Anti-Seismic Numerical Analysis of Water Intake Structure of Pakistan Karachi K-2/K-3 Nuclear Power Plant

Wang Guixuan*,[1], Yin Xunqiang*,[1] and Zhao Jie[1]

[1]*Civil Engineering Technology Research and Development Center of Dalian University, Dalian, Liaoning, China PR*

**E-mail:* tumuxinxi@163.com

ABSTRACT: Based on the actual conditions of Pakistan Karachi K2/K3 Nuclear Power Plant (NPP), the special topic of seismic numerical simulation calculation and anti-seismic numerical analysis of water intake structure are introduced. Firstly, the project profile of K2/K3 NPP is briefly presented, including the preliminary design, the soil conditions of site, and the purpose and contents of the proposed special topics. Then, the physical and mechanical qualities of the foundation are introduced. Next, the method for calculating the designed ground motion parameters of engineering site are proposed and parts of results are listed which meet the provisions of standard and can be used as an input data for the anti-seismic analysis. Finally, anti-seismic analysis of water intake gate shaft, water intake tunnel, and diversion dike and bank revetment of water intake channel is described, respectively. Through the numerical analysis, it can be concluded that the design scheme put forward in the design can adopt appropriate reinforcement measures and the marine structure is stable under SL2 earthquake loading.

KEYWORDS: Pakistan Karachi K-2/K-3 Nuclear Power Plant, Anti-Seismic Analysis, Water Intake Structure, Seismic Wave Fitting

1. PROJECT PROFILE

The proposed Karachi K-2/K-3 nuclear power plant (K-2/K-3 NPP) is located in the west to Karachi of Pakistan and the north bank of the Arabian Sea, with the geographic coordinate of 66°45′30″~ 66°47′30″ (eastern longitude) and 24°50′30″~24°52′00″ (northern latitude). The proposed K-2/K-3 NPP is approximately 25km from Karachi downtown.

The preliminary design of Karachi K-2/K-3 NPP is based on “Regulation for content and depth of preliminary design document of nuclear power plant” (Han, G. Q., 2013). K-2/K-3 NPP project is design as Advanced China Pressurized water reactor 1000 MW (ACP1000), single unit layout with the safety and economy level of 3rd generation NPP. The main features include: Nuclear Steam Supply System (NSSS) rate thermal power 3060MWt; unit nominal power ≥ 1100MWe; unit average availability ≥ 90%; service life is 60 years; using annually refuelling management strategy for the 1st cycle, and transition to balance cycle of 18 months refuelling by the increase of the fuel enrichment from the 2nd cycle; average depth of fuel burn about 45000MWd/tU; CF2/CF3 fuel assemblies are adopted in the core; Average linear power rate of 173.8 W/cm. The plant comprises of main building area, circulating water facilities area, switch-yards area, auxiliary facility area, administration building area, etc.

The site area is composed of the Pliocene to recent sedimentary deposits of shallow marine, deltaic to alluvial depositional environments (International Atomic Energy Agency, 1993). The stratigraphic sequence is Nari Formation, Gaj Formation, Manchar Formation, and Cover Sediments. A large part of the project area is occupied by horizontal, Sub recent to recent alluvial deposits which are not deformed. The alluvial fans unconformably rest over Manchar formation. There exists no liquefaction stratum in the subgrade layer at the site. The project area lies at the north-western limb of Karachi synclinorium that is bounded by Ran Pathani Anticline in east and Allah Bano Anticline in west. The south eastern limb of the Allah Bano Anticline is cross folded into Mango Pir Anticline and Lalji syncline. The cross folds are located in the northeast of the project area. There are no chances of any magmatic or volcanic activity, and potential of geothermal energy in the site. There are no potential of uplift or subsidence and chances of the subgrade liquefaction. The collapse of narrow channels, however, cannot be ruled out under earthquake loading.

Bed rocks on the proposed construction site are generally made of the pliocene epoch Manchar stratums, with the upper covering layer made of quaternary alluviation and proluvial sediments. The rocks are mainly mud rocks and argillaceous sandstones. The mud stones are mainly greyish yellow or steel grey, etc., in politic texture and stratified structure. With a single layer thickness of 0.1~0.5m, it is the medium bed. The mudstone shows developmental joint fissure in rust color, with thin stratified structure of argillaceous sandstones in some areas. The mud rocks show certain expansibility and will expand after absorbing water or shrink if dry. The repeated expansion and shrinkage may finally result in crush and disintegration of the mud rock. The argillaceous sandstones are greyish yellow, grey or greyish green, with medium to fine particles, silty texture and stratified structure. The stratum facets fluctuate like waves, almost in medium thickness.

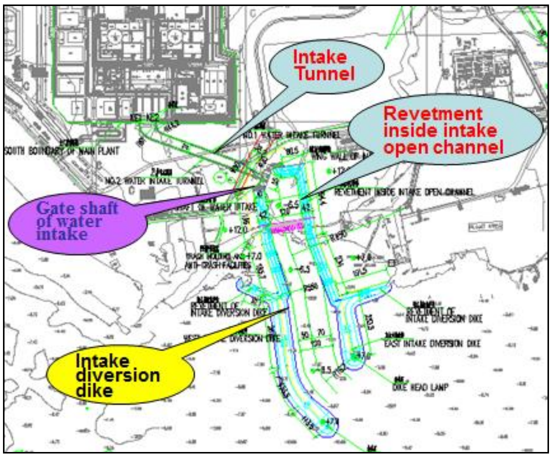


Figure 1 Layout chart of water intake marine structure

Here, the intake tunnels and gate structures will be subject to an acceleration of 0.3 g (Safety Shutdown Earthquake, SSE for short), while other structures be subject to a magnitude 7 (0.18g). In accordance with the technical contract for Subject Studies on Anti-Seismic Analysis of Marine Hydraulic Structure for Marine Water taking of K-2/K-3 NPP in Karachi, Pakistan-jointly signed by China Ocean Engineering Construction General Bureau (COECGB), this subject work for earthquake in this project have been jointly undertaken by Civil Engineering Technology Research and Development Center of Dalian University and Civil Engineering Research Institute of Dalian Lianda School. The layout chart of water intake marine structure is shown in Figure 1, which consists of water intake gate shaft, water intake tunnel, water intake channel diversion dike and intake diversion dike. This paper mainly includes two parts as follows:

(1) Special topic of the seismic numerical simulation calculation: By sorting the existing seismic safety evaluation data

obtained from the project site of K-2/K-3 NPP in Karachi, Pakistan, we can specify the earthquake resistance level adopted in this project, listing the design parameters of ground motion given by seismic safety assessment report obtained from the project site of K-2/K-3 NPP in Karachi, Pakistan. Based on this, synthesis of history of artificial earthquake wave can be performed to give out the damping ratio of respectively 5% and 7% to be deemed as a synthesis of history for artificial earthquake motion deeded for anti-seismic analysis of hydraulic structure in this project, to determine the input of earthquake motion in this project.

(2) For the purpose of implementing the policy of "prevention first, safety first" for seismic works and civil nuclear facilities and realizing the design target of "safe operation, reliable quality, advanced technology and rational cost", numerical simulation analysis is used to validate seismic stability of nuclear-related marine structures, to validate and optimize the design section and finally to provide basic data for the design of and safety assessment on the marine structures.

2. PHYSICAL AND MECHANICAL QUALITIES OF THE FOUNDATION INSTRUCTIONS

The rocks in the site are classified into highly weathered, moderately weathered and slightly weathered rocks. Drilling at the site revealed that the rock below site grade level (12 m elevation) is highly weathered and then is moderately weathered rock.

Highly weathered mudstone can be classified as fractured rock. Moderately weathered mudstone, slightly weathered mudstone and sand stone are classified as blocky rock according to the rock integrity index and joint develop degree.

Bedrock at the site is overall classified as category "V" rock which means that the quality of rock is very poor. Ground water was not encountered up to the maximum investigated depth which means that no groundwater at the site affects the stability of subgrade.

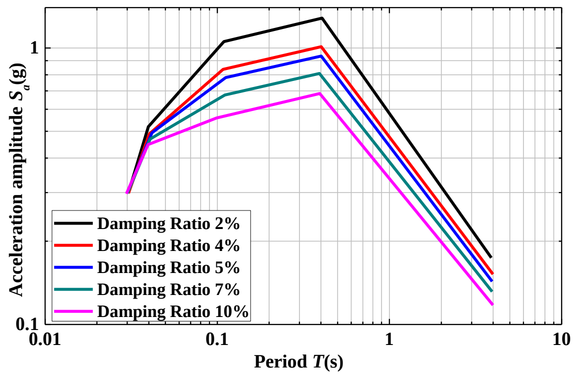
The soil calculation parameters adopted by the calculation and analysis process are shown in Table 1, in which $\boldsymbol{E_s}$ the elastic modulus, $\boldsymbol{E_d}$ the Dynamic elastic modulus, $\boldsymbol{v}$ the Poisson's ratio, $\boldsymbol{\rho}$ the unit weight, $\boldsymbol{\varphi}$ the angle of internal friction, and $\boldsymbol{C}$ the cohesion force.

Table 1 Selection of various material calculation parameters of water intake structure

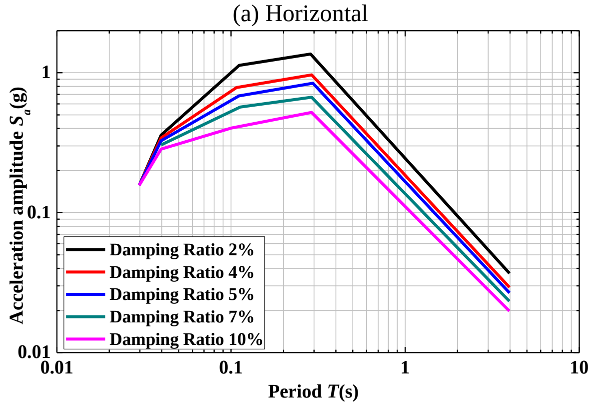
Rock-soil name	E_s (GPa)	E_d (GPa)	v	ρ kN/m^3	φ Degree	C kPa
Gravelly sand	0.17	0.52	0.41	16.6	30.3	51
Strongly-weathered mudstone	0.43	1.29	0.38	22.8	15.0	220
Moderately-weathered mudstone	1.91	3.2	0.32	22.8	20.6	630
Slightly-weathered mudstone	2.01	6.25	0.26	23.2	24.7	1100
Backfill block stone	0.1	0.3	0.42	18.0	42	0

3. DESIGNED GROUND MOTION PARAMETERS OF ENGINEERING SITE

By sorting seismic risk assessment from Pakistan side (International atomic energy agency.1993; Fazal, A. and Haroon, P. D., 1995; Samad, M. A., et al., 1997; International Atomic Energy Agency, 1998; Samad, M. A., et al., 2005; Samad, M. A., et al., 2015) seismic parameters are set as below initially: under ultimate safety ground motion (SL2), horizontal acceleration design peak value of rock is set 0.30g and vertical acceleration design peak value is set 0.30g. Under operation safety ground motion (SL1), horizontal acceleration design peak value of rock is set 0.10g and vertical acceleration design peak value is set 0.10g. The response spectra of Safe Shutdown Earthquake (SSE) are shown in Figure 2 by using the response spectrum stipulated in Revised Version of American Regulatory Guide 1.60 (RG1.60) (U.S. Atomic Energy Commission, 2014), in which the horizontal axis represents period and the vertical axis represents acceleration amplitude of response spectrum.



(a) Horizontal



(b) Vertical

Figure 2 U.S. RG1.60 response spectrums

3.1 Artificial Seismic Wave Fitting Criterion

In the current engineering seismic protection aspect, the most frequently used method for fitting of time generated seismic waves is the response spectrum method which is to build an approximate stationary Gaussian process and then multiple it with the strength envelops to form an initial artificial wave which approximately describes the non-stationary seismic motion at the acceleration time course. Later, technological means are taken to iteratively adjust the Fourier spectral signatures of such an artificial wave to make it satisfy given precision requirements on the target response spectrum.

3.2 Method for Seismic Wave Fitting

3.2.1 Generation of Initial Wave

Along with the extensive application of the response method in most standards and the fact that the artificial ground motion generated based on power spectrum needs the response spectrum for verification, the idea to directly use the response spectrum as the target parameters for simulation came into being (Maharaj, K. K., 1978). By introducing the theoretical transformation relation between the Fourier amplitude spectrum and the power spectrum, the approximation relation between the Fourier amplitude spectrum and the response spectrum can be obtained. Further, according to the uniform distribution hypothesis of random phase angle, or the statistical law of phase difference spectrums, random phase angles corresponding to Fourier spectral lines are generated.

Finally, the initial artificial ground motion time histories are generated by the way of trigonometric series superposition. See below:

$$x(t) = \sum_{k=0}^{N-1} \left| F(\omega_k) \right| \cos(\omega_k t + \varphi_k) \tag{1}$$

In which, $\left| F(\omega_k) \right|$ the Fourier amplitude spectrum of the harmonic component number k, ω_k the corresponding angle frequency, and φ_k the Fourier phase spectrum. Points 0~N-1 are time points of the artificial initial wave time history data.

3.2.2 Function and Waveform Control

For the purpose to reflect the non-stationary of artificial fit seismic wave, a conclusive strength envelop changing with time is added on the original initial stationary random process, for the purpose to show the strength non-stationary of ground motion and make the man-made ground motion time history be closer to the real seismic records. The common model of envelop functions used in seismic design regulations of Japan and the U.S.A. and that used for synthetic ground motion in the seismic micro zonation on the existing key engineering sites of China can be described as:

$$\psi(t) = \begin{cases} (t/t_1)^2 & 0 < t \le t_1 \\ 1 & t_1 < t \le t_2 \\ e^{-c(t-t_2)} & t_2 < t \le T \end{cases} \tag{2}$$

In which, 0~t_1 the ascent stage, t_1~ t_2 the stationary stage of the peak, T the duration, and c the exponential form attenuation coefficient in the descent stage.

3.2.3 Frequency Domain Method for Target Response Spectrum Iteration Fitting

Since the transformation formula between the response spectrum and the power spectrum is the approximation relationship, the response spectrum obtained according to the initial time histories is generally approximate to the target response spectrum only and the degree of coincidence is probability average. However, multiplying it with the strength envelop curve based on the initial wave form may further influence the calculation of the response spectrum and make the difference between the calculation response spectrum and the target response spectrum larger. The Fourier amplitude spectrum adjustment method in the frequency domain is adopted to reduce such difference.

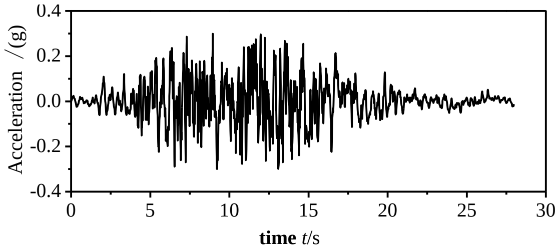
3.2.4 Frequency Domain Method for Target Response Spectrum Iteration Fitting

Firstly, synthesize the initial acceleration time course targeting to give out peak acceleration, response spectrum and strength envelop, with methods used in the traditional frequency domains to adjust the Fourier amplitude spectrum **(**Zhao, F. X., and Zhang, Y. SH., 2007). Later, further adjust such initial artificial wave to further improve the fitting precision of such wave to the target response spectrum, by inputting the inversion formula based on the linear SDOF (single degree of freedom) system and superposition narrowband time courses in the time domain.

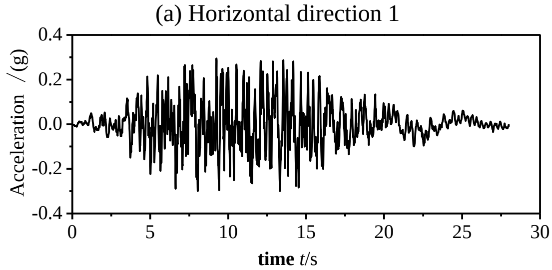
3.3 Method for Seismic Wave Fitting

Combined with the regulation of ground motion duration envelop function in American nuclear code American Society of Civil Engineers (ASCE) 4-98 (ASCE4-98, 2000), rise time of artificial earthquake wave is 5.0s; strong motion duration is 13s; decay time is 10s. The initially selected ground motion time-histories attains to 28s, so that it can be performed for the fitting of ground motion time-history curves under different damping ratios as well as target response spectrum and a comparison of fitting condition for computing the response spectrum. However, only the ground motion time-history curves under 5% damping ratios are shown in Figure 3 in consideration of space.

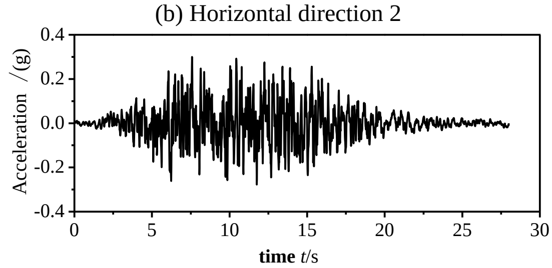
The fitting result shows that there are 77 control frequency points of artificial earthquake motion time-histories; the specific value for average response spectrum to design spectrum ≧1; there is no average spectrum point being lower than 10% of design spectrum. An absolute related coefficient in every two artificial ground motion time-histories should not be greater than 0.3 and all of absolute related coefficients shall meet the provisions of standard ASCE 4-98.



(a) Horizontal direction 1



(b) Horizontal direction 2



(c) Vertical direction

Figure 3 Improved RG1.60 earthquake wave time-history curves (Damping ratio: 5%)

4. NUMERICAL ANALYSIS OF WATER INTAKE STRUCTURE

4.1 Anti-Seismic Analysis of the Water Intake Gate Shaft

Anti-seismic analysis on the water intake gate shaft in the project was performed with the multi-point excitation elastic support tri-dimensional dynamic finite element numerical simulation method and the universal business software ANSYS. In this project, a near-field three-dimensional finite element model was established for simulating the complex geological region which contains uneven strata. On this software basis, a calculation module which can carry out structure-foundation dynamic interaction analysis was developed. In the meanwhile, post-processing modules for internal force extraction and foundation bearing capability and stability analysis were developed in order to achieve the functions of dynamic ground motion input and infinite foundation radiation damping simulation. Therefore, the static and dynamic finite element analysis for structure-foundation system and the extraction of internal force of typical walls and plates were carried out. Then, the bearing capability analysis and solution for stability safety factor were implemented.

4.1.1 Analysis Methods

(1) Structure-Foundation Dynamic Interaction Analysis

Under the earthquake effect, the viscous-elastic boundary is imposed on the external boundary of foundation to simulate the radiation damping effect of infinite foundation. Developed on the basis of viscous boundary (Lysmer and Kulemeyer, 1969), the viscous-elastic boundary can not only simulate the radiation damping effect of far-field infinite foundation but also demonstrate the elastic supporting effect. This boundary condition is expressed by a series of spring-damping units as shown in Figure 4. These boundary conditions are deducted according to the wave form and the solution to elastic wave in infinite media.

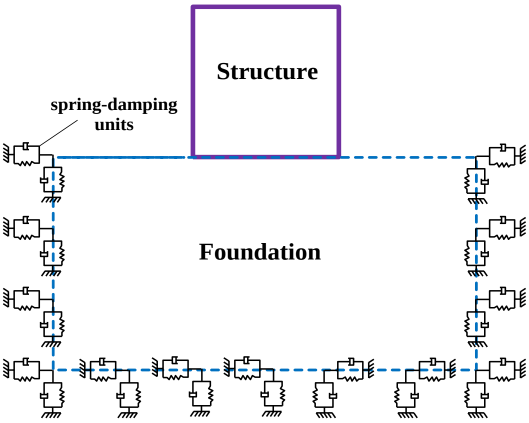


Figure 4 Schematic diagram of viscous-elastic artificial boundary in the numerical model

(2) Simulation of dynamic hydraulic pressure

It is generally assumed that the dynamic hydraulic pressure has a huge influence on the dynamic response of water intake architectural structure under seismic loading and serves as the major dynamic load for seismic design of water intake structures. In accordance with relevant regulations in *Seismic Design Code for Nuclear Power Plant* (GB 50267-97, 1998) and *ASCE 4-98*, the pulse effect and convection effect need to be considered, considering the earthquake causes shock in the water body inside the water intake structures. Pulse pressure is a hydraulic pressure effect correlated with inertia force caused by the pulse motion of water intake structures. The convection pressure is a hydraulic dynamic pressure effect produced on the container wall and bottom plate by shock of water body inside the water intake structures. In the meanwhile, the influence of horizontal motion component and the vertical motion component should be calculated.

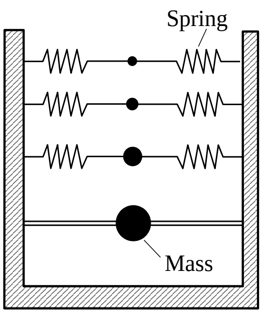


Figure 5 Housner spring-mass system to calculate dynamic hydraulic pressure

The seismic dynamic hydraulic pressure acting inside or outside the water intake structure should be considered by the additional dynamic hydraulic mass and corresponding acceleration of the node. As for the upstream surface of water intake structure, the downstream seismic dynamic hydraulic pressure is calculated according to Westergaard formula; the horizontal dynamic hydraulic pressure acting among the internal walls of water intake structure should be in reference to the relevant regulations in current *Seismic Design Code for Nuclear Power Plant* and ASCE 4-98 and adopt calculation model put forward by Housner (Housner, G. W., 1957; Housner, G.W., 1963). The schematic diagram for calculation is shown in Figure 5, in which the spring-mass system is used to calculate the pulsating pressure and convection pressure produced by complex movement of fluid impacting the structure.

4.1.2 Calculation Model

According to the design scheme for water intake gate shaft which is the RC structure of Karachi K-2/K-3 NPP provided by China Shipbuilding National Development and Research Institutes (NDRI) Engineering Co., Ltd, the structural size of water intake gate shaft is 55m × 29m × 28.5m (length × breadth × height). The simulation range of foundation extends 60m from both sides of gate shaft structure, downwards the bottom plate and alongside the axial direction of water intake gate shaft. The foundation boundary conditions in static analysis include full restraint at the bottom and normal restraint at the sides. The viscous-elastic artificial boundary model is imposed in dynamic analysis.

Based on ANSYS software, the 3D finite element model of water intake gate shaft structure-foundation system is shown in Figure 6, where the material for structure is assumed as linear elastic according to the American Standard Review Plan (SRP) 3.7.2-17 (U.S. Nuclear Regulatory Commission, 2013). The finite element model of water intake gate shaft structure is shown in Figure 7. The properties of the foundation rocks are determined by the relative position of element center and strata interface. Different colors represent different material properties in the model, where sky blue represents C55 concrete, deep purple represents gravelly sand, read represents strongly-weathered mudstone, blue represents moderately-weathered mudstone, pink represents slightly-weathered mudstone, green represents backfill block stone, light purple represents block stone concrete, and the computational mechanical parameters of materials are shown in Table 1.

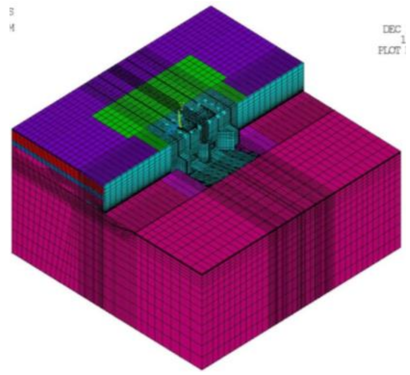


Figure 6 3D finite element model of water intake gate shaft structure-foundation system

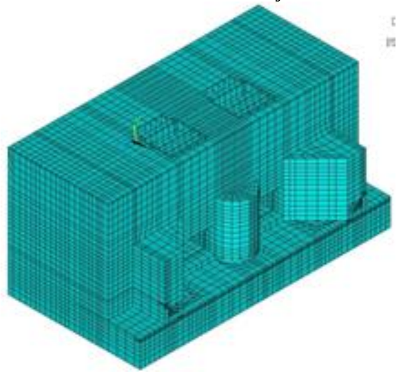


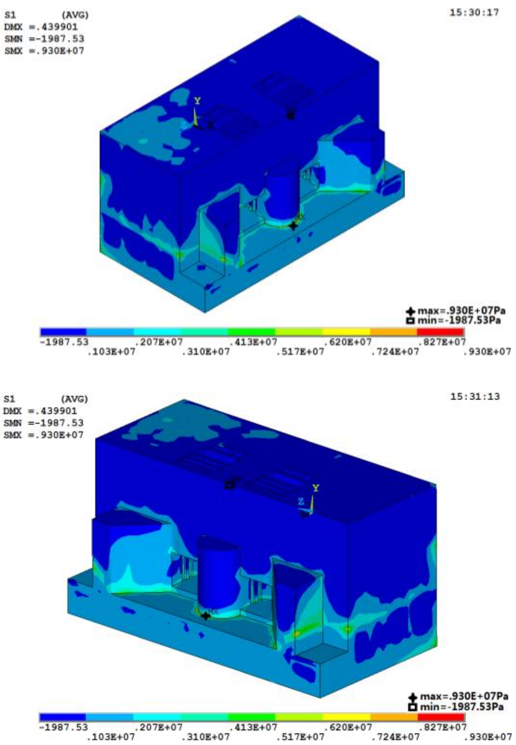
Figure 7 3D finite element model of water intake gate shaft

4.1.3 Description of Working Cases for Calculation

According to the water level, inspection status and seismic loading (consider the effects of normal operation action in combination with severe environmental action and extreme environmental action, respectively), the specific working cases for calculation include 12 working cases for three-dimensional seismic analyses, 12 working cases for foundation bearing capability and stability checking calculation and 12 working cases for structural cross-sectional strength checking calculation.

4.1.4 Analysis of Calculation Results

Owing to space constraints, this paper introduces only part of results, as show in Figure 8 and 9. It can be found from the stress cloud in Figure 8 that the stress which exceeds the limiting tensile strength is mainly distributed in the small region of the external walls in front of water intake gate shaft and the corners of other walls. Besides, there is tensile stress exceeding the limit in a small range on the corners of bottom plate. It can be seen that the areas where the tensile stress exceeds the designed tensile strength are mainly located at the edges and corners of bottom plate, which demonstrates a phenomenon of concentrated stress. The majority of others are distributed around 1.0MPa. In addition, it can be seen from the cloud diagram of the 1st principal stress under the combination of action effects in other working cases that these combinations of actions effects demonstrate a similar changing trend.



(a) The overall water intake gate shaft structure

Figure 8 Working case4 (SL2 Maximum in forward seismic direction) 1st principal stress distribution

It can be found from the stress cloud diagram (Figure 9) that there are larger stresses in a small area on the external wall in front of the water intake gate shaft and other corner areas, which demonstrates a phenomenon of concentrated stress. The majority of others are distributed around -2.0MPa. The structure satisfies limiting compressive stress requirement. The same conclusion can be drawn from the cloud diagram of the third principal stress under combination of action effects in other working cases and the structure satisfies the limiting compressive stress requirement.

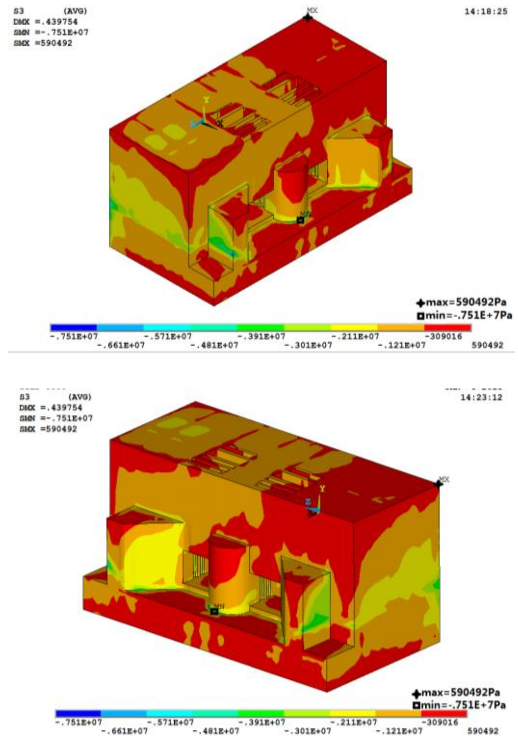


Figure 9 Working case12 (SL2 Maximum in forward seismic direction) 3rd Stress distribution diagram of the overall water intake gate shaft structure

4.2 Anti-Seismic Analysis of Water Intake Tunnel

4.2.1 Analysis Methods

(1) FLAC3D dynamic analytic methods

FLAC3D **(**Itasca Consulting Group. 2012**)** was used for seismic analysis on the water intake tunnel. As an internationally universal analytic procedure for geotechnical engineering, FLAC/FLAC3D showed powerful calculation function and extensive simulation capability. Fully non-linear methods were adopted, making it easy to realize nonlinear dynamic analysis on the structures.

To correctively simulate transmission of the seismic wave in actual site and eliminate reflective effects of seismic wave on the artificial model boundaries, viscous boundary can be set in FLAC3D to absorb or consume wave energies transmitted to the outwards the boundary. Thus, transmission process of the seismic wave can be reflected truthfully. The viscous boundary model is shown in Figure 10, where the sizes of the numerical model of water intake tunnel are usually 5 times the tunnel diameter.

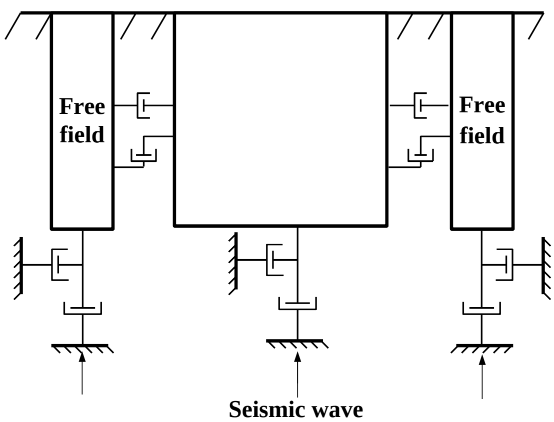


Figure 10 Dynamic loading and boundary constraint modes in FLAC3D

(2) Constitutive Model

In this project, the Mohr-Coulomb plastic model was used to simulate the rock constitutive model. Mohr-Coulomb plastic constitutive model in the FLAC3D is the combined yield constitutive model of the tension stress truncated model according to the Mohr-Coulomb criterion, as shown in Figure 11. The three major stresses are marked as $\sigma_1 \le \sigma_2 \le \sigma_3$, while the yield criterion is defined with $\sigma_1\sigma_3$ and set up in the $\sigma_1\sigma_3$ stress plane. Here, the compression stress is defined as negative and the failure envelop function is $f(\sigma_1,\sigma_3)=0$.

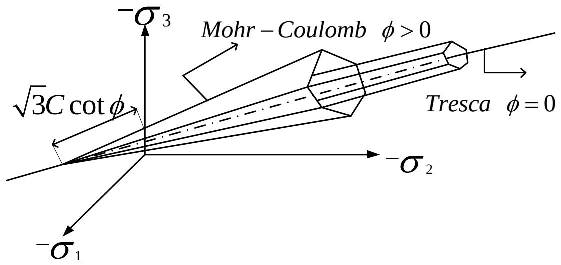


Figure 11 Yield surface of Mohr-Coulomb in the main stress space

4.2.2 Calculation Model

In accordance with the design scheme, the water intake tunnel is divided into entrance section, body section, exit section and open excavation transition section. Owing to spatial confined, the paper enumerates the results of body section only. As is shown in Figure 12, the horseshoe shaped cross-sectional is employed for the water intake tunnel. The clear height and width of the cross-sectional are 6.36m. The length of water intake tunnel is 567.05m. Construction technology of open excavation is adopted in both ends of the tunnel (total length of approximately 30 m), and hidden underground construction for the rest. In addition, the changes to internal force and deformation of lining at different parts under static and seismic ground motion loading are obtained in the body section of water intake tunnel. There are 24 internal lining force control points alongside the lining circular direction and 4 lining deformation control points (respectively on vault top, side wall midpoint, lining bottom and two vault feet).

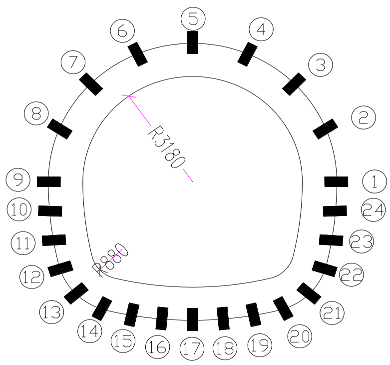
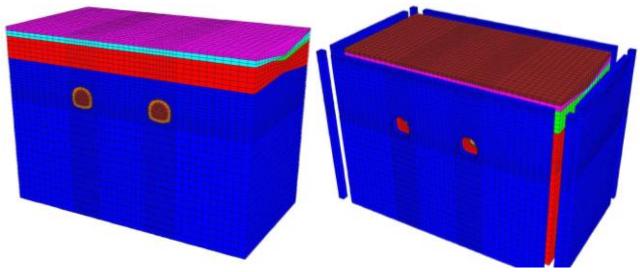


Figure12 Dimensions of the tunnel and configuration of tunnel internal force and deformation control points

The model for 3D dynamic analysis of body section of water intake tunnel is consistent with the size and scope of static model as shown in Figure 13. The calculation scope covers 5 times the tunnel diameter from both sides and 50 m downwards the bedrock from the tunnel center. Adhesive boundary was set at the model bottom during calculation and both sides adopt energy penetration boundary. The maximum grid breadth in this calculation is 7.50 m and the influence from major frequency component in seismic wave can be considered. In FLAC3D, Beam element was adopted to simulate steel support, the initial lining adopts solid element, the secondary lining adopts SHELL element and Cable element was adopted to simulate system anchor rod and fore-piling anchor rod. In the geotechnical projects, the support function of anchor rod can be summarized as: solidify, combine, arching and overhanging. The anchor rod can be combined with nearby rocks into a whole so that to improve the bending and shearing resistance of the ambient rocks.



(a) Static model (b) Dynamic model

Figure 13 Three-dimensional analysis model of body section of water intake tunnel

It is important to note that the static model and the dynamic model need be established respectively because the boundary condition is different.

4.2.3 Description of Working Cases for Calculation

As for the static analysis, three conditions were taken into consideration: action of the construction bracing only, action of the permanent support only, and joint action of the former two and their effect combination. In the 3D dynamic analysis, two kinds of seismic wave (SL1 and SL2) were taken into consideration and the ground motion was inputted in three directions. After taking reference from seismic analysis cases of previous tunnel projects (Wang G. X., 2013), only the combined effects of internal water pressure and seismic action were taken into consideration for the water intake tunnel, while the combined working condition between construction overhauling and earthquake was excluded from consideration.

4.2.4 Analysis of Calculation Results

Owing to space constraints, this paper introduces only part of results.

(1) 3D static analysis

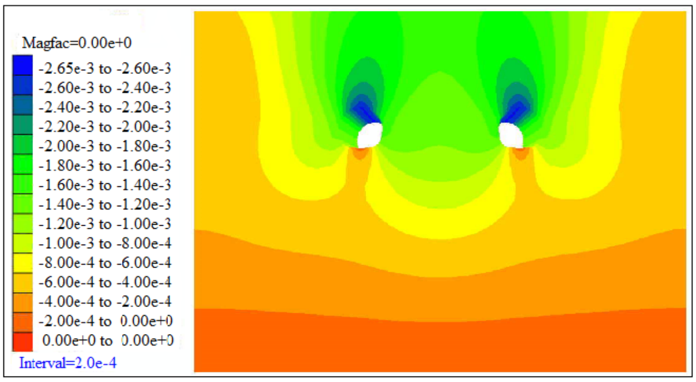


Figure 14 Tunnel deformation diagram

Figures 14 and 15 present the tunnel deformation and lining internal force in 3D analysis respectively when only permanent support is considered. It can be seen that the deformation to side walls after the left tunnel is excavated, is about 0.797mm, the downward deformation of vault is about 2.632mm, the downward deformation of the bottom is about 0.105mm, the downward deformation of the left bottom corner is about 0.995mm and the downward deformation of the right bottom corner is about 1.150mm. The maximum forward bending torque of lining is 620.4kNm, located at the right bottom corner; the maximum negative bending torque is 431.7kNm, located at the bottom; the lining is compressive structure, the maximum axial force is 2551kN, located on the side wall while the minimum axial force is 629.8kN, located on the bottom; the maximum positive shearing force of lining is 442kN, located at the left bottom corner; the maximum negative shearing force is -445.5kN, located at the right bottom corner.

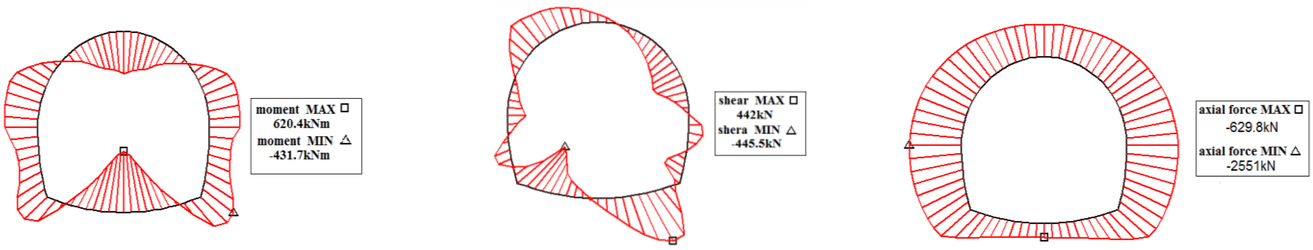
The deformation to side walls after the right tunnel is excavated is about 0.571mm, the downward deformation of vault is about 2.631mm, the downward deformation of the bottom is about 0.103mm, the downward deformation of the left bottom corner is about 1.144mm and the downward deformation of the right bottom corner is about 0.997mm. The maximum bending torque of lining is 620.5 kNm, located at the left bottom corner; the maximum negative bending torque is 432.4kNm, located at the tunnel bottom; the lining is compressive structure, the maximum axial force is 2549kN, located on the right side wall while the minimum axial force is 629.7kN, located on the bottom; the maximum positive shearing force of lining is 409kN, located at the left bottom corner; the maximum negative shearing force is -447.2kN, located at the right bottom corner.

The above results show that the deformation of two tunnels is substantially symmetric and the maximum deformation is 2.632 mm and occurs at the top of left tunnels, meanwhile the right tunnel deformation of 2.631mm also occurs at the top. This indicates that the maximum deformation satisfy regulation requirements.

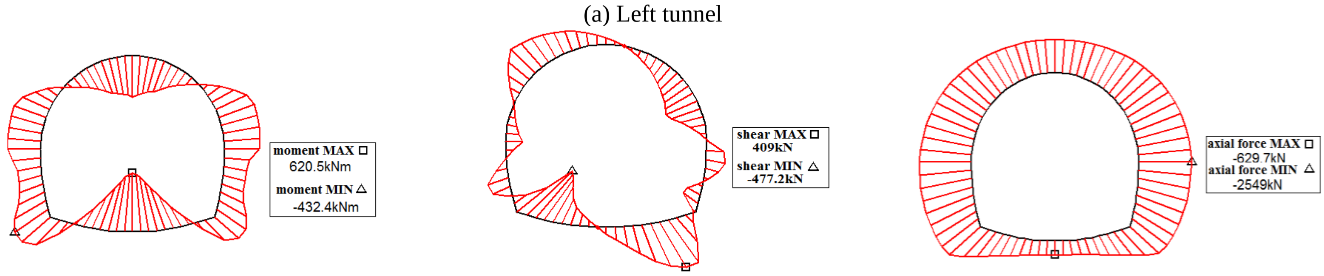
(2) 3D dynamic analysis

Figure 16 presents the tunnel maximum internal force and time in three-dimensional analysis when permanent support and internal hydraulic pressure were considered under the action of SL1 earthquake. It can be seen that the maximum forward bending torque of left tunnel lining is 730.9kNm, located at the bottom; the lining is compressive structure, the maximum axial force is 3095 kN, located on the left side wall; the maximum positive sheer force of lining is 474.6 kN; the maximum forward bending torque of right tunnel lining is 732.6 kNm, located at the bottom; the lining is compressive structure, the maximum axial force is 2820 kN, located on the right side wall; the maximum positive shearing force of lining is 426.8 kN.

Through the comparison and analysis of internal force of 3D static analysis and 3D dynamic analysis, it could be seen that the corresponding control working case is SL1. In the meanwhile, the reinforcement scheme was calculated by the calculated lining internal force in control working case, and the reinforcement scheme finally determined for tunnel lining structure under pressure was E28@150 (E represents HRB400 classes of steel bar, 28 mm represents bar diameter, and 150 mm represents centre distance), which can satisfy seismic requirement..

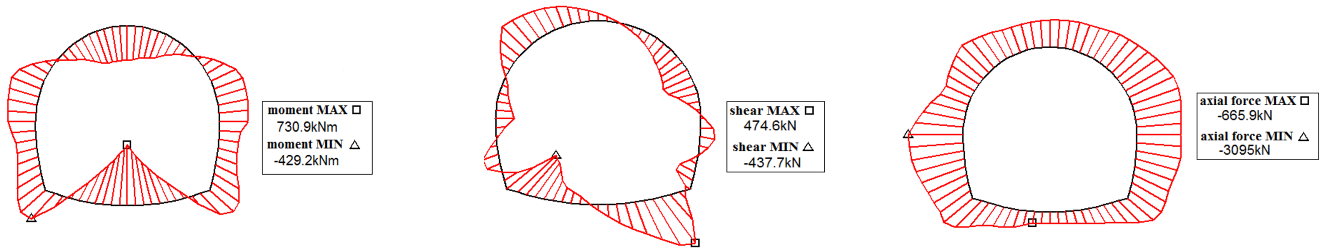


(a) Left tunnel

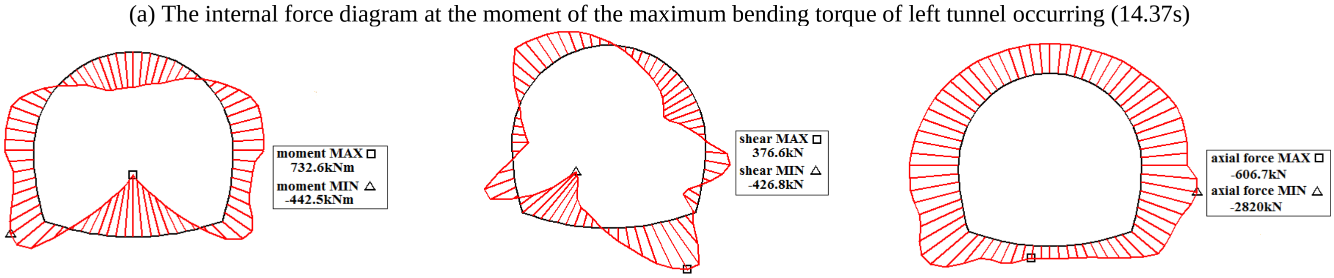


(b) Right tunnel

Figure 15 Tunnel lining internal force diagram



(a) The internal force diagram at the moment of the maximum bending torque of left tunnel occurring (14.37s)



(b) The internal force diagram at the moment of the maximum bending torque of right tunnel occurring (14.40s)

Figure 16 The time and corresponding distribution diagram of maximum internal force

Above all, it is shown by the static and dynamic analysis results of tunnel that the construction support undertakes a certain proportion of pressure from surrounding rock and reduces the internal force of permanent support. However, the effect evaluation of the permanent support should be obtained from monitoring in tunnel construction period. To carry out water intake tunnel design in an economic and reasonable way, various monitoring activities should be carried out in construction period for convenience of dynamic design.

4.3 Anti-Seismic Analysis of Diversion Dike and Bank Revetment of Water Intake Channel

Anti-seismic mathematical analysis and calculation were carried out on the bank revetment and diversion dike of intake channel in Pakistan Karachi K-2/K-3 NPP Marine Engineering Project and then the structural anti-seismic stability of all the structures were verified in order to provide secure and reliable basis for structural design of the marine engineering project.

4.3.1 Analysis Methods

In accordance with the regulations in *Code for Anti-Seismic Design of Nuclear Power Plant* (GB50267-97), *Design Code for Maritime Structures of Nuclear Power Plant* (NB/T25002-2011), *Code for Hydraulic Design of Nuclear Power Plant* (NB/T25046-2015) and requirements from the entrusting party, the analysis in this project adopted sliding surface method and dynamic finite element method successively, which are described briefly as follows.

(1) Sliding surface method

When the method of slices is adopted to calculate the stability of marine structures under earthquake loading, Swedish method or the simplified Bishop's method can be adopted. Swedish method has a long history and has accumulated rich operation experiences in numerous projects, while Bishop's method is featured by a simple theoretical background and high calculation precision, which makes it widely applied in many projects. Therefore, this project adopts Swedish method and Bishop's method to analyse the stability of marine structures.

The earthquake load is simplified as constant acceleration load in horizontal or vertical directions; this acceleration load produces an inertial force impacting on the centroid of unstable object. Thus, static force and earthquake inertial force are superimposed on the soil slices. Based on the limit equilibrium method, a force equilibrium condition is established to obtain the factor of safety.

(2) Dynamic finite element method

Dynamic finite element calculation is to calculate the stress distribution and deformation distribution inside the soil structures and then to evaluate the stability based on these results. In the dynamic response analysis of soil structure in diversion dike, it is assumed that the soil structural material is a nonlinear elastic agent with adhesive damping. Its dynamic balance equation is given as:

$$[M]\{\ddot{a}\}+[C]\{\dot{a}\}+[K]\{a\}=\{F\} \tag{7}$$

Table 2 Dynamic shearing modulus ratio and damping ratio under different shearing strains

Rock-soil name		shearing strain γ_d (*10^{-4})						
		0.05	0.1	0.5	1	5	10	50
Gravelly sand	G_d/G_{dmax}	0.99	0.99	0.97	0.93	0.73	0.58	0.22
	λ_d	0.001	0.002	0.01	0.02	0.06	0.10	0.20
Backfill block stone	G_d/G_{dmax}	0.99	0.98	0.95	0.92	0.81	0.70	0.45
	λ_d	0.01	0.012	0.02	0.03	0.05	0.07	0.08

Step-by-step integration method (Wilson-θ) was adopted to solve above dynamic equation. The displacement, velocity and acceleration obtained were used to determine the stress-displacement relation according displacement shape function in order to obtain the unit dynamic strain. Then, the dynamic stress of each unit was obtained by the stress-strain relation on the basis the constitutive models as below.

It is worth noting that the accelerations adopted are shown in Figure 3. In addition, the Equivalent Linear Model was used to simulate the nonlinear characteristic of gravelly sand and backfill block stone (the calculation parameters are shown in Table 2, where G_d/G_{dmax} is shearing modulus ratio, λ_d is damping ratio), and the linear elastic model were used for the other rock constitutive models.

4.3.2 Description of Working Cases for Calculation

The impacts from designed high water level, low water level and different seismic ground motions were considered separately in the analysis. According to the design scheme provided by China Shipbuilding NDRI Engineering Co., Ltd, the calculation water levels are as follows: (1) Designed high water level (high tidal level once every 50 years): 1.86m; (2) Designed low water level (low tidal level once every 50 years): -2.42m.

The anti-seismic mathematical modelling of this project mainly calculated the failure condition of bank revetment and diversion dike under SL2 earthquake loading. As non-nuclear safety items, the bank revetment and diversion dike should be verified that their failure will not influence the water intake safety of key plant water supply system under earthquake loading.

4.3.3 Sliding Surface Method Analysis

(1) Calculation and analysis model

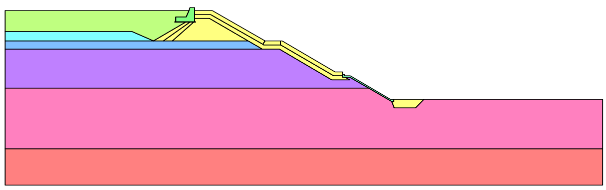


Figure 17 Analysis model of A-A section (bank revetment of intake channel)

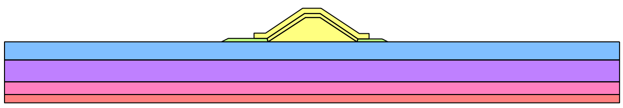


Figure 18 Analysis model of C-C section (diversion dike of intake channel)

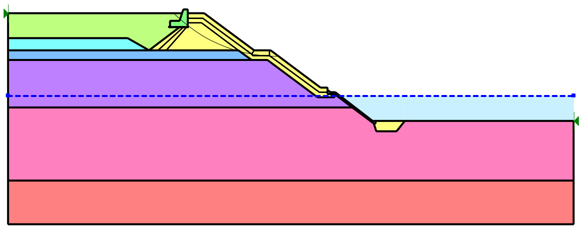
According to the design scheme provided by China Shipbuilding NDRI Engineering Co., Ltd, typical sections analysis models of bank revetment A-A and diversion dike C-C of intake channel were established respectively, as specifically shown in Figure 17 and 18.

(2) Results of anti-seismic calculation and analysis

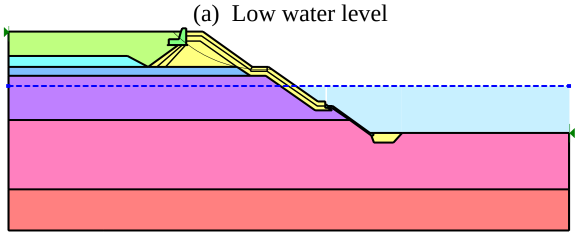
Table 3 Safety factors of bank revetment slip surface

section	water level	sliding direction	Bishop method	Swedish method
A-A	High	-	1.528	1.456
	Low	-	1.570	1.496
C-C	High	outer sea side	2.092	1.613
		water intake side	1.899	1.418
	Low	outer sea side	1.845	1.510
		water intake side	1.842	1.495

Table 3 summarizes the results of safety factors in two sections. Figure19 and 20 present the shape and location of sliding surface in two sections respectively. It can be seen that under SL2 earthquake loading, the safety factors of two sections are larger than 1.4 no matter in what working conditions, thus providing relatively large safety reserve.

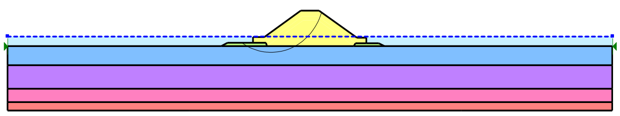


(a) Low water level

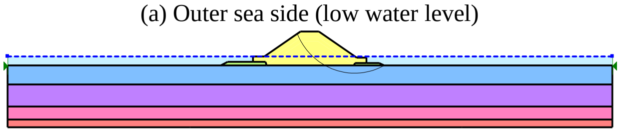


(b) High water level

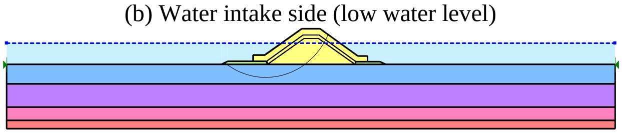
Figure 19 The most dangerous sliding arc in A-A section



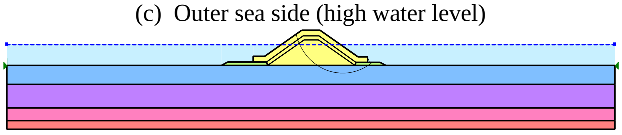
(a) Outer sea side (low water level)



(b) Water intake side (low water level)



(c) Outer sea side (high water level)



(d) Water intake side (high water level)

Figure 20 The most dangerous sliding arc in C-C section

4.3.4 Dynamic Finite Element Stability Analysis of Marine Structure

(1) Calculation and analysis model

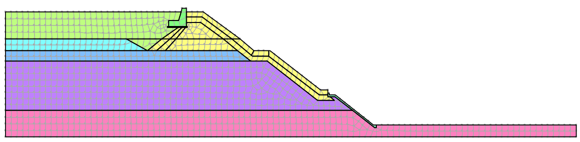


Figure 21 Finite element analysis model of A-A section

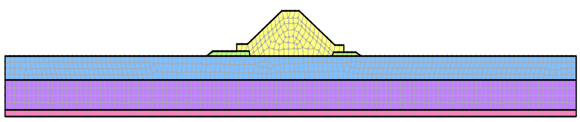


Figure 22 Finite element analysis model of C-C section

Finite element analysis models for bank revetment and diversion dike in Intake channel were established respectively in accordance with the design scheme provided by China Shipbuilding NDRI Engineering Co., Ltd as specifically shown in Figure 21 and 22.

The left and right boundary range of finite element models of both fault surfaces are larger than 2.5 times of the diversion dike breadth or bank revetment breadth. The bottom depth is set as the moderately-weathered bedrock surface provided in the geotechnical engineering prospect report. In addition, the mesh density of the above models is satisfied with the requirement of waves propagating.

(2) Results of anti-seismic calculation and analysis

In accordance with the requirements in *Code for Anti-Seismic Design of Nuclear Power Plant*, the earthquake stability check calculation is carried out according to following steps using Dynamic Finite Element Method:

1) Calculation of stress caused by self-weight inside the soil;

2) Calculation of stress inside soil structure under static vertical earthquake loading;

3) Calculation of response values (stress, acceleration and displacement inside the soil) under horizontal earthquake loading;

4) Calculation of stress combining above three types of stresses and check calculation of sliding resistance.

The safety coefficient of soil structure at a certain moment can be defined as: the ratio between the sum of shear resistance intensities and the sum of sliding shear stresses on sliding surface

Table 4 summarizes the results of safety factors of different slip surfaces in different working conditions. It can be seen that under SL2 earthquake loading, the minimum dynamic safety factors of both bank revetment and diversion dike structure are larger than 1.0, thus providing relatively large safety reserve.

Table 4 Safety factor in dynamic finite element stability analysis of bank revetment slip surface

section	water level	SL2	section	water level	sliding direction	SL2
A-A	High	1.074	C-C	High	outer sea side	1.374
					water intake side	1.371
	Low	1.074		Low	outer sea side	1.312
					water intake side	1.232

5. CURRENT SITUATIONS

According to the main contract of K2/K3 Project signed by China National Nuclear Corporation (CNNC) and PNRA in Beijing on February 18, 2013, concrete pouring for the first reactor facility of K2/K3 have been done on Jan. 31th, 2015. In order to achieve that goal above, the preliminary design work was developed by China Nuclear Engineering Co. Ltd (CNPE) associated with Nuclear Institute of China (NPIC) and East China Electric Design Institute (ECEPDI). Currently, K-2/ K-3 NPP are still under construction and it is expected to be completed by 2020 and 2021. In addition, the anti-seismic numerical analysis of water intake structure of Pakistan Karachi K-2/K-3 NPP has entirely been completed. In addition, the monitoring activities have been suggested to the owner and it is still under discussion so that the monitoring data cannot be provided.

6. CONCLUDING REMARKS

In this paper, the project profile of Pakistan Karachi K-2/K-3 Nuclear Power Plant was briefly introduced. This manuscript emphasized two special topics of Water Intake Structure. Hereinto, the special topic of the seismic numerical simulation calculation was to specify the earthquake resistance level adopted in this project and to determine the input of earthquake motion in this project. And then, several different numerical simulation analyses were used to validate seismic stability and optimize the design section of nuclear-related marine structures, including water intake gate shaft, water intake tunnel, diversion dike, and bank revetment of water intake channel. The results finally provided basic data for the design of and safety assessment on the marine structures.

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