Foreword

According to the requirements of Document JIANBIAO [2012] No. 5 issued by the Ministry of Housing and Urban-Rural Development (MOHURD) of the People's Republic of China—"Notice on Printing and Distributing 'the Development and Revision Plan of National Engineering Construction Standards in 2012'", and after extensive investigation and research, summarization of practical experience, and wide solicitation of opinions, the drafting group has prepared this standard.

This standard comprises 14 chapters and 2 appendixes with the main technical contents on seismic design of hydraulic structures of hydropower plant, covering: general provisions; terms and symbols; basic requirements; site, foundation and slope; seismic action and seismic calculation; embankment dam; gravity dam; arch dam; sluice; underground hydraulic structures; intake tower; penstock and surface powerhouse of hydropower station; aqueduct; shiplift, etc.

The provisions printed in bold type are mandatory ones and must be implemented strictly.

The Ministry of Housing and Urban-Rural Development of the People's Republic of China is in charge of administration of this standard and explanation of its mandatory provisions, the Ministry of Water Resources of the People's Republic of China is responsible for its routine management, China Institute of Water Resources and Hydropower Research is in charge of explanation of specific technical contents. During implementation of this standard, any comments and advices can be posted or passed on to China Institute of Water Resources and Hydropower Research (Address: No.20, Chegongzhuang West Road, Haidian District, Beijing, Postcode: 100048).

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Contents

1 General provisions

1.0.1 This standard is formulated in accordance with the Law of the People's Republic of China on Protecting Against and Mitigating Earthquake Disasters, and with a view to carrying out the policy of prevention first, to mitigate earthquake damage and prevent secondary disasters through seismic design of hydraulic structures.

1.0.2 The hydraulic structure designed as per this standard shall be able to resist the seismic action of the design intensity, and remain functional after repair of local damages, if any.

1.0.3 This standard is mainly applicable to seismic design of Grade 1, Grade 2 and Grade 3 hydraulic structures with design intensity of Ⅵ, Ⅶ, Ⅷ and Ⅸ, such as the roller-compacted embankment dam, concrete gravity dam, concrete arch dam, sluice, underground hydraulic structures, intake tower, penstock and surface powerhouse of hydropower station, aqueduct, shiplift, etc.

For hydraulic structures with design intensity of Ⅵ, seismic calculation may not be required, but seismic measures shall still be taken in accordance with this standard.

For hydraulic structures with design intensity above Ⅸ, and water-retaining structures higher than 200m or with unfavorable conditions, special study and demonstration shall be carried out on their seismic safety.

1.0.4 For general projects, the design peak ground acceleration (PGA) on the project site and corresponding design intensity shall be determined in accordance with the current national standard GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*.

1.0.5 For large-scale (Rank 1) projects with a dam height over 200m or reservoir storage capacity over 10 billion m^3 in the regions with a basic intensity of Ⅵ or above, and large-scale (Rank 1) projects with a dam height over 150m in the regions with a basic intensity of Ⅶ or above, the design peak ground acceleration on the project site and corresponding design intensity shall be determined based on site-specific seismic safety evaluation.

1.0.6 For Grade 1 and Grade 2 dams with a height over 90m, main structures of Rank 1 pumped storage power stations and important structures of water diversion projects in the regions with a basic intensity of Ⅶ or above, the design peak ground acceleration on the project site and corresponding design intensity may be determined based on site-specific seismic safety evaluation after techno-economic demonstration.

1.0.7 In addition to this standard, the seismic design of hydraulic structures shall comply with other current relevant standards of the nation.

2 Terms and symbols

2.1 Terms

2.1.1 seismic design

Special design of engineering structures in earthquake regions, generally including seismic calculation and seismic measures.

2.1.2 basic intensity

Seismic intensity of general site with a 10% probability of exceedance in 50 years, which is usually determined according to the peak ground acceleration specified in the current national standard GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*, and corresponding to the seismic intensity specified in the Appendix. For major projects, it shall be determined through site-specific seismic safety evaluation.

2.1.3 design intensity

Seismic intensity for engineering fortification determined on the basis of basic intensity.

2.1.4 reservoir earthquake

Earthquake related to reservoir impounding, whichever occurs within a scope of less than 10km away from the reservoir rims.

2.1.5 maximum credible earthquake(MCE)

Earthquake with potential maximum ground motion assessed based on the regional geological and seismological conditions around project site.

2.1.6 scenario earthquake

Earthquake having a particular magnitude and epicentral distance, with the maximum probability of exceedance of design peak ground acceleration in a source that makes the maximum contribution to design peak ground acceleration on a project site among potential seismic sources, based on the result of site-specific seismic safety evaluation.

2.1.7 seismic ground motion

Ground motion induced by earthquake.

2.1.8 seismic action

Dynamic actions of seismic ground motion on structures.

2.1.9 hanging wall effect

Phenomenon that seismic ground motion of hanging wall above the inclined seismogenic fault is larger than that of footwall.

2.1.10 peak ground acceleration(PGA)

Maximum absolute value of ground mass point motion acceleration during earthquake.

2.1.11 design earthquake

Seismic ground motion for seismic fortification corresponding to design intensity, whose parameters include peak ground acceleration, response spectrum, duration, and acceleration time history.

2.1.12 design peak ground acceleration

Peak ground acceleration of fortification probability level specified by site-specific seismic safety evaluation on project site, or generally corresponding to the design intensity.

2.1.13 seismic effect

Dynamic effect such as structure internal force, deformation, sliding and cracking caused by seismic action.

2.1.14 seismic liquefaction

Process in which, induced by the seismic ground motion, the particles of saturated cohesionless soil or less cohesive soil grow denser, soil pore water pressure increases, and the effective stress of the soil approaches zero.

2.1.15 design response spectrum

Curve that plots the maximum ground acceleration as a function of the natural vibration period of single-degree-of-freedom (SDOF) system considering a given damping ratio, which may be expressed by the ratio of the maximum acceleration response to the peak ground acceleration.

2.1.16 dynamic method

Method to analyze seismic effect of structure based on the theory of structural dynamics.

2.1.17 time history analysis method

Method to analyze seismic effect in whole time history by integrating the governing motion equation of structure with accelerogram as seismic input.

2.1.18 mode decomposition method

Method to analyze seismic effect of structure, in which the total seismic effect of the structure is obtained by superposition of seismic effect of each mode. It is called the mode decomposition time history analysis method, when the time history analysis is used to obtain the seismic effect of each mode. It is called the mode decomposition response spectrum method, when the response spectrum is used to obtain the seismic effect of each mode.

2.1.19 square root of the sum of squares (SRSS) method

Method to evaluate the maximum response of structure by the square root of the sum of the squares of various mode seismic effects.

2.1.20 complete quadratic combination (CQC) method

Method to evaluate the maximum response of structure by the square root of the sum of quadratic terms of various mode seismic effects and coupling terms.

2.1.21 seismic hydrodynamic pressure

Dynamic pressure of water on structure caused by earthquake.

2.1.22 seismic earth pressure

Dynamic pressure of soil mass on structure caused by earthquake.

2.1.23 pseudo-static method

Static analysis method taking the product of gravity action, ratio of design seismic peak acceleration to gravity acceleration, specified seismic effect reduction factor and dynamic distribution coefficient as the design seismic action.

2.1.24 seismic effect reduction factor

Reduction factor for seismic effects introduced due to simplification in analysis method.

2.1.25 natural vibration period

Time interval for structure to complete a free vibration cycle in a certain vibration mode. The natural vibration period corresponding to the first vibration mode is called the fundamental period.

2.1.26 seismic measures

Seismic design except the calculation of seismic action and resistance, including details of seismic design.

2.1.27 details of seismic design

Various detailed measures that must be taken for structural and non-structural members without justification by seismic calculation, according to basic requirements of seismic design.

2.2 Symbols

2.2.1 Actions and effects:

a_h—representative value of horizontal design peak ground acceleration;

a_v—representative value of vertical design peak ground acceleration;

E_i—representative value of horizontal seismic inertial force acting on mass point i;

F_E—representative value of seismic active earth pressure;

F_0—representative value of total seismic hydrodynamic pressure on water-contact face per unit width of structure;

g—gravity acceleration, which is taken as 9.81m/s^2;

G_E—characteristic value of structure total gravity action that produces seismic inertial force;

$P_w(h)$—representative value of seismic hydrodynamic pressure at water depth h;

α_i—dynamic distribution coefficient of seismic inertial force of mass point i;

β—design response spectrum;

ξ—seismic effect reduction factor.

2.2.2 Material properties and geometric parameters:

a_k—characteristic value of geometric parameter;

f_k—characteristic value of material property;

K_L—characteristic values of longitudinal stiffness coefficient of unit length of tunnel surrounding mass;

K_T—characteristic values of transverse stiffness coefficient of unit length of tunnel surrounding mass;

N—blow count of standard penetration test;

N_{cr}—critical blow count;

v_p—characteristic value of compression wave velocity;

v_s—characteristic value of shear wave velocity;

ρ_w—characteristic value of water mass density.

2.2.3 Limit state design using partial factor:

E_k—representative value of seismic action;

G_k—characteristic value of permanent action;

Q_k—characteristic value of variable action;

R—resistance of structure;

S—action effect of structure;

γ_0—importance factor of structure;

γ_d—structural factor, safety margin introduced for non-random uncertainty on the ultimate limit state of bearing capacity;

γ_E—partial factor for seismic action;

γ_G—partial factor for permanent action;

γ_m—partial factor for material property;

γ_Q—partial factor for variable action;

ψ—design situation factor.

2.2.4 Others:

T_g—characteristic period;

T—natural vibration period of structure;

λ_m—mass ratio of appurtenant structure to main structure;

λ_f—fundamental frequency ratio of appurtenant structure to main structure.

3 Basic requirements

3.0.1 The seismic fortification class of hydraulic structures shall be determined based on their importance and basic seismic intensity on their sites according to Table 3.0.1.

Table 3.0.1 Classification of seismic fortification

Seismic fortification class	Grade of structure	Site basic intensity
A	Water-retaining and important water-releasing structures of Grade 1	≥Ⅵ
B	Non-water-retaining structure of Grade 1 and water-retaining structure of Grade 2	
C	Non-water-retaining structure of Grade 2 and structure of Grade 3	≥Ⅶ
D	Structure of Grade 4 and Grade 5	

Note: Important water-releasing structures refer to those whose failure might endanger the safety of water-retaining structures.

3.0.2 The seismic fortification class of hydraulic structures shall be represented in terms of design intensity and horizontal design peak ground acceleration on flat ground surface after site class adjustment, and shall comply with Article 3.0.3 to Article 3.0.8 in this standard.

3.0.3 For hydraulic structures whose seismic fortification classes are determined in accordance with the current national standard GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*, in the case of general projects, the value of the peak ground acceleration on their sites shall be taken from zonation map as the representative value of the horizontal design peak ground acceleration, and the corresponding basic intensity is taken as the design intensity. In the case of hydraulic structures assigned to seismic fortification Class A, their design intensity shall be one level higher than the basic intensity, and the representative value of the horizontal design peak ground acceleration shall be doubled accordingly.

3.0.4 For projects whose seismic fortification criteria are based on site-specific seismic safety evaluation, the probability of exceedance of the representative values of horizontal design peak ground acceleration, P_{100}, on the flat rock foundation surface shall be 0.02 in 100 years for water-retaining structures and important water-releasing structures assigned to seismic fortification Class A. An probability of exceedance in 50 years, P_{50}, shall be 0.05 for Grade 1 non-water-retaining structures. An probability of exceedance in 50 years, P_{50}, shall be 0.10 for hydraulic structures assigned to other seismic fortification classes than Class A, and the corresponding peak ground acceleration shall not be lower than that specified in the current national standard GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*.

3.0.5 For hydraulic structures assigned to seismic fortification Class A whose design seismic parameters shall be provided by the site-specific seismic safety evaluation, a special demonstration on safety margin under the maximum credible earthquake (MCE) shall be carried out on disaster prevention of the uncontrolled release of reservoir in addition to the seismic design under design peak ground acceleration. A special report on seismic safety shall be prepared. The MCE of the site shall be determined by the deterministic method or the probabilistic method with an probability of exceedance of 0.01 in 100 years.

3.0.6 In the special report on seismic safety, relevant site-specific design response spectrum should be determined based on scenario earthquake corresponding to horizontal design peak ground acceleration, and artificial accelerograms are generated. For analyzing the seismic effect on structures with strong non-linearity, the influence arising from non-stationary frequency of ground motion should be studied when conditions permit. When the distance from the seismogenic fault to the site is less than 30km and its inclination angle is smaller than 70°, hanging wall effect should be considered. When the distance is less than 10km and the magnitude is over 7.0, the rupture process of seismogenic fault as the area source of the near-field strong earthquake ground motions should be studied to generate directly the random time histories of ground motions, and then to select the time histories with the peak period of evolutionary spectrum closest to the fundamental period of structure.

3.0.7 When the grade of water-retaining structure is raised due to the dam height, special study on the seismic fortification standard shall be performed and reported to competent authorities for approval.

3.0.8 Seismic actions may not be involved in the case of relatively short period of construction.

3.0.9 For new reservoirs with the dam higher than 100m and storage capacity larger than 500 million m^3, an evaluation of reservoir earthquake shall be conducted. In the case of potential reservoir earthquake of magnitude higher than 5.0 or epicentral intensity higher than Ⅶ, a reservoir earthquake monitoring network shall be established and put into operation at least one year prior to the initial impoundment.

3.0.10 The seismic design for hydraulic structures shall include seismic calculation and seismic measures, and shall be compliance with the following requirements:

1 Select the region, site and structure type favorable for seismic resistance according to the seismic requirements.

2 Prevent stability failure of foundation and slopes adjacent to the structures.

3 Select safe and cost-effective structures and measures for earthquake resistance.

4 Propose the construction quality control measures meeting the seismic safety requirements in design documents.

5 Arrange water-releasing facilities that can lower the reservoir level as quickly as possible.

6 Conduct seismic designs for non-structural elements, appurtenant electromechanical equipment and their connections with main structures in hydraulic structures, such as sluice, intake tower and shiplift.

3.0.11 The requirements for emergency plan to prevent and mitigate earthquake hazard shall be proposed in the design document for hydraulic structures with seismic requirements.

3.0.12 Dynamic model test should be conducted for dams assigned to seismic fortification Class A with the design intensity of Ⅷ and above, and a height of more than 150m.

3.0.13 The seismic monitoring array design for strong-motion observation shall meet the requirements of the current professional standard SL 486 *Technical Specification of Strong Motion Monitoring for Seismic Safety of Hydraulic Structures* or DL/T 5416 *Specification of Strong Motion Safety Monitoring for Hydraulic Structures*.

4 Site, foundation and slope

4.1 Site

4.1.1 In site selection for a hydraulic structure, a comprehensive evaluation shall be performed in terms of tectonic activity, the stability of site foundation and slope, and the risk of secondary disasters, etc., based on engineering geological and hydrogeological exploration and seismicity investigation. The site shall be classified into four categories: favorable, normal, unfavorable and hazardous according to Table 4.1.1. Favorable or normal site for seismic safety should be selected, while unfavorable and hazardous sites should be avoided. A thorough seismic safety evaluation must be conducted for a dam constructed in unfavorable and hazardous sites.

Table 4.1.1 Classification of site

Site class	Tectonic activity	Stability of site foundation and slope	Risk of secondary disaster
Favorable	No active fault within 25km around the site, with basic intensity of Ⅵ	Good	Very low
Normal	No active fault within 5km around the site, with basic intensity of Ⅶ	Fair	Low
Unfavorable	There are active faults of less than 10km in length within 5km around the site, and seismogenic structures with a magnitude less than 5.0. The basic intensity is Ⅷ	Poor	High
Hazardous	There are active faults not shorter than 10km within 5km around the site, and seismogenic structures with a magnitude greater than 5.0. The basic intensity is Ⅸ	Very poor	Very high

4.1.2 The site soils after excavation and treatment for a hydraulic structure should be classified according to the shear wave velocity of soil layers shown in Table 4.1.2, and shall be in accordance with the following requirements:

1 The shear wave velocity v_s of site soil, or the equivalent shear wave velocity of each soil layer beneath the foundation in the case of multi-layered site soil, shall be calculated according to the following formula:

$$v_s = \frac{d_0}{\sum_{i=1}^{n}(d_i / v_{si})} \tag{4.1.2}$$

where d_0—overburden thickness (m);

d_i—thickness of the ith soil layer (m);

v_{si}—shear wave velocity of the ith soil layer (m/s);

n—number of overburden soil layers.

2 The determination of overburden thickness d_0 shall be in accordance with the following requirements:

1) The thickness shall be determined by the distance from the ground or foundation surface to the top of the soil layer, whose shear wave velocity is more than 500m/s and the shear wave velocity of layers beneath which is not less than 500m/s.

2) The thickness shall be determined by the distance from the ground or foundation surface to the top of the layer, whose depth is more than 5m and shear wave velocity is more than 2.5 times the overlying soil layer and the shear wave velocity of itself and underlying layers is not less than 400m/s. The boulders and lenticles with a shear wave velocity greater than 500m/s shall be deemed the same as surrounding soil layer.

3) The hard rock layer intercalated in soil shall be considered as rigid body and its thickness shall be deducted from the overburden thickness.

Table 4.1.2 Classification of site soil

Site soil class	Shear wave velocity v_s(m/s)	Descriptions and properties
Hard rock	$v_s>800$	Stiff, hard and intact rocks
Soft rock and hard soil	$800\geqslant v_s>500$	Fractured and partially fractured rocks, or soft and intermediate rocks; dense sandy gravels
Moderately hard soil	$500\geqslant v_s>250$	Moderate-dense and slight-dense sandy gravels; dense coarse sand and medium sand; hard clay or silt
Moderately soft soil	$250\geqslant v_s>150$	Slight-dense gravels, coarse sand, medium sand and fine sand and silty sand; ordinary clay and silt
Soft soil	$v_s\leqslant150$	Muck; mucky soil; loose sandy soil; miscellaneous fill

4.1.3 Sites shall be classified into five classes, namely Ⅰ$_0$, Ⅰ$_1$, Ⅱ, Ⅲ, and Ⅳ, according to the type of site soil and overburden thickness, as shown in Table 4.1.3.

Table 4.1.3 Classification of site

Site soil class	Overburden thickness d_s(m)						
	0	$0<d_s\leqslant3$	$3<d_s\leqslant5$	$5<d_s\leqslant15$	$15<d_s\leqslant50$	$50<d_s\leqslant80$	$d_s>80$
Hard rock	Ⅰ$_0$	–					
Soft rock and hard soil	Ⅰ$_1$	–					
Moderately hard soil	–	Ⅰ$_1$		Ⅱ			
Moderately soft soil	–	Ⅰ$_1$	Ⅱ			Ⅲ	
Soft soil	–	Ⅰ$_1$	Ⅱ		Ⅲ		Ⅳ

4.2 Foundation

4.2.1 In seismic design of foundation for hydraulic structures, the type, load, hydropower and operating conditions of structures, as well as engineering geological and hydrogeological conditions of foundation and bank slope shall be considered comprehensively.

4.2.2 For foundation and bank slope of water-retaining structures, such as dam and sluice, the criteria on stability against earthquake liquefaction, earthquake subsidence of weak clay and seepage deformation under design seismic action shall be met. The detrimental deformation to the structures shall be avoided.

4.2.3 For weak discontinuities in foundation and bank slope of hydraulic structures, such as faults, fractured zones, dislocation zones, and especially, low-dip clay-interbedded layers and argillization-liable rock layers, the stability and allowable deformation under design seismic action shall be verified according to their occurrence, buried depth, boundary conditions, seepage, physical and mechanical properties and design intensity of structures, seismic measures shall be taken if necessary.

4.2.4 For seepage control system and its connections, drainage and filters of foundation and bank slope for hydraulic structure, effective measures shall be taken to prevent hazardous cracks or seepage damage under earthquakes.

4.2.5 For heterogeneous foundations, whose material properties and thickness vary greatly in horizontal direction, measures shall be taken to prevent large differential settlement, sliding and concentrated seepage, and to improve the capacity of structure to tolerate differential settlement of the foundation.

4.2.6 Liquefaction of soil layer in foundation shall be identified according to the current national standards GB 50287 *Code for Hydropower Engineering Geological Investigation* and GB 50487 *Code for Engineering Geological Investigation of Water Resources and Hydropower*.

4.2.7 For potential liquefaction soil layers in foundation, the following seismic measures may be taken according to the type of structures and specific conditions of the project:

1 Remove liquefiable soil layers and replace with non-liquefiable soil.

2 Use artificial compaction strengthening methods, including vibroflotation and strong ramming, etc.

3 Adopt counter weight and drainage measures.

4 Adopt compound foundation like vibration-compacted stone column, or foundation with piles penetrating through liquefiable soil layer into non-liquefiable soil layer.

5 Confine liquefiable foundation soil by continuous concrete walls or other measures.

4.2.8 For soft clay layers in the foundations of hydraulic structure assigned to seismic fortification Class A or Class B, special seismic test and analysis shall be carried out. Unless otherwise specified, foundation soil may be identified as a soft clay layer if any one of the following criteria is met:

1 Liquidity index I_L greater than or equal to 0.75.

2 Unconfined compressive strength q_u less than or equal to 50kPa.

3 Blow count of standard penetration test N less than or equal to 4.

4 Sensitivity S_t greater than or equal to 4.

4.2.9 For soft clay layers in foundation, the following seismic measures may be taken according to the type of structures and specific conditions:

1 Remove or replace soft clay in the foundation.

2 Consolidate the layers with the preloading method.

3 Adopt counter weight and sand well drain or plastic drainage board.

4 Adopt composite foundation, such as pile foundation, vibration-compacted stone column, etc.

4.3 Slope

4.3.1 For hydraulic structures site where complicated rock mass structures, weak discontinuities or unfavorable combinations of clay-interlayer exist, and with poor stability of slope, the distribution of unstable slopes under design seismic action shall be identified, the potential hazard shall be analyzed and treatment measures shall be proposed.

4.3.2 The design intensity and representative value of design peak ground acceleration of slope shall be determined based on comprehensive demonstration of the seismic fortification class of the relevant hydraulic structures, the correlation between the slope and the hydraulic structures, and the impact of the slope failure on the hydraulic structures, etc.

4.3.3 The rigid limit equilibrium method may be adopted for calculation of slope seismic stability. Dynamic amplification effect of slope seismic inertial force may be neglected and the shear strength with cohesion may be taken as the static shear strength.

4.3.4 The seismic analysis and the selection of safety factor for slopes shall comply with the current professional standard SL 386 *Design Code for Engineered Slopes in Water Resources and Hydropower Projects* or DL/T 5353 *Design Specification for Slope of Hydropower and Water Conservancy Project*.

4.3.5 For high slopes with complicated geological conditions, special studies should be conducted based on dynamic analysis. The deformation and seismic stability safety of the slope shall be analyzed comprehensively on seismic effects such as the displacements, residual displacements or opening of sliding plane of the slopes.

5 Seismic action and seismic calculation

5.1 Seismic ground motion components and combination

5.1.1 In general, it may be permitted to take into account only the horizontal seismic actions for hydraulic structures other than aqueducts.

5.1.2 For aqueducts with design intensity of Ⅶ and above, as well as Grade 1 or Grade 2 hydraulic structures with design intensity of Ⅷ or Ⅸ including embankment dams, gravity dams and other water-remaining structures, long cantilevered, large-span or high-rising hydraulic concrete structures, both horizontal and vertical seismic actions shall be taken into account. The representative value of vertical design peak ground acceleration may be taken as 2/3 of the representative value of the horizontal design peak ground acceleration, but shall be the representative value of the horizontal design peak ground acceleration for near-field earthquakes.

5.1.3 For arch dams of special types, obviously asymmetric or hollow ones, and for Grade 1 and Grade 2 double-curvature arch dams with design intensity of Ⅷ or Ⅸ, the vertical seismic effects should be studied specially.

5.1.4 For embankment dams and concrete gravity dams, it may be permitted to take into account only the horizontal seismic actions in the stream direction on seismic design. For monolith of gravity dam on steep abutment, the horizontal seismic actions along cross-stream direction should be considered. For important embankment dams, the horizontal seismic actions along cross-stream direction should be specially studied.

5.1.5 For concrete arch dams and sluices, the horizontal seismic actions in both stream and cross-stream directions shall be considered.

5.1.6 For intake towers, frames on the top of sluices and other hydraulic concrete structures with similar stiffness along the two principal axial directions, horizontal seismic actions along the two principal axial directions of structures shall be considered.

5.1.7 When the seismic effects in orthogonal directions are calculated simultaneously by the mode decomposition response spectrum method, the overall seismic effects may be taken as the square root of the sum of squares (SRSS) of seismic effects in each direction.

5.2 Classification of seismic actions

5.2.1 Seismic actions to be considered in seismic calculation of hydraulic structures shall include the inertial force induced by dead weight of structures and facilities, seismic earth pressure and seismic hydrodynamic pressure, as well as seismic pore water pressure.

5.2.2 Seismic hydrodynamic pressure may be ignored for embankment dams except concrete face rockfill dams (CFRDs).

5.2.3 Seismic effect on wave pressure, seepage pressure and uplift pressure may be ignored.

5.2.4 Generally, seismic effect on silt pressure may be ignored, but the water depth in front of structure shall include silt deposit depth in seismic hydrodynamic pressure calculation; if a high dam has an extremely deep silt deposit, the seismic effect on the silt pressure shall be studied specially.

5.3 Design response spectrum

5.3.1 For hydraulic structures assigned to seismic fortification Class A requiring site-specific seismic safety evaluation, the site-specific design response spectrum specified in Article 3.0.6 in this standard shall be adopted as the design response spectrum. For other structures, standard design response spectrum shall be adopted as the horizontal and vertical design response spectrum.

5.3.2 The shape parameters of standard design response spectrum β (Figure 5.3.2) shall be in accordance with the following requirements:

1 For the period less than 0.1s, $\beta(T)$ is taken as a straight line ranging from 1.0 to β_{max}; where T is the natural vibration period of structure.

2 For the period between 0.1s and the characteristic period T_g, $\beta(T)$ is equal to the maximum value β_{max}.

3 For the period between the characteristic period T_g and 3.0s, $\beta(T) = \beta_{max}(T_g/T)^{0.6}$.

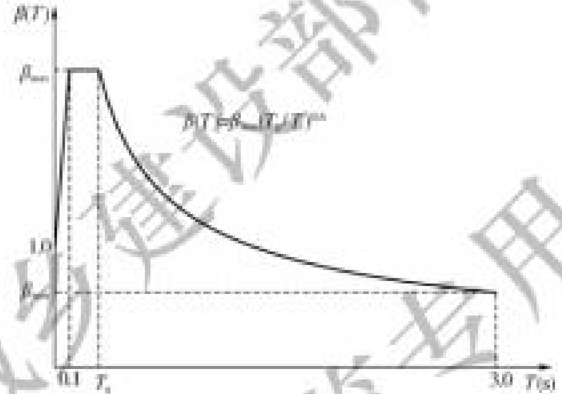


Figure 5.3.2 Standard design response spectrum

5.3.3 The representative value of maximum value of standard design response spectrum β_{max} for various hydraulic structures shall be taken in accordance with Table 5.3.3.

Table 5.3.3 Representative value of maximum value of standard design response spectrum β_{max}

Structure type	Embankment dam	Gravity dam	Arch dam	Other structures including sluice, intake tower, etc., and slope
β_{max}	1.60	2.00	2.50	2.25

5.3.4 The representative value of minimum value of standard design response spectrum β_{min} shall not be less than 20% of the representative value of maximum value of design response spectrum.

5.3.5 The characteristic periods of standard design response spectrum T_g for different site classes may be selected according to the site location specified in the current national standard GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*, and adjusted in accordance with Table 5.3.5.

Table 5.3.5 Adjustment for characteristic period of standard design response spectrum of site

Characteristic period of basic response spectrum of site Class Ⅱ	Site class				
	$Ⅰ_0$	$Ⅰ_1$	Ⅱ	Ⅲ	Ⅳ
0.35s	0.20s	0.25s	0.35s	0.45s	0.65s
0.40s	0.25s	0.30s	0.40s	0.55s	0.75s
0.45s	0.30s	0.35s	0.45s	0.65s	0.90s

5.4 Combination of seismic action with other actions

5.4.1 Upstream pool level in seismic calculation of hydraulic structures may adopt the normal pool level; for overyear storage reservoir, it may be lower than the normal pool level after demonstration.

5.4.2 For upstream slope of embankment dam, a frequently occurred pool level most unfavorable to its seismic stability shall be adopted in seismic calculation according to the operation condition. If necessary, the joint action of earthquake motions and frequent pool level draw-down may also be considered.

5.4.3 The seismic strength calculation of important arch dam and sluice should be supplemented by checking of the combination of seismic action with frequent low pool level.

5.5 Structural modeling and calculation method

5.5.1 The structural modeling of seismic effect for various hydraulic structures shall be the same as that stipulated in their corresponding design standards.

5.5.2 Except for embankment dams in narrow valleys and gravity dams with grouted transverse joints, the seismic calculation of gravity dams, sluices and embankment dams may be conducted in terms of unit width of dam or sluice or a monolith of dam or sluice.

5.5.3 In addition to the relevant provisions in this standard, the calculation method of the seismic effects for hydraulic structures shall be adopted according to the seismic fortification classes, specified in Table 5.5.3.

Table 5.5.3 Calculation method of seismic effect

Seismic fortification class	Calculation method of seismic effect
A	Dynamic method, and for embankment dams pseudo-static method may be adopted at the same time
B, C	Dynamic method or pseudo-static method
D	Pseudo-static method or taking seismic measures without calculation

5.5.4 For linear elasticity analysis of hydraulic structures, the mode decomposition response spectrum method or the mode decomposition time history analysis method may be adopted for calculating seismic effects, in which only the elasticity influences of foundation are taken into account. The damping ratios for various hydraulic structures may be taken as follows: 20% for embankment dams, 10% for gravity dams, 5% for arch dams, 7% for sluices, intake towers and other structures. For damping ratio of slope, special studies shall be carried out.

5.5.5 For concrete gravity dams and arch dams assigned to seismic fortification Class A, the following factors shall be taken into account in the dynamic analysis model for special seismic study:

1 Dynamic interaction of structure-foundation-reservoir system, foundation mass, rock type and tectonic structures of near-field foundation, the far-field radiation damping, non-uniform ground motion along the dam-foundation interface.

2 Opening and closing, and sliding of arch dam transverse joints during earthquake.

3 The massless foundation model may be used to consider the dynamic interaction between structure and foundation for other hydraulic structures when the seismic effects are calculated by the dynamic method.

5.5.6 When the mode decomposition response spectrum method is used, seismic effects of various modes may be combined by the square root of the sum of squares (SRSS) method. If the ratio of the absolute value of the frequency difference between two modes to the minor frequency is less than 0.1, seismic effects should be calculated by the complete quadratic combination (CQC) method:

$$S_E = \sqrt{\sum_i^m \sum_j^m \rho_{ij} S_i S_j} \tag{5.5.6-1}$$

$$\rho_{ij} = \frac{8\sqrt{\zeta_i \zeta_j}\,(\zeta_i + \gamma_\omega \zeta_j)\gamma_\omega^{3/2}}{(1-\gamma_\omega^2)^2 + 4\zeta_i \zeta_j \gamma_\omega (1+\gamma_\omega^2) + 4(\zeta_i^2 + \zeta_j^2)\gamma_\omega^2} \tag{5.5.6-2}$$

where m—number of modes involved in calculation;

S_E—seismic effect;

S_i, S_j—seismic effects of ith and jth modes, respectively;

ρ_{ij}—correlation coefficient of ith and jth modes;

ζ_i, ζ_j—damping ratios of ith and jth modes, respectively;

γ_ω—circular frequency ratio, $\gamma_\omega = \omega_j / \omega_i$;

ω_i, ω_j—circular frequencies of ith and jth modes, respectively.

5.5.7 Higher-order modes with contribution of not more than 5% to seismic effects may be ignored. When the lumped mass model is used, the number of lumped masses should not be less than four times that of modes used in calculation of seismic effects.

5.5.8 When the time history analysis method is used for calculating seismic effects, design response spectrum with 5% damping ratio shall be taken as target spectrum. At least three sets of artificial accelerograms shall be generated as the design seismic accelerograms, and the correlation coefficient between components of each set of those accelerograms shall not be larger than 0.3. The calculated results with various seismic accelerograms shall be analyzed comprehensively to determine the seismic effects for design.

5.5.9 When the pseudo-static method is used for calculation of seismic effect, the representative value of horizontal seismic inertial force acting on mass point i shall be calculated according to the following formula:

$$E_i = a_h \xi G_{Ei} \alpha_i / g \tag{5.5.9}$$

where E_i—representative value of horizontal seismic inertial force acting on mass point i;

a_h—representative value of horizontal design peak ground acceleration;

ξ—seismic effect reduction factor, which shall be taken as 0.25 except for reinforced concrete structures analyzed using the dynamic method;

G_{Ei}—characteristic value of gravity action concentrated on mass point i;

α_i—dynamic distribution coefficient of seismic inertial force of mass point i, which shall be taken in accordance with the relevant provisions of this standard;

g—gravity acceleration.

5.6 Dynamic properties of concrete and foundation rock for hydraulic structures

5.6.1 For mass concrete hydraulic structures assigned to seismic fortification Class A, the dynamic properties of concrete shall be determined by test.

5.6.2 For mass concrete hydraulic structures whose dynamic properties of concrete are not determined by test, the characteristic values of concrete dynamic strength may be determined in accordance with Table 5.6.2, and the corresponding partial factors for material properties may be taken as 1.5; the characteristic values of dynamic elastic modulus may be 150% of static ones; the characteristic value of dynamic tensile strength may be 10% of the characteristic value of dynamic compressive strength.

Table 5.6.2 Characteristic value of dynamic compressive strength of dam concrete (MPa)

Concrete strength grade	C5	C7.5	C10	C15	C20	C25	C30
Conventional concrete	–	9.1	11.8	17.2	22.2	26.9	31.4
Roller-compacted concrete	8.6	12.5	16.2	23.5	30.1	37.2	–

5.6.3 For seismic calculation of concrete hydraulic structures, dynamic deformation modulus of foundation rock mass may be taken as static deformation modulus. When the dynamic method is adopted to calculate seismic effect, the characteristic values of dynamic shear strength parameters of foundation rock mass as well as the interface between rock foundation and concrete may be taken as the static ones. When the pseudo-static method is adopted to calculate seismic effects, the characteristic values of dynamic shear strength parameters of foundation rock mass as well as the interface between rock foundation and concrete shall be the mean values of static shear strength parameters.

5.7 Seismic design for ultimate limit states with partial factors

5.7.1 The seismic strength and stability under the most unfavorable combinations considering both static and dynamic actions for various hydraulic structures shall satisfy the following formula for ultimate limit states:

$$\gamma_0 \psi S(\gamma_G G_k, \gamma_Q Q_k, \gamma_E E_k, a_k) \leq \frac{1}{\gamma_d} R\left(\frac{f_k}{\gamma_m}, a_k\right) \tag{5.7.1}$$

where γ_0—importance factor of structure, which shall be taken in accordance with GB 50199 *Unified Standard for Reliability Design of Hydraulic Engineering Structures*;

ψ—design situation factor, taken as 0.85;

$S(\cdot)$—action effect function of structure;

γ_G—partial factor for permanent action;

G_k—characteristic value of permanent action;

γ_Q—partial factor for variable action;

Q_k—characteristic value of variable action;

γ_E—partial factor for seismic action, taken as 1.0;

E_k—representative value of seismic action;

a_k—characteristic value of geometric parameter;

γ_d—structural factor for ultimate limit states;

$R(\cdot)$—resistance function of structure;

f_k—characteristic value of material property;

γ_m—partial factor for material property.

5.7.2 Limit states to be checked and corresponding structural factors for various hydraulic structures under seismic actions shall comply with the relevant requirements in this standard.

5.7.3 Partial factors and characteristic values of static actions combined with seismic action shall be assigned in accordance with corresponding design standards for various structures. When seismic effect reduction factor is introduced in seismic checking, partial factors shall be taken as 1.0.

5.7.4 In seismic design of reinforced concrete structural elements, the seismic checking of their sectional bearing capacity shall be conducted in accordance with the current professional standard SL 191 *Design Code for Hydraulic Concrete Structures* or DL/T 5057 *Design Specification for Hydraulic Concrete Structures* after the seismic effect determined by this standard. When the dynamic method is used to calculate seismic effects, the seismic effect reduction factor shall be taken as 0.35. When the pseudo-static method is used to calculate seismic effects of reinforced concrete structural elements, the seismic effect reduction factor shall be taken as 0.25 in seismic inertial force calculation.

5.7.5 The partial factors for dynamic properties of material may take the static ones.

5.8 Seismic calculation for appurtenant structures

5.8.1 In calculation of seismic effects for appurtenant structures of hydraulic structure, if the mass ratio λ_m and fundamental frequency ratio λ_f of appurtenant structures to main structure satisfy either of the following conditions, coupling analysis of appurtenant structures and main structure may not be performed:

1 $\lambda_m < 0.01$.

2 $0.01 \leqslant \lambda_m \leqslant 0.1$, and $\lambda_f \leqslant 0.8$ or $\lambda_f \geqslant 1.25$.

5.8.2 For an appurtenant structure without coupling analysis, the seismic input in seismic effect calculation may take the acceleration at the connection with the main structure.

5.8.3 For an appurtenant structure and main structure without coupling analysis, the mass of the appurtenant structure shall be considered as an added mass of the main structure if their connections may be regarded as rigid.

5.9 Seismic earth pressure

5.9.1 The representative value of seismic active earth pressure may be calculated by Formula(5.9.1-1) and shall take the greater value of the calculated results with plus or minus signs in Formula(5.9.1-1).

$$F_E=\left[q_0\frac{\cos\psi_1}{\cos(\psi_1-\psi_2)}H+\frac{1}{2}\gamma H^2\right](1\pm\xi a_v/g)C_e \tag{5.9.1-1}$$

$$C_e=\frac{\cos^2(\varphi-\theta_e-\psi_1)}{\cos\theta_e\cos^2\psi_1\cos(\delta+\psi_1+\theta_e)(1+\sqrt{Z})^2} \tag{5.9.1-2}$$

$$Z=\frac{\sin(\delta+\varphi)\sin(\varphi-\theta_e-\psi_2)}{\cos(\delta+\psi_1+\theta_e)\cos(\psi_2-\psi_1)} \tag{5.9.1-3}$$

where F_E—representative value of seismic active earth pressure;

q_0—load per unit length on earth surface;

ψ_1—included angle of retaining wall surface with vertical plane;

ψ_2—included angle of earth surface with horizontal plane;

H—height of earth;

γ—characteristic value of gravitaty density of earth;

φ—internal friction angle of earth;

θ_e—seismic coefficient angle, $\theta_e = \arctan\frac{\xi a_h}{g \pm \xi a_v}$, where a_v is the representative value of vertical design peak ground acceleration;

δ—friction angle between retaining wall surface and earth;

ξ—seismic effect reduction factor; when the dynamic method is used for calculating the seismic effect, it shall be taken as 1.00; when the pseudo-static method is used, it shall be taken as 0.25, and 0.35 for reinforced concrete structure.

5.9.2 The seismic passive earth pressure shall be determined through special study.

6 Embankment dam

6.1 Seismic calculation

6.1.1 The seismic calculation of embankment dams shall include seismic stability calculation, residual deformation calculation, safety evaluation of impervious body, discrimination of liquefaction, etc. Comprehensive evaluation of seismic safety shall be performed in combination with seismic measures.

6.1.2 For seismic stability calculation of embankment dams, the pseudo-static method shall generally be adopted to calculate seismic effect. The finite element method (FEM) shall also be adopted to analyze the seismic effect on dam body and foundation to comprehensively evaluate the seismic stability in one of the following conditions:

1 With a design intensity of Ⅶ and a dam height over 150m.

2 With a design intensity of Ⅷ or Ⅸ and a dam height over 70m.

3 With liquefiable soil layers in the dam foundation.

For embankment dam with an overburden thickness over 40m, dynamic analysis should be performed.

6.1.3 When the pseudo-static method is adopted to calculate seismic effects and seismic stability for embankment dams, the slip-circle method considering effect of inter-slice forces should be adopted in compliance with Article 5.7.1 in this standard, using the formulas given in Appendix A of this standard. The sliding wedge method may be adopted for foundations intercalated with thin soft clay layers, and dams with thin inclined clay core or thin clay core.

6.1.4 When the pseudo-static method is adopted to calculate seismic effects and seismic stability for embankment dams, the dynamic distribution coefficient α_i (Figure 6.1.4) of mass point i shall be taken in accordance with the following requirements:

1 The dynamic distribution coefficient at the dam base α is taken as 1.0.

2 The dynamic distribution coefficient at the dam crest α_m is taken as 3.0, 2.5 and 2.0 for design intensity of Ⅶ, Ⅷ and Ⅸ, respectively.

3 When the dam height H is not greater than 40m, the dynamic distribution coefficient of each elevation is obtained by linear interpolation of the dynamic distribution coefficients of dam crest and dam base.

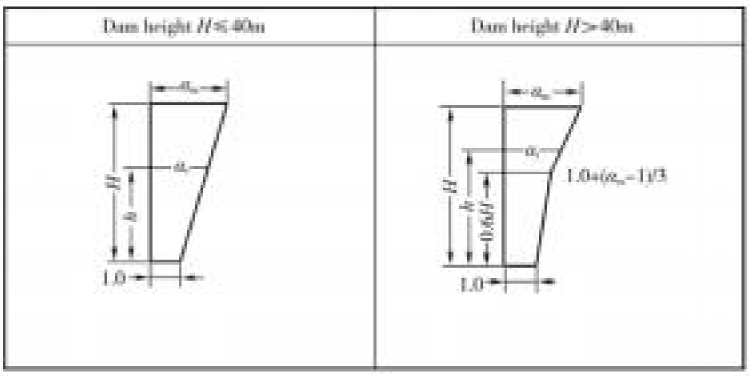


Figure 6.1.4 Sketch for dynamic distribution coefficient of seismic inertial force in embankment dams

4 When the dam height is greater than 40m, the dynamic distribution coefficient α_i is taken as $1.0+(\alpha_m-1)/3$ at $0.6H$, the dynamic distribution coefficient α_i at mass point above $0.6H$ is obtained by linear interpolation of the dynamic distribution coefficients at $0.6H$ and the dam crest, the dynamic distribution coefficient α_i at mass point below $0.6H$ is obtained by linear interpolation of the dynamic distribution coefficients at $0.6H$ and the dam base.

6.1.5 When the pseudo-static method is adopted to calculate the seismic effects and stability for Grade 1 and Grade 2 embankment dams, the dynamic shear strength of soil should be determined through dynamic tests. If the tested dynamic strength is higher than the corresponding static strength, the static strength shall apply.

When dynamic test data is unavailable for non-liquefiable soil like cohesive soil and dense sandy gravel, the static effective shear strength may be adopted; and for coarse cohesionless soil like rockfill and sandy gravels, the non-linear static shear strength considering confining pressure should be adopted.

6.1.6 Dynamic analysis of seismic effects on embankment dams with FEM should be conducted in accordance with the following requirements:

1 Calculate the initial stress state before earthquake with non-linear stress-strain relations of the materials.

2 Determine the characteristic parameters of dynamic deformation, dynamic residual deformation, and dynamic strength of dam materials by material dynamic tests, and should be in combination with engineering analogy.

3 Calculate seismic response with dynamic non-linear stress-strain relations of dam materials.

4 Analyze seismic stability along potential sliding plane based on seismic effects and calculate residual deformation of dam body caused by earthquake.

5 Evaluate the seismic safety comprehensively based on seismic response analysis in such aspects as slope stability, residual deformation, safety of impervious body, and liquefaction judgement, in accordance with Article 6.1.11 in this standard.

6.1.7 The material samples for dynamic tests shall be representative, and test conditions shall reflect compactness state and consolidated stress state of dam body and foundation soil. Dynamic characteristic parameters of dam materials, if conditions permit, should be determined through lab tests combined with in-situ tests.

6.1.8 For calculation residual deformation of dam, the influence of residual volumetric strain and residual shear strain should be taken into account.

6.1.9 The hydrodynamic pressure of CFRD may be determined in accordance with Article 7.1.12 to Article 7.1.14 in this standard.

6.1.10 When the slice method with inter-slice forces counted is adopted for seismic stability calculation, the structural factor shall not be smaller than 1.2. When the slice method without inter-slice forces counted is adopted, the structural factor shall not be smaller than 1.1.

6.1.11 Comprehensive seismic safety evaluation based on dynamic calculation results should be conducted in accordance with the following requirements:

1 Evaluate the seismic sliding stability of dam slope and its influence on overall seismic safety of dam comprehensively according to the location, depth, range of sliding circle, and duration and degree of over-limit in time history of the safety factor of seismic stability.

2 Determine the distribution range of local shear failure or liquefaction failure of dam body, and evaluate the possibility of inducing global failure of dam.

3 Provide the magnitude and distribution of residual deformation of dam body, and evaluate the seismic safety of dam and impervious body comprehensively according to the maximum seismic settlement ratio and the unevenness of deformation.

6.2 Seismic measures

6.2.1 For embankment dam in strong seismicity areas, straight axis or convex axis toward upstream should be adopted, and axis convex toward downstream or with broken or S-shape line should not be used.

6.2.2 When the design intensity is Ⅷ or Ⅸ, rockfill dam should be selected, but impervious body should not be of rigid core. When a homogeneous dam is selected, an internal drainage system shall be established to lower the phreatic line.

6.2.3 The freeboard of embankment dam in strong seismicity area shall accommodate the seismic surge and earthquake-induced settlement, and it shall be determined by the following principles:

1 The seismic surge height may be taken as 0.5m to 1.5m according to the design intensity and water depth in front of the dam.

2 When the design intensity is Ⅶ, Ⅷ or Ⅸ, the additional settlement of dam body and foundation due to seismic actions shall be incorporated into the freeboard.

3 The possible surge resulted from earthquake-induced massive collapse or landslide, etc., shall be specially studied.

6.2.4 When the design intensity is Ⅷ or Ⅸ, the dam crest should be widened and the upper dam slope should be gentler. The dam slope toe may be strengthened with blanket or counterweights, upper dam slope may be strengthened with masonry revetment, and interior of upper dam abutments may be strengthened with reinforcing bars, geosynthetics or concrete grids, etc.

6.2.5 The impervious body of embankment dam shall be strengthened, especially for such parts prone to cracking during earthquake as upper dam body and crest, interfaces between dam body and bank slopes or other concrete structures. The interface between impervious body and bank slopes or concrete structures should not be too steep. Slope angle should not be too large and reverse slope or sudden slope change shall be avoided. The impervious body, upstream and downstream filters, and transition zones shall be appropriately thickened.

6.2.6 Well-graded embankment material with favorable dynamic properties and seepage stability shall be adopted. Uniform medium sand, fine sand, silt and silty soil should not be used for embankment in strong seismicity areas.

6.2.7 Compaction performance and compactness of cohesive soil, and dry density or porosity of rockfill shall be determined in accordance with the current professional standards SL 274 *Design Code for Rolled Earth-rock Fill Dams* or DL/T 5395 *Design Specification for Rolled Earth-rock Fill Dams*, SL 228 *Design Code for Concrete Face Rockfill Dams* or DL/T 5016 *Design Specification for Concrete Face Rockfill Dams*. The upper limits stipulated should be taken when the design intensity is Ⅷ or Ⅸ.

6.2.8 For compaction of cohesionless soil, the relative density of material above phreatic line shall not be lower than 0.75, and that below phreatic line shall not be lower than 0.80. For sand-gravel material, if the content of coarse grain greater than 5mm is less than 50%, the relative density of fine materials shall meet the compaction requirements of cohesionless soil mentioned above, and according to relative density, the compact dry densities of sand-gravel materials with varying degrees of gravel content shall be used as filling criteria.

6.2.9 For Grade 1 and Grade 2 embankment dams, buried water-conduit should not be set under the dam. If unavoidable, reinforced concrete conduit or cast iron pipes should be used and placed in the trench in bedrock and the top of the pipe should not be higher than the bottom of dam with backfilling concrete outside the pipe, and the Anti-seepage and sealing at the joints of conduit shall be reliable, and the control valve of the conduit shall be installed in the intake or the upstream side of impervious body. Reliable filter shall be arranged at conduit outlet and joints.

6.2.10 The liquefiable strata or soft clay foundation of embankment dams in seismic zone should be treated according to Article 4.2.7 or Article 4.2.9 of this standard.

6.2.11 For CFRDs, the following seismic structural measures should be taken:

1 Thicken the cushion zone and strengthen its connection with foundation and bank slopes, increase the length of interface between cushion zone and bedrock for steep abutments, and adopt finer cushion material.

2 Fill the vertical joints of face slab at the middle of riverbed with asphalt coated board or other materials of desirable strength and flexibility.

3 Increase the reinforcement ratio for upper part of the face slab at the middle of riverbed, especially along slope direction.

4 Make the construction joints of face slab constructed in stages perpendicular to facing and provide double-layer reinforcement and stirrups in the vicinity of joints.

5 Use water-stop structures with good deformation performance and minimize their reduction to the face slab cross section.

6 Increase the compaction density of rockfill material and pay special attention to compaction quality at abrupt changes of terrain.

7 Arrange the internal drainage zone when filling the dam with soft rocks and sand-gravel materials to ensure smooth drainage, and fill a certain area downstream of dam slope with rockfill materials.

7 Gravity dam

7.1 Seismic calculation

7.1.1 For gravity dams, seismic analysis of stress of dam body and overall sliding stability along dam-foundation interface shall be carried out. For roller-compacted concrete(RCC) gravity dams, the seismic analysis of sliding stability along lift joints shall also be carried out.

7.1.2 The highest monolith of different dam sections may be selected for seismic analysis. For gravity dam with sound integrity, the analysis of whole dam should be carried out.

7.1.3 Under design seismic action, for strength safety analysis of gravity dams, FEM analysis shall be conducted as supplement to the dynamic and static analysis using the cantilever method. For gravity dams assigned to seismic fortification Class A, or with complex structure or complicated geological conditions, material non-linearity shall also be considered in FEM analysis. For seismic calculation of gravity dams under the action of MCE, FEM analysis shall be carried out considering non-linearity of dam body and foundation.

7.1.4 The analysis of overall sliding stability along dam-foundation interface for gravity dams and the analysis of sliding stability along lift joints for RCC gravity dams shall be carried out by the shear strength formula with cohesion for the rigid body limit equilibrium method. For deep-seated sliding stability, the rigid body limit equilibrium method based on the equal safety factor(equal-*K* method) shall be used. For gravity dams with complex foundation conditions, the non-linear FEM analysis should be carried out as a supplement.

7.1.5 The dynamic method or the pseudo-static method may be adopted for seismic calculation of gravity dams. For gravity dams assigned to seismic fortification Class A, or assigned to seismic fortification Class B and Class C with design intensity of Ⅷ or above, or with a dam height over 70m, the dynamic method shall be adopted.

7.1.6 Under design seismic action, the mode decomposition method shall be adopted for the dynamic analysis of gravity dam. For gravity dams assigned to seismic fortification Class A, a non-linear FEM analysis shall be added.

7.1.7 Under design seismic action, the dynamic method is adopted in checking the strength of gravity dams and sliding stability along dam-foundation interface and RCC lift joints. When the cantilever method or FEM with equivalent stress treatment is adopted, the structural factors for compressive strength and tensile strength shall not be less than 1.3 and 0.7 respectively. The structural factor for sliding stability along dam-foundation interface or RCC lift joints shall not be less than 0.65, or shall be further assessed by the time history analysis method.

7.1.8 When the dynamic method is used to check the deep-seated sliding stability of gravity dam, the parameters of rock mass shear strength shall take the static mean values, its partial factor shall be taken as 1.0, the structural factor for deep-seated sliding stability shall not be less than 1.4 or shall be assessed comprehensively by the time history analysis method.

7.1.9 The comprehensive evaluation of seismic sliding stability along gravity dam-foundation interface, RCC lift joints and deep-seated sliding plane shall be carried out by the time history analysis method according to the following steps:

1 In each time step, structural factor for sliding stability is calculated by the rigid body limit equilibrium method and a time history of structural factor is provided. The sliding stability shall be evaluated based on the minimum structural factor in the time history.

2 If the minimum value of structural factor in the time history cannot meet the requirements of Article 7.1.7 or Article 7.1.8 in this standard, the dam sliding stability should be evaluated comprehensively based on the duration and degree of limit exceedance.

7.1.10 In seismic safety demonstration for dams under the action of MCE, the finite element model of dam-foundation-reservoir system shall be established with parameters determined for analysis, with consideration of radiation damping effect of far-field foundation, material non-linearity of dam concrete and near-field rock mass, etc. For important gravity dams specified in Article 3.0.6 of this standard, dynamic model test should be performed for verification. A comprehensive evaluation based on calculation, model test results and engineering analogy shall be conducted to meet the seismic safety requirement on preventing uncontrolled release of reservoir.

7.1.11 When the pseudo-static method is used to calculate the seismic effect on gravity dams, the representative values of horizontal seismic inertial force of various mass points shall be calculated in accordance with the requirements in Article 5.5.9 of this standard, in which the dynamic distribution coefficient α_i shall be determined according to the following formula.

$$\alpha_i = 1.4 \times \frac{1+4(h_i/H)^4}{1+4\sum_{j=1}^{n}\frac{G_{Ej}}{G_E}(h_j/H)^4} \qquad (7.1.11)$$

where n—total number of calculation mass points of dam body;

H—dam height, and it shall be taken up to the top of pier for an overflow dam;

h_i, h_j—heights of mass points i and j, respectively;

G_{Ej}—characteristic value of gravity action concentrated on mass point j;

G_E—characteristic value of total gravity action of dam that produces seismic inertial force.

7.1.12 When the pseudo-static method is used to calculate the seismic effects on gravity dams, the representative value of seismic hydrodynamic pressure at water depth h shall be calculated according to following formula.

$$P_w(h) = a_h \xi \psi(h) \rho_w H_0 \qquad (7.1.12\text{-}1)$$

where $P_w(h)$—representative value of seismic hydrodynamic pressure on vertical waterward face of dam at water depth h;

$\psi(h)$—distribution coefficient of seismic hydrodynamic pressure at water depth h, which shall be taken in accordance with Table 7.1.12;

ρ_w—characteristic value of water mass density;

H_0—total water depth.

The representative value of the total seismic hydrodynamic pressure per unit width of dam face with its action point at water depth $0.54H_0$ shall be calculated according to the following formula.

$$F_0 = 0.65 a_h \xi \rho_w H_0^2 \qquad (7.1.12\text{-}2)$$

Table 7.1.12 Distribution coefficient of hydrodynamic pressure on gravity dam

h/H_0	$\varphi(h)$	h/H_0	$\varphi(h)$
0.0	0.00	0.6	0.76
0.1	0.43	0.7	0.75
0.2	0.58	0.8	0.71
0.3	0.68	0.9	0.68
0.4	0.74	1.0	0.67
0.5	0.76	-	-

7.1.13 The representative value of hydrodynamic pressure calculated by Formula (7.1.12-1) shall be multiplied by a reduction factor η_c for the dam with an angle θ between sloping upstream surface and horizontal plane:

$$\eta_c = \theta/90 \tag{7.1.13}$$

When there is a batter in the slope of the upstream surface of dam and the height of the vertical portion below the water surface is equal to or greater than half of the water depth, the upstream surface may be regarded approximately vertical. Otherwise, the line connecting the highest wetted point and the heel shall be regarded as the slope.

7.1.14 When the dynamic method is used, the added mass normal to the dam surface m_w corresponding to the horizontal seismic hydrodynamic pressure at water depth h shall be calculated according to the following formula.

$$m_w(h) = \frac{7}{8}\rho_w\sqrt{H_0 h} \tag{7.1.14}$$

7.1.15 When the pseudo-static method is used to check the strength of dam body, sliding stability along dam-foundation interface, RCC lift joints and deep-seated sliding plane, the structural factors for compressive strength and tensile strength shall not be less than 2.8 and 2.1 respectively, and the structural factor for sliding stability shall not be less than 2.7.

7.2 Seismic measures

7.2.1 A gravity dam should adopt straight axis rather than broken line axis in the layout.

7.2.2 A gravity dam shall be simple in shape, avoiding abrupt changes in dam slope, and the slope break near dam crest should be arc. The dam crest should not be excessively inclined toward upstream. For upper part of dam body, the weight should be reduced and the stiffness increased, the concrete strength grade shall be increased or reinforcement included appropriately.

7.2.3 Appurtenant structures on dam crest should be light, simple, well-integrated and minimized in height, and high tower structures should be avoided. The connections between the access bridge and the piers on overflow section as well as lateral stiffness of piers should be strengthened.

7.2.4 Weaknesses in foundation such as faults, fractures and weak intercalated layers shall be treated with engineering measures, and the strength grade of bottom concrete should be increased and clay blanket may be provided at dam heel.

7.2.5 For locations where the dam cross sections differ significantly along the dam axis or longitudinal topographic and geological conditions change abruptly, contraction joints shall be provided, and joint sealing and water-stop material with good flexibility should be selected.

7.2.6 For gravity dams assigned to seismic fortification Class A and design peak ground acceleration greater than 0.2g, contraction joints between dam monoliths should be provided with shear keys or grouting to improve dam integrity. Water-stop design for contraction joints shall be emphasized, and joint sealing and water-stop material with good flexibility should be selected.

7.2.7 Seismic vulnerable parts like orifice perimeter and connections between gate pier and weir surface of overflow dam shall be reinforced additionally.

8 Arch dam

8.1 Seismic calculation

8.1.1 Seismic calculation of arch dam shall include the analysis of dam body stress and arch abutment stability under design earthquake. Deformation analysis of dam-foundation system shall also be made additionally when calculation under MCE is required.

8.1.2 Under design seismic action, the stress analysis of arch dams shall be conducted by FEM in addition to the dynamic and static trial-load methods. For arch dams assigned to seismic fortification Class A, or with complex structure or complicated geological conditions, material non-linearity shall be considered in FEM analysis.

8.1.3 Seismic calculation of arch dam may be conducted with the dynamic method or the pseudo-static method. The dynamic method shall be used to calculate seismic effects on arch dams assigned to seismic fortification Class A, or assigned to seismic fortification Class B or Class C but with a design intensity of Ⅷ or above, or with a dam height over 70m.

8.1.4 Under design seismic action, the mode decomposition method shall be adopted in the dynamic analysis of arch dams. For arch dams assigned to seismic fortification Class A, non-linear FEM analysis shall be made as well.

8.1.5 The added mass normal to the dam surface may be taken as 1/2 of the value calculated by Formula (7.1.14) of this standard, where H_0 is the water depth of the calculated section. When the pseudo-static method is adopted for analysis, the horizontal seismic hydrodynamic pressure shall be determined by the added mass on the dam surface multiplied by the dynamic distribution coefficient α_i specified in Article 8.1.13 of this standard and the seismic effect reduction factor ξ.

8.1.6 When the dynamic method is used to check dam strength under design seismic action, the structural factors for compressive and tensile strengths shall not be less than 1.3 and 0.7, respectively.

8.1.7 The stability of arch abutments under design seismic actions shall be calculated by shear strength formula with cohesion of the rigid body limit equilibrium method. For arch dams assigned to seismic fortification Class A, the safety evaluation concerning overall stability of the arch dam-foundation system shall also be carried out in accordance with Article 8.1.11 of this standard.

8.1.8 Seismic stability calculation for arch abutments (including thrust block) under design seismic action may be conducted according to the following requirements and Article 8.1.9 and Article 8.1.10 of this standard. It may also be evaluated comprehensively by comparison of various methods.

1 After identifying the possible sliding rock blocks, the maximum value and direction of the thrust of arch abutments shall be determined by the most unfavorable combination of the results of dynamic and static calculation of dam body.

2 The representative value of seismic inertial force of possible sliding rock blocks shall be calculated by Formula (5.5.9) of this standard, where α_i is taken as 1.0. When the maximum thrust at arch abutments is determined by the dynamic method, the seismic effect reduction factor ξ of seismic inertial force of the rock shall be taken as 1.0, assuming that representative value of rock seismic inertial force and the maximum thrust at arch abutment occur simultaneously. For calculation of seismic inertial force of the rock block, the PGA in each direction shall be combined as follows:

1) When PGA in cross-stream direction is taken as the design value, the PGAs in stream direction and vertical direction are taken as 1/2 of the design value.

2) When PGA in stream direction is taken as the design value, the PGAs in cross-stream direction and vertical direction are taken as 1/2 of the design value.

3) When PGA in vertical direction is taken as the design value, the PGAs in stream direction and cross-stream direction are taken as 1/2 of the design value.

3 The most unfavorable time-independent sliding mode shall be selected based on the geometrical features of the potential sliding rock blocks.

4 The influence of earthquake on seepage pressure variation in rock mass may be ignored.

8.1.9 When the dynamic method is used to check the stability of rock mass of arch abutment under design seismic action, the shear strength parameters of the rock mass shall take the static mean values, the partial factor shall be taken as 1.0, and the structural factor for sliding stability shall not be less than 1.4; or the sliding stability of potential sliding rock mass in arch abutments shall be further assessed by time history analysis method.

8.1.10 The comprehensive evaluation of seismic stability of potential sliding rock mass in arch abutment by time history analysis method shall follow the procedures below:

1 Under the actions of three components of design ground motion, the resultant force time history of static and dynamic composite effect on the arch abutment is calculated by time history analysis method and is exerted on potential sliding rock blocks together with the time history of the inertial force of rock mass ignoring dynamic amplification.

2 In each time step, the structural factor for stability of abutment rock mass is calculated by the rigid limit equilibrium method and the time history of structural factor is provided. The abutment seismic stability shall be evaluated according to the minimum structural factor in time history.

3 If the minimum value of structural factor in time history does not meet the requirements in Article 8.1.9 of this standard, the abutment sliding stability and its effects on overall dam safety should be evaluated comprehensively based on the duration and degree of limit exceedance.

8.1.11 For arch dams assigned to seismic fortification Class A, non-linear numerical calculation shall be performed to analyze and evaluate the overall stability safety of the arch dam-foundation system under design seismic actions. For important arch dams designed to resist MCE, the safety evaluation concerning overall stability under MCE shall also be conducted. In the calculation and analysis, the influence of the contact non-linearity of contraction joints on dam body and the slip surface of critical sliding rock mass, material non-linearity of major weak zone in near-field foundation, and radiation damping effect of far-field foundation, shall be considered. For important arch dams specified in Article 3.0.6 of this standard, dynamic model test verification should be conducted. Comprehensive evaluation based on numerical analysis and model test results combining with engineering analogy shall be conducted to meet the seismic safety requirement on preventing uncontrolled release of reservoir.

8.1.12 When the seismic safety evaluation of arch dam is conducted according to Article 8.1.11 of this standard, turning point on curves of deformation at typical locations on dam body or on foundation rock mass which varies with the increase in seismic action may be used to evaluate the safety of dam foundation system, and the ratio of the input acceleration at the turning point to the design peak ground acceleration may be taken as the safety margin preventing uncontrolled release of reservoir.

8.1.13 When the seismic effects of arch dam are calculated by the pseudo-static method, the representative values of horizontal seismic inertial forces of various mass points on different arch rings acting in normal direction shall be calculated according to Article 5.5.9 of this standard, in which the dynamic distribution coefficients of dam crest and the lowest elevation of dam shall be take as 3.0 and 1.0, and respectively, and shall be interpolated along the elevation direction, distributed uniformly along arch rings.

8.1.14 When the strength of dam body and stability of arch abutments are checked with the pseudo-static method, the structural factors shall comply with Article 7.1.15 of this standard.

8.2 Seismic measures

8.2.1 The dam shape shall be selected reasonably to improve thrust direction of arch abutments and reduce tensile stress zones in upper and middle parts of dam and near the foundation under seismic action. For a double curvature arch dam, overhanging towards upstream side should be checked, and overhanging towards downstream should be increased for the upper arch crown.

8.2.2 The seismic stability of abutments shall be strengthened. Excessively large difference in rock properties and structure of rock mass between both abutments shall be avoided and dam abutments shall not sit on a thin mountain ridge. Weak zones in rock foundation may be strengthened by grouting, concrete plug, local anchorage, supporting, etc. The construction quality of the contact surface between top arch and abutments shall be controlled strictly, and measures such as thickening the arch base and anchoring by deep embedment may be taken near the top arch base. To minimize seepage pressure in rock mass, curtain and drainage measures shall be provided in dam foundation, and the pressure tunnel shall not be close to the dam abutment.

8.2.3 Details of the dam joints, in particular, waterstop, grouting temperature control and shear keys, shall be designed carefully. The shape and material of waterstops shall be able to accommodate repeated joint opening and closing during earthquake. If calculation results indicate that excessive contraction joint deformation under seismic action endangers the waterstop, dampers on crest and joint-crossing rebars at upper dam section should be considered.

8.2.4 In high tensile stress zone of arch dam faces, especially the middle part of downstream face, higher strength concrete and seismic reinforcement may be provided, and the dam crest with lighter weight and higher stiffness may be preferred. Clay blanket may be provided at upstream dam heel.

8.2.5 Light, simple and well-integrated appurtenant structures on the dam crest should be adopted, and the part extruding out of the dam body should be minimized. Arch thrust transferring structures should be provided between piers of the overflow section. Connections on the crest, such as access bridge shall be strengthened to prevent them from falling down during earthquake.

9 Sluice

9.1 Seismic calculation

9.1.1 The seismic calculation of sluice shall include the check of seismic stability and structural strength. Seismic stability calculation for sluice chamber and side connections and their foundation shall be carried out; the seismic stress calculation of structural components of sluice shall be carried out. For sluice structure, seismic design shall be conducted for non-structural components, appurtenant electromechanical equipment and their connections with the main structure.

9.1.2 The seismic effect of sluice may be calculated by the dynamic method or the pseudo-static method. The dynamic method shall be adopted for Grade 1 and Grade 2 sluices with design intensity of Ⅷ and Ⅸ or seated on liquefiable soil.

9.1.3 When the pseudo-static method is used to calculate the seismic effect on a sluice, the representative values of horizontal seismic inertial forces of various mass points shall be calculated in accordance with the requirements in Article 5.5.9 of this standard, in which the dynamic distribution coefficient α_i (Figure 9.1.3) of seismic inertial force shall be taken in accordance with the following requirements:

1 For seismic actions in vertical direction and stream direction, the dynamic distribution coefficient is taken as 1.0 at the bottom of the gate pier, and 2.0 at the top. For seismic action in cross-stream direction, the dynamic distribution coefficient is taken as 1.0 from the bottom of the gate pier to 1/2 of its height, and 3.0 at the top. The dynamic distribution coefficients from 1/2 height of the gate pier to the top are determined by linear interpolation.

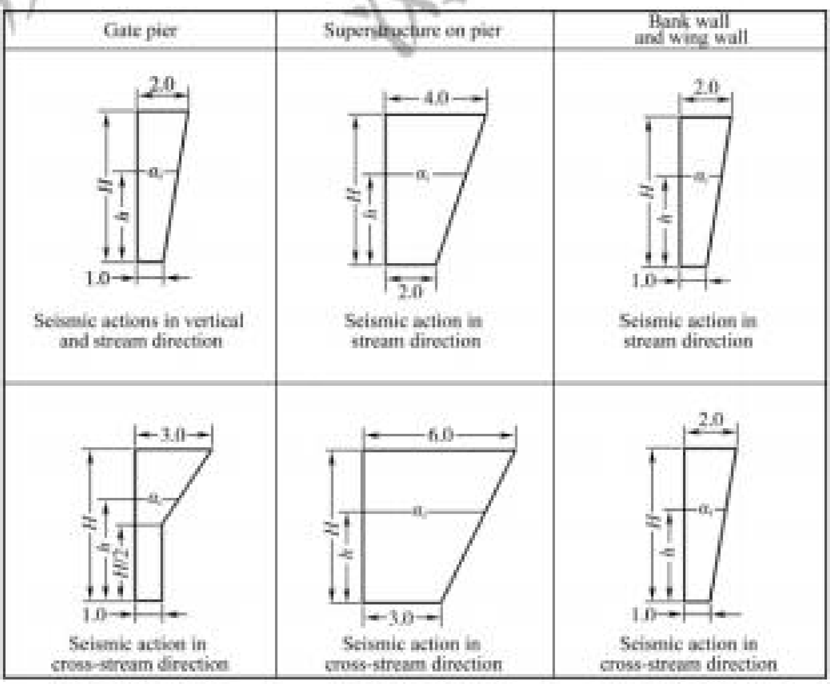


Figure 9.1.3 Sketch of dynamic distribution coefficient α_i of seismic inertial force acting on a sluice

2 For seismic action in stream direction, the dynamic distribution coefficient is taken as 2.0 at the bottom of superstructure on pier, and 4.0 at the top. For seismic action in cross-stream direction, the dynamic distribution coefficient is taken as 3.0 at the bottom of superstructure on pier, and 6.0 at the top. The dynamic distribution coefficients from the bottom to the top are determined by linear interpolation.

3 The dynamic distribution coefficient are taken are 1.0 at the bottom of the bank wall and wing wall, and 2.0 at the top. The dynamic distribution coefficients from the bottom to the top are determined by linear interpolation.

9.1.4 When the dynamic method is used for calculating the seismic effect of sluice, the gate chamber shall be considered as an integral three-dimensional structure.

9.1.5 The influence of radial gate rigidity on the seismic performance of sluice should be calculated, and the dynamic analysis of involved corbels shall be conducted.

9.1.6 In check of seismic strength for supports of access bridges and service bridges, the representative value of horizontal seismic inertial force E on simply-supported beam supports shall be calculated by the following formula and the representative value of horizontal seismic transverse inertial force shall be borne by supports on both sides:

$$E = 1.5 a_h G_{EL} / g \tag{9.1.6}$$

where G_{EL}—characteristic value of structural gravity action; for fixed supports, the dead weight of superstructure over one bridge span is taken; for non-fixed supports, 1/2 of the dead weight of superstructure over one bridge span is taken.

9.1.7 The representative value of seismic hydrodynamic pressure acting on a sluice may be calculated in accordance with Article 7.1.12 to Article 7.1.14 of this standard.

9.1.8 The representative values of seismic active earth pressures acting on side piers, bank walls or wing walls of sluice may be calculated in accordance with Article 5.9.1 of this standard.

9.1.9 The structural strength of sluice components shall be checked in accordance with Article 5.7.4 of this standard and shall also meet the requirements of the current professional standard SL 265 *Design Specifications for Sluices*. The influences of structural deformation of sluice component during earthquake on operation of gates and hoists shall be checked.

9.1.10 When checking the sliding stability along sluice base, the seismic effects shall be determined according to Article 9.1.3 or Article 9.1.4 in this standard and shall meet the requirements of the current professional standard SL 265 *Design Specifications for Sluices*. When the dynamic method is used to calculate sluice seismic stability, seismic effects consistent with that adopted for strength check shall be adopted.

9.1.11 For a sluice seated on bedrock, the sliding stability along its base plane or shallow foundation plane may be checked in accordance with Article 7.1.7 or Article 7.1.15 in this standard by the dynamic method or the pseudo-static method. When checking the sliding stability of sluice on soil foundation along its base plane or shallow foundation plane by the pseudo-static method, the structural factor shall not be less than 1.2.

9.2 Seismic measures

9.2.1 When pile foundation is adopted, the connection between pile foundation and sluice base slab and the seepage control measures shall be designed well. Anti-seepage wall, cutoff wall, end sill or other measures may be taken at base slab to prevent piping or concentrated seepage due to the separation of foundation and base slab by seismic action.

9.2.2 The layout of sluice chamber should be arranged and integrated well. The sluice chamber should adopt integrated reinforced concrete structure. Waterstops shall adopt structural type and material with durability and deformation adaptability. Waterstops at key joints shall be strengthened.

9.2.3 The height of the hoist frame bridges should be decreased through proper type selection and arrangement of gate and hoist to reduce the weight of rack top section.

9.2.4 The hoist frame should adopt frame structure and connection of column of the hoist frame, pier and deck should be strengthened by increasing section area and reinforcement in the connection positions. When prefabricated concrete beams and non-fixed supports are used for hoist frame, measures such as guard board, bolt connection or steel clip plate connection shall be adopted for beam support to prevent falling during earthquake. The stirrup spacing at upper and lower ends within 1/4 clear height of rack column shall be closer. In the case of design intensity of Ⅸ, the stirrup spacing at whole height range shall be closer.

9.2.5 The backfill slope against side pier should be lowered, and buildings or stacking loads on the bank adjacent to side pier should be avoided, to reduce the seismic deformation of river bank and deformation of sluice due to additional lateral loads during earthquake.

9.2.6 For Grade 1, Grade 2 and Grade 3 sluices, the upstream blanket should be made of concrete, which should be reinforced. The construction quality of waterstops of contraction joints, drainage of seepage downstream and on both banks of the sluice chamber shall be ensured.

10 Underground hydraulic structures

10.1 Seismic calculation

10.1.1 For underground structures with a design intensity of Ⅸ or Grade 1 underground structures with a design intensity of Ⅷ, the seismic safety of the structures and stability of their surrounding rocks shall be checked. For underground structures with a design intensity of Ⅶ and above, the seismic stability of rock mass at portals shall be checked. For Grade 1 underground structures in soil with a design intensity of Ⅶ and above, the seismic safety and the seismic subsidence of their foundation shall be checked.

10.1.2 In seismic calculation of underground structures, the maximum displacement of the site and its distribution in depth shall be obtained from site response analysis. The site is assumed to be a horizontal layered medium and analyzed by the one-dimensional wave propagation method, in which the non-linear soil model shall be employed. The maximum displacement of bedrock surface may also be calculated through the representative value of design peak ground acceleration and the predominant period of the site. The representative value of acceleration may be halved at 50m below the bedrock surface or lower, and may vary linearly within the depth of 50m.

10.1.3 The seismic effect of underground structures shall be calculated by the displacement response method or the acceleration response method. The numerical model shall involve the underground structure and the surrounding medium in a certain range.

10.1.4 For straight tunnel sections in rock, the representative values of axial stress σ_N, axial bending stress σ_M and shear stress σ_V, caused by seismic wave propagation, may be calculated according to the following formulas:

$$\sigma_N = \frac{a_h T_g E}{2\pi v_p} \tag{10.1.4-1}$$

$$\sigma_M = \frac{a_h r_0 E}{v_s^2} \tag{10.1.4-2}$$

$$\sigma_V = \frac{a_h T_g G}{2\pi v_s} \tag{10.1.4-3}$$

where v_p, v_s—characteristic values of compressive and shear wave velocity of surrounding rock mass, respectively;

E, G—characteristic values of dynamic elasticity modulus and shear modulus of tunnel structural material, respectively;

r_0—characteristic value of equivalent radius of tunnel section.

10.1.5 For straight tunnel sections in soil mass, the representative values of axial stress σ_N, axial bending stress σ_M and shear stress σ_V, caused by seismic wave propagation, may be calculated according to the following formulas:

$$\sigma_N = \max \left| \begin{array}{l} \beta_N \dfrac{ET_g}{2\pi v_p} a_h^t = \beta_N \dfrac{EV_h}{v_p} \\ \beta_N \dfrac{ET_g}{4\pi v_s} a_h^t = \beta_N \dfrac{EV_h}{2v_s} \end{array} \right. \tag{10.1.5-1}$$

$$\beta_N = \frac{1}{1 + \frac{EA}{K_a}\left(\frac{2\pi}{L}\right)^2} \tag{10.1.5-2}$$

$$\sigma_M = \beta_M \frac{E r_0}{v_s^2} a'_h \tag{10.1.5-3}$$

$$\sigma_V = \beta_M \frac{G T_g}{2\pi v_s} a'_h = \beta_M \frac{G V_h}{v_s} \tag{10.1.5-4}$$

$$\beta_M = \frac{1}{1 + \frac{EI}{K_t}\left(\frac{2\pi}{L}\right)^4} \tag{10.1.5-5}$$

where a'_h—maximum value of horizontal acceleration response at the surrounding mass of tunnel;

V_h—maximum value of horizontal velocity response at the surrounding mass of tunnel;

β_N, β_M—reduction factors for axial stress σ_N and axial bending stress σ_M, respectively;

EA, EI—characteristic values of axial stiffness and axial bending stiffness of tunnel structure, respectively;

K_a, K_t—characteristic values of longitudinal and transverse stiffness coefficient of unit length of tunnel surrounding mass, respectively;

L—characteristic value of seismic apparent wavelength.

10.1.6 The seismic effect of underground structures with complicated topographical and geological conditions, such as underground powerhouse, tunnel and other deep-buried underground caverns, or shallow-buried caverns such as intake and outlet on river bank, should be analyzed with three-dimensional structural models, in which the dynamic interaction between the structure and surrounding mass is taken into consideration.

10.2 Seismic measures

10.2.1 Underground structures should be kept away from active faults, shallow and thin ridge, and should not be close to hillside or unstable areas. Potentially liquefiable soil foundation should be avoided. The alignment with a greater buried depth is preferred and should be away from weathered rock mass.

10.2.2 The turning radius of tunnel and the intersection angle of two tunnels should not be too small.

10.2.3 Concealed excavation should be taken when construction conditions permit.

10.2.4 The portals of underground structure should be arranged at a location where topographical and geological conditions are favorable. When complex geological conditions are unavoidable, measures such as gentle portal slope, shotcrete and bolting or lining protection, and entrance extension outward should be taken. The portal structures shall be of reinforced concrete.

10.2.5 Consolidation grouting should be conducted well to strengthen the combined effect of lining and surrounding rocks.

10.2.6 Seismic joints should be set at turning, bifurcation and transition sections with abrupt change in structure or properties of surrounding mass. The number, spacing and structural form of seismic joints shall meet the requirements of structural deformation and water sealing.

10.2.7 Elastic joints should be used at the joints between central columns and beams or top plates in underground structures, and rigid joints should not be used.

11 Intake tower

11.1 Seismic calculation

11.1.1 The seismic calculation of intake tower shall include the check of stresses or internal forces, overall sliding stability and overturning stability, and foundation bearing capacity. Seismic design shall be conducted for non-structural components, appurtenant electromechanical equipment and their connections with main structure in intake tower.

11.1.2 The seismic effect calculation of intake tower shall be conducted by the dynamic method or the pseudo-static method. The dynamic method should be adopted to calculate the seismic effect of intake towers assigned to seismic fortification Class A or with design intensity of Ⅷ and above or unreinforced concrete intake towers higher than 40m or reinforced intake towers not higher than 40m.

11.1.3 The influence of water body inside and outside of intake tower as well as foundation shall be considered for dynamic analysis of seismic effect of intake tower. The mode decomposition method should be adopted.

11.1.4 The seismic effects of intake tower may be calculated by the cantilever method or FEM. The seismic calculation model shall be the same as that adopted in the analysis of static load combination.

11.1.5 When the pseudo-static method is adopted to calculate seismic effect of intake tower, the representative value of horizontal inertial force of each mass point shall be calculated in accordance with the requirements in Article 5.5.9 of this standard, where G_{Ei} is the representative value of gravity actions concentrated on mass point i of tower body, frame and appurtenant equipment; and the dynamic distribution coefficient α_i(Figure 11.1.5.)of seismic inertial forces shall be taken in accordance with the following requirements:

1 The dynamic distribution coefficient of intake tower is taken as 1.0 from the tower bottom to 1/2 of its height.

2 When the height of intake tower is 10m to 30m, the dynamic distribution coefficient at the tower top is taken as 3.0; when the tower is higher than 30m, the dynamic distribution coefficient at the tower top is taken as 2.0; the dynamic distribution coefficients from 1/2 of the tower height to the top are determined by linear interpolation.

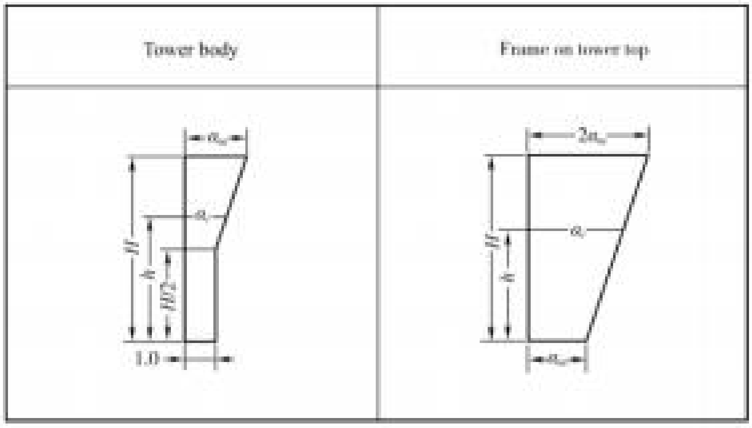


Figure 11.1.5 Sketch of dynamic distribution coefficient α_i of seismic inertial force of intake tower

3 When the height of intake tower is 10m to 30m, the dynamic distribution coefficient is taken as 3.0 at the bottom of frame on tower top and 6.0 at the top; when the tower is higher than 30m, the dynamic distribution coefficient is 2.0 at the bottom of frame on tower top and 4.0 at the top; the dynamic distribution coefficients of frame on tower top are determined by linear interpolation along the height.

11.1.6 When the dynamic method is adopted to calculate seismic effect of intake tower, the hydrodynamic pressure inside and outside of tower may be considered respectively as the added mass of the interior and exterior surfaces of the tower, which is calculated according to the following formula:

$$m_w(h)=\psi_m(h)\rho_w\eta_w A\left(\frac{a}{2H_0}\right)^{-0.2} \tag{11.1.6}$$

where $m_w(h)$—representative value of added mass of hydrodynamic pressure per unit height at water depth h;

$\psi_m(h)$—distribution coefficient of added mass, taken as 0.72 for the hydrodynamic pressure inside tower, and shall be taken according to Table 11.1.6-1 for the hydrodynamic pressure outside tower;

η_w—shape coefficient, taken as 1.0 for inside of the tower and outside of the circular tower and shall be taken according to Table 11.1.6-2 for outside of the rectangular tower;

A—area enclosed by the section-water contact line, where the section is the average section of the tower along height;

a—mean value of the maximum width of structure-water contact surface along the tower height perpendicular to the direction of seismic action.

Table 11.1.6-1 Distribution coefficient of added mass $\psi_m(h)$

h/H_0	$\psi_m(h)$	h/H_0	$\psi_m(h)$
0.0	0.00	0.6	0.59
0.1	0.33	0.7	0.59
0.2	0.44	0.8	0.60
0.3	0.51	0.9	0.60
0.4	0.54	1.0	0.60
0.5	0.57	-	-

Table 11.1.6-2 Shape coefficient η_w for outside of the rectangular tower

a/b	η_w	a/b	η_w
1/5	0.28	3/2	1.66
1/4	0.34	2	2.14
1/3	0.43	3	3.04
1/2	0.61	4	3.90
2/3	0.81	5	4.75
1	1.15	-	-

Note: b is the width of tower along the direction of seismic action.

11.1.7 When the pseudo-static method is adopted to calculate seismic effect of intake tower, the representative value of hydrodynamic pressure may be directly calculated according to the following formula:

$$F_T(h)=a_h\xi\rho_w\psi(h)\eta_w A\left(\frac{a}{2H_0}\right)^{-0.2} \tag{11.1.7-1}$$

where $F_T(h)$—representative value of the resultant force of hydrodynamic pressure on tower surface per unit height at water depth h;

$\psi(h)$—distribution coefficient of hydrodynamic pressure at water depth h, taken as 0.72 for hydrodynamic pressure inside the tower, and taken according to Table 11.1.7 for hydrodynamic pressure outside the tower.

The representative value of resultant force of hydrodynamic pressure exerting on whole tower surface may be calculated according to the following formula with its action point at water depth $0.42H_0$:

$$F_T=0.5a_h\xi\rho_w\eta_w AH_0\left(\frac{a}{2H_0}\right)^{-0.2} \tag{11.1.7-2}$$

Table 11.1.7 Distribution coefficient $\psi(h)$ of hydrodynamic pressure of intake tower

h/H_0	$\psi(h)$	h/H_0	$\psi(h)$
0.0	0.00	0.6	0.48
0.1	0.68	0.7	0.37
0.2	0.82	0.8	0.28
0.3	0.79	0.9	0.20
0.4	0.70	1.0	0.17
0.5	0.60	-	-

11.1.8 When the water depth is different in front of and behind the tower, the representative value of hydrodynamic pressure or the added mass at a certain elevation may be calculated respectively in accordance with in front of and behind water depths and then the mean value can be taken.

11.1.9 For tower group connected in a row, when ratio of average width of structure-water contact surface perpendicular to the seismic action direction to the maximum water depth in front of tower a/H_0 is greater than 3.0, the resultant force for the pseudo-static method and added mass for the dynamic method, which are used in calculation of the hydrodynamic pressure per unit height at water depth h outside of the tower, may be calculated according to Formula(11.1.9-1)and Formula(11.1.9-2), respectively:

$$F_T(h)=1.75a_h\xi\rho_w a\sqrt{H_0 h} \tag{11.1.9-1}$$

$$m_w(h)=1.75\rho_w a\sqrt{H_0 h} \tag{11.1.9-2}$$

11.1.10 Distribution of representative values of hydrodynamic pressure and the added mass on horizontal section may be taken uniformly on both structure-water contact surfaces perpendicular to the seismic action direction for rectangular cross section tower; and may be taken as $\cos\theta_i$ for circular-section tower, where θ_i is the acute angle intersected by the normal direction of point i on structure-water contact surface and the direction of seismic action. The maximum distribution strength of the hydrodynamic pressure and added mass may be calculated according to the following formulas, respectively:

$$F_0(h)=\frac{2}{\pi a}F_T(h) \tag{11.1.10-1}$$

$$m_0(h)=\frac{2}{\pi a}m_w(h) \tag{11.1.10-2}$$

where $F_i(h)$, $m_i(h)$—maximum distribution strength of hydrodynamic pressure and added mass on horizontal section at water depth h, $F_i(h)$ acting on both structure-water contact surfaces shall be in the same direction.

11.1.11 When checking the sliding stability, overturning stability and foundation bearing capacity of tower under seismic action, seismic effect by the dynamic method shall be multiplied by the seismic effect reduction factor.

The seismic sectional bearing capacity of reinforced concrete intake tower shall be checked according to Article 5.7.4 of this standard. For seismic check of sliding stability, overturning stability and foundation bearing capacity of tower, the seismic effect shall be in consistent with the check of seismic strength.

11.1.12 Under seismic action, the partial factor for rock property of tower foundation may be taken as the static one, whereas the characteristic value of dynamic bearing capacity may be taken as 1.5 times the static one.

11.1.13 The sliding stability of intake tower shall be checked by the shear strength formula with cohesion.

11.1.14 When checking the foundation bearing capacity of intake tower, the vertical normal stress acting on the foundation surface shall be calculated by the cantilever method.

11.1.15 In seismic check of intake tower, the structural factor for sliding stability shall not be smaller than 2.70. In this case, the parameters of shear strength shall be taken the static mean values. The structural factor for overturning stability shall not be smaller than 1.40. The structural factor for foundation bearing capacities of average vertical normal stress and maximum vertical normal stress on foundation plane shall not be smaller than 1.20 and 1.00, respectively.

11.2 Seismic measures

11.2.1 For intake tower with high water head and large flow, the earthquake-resistant box and barrel structure with high rigidity, strong capacity against overturning, large bearing capacity and good integrity should be selected. For frame structure, the strength and rigidity of connecting points and bracing members shall be strengthened to ensure the integrity and torsional rigidity.

11.2.2 On the prerequisite of meeting operational requirements, the intake tower structure shall be simple and symmetric, with gentle variation of mass and stiffness, low stress concentration, and sufficient lateral stiffness. Lateral support shall be provided along the tower height, and the support stiffness should be strengthened at abrupt change section.

11.2.3 The tower should be built on the rock foundation with sufficient bearing capacity, buried depth, and consolidation grouting.

11.2.4 For river-side intake tower, the gaps between the tower and excavated rock mass should be backfilled.

11.2.5 The weight of hoist room on the top of tower shall be minimized. For seismic vulnerable parts such as pier and the connection between tower and access bridge, such measures as increasing connection area between bridge and the top of tower, flexible connection, and falling prevention of access bridge of hoist during earthquake shall be taken, and the seismic resistance of bridge pier shall be strengthened.

11.2.6 The intake tower group should be lined up and connected to each other so as to increase the lateral rigidity.

11.2.7 Emergency gates must be equipped for Grade 1 and Grade 2 intake towers. The inlet gate slot shall be provided with baffle plates which do not affect ventilation, to prevent debris from falling into the gate slot and hindering the opening and closing of the gate during earthquake.

11.2.8 The seismic measures for concrete intake tower in details, materials and reinforcements shall comply with the current professional standard SL 191 *Design Code for Hydraulic Concrete Structures* or DL/T 5057 *Design Specification for Hydraulic Concrete Structures*.

12 Penstock and surface powerhouse of hydropower station

12.1 Penstock

12.1.1 The seismic effects of exposed penstock may be calculated by the pseudo-static method. The representative value of horizontal seismic inertial force of each mass point may be calculated by Formula (5.5.9) of this standard, where G_{Ei} is the characteristic value of gravity actions including water in the penstock, concentrated on mass point i. The dynamic distribution coefficient α_i (Figure 12.1.1) shall be taken in accordance with the following requirements.

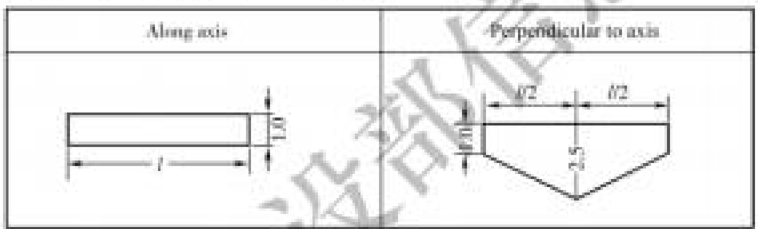


Figure 12.1.1 Sketch of dynamic distribution coefficient α_i of seismic inertial force of penstock

1 For seismic action along axis, the dynamic distribution coefficient of seismic inertial force at each point of the penstock is taken as 1.0.

2 For seismic action perpendicular to axis, the dynamic distribution coefficient of the seismic inertial force is taken as 1.0 at both ends of the penstock and 2.5 at the middle of the penstock. The dynamic distribution coefficient of the seismic inertial force at each point from the middle to the ends of the penstock is determined by linear interpolation.

12.1.2 The strength and stability of penstock under seismic action may be checked in accordance with the current professional standard SL 281 *Design Specification for Steel Penstocks of Hydroelectric Stations* or NB/T 35056 *Design Code for Steel Penstocks of Hydroelectric Stations*.

12.1.3 The seismic check may not be required for the penstock embedded in gravity dam.

12.1.4 The penstock shall be arranged on sound rock foundation without abrupt change in slope, and kept away from cliff surface, depression, potential avalanche and landslide. The alignment of penstock should conform to topographic slope direction. The powerhouse shall be prevented from inundation due to penstock damage during earthquake.

12.1.5 Anchor blocks of exposed penstock shall be placed on bedrock. Foundation treatment shall be done for supporting piers on soil. The span between the supporting piers should be shortened. The cross section and anchor bars should be appropriately increased. The reinforcement should be increased in stress concentration zones.

12.1.6 The flexibility of connecting structures of penstock should be increased, and penstock falling down from supporting piers shall be prevented during earthquakes.

12.1.7 Joints and connecting structures at the outlet of penstock embedded in gravity dam shall have a good earthquake resistance.

12.2 Surface powerhouse

12.2.1 The principle and method for seismic calculation of substructure for powerhouse shall meet the relevant requirements of Section 7.1 of this standard.

12.2.2 The overall sliding stability under design seismic action may be calculated by the shear strength formula with cohesion or shear strength formula without cohesion. The structural factor for overall sliding stability of powerhouse shall not be less than the specified values in Table 12.2.2.

Table 12.2.2 Structural factor for overall sliding stability of powerhouse

Foundation type	Calculation method for seismic effect	Calculation method for sliding stability	Structural factor
Rock foundation	Dynamic method	Shear strength formula with cohesion	0.65
	Pseudo-static method	Shear strength formula with cohesion	2.70
		Shear strength formula without cohesion	1.20
Non-rock foundation	Pseudo-static method	Shear strength formula without cohesion	1.25

12.2.3 The vertical normal stress at the powerhouse foundation surface under design seismic action shall be calculated by the cantilever method. When calculating a powerhouse on rock foundation using the pseudo-static method, the vertical normal stress on the surface of powerhouse foundation shall comply with the following provisions:

1 The maximum vertical normal stress shall not exceed the allowable bearing capacity of the foundation, and the characteristic value of bedrock dynamic bearing capacity may be taken as 1.50 times the static one.

2 The minimum vertical normal stress, including uplift pressure, of the foundation surface of the riverbed powerhouse shall not be greater than 0.1MPa.

3 When the minimum vertical normal stress, including uplift pressure, of the foundation surface of powerhouse at dam toe or river-side powerhouse is greater than 0.2MPa, special study shall be carried out.

12.2.4 Under design seismic action, the foundation stress at the foundation surface of powerhouse seated on non-rock foundation calculated by the pseudo-static method shall meet the requirements of the current professional standard SL 266 *Design Code for Hydropower House* or NB 35011 *Design Code for Powerhouses of Hydropower Stations*.

12.2.5 The sectional bearing capacity of the superstructure of powerhouse shall be checked according to Section 5.7 of this standard, and the acceleration at the top of substructure shall be taken as the seismic input of powerhouse superstructure.

12.2.6 Joint type and waterstop for the submerged portion of powerhouse shall meet the requirements for deformation induced by earthquake, and the waterstop material and type with good seismic resistance should be adopted.

12.2.7 Seismic measures for the superstructure of powerhouse shall comply with the current professional standard SL 191 *Desing Code for Hydraulic Concrete Structures* or DL/T 5057 *Design Specification for Hydraulic Concrete Structures*, and the current national standard GB 50011 *Code for Seismic Design of Buildings*.

12.2.8 The reinforcement should be enhanced at the joint between the air duct of the machine hall and turbine pier of the powerhouse.

12.2.9 The river-side powerhouse should be seated on the foundation of stable bank slope and favorable geological conditions, and the slope against powerhouse should be away from high, steep and dangerous cliff or potentially unstable slopes. The rock slope against powerhouse shall be excavated into stable slope, and shotcrete and rock bolt support should be applied. Protective measures shall be taken at the slope side of powerhouse.

13 Aqueduct

13.1 Seismic calculation

13.1.1 For aqueducts with design intensity of Ⅶ and above, the seismic actions in longitudinal, transversal and vertical directions shall be taken into account simultaneously.

13.1.2 For Grade 1 aqueduct, the dynamic method shall be used for seismic calculation with a three-dimensional model integrating effects of adjacent structures and boundary conditions. For Grade 2 aqueduct, the dynamic method may be used for seismic calculation of pier and upper aqueduct body modeled as a cantilever and a simply supported beam, respectively. For aqueduct of Grade 3 and below, the pseudo-static method may be used for seismic calculation of pier and aqueduct body respectively in accordance with Article 5.5.9 of this standard, where the dynamic distribution coefficient α_i of seismic inertial force of piers may refer to Article 9.1.3 of this standard; the dynamic distribution coefficient α_i of seismic inertial force of aqueduct body may be calculated according to Article 12.1.1 of this standard concerning penstocks of hydropower station.

13.1.3 When pile foundation is adopted, the effect of pile-soil interaction shall be considered, which may be modeled by equivalent springs of soil. According to the requirements in the current professional standard JGJ 94 *Technical Code for Building Pile Foundations*, the soil is treated as elastic medium, with its horizontal resistance coefficient varying linearly with depth (the *m*-method).

13.1.4 In calculation of Grade 1 or Grade 2 aqueduct, the hydrodynamic pressure within the aqueduct shall be considered, and may be calculated by the formulas specified in Appendix B in this standard.

13.1.5 The dynamic analysis of aqueduct may be conducted by the mode decomposition response spectrum method. For Grade 1 aqueduct, the time history analysis method shall be used according to Article 5.5.8 of this standard.

13.1.6 When there is significant difference of geological conditions or abrupt change of topographic features along the aqueduct, the spatial variation effect of seismic ground motion should be studied.

13.1.7 When the dynamic method is adopted in checking the cross section bearing capacity of pre-stressed reinforced concrete aqueduct body, the seismic effect reduction factor should be taken as 1.0.

13.1.8 The hydrodynamic pressure acting on the aqueduct pier in river channel may be calculated according to the current national standard GB 50111 *Code for Seismic Design of Railway Engineering*.

13.2 Seismic measures

13.2.1 For aqueducts with design intensity of Ⅶ and above, shock absorption or isolation devices such as lead-core rubber bearing, spherical damping bearing or pot bearing meeting the requirements for bearing capacity should be installed between the aqueduct and pier.

13.2.2 For aqueduct with shock absorption and isolation devices, the resonance of aqueduct structure during earthquake shall be considered for the lower supporting structure with low rigidity and on soft soil foundation.

13.2.3 Guard blocks shall be installed on top of pier to prevent aqueduct body from lateral falling. The ends of aqueduct shall have sufficient overlapping length on the pier abutment so as to prevent aqueduct from longitudinal falling.

13.2.4 The connection point between aqueduct body end and bearing as well as the top of pile foundation shall be strengthened by reinforcement.

13.2.5 The waterstop type and material meeting the seismic requirements shall be selected for the joint between adjacent aqueduct sections.

14 Shiplift

14.1 Seismic calculation

14.1.1 The seismic calculation of shiplift tower shall include the check of deformation, strength, and overall sliding stability and overturning stability.

14.1.2 For shiplift with design intensity of Ⅷ and above, the effect of vertical seismic action shall be considered in seismic calculation.

14.1.3 For structures with non-uniform or unsymmetrical mass or rigidity distribution, torsion effect under horizontal seismic action shall be studied.

14.1.4 For tower structure not higher than 30m, the pseudo-static method may be adopted for seismic calculation, and the dynamic distribution coefficient of seismic inertial force may be taken in accordance with Article 11.1.5 of this standard.

14.1.5 For tower structure higher than 30m, the mode decomposition response spectrum method shall be used to calculate seismic effect. The time history analysis method should be adopted for Grade 1 shiplift tower.

14.1.6 For rack and pinion vertical shiplift, dynamic interaction between ship chamber and tower structures and dynamic fluid-solid interaction between ship chamber and water shall be considered. The value of hydrodynamic pressure on steel ship chamber may be determined according to Article 13.1.4 of this standard.

14.1.7 For dynamic analysis of tower column, the connection shall be modeled by springs with the same stiffness of guide wheel and rail, and dynamic coupling analysis shall be conducted if the structure is connected with counterweight. In simplified analysis, 30% of counterweight mass may be added to tower column to simulate the interaction between counterweight and tower.

14.1.8 Seismic design shall be conducted for non-structural components, appurtenant electromechanical equipment and their connections with main structure of shiplift.

14.2 Seismic measures

14.2.1 For shiplift tower, the earthquake-resistant box and barrel structure with high rigidity, strong stability against overturning, large bearing capacity and good integrity should be selected. Seismic joints should be provided between tower units seated on different types of foundation.

14.2.2 The tower should be regular and symmetrical in shape. The mass, stiffness, and bearing capacity of lateral-force resisting members of same type should be uniformly distributed. The eccentricity between stiffness center and centroid should be minimized. Abrupt change of stiffness in adjacent layers and bearing capacity of lateral-force resisting structures should be avoided.

14.2.3 For structural design of shiplift, the load transmission path under seismic action shall be clear and simple; members and their joints on the transmission path shall not allow brittle failure, and the failure of partial structures or members shall not cause bearing stability failure of the whole structure system.

14.2.4 For rack and pinion vertical shiplifts, damping devices should be provided at the guide mechanism coupling ship chamber and tower.

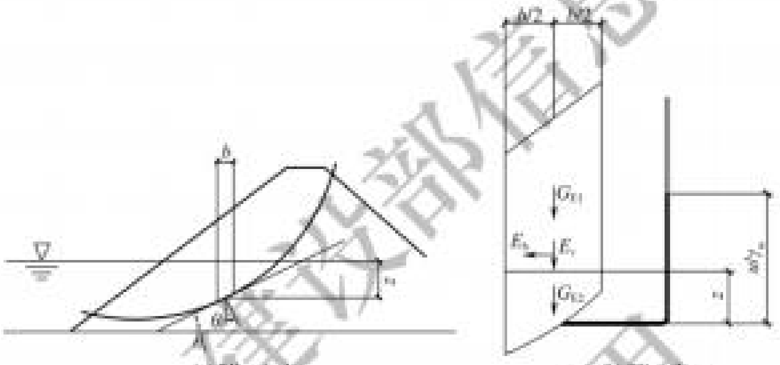
14.2.5 The non-structural members of floor, roof and non-load-bearing walls at the stairway shall be reliably connected with the main structure to prevent human injury and important equipment damage associated with falling hazards in the event of an earthquake.

14.2.6 The supports and connecting parts of electromechanical equipment mounted on structures shall comply with the current professional standard SL 660 *Design Code for Shiplift*.

14.2.7 For shiplift of concrete tower column structure, the seismic measures in details, materials and reinforcements shall comply with the current professional standard SL 191 *Design Code for Hydraulic Concrete Structures* or DL/T 5057 *Design Specification for Hydraulic Concrete Structures*.

Appendix A Seismic stability calculation of embankment dams by pseudo-static method

A.0.1 When the slip circle method with inter-slice forces counted, derived from the simplified Bishop method, is adopted, the representative value of action effect S and the characteristic value of resistance R for seismic stability of dam slope may be determined by Formulas (A.0.1-1) and (A.0.1-2) as shown in Figure A.0.1.



(a) Slip circle (b) Slice forces

Figure A.0.1 Sketch of slip circle and slice forces

$$S=\sum\left[(G_{E1}+G_{E2}\pm E_v)\sin\theta_i+M_h/r\right] \tag{A.0.1-1}$$

$$R=\sum\left\{\left[(G_{E1}+G_{E2}\pm E_v)\sec\theta_i-(u-\gamma_w z)b\sec\theta_i\right]\frac{\tan\varphi}{\gamma_f}+\frac{c}{\gamma_c}b\sec\theta_i\right\}\left[1/(1+\tan\theta_i\tan\varphi/\gamma_R)\right] \tag{A.0.1-2}$$

$$\gamma_R=\frac{\gamma_0\psi\gamma_d(1+\rho_c)}{\dfrac{1}{\gamma_f}+\dfrac{1}{\gamma_c}\rho_c} \tag{A.0.1-3}$$

$$\rho_c=\frac{cb\sec\theta_i}{\left[(G_{E1}+G_{E2}\pm E_v)\sec\theta_i-ub\sec\theta_i\right]\tan\varphi} \tag{A.0.1-4}$$

where G_{E1}—characteristic value of actual weight of the slice part above corresponding water level in a slice;

G_{E2}—characteristic value of buoyant weight of the slice part below corresponding water level in a slice;

E_h—representative value of horizontal seismic inertial force acting on the centroid of a slice, i.e., characteristic value of actual weight of the slice multiplied by $a_h\xi\alpha_i/g$ at the centroid, where a_h is the representative value of horizontal design peak ground acceleration, ξ is the seismic effect reduction factor, commonly taken as 0.25, α_i is the dynamic distribution coefficient of mass point i, and g is the gravity acceleration, taken as 9.81m/s^2;

E_v—representative value of vertical seismic inertial force acting on centroid of a slice, i.e., characteristic value of actual weight of the slice multiplied by $a_h\xi\alpha_i/3g$ at the centroid, in the acting direction of upward (−) or downward (+) whichever is unfavorable to stability;

M_h—moment of E_h to the circle center;

r —radius of the slip circle;

θ_i—included angle between slip circle radius through midpoint of slice bottom and vertical line going through radius of slip circle, if the radius deviates from the vertical line to the dam axis, plus sign is taken, otherwise, minus sign is taken;

b—slice width;

u—representative value of pore water pressure at midpoint of slice bottom;

z—vertical distance from corresponding water level to midpoint of slice bottom;

γ_w—unit weight of water;

c, φ—internal cohesion and friction angle of soil under seismic action, respectively;

γ_0—importance factor of structure, taken according to the current national standard GB 50199 *Unified Standard for Reliability Design of Hydraulic Engineering Structures*;

ψ—design situation factor, taken as 0.85 according to Article 5.7.1 of this standard;

γ_E—partial factor for seismic action, taken as 1.0 according to Article 5.7.1 of this standard;

γ_c, γ_f—partial factors for material properties of soil shear strength. $\gamma_c = 1.2$, $\gamma_f = 1.05$; the partial factor for material properties of non-linear friction angle of coarse-grained materials like rockfill and gravels may take $\gamma_f = 1.10$;

ρ_c—ratio of cohesion to friction of soil slices;

γ_d—structural factor.

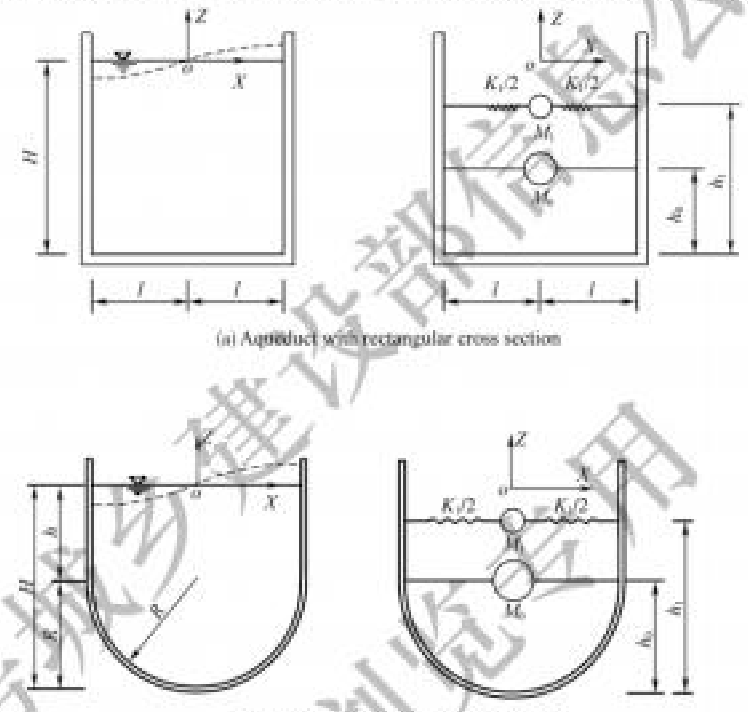
A.0.2 When the slip circle method without inter-slice forces counted, derived from the Swedish method, is adopted, the representative value of action effect S and the characteristic value of resistance R for seismic stability of dam slope may be determined by Formulas (A.0.2-1) and (A.0.2-2).

$$S = \sum\left[(G_{E1} + G_{E2} \pm E_v)\sin\theta_i + M_h/r\right] \tag{A.0.2-1}$$

$$R = \sum\left\{\left[(G_{E1} + G_{E2} \pm E_v)\cos\theta_i - (u - \gamma_w z) b\sec\theta_i - E_h\sin\theta_i\right]\frac{\tan\varphi}{\gamma_f} + \frac{c}{\gamma_c} b\sec\theta_i\right\} \tag{A.0.2-2}$$

Annex B Calculation of hydrodynamic pressure in aqueduct

B.0.1 In seismic calculation of Grade 1 aqueduct, the hydrodynamic pressure acting on aqueduct body with rectangular or U-shaped cross section may be divided into impulsive pressure and convective pressure (Figure B.0.1), and shall be in accordance with the following requirements:



(a) Aqueduct with rectangular cross section

(b) Aqueduct with U-shaped cross section

Figure B.0.1 Sketch of hydrodynamic pressure model

1 Under transverse seismic action on aqueduct, the impulsive pressures distributed in aqueduct may be converted into horizontally added masses m_{wh} fixed on each side wall along elevation direction and calculated using Formula (B.0.1-1) if $|z/l| \leqslant 1.5$ and Formula (B.0.1-2) if $|z/l| > 1.5$, respectively:

$$m_{wh}(z) = \frac{M}{2l}\left[\left|\frac{z}{H}\right| - \frac{1}{2}\left(\frac{z}{H}\right)^2\right]\sqrt{3}\tanh\left(\sqrt{3}\frac{l}{H}\right) \quad \text{(B.0.1-1)}$$

$$m_{wh} = \frac{M}{2H} \quad \text{(B.0.1-2)}$$

For aqueduct bottom, when $H/l \leqslant 1.5$, the impulsive pressure p_{bh} may be calculated according to the following formula.

$$p_{bh}(x,t) = \frac{M}{2l} a_{wh}(t) \frac{\sqrt{3}}{2} \frac{\sinh\left(\sqrt{3}\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\frac{l}{H}\right)} \quad \text{(B.0.1-3)}$$

where M—total water mass per unit length along aqueduct axis direction, taken as $2\rho_w Hl$ for rectangular cross section and $\rho_w(2hR + 0.5\pi R^2)$ for U-shaped cross section;

z—vertical coordinate of the side wall;

x—horizontal coordinate of the aqueduct bottom;

t—time instant;

$a_{wb}(t)$—horizontal acceleration response at center of aqueduct bottom of each cross section;

ρ_w—water mass density;

H—water depth in aqueduct;

$2l$ or 2R—internal width of aqueduct.

When $H/l > 1.5$, the impulsive pressure at aqueduct bottom is distributed linearly.

2 Under transverse seismic action on aqueduct, in aqueduct the convective pressure may be considered as spring-mass system connected with aqueduct side wall at height h_1. For aqueduct with rectangular cross section, the equivalent mass M_1, equivalent spring stiffness K_1 and height h_1 may be calculated according to the following formulas, respectively:

$$M_1 = 2\rho_w Hl\left[\frac{1}{3}\sqrt{\frac{5}{2}}\frac{l}{H}\tanh\left(\sqrt{\frac{5}{2}}\frac{H}{l}\right)\right] \tag{B.0.1-4}$$

$$K_1 = M_1\frac{g}{l}\sqrt{\frac{5}{2}}\tanh\left(\sqrt{\frac{5}{2}}\frac{H}{l}\right) \tag{B.0.1-5}$$

$$h_1 = H\left[1 - \frac{\cosh\left(\sqrt{\frac{5}{2}}\frac{H}{l}\right) - 2}{\sqrt{\frac{5}{2}}\frac{H}{L}\sinh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}\right] \tag{B.0.1-6}$$

For aqueduct with U-shaped cross section, the equivalent mass M_1, equivalent spring stiffness K_1 and height h_1 may be calculated according to the following formulas, respectively:

$$M_1 = M\left\{0.571 - \frac{1.276}{\left(1+\frac{h}{R}\right)^{[illegible]}}\left[\tanh\left(0.331\frac{h}{R}\right)\right]^{0.932}\right\} \tag{B.0.1-7}$$

$$K_1 = M_1\omega_1^2 \tag{B.0.1-8}$$

$$\frac{R}{g}\omega_1^2 = 1.323 + 0.228\left[\tanh\left(1.505\frac{h}{lR}\right)\right]^{0.788} - 0.105\left[\tanh\left(1.505\frac{h}{R}\right)\right]^{4.609} \tag{B.0.1-9}$$

$$h_1 = H\left[1 - \left(\frac{h}{R}\right)^{0.884}\frac{0.394 + 0.097\sinh\left(1.534\frac{h}{R}\right)}{\cosh\left(1.534\frac{h}{R}\right)}\right] \tag{B.0.1-10}$$

3 Under vertical seismic action, only the impulsive pressure may be considered. For aqueduct bottom, the impulsive pressure may be taken as uniformly-distributed added mass m_{wv} fixed on it and calculated according to the following formula:

$$m_{wv} = 0.4\frac{M}{l} \tag{B.0.1-11}$$

For aqueduct side wall, the impulsive pressure may be considered as horizontal pressure distributed along elevation and calculated by Formula(B.0.1-12). The impulsive pressures p_{wv} on both side walls of aqueduct at each instant are in the same direction:

$$p_{wv}(z,t)=0.4\frac{M}{l}a_{wv}(t)\cos\left(\frac{\pi}{2}\frac{H+z}{H}\right) \tag{B.0.1-12}$$

where $a_{wv}(t)$—vertical acceleration response at center of aqueduct bottom of each cross section.

B.0.2 For Grade 2 aqueduct, both the mass of aqueduct body and added mass of hydrodynamic pressure of two adjacent half spans shall be taken as additional concentrated mass at pier top in calculation of transverse seismic effects of aqueduct pier.

In calculation of aqueduct structure, if $H/l \leqslant 1.5$, the impulsive pressure under horizontal seismic action may be taken as additional concentrated mass M_0 in transverse direction at depth h_0 on aqueduct side wall and calculated according to the following formulas, respectively:

$$M_0=M\frac{\tanh\left(\sqrt{3}\frac{l}{H}\right)}{\sqrt{3}\frac{l}{H}} \tag{B.0.2-1}$$

$$h_0=\frac{3}{8}H\left\{1+\frac{4}{3}\left[\frac{\sqrt{3}\frac{l}{H}}{\tanh\left(\sqrt{3}\frac{l}{H}\right)}-1\right]\right\} \tag{B.0.2-2}$$

If $H/l>1.5$, horizontal added mass uniformly distributed on aqueduct inside walls below the point of $|z|=1.5l$ may be corrected by Formula (B.0.1-2), and linearly distributed impulsive pressure acting on aqueduct bottom may also be corrected accordingly.

Effect of convective pressure may be considered as spring-mass system connected to aqueduct inside wall at height h_1. For aqueduct with rectangular cross section, the equivalent mass M_1, equivalent spring stiffness K_1 and height h_1 may still be calculated by Formulas (B.0.1-4) to (B.0.1-6), respectively. For aqueduct with U-shaped cross section, they may still be calculated by Formulas (B.0.1-7) to (B.0.1-10), respectively.

The acceleration response at pier top shall be taken as the seismic input for support abutment at the bottom of aqueduct body.

Explanation of wording in this standard

1 Words used for different degrees of strictness are explained as follows in order to mark the differences in implementing the requirements of this standard.

1) Words denoting a very strict or mandatory requirement:

"Must" is used for affirmation, "must not" for negation.

2) Words denoting a strict requirement under normal conditions:

"Shall" is used for affirmation, "shall not" for negation.

3) Words denoting a permission of a slight choice or an indication of the most suitable choice when conditions permit:

"Should" is used for affirmation, "should not" for negation.

4) "May" is used to express the option available, sometimes with the conditional permit.

2 "Shall meet the requirements of..." or "shall comply with..." is used in this standard to indicate that it is necessary to comply with the requirements stipulated in other relative standards and codes.

List of quoted standards

GB 50011 *Code for Seismic Design of Buildings*
GB 50111 *Code for Seismic Design of Railway Engineering*
GB 50199 *Unified Standard for Reliability Design of Hydraulic Engineering Structures*
GB 50287 *Code for Hydropower Engineering Geological Investigation*
GB 50487 *Code for Engineering Geological Investigation of Water Resources and Hydropower*
GB 18306 *Seismic Ground Motion Parameters Zonation Map of China*
DL/T 5016 *Design Specification for Concrete Face Rockfill Dams*
DL/T 5057 *Design Specification for Hydraulic Concrete Structures*
NB/T 35056 *Design Code for Steel Penstocks of Hydroelectric Stations*
DL/T 5353 *Design Specification for Slope of Hydropower and Water Conservancy Project*
DL/T 5395 *Design Specification for Rolled Earth-rock Fill Dams*
DL/T 5416 *Specification of Strong Motion Safety Monitoring for Hydraulic Structures*
JGJ 94 *Technical Code for Building Pile Foundations*
NB 35011 *Design Code for Powerhouses of Hydropower Stations*
SL 191 *Design Code for Hydraulic Concrete Structures*
SL 228 *Design Code for Concrete Face Rockfill Dams*
SL 265 *Design Specifications for Sluices*
SL 266 *Design Code for Hydropower House*
SL 274 *Design Code for Rolled Earth-rock Fill Dams*
SL 281 *Design Specification for Steel Penstocks of Hydroelectric Stations*
SL 386 *Design Code for Engineered Slopes in Water Resources and Hydropower Projects*
SL 486 *Technical Specification of Strong Motion Monitoring for Seismic Safety of Hydraulic Structures*
SL 660 *Design Code for Shiplift*