided in reservoir operation;
  - Evaluation of risk of reservoir slopes; consequences of failures; tolerable risk; risk management; importance of monitoring; prioritization of slopes for stabilization actions; and strategy of slope maintenance.

Summary

The stability of the large-scale soil slopes in the TGRZ under rainfall infiltration and reservoir level changes is a great concern. The design of these slopes involves several sources of uncertainties; namely uncertainties in the pore-water pressures caused by rainfall and reservoir level changes, shear strength parameters, the search for critical slip surfaces; and water infiltration and slope stability analysis models. Accordingly, these slopes pose hazards to local residents and properties, and a realistic evaluation of risks is important in hazards mitigation. Engineers should play a crucial role in analysis and design to provide useful information for defensible decision. The use of design scenario tree facilitates comparison of design / policy options.

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