Analysis of seismic Response and Slope Stability for Intake Tunnel Entrance of Hongyan River Nuclear Project

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Abstract. The principle of dynamic analysis of Flac3D, setting of boundary condition and the solving method of dynamic safety factor for slope are introduced in this article firstly, and the three-dimensional finite difference model of the portal of intake tunnel is built by Flac3D which uses the nonlinear dynamic analytical method to simulate response regularity of the portal of intake tunnel and the high slope backfilled here under seismic load for Hongyan River nuclear intake tunnel first-stage project in Liaoning province. The corresponding change rule of the internal force of tunnel entrance under the effect of running safe seismic load SL1 and ultimate safe seismic load SL2 is gotten by time-histories changing curve under the effect of different seismic and the comparison of tunnel lining internal force diagram; the location sliding surface and dynamic safety factor time-histories curve of the high slope backfilled at the tunnel entrance are gotten by building two-dimension finite element model and doing stability analysis under seismic. Analysis result shows that for tunnel entrance, the analysis method and conclusions of the seismic weak position of the tunnel portal are the sidewall and getting the location of sliding surface and minimum dynamic safety factor time-histories curve of high slope. For similar nuclear power intake tunnel project, the conclusion can be a reference and have a certain reference value for the seismic design of tunnel.

1. INTRODUCTION

As a part of power industry, nuclear power is important in China. Generally, because the intake tunnel transported nuclear safety water to conventional island, the safe stability of it played a key role in safety use of nuclear power. In terms of code for seismic design of nuclear power plant [1], intake tunnel has strict seismic requirement as nuclear safety class I. Many geological disasters induced by earthquake of tunnel demonstrate that [2,3]: the break such as the dehiscence in tunnel lining and the collapse and deformation of terrane is induced by earth surface cracks, landslide and soften earthquake subsidence and other geological hazard under strong earthquake. Therefore, the seismic analysis of tunnel and other underground structure turn into one of the key study objects for international researchers.

The entrance is the vulnerable spot of seismic for tunnel as the exposed part of the tunnel and it has the poorer geological conditions [4,5,6]. The terrane of the tunnel portal has the feature of inhomogeneity, nonlinearity and anisotropy and it always has serious accumulation body weathered. This kind of terrane often causes natural slope failure, landslide and collapse. The seismic response of underground structures is obviously different from above ground structure[7]. Therefore, it is significant to undertake the anti-seismic analysis of the tunnel portal in earthquake area for both theoretical and practical use. There are two parts of the seismic stability analysis of tunnel entrance,
including rock stability and high slope at the entrance stability. In article of seismic about slope and tunnel at present, Ervin Paci [8] and Youssef M.A.Hashash [9] discussed the seismic of slope and tunnel in mountain tunnel but no in-depth exploration, and Yamshita Atsuki [10] and Tashi Tshering [11] undertook research of seismic of slope and tunnel in mountain tunnel but the slope is rock slope only. Atsuki Kusuka [12] discussed seismic of slope and tunnel, however, the axial direction of tunnel was parallel to the slope surface in his study. The issues of tunnel portal were discussed with respect to seismic analyses and design strategies. By test of shaking table model and analysis, Xiang Zhao [13] concluded that cracks occur on the side slope above the tunnel and near the tunnel portal. Jia-Jia Xin [14] studied the seismic dynamic response, including the displacement and stress, of tunnel portal. Mallika S [15] and G.Lanzano [16] focused on uncertainty modeling and limit state reliability of tunnel supports under seismic effects. Hack Robert [17] and Siming He [18] developed the slope stability evaluation standard and the slope limit aseismic. Research mentioned above provided fundamental basis for this study. At present, it is required to better understand the effects of tunnel portal and slope stability on earthquakes.

In this article, the seismic analysis of portal area of intake tunnel in HongYan River nuclear power first stage construction is presented, and internal force envelopes of lining is calculated based on it. The transformation law of lining internal force and seismic weak zones of tunnel portal are studied and seismic stability of high slope portal is analyzed to provide references for similar project.

In this paper, the part two introduced fundamental of dynamic analysis, the analytical method of dynamic balance, constitutive relation, the set of dynamic boundary condition, and dynamic bucking safety factor of slope and its solution, which provide fundamental basis for part three. The part three is building calculation model by actual data of nuclear power project of HongYan River, analyzing stability of high slope, and calculating after import the data of seismic wave. The part four is analyzing the result and getting the conclusion of the seismic weakness part of tunnel portal and slope stability.

2. THE BASIC PRINCIPLE OF DYNAMIC ANALYSIS

2.1. Method of Dynamic Balance Analysis

In dynamical response analysis of this paper, rock mass material was assumed to be nonlinear elastic medium which have viscous damping. To express the interrelations among the response displacement, response velocity and response acceleration of dynamic analysis, the equation of dynamic balance [19] can be formulated by:

\[ [M]\dddot{a} + [C]\ddot{a} + [K]a = F \]  \hspace{1cm} (1)

where \([M]\) is mass matrix; \([C]\) is damping matrix; \([K]\) is stiffness matrix; \(\{a\}, \{\dot{a}\}, \{\ddot{a}\}\) are relative displacement, velocity and acceleration of node respectively; and \(F\) is nodal forces.

2.2. Constitutive Relation

The constitutive model of rock is assumed to be Mohr-Coulomb model [20], which has been proved that can describe soil better than Drucker-Prager, because the latter one always tend to be very inaccurate for the condition of axial symmetry.

\[ F = \frac{I_1}{3} \sin \phi + \sqrt{J_2} \cos \theta_s - \frac{J_3}{\sqrt{3}} \sin \phi \cos \theta_s - c \cos \phi = 0 \]  \hspace{1cm} (2)

where \(I_1\) is the first invariant of stress tensor; \(J_2\) is the second invariant of deviator stress; \(\theta_s\) is Lode angle of stress.

When associated flow rule is used, yields:

\[ \alpha = \frac{\sin \varphi}{\sqrt{3(3 + \sin^2 \varphi)}} \]  \hspace{1cm} (3)

\[ k = \frac{3c \cos \phi}{\sqrt{3(3 + \sin^2 \varphi)}} \]  \hspace{1cm} (4)

when non-associated flow rule is used, there will be:
\[ \sin 3\phi = \frac{1}{3} \]  

(5) 

\[ k = c \cos \phi \]  

(6) 

if the dilation angle is introduced and used to replace \( \phi \), yields:

\[ Q = \frac{4}{3} \sin \phi + \sqrt{6} \cos \theta_\sigma - \sqrt{3} \sin \phi \cos \theta_\sigma - c \cos \phi = 0 \]  

(7)

For the soil structure, using associated flow rule lead to volume expansion which is not credible on the physical during the period of yield usually, non-associated flow rule is quoted in this paper, and dilation angle \( \phi \) equals to 0.

2.3. Options of Dynamic Boundary Conditions

Seismic wave will generate refracted wave and reflected wave when it incidence the interface have different medium according to wave theory, meanwhile, the wave have approximate frequency will intervene. For simulating course of seismic wave diffuse in actual conditions and eliminating reflecting effect induced by seismic wave on the boundary of artificial model, viscous boundary [21] was installed in model to consume or absorb wave which will diffuse to outside of boundary, and the model can reflect the factual spread of seismic wave.

Viscous boundary is firstly proposed by Lysmer and Kuhlemeyer (1969). Viscous boundary is setting the nominal and tangential dashpots and the dashpots generate positive and tangential viscous resistance \((t_n, t_t)\), it can be formulated as:

\[ t_n = -\rho C_p v_n \]  

\[ t_s = -\rho C_s v_s \]  

(8)

where \( v_n \) is nominal component of velocity on the boundary and \( v_t \) is tangential, and \( \rho \) is density of medium; \( C_p \) is propagation velocity of P wave in medium and \( C_s \) is S wave.

2.4. The Safety Factor and Solution For Dynamic Stability of Slope

As the important indicator which reflects the slop dynamic stability, safety factor of slope can be defined as the ratio of sum of shear strength and sum of glide shear force on the potential slip surface at the moment:

\[ F(t) = \frac{\sum \sigma_r}{\sum S_m} \]  

(9)

Where \( S_r \) is shear strength of each point on the sliding surface, \( S_m \) is shear force of each point on the sliding surface.

Because stress distribution is given by element mesh of plane finite element stress analysis, as shown in Fig. 1, the element which point C on curve AB belong to should be determine first if its stress needs to be calculated. The stress state is worked out by interpolation function firstly, and normal stress \( \sigma_n \) and shearing strength \( \tau \) on slide plane are obtained by equation (10). The safety factor value of slope can be analyzed and obtained by the normal stress \( \sigma_n \) and shearing strength \( \tau \). The dynamic stress at each moment is gotten by dynamic finite element calculation and distribution of interpolation, while overlapping the static stress filed is used for analyzing the dynamic stability.

\[ \sigma_n = \frac{1}{2} (\sigma_s + \sigma_t) + \frac{1}{2} (\sigma_s - \sigma_t) \cos 2\alpha + \tau_\sigma \sin 2\alpha \]  

\[ \tau = \frac{1}{2} (\sigma_s - \sigma_t) \sin 2\alpha - \tau_\sigma \cos 2\alpha \]  

(10)

with:

\[ \sin 2\alpha = 2 Y_n'(1 + Y_n^2) \]  

\[ \cos 2\alpha = (1 - Y_n^2)/(1 + Y_n^2) \]

Where \( Y_n' \) is the normal slope of any point on the sliding surface.
3. APPLICATION AND EXAMPLE

3.1. General Situation of Engineering
The mode of water intaking, one machine one tunnel, was adopted in nuclear power unit in first-stage project of LiaoNing HongYan River nuclear power engineering, and the water was channeled to PX pumping room by two intake tunnels which has dimension of 5.5m. The section configuration of intake tunnel was roundness, and the inner diameter of tunnel was 5.5m, diameter of excavation was 6.5m. The steel arch bridge, system rock bolts and advance rock bolts were combined used in excavation of tunnel, as shown Figure 2 [22]. According to geologic survey report of this nuclear plant construction documents design phase, the geology of water intaking area was stable bed rock, and the lithological character of bed rock was granite and gneiss; weathered state was intense weathering and moderate weathered mainly; and rock mass was rather broken. The division of surrounding rock of tunnel was: strongly weathered granite, strongly weathering gneiss, moderately weathered granite and moderately weathering gneiss.

3.2. Calculation Model
Category of surrounding rocks was considered as gneiss of class V because the surrounding rock at intake tunnel portal was worse. The 3D dynamic analysis model was shown as Figure 3. The model extended distance of five-fold of the tunnel diameter at left and right sides, and extended 50m downward from bottom of tunnel. The model dimension as follows, the length of tunnel was 80m, axial length was 120m, width was 128m. In calculation model of FLAC3D, if the propagation process of seismic wave was expected to be simulated accurately, the value of space unit size Δl should be less than 1/10 to 1/8 of the wave length which corresponding to the maximum frequency of import wave, that is Δl < λ/10. The main frequency component of earthquake ground motion was about 10Hz, the minimum shear wave velocity of rock material was 925m/s as the value in section 3.3, and the corresponding wave length was 92.5m, that was the corresponding size of mesh opening was 9.25m. In this paper, the largest grid size was 8m, so the value met the requirement. The null element was used to simulate excavation of tunnel, CABLE element for rock bolt, BEAM element for steel bracing and pipe shed, entity element for the first lining, and SHELL element for secondary lining. The damping use local damping, and dynamic boundary conditions used viscous boundary.
For further analyzing the stability of high slope at the tunnel mouth, 2D finite element models of high slope at portal section are established after considering the elastoplasticity of rock mass. The profiles were shown in Figure 4 and Figure 5.

Moreover, the thickness of the bedrock 50 m below the intermediary weathered sandstone, the vertical constraints on both sides of the foundation, and damped boundary in horizontal direction were adopted to simulate the radiation damping, we backfilled the excavation section of intake structure’s slope to elevation of 7.7 meters, and the backfill materials was macadam.

Table 1. Calculate Parameter for Intake Tunnel

| Project               | Lithology          | Intensely weathered granite | Intermidiate weathered granite | Intensely weathered gneiss | Intermidiate weathered gneiss | Backfill crushed stone |
|-----------------------|--------------------|-----------------------------|-------------------------------|---------------------------|-------------------------------|------------------------|
| Self-weight (kN/m³)   | 23.3               | 25.0                        | 23.0                          | 24.5                      | 18.0                          |                        |
| Cohesion of rock mass (kPa) | 60                 | 400                         | 45                            | 300                        | 0.0                           |                        |
| Internal friction angle of rock mass (°) | 27                 | 32                          | 24                            | 30                         | 36.0                          |                        |
| Elasticity modulus (GPa) | 2.50               | 8.00                        | 1.50                          | 4.00                       | 0.1                           |                        |
| Poisson ratio         | 0.38               | 0.29                        | 0.40                          | 0.33                       | 0.33                          |                        |
| Compressional wave (m/s) | 2043.7              | 3951                        | 925                           | 3150                       | 700                           |                        |
| Shear wave (m/s)      | 998.7              | 1834                        | 925                           | 1500                       | 350                           |                        |
| The dynamic modulus of elasticity(GPa) | 5.8                | 23.8                        | 5.2                           | 14.6                       | 0.2                           |                        |
| The dynamic shear modulus(GPa) | 2.2                | 8.8                         | 1.9                           | 5.4                        | 0.07                          |                        |
| The poisson ratio     | 0.36               | 0.35                        | 0.37                          | 0.35                       | 0.40                          |                        |
| Damping ratio %       | 5                  | 5                           | 5                             | 5                          | 5                             |                        |

3.3. The Calculating Parameters of Rock Mass

The calculating parameters according to the geological detailed survey report were shown in the Table 1 [22]. The rock mass level of intensely weathered granite and intensely weathered gneiss were class V, moderately weathered granite and moderately weathered gneiss are class IV.
3.4. Partial Factor of Load
According to the requirements of <Nuclear power plant seismic design code>, there were two kinds of load on intake tunnel, dead load (permanent action) and seismic load. The selection of partial factor was shown as Table 2:

| Partial factor | Dead load | Seismic load |
|----------------|-----------|--------------|
| SL1            | 1.4       | 1.7          |
| SL2            | 1.0       | 1.0          |

3.5. Input of Seismic Wave on Bed Rock
The input of seismic wave was according to earthquake security evaluation report audited by china seismological bureau when did the dynamic analysis. On the base of the code for seismic design of nuclear power plant, the running safe seismic load SL1 and the ultimate safe seismic load SL2 was imported in three directions, as shown in Figure 6. X direction of seismic was perpendicular to the tunnel axis and horizontal 1; Y direction was along the tunnel axis and horizontal 2; and the Z direction of seismic was vertical. The SL1 lasted 25 seconds. The peak acceleration of SL1 on the horizontal direction was 0.1g, and the vertical direction was 0.08g. The SL2 lasted 25 seconds, and the peak acceleration of it was 0.18g on the horizontal direction, and it was 0.12g the vertical direction.

Figure 6. Seismic Time History Curve of seismic: (a) Horizontal Seismic Wave H1; (b) Horizontal Seismic Wave H2; (c) Vertical Seismic Wave V

4. RESULT AND ANALYSIS

4.1. Tunnel Portal
To monitor the variation of internal force of lining, 16 reference point of internal force were lay out on ring direction, and the result could be used to analyze the transformation law of internal force at different parts of lining of tunnel portal, as shown in Figure 7.

The initial ground stress of tunnel entrance, construction supporting, pressure of wall rock, effect of permanent rock support, internal water pressure, the operation safety earthquake load SL1 and the ultimate safety earthquake load SL2 and the conditions among them, are considered and shown in Table 2. In this table, the internal pressure was adding water on tunnel lining as additional mass, and dynamic water pressure was lining which adds mass.
The change rule of internal force of lining which effected by earthquake wave can be gotten by analysis, as shown in Figure 8 (as reference point 3 of working condition 1 for example), and the amplitude of changes of internal force at the tunnel portal could be gotten. As shown in Figure 9 and Figure 10 and Table 3 [22] (in tunnel 1, for example). The direction of shearing force is positive when isolated body rotates by clockwise.

From Figure 9, max positive moment in the working condition 1 is 131.8 kNm, and the point is at the sidewall of tunnel; max negative moment is 87.62 kNm, and the point is at the bottom of tunnel; as compression member, lining has the max axial force 3135 kN at the base of the tunnel and has the min axial force 1334 kN near the sidewall of the tunnel; max positive shear is 88.57 kN which is at the left wall and the max negative shear is 77.47 kN in the impost. In Figure 10, the change features of the tunnel of working condition 2 is similar as working condition 1 and the peak value of the internal force of SL2 is larger than the peak value considering the running safe seismic load SL1.
Table 3. Different Combination Conditions

| Position   | Working condition | Combination of actions                                                                 |
|------------|-------------------|----------------------------------------------------------------------------------------|
| Tunnel portal | Condition1         | Wall rock pressure + construction support + permanent support + internal tariff water pressure + running safe seismic load |
|            | Condition2         | Wall rock pressure + construction support + permanent support + internal tariff water pressure + ultimate safe seismic load |

Table 4. The Extremum Value of Internal Force in Lining in Different Working Conditions

| Position   | Working condition | The minimum internal force of lining (kNm) | The maximum internal force of lining (kNm) |
|------------|-------------------|-------------------------------------------|-------------------------------------------|
| Tunnel portal | ①                | -87.62                                   | 131.8                                    |
|            | ②                | -86.77                                   | 132.7                                    |

From the comparison of two conditions, the crest value of internal forces in lining are close relatively, and change rule of the internal force are with one accord; the support can reduce part of pressure from the surrounding rock. The support can decrease the value of permanent support role and part of the bending deformation; construction support can undertake part pressure from surrounding rock and decrease the value of internal force of deformation of tunnel and the permanent supporting.

On lining, the seismic weak positions of tunnel portal can be gotten by the transformation law of static load and dynamic load. The monitoring data of 16 points show that the bending moment combined by initial ground stress, stress of wall rock, effect of permanent supporting, internal water
pressure, the operation safety seismic load SL1 and the ultimate safety seismic load SL2 minus the result of bending moment of statical analysis to get the value of increment of bending moment cause by seismic, as shown in Figure 11 and Figure 12.

The conclusion that the change value of bending moment at the monitoring 1, 2, 8, 9, 10 and 16 is larger and the values of the roof and the base of the lining are smaller can be get by entering the combination of two different conditions.

The seismic weak positions of tunnel portal are near the sidewall and these part need to reinforce.

4.2. Tunnel Entrance Section of High Slope
Taking account of stability of backfill slope at tunnel portal, the location of riskiest sliding surface will change in process of dynamic effect, and the minimum safety faction and the corresponding riskiest sliding surface can be located by change the range of slip in area and slip out area. This paper adopted QUAKE/W module and SLOPE/W module of software GeoStudio, and the position of sliding surface and the corresponding safety coefficient curve can be given by Figure 13 and Figure 14 under the impact of seismic wave.

![Figure 13. The Sliding Surface under The Action under of Earthquake](image1)

From the above two figures, we can see that the stability coefficient calculated by dynamic stability analysis after backfill is greater than 1.2. Moreover, stress strain and deformation characteristics of soil slope can be obtained intuitively and the slope is stable under seismic action after backfill.

5. CONCLUSION
In this article, by the feature of the intake tunnel, the seismic response of tunnel portal and high slope near portal of a nuclear power intake tunnel are analyzed, in which a 3D tunnel model is built by software of Flac 3D and 2D finite element model of slope is built by Geo-Slope. Main conclusions are as follows:

1) The results which calculate different seismic load and do dynamic analysis of time history show that: For portal section of tunnel, the seismic structural weak parts can be confirmed, i.e., tunnel sidewall need enhance.

2) By using dynamic finite element method in backfill slope seismic stability computations, it can better reflect the dynamic response rule of hole slope. And the sliding surface and the minimum power factor of safety time-history curves can be obtained based on it. The conclusions and analysis method can be a reference for seismic slope stability analysis of tunnel portal.
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