

中华人民共和国电力行业标准

# 水电水利工程边坡设计规范

**Design specification for slope of hydropower  
and water conservancy project**

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## 1. Scope

This Specification stipulates the following contents including the safety class, safety design criteria, stability analysis methods, synthetic rehabilitation measures and safety monitoring as well as early-warning for design of slopes in the major structure areas and reservoir bank slopes near dams for hydropower and water resources projects.

This Specification is applicable to the engineering design of slopes in major structure areas and natural reservoir bank slopes near dams which may influence the project's normal and safe operation for large- and medium-sized hydropower and water resources projects. For design of other slopes in reservoir areas, this Specification may be referred.

For extra large-sized slopes and very important or very complicated slopes, special study and demonstration shall be carried out on the safety design criteria, stability analysis and assessment, as well as synthetic stabilization measures.

## 2 Normative References

The following references contain provision which through reference in this Standard, constitute the provisions of the Standard. For dated reference, subsequent amendments to, or revisions of, any of these publications do not apply. All standards are subject to revision, and parties to agreements based on this Standard are encouraged to investigate the possibility of applying the most recent editions of the standard indicated below. For undated references, the latest edition of the normative document referred to applies

- GB50287 Code for hydropower engineering geological investigation
- GB50330 Technical code for building slope engineering
- DL/T5057 Design code for hydraulic concrete structures
- DL5073 Specifications for seismic design of hydraulic structures
- DL5077 Specifications for load design of hydraulic structures
- DL/T5176 Design specification of prestressed anchorage for hydropower project
- SL55 Specification of engineering geological investigation for medium-small water conservancy and hydropower development

### **3 Terminology and Definitions**

#### **3.0.1 Slope**

A geological body on the crust surface with lateral free faces, consisting of slope crest, slope surface, slope toe and the mass below them in a certain depth.

#### **3.0.2 Natural slope**

Slope formed by natural agents.

#### **3.0.3 Engineered slope**

Man reformed slope or slope influenced by engineering works.

#### **3.0.4 Stable and undeformed slope**

A slope keeping its original state without any traces of deformation and instability.

#### **3.0.5 Deforming slope**

A slope deformed or under deformation

#### **3.0.6 Slope engineering / works**

Engineering works to modify a slope for a certain purpose.

#### **3.0.7 Buckling failure**

Or called slip-bending failure, a failure mode that occurs in a bedded consequent slope with the upper part sliding along the bedding plane and the lower part bending and dilating outward

#### **3.0.8 Toppling**

A failure mode that occurs in bedded or quasi-bedded rock mass with the rock mass toppling toward the free face and breaking at the toe.

#### **3.0.9 Slope geological model**

A generalized and simplified model that indicates the distribution of geo-materials, discontinuities and slip planes in a slope, generally represented by plans, profiles and horizontal sections.

#### **3.0.10 Factor of safety**

An index expressing the slope's stability state against sliding. It is a ratio of the resultant slide-resisting force to the resultant driving force. Strictly speaking, it is the reduced times of the shear strength parameters by supposing that the rock/soil mass reaches a limit equilibrium along a determined sliding plane.

#### **3.0.11 Designed factor of safety**

The lowest allowable factor of safety for a slope to attain an expected safety level.

#### **3.0.12 Limit equilibrium methods**

Methods for determining the factor of safety against sliding of a slope by establishing and resolving the static limit equilibrium equations according to the Mohr-Coulomb strength criterion.

#### **3.0.13 Upper bound solution**

For an integral sliding or a disintegrated sliding failure mode, corresponding to a certain permissible displacement field, if the limit equilibrium state is reached along the sliding

plane and the discontinuities within the sliding mass (or at every point within the sliding mass for a homogeneous soil slope), the factor of safety must be greater than or equal to the corresponding true value, and the solution is defined as the upper bound solution.

#### **3.0.14 Lower bound solution**

For an integral sliding mode, if a limit equilibrium state is reached along the slide plane and the stress in the sliding body is limited under the yield surface, then the factor of safety must be less than the corresponding true value, and the solution is defined as the lower bound solution.

#### **3.0.15 Method of information design**

A design method of reviewing slope stability and modifying the original design according to the latest investigation data obtained during construction, and the feedback information from the permanent or temporary monitoring system.

#### **3.0.16 Risk analysis**

A method for determining annual failure probability of slopes, prediction of consequences of failure including life and property losses, and proposal of risk assessment.

#### **3.0.17 Subsurface drainage system**

Underground drainage facilities composed of drainage holes, tunnels and wells linked together within a slope.

#### **3.0.18 Slide-resisting pile**

A pile built in a landslide or potential landslide and vertically penetrating through the sliding plane to a certain depth to improve the slope stability.

#### **3.0.19 Shear resisting plug**

A reinforced concrete plug built in a tunnel horizontally excavated across the sliding plane in a rock slope to key the upper and lower walls together with its axis generally perpendicular to the sliding direction

#### **3.0.20 Retaining concrete plug**

A reinforced concrete plug built in a tunnel horizontally or inclined excavated across the sliding plane in a rock slope or landslide for stabilization purpose with its axis generally parallel to the sliding direction.

#### **3.0.21 Retaining wall with anchors**

A supporting structure composed of a retaining wall and anchoring bolts (cables)

#### **3.0.22 Anchored frameworks**

A lattice structure fixed on slope surfaces.

#### **3.0.23 Protecting wire mesh**

A flexible wire mesh covering the slope surface to prevent falling rock blocks or fixed at slope toe to catch falling rock blocks.

## **4 General Provisions**

**4.0.1** The design of slopes for hydropower and water resources projects shall match the design phase of the associated structures and meet the requirements of being reliable and safe, cost effective, advanced in technology and realistic.

**4.0.2** The design of a slope shall be carried out on the basis of the results of geological investigation and geotechnical tests. The geological investigation and geotechnical tests shall be carried out in conformity to GB50287 and associated test specifications.

**4.0.3** During the comparison and selection of project sites and the hydraulic structure layout alternatives, the topographical and geological conditions and the structure layout requirements shall be analyzed in detail, the potential problems concerning slope stability shall be considered previously to minimize the difficulties of the slope engineering treatments.

**4.0.4** According to the topographic and geologic conditions in the slope area, and the expected excavation shape, the study on the slope structure and on the deformation and sliding failure mode as well as the analysis and assessment on the stability conditions shall be carried out. The slope type and slope structure shall be classified following Appendices A and B; the deformation and failure mode of slopes shall be determined according to Appendix C.

**4.0.5** After determination of the project layout, the importance, the failure risk and the failure consequences of slopes surrounding the main structures shall be analyzed and studied; then the safety class of slope shall be defined according to the clause 5.0.1 of this Specification, based on which the slope design principles and basic requirements are decided.

**4.0.6** Slope excavation shall be designed by referring to the recommendations by geologists about the slope inclination and by taking consideration of the factors including the project purpose, stabilization measures, berms and drainage, and requirement of access road, construction and maintenance.

**4.0.7** Determination of the stability level and the deformation limit of slopes, which must be satisfied for the slope design, and selection of suitable stability analysis method shall be carried out in accordance with the project purpose, engineering geological conditions and the failure mode. Stabilization measures shall be proposed through comprehensive technical and economical comparison.

**4.0.8** The Limit equilibrium analysis is a basic approach for slope stability analysis and applicable for slopes of sliding failure mode. For slopes of the class 1 and class 2, two or

more calculation methods including FEA and DEA methods shall be used for deformation and stability analysis to comprehensively evaluate the safety against deformation and sliding failure of the slopes.

**4.0.9** For very important or geologically complicated high slopes, a special stress-deformation analysis or emulation analysis shall be done to study the failure mechanism, the failure mode and the effective stabilization measures. A geomechanical model test may be conducted if necessary. A simplified reliability analysis method (see Appendix D) is recommended by this Specification in case the reliability analysis is required.

**4.0.10** In the process of slope design, the results of in-situ investigation and geological analysis, including the observation results of slope deformation and groundwater, shall be used sufficiently. During slope construction, the design parameters shall be checked and adjusted on the basis of the deformation and stability analysis results, in combination of with the geological observation and prediction, geological mapping, and the feedback information from monitoring results. The information design shall be implemented throughout the slope construction.

## 5 Slope classification and design factor of safety

**5.0.1** Slopes for hydropower and water resources projects shall be categorized and classified according to project scale and structure grade, slope location and the importance of the slopes and the catastrophic consequences as shown in Table 5.0.1.

**Table 5.0.1 Categorization and classification of slopes for hydropower and water resources projects**

Class	Group A Slopes in main structure area	Group B Reservoir bank slopes
1	Affect the safety of grade 1 hydraulic structures	May threaten the safety of grade 1 structures by landslide or the resulted hazardous surge
2	Affect the safety of grades 2 and 3 hydraulic structures	Affect the safety of grades 2 and 3 structures by possible landslide
3	Affect the safety of grades 4 and 5 hydraulic structures	Global stability is required, but local failure or slow slip is permitted

**5.0.2** Provided the slope failure in project area only affects the normal operation of some structure but does not threaten the safety of structures and lives, the slope class may be decreased by one class.

**5. 0. 3** Provided it has been confirmed that the reservoir landslides or potentially instable bank slopes are classified as the creeping failure mode and the changes in stability can be predicted and forecasted through safety monitoring, and preventive measures can be taken against the failures, then the class of these slopes or landslides may be decreased by one or two classes.

**5. 0. 4** Stability analysis of slopes for hydropower and water resources projects shall be made according to different loading combinations or operation conditions. When the lower bound solution method of limit equilibrium is adopted, the design factor of safety shall not be less than the specified values listed in Table 5.04.

**Table 5.0.4 Design factor of safety for slopes of hydropower and water resources projects**

Mode Class	Group A Slopes in main structure area			Group B Reservoir bank slopes		
	sustained	transient	accidental	sustained	transient	occasional
1	1.30~1.25	1.20~1.15	1.10~1.05	1.25~1.15	1.15~1.05	1.05
2	1.25~1.15	1.15~1.05	1.05	1.15~1.05	1.10~1.05	1.05~ 1.00
3	1.15~1.05	1.10~1.05	1.00	1.10~1.00	1.05~1.00	≤1.00

**5.0.5** The design safety factor adopted for a specific slope shall be selected within the range specified in Table 5.0.4 according to analysis of the factors including the relationship between the slope and structure, the engineering scale of slope, the geological complexity and the uncertainties of slope stability. For slopes with high failure risks or greater uncertainties in stability analysis, the adopted design factor of safety should be the upper limit value; otherwise the lower limit value should be adopted.

**5.0.6** For very important slopes or slopes with requirements on deformation limit, the design factors of safety shall be determined through the stress-deformation analysis and generally be higher than the values specified in Table 5.0.2.

## **6 Slope structure and failure modes**

### **6.1 General**

**6.1.1** Slope stability analysis, excavation design and synthetic stabilization design shall be carried out based on the following works: collection and analysis of the data and information about the meteorology, hydrology, seismicity, engineering geology, hydrogeology and the structure layout; study on the history and the existing situation of slopes and the possible effects of human activities on slopes.

**6.1.2** In accordance with the engineering geological analysis and assessment, divide the studied slopes roughly into sections, determine the geological type, judge the basic stability conditions, possible deformation mechanism and failure modes, and finally delimit the slope extent for stability analysis and stabilization design.

**6.1.3** For slopes that need comprehensive stabilization treatment, a safety monitoring system should be established as early as possible in combination with the geological exploration and engineering construction in order to timely get the slope dynamic situation.

### **6.2 Structure model of slope**

**6.2.1** Slopes may be classified, according to engineering geological zoning, into rock slopes, soil slopes and rock-soil composite slopes; or, according to the engineering geological assessment, into stable slopes, potentially instable slopes, deforming slopes, instable slopes and failed slopes. See Appendix A.

#### **6.2.2 Structure model of rock slope**

1. Structure type of rock slope shall be determined according to Table B.1 in Appendix B.

2. Identify and analyze the possible combination of the discontinuities with different types and different scales based on geological data; analyze and recognize the potentially instable rock mass or blocks with various dimensions by using stereographic projection or other methods.

For slopes with combination of many discontinuities, analysis shall be carried out firstly on the determinative blocks formed by combination of weak discontinuities, weak zones or seams and thorough discontinuities; secondly on the semi- determinative blocks formed by weak discontinuities, weak zones or seams in combination with joint sets or bedding fissures.

For slopes free from weak and thorough discontinuities, analysis shall be made on random blocks formed by joint sets or bedding fissures.

3. For stratified rock slopes, the structure types shall be classified according to the relationship between the attitudes of bedding planes and slope faces, and the possible deformation and failure mode shall be judged.

4. For the rock slopes of blocky or stratified structure subject to slip failure, rational

stability analysis method shall be selected according to different sliding modes including plane sliding, wedge sliding or the composite plane sliding.

5. For the rock slope with cataclastic structures, analysis shall be made on the circular slip mode in addition to the abovesaid three sliding modes

6. For the rock slope with loose structure, the stability analysis may be conducted as for the soil slopes.

#### **6.2.3 Soil slope structure model**

1. The types of soil slopes shall be determined according to Table B.2 in Appendix B.

2. The soil slopes are divided into homogeneous soil slopes, stratified soil slopes and heterogeneous soil slopes according to geological data.

The homogeneous soil slopes are subdivided into sandy soil slopes and clayey soil slopes.

The accumulated slopes are divided into homogeneous, stratified and heterogeneous slopes by analogy with soil slopes.

3. The soil slopes of different types shall be analyzed according to the sliding failure modes described as follows:

Planar sliding for homogeneous sandy soil slopes;

Circular sliding for homogeneous clayey soil slopes;

Sliding along bedding plane or combined bedding planes for stratified soil slopes.

Sliding may occur along weak planes for heterogeneous soil slopes;

Sliding may occur along the contact face between soil and bedrock or within the soil or bedrock for slopes composed of soil on the upper and rock on the lower.

4. The failure modes of soil slopes composed of loess, soft soil, swelling soil or other special soils shall be determined according to the engineering geological conditions and deformation characteristics.

#### **6.2.4 Deforming slope**

1. The deformation and failure modes for deforming slopes shall be determined according to Table C.1 in Appendix C.

2. For deformed, loosened and creeping slopes of sliding failure mode, the extent, boundary, inner intersecting planes and potential slide planes shall be determined according to geological data.

3. For slopes of toppling, buckling, falling, plastic flowing and other non-sliding failure modes, the extent and influence depth shall be ascertained according to the geological data.

#### **6.2.5 Landslides and failed slopes**

1. The failure modes of slopes and landslide types shall be determined according to Table C.1 and Table C.2 in Appendix C.

2. The boundary of landslides, collapse-slip bodies or accumulated bodies of failed slopes shall be determined according to geological data, including the base sliding planes, rear tension planes, lateral cutting planes and secondary intersecting planes within the landslide.

### **6.3 Analysis on kinematic features of a failure slope**

**6.3.1** For the slope where the failure modes are important for treatment decision making or project layout, the failure movement features shall be analyzed.

**6.3.2** For analysis of the movement features of a failure slope, the failure types shall be classified according to geological investigation, and the failure process and movement pattern shall be predicted.

**6.3.3** For a sliding-failure slope, the major sliding plane and the secondary sliding planes shall be defined before calculation of the factor of safety of the integral sliding body and local sliding bodies using the limit equilibrium method. On the basis of the calculated results, a prediction should be made of the possibility of disintegrated sliding, the sliding sequence and accumulation form of local failures, and the maximum sliding volume of one slide.

**6.3.4** For important slopes, the finite element method, discrete element method, discontinuous deformation analysis or other block movement analysis methods should be adopted to study and predict the failure and movement modes of slopes.

**6.3.5** For large- and medium-sized landslides on the reservoir bank near the dam, the velocity and distance of the possible sliding movement shall be analyzed and predicted; the probable surge induced by the landslide in front of the dam shall be calculated and predicted by means of formulas or model test, on the basis of which the early-warning and preventative measures shall be prepared.

### **6.4 Representative profiles of slopes**

**6.4.1** The representative profiles of a slope shall be made along the direction normal to the strike of the slope comprising detail information about the rock types, weathering, stress releasing, geological structure, groundwater and other necessary geological and hydro-geological data. In case the slope is divided into sections according to the geological conditions and the stability state, not less than one representative profile shall be made for each section.

**6.4.2** For a potential landslide or a possibly instable slope, representative profiles shall be made as a two-dimensional strain model along the possible moving direction with spacing not larger than 30m. In the direction normal to the moving direction, not less than 2 cross sections shall be made.

**6.4.3** Both the horizontal and vertical scales of a representative profile shall be the same. The profile scale shall satisfy the requirements for analysis and calculation. The scale of plans and profiles used for slope design should be no less than 1/1000.

## **7 Slope Stability Analysis**

### **7.1 General**

**7.1.1** The methods and formulae for slope stability calculation are shown in Appendix E. For toppling deformation and buckling failure of stratified rock slopes, engineering geological qualitative and semi-quantitative methods shall be adopted for stability analysis. The possible toppling and buckling positions shall be determined and then stability against sliding shall be analyzed for the rupture plane formed after occurrence of toppling or buckling.

**7.1.2** For rock falling failure slopes, the distribution area of dangerous rock blocks and unstable rock mass shall be defined according to the geological data. The stability shall be assessed by means of qualitative and semi-quantitative methods.

**7.1.3** For a slope located in an important area, its stability in different construction phases shall be analyzed according to the excavation and anchorage process, in addition to stability analysis of the slope in the natural and final state.

**7.1.4** For a slope under construction, its stability shall be checked according to the feedback information obtained from the permanent or temporary monitoring system.

**7.1.5** The following basic data shall be available for slope stability analysis:

1. Engineering geology

The engineering geological plans, profiles and horizontal sections;

The seismic dynamic parameters; in case the basic seismic intensity of the site is higher than VIII, the seismic safety assessment data and the corresponding dynamic parameters shall be available.

2. Hydrogeology

Groundwater contour map;

Long term observation data of ground water;

Permeability coefficient of strata

3. Physical and mechanical parameters of rocks and soils

Test standard values and recommended parameters of density, porosity, water content, compressive strength, shear strength, deformation modulus, elastic modulus, Poisson ratio and other properties.

For important slopes, the stress-displacement curves of controlling discontinuities from shear test and the loading-unloading curves of rock mass from deformation test shall be available.

4. Hydrology and meteorology

Characteristic water levels of natural river and reservoir during construction and operation;

Data of rainfall, rain intensity and process;

Extent and intensity of the splashing and atomized water induced by flood releasing

from the reservoir.

#### 5. Project layout

General layout of main structures;

Plans and profiles of main structures.

### 7.2 Actions on slopes and action combinations

#### 7.2.1 Dead load of rock and soil mass

1. For rock and soil mass above groundwater table, the natural unit weight shall be used; for the rock and soil mass under groundwater table, the unit weight shall be chosen rationally depending on the calculation method. The saturated unit weight shall be adopted if the water pressure is considered as a force acting on the surfaces of boundaries and slices. The buoyant unit weight shall be used if the water pressure is taken as a volume force, and the hydrostatic pressure on the slide plane shall be deducted which is accounted at the water table out of the slope at the same time. For unsaturated rock and soil mass during rainy season, a unit weight with certain water content shall be used, which can be determined by test or estimation. The average values shall be taken for all the unit weight values above mentioned.

2. The weights of the structures, including the reinforcing structures, located on/in the slope shall be considered as a part of the dead load of the slope. For the unit weights of all materials, the average values, or the values specified or recommended in Appendix B of DL5077 shall be taken.

#### 7.2.2 Action of groundwater

1. The pressure of pore water, fissure water or artesian water in the slope shall be determined according to the hydrogeological data and the long-term observation record. The highest groundwater recorded shall be taken as the sustained table; the short-time high groundwater table caused by the extra heavy rain or the long-duration rain or the possible atomized rain created by the flood discharging shall be taken as the transient table, as shown in Appendix F.

2. For the slope installed with drainage facilities, the groundwater table after drainage shall be ascertained before determination of groundwater pressure. In order to improve the reliability of calculation, the groundwater pressure shall be multiplied by a coefficient greater than 1 according to the practical situation. The local failure of drainage or uncompleted drainage during construction shall be taken as the transient condition.

3. The groundwater pressure acting on the surface of thorough discontinuities and the fissures developed in the strongly unloaded zone of the rock mass below the groundwater table shall be determined by means of interpolation or extrapolation from the isolines of the groundwater.

4. For a deep-seated potentially instable mass of a rock slope, if its boundary planes do not penetrate entirely, the fissure water pressure may be reduced correspondingly by multiplying a reduction coefficient less than 1 as the approach used for reducing the water

pressure acting on an impervious curtain for a dam or acting on an underground chamber.

5. For the rock/soil mass under groundwater table with seepage flow, if the volume force method and buoyant unit weight are used for analysis, the seepage pressure shall be taken into consideration. For the slide mass above the river water level, the seepage pressure or hydrodynamic pressure  $P_{wi}$  shall be calculated according to the following formula:

$$P_{wi} = \gamma_w V_i J_i \quad (7.2.2)$$

where:  $\gamma_w$  — unit weight of water ( $\text{kN/m}^3$ );

$V_i$  — submerged volume of the unit width rock mass of the  $i$ -th slice ( $\text{m}^3/\text{m}$ );

$J_i$  — gradient of groundwater in the  $i$ -th slice.

6. The groundwater level in the reservoir banks after impoundment should be determined by actual observation. In case the actual observation record is not available or the reservoir has not been impounded, the groundwater level may be estimated by submerge calculation. Attention shall be paid to the possible variations of river or reservoir levels and groundwater level during construction and operation, and their influence on the physical and mechanical properties of rocks and soils.

7. For slope stability analysis under the situation of the short-time rising of groundwater caused by either the natural rain or the atomized water due to flood releasing, and rapid drawdown of reservoir level, the minimum average value for permeability coefficient shall be taken and the groundwater level should be estimated according to the unstable seepage flow. The transient groundwater pressure caused by raining may be estimated by the methods recommended in Appendix F of this Specification.

8. For slopes subject to atomized raining induced by flood releasing, the influencing extent and intensity of atomized raining shall be estimated firstly according to experiences and project analog, and then the transient groundwater pressure shall be estimated in reference to Appendix F.

### 7.2.3 Reinforcement action

1. Reinforcement action means the forces imposed on the unstable (or potentially unstable) rock mass by the reinforcing structures to fix the rock mass with the underlying stable rock mass.

2. For calculation of the factor of safety, the reinforcing forces shall be treated as additional slide-resisting forces.

### 7.2.4 Seismic actions

For the area with a basic seismic intensity of VII or higher, the seismic forces shall be included in calculation. The action of earthquake on slopes and the aseismatic design shall be in accordance with DL5073.

7.2.5 For slope design, two kinds of action combinations shall be considered:

1. Basic combination: deadweight + external water pressure acting on the bank slope +

groundwater pressure + reinforcing forces

2. Accidental combination: Basic combination + seismic action

**7.2.6** The slope engineering shall be designed according to the following three working conditions:

1. Sustained conditions: mainly include the normal operation condition, for which the basic combination of actions shall be applied;

2. Transient conditions: include conditions without reinforcing forces totally or partially and the ground water rising due to lacking of drainage measures or the construction water infiltration during construction; the ground water rising due to rainstorm, durable raining, atomized raining by the flood discharging, as well as malfunction of drainage facilities during operation period; and rapid draw down of reservoir level. Under the abovesaid conditions, the basic combination of actions shall be applied;

3. Accidental conditions: mainly include the conditions of earthquake or the emergency emptying of reservoir. Under these conditions, the occasional action combination shall be applied.

### **7.3 Physical and mechanical properties of rock, soil and reinforcing structures**

**7.3.1** The applied physical and mechanical parameters of the rock mass and soil mass for slope stability analysis shall be selected according to the engineering geological conditions and the recommended values. In case these parameters are not available, they may be selected by reference to the recommended values in Appendix G and Appendix H.

**7.3.2** Generally the effective stress method is used for slope stability analysis. The shear strength  $\tau_f$  of the sliding plane shall be determined by the following formula:

$$\tau_f = c' + \sigma' \tan \phi' = c' + (\sigma - u) \tan \phi' \quad (7.2)$$

where:  $\sigma$ ,  $\sigma'$  —total and effective normal stresses on fracture plane respectively;

$c'$  —effective cohesion;

$\phi'$  —effective internal friction angle;

$U$  — pore water pressure.

**7.3.3** For stability analysis of rock slopes, the effective stress method shall be used. The in-situ shear test of weak discontinuities shall be carried out with slow rate; the drained and undrained direct shear test shall be carried out in laboratory. For the undrained direct shear test, the pore pressure shall be measured to obtain the effective stress strength.

**7.3.4** For stability analysis of soil slopes, in case the pore pressure can be determined, the effective stress method should be adopted and the effective shear strength parameters shall be used. In case the stability is analyzed by the total stress method, the total stress strength shall be used.

**7.3.5** For clayey soil slopes, the total stress method may be adopted for analysis by using the strength parameters of undrained shear tests (triaxial shear tests) or quick shear tests

( direct or in-situ shear test) under the following conditions:

1. For slopes rapidly filled up with unsaturated clayey soil, the shear strength  $\tau_f$  of the fracture plane shall be determined by the following formula:

$$\tau_f = c_{uu} + \sigma \tan \phi_{uu} \quad (7.3)$$

where:  $\sigma$  — total stress on the fracture plane;

$c_{uu}$ ,  $\phi_{uu}$ —cohesion and internal friction angle obtained by the unconsolidated-undrained shear test, respectively.

2. For slopes formed by rapid filling on or cutting in the saturated clayey soil, the shear strength of the fracture plane shall be determined by the following formula:

$$\tau_f = c_{cu} + \sigma'_c \tan \phi_{cu} \quad (7.4)$$

where:  $\sigma'_c$  —effective stress on the fracture plane before load variation;

$c_{cu}$ ,  $\phi_{cu}$ —cohesion and internal friction angle obtained by consolidated- undrained shear test, respectively.

3. For water retaining slopes composed of saturated clayey soil, when the water level outside the slope drops down rapidly, the consolidated-undrained shear strength should be used.

**7.3.6** The in-situ test in the sandy or saturated clayey soil foundation should be carried out by the following approaches:

1. The standard penetration test, the static penetration test and the large-scale cone penetration test are mainly used for the sandy soil, from which the effective friction angle  $\phi'$  of the soil is obtained;

2. The vane-shear test, the static penetration test and the pressure meter test are generally used for the saturated clayey soil, and the result obtained is the consolidated-undrained total stress strength  $\tau_f$  at different depth of the foundation soil. The result may be used for total stress analysis, i.e., the strength parameters at different depths of foundation are  $c_{cu}=\tau_f$  and  $\phi_{cu}=0$ .

**7.3.7** For deforming and failed slopes, the strength parameters of the sliding plane under critical state may be obtained by back calculation. When these parameters are used for slope stability analysis, they shall be deducted properly with a deduction coefficient of 0.8 in general. The parameters obtained by back calculation with two-dimensional analysis shall not be used for three-dimensional analysis, and vice versa.

**7.3.8** The characteristic parameters of strength and deformation of concrete and reinforced concrete used in the reinforcing structures for a slope shall be determined in accordance with DL/T5057.

**7.3.9** The characteristic parameters of strength and deformation of rock bolts (anchor cables) used in the reinforcing structures for a slope shall be determined in accordance with DL/T5176.

## **7.4 Slope stability analysis against sliding**

**7.4.1** The basic methods for slope stability analysis specified herein are the two-dimensional limit equilibrium methods of lower bound solution, while the methods of upper bound solution may be used in case of sufficient demonstration, for which the design factor of safety maintains unchanged as specified in Table 5.2. When more than one method is used for analysis, the maximum value obtained by the lower bound solution methods shall be taken but the value shall not be higher than the minimum value obtained by the upper bound solution methods; and the minimum value obtained by the upper bound solution methods shall be taken. The limit equilibrium methods recommended by this Specification are shown in Appendix E.

**7.4.2** Generally the two-dimensional strain analysis is used for slope stability analysis. The three-dimensional stability analysis shall be conducted with the same strength parameters in case the three-dimensional effect is evident, for which the design factor of safety remains unchanged as specified in Table 5.2.

**7.4.3** For two-dimensional analysis, provided the factors of safety for the representative profiles of various sections of a landslide or a potentially instable rock mass are different from each other, which is obtained with the same calculation method, the overall factor of safety may be calculated on the average weight of various sections with the weighted average method according to the rock mass weight of the various sections, or it may be expressed with a calculated range. When the difference among the factors of safety is relatively large, local stability shall be studied.

**7.4.4** The stability analysis of landslides shall be carried out in accordance with the following stipulations:

1. For rockslides or soil slides with circular slip surfaces, the simplified Bishop method is recommended and Janbu method may also be used. For slopes with composite slip surfaces, the Morgenstern-Price method is recommended and the method of transmission coefficient (method of unbalanced thrust) may also be used.

2. For landslides with secondary slip surfaces, the overall stability and local stability along different slip surfaces or combined slip surfaces shall be calculated and analyzed.

3. For landslides with definite slip surface, provided the design factor of safety has been satisfied after stabilization treatment, the probability of a failure along some newly generated slip surfaces inside a slope shall be checked.

**7.4.5** For rock slopes, the stability analysis shall be carried out in accordance with the following stipulations:

1. For newly cut slopes or long-term steady natural slopes with intact rock mass, their stability may be analyzed by upper bound solution methods. The recommended methods include Sarma method with oblique slices, Pan's block limit equilibrium method and the energy method (EMU). The obliquity of slices or block surfaces shall be determined according to the occurrence of the associated discontinuities in the rock mass.

2. For weathered and stress released natural slopes, slopes excavated without

pre-splitting and preventive measures, and slopes with loosened structure or obvious deformation evidences, the lower bound solution method shall be used for the stability analysis. The recommended method is Morgensern-Price method, and Janbu method and the transmission coefficient method may be used.

3. For potentially instable wedges, the wedge analysis method is recommended.

4. In case there are many controlling weak planes in the rock slope, the stability analysis shall be carried out on the possible combination of these weak planes to assess the integral and local stability of the slope.

5. For slopes with cataclastic structure, loose structure or stratified structure with multiple slip surfaces of an identical dip, the stability analysis shall be carried out by means of trial calculation to determine the most critical slide surface and the corresponding factor of safety.

**7.4.6** For soil slopes, the stability analysis shall be carried out in accordance with the following stipulations:

1. For deposits of sand, debris or gravel, the calculation should be conducted as per planar slip surface, and the factor of safety against sliding is defined as the ratio of the tangent of internal friction angle to the tangent of the slope inclination.

2. For clayey soil, mixed soil and homogeneous soil deposits, the calculation should be made as per circular slip surface, and the stability analysis should be carried out by lower bound solution methods. The simplified Bishop method is recommended to determine the most critical slip surface and the factor of safety, and the Janbu method may also be used.

3. In case the sliding occurs along the bottom of the soil deposit or along a definite weak plane inside the slope, the lower bound solution method should be used in calculation as per composite slip surfaces. The Morgenstern-Price method is recommended, and the method of transmission coefficient may also be used.

4. In case the failure occurs in dense soil or dense deposits, the stability analysis can be carried out with the upper bound solution method. The energy method (EMU) is recommended to determine the most critical slip surface and the associated factor of safety.

5. For slopes composed of homogeneous soil or multi-layer soil, the most critical slip surface and the associated factor of safety shall be determined with a trial calculation.

## **7.5 Stress-strain analysis of slopes**

**7.5.1** For important or geologically complicated slopes, they may be assumed as continuous or discontinuous media, and the numerical method shall be adopted for analyzing the deformation, stability and movement modes, such as the finite element method, the discrete element method, the block element method, the finite differential method and the flow element method.

**7.5.2** The extent of stress-strain analysis of a slope shall cover the height and depth influenced by gravitational stress. If necessary, the necessity of a three-dimensional analysis shall be considered.

**7.5.3** The grid division of the finite element shall meet the simulation requirements of the strata, controlling discontinuities, sliding resistant mass, drainage tunnels and holes, and the accuracy requirements of the stress and displacement calculation.

**7.5.4** The soil or rock mass of a slope may be generalized into isotropic, anisotropic or orthotropic continuous elements according to their properties. The weak planes or controlling discontinuities may be generalized into joint elements. The elastic-plastic or the nonlinear constitutive relations shall be selected according to the stress-strain relationship determined by geotechnical tests.

**7.5.5** The physical and mechanical parameters of the rock mass or soil mass of slopes shall be determined according to the following stipulations:

1. For specific strata, discontinuities and sliding resistant masses, the physical and mechanical parameters shall be selected in accordance with the relevant specifications. For faults with multiple zones, simplification should be made by using the averaged thickness and equivalent deformation modulus.

2. For the passive sliding-resisting structures such as piles and plugs, their shear strength parameters shall be deducted by a structure safety factor. The designed tensile strength of the prestressed cable shall be adopted.

**7.5.6** For common slopes, only the gravitational stress field shall be considered. In case the residual geotectonic stresses exist, the crustal stresses obtained by regression analysis of the measured results should be applied on the calculation boundary.

**7.5.7** Loading or unloading simulation shall be made in accordance with the load variations during the slope excavation, reinforcement and operation.

**7.5.8** The overall factor of safety calculated by finite element analysis shall be calculated by decreasing the strength parameters. The factor of safety achieved when the deformation begins to diverge during calculation shall be taken as the factor of safety of the slope.

**7.5.9** The results of finite element analysis shall meet the following requirements:

1. The initial displacement field of a slope under natural condition is considered as a zero displacement field. The analysis results shall be the stress field and displacement field after variation of the slope and its loading conditions.

2. The results shall include the charts of stress vectors and isolines, the charts of displacement vectors and isolines, the charts of point safety distribution, the charts of distribution of plastic area, tension area, fissures and abnormal deformation.

## **8 Slope treatment design**

### **8.1 General principles**

**8.1.1** In design of hydropower and water resources projects, the followings shall be done: identifying the objects jeopardized or affected by slopes; defining the types and safety classes of slopes; determine the design factor of safety; and analyzing the risk of slope failure. Slope treatment includes excavation, drainage and cutoff of surface water and ground water, slope reinforcement and supporting.

1. The extent and mode of probable failure, the accumulation shape after failure and the possible damage brought by the failure shall be estimated and assessed.

2. For slopes to be strengthened, the reinforcing measures such as piles, anchors or their combinations shall be compared, and then budget and prediction shall be given taking construction conditions, construction period, cost and effectiveness into account. The benefit-cost analysis shall be conducted.

3. Alternatives of avoiding risks or reducing protection criteria may be accepted in combination with intensified monitoring, prediction and early-warning measures in order to avoid or minimize the damages.

**8.1.2** A slope which needs to be treated shall be zoned according to the engineering geological conditions, rock mass types, and deformation and failure modes. The treatment target and criteria shall be made clear and definite, based on which a unified plan and a basic scheme for slope treatment shall be worked out.

**8.1.3** In slope treatment design, the environment protection must be considered, and the state and local government laws and regulations must be obeyed.

**8.1.4** When a natural slope can not meet the design requirements in the aspects of stability and deformation, priority shall be given to stabilizing measures increasing the stability of the rock mass or soil mass itself, mainly including decreasing ground water pressure (surface and sub-surface drainage) and flattening slope (cutting at top and counter weight fill at toe). In case these measures are difficult to be put into effect or still cannot meet the design requirements, the reinforcing measures shall be additionally adopted. The stability and deformation analysis shall be carried out in steps according to the execution process of the measures.

**8.1.5** The unfavorable influences of engineering activities on a slope shall be analyzed. In case high pressure grouting or water pressure test is necessary in the upper part of a potentially unstable slope, reliable monitoring shall be provided and preventive measures must be taken.

### **8.2 Basic stipulations**

**8.2.1** The slope treatment works shall be designed according to the topographical and

geological conditions in combination with the layout of the hydraulic structures or other structures, slope types (permanent or temporary), and local conditions.

**8.2.2** The force against sliding for a slope shall be determined based on the stability analysis and the design factor of safety. The residual sliding force of each slice, which needs to be balanced to achieve the design factor of safety, shall be calculated by the method of slices. The slide resistant structures and their locations and depths shall be selected and determined to fit the geological and construction conditions. The forces against sliding offered by different structures shall be calculated according to the principle of force composition.

**8.2.3** The force against sliding or the prestressed anchoring force offered by slide resistant structures shall be obtained in the way that the needed force against sliding required by the design factor of safety divided by a strength utilization factor of less than 1, or multiplied by a strength reserve factor of more than 1 (i.e., structure partial factor in the limiting bearing capacity analysis), depending on the type, structure and materials of the slide-resistant structures.

**8.2.4** For systematic or local anchorage structures used for stabilization of rock blocks in the superficial layer of a slope, such as systematic anchor bars or micro-piles, the depth and anchor capacity shall be determined according to experiences and estimation, taking the following factors into considerations: actual location, thickness of moveable blocks, thickness of weathered and stress-released rock mass, or buried depth of poor discontinuities and their strength. If necessary, the stability analysis of rock blocks shall be made. The anchorage depth and capacity shall be determined so as to meet the design factor of safety.

## **9 Slope excavation design**

**9.0.1** In layout selection and structure design, the high and steep cut slope shall be avoided as far as possible.

If a high slope is necessary, stability of the cut slope shall be studied sufficiently according to the geological conditions and the geotechnical characteristics, and the shape and inclination of the slope shall be determined according to empirical judgment or stability analysis.

**9.0.2** For a cut slope, the deep and thick deposits, the major faults and the area where unfavorably consequent weak layers are developed shall be avoided. In the area of high crust stress, the relationship between the orientations of the slope and the crust principle stresses shall be studied, and measures shall be taken to avoid or prevent serious unloading effect induced by cutting.

**9.0.3** For a cut slope, the shape, berm width, flight height and gradient shall be designed by referring to slope inclination recommended by geologist, fitting to the project layout and construction conditions and methods, and the requirements of monitoring, maintenance and inspection. Commonly the width of the berm should be no less than 2m, and the height of the flight should be no more than 30m for rock slopes and no more than 10m for soil slopes.

**9.0.4** Generally, the slope inclination adopted shall make the cut slope self stable under the drained situation. For a slope with local geological defects, its self stability shall be ensured with the support of temporary bolts and shotcrete. For a slope of consequent bedding structure, the orientation and dip of the bedding planes shall be taken into account to avoid cutting through the toe possibly.

**9.0.5** In design of a cut slope, removal and cleaning-up of the resting dangerous rock mass and blocks on the cut slope shall be considered first. Then the slope inclination shall be designed according to the geotechnical characteristics of the rock and soil mass, the conditions of weathering and unloading, the conditions of discontinuity development, and the requirement of the slope self-stability. The slope cutting shall be executed in benches from top to down.

The controlled blasting shall be required for slope cutting. For the locations with unfavorable combination of discontinuities, serious stress releasing and cracking, possible sliding, toppling and buckling, as well as for the area near the cutting peripheries and the portal of opening, the technique of advanced bolting, pre-reinforcing or anchorage in time shall be adopted.

**9.0.6** In case, tensile cracks, local slip or collapse failure occurs during or after the slope cutting, analysis shall be made to find out the causes including influences of construction execution. Then, the remedial measures shall be studied and the corresponding supplementary design document shall be proposed.

**9.0.7** The small-sized instable rock mass or soil mass, deposits, talus and landslides should be entirely removed. For the instable rock mass or soil mass, deposits and landslides with a

large volume, study on the possibility of change in the slope shape shall be first conducted so as to improve the slope stability.

**9.0.8** The slope stability may be improved by cutting (for unloading purpose), flattening, counter weight at toe and their combinations according to the situation of the surrounding objects to be protected, the structure layout and the topographic conditions. The location, shape and volume of earth-rock excavation and fill shall be demonstrated by stability analysis and calculation.

**9.0.9** The materials filled at a slope toe shall be permeable enough; otherwise a special pervious layer composed of rock blocks, debris or sand and gravels shall be provided. The filling materials shall be compacted layer by layer, and the necessary measures such as water catch, drainage and surface protection shall be taken.

## **10 Slope drainage design**

### **10.1 Surface drainage**

**10.1.1** In comprehensive treatment of a slope, surface water intercepting and draining system shall be designed to suit the local topographical and geological conditions.

**10.1.2** The following items shall be included in design of surface water intercepting and draining system:

- 1 Water intercepting and draining ditches outside the boundary of the cut or treated slope;
- 2 Water intercepting and draining ditches inside the boundary of the cut or treated slope;
- 3 Surface waterproof measures such as crossing structures, filling and compaction, etc.

**10.1.3** The design criteria of the discharge flow for the surface water intercepting and draining ditches shall be determined according to a comprehensive analysis on the following factors: slope importance, local rainfall characteristics, catchment area, influence of surface water infiltration on the slope stability, etc. Generally the discharge flow is calculated according to two to twenty-year rain intensity. A special study on design criteria of discharging flow shall be conducted for slopes influenced by atomized water.

**10.1.4** The sectional size and gradient of water intercepting or draining ditches shall be determined according to the results of hydraulic calculation and the topographic conditions.

**10.1.5** The water intercepting and draining ditches should be arranged in the way that the surface flow is directed to the nearby gulley or stream without any scouring, for which the energy dissipating and erosion protecting measures may be taken if necessary.

**10.1.6** The water intercepting and draining ditches should be of trapezoid or rectangular cross section. Cemented masonry or concrete may be used as facing material in a thickness of no less than 20 to 30mm. The strength grade of mortar or concrete should not be less than C15.

**10.1.7** The pervious faults, joints, fissures and cracks in slope surface should be sealed with compacted clay, mortar, concrete, bitumen or other materials. In case the intercepting or draining ditches cross these geological defects, the crossing structures shall be provided.

**10.1.8** The water storage and supply facilities should be located on stable slopes with good drainage conditions, and the leak proofing measures shall be taken. Drainage ditches shall be arranged for the water storage and supply facilities and connected with the slope drainage system so as to prevent the leakage and spilled water from entering the slope.

### **10.2 Subsurface drainage**

**10.2.1** Layout of the subsurface water intercepting and drainage system shall be designed in accordance with location of the slope, its relations with structures, and the engineering geological and hydro-geological conditions.

**10.2.2** The subsurface water intercepting and drainage engineering measures mainly include water catch and percolation ditches, drainage holes, wells and tunnels.

**10.2.3** For important slopes, a subsurface drainage system comprising multi-level drainage tunnels should be provided. If necessary, a drainage curtain comprising drainage holes may be adopted, connecting the tunnels on different levels. The height difference between two adjacent drainage tunnels should be no more than 40m. The impervious curtain and the drainage system in the slope of dam abutment shall be designed according to the design specification of dams.

**10.2.4** The systematical drainage holes shall be provided for bolting-shotcreting supports, girder grillages and retaining walls of a slope; if necessary, filter shall be provided. The systematical drainage hole of a rock slope shall have a diameter of no less than 50mm, and a depth of no less than 4m. The altitude angle of drainage holes should not be less than  $5^{\circ}$ .

**10.2.5** The groundwater in rock slopes, deposit slopes and landslides should be drained through tunnels. In layout of drainage tunnels, discharge path for the perched water upon the confining weak layer, slip surface or slip zone and the confined water beneath them shall be considered.

**10.2.6** The cross section of the drainage tunnel should not be less than  $1.5\text{m} \times 2\text{m}$  (width  $\times$  height) and an inspection access shall be provided. The gradient of a drainage tunnel should not be less than 1%. At one side of the tunnel, a discharge ditch shall be provided so as to have the water flowing out by gravity possibly.

**10.2.7** The inlet portal of the drainage tunnel should be located in stable rock mass. The main tunnel shall be arranged in the bottom wall of the slip surface and parallel to its strike. The branch tunnels shall be normal to the strike of the slip surface and penetrate the weak confining layer or slip zone. In case the surrounding rock mass is less pervious and the effect of drainage is not obvious, the radial drainage holes in crown and sidewalls of the tunnel shall be drilled, with a diameter of not less than 50mm. The drainage holes shall be protected with filters.

**10.2.8** In case the drainage tunnel passes through fractured rock mass or weak layers, necessary lining shall be provided, and the drainage holes shall be protected with filters.

**10.2.9** In case the elevation of the drainage tunnel is lower than the surface drainage, water collection wells with sufficient capacity shall be provided in the tunnel, and the water shall be pumped out.

**10.2.10** The percolation ditches or trenches may be used at the periphery of soil slopes or landslides for intercepting and draining the groundwater in superficial layer. The percolation ditches or trenches should be no deeper than 3m, and shall be backfilled with pervious sand and gravels and sealed at surface with compacted clayey soil of about 0.3m thick.

**10.2.11** Drainage wells may be provided for lowering the groundwater table in soil slopes or landslides. However, the excavation, supporting and dewatering during construction are rather difficult and pumping facilities are required during operation, hence, whether the drainage wells are provided or not shall be considered cautiously.

## **11 Slope reinforcement design**

### **11.1 Slope surface protection**

**11.1.1** If a slope with damaged surfaces will impact the safety of project, the surface protection design shall be prepared.

**11.1.2** The protecting measures of slope surface include following ones: shotcrete, facing concrete, molding-bag concrete, gabions, masonry, geotextile and vegetation cover. The protecting measures shall be selected in accordance with the topographical, geological and environmental conditions, and the environmental protection requirements.

**11.1.3** For accumulated mass or soil slopes, gabion, masonry, geotextile and molding-bag concrete may be used as their surface protection. All the surface protecting structures shall guarantee their self stability on the slope surface.

**11.1.4** For a rock slope prone to be weathered or with poor integrity, shotcrete combined with surface anchorage may be used as protection.

**11.1.5** For high and steep rock slopes stable but with sporadically unstable or loosened rock blocks on the surface, local removal, local anchoring, rock arresting wires, trap trenches and walls may be used as protection.

### **11.2 Slope superficial reinforcement**

**11.2.1** In case superficial unstable blocks or wedges formed by such unfavorable discontinuities as bedding planes, schistositities, joints, fissures and faults are frequent on the slope and prone to fail in form of sliding, toppling or buckling, the stability analysis and the reinforcement design shall be carried out.

**11.2.2** The superficial reinforcement measures include following ones: bolts, wire mesh, shotcrete, facing concrete and concrete lattice. The reinforcement measures shall be selected and the design parameters shall be proposed in accordance with geotechnical characteristics, slope structure, deformation and failure mechanisms.

**11.2.3** For a slope with its superficial layer relatively intact, systematic or random bolting measures may be taken for reinforcement. For a slope with its superficial layer highly weathered or fractured, such combined reinforcement measures shall be taken as bolting with wire mesh and shotcrete, or bolting with facing concrete, or bolting with concrete lattice.

**11.2.4** The depth of bolts for reinforcement of superficial layer may be determined according to the buried depth of instable blocks, the weathered extent of rock mass or the

depth of unloaded and loosened rock mass. The tension-sheared bolts should be adopted with the anchorage orientation and optimal angle determined according to the sliding direction of instable block and construction conditions.

**11.2.5** The diameter and spacing of bolts shall be determined based on the calculation results of instable block sliding force or by engineering analogy.

**11.2.6** In case facing concrete or concrete lattice plays the role of sliding resistance, their capacity of bending resistance and shearing resistance shall be calculated.

**11.2.7** The facing concrete or concrete lattice shall be competent to keep their own stability on the slope surface and be fixed with systematic bolts.

### **11.3 Slide-resisting piles**

**11.3.1** In case of a reliable anchorage force can be provided by a stable bedrock or dense and compact soil layer underlain the slide surface of a slope, slide-resisting piles may be used for slope reinforcement. The slide-resisting pile design and calculation shall comply with the specifications of Appendix I.

**11.3.2** According to the engineering geological conditions and the sliding force, the following reinforcement measures may be chosen: cantilever piles, chair-shaped pile-wall, gate-shaped steel frame piles, bent piles, bolting piles, pile-plug combined structure and prestressed cable piles.

**11.3.3** The slide-resisting piles shall be arranged in accordance with the following specifications:

1. The piles should be set at the slide-resisting zone in the front fringe of slope or at the front part of sliding section;
2. The piles shall be arranged in rows in the extending directions perpendicularly to the main sliding direction.
3. The net spacing of pile should be 5m~10m.

**11.3.4** The length of a slide-resisting pile should not be more than 40m. The embedded length below the sliding surface shall be determined according to the strength and deformation characteristics of rock mass or soil mass, 1/3~2/5 of the total pile length in general, or 1/4 of the total length in competent rock mass.

**11.3.5** Generally the cross section of a slide-resisting pile is rectangular with its shorter side perpendicular to the sliding direction. In case the sliding direction is changeable or uncertain, a circular cross section may be adopted.

**11.3.6** The driving force borne by a single pile in one row shall be determined based on the

sliding force distributed in the region with a width of two 1/2 center distances from each side of the pile center, and the requirement of the design factor of safety shall be considered in calculation.

**11.3.7** The ground pressure at the bottom and the lateral sides of a pile shall be less than the design strength of the corresponding rock mass or soil mass.

**11.3.8** The cross section area, concrete strength class and reinforcement ratio for a pile shall be determined based on the shear force and bending moment acting on it according to the specifications of DL/T5057.

**11.3.9** In case the slide-resisting piles are used as foundation piles of a structure, the vertical loading capacity, subsidence, horizontal displacement and deflection for the pile foundation shall be checked according to JGJ 94 “Technical specifications for pile foundation of buildings” and the influence of additional loading at ground surface on the stress condition and stability of piles shall be considered.

**11.3.10** In case the groundwater is chemically corrosive, the corrosion-resisting measures shall be taken for the pile concrete.

#### **11.4 Shear-resisting plugs and retaining concrete plugs**

**11.4.1** For originally stable slopes, provided the slope is likely to slide along a deep-seated weak discontinuity, while the rock mass at top wall and bottom wall of the discontinuity is sound and intact, shear-resisting plugs may be used for reinforcing the slope. Shear-resisting plugs shall be set horizontally along the strike of potential slide plane, and the potential slide zone shall be replaced by concrete or reinforced concrete.

**11.4.2** Retaining plugs formed by backfilled tunnels should be located where the slide plane is relatively steep and the surrounding rock mass is sound and intact, and the axis of the plug shall be parallel to the sliding direction. The retaining plugs should be inclined toward the slope and perpendicular to the sliding surface as far as possible in order to avoid tension and shear failure occurred under sliding force. Plugs reformed from existing exploration adits or construction access adits, or combined with drainage tunnels, should be only served as one of reinforcing measures, and be used for calculation of stability against sliding together with other reinforcement measures.

**11.4.3** For shear resisting plugs or retaining plugs, the sliding potentiality of slope, along some secondary slide surface in the rock-soil mass above or below the shear resisting plug, or retaining plug or along the contact borders between concrete and rock-soil mass, shall be checked.

**11.4.4** The cross section of a shear resisting plug or a retaining plug shall be determined on the basis of slope stability analysis. The calculation of reinforced concrete backfilled shall satisfy the requirements of DL/T5057.

**11.4.5** The crown of a shear resisting plug or a retaining plug shall be backfilled with grouting. If necessary, consolidation grouting in surrounding rock mass may be carried out.

**11.4.6** The embedded depth of a shear resisting plug in hard rock mass at top wall and bottom wall of the sliding plane shall be no less than 3m. A retaining plug shall have an adequate embeded depth in stable rock mass, generally not less than 2 times of the plug diameter.

## **11.5 Prestressed anchor cables**

**11.5.1** Prestressed anchor cables is a kind of active slide resisting structures and applicable for slope reinforcement or pre-reinforcement where prestress can be applied.

**11.5.2** The total design anchoring force of prestressed anchor cables shall be determined according to the slope stability and stress-deformation analysis. The cable arrangement and design parameters shall be determined according to the geotechnical properties of ground mass and the construction conditions.

**11.5.3** For a rock slope, the total slide resisting force shall be calculated as the sum of two forces: one is the component derived from the total anchorage force acting along the slide plane, the other is the slide resisting force generated from the force normal to the slide plane. For a soil slope or accumulated slope, only the slide resisting force acting along the slide surface shall be calculated.

**11.5.4** The prestressed cables should be spaced at 4m~10m. In case the spacing is less than 4m, the interference effect of multi-anchors shall be analyzed. The length of prestressed anchor cable should not be more than 50m. The design capacity of a single cable should not be larger than 3000kN.

**11.5.5** The direction of bore hole for prestressed anchor cable shall be determined according to the potential sliding direction of slope; the dipping angle of bore hole shall be determined according to the slope stability analysis, topography and geology conditions, and construction conditions.

**11.5.6** For bore holes of freely grouted anchor cables, the dip angle should not be less than 10° dipping into slope, otherwise the pressure grouting with grout stop ring shall be conducted.

**11.5.7** The length of inner anchorage section may be determined based on the calculation

of the bonding strength between mortar and cable or mortar and rock, or by project analogue or by in-situ pull-out tests.

**11.5.8** The anchorage section of prestressed cable shall be set in stable rock mass inside the slope. The inner anchorage section may be treated with pre-grouting if necessary. In case the anchorage section is located below the groundwater table, drainage facilities should be provided before anchoring. The contra-pulling prestressed cables should be adopted if conditions permit.

**11.5.9** In case the prestressed cables are installed in groups, any large tensile zone shall be avoided being formed inside a slope. The inner anchorage section shall be set in alternating depths, the depth difference should not be less than 1/2 of the anchorage length. During prestressing the interference effect of adjacent anchors shall be considered so as to guarantee the loading uniformly distributed and generally synchroual.

**11.5.10** The foundation of outer anchor pier for a prestressed cable shall be rigid enough. In case the slope surface is composed of weathered and fractured rock mass or soil and talus accumulation, an outer anchor pier with extended foundation or a composite structure with concrete beams or lattice frame shall be provided.

**11.5.11** Commonly the pretressed anchor cables shall be locked according to the designed capacity. In case the slope rock mass is loosened and the expected prestress loss is relatively great, the cables shall be locked in extra-pretension. In case the rock mass of reinforced slope is intact and hard, the unloading rebound is expected to be relatively great, or the prestressed cables act together with the slide-resistance piles, the cables shall be locked in under-pretension.

**11.5.12** The structure of prestressed cables shall be designed according to the stipulations of DL/T5176. The monitoring measures for the prestressed cables shall be determined according to the importance of the slope.

## **11.6 Slope retaining structures**

**11.6.1** The retaining structures should be provided for the slopes to be protected in the project area, such as soil slopes, talus slopes, backfill slopes or rock slopes with their toes subjected to water scouring, or composed of weathered, fractured or weak rocks.

**11.6.2** The retaining structure shall be designed on the basis of slope stability analysis in combination with other stabilizing measures, such as drainage, load reduction and reinforcement in order to meet the requirement of slope overall stability.

**11.6.3** The retaining structures can be classified into bolted (cabled) retaining wall, resting

retaining wall, gravitational retaining wall and buttressed retaining wall. The bolted retaining wall can be subdivided into retaining walls with ribbed plates, lattice frame, piles in line, nonprestressed bolts and prestressed bolts. The type of supporting structure shall be chosen according to the slope composition, genetic type, slope height and stability.

**11.6.4** During the foundation excavation in soil or accumulation slope, the bolted retaining wall with row piles should be adopted for the high slope with poor stability; and the bolted retaining wall with ribbed plate or lattice frame may be adopted for the fairly stable slope.

**11.6.5** For slopes in water, the protective measures against scouring should be considered for the foundations and the walls.

**11.6.6** For the bolted retaining wall of backfilled slopes, the effective measures shall be considered to avoid an excessively additional tensile stress of anchorage bolts in design and construction. For high slopes newly backfilled, the bolted retaining wall should not be adopted.

**11.6.7** The height of gravitational retaining wall should be no higher than 8m for soil slopes and no higher than 10m for rock slopes. The gravitational retaining wall should not be used for the slope with strictly deformation limit or the toe excavation of which endangering the slope stability.

**11.6.8** The buttress retaining wall is suitable for the slopes filled with earth. The wall height should be no higher than 10m. The foundation of buttress retaining wall shall be located in stable strata.

**11.6.9** The retaining structure is applicable to be used independently as a structure against sliding only for small volume and shallow landslides. The gravitational retaining wall may be adopted for the landslide with a thickness of less than 6m. The bolted retaining wall should be adopted for the landslide with a thickness of over 6m.

**11.6.10** The rock-earth pressure subjected by a retaining structure shall be calculated according to the residual sliding force and the active earth pressure separately, and then the larger one shall be adopted.

**11.6.11** For retaining walls of various types, drainage holes crossing the wall shall be preformed in order to drain ground water behind the wall.

**11.6.12** In the retaining wall design of a slope in water, the unfavorable water pressure inside slope induced by the fluctuation of water table shall be considered. The necessity of drainage measures inside the retaining wall shall be studied.

## **12 Safety Monitoring and Early-Warning System Design**

### **12.1 General**

**12.1.1** The slope monitoring includes the observation in pre-construction stage, the safety monitoring during construction and in the operation period. In design of safety monitoring, the monitoring purpose and contents for different phases shall be considered thoroughly and comprehensively in order to keep continuity and integrity of data.

**12.1.2** The safety monitoring design for slopes in the main structure area of a hydropower or water resources project (Group A) shall be carried out in combination with the safety monitoring design of the dam, and the slope monitoring system shall be included in the dam monitoring system. The safety monitoring design for slopes in the reservoir area (Group B) shall be carried out in combination with reservoir bank protection, land utilization and resettlement planning.

**12.1.3** The purpose and required accuracy of monitoring shall be defined and a reasonable safety monitoring program and monitoring system shall be prepared according to the type, class, geological conditions, deformation and failure mechanism, and failure risks of slopes,. The geological visual inspection and judgment shall be included in the safety monitoring system.

**12.1.4** The monitoring items and the arrangement of monitoring points shall be able to reflect the deformation and displacement of slopes and the load-bearing characteristics of reinforcement structures. The surface and subsurface monitoring shall be combined to establish a tridimensional monitoring system. The monitoring items should be in small numbers but in high precise.

**12.1.5** The measuring range and accuracy of monitoring apparatus shall be determined on the basis of the deformation and stability analysis or the engineering experiences, and then the instrument type will be selected. Under the precondition that the monitoring requirements are met, the apparatus shall be simple, durable and easy to maintain.

**12.1.6** The protecting devices shall be provided for the monitoring apparatus to avoid any damage. For the steep slopes with poor stability, measures shall be taken to protect the safety of monitoring staffs. The roads for monitoring and tour inspection shall be safe.

**12.1.7** The monitoring cycle shall be determined according to the engineering experiences and the monitoring results of early stage. The observation results shall be collected and analyzed in time. The observation results obtained in the construction period shall be fed back to the designer and the contractor in time so that they can regulate and modify the design and construction schemes for the slope excavation and treatment if necessary. The observation results in the operation period shall be timely reported to the competent authority.

### **12.2 Safety monitoring system**

**12.2.1** The design of safety monitoring system shall be carried out according to the

characteristics of slopes. For the common slopes, the integrate stability shall be the main monitoring item, at the same time, attention will be given to the local stability. The deformation of slope is the main item of stability monitoring. For the landslide with determined slide surface, the deformation of ground surface is the main monitoring item.

**12.2.2** For the slopes of classes 1 and 2 or the slopes with a height of more than 100m, the following observations shall be carried out, for the other slopes the observation items may be reduced properly.

1. Displacement and deformation monitoring

External monitoring: displacement and subsidence of the slope surface; cracking length and aperture at the slope surface.

Internal monitoring: underground deformation; movement of slide plane or faults.

2. Ground water monitoring: underground water table or water pressure; spring discharge; hydrochemistry.

3. Monitoring of reinforcement structures for slopes: stress and strain of slide resisting piles, shear resisting plug, retaining plug, anchoring bar and cable and retaining wall.

4. Other specific items: rain fall, ground stress and seismic activity, etc, made for the slopes of important projects.

**12.2.3** One or more representative profiles shall be established according to the slope geological conditions and reinforcing structure features and no less than 3 monitoring points should be provided for each profile. The monitoring profiles shall be combined with the exploration profiles and the stability analyzing profiles as much as possible. The arrangement of monitoring points on ground surface shall be incorporated with the monitoring points for underground deformation so that the correlation of displacement and deformation between ground surface and underground can be established.

**12.2.4** Ground surface displacement monitoring: triangulation and leveling networks shall be established and the geodetic surveying method shall be adopted for the important slopes. The simple methods such as the collimation line method may be used for the common slopes.

**12.2.5** Monitoring of surfacial and deep cracks: 3-dimensioned or simple crack meters may be used depending on the importance of slopes. For the cracks on ground surface, their extension, number and length shall be monitored in geological tour inspection.

**12.2.6** Subsurface deformation monitoring

1. Borehole inclinometer: inclinometers are used for investigation of the slip plane location or the relative displacement of multiple-level slip planes. The max. depth of an inclinometer hole should not be more than 90m. For monitoring the slopes higher than 100m, combination of multi-step inclinometer holes may be adopted. Since the measuring range of inclinometer is limited, it should not be used for the slides with large displacement. The inclinometer hole may be combined with the groundwater observation hole, which is favorable for the multi-factor correlation analysis.

2. Multipoint-extensometer: multipoint-extensometers are used for investigation of tensile deformation of slopes. The borehole depth, number and location of measuring points in the hole shall be determined according to the results of stability and deformation analysis

and by analogy with similar projects.

3. For the slopes of important projects, inclinometers, converge-meters and extensometers may be used for monitoring in a monitoring gallery. In special situations the plumb-line may be used for monitoring in a vertical hole.

**12.2.7** For slopes of Classes 1 and 2 or slopes higher than 100m, the groundwater table or pore water pressure, seepage flow from gallery or ground surface shall be monitored. During rain season or in the period of reservoir level fluctuation, the monitoring frequency shall be increased. The leakage into a gallery from reservoir should be observed automatically by setting a specific measuring weirs.

**12.2.8** For the slopes of important projects, the natural precipitation or the splashing and atomized water during flood discharging shall be monitored, and the correlation analysis shall be carried out between the measured results and the slope deformation.

**12.2.9** For the slopes reinforced by anchorage bolts (cables) or concrete slide resisting structures, the hydrochemistry of groundwater shall be monitored.

**12.2.10** Monitoring of slope reinforcement structures

1. For the slope reinforced with prestressed anchorage bolts (cables), dynamometers (prestress-sensor) shall be equipped for monitoring the bolts (cables), the number of dynamometers shall not be less than 5% of the total bolts (cables).

2. The stress-strain monitoring of slide resisting piles, shear resisting plugs, concrete retaining plugs and retaining walls shall be carried out at the selected representative sections. If necessary, the stress-strain of slope shall also be monitored.

3. The water discharge, water quality and educts in drainage galleries and major drainage holes shall be monitored.

**12.2.11** The geological tour inspection shall be carried out for the following phenomena: topographic and geomorphic changes, recharge and discharge of groundwater, deformation and damage of reinforcement structures and buildings, and changes of slope environment. Based on the inspection findings, the slope stability shall be analyzed and judged.

**12.2.12** For slopes in the area with the basic seismic intensity of VII or higher, or in the area adjacent to the site constructed by explosives, the seismic response or the particle vibration speed shall be monitored.

**12.2.13** For a slope which its stability will threaten the safety of an important project, the monitoring should be complemented by GPS and other methods in addition to the above mentioned ones.

### **12.3 Slope Monitoring and Early-Warning**

**12.3.1** For the slope, its failure will be serious in consequence and will possibly threaten the project safety and cause personnel casualty and property loss, the safety warning level and corresponding early-warning criterion shall be established according to its type and safety class, the deformation and failure mode, and the results of stability analysis and failure risk analysis.

**12.3.2** The main contents of early-warning and forecast include the slope failure time, volume and possible landslide-covered extent as well as the affected area of the

possibly-induced secondary disasters.

**12.3.3** The early-warning of sliding failure of a slope is classified in three levels according to the development levels of the landslide and the corresponding countermeasures which shall be taken:

First level: the abnormal deformation has been noticed and confirmed; a few reinforcing structures have been damaged, such as prestressed cables broken, retaining wall cracked locally etc. The monitoring results shall be reported to the authorities concerned, and the monitoring frequency and monitoring items shall be increased if necessary; and the tour inspection shall be carried out every day.

Second level two: the slope deformation does not tend to converge and the reinforcing structures in local area have been damaged, it is confirmed that a progressive failure of the slope are underway. Continuous monitoring and tour inspection shall be carried out; an internal warning report shall be sent to the local departments concerned; the persons working on and down the slope shall be evacuated.

Third level: it is confirmed that the slope has entered into an accelerating deformation stage and slope failure will occur in 3 to 5 days. The continuous remote monitoring shall be carried out for some representative points. A public warning and alarm in the district concerned shall be issued. All the personnel in the area possibly affected by slope failure shall be evacuated.

**12.3.4** Principles of establishing the early-warning criteria

1. For slopes of the first level, the deformation is strictly limited; the early-warning criteria for the surface deformation shall be established according to the maximum allowable deformation on the ground surface;

2. For slopes, which is failed in the form of sliding, the early-warning criteria shall be established generally according to the critical displacement rate of the representative monitoring points of ground surface;

3. For slopes, the stability is affected significantly by the groundwater, the early-warning criteria about the critical groundwater table or seepage pressure of the instable slope should be established on the basis of the stability analysis and the monitored dynamic rule analysis. For the landslide on the banks of reservoirs or rivers, the warning criteria for rapid drawdown or rate of water level shall be established.

4. For slopes, which is failed in the form of collapse, plastic flow, fail under water scouring or debris flow, the early-warning criteria may be established according to the rainfall intensity or the cumulative rainfall in the period of slope failure.

**12.3.5** The sliding failure of an excavated rock slope may be predicted according to the displacement rate obtained from the displacement-time curves. The early-warning criteria for slope failure shall be established according to the analysis of the stability, failure mode, dynamic variation and actual monitoring results.

## Appendix A (Normative)

### Classification of slopes for hydropower and water conservancy projects

**A.1** Slopes for hydropower and water conservancy projects may be classified according to Table.A.1.

**Table.A.1 Classification of Slopes for Hydropower and Water Resources Projects**

Classification	Name	Definition
Genesis	Natural slopes	Slopes formed by natural agents
	Engineered slopes	Man reformed slope or slope influenced by engineering works
Material	Rock slopes	Slopes consisting of rock mass
	Soil slopes	Slopes consisting of soil or accumulations
	Rock-soil slopes	Slopes consisting of rock and soil
Structure	Slopes with Consequent bedding	Slopes with bedding planes parallel to river valley and dipping outwards
	Slopes with Reverse bedding	Slopes with bedding planes parallel to river valley and dipping inwards
	Slopes with transverse bedding	Slopes with bedding planes perpendicular to river valley and dipping upstream or downstream
	Slopes with oblique bedding	Slopes with bedding planes oblique to stream valley and dipping upstream or downstream
	Slopes with horizontal bedding	Slopes with horizontal bedding planes
Relation to buildings	Slopes of structure foundation	The requirement of stability and limited deformation must be satisfied
	Slopes of structure surrounding	The requirement of stability must be satisfied
	River or reservoir slopes	Stability is required or certain limited failure is allowable
Existence	Permanent slopes	Stability is required during the serve life of project
	Temporary slopes	Stability is required during the construction period
Stability situation	Stable slopes	Stability or limited deformation can be kept
	Potentially unstable slopes	Definite unstable factors existed but the slope is temporarily stable
	Deforming slopes	Slopes with evidence of deformation or creeping
	Unstable slopes	Slopes under overall sliding or local collapse occur from time to time
	Post-failure slopes	Residual slopes after sliding
Height	extreme high slopes	Height >300m
	Super high slopes	Height >100m to ≤300m
	High slopes	Height >30m to ≤100m
	Medium slopes	Height >10m to ≤30m
	Low slopes	Height ≤10m

## Appendix B (Normative)

### Classification of slope structure for hydropower and water conservancy projects

**B.1** The structure of rock slopes for hydropower and water conservancy projects may be classified according to Table.B.1.

**Table.B.1 Classification of Rock Slope Structure for Hydropower and Water Resources Projects**

No.	Structure of slope		Rock type	Rock mass characteristics	Slope stability behavior
1	Massive / Blocky		Magmatic rock, meso-metamorphic rock, thick layer sedimentary rock, thick layer volcanic rock	Discontinuity poorly / fairly developed , mainly hard planes, a few weak planes	Slope failure mainly falls into the category of rockfall and block slide. Stability is controlled by fracture discontinuities.
2	bedding	Consequent bedding	Sedimentary rock with different layer thickness, foliated metamorphic, multiple recurrent eruption volcanic rock	Slope surface and bedding planes dip in the same direction with strike angle generally less than 30°. Bedding fissures or shear zones are developed	Toe cutting may lead to slide. For thinly bedded and steeply dipping strata the buckling failure is prone to occur. The bedding planes, weak intercalations or bedding discontinuity often constitute slip surfaces
		Reverse bedding		Slope surface and bedding planes dip to the opposite direction with strike angle generally less than 30°. Bedding fissures or shear zones are developed	Toppling is easy to occur in steeper strata and is popular in the slope surface composed of phyllite and rock with thin-bedded structure. The stability against sliding is good and controlled by fracture discontinuities.
		Transverse bedding		Strike angle of the slope surface and bedding planes is generally greater than 60°. Bedding fissures or shear zones are developed	The slope stability is good and controlled by fracture discontinuities.
		Oblique bedding		Strike angle of the slope surface and bedding planes is generally greater than 30° and less than 60°. Bedding fissures or shear zones are developed	The slope stability is fair. In the shallow depth of the slope with oblique-consequent bedding structure, wedge slide is prone to occur. The stability of slope is controlled by the combination of the bedding and fracture discontinuities.

		Horizontal bedding		The bedding planes are approximately horizontal. It is mostly of sedimentary rock. The shear zones are generally undeveloped	The slope stability is good. The lateral extension or flow along weak bedding planes may occur.
3	Cataclastic		Fault zone, shear zone, cleavage zone, intensive fissure zone	Discontinuities or original joints and weathered joints are developed. The rock mass is fairly fractured.	The slope stability is poor. Rock-falling and scaling are prone to occur. The stability against sliding is controlled by fracture discontinuities.
4	Loosen		Uncemented fault fractured zone, completely weathered zone, loosened rock mass	Composed of rock fragments, pieces, debris and argillaceous materials	The slope stability is very poor. The circular slide and slide along the bottom of accumulation (deposits or talus) are prone to occur.

**B.2** The structure of soil slopes for hydropower and water conservancy projects may be classified according to Table.B.2.

**Table.B.2 Classification of Soil Slope Structure for Hydropower and Water Resources Projects**

No.	Type of slope	Primary characteristics	Slope stability behaviors
1	Clayey soil	The main component is clayey particles. Generally it is stiff and prone to crack when it is dry; it expands and crumbles when it is exposed to water. The effect of wetting-drying is obvious. Some clay possess macro pore (in the south of Shanxi province); some clay is very stiff (red soil with reticulate pattern in the South China); Some clay is in form of semi rock, but with high content of soluble salt (in the upstream reaches of the Yellow River); some clay possess horizontal lamination (in the downstream reaches of Huai River).	The main factors affecting the slope stability include: the mineral components, especially the content of hydrophilic, expansible and soluble minerals; the joint developing conditions; the action of water; the effect of freezing-thawing. The main failure modes include slide, flaking off due to freezing-thawing and collapse.
2	Sandy soil	The main component is sandy soil with porous structure, its major characteristics is low cohesion strength. The permeability is relatively high. This soil type includes thick residual soil formed by completely weathered granite.	The slope stability is controlled mainly by the factors: grain composition and homogeneity; moisture content; vibration; action of surface water and ground water and compactness. The water saturated homogeneous sandy soil slopes under vibration are prone to be liquefied and slide down. The other failure modes are piping, earth flow; slumping and flaking off.

3	Loess	The main component is silt with homogeneous texture. The content of calcium is generally high. There is no bedding but developed columnar jointing is developed. The natural moisture content is low. It is stiff when dry. Part of loess settles when it is exposed to water. Some loess is in consolidated form. Some times it is composed of multiple-unit structure.	The slope stability is controlled mainly by the action of water. The wetting leads to collapse, the inundation and infiltration result in argillization of the water-resisting clay and lower its strength. The main failure mode includes collapse, cracking, collapse by wetting and landslide.
4	Soft soil	The main components are mud, turf and muddy soils, the shear strength of which is very low. The plastic flow is serious.	The landslide, plastic flowing and slumping are prone to occur. It is difficult to form a slope.
5	Swelling soil	The physical and mechanical properties are special, since it is rich in montmorillonite; the internal friction angle is very low. The wetting-drying effect is obvious.	The change of wetting-drying cycle and the action of water strongly influence the slope stability. The shallow landslide and crumbling are prone to occur.
6	Scree and soil mixed	The slopes are composed of rock fragments and sandy soils or gravelly soils. They can be divided into accumulation, residual and slopes with mixed structure and polygenetic multiple-unit structure.	The slope stability is controlled by the content and distribution of the clayey particles, the moisture content of the soil and the occurrence of the underlying bedrock surface. The landslide and slumping are prone to occur.
7	Rock and soil mixed	The slope upper part is composed of soil while the lower is rock, or the upper is rock and lower is soil (completely weathered rock), or multiple layered.	The occurrence of the underlying bedrock surface, the inundation and infiltration of the water seriously affect the slope stability. The soil mass sliding along the underlying bedrock surface, local slumping and upper lying rock mass creeping or slumping are prone to occur.

## Appendix C (Normative)

### Classification of slope deformation and failure modes and classification of landslides

**C.1** The slope deformation and failure mode may be classified according to Table.C.1.

**Table.C.1 Classification of Deformation and Failure Mode of Slopes**

Failure mode		Characteristics of deformation and failure
Falling		Rock mass falls and rolls down
Slide	Planar	Rock mass slides along a planar surface
	Circular	Rock mass of cataclastic struture or loosen structure or soil mass slides down along a circular surface
	Wedge	A wedge formed by discontinuities slides down along the intersecting line of discontinuities
Creep	Toppling	The superficial layers of slopes with reverse bedding structure bend and tilt outwards
	Buckling	The lower part of rock slopes with consequent bedding structure and the dip angle of the strata are similar to the slope inclination bulge progressively leading to bedding cracking and buckling. The continuing development results in sliding along the bedding at the upper part and crossing the bedding at the toe.
	Lateral extension	For slopes with dualistic structure, the lower soft rock creeps or flows leading to latter expansion, cracking, dislocation and subsidence of the upper rock
Flowing		Talus, accumulated rock fragments, debris and soil flow down from slope

**C.2** Landslides may be classified according to Table.C.2.

**Table.C.2 Classification of Landslides**

Classification	Name	Definition
Origin	Natural	Slides led by natural factors, such as erosion by streams, rainfall, freeze-thaw, earthquake, etc.
	Unnatural	Slides caused by engineering activities, such as excavation, water discharge, construction loading, etc.
Characteristics	Bedding consequent	Rock slides along bedding planes. There is possibility of multi-surface sliding
	Bedding cut	Sliding along discontinuities
	Compound	The slip surface is composed of bedding planes and discontinuities of different types
	accumulation	The slide mainly occurs along the bottom of deposits of different origin, or a circular slide inside the accumulation

	Toppling-slide	Slide along the fractured bottom of toppling, however it occurs rarely
	Buckling-slide	Sliding along bedding plane in the rear part and buckling at the toe forming a compound fractured slide zone
Mechanism	Pushing	The driving force is generated upper pushing the lower part to slide. Mainly integral sliding with high velocity
	Dragging	The lower part slides at first dragging the upper part to lose stability. Mostly disintegrated sliding with low velocity
Behavior	Active	Active or seasonally active landslides
	Inactive	The landslides, for which the factors leading to slide still exist but they are kept in stability for a long time
	Dead	The landslides which have lost the possibility for sliding under natural conditions
Thickness	Shallow	Thickness <10m
	Medium	Thickness =10m~25m
	Deep	Thickness >25m
Volume	Small	$<0.1 \times 10^6 \text{ m}^3$
	Medium	$=0.1 \times 10^6 \text{ m}^3 \sim <1 \times 10^6 \text{ m}^3$
	Large	$= 1 \times 10^6 \text{ m}^3 \sim <10 \times 10^6 \text{ m}^3$
	Very large	$= 10 \times 10^6 \text{ m}^3 \sim <100 \times 10^6 \text{ m}^3$
	Huge	$\geq 100 \times 10^6 \text{ m}^3$

## Appendix D (Normative)

### Reliability assessment of slope stability

**D. 1** If it is necessary to carry out the reliability assessment, the method based on the factor of safety shall be used. The performance functions shall be defined on the basis of the factor of safety

$$F(x_1, x_2, \dots, x_n) - 1 = 0 \quad (D.1)$$

$$\ln F(x_1, x_2, \dots, x_n) = 0 \quad (D.2)$$

where:  $F$ —factor of safety;

$x_1, x_2, \dots, x_n$ —factors influencing the factor of safety, such as dead weight of the rock mass, ground water pressure, shear strength parameters of the rock mass etc.

The corresponding reliability index is:

$$\beta = (\mu_F - 1) / \sigma_F \quad (D. 3)$$

or 
$$\beta = (\mu_F - 1) / \mu_F V_F \quad (D. 4)$$

where:  $\mu_F$  —average of the factor of safety;

$\sigma_F$  —standard deviation of the factor of safety;

$V_F$ —variation coefficient of the factor of safety.

**D.2** A simplified approach recommended by J.M.Duncan can be used to determine the standard deviation of the factor of safety. The steps are as follows:

**D.2.1** Determine the most probable value of each factor which influence the slope stability and calculate the most probable value of the factor of safety  $F_{MLV}$  by using conventional methods. Since the reliability analysis is a procedure based on the statistical probability, the average values of the physical and mechanical parameters of the geo-materials shall be applied.

**D.2.2** Estimate the standard deviation of each uncertain parameter by means of test-statistics, or by using empirical average value and variation coefficient, or by using the method of “3  $\sigma$  Rule”. These uncertain parameters include generally the ground water pressure, the shear strength parameters of rock mass and slip surface  $f$  (or internal friction angle  $\phi$ ) and  $c$ . The “3  $\sigma$  Rule” means that if these uncertain parameters obey the normal distribution, then the distribution extent limited by the boundary values obtained by the average value ( the most probable value for a normal distribution, similarly hereinafter) plus and minus 3 times of the standard deviation covers 99.73% of the total probability distribution. The upper and lower limits of the probable range of the parameters can be estimated by experienced experts. Dividing the difference of the upper and lower limits of a parameter by 6 then the quotient can be defined as the standard deviation of this parameter. For example, for the parameter of friction coefficient  $f$  there are correlations as follows:

$$\sigma_f = (f_{ub} - f_{lb}) / 6 \quad (D. 5)$$

$$\mu_f = f_{ub} - 3 \sigma_f \quad (D. 6)$$

$$\text{or } \mu_f = f_{lb} + 3 \sigma_f \quad (\text{D. 7})$$

where:  $\mu_f$  — average of the friction coefficient;

$\sigma_f$  — standard deviation of the friction coefficient;

$f_{lb}$  — lower empirical limit of the friction coefficient;

$f_{ub}$  — upper empirical limit of the friction coefficient.

The other uncertain parameters can be similarly derived. The standard deviations and average values of the parameters may be obtained from their upper and lower empirical limits.

**D.2.3** Calculate the factors of safety  $F^+$  and  $F^-$  by adding and subtracting the standard deviation to the most probable value for each parameter respectively while keep the most probable values of the other parameters constant. If there are  $N$  parameters, the calculation shall be carried out  $2N$  times, and the number of obtained  $F^+$  and  $F^-$  will be  $N$ . On the basis of the obtained  $F^+$  and  $F^-$  calculate  $\Delta F$ . The standard deviation  $\sigma_F$  and variation coefficient  $V_F$  of the factor of safety can be calculated by using the following two formulas:

$$\sigma_F = ((\Delta F_1/2)^2 + (\Delta F_2/2)^2 + \dots + (\Delta F_n/2)^2)^{1/2} \quad (\text{D.8})$$

$$V_F = \sigma_F / F_{MLV} \quad (\text{D.9})$$

$$\Delta F_1 = (F_1^+ - F_1^-) \quad (\text{D.10})$$

where  $F_1^+$  — calculated factor of safety, obtained by adding a standard deviation to the most probable value of the first parameter;

$F_1^-$  — calculated factor of safety, obtained by subtracting a standard deviation from the most probable value of the first parameter;

For example, in a landslide stability analysis the pore pressure  $U$ , the friction coefficient  $f$  and cohesion  $c$  of the slip surface are uncertain parameters. The standard deviation of the factor of safety can be calculated in the following steps:

**D2.3.1** At first, keeping the average values of friction coefficient and cohesion unchanged, that means keeping the  $\mu_f$  and  $\mu_c$  constant, add and subtract the standard deviation of the pore pressure  $\sigma_U$  to and from the average of the pore pressure  $\mu_U$  respectively, namely:  $\mu_U^+ = \mu_U + \sigma_U$ ;  $\mu_U^- = \mu_U - \sigma_U$ , substitute  $\mu_U^+$  and  $\mu_U^-$  together with the  $\mu_f$  and  $\mu_c$  respectively into the stability calculation formula and obtain the two factors of safety  $F_U^+$  and  $F_U^-$ . Then the difference value of the two factors of safety can be calculated as  $\Delta F_U = F_U^- - F_U^+$ .

**D2.3.2** Keep the average values of the pore pressure and cohesion unchanged, that means keeping the  $\mu_U$  and  $\mu_c$  constant, add and subtract the standard deviation of the friction coefficient  $\sigma_f$  to and from the average of the friction coefficient  $\mu_f$  respectively, namely:  $\mu_f^+ = \mu_f + \sigma_f$ ;  $\mu_f^- = \mu_f - \sigma_f$ , substitute  $\mu_f^+$  and  $\mu_f^-$  together with the  $\mu_U$  and  $\mu_c$  respectively into the stability calculation formula and obtain the two factors of safety  $F_f^+$  and  $F_f^-$ . Then the difference value of the two factors of safety can be calculated as  $\Delta F_f =$

$$F_f^+ - F_f^-;$$

**D2.3.3** Keep the average values of pore pressure and friction coefficient unchanged, that means keeping the  $\mu_U$  and  $\mu_f$  constant, plus and minus the standard deviation of the cohesion  $\sigma_C$  to and from the average of the cohesion  $\mu_C$  respectively, namely:  $\mu_C^+ = \mu_C + \sigma_C$ ;  $\mu_C^- = \mu_C - \sigma_C$ , substitute  $\mu_C^+$  and  $\mu_C^-$  together with the  $\mu_U$  and  $\mu_f$  respectively into the stability calculation formula and obtain the two factors of safety  $F_C^+$  and  $F_C^-$ . Then the difference value of the two factors of safety can be calculated as  $\Delta F_C = F_C^+ - F_C^-$ ;

**D2.3.4** The standard deviation of the factor of safety can be calculated by substituting the obtained  $\Delta F_U$ ,  $\Delta F_f$  and  $\Delta F_C$  into the following formula:

$$\sigma_F = [(\Delta F_U/2)^2 + (\Delta F_f/2)^2 + (\Delta F_C/2)^2]^{1/2} \quad (D.11)$$

**D2.3.5** The variation coefficient of the factor of safety can be calculated with the following formula

$$V_F = \sigma_F / \mu_F$$

**D.3** If the factor of safety obeys the log normal distribution, the reliability index expressed as  $\beta_{LN}$  can be calculated with the following formula:

$$\beta_{LN} = (\ln F_{MLV} / [(1 + V_F^2)^{1/2}] / (\ln (1 + V_F^2))^{1/2} \quad (D.12)$$

where:  $F_{MLV}$  — the most probable value of safety factor;

$V_F$  — the variation coefficient of the safety factor.

(Note: J.M.Duncan holds that the assumption of log normal distribution for factor of safety is a reasonable approximation. This assumption does not imply that the values of the individual variables, such as  $\gamma_{ef}$ ,  $\tan \phi$ ,  $\gamma_{bf}$ ,  $\gamma_c$ , must be distributed in the same way. It is not necessary to make any particular assumption concerning the distributions of the variables to use the method described above)

If the normal distribution is used, the calculation may be performed with formula (D.3)

**D.4** Having obtained  $F_{MLV}$ ,  $V_F$  or  $\beta_{LN}$  and  $\beta$ , the failure probability can be calculated according to the following formula:

$$P_f = 1 - \Phi(\beta) \quad (D.13)$$

where:  $\Phi(\beta)$  is a standard normal distribution function. The relation between  $P_f$  and  $\beta$  can be found in the normal distribution table.

The main corresponding values are listed in Table D.1.

**Table D.1 Failure probability  $P_f$  and corresponding probability index  $\beta$**

Failure probability $P_f$	probability index $\beta$
0.50	0
0.25	0.67
0.10	1.28
0.05	1.65
0.01	2.33
0.001	3.10
0.0001	3.72
0.00001	4.25

For different variation coefficients the factor of safety  $F$  and corresponding failure probability  $P_f$  can be obtained by interpolating the values listed in Table D.2. It should be noticed that the factor of safety is calculated by using the average value of mechanical strength of rock or soil.

**Table D.2 Factor of safety and failure probability (%)**

Factor of safety $F$	Variation coefficient of factor of safety $*V_F$									
	0.10		0.15		0.20		0.25		0.30	
	A	B	A	B	A	B	A	B	A	B
1.05	33.02	31.70	40.03	37.55	44.14	40.59	47.01	42.45	49.23	43.69
1.10	18.26	18.17	28.63	27.22	35.11	32.47	39.59	35.81	42.94	38.09
1.15	8.831	9.606	19.42	19.23	27.20	25.71	32.83	30.09	37.10	33.19
1.20	3.771	4.779	12.56	13.33	20.57	20.23	26.85	25.25	31.77	28.93
1.25	1.437	2.275	7.761	9.121	15.20	15.87	21.68	21.19	26.98	25.25
1.30	0.494	1.051	4.606	6.197	11.01	12.43	17.30	17.80	22.76	22.09
1.40	0.044	0.214	1.459	2.841	5.480	7.656	10.69	12.66	15.88	17.05
1.50	0.003	0.043	0.410	1.313	2.569	4.779	6.380	9.121	10.85	13.33
1.60	0.000	0.009	0.105	0.621	1.148	3.040	3.707	6.681	7.294	10.57
1.80		0.000	0.006	0.152	0.206	1.313	1.178	3.772	3.176	6.924
2.00			0.000	0.043	0.034	0.621	0.355	2.275	1.340	4.779
3.00			0.000	0.000	0.000	0.043	0.001	0.383	0.016	1.313
Note: A — The factor of safety is considered to obey the log normal distribution. The calculation is carried out by using formula (D.12); B — The factor of safety is considered to obey the normal distribution. The calculation is carried out by using formula (D. 3). * Suppose the change of the dead load of the rock and soil can be ignored and the ground water pressure is considered to be the maximum value and be treated as constant, then the variation coefficient is that for shear strength of the rock or soil.										

**D.5** The annual failure probability is obtained by dividing the failure probability by the design basic period of service. The design basic period of service of slopes is determined according to the same one of the associated hydraulic structures, for common slopes it may be determined as 50 years.

## Appendix E (Informative)

### Calculation of stability against sliding

#### E.1 Stability calculation by slicing methods

E.1.1 The simplified Bishop method (see Fig. E.1) equation (E.1) shall be used

$$K = \frac{\sum \{[(W_i + V_i) \sec \alpha - u_i b_i \sec \alpha_i] \tan \phi'_i + c'_i b_i \sec \alpha_i\} / (1 + \tan \alpha_i \tan \phi'_i / K)}{\sum [(W_i + V_i) \sin \alpha_i + M_{Qi} / R]} \quad (\text{E. 1})$$

where:  $W_i$  —weight of the i-th sliding slice;

$Q_i$ 、 $V_i$  —horizontal and vertical components of external forces acting on the i-th sliding slice (downward is positive and the same below) respectively, including seismic action, reinforcing forces provided by anchoring cables and piles, and surface loads);

$u_i$  —pore pressure acting on the base surface of the i-th sliding slice;

$\alpha_i$  —dipping angle of the base surface of the i-th sliding slice;

$b_i$  —width of the i-th sliding slice;

$c'_i$ 、 $\phi'_i$  —effective cohesion and internal friction angle of the i-th sliding slice;

$M_{Qi}$  —moment of horizontal external forces acting on the i-th sliding slice to circle center;

$R$  —arc radius;

$K$  —factor of safety.

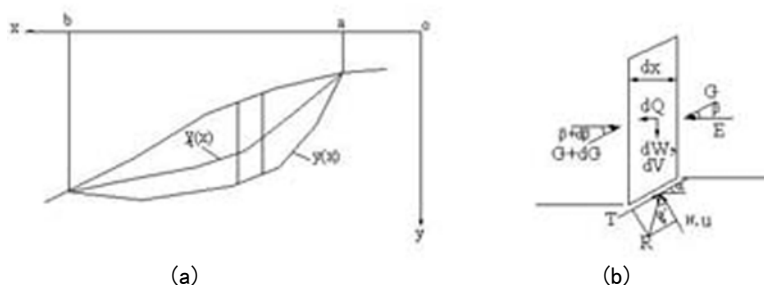


Figure E.1 A sketch of slicing method for circular slide and the treatment of water level outside the slope

E.1.2 For calculation with Morgenstern- Price method (see Fig. E.2), the following equations (E.2-E.10) shall be used:

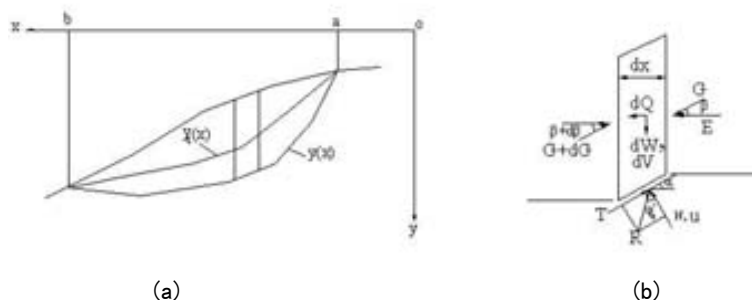


Figure E.2 A sketch of Morgenstern-Price method

$$\int_a^b p(x)s(x)dx = 0 \quad (E.2)$$

$$\int_a^b p(x)s(x)t(x)dx - M_e = 0 \quad (E.3)$$

where:

$$p(x) = \left(\frac{dW}{dx} + \frac{dV}{dx}\right) \sin(\tilde{\varphi}' - \alpha) - u \sec \alpha \sin \tilde{\varphi}' + \tilde{c}' \sec \alpha \cos \tilde{\varphi}' - \frac{dQ}{dx} \cos(\tilde{\varphi}' - \alpha) \quad (E.4)$$

$$s(x) = \sec(\tilde{\varphi}' - \alpha + \beta) \exp \left[ - \int_a^x \tan(\tilde{\varphi}' - \alpha + \beta) \frac{d\beta}{d\zeta} d\zeta \right] \quad (E.5)$$

$$t(x) = \int_a^x (\sin \beta - \cos \beta \tan \alpha) \exp \left[ \int_a^\xi \tan(\tilde{\varphi}' - \alpha + \beta) \frac{d\beta}{d\zeta} d\zeta \right] d\xi \quad (E.6)$$

$$M_e = \int_a^b \frac{dQ}{dx} h_e dx \quad (E.7)$$

$$\tilde{c}' = \frac{c'}{K} \quad (E.8)$$

$$\tan \tilde{\varphi}' = \frac{\tan \varphi'}{K} \quad (E.9)$$

$$\tan \beta = \lambda f(x) \quad (E.10)$$

The definitions of the physical quantities are as follows:

$dx$  ———width of slices;

$c'$  、  $\varphi'$  ———effective cohesion and internal friction angle of the sliding slice;

$dW$  ———weight of the sliding slice;

$u$  ———pore pressure acting on the base surface of the sliding slice;

$\alpha$  ———included angle between the base surface of the sliding slice and horizontal plane;

$dQ$  、  $dV$  ———horizontal and vertical components of the external forces acting on the sliding slice, respectively, including seismic action, reinforcing forces provided by anchoring cables and piles, and surface loads;

$M_e$  ———moment of  $dQ$  to the center of the slice;;

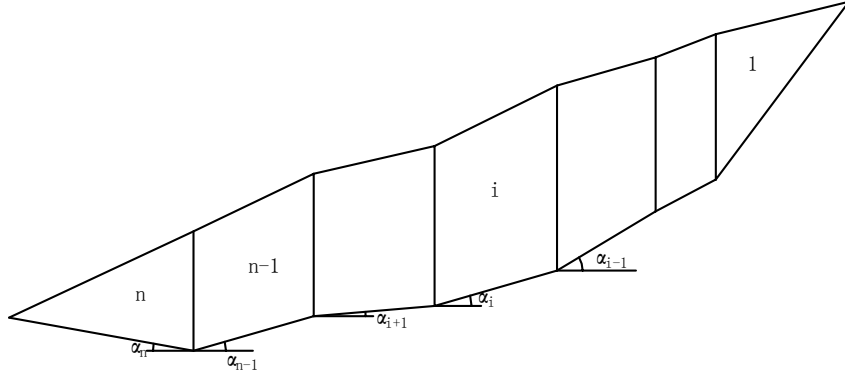
$h_e$  ———vertical distance from the application point of  $dQ$  to the center of the slice;

$f(x)$  — — distribution of  $\tan \beta$  at direction  $x$ , generally selected  $f(x)=1$ ;

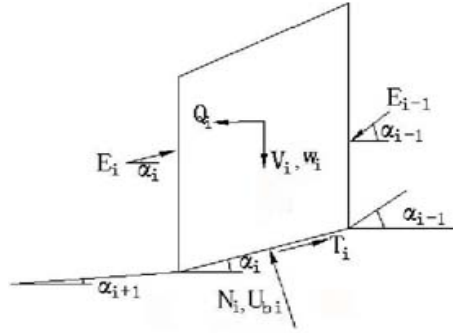
$\lambda$  ———An undetermined coefficient for determination of  $\tan \beta$ .

There are two unknown quantities in the equations (E.2) and (E.3) . The factor of safety  $K$  is implied in the equations (E.8) and (E.9), the other undetermined coefficient  $\lambda$  is implied in the equation (E.10), these two unknown quantities can be gained by iterative operation.

**E.1.3** For the method of unbalanced thrust force transmission the following equation (E.11) ~ (E.15) shall be used.



**Figure E.3 A sketch of slide surface by the method of unbalanced thrust force transmission**



**Figure E.4 A Sketch of calculation by method of unbalanced thrust force transmission**

$$K = \frac{\sum_{i=1}^{n-1} (R_i \prod_{j=i+1}^n \psi_j) + R_n}{\sum_{i=1}^{n-1} (T_i \prod_{j=i+1}^n \psi_j) + T_n} \quad (\text{E.11})$$

where:

$$R_i = [(W_i + V_i) \cos \alpha_i - U_{bi} - Q_i \sin \alpha_i] \tan \phi'_i + c'_i l_i \quad (\text{E.12})$$

$$T_i = [(W_i + V_i) \sin \alpha_i + Q_i \cos \alpha_i] \quad (\text{E.13})$$

$$\psi_i = \cos(\alpha_{i-1} - \alpha_i) - \sin(\alpha_{i-1} - \alpha_i) \tan \phi'_i / K \quad (\text{E.14})$$

The definitions of the physical quantities are as follows:

$R_i$ —anti-sliding force at the base plane of the  $i$ -th sliding slice;

$T_i$ —slide-driving force at the base plane of the  $i$ -th sliding slice;

$\psi_i$ —transmission coefficient for determination of the thrust force at the border of the  $i$ -th sliding slice,  $\psi_1 = 1$ ;

$W_i$ —weight of the  $i$ -th sliding slice;

$Q_i$ 、 $V_i$ —horizontal and vertical components respectively of the external forces acting on the  $i$ -th sliding slice, including seismic action, reinforcing forces provided by anchoring cables and piles, and surface loads;

$U_{bi}$ —pore pressure acting on the base plane of the sliding slice;

$E_{i-1}$ —thrust force transmitted from  $i-1$ th sliding slice to the  $i$ -th sliding slice;

$E_i$ —reaction force of the  $i+1$ th sliding slice to the  $i$ -th sliding slice at the lateral side. The reaction is the same as that of the thrust force of the  $i$ -th sliding slice in quantity but reverse in direction.

$\alpha_i$ —included angle between base plane of the  $i$ -th sliding slice and horizontal plane;

$l_i$ —length of base plane of the  $i$ -th sliding slice;

$c'_i$ 、 $\phi'_i$ —effective cohesion and internal friction angle on base plane of the  $i$ -th sliding slice;

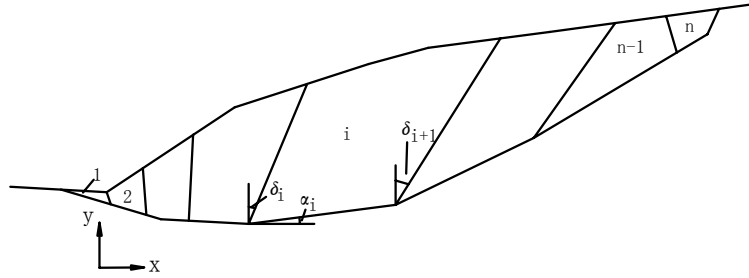
$K$ —factor of safety.

The thrust force acting on the slice interface  $E_i$  is determined by the formula as follows:

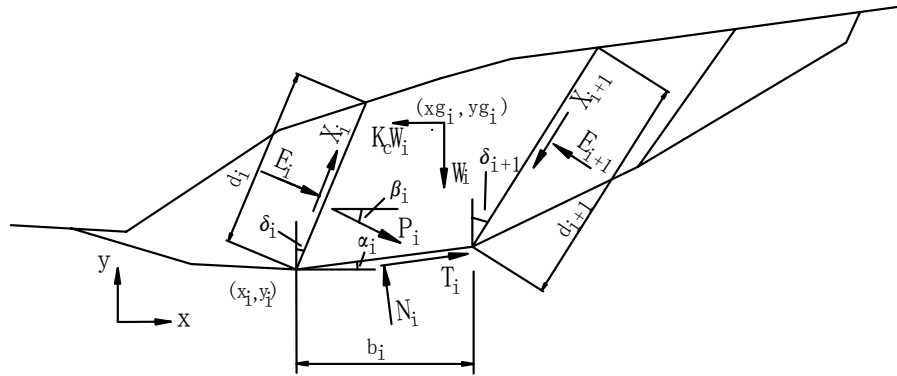
$$E_i = T_i - R_i / K + \psi_i E_{i-1} \quad (\text{E. 15})$$

**E.1.4 Sarma method** (see Fig. E.5 and E.6, the external forces acting on slices are not included)

The critical horizontal force coefficient  $K_c$  making the slope in limit equilibrium state corresponding to a certain factor of safety  $K$  is calculated by Equation (E.16). The factor of safety  $K$  is the corresponding value, which makes the  $K_c$  equal to zero and can be obtained by iteration.



**Figure E.5 A sketch of slide surfaces by Sarma method**



**Figure E.6 A sketch calculation by Sarma method**

$$K_c = \frac{\alpha_n + \alpha_{n-1}e_n + \alpha_{n-2}e_n e_{n-1} + \dots + \alpha_1 e_n e_{n-1} \dots e_3 e_2 + E_1 e_n e_{n-1} \dots e_1 - E_{n+1}}{p_n + p_{n-1}e_n + p_{n-2}e_n e_{n-1} + \dots + p_1 e_n e_{n-1} \dots e_3 e_2} \quad (\text{E16})$$

$$\alpha_i = \frac{R_i \cos \tilde{\varphi}'_{bi} + W_i \sin(\tilde{\varphi}'_{bi} - \alpha_i) + S_{i+1} \sin(\tilde{\varphi}'_{bi} - \alpha_i - \delta_{i+1}) - S_i \sin(\tilde{\varphi}'_{bi} - \alpha_i - \delta_i)}{\cos(\tilde{\varphi}'_{bi} - \alpha_i + \tilde{\varphi}'_{si+1} - \delta_{i+1}) \sec \tilde{\varphi}'_{si+1}} \quad (\text{E.17})$$

$$p_i = \frac{W_i \cos(\tilde{\varphi}'_{bi} - \alpha_i)}{\cos(\tilde{\varphi}'_{bi} - \alpha_i + \tilde{\varphi}'_{si+1} - \delta_{i+1}) \sec \tilde{\varphi}'_{si+1}} \quad (\text{E.18})$$

$$e_i = \frac{\cos(\tilde{\varphi}'_{bi} - \alpha_i + \tilde{\phi}'_{si} - \delta_i) \sec \tilde{\varphi}'_{si}}{\cos(\tilde{\varphi}'_{bi} - \alpha_i + \tilde{\varphi}'_{si+1} - \delta_{i+1}) \sec \tilde{\varphi}'_{si+1}} \quad (E.19)$$

$$R_i = \tilde{c}_{bi} b_i \sec \alpha_i - U_{bi} \tan \tilde{\varphi}'_{bi} \quad (E.20)$$

$$S_i = \tilde{c}_{si} d_i - U_{si} \tan \tilde{\varphi}'_{si} \quad (E.21)$$

$$S_{i+1} = \tilde{c}_{si+1} d_{i+1} - U_{si+1} \tan \tilde{\varphi}'_{si+1} \quad (E.22)$$

$$\tan \tilde{\varphi}'_{bi} = \tan \varphi'_{bi} / K \quad (E.23)$$

$$\tilde{c}'_{bi} = c'_{bi} / K \quad (E.24)$$

$$\tan \tilde{\varphi}'_{si} = \tan \varphi'_{si} / K \quad (E.25)$$

$$\tilde{c}'_{si} = c'_{si} / K \quad (E.26)$$

$$\tan \tilde{\varphi}'_{si+1} = \tan \varphi'_{si+1} / K \quad (E.27)$$

$$\tilde{c}'_{si+1} = c'_{si+1} / K \quad (E.28)$$

The physical quantities are defined as follows:

$c'_{bi}, \varphi'_{bi}$ ——effective cohesion and internal friction angle of the base plane of i-th slice;

$c'_{si}, \varphi'_{si}$ ——effective cohesion and internal friction angle of i-th lateral surface of i-th slice;

$c'_{si+1}, \varphi'_{si+1}$ ——effective cohesion and internal friction angle of i+1-th lateral surface of i-th slice;

$U_{si}, U_{si+1}$ ——pore pressures at i-th and i+1-th lateral surfaces respectively of i-th slice

$U_{bi}$ ——pore pressure at the base plane of i-th slice;

$\delta_i, \delta_{i+1}$ ——dip angles of i-th and i+1-th lateral surfaces of i-th slice (beginning from the vertical line, the clockwise is positive and counterclockwise is negative);

$\alpha_i$ ——included angle between base plane of i-th slice and horizontal plane;

$b_i$ ——horizontal projected length of base plane of i-th slice;

$d_i, d_{i+1}$ ——lengths of i-th and the i+1-th lateral surfaces of i-th slice, respectively;

$K_c$ ——critical horizontal force coefficient.

**E.1.5** For the energy method (EMU-energy method upper bound limit analysis), the following equation ( E.29) shall be used ( see fig. E.7):

$$\begin{aligned} & \sum_{i=1}^n \lambda_i [(\tilde{c}'_{bi} \cos \tilde{\varphi}'_{bi} - u_{bi} \sin \tilde{\varphi}'_{bi}) b_i \sec \alpha_i] \\ & + \sum_{i=1}^{n-1} \lambda_{i+1} [(\tilde{c}'_{si} \cos \tilde{\varphi}'_{si} - u_{si} \sin \tilde{\varphi}'_{si}) \sec(\alpha_i + \delta_i - \tilde{\varphi}'_{bi} - \tilde{\varphi}'_{si}) \sin(\Delta \alpha_i - \Delta \tilde{\varphi}'_{bi}) d_i] \\ & = \sum_{i=1}^n \lambda_i [(W_i + V_i) \sin(\alpha_i - \tilde{\varphi}'_{bi}) + Q_i \cos(\alpha_i - \tilde{\varphi}'_{bi})] \end{aligned} \quad (E.29)$$

where:

$$\lambda_i = \begin{cases} 1 & i = 1 \\ \prod_{k=2}^i \frac{\cos(\alpha'_j + \delta_j - \tilde{\varphi}'_{bj} - \tilde{\varphi}'_{sj})}{\cos(\alpha'_j + \delta_j - \tilde{\varphi}'_{bj} - \tilde{\varphi}'_{sj})} & i = 2, 3, \dots, n-1 \end{cases} \quad (E.30)$$

$$\tan \tilde{\varphi}'_{bi} = \frac{\tan \varphi'_{bi}}{K} \quad (\text{E. 31})$$

$$\tilde{c}'_{bi} = \frac{c'_{bi}}{K} \quad (\text{E. 32})$$

$$\tan \tilde{\varphi}'_{si} = \frac{\tan \varphi'_{si}}{K} \quad (\text{E. 33})$$

$$\tilde{c}'_{si} = \frac{c'_{si}}{K} \quad (\text{E. 34})$$

$$\tan \tilde{\varphi}^l_{bj} = \frac{\tan \varphi^l_{bj}}{K} \quad (\text{E.35})$$

$$\tan \tilde{\varphi}^r_{bj} = \frac{\tan \varphi^r_{bj}}{K} \quad (\text{E.36})$$

$$\tan \tilde{\varphi}_{sj} = \frac{\tan \varphi'_{sj}}{K} \quad (\text{E.37})$$

where

$W_i$ ——dead weight of the i-th slice;

$H_i, Q_i, V_i$ ——horizontal and vertical components of the external forces acting on the i-th slice, including seismic action, reinforcing forces provided by anchoring cables and piles, and surface loads, respectively;

$u_{bi}$ ——pore pressure acting on base plane of the i-th sliding slice;

$b_i$ ——horizontal projected length of base plane of the i-th slice;

$d_i$ ——length of the i-th lateral surface;

$\alpha_i$ ——dip angles of base plane of the i-th slice;

$\delta_i, \delta_j$ ——dip angles of the i-th and j-th lateral surfaces (the turning from positive y axis to positive x axis is positive), respectively

$c'_{bi}, \varphi'_{bi}$ ——effective cohesion and internal friction angle of the base surface of the i-th slice;

$c'_{si}, \varphi'_{si}$ ——effective cohesion and internal friction angle of the lateral surface of the i-th slice;

$\varphi^l_{bj}, \varphi^r_{bj}$ ——internal friction angle of base plane of the left and right adjacent slices of the j-th lateral surface, respectively;

$\varphi_{sj}$ ——internal friction angle of the j-th lateral surface;

$\alpha_j^l, \alpha_j^r$ ——dipping angle of base planes of the left and right adjacent slices respectively of the j-th lateral surface;

$\Delta\alpha_i$  and  $\Delta\varphi_{bi}$ ——increments of  $\alpha_i$  and  $\varphi_{bi}$  of the adjacent right slice compared to the left slice of the i-th lateral surface.

The two items in the left side of Fig. E.7 are internal energy consumption happened at base surfaces of n slices and n-1 lateral surfaces, and the item in the right side of the equation is the work done by external forces.  $\lambda_i$  is the i-th slice plastical displacement rate of the i-th slice. The factor of safety K implied in equations (E.31) to (E.37) can be resolved by

It has been proven theoretically that the energy method is completely equivalent to the Sarma method, any one of which can be selected for stability analysis of slopes.



**E.2.1** For stability calculation of wedge sliding along the intersection line, CO equations E.38 to E.54 shall be used:



Where:

$$s = (m_{ab}m_{pb} - m_{pa})/(1 - m_{ab}^2) \quad (\text{E. 41})$$

$$x = (m_{ab}m_{Wa} - m_{Wb}) / (1 - m_{ab}^2) \quad (E. 42)$$

$$y = (m_{ab}m_{ca} - m_{cb}) / (1 - m_{ab}^2) \quad (E. 43)$$

$$z = (m_{ab}m_{Pa} - m_{Pb}) / (1 - m_{ab}^2) \quad (E. 44)$$

$$m_{ab} = \sin \psi_a \sin \psi_b \cos(\alpha_a - \alpha_b) + \cos \psi_a \cos \psi_b \quad (E. 45)$$

$$m_{Wa} = -\cos \psi_a \quad (E. 46)$$

$$m_{Wb} = -\cos \psi_b \quad (E. 47)$$

$$m_{ca} = \sin \psi_a \sin \psi_c \cos(\alpha_a - \alpha_c) + \cos \psi_a \cos \psi_c \quad (E. 48)$$

$$m_{cb} = \sin \psi_b \sin \psi_c \cos(\alpha_b - \alpha_c) + \cos \psi_b \cos \psi_c \quad (E. 49)$$

$$m_{Pa} = \cos \psi_P \sin \psi_a \cos(\alpha_P - \alpha_a) - \sin \psi_P \cos \psi_a \quad (E. 50)$$

$$m_{Pb} = \cos \psi_P \sin \psi_b \cos(\alpha_P - \alpha_b) - \sin \psi_P \cos \psi_b \quad (E. 51)$$

$$m_{WS} = \sin \psi_S \quad (E. 52)$$

$$m_{CS} = \cos \psi_S \sin \psi_c \cos(\alpha_S - \alpha_c) - \sin \psi_S \cos \psi_c \quad (E. 53)$$

$$m_{PS} = \cos \psi_S \cos \psi_P \cos(\alpha_S - \alpha_P) + \sin \psi_P \cos \psi_S \quad (E. 54)$$

where:

$A_A$ 、 $c'_A$ 、 $\phi'_A$  —— area, effective cohesion and inner friction angle of the slide surface A;

$A_B$ 、 $c'_B$ 、 $\phi'_B$  —— area, effective cohesion and inner friction angle of the slide surface B;

$\psi_a$ 、 $\alpha_a$  —— dip angle and direction of the slide surface A;

$\psi_b$ 、 $\alpha_b$  —— dip angle and direction of the slide surface B;

$\psi_c$ 、 $\alpha_c$  —— dip angle and direction of the pull crack surface C;

$\psi_P$ 、 $\alpha_P$  —— dip angle and direction of the anchoring force P;

$\psi_S$ 、 $\alpha_S$  —— dip angle and direction of the intersection line OC between slide surfaces A and B;

$U_A$  —— pore pressure acting on slide surface A;

$U_B$  —— pore pressure acting on slide surface B;

$U_C$  —— pore pressure acting on pull crack surface C;

$W$  —— dead weight of the wedge;

$P$  —— anchoring force.

In figure E.8,  $N_a$ ,  $N_b$ , and  $U_C$  are the effective normal reaction forces of the slide surface A, B, and the pull crack surface C respectively, the dip angle and direction of which are as follows:

For  $N_a$ :  $\psi_a - 90^\circ$  and  $\alpha_a$ ;

For  $N_b$ :  $\psi_b - 90^\circ$  and  $\alpha_b$ ;

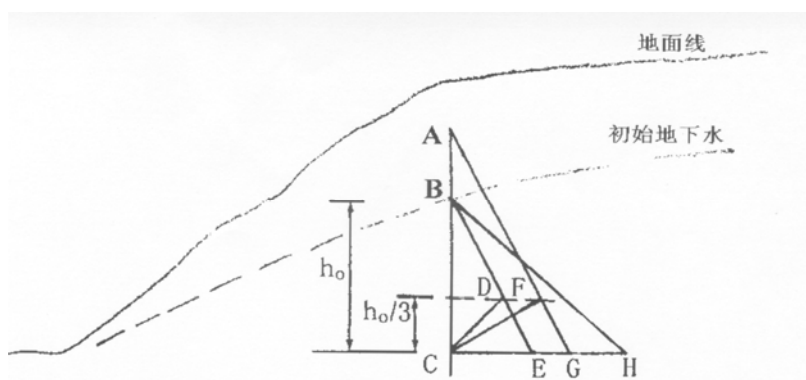
For  $U_C$ :  $\psi_c - 90^\circ$  and  $\alpha_c$ ;

For  $W$ :  $90^\circ$ .

## Appendix F (Normative)

### Estimation method of water pressure under sustained and transient design conditions

**F.1** Sustained design condition: for condition without rainfall the maximum recorded ground water table during rainy season is taken as the reference value or initial value. About water pressure estimation for slope design refer to Figure F.1.



**Fig.F.1** A sketch of distribution of the most unfavorable transient water pressure caused by rainfall

**F1.1** The initial value of the water pressure shall be calculated according to the static water pressure of ground water table multiplied by a reduction coefficient  $\beta$ , which should be selected in the range of  $\leq 1$  and  $> 0$  based on different conditions. When it is possible osmometer can be fixed at different elevations in a same bore hole, by which the value of  $\beta$  can be obtained according to the observed pressure or water table in the hole; the  $\beta$  also can be obtained by analysis of the initial osmotic pressure field. This means the static water pressure is calculated according to the  $\triangle BCE$  in Fig. F.1.

In case the consequent stress-released joints are developed whereas the infiltrated rainwater is difficultly be discharged, the transient water pressure caused by rainfall can be calculated by taken  $\beta = 1$ , namely according to the  $\triangle BCH$  in Fig. F.1.

**F.1.2** In case that drainage facilities have been provided at depth  $h_0$  under the groundwater table, where the static water pressure can be assumed to be zero and the applying water pressure can be calculated according to the  $\triangle BCD$  in Fig. F.1. That means taking the depth of  $(2/3) h_0$  as a boundary, the static water pressure upper which is distributed in accordance with a positive right triangle and that lower which is distributed in an inversed right triangle.

**F.2** Transient design condition: in case of rainfall, the transient ground water rises

for a height of  $\Delta h$ , the static water pressure at all the depth shall be calculated by adding an increment of  $\beta \gamma \Delta h$ .

**F.2.1** In case of lacking drainage facilities, the static water pressure shall be calculated according to the  $\Delta ACG$  in Fig. F.1.

**F.2.2** In case that drainage facilities have been provided at depth  $h_0$  under the groundwater table, the static water pressure at  $h_0$  can be assumed to be zero and the applying water pressure can be calculated according to the  $\Delta ACF$  in Fig. F.1. That means taking the depth of  $(2/3) h_0$  as a boundary, the static water pressure upper which is distributed in accordance with a positive right triangle and that lower which is distributed in an inversed right triangle.

**F.2.3** In the rainy South China or where the continuous heavy rainfall of more than 5 hours has been recorded and there is no impervious layer at the ground surface, the ground water can reach ground surface. In the arid North China or where the ground surface is provided with an impervious layer, the ground water table shall be properly lower than ground surface.

**F.3** Provided the subsurface drainage facilities can not work effectively, the static water pressure there shall be more than zero and can be determined by analysis and judgment.

## Appendix G (Informative)

### Parameters of shear strength for rock and soil slopes

#### G.1 Design parameters of shear strength for rock slopes

##### G.1.1 Shear (rupture) strength of rock mass

**G.1.1.1** Provided the rock mass is composed of hard rock with massive-blocked and bedded structure and the failure is of brittle type, the value corresponding 0.2 in the probability distribution curve, or the average value of the small values, or the lower limit value determined by the optimized gradient method of the peak strength Shall be taken as the standard value.

**G.1.1.2** Provided the rock mass is of interlocked cataclastic structure or cataclastic structure with developed micro, unfilled and tight joints, and the failure is of plastic or elastic-plastic type ,the average value of yield strength shall be used as the standard value.

**G.1.1.3** Provided the slope rock mass is of isotropic jointed structure and without specially controlling discontinuities, the shear strength of which can be determined by Hoek-Brown's failure criterion of rock mass and GSI or RMR systems in combination with the rock mass quality class and test results. See the method described in Appendix H.

**G.1.2** During planning and pre-feasibility stages, when the geotechnical test data of rock mass are not adequate, the rock mechanical parameters used for geological suggestion or design assumption can be selected according to Tab.G.1( quoted from Appendix D.0.3 of GB50287) after a proper reduction in accordance with the geological conditions of the slope.

**Table .G.1 Rock mechanical parameters referred for slope design**

Rock mass class	Contact surface of concrete and rock mass		Rock mass		Deformation modulus
	$f'$	$c'$ (MPa)	$f'$	$c'$ (MPa)	$E_0$ (GPa)
I	$1.50 \geq f' > 1.30$	$1.50 \geq c' > 1.30$	$1.60 \geq f' > 1.40$	$2.50 \geq c' > 2.00$	$> 20.0$
II	$1.30 \geq f' > 1.10$	$1.30 \geq c' > 1.10$	$1.40 \geq f' > 1.20$	$2.00 \geq c' > 1.50$	$20.0 \geq E_0 > 10.0$
III	$1.10 \geq f' > 0.90$	$1.10 \geq c' > 0.70$	$1.20 \geq f' > 0.80$	$1.50 \geq c' > 0.70$	$10.0 \geq E_0 > 5.0$
IV	$0.90 \geq f' > 0.70$	$0.70 \geq c' > 0.30$	$0.80 \geq f' > 0.55$	$0.70 \geq c' > 0.30$	$5.0 \geq E_0 > 2.0$
V	$0.70 \geq f' > 0.40$	$0.30 \geq c' > 0.05$	$0.55 \geq f' > 0.40$	$0.30 \geq c' > 0.05$	$2.0 \geq E_0 > 0.2$
Note :1 In the table $f'$ 、 $c'$ are parameters of shear failure strength; 2 The parameters in this table are limited for hard rocks; if using for soft rocks, these parameters shall be reduced according to the softening coefficient.					

**G.1.3** During planning and pre-feasibility stages, when the geotechnical test data for joints, seams and faults are not adequate, the mechanical parameters used for geological suggestion

or design assumption can be selected according to Table.G.2( quoted from Appendix D.0.5 of GB50287-99 ) after a proper reduction in accordance with the geological conditions of the slope.

**Table.G.2 Shear failure strength parameters of joints, seams and faults**

Type of discontinuities		$f'$	$c'$ (MPa)
Hard discontinuities	Cemented	0.80~0.60	0.250~0.100
	Without fillings	0.70~0.45	0.150~0.050
weak discontinuities (with fillings)	Rock fragments and pieces	0.55~0.45	0.250~0.100
	Rock pieces with mud	0.45~0.35	0.100~0.050
	Mud with rock pieces	0.35~0.25	0.050~0.010
	Clay film, gouge, argillaceous seam	0.25~0.18	0.01~0.002

**G.1.4** For engineered slopes, which are stable and without obvious deformation trace, the adopted design parameters of the shear strength (including shear failure strength) of rock mass, soil mass and discontinuities shall be determined taking the deformation and stability criteria into account.

**G.1.4.1** The parameters of physical and mechanical properties for the foundation slopes are suitably determined according to the following principles:

1. For weak discontinuities, the standard value of the strength corresponding with the allowable deformation, or the yield strength, or the rheological strength, or the recommended value by geologist shall be adopted as the design value;
2. For hard discontinuities the standard value of the strength corresponding with the proportional limit or the recommended value by geologist shall be adopted as the design value;
3. For joint sets, the strength standard value of proportional limit or the recommended value by geologist shall be adopted as the design value, taking the joint continuity along the dominant direction and rock bridge effect into consideration.
4. For deformation modulus or elastic modulus of rock mass, the standard value, based on the load-deformation relationship corresponding to the maximum loading applied by the structures, or the recommended value by geologist shall be adopted as the design value.

**G.1.4.2** Common slopes (non-foundation): The stable natural slopes and engineered slopes formed by controlled blasting, simultaneous anchoring or pre-anchoring during excavation, the shear strength parameters of which can be determined according to the

following principles:

1. For weak discontinuities: the standard value of the peak shear strength or the recommended value by geologist shall be adopted as the design value;
2. For hard discontinuities: the standard value of the peak shear failure strength or the recommended value by geologist shall be adopted as the design value;
3. For joint sets, the strength standard value of peak shear failure strength or the recommended value by geologist shall be adopted as the design value, taking the joint continuity along the dominant direction and rock bridge effect into consideration.

**G.1.5** The geotechnical parameters for stability analysis of the deformed or deforming slopes should be determined according to the following stipulations:

The composite parameter of shear strength can be obtained by regressive estimation from limit equilibrium condition with the factor of safety equal to 1.05 to 1 times the traditional one according to the deformation intensity of the slope rock mass. When the slope is likely to fail, the factor of safety can be assumed equals to 1.

**G.1.6** The physical and mechanical parameters for stability analysis of landslides or failed rock slopes are preferably determined according to the following stipulations:

1. For the friction coefficient of slide surface and the cracked discontinuities inside the sliding mass the residual strength is taken as the standard value while the cohesion can be ignored.
2. The shear strength of slide surface at the moment when the landslide happens can be obtained by regressive estimation taking the factor of safety equal to 0.95 to 0.99 according to the sliding velocity and the destruction intensity of the rock mass, for which the cohesion can be assumed very small or even ignored during regressive estimation.

## **G.2 Design parameters of shear strength for soil slopes**

### **G.2.1 Value of shear strength for soil slopes**

**G.2.1.1** For shear strength of soil obtained by direct shear test, the peak value should be adopted. The average of the small values or the value corresponding with the 0.2 on the probability distribution curve shall be taken as the standard value of strength index.

**G.2.1.2** The soil samples for test shall be undisturbed possibly, unless it is for an artificially accumulated slope. In case it is difficult to get undisturbed samples, disturbed ones with features simulating the undisturbed situation should be adopted.

**G.2.1.3** For soil mass above saturation line, the test results of natural undisturbed samples shall be adopted. For soil mass under saturation line, the test results of saturated undisturbed

samples shall be adopted.

**G.2.1.4** For sandy soil slopes, the factor of safety should be calculated by the method of effective stress. The parameters of shear strength can be obtained by triaxial consolidated drained (CD) test or slow direct shear (S) test.

**G.2.1.5** For clayey soil slopes, the factor of safety should be calculated by the method of effective stress. The parameters of shear strength can be obtained by triaxial consolidated drained (CD) test or consolidated undrained (CU) test with measured pore pressure or slow direct shear (S) test. Provided the calculation is carried out by the total stress method, the corresponding test methods are triaxial consolidated undrained (CU) test and consolidated quick shear test (CQ).

**G.2.1.6** For special soil slopes with rheological characteristics the calculation shall be carried out by using the rheological strength.

**G.2.1.7** For the soil of slide zone in landslides and soil slopes with large deforming, the average of small values of residual strength of disturbed samples can be used. The attention shall be paid to the influence of the moisture content variation on the soil strength and the natural moisture content or saturated moisture content shall be used.

**G.2.2** According to the slope stability situation the corresponding shear strength parameters should be used: for stable and deforming slopes the parameters are selected on the basis of peak strength; for failed slopes the parameters are selected on the basis of residual strength.

**G.2.3** The composite parameter of shear strength of slide surface can be obtained by regressive estimation from the critical stability situation of the slope. Generally, the factor of safety for deforming or deformed slopes can be assumed as 1.05 to 1.00; that for failed slopes can be assumed as 0.95 to 0.99.

## Appendix H (Informative)

### RMR and GSI systems for estimation of shear strength of isotropic jointed rock mass

**H.1** The rock mass quality index RMR is determined according to Table.H.1







**Table.H.1 RMR classification and rating**

parameter			Range of value						
1	C.S of intact rock	Index of point loading	>10	4~10	2~4	1~2			
		U.C.S (MPa)	>250	100~250	50~100	25~50	5~25	1~5	<1
	Rating		15	12	7	4	2	1	0
2	RQD %		90~100	75~90	50~75	25~50	<25		
	Rating		20	17	13	8	3		
3	Joint spacing    cm		>200	60~200	20~60	6~20	<6		
	Rating		20	15	10	8	5		
	Joint condition		Very rough surfaces. Not continous.CI osed. Unweathered wall	Slightly rough surfaces. thickness<1 mm. Slightly weathered wall	Slightly rough surfaces. thickness<1 mm. Highly weathered wall	Slick surface or with soft seam<5mm thick. joint aperture 1 - 5 mm. joint continous	Soft seam >5mm. continuous joints with aperture of>5mm. joint continous		
	Rating		30	25	20	10	0		
5	Ground water	Ground water flow per 10m tunnel    l/min	0	<10	10~25	25~125	>125		
		Ratio of joint water pressure / max. principle stress	0	<0.1	0.1~0.2	0.2~0.5	>0.5		
		General condition	Completely dry	Wet	Dripping	Flowing			
		Rating	15	10	7	4	0		
Note : When the RMR is applied for estimation of the strength parameters, the ground water rating is adopted to be 15 for all conditions. The influence of groundwater will be considered by taking pore pressure into account during the stability analysis. It should not be considered repeatedly.									

**H. 2** The geological strength index GSI for jointed rock masses is determined according to

the Table.H.2

**Table H.2 Geological strength index for jointed rock masses (GSI)**

Geological strength indexes of jointed rocks (Hoek and Marinos, 2000). The average GSI value of rock mass can be determined based on the rock mass structure and discontinuity surface conditions. It is not necessary to try to determine the GSI value too precisely. It is more realistic to define a range for the GIS value (eg. From 33 to 37) than to fix GSI=35. This table is not applicable to rock mass, failure of which is controlled by its discontinuities. Planar and straight discontinuities with unfavourable occurrences with respect to the excavated surface would dominate the rock mass behaviours. For rock mass with ground water, the shear strength would be reduced due to change of the ground water conditions. During excavation in very poor rock mass, if ground water is discovered in the rock mass, GSI value lower than that listed in the table shall be used. Influence of the ground water shall be alleviated through the effective stress analysis and appropriate treatment.		Characteristics of discontinuity surface				
		Very good: very rough, fresh unweathered surfaces	Good: rough, slightly weathered, iron stained surfaces	Fair: Smooth, moderately weathered and altered surfaces	Poor: Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments	Very poor: Slickensided, highly weathered surfaces with soft clay coatings or fillings
Rock struture		Lowering quality of discontinuity surface→				
	① intact or massive – intact rock specimens or massive in situ rock with few widely spaced discontinuities	90	80	70	60	N/A
	② blocky – well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets	80	70	60	50	40
	③ mosaic – interlocked, partially disturbed mass with multi-face angular blocks formwd by 4 or more joint sets	70	60	50	40	30
	④ blocky/disturbed/seamy – folded with anglur blocks formed by many intersecting discontinuity sets. Continuous bedding planes or schistosity planes	60	50	40	30	20
	⑤ weathered rock mass – poorly interlocked, heavily broken rock mass composed of mixture of angular and rounded rock pieces	50	40	30	20	10
	⑥ Laminated/sheared zone – only containing very few rock blocks due to closely spaced weak schistosity or shear planes	N/A	N/A	N/A	N/A	N/A

### H. 3 Relationship of RMR and GSI

The empirical relationship between RMR determined by H.1 and GSI determined by H.2:

$$GSI=RMR-5 \quad (H.1)$$

### H. 4 Estimation of the shear strength parameters by GSI or RMR

The shear strength parameters are estimated according to GSI or RMR in the

following steps:

#### H.4.1 Determine the constant $m_i$ for intact rock according to Table H.3

**Table H.3 Constant  $m_i$  for intact rock by rock group**

Rock type	class	group	Textture			
			coarse	medium	fine	Very fine
sedimentary	clastic		Conglomerate* Breccia*	Sandstone 17±4	Siltstone 7±2 graywacke (18±3)	Claystone 4±2 Shale (6±2) Marls (7±2)
	Non-clastic	carbonates	Crystalline limestone (12±3)	Sparry limestone (10±2)	Microcrystalline limestone (9±2)	dolomites (9±3)
		evaporites		Gypsum 8±2	Anhydrite 12±2	
		organic				Chalk 7±2
metamorphic	Non foliated		Marble 9±3	Hornfels (19±4) Metamorphic sandstone (19±3)	Quartzite 20±3	
	Slightly foliated		Migmatite (29±3)	Amphibolite 26±6	Gneiss 28±5	
	Laminated**			Schist 12±3	Phyllite (7±3)	Slate 7±4
Igneous	plutonic	light	Granite 32±3      Diorite 25±5 Granodiorite (29±3)			
		dark	Gabbro 27±3      Dolerite (16±5) Norite 20±5			
	hypabyssal		Porphyries (20±5)		Diabase (15±5)	Peridotite (25±5)
	volcanic	lava		Rhyolite (25±5) Andesite 25±5	Dacite (25±3) Basalt (25±5)	
		pyroclastic	Agglomerate (19±3)	Breccia (19±5)	Tuff (13±5)	

\* Conglomerates and breccias may present a wide range of  $m_i$  values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone, to values used for fine grained sediments (even under 10).

\*\* These values are for intact rock specimens tested normal to bedding or foliation. The value of  $m_i$  will be significantly different from that of point load test if failure occurs along a weakness plane.

\*\*\* Note that values in parenthesis are estimates. The range of values quoted for each material depends upon the granularity and interlocking of the crystal structure – the higher values being associated with tightly interlocked and more frictional characteristics.

#### H.4.2 The general formula of Hoek-Brown criteria for estimation of the strength based on GSI is

$$\sigma_1 = \sigma_3 + \sigma_c \left( m_b \frac{\sigma_3}{\sigma_c} + s \right)^a \quad (\text{H.2})$$

where  $m_b$ ,  $s$  and  $a$ :

for  $GSI > 25$  (undisturbed rock mass)

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \quad (\text{H.3})$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad (H.4)$$

$$a=0.5 \quad (H.5)$$

For  $GSI < 25$  (undisturbed rock mass)

$$s=0 \quad (H.6)$$

$$a = 0.65 - \frac{GSI}{200} \quad (H.7)$$

#### H.4.3 Estimate the shear strength parameters

The shear strength parameters can be estimated according to formula H.2 by fitting (A spreadsheet for carrying out these calculations prepared by Hoek can be referred). The following approximate method can be used too. Corresponding with a certain effective normal stress  $\sigma_n$  the shear strength  $\tau$  can be expressed as

$$\tau = (\cot \phi'_i - \cos \phi'_i) \frac{m\sigma_c}{8} \quad (H.8)$$

where:  $\sigma_c$ —uniaxial compression strength for intact rock;

$$\phi'_i = \arctan \frac{1}{\sqrt{4h \cos^2 \theta - 1}} \quad (H.9)$$

$$\theta = 30 + \frac{1}{3} \arctan \frac{1}{\sqrt{h^3 - 1}} \quad (H.10)$$

$$h = 1 + \frac{16(m\sigma_n + s\sigma_c)}{3m^2\sigma_c} \quad (H.11)$$

Having determined shear stresses  $\tau$  under different levels of normal stress  $\sigma_n$  according to formula H.8, the values  $c$  and  $\phi$  under a certain level of normal stress can be obtained by linear regression after linearization.

#### H. 5 Some stipulations for estimation of shear strength parameters by RMR and GSI

1. For slopes excavated without well controlled blasting the estimated rating of GSI should be subtracted by 10.
2. When the results of strength parameters are estimated by using the spreadsheet prepared by Hoek the value  $c$  should be decreased by 25%.
3. If there is ground water in the rock mass, the saturated uniaxial compression strength  $\sigma_c$  should be adopted.
4. During the rating of RMR and GSI and the estimation of shear strength by using formula G.8, all the imputed strength parameters should be the values corresponding with that of 0.2 on the probability distribution curves.

## Appendix I (Informative)

### Calculation of slide-resisting pile

**I.1** The direction of thrust force acting on a single pile is parallel with the sliding direction. The thrust force subjected by a single pile standing in a single line with other piles is determined according to the residual sliding force, which needs to be balanced to meet the design factor of safety of the sliding rock/soil mass, distributed in the area with a width of 1/2 center distance from the center on each side of the pile.

**I.2** The distribution of thrust force acting on slide-resisting piles can be considered in a form of triangle, rectangle or trapezoid in accordance with the material properties and the deformation sliding features of the landslide mass.

**I.3** The load bearing area of a single pile shall be calculated according to the equivalent width of the pile  $b_p$ . For a single pile or piles in a single row, it will be:

for piles with rectangle section,

$$\text{when } b \geq 1\text{m}, \quad b_p = b + 1, \text{ m} \quad (\text{I.1})$$

$$\text{when } b < 1\text{m}, \quad b_p = 1.5b + 0.5, \text{ m} \quad (\text{I.2})$$

for piles with circle section,

$$\text{when } d \geq 1\text{m}, \quad b_p = 0.9 (d + 1), \text{ m} \quad (\text{I.3})$$

$$\text{when } d < 1\text{m}, \quad b_p = 0.9 (1.5d + 0.5), \text{ m} \quad (\text{I.4})$$

where:  $b_p$ - the equivalent width of a single pile, m;

$b$ - the width of the rectangle-shaped pile, m;

$d$ - the diameter of the circle-shaped pile, m.

The total widths of  $n$  piles in a single row shall satisfy that

$$nb_p \leq B + 1, \text{ m} \quad (\text{I.5})$$

where:  $B$  is the total width enclosed by the out sides of the two border piles.

**I.4** In case the soil in front of piles is stable and no scouring failure occurs, the earth pressure in front face of piles can be assumed to be the residual slide-resisting force of the sliding mass in front of piles, the distribution of which is of rectangle-shape. When the passive earth pressure shall be less than the residual slide-resisting force, the slide-resisting force in front of piles is calculated according to the passive earth pressure. The passive earth pressure in front of piles can be calculated by the following equation:

$$E_p = (1/2)\gamma_1 \times h_1 \times \tan^2 (45^\circ + (\Phi/2)) \quad (\text{I.6})$$

where:  $E_p$  – passive earth pressure (kN/m)

$\gamma_1, \Phi$  – bulk density of the soil (kN/m<sup>3</sup>) in front of piles and its internal friction angle or equivalent internal friction angle( ° ) respectively;

$h_1$  – load bearing section length of a pile (m).

**I.5** The embedded length of pile is assumed to be of Winkler foundation. It is assumed that the horizontal displacement of piles equals that of the in-situ rock or soil; only the compressive stress can be transferred between pile and rock or soil but the tensile or shearing

stress can not be transferred. Provided the top of piles is at the ground surface level, under the action of horizontal force and moment, horizontal displacement and tilting will take place at the top of piles.

The horizontal resistance coefficient of foundation reinforced by piles embedded section, hereafter will be called foundation coefficient, of the length of piles can be calculated by the following formula:

$$K = m (y + y_0)^n \quad (I.7)$$

Where:  $K$  – foundation coefficient of the pile embedded section ( $\text{kN}/\text{m}^3$ );

$m$  – ratio coefficient of the foundation coefficient increasing with the depth;

$n$  – parameter related with rock-soil properties;

$y$  – thickness of the sliding mass in front of slide-resisting piles (m);

$y_0$  – distance from the bottom of embedded section up to sliding surface (m).

Foundation coefficient is related to the rock mass properties of the slide bed, that usually used in engineering can be simply summarized as the following three situations:

(1) K-method: the foundation coefficient is a constant,  $K=m$ ; that means in the formula (I.7)  $n=0$ . Generally it is considered that this method is applicable for the materials with cohesion being the major shear strength, or for the situation with small displacements of the piles;

(2) m-method: the foundation coefficient increases with the depth linearly,  $K=my$ ; that means in the formula (I.7)  $n=1$ ,  $y_0=0$ . Generally it is considered that this method is applicable for the materials with internal friction being their major shear strength, or for the situation with large displacement of piles.

(3) C-method: the foundation coefficient changes with the depth in a convex parabolic pattern; that means in the formula (I.7)  $0 < n < 1$ , generally it is assumed  $y_0=0$  and  $n=0.5$ .

The K- method and m- method are used generally, and applicability of the third one shall be determined through tests in situ.

The empirical values of the foundation coefficient for slide resisting pile design can be found in Tab.I.1 for reference.

**I.6** The earth pressure at the bottom and the lateral sides of slide resisting piles shall be lower than the design value of rock or soil strength. The allowable lateral pressure of the surrounding rock of embedded length of piles can be determined by the following formulas:

(1) For fairly massive and sound rock mass

$$\sigma_{\text{MAX}} \leq \rho_1 \times R \quad (I.8)$$

where:  $\sigma_{\text{MAX}}$  - Allowable maximum lateral pressure of surrounding rock for the embedded length of piles (MPa);

$\rho_1$  - Reduction coefficient depending on the rock mass properties, which can be taken as 0.1~0.5 according to the actual conditions;

$R$  - Unconfined compressive strength of rock (MPa).

(2) For severely weathered and fragmented rock, accumulated rock blocks or soil mass

$$\sigma_{\text{MAX}} \leq \rho_2 \times (\sigma_P - \sigma_A) \quad (I.9)$$

where:  $\sigma_{MAX}$  - Allowable maximum lateral pressure of rock-soil mass of pile embedded length (MPa);

$\rho_2$  - Reduction coefficient depending on rock-soil mass properties, which can be taken as 0.1~0.5 according to the actual conditions;

$\sigma_P$  - Passive earth pressure in front of piles (MPa);

$\sigma_A$  - Active earth pressure behind piles (MPa).

**Table.I.1 Physical and mechanical properties of rock and foundation coefficients of slide resisting piles K**

Rock type	Internal friction angle	Young's modulus $E_0$ 10 <sup>4</sup> kPa	Poisson's ratio $\mu$	Foundation coefficient K 10 <sup>4</sup> kPa/m (kN/m <sup>3</sup> )
Fine-grained granite, Syenite, Diabase, Porphyrite	More than 80°	5430~6900 6700~7870	0.25~0.30 0.28	2.0~2.5 2.5
Medium-grained granite, Coarse-grained syenite, Hard dolomite	More than 80°	5430~6500	0.25 0.25	1.8~2.0
Hard limestone, Hard sandstone, Marble, Coarse-grained granite, Granite gneiss	More than 80°	4400~10000 4660~5430 5430~6000	0.25~0.30	1.2~2.0
Fairly hard limestone Fairly hard sandstone Non-hard granite	75°~80°	4400~9000 4460~5000 5430~6000	0.25~0.30	0.8~1.2
Hard shale rock Common limestone Common sandstone	70°~75°	2000~5500 4400~8000 4600~5000	0.15~0.30 0.25~0.30 0.25~0.30	0.4~0.8
Hard marlstone Fairly hard shale rock Non-hard limestone Non- hard sandstone	70°	800~1200 1980~3600 4400~6000 1000~2780	0.29~0.38 0.25~0.30 0.25~0.30 0.25~0.30	0.3~0.4
Fairly hard marlstone Common shale rock Soft limestone	65°	700~900 1900~3000 4400~5000	29~0.38 15~0.20 0.25	0.2~0.3
Non-hard marlstone Stiff clay Soft schist Hard coal	45°	30~500 10~300 500~700 50~300	0.29~0.38 0.30~0.37 0.15~0.18 0.30~0.40	0.06~0.12
Dense clay Common coal Cemented gravels Soil mixed with stone	30°~45°	10~300 50~300 50~100 50~100	0.30~0.37 0.30~0.40	0.3~0.4
Note: quoted from "Regulation for design of the retaining structures for railway foundation" (TBJ25-90)				

**I.7** During the internal force calculation the first thing is to judge whether the slide resisting piles are rigid ones or elastic ones, by which the corresponding calculation formulas are selected. The attribution of the slide resisting piles is judged by the following formulas

depending on the deformation coefficient of the piles.

(1) For K-method, the deformation coefficient of a pile is  $\beta$ , ( $\text{m}^{-1}$ )

$$\beta = (Kb_p/4EI)^{1/4} \quad (\text{I.10})$$

where:  $K$ — foundation coefficient ( $\text{kN/m}^3$ ) ;

$b_p$ — calculation width of a pile (m) ;

$E$ — elastic modulus of a pile (kPa) ;

$I$ — inertia moment of pile cross section ( $\text{m}^4$ ) .

The criterion:

$\beta h_2 \leq 1.0$ -rigid pile;

$\beta h_2 > 1.0$ -elastic pile;

where  $h_2$  is the embedded length of a pile.

(2) For m-method, the deformation coefficient of a pile is  $\alpha$ , ( $\text{m}^{-1}$ )

$$\alpha = (mb_p/EI)^{1/5} \quad (\text{I.11})$$

where:  $m$ —ratio coefficient of the foundation coefficient increasing with the depth;

The other signs are the same in formula (I.10).

The criterion:

$\alpha h_2 \leq 2.5$ -rigid pile;

$\alpha h_2 > 2.5$ -elastic pile;

where  $h_2$  is the embedded length of a pile (m).