Seismic Response of Underground Opennings: with an Insight into Siah Bisheh Caverns

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ABSTRACT

Response of underground structures exposed to seismic loading is a concern for designers, especially in large span opening. Siah Bisheh is one of the largest power plant projects in Iran that has three main caverns and is located in Alborz seismo-tectonic region, with high risk of seismic events. Seismic response of these caverns is considered in this study. For this purpose, the result of a probabilistic seismic hazard analysis that has been conducted in this region is used to determine the maximum design earthquake in the studied site. Numerical analyses are performed in three different media: continuum, semi-continuum and discontinuum media using two different software. PHASE2 V.5 software was used for modeling of the first and the second media, while UDEC software was applied for the third media simulation. The obtained results show that the discontinuum modeling, as compared to the continuum modeling, exhibits a good agreement with monitoring data in the static modeling. Furthermore, the wall between the Guard Gate cavern and the Power House cavern would be in the risk of instability.

Keywords:
Seismic response; Cavern; Siah Bisheh; Discontinuum modeling; Numerical modeling

1. Introduction

Although underground excavations are relatively more resistant to dynamic loading than the surface excavations, they are still subjected to failure under impulsive loading induced by explosions, earthquakes and mining-induced seismicity [1]. It is, therefore, widely believed that the design of an underground excavation should take the probable dynamic loading to be experienced in its functional life into account.

Underground power projects play a very important role in meeting the requirements of present day civilization. These days, several underground structures are being built in the highly seismic zones; hence their failure would be disastrous to the human life and property. To avoid any serious damages, it is thus essential that underground structures be designed by checking anticipated earthquake effects [2].

Seismic response of underground opening has been considered in many previous studies. Chen et al presented an efficient numerical method to determine the dynamic response of the unbounded medium-structure system in the basis of refined DSE method [3]. Liu and Song, studied the seismic performance of a subway station in saturated liquefiable soils using fully coupled dynamic Finite Element method. They employed a generalized plasticity model capable of modeling cyclic liquefaction and pressure dependency of soil behavior to simulate the cyclic behavior of the liquefiable soils [4]. Vardakos et al used distinct element modeling for back-analysis of Shimizu tunnel in Japan. They indicated that convergence-confinement method can be adapted to tunneling in jointed rock masses by use of the distinct element method [5].

Ding et al presented a three-dimensional numerical simulation method for large-scale seismic response calculation based on a newly built immersed tunnel in Shanghai [6]. The results of their research provide a global understanding of this immersed tunnel under seismic loading and reveal the weak parts. It also provides relevant data and references for the seismic design of immersed tunnel and flexible joints in the future.

Although numerical modeling has been used to evaluate the response of underground openings in many studies previously, there is not a considerable insight about the difference of various numerical methods in dynamic analysis of underground openings. In this study, numerical analyses are performed in three different media: continuum, semi-continuum and discontinuum media using two different software. In other words, this study tries to provide a global understanding on different response continuum and discontinuum modeling.

2. Seismic Hazard Analysis

Seismic hazard analysis involves the quantitative estimation of ground shaking hazards at particular area. Seismic hazards can be analyzed deterministically as and when a particular earthquake scenario is assumed, or probabilistically, in which uncertainties in earthquake size, location, and time of occurrence are explicitly considered [7].

With three main and active faults like Mosha-Fasham, Taleghan and Kandovan, and based on seismic history, it is observed that the proposed region, in past decades, has experienced quakes with magnitude more than 7 on Richter scale, see Figure (1).

As power plants have a longer life span, the maxim deterministic level criteria are used for hazard analysis. Moreover, a probabilistic seismic hazard analysis method is applied to obtain the maximum design earthquake [8]. Considering the results, horizontal and vertical accelerations are 0.5 and 0.37, respectively. Indeed, seismic response of studied structures is considered using an acceleration time history with a PGA equal to 0.5. During the analysis, for dynamic loading, a proper time history was selected from Avaj Accelerometer Station [9]. The accelerometer was on bedrock, which is suitable for underground structure enclosed in rocks. Thereafter, the related correction of frequency and damping was done, and acceleration time history scaled according to earthquake design level [9]. As shown in Figure (2), the peak ground acceleration of Avaj time history is 0.5 g, and then the selected time history doesn't need to be scaled.

3. Description of Case Study

Iran's Siah Bisheh pump/storage project consists of two concrete face rock fill dams (CFRD), a waterway and a power cavern with a pump/turbine system.

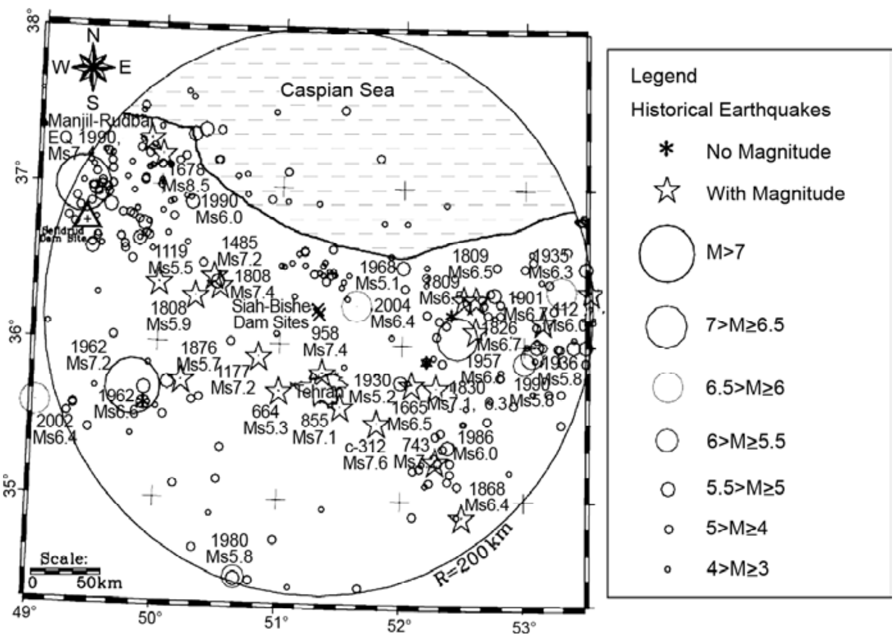

Figure 1. Siah Bisheh pumped storage power plant project and historical earthquakes [8].

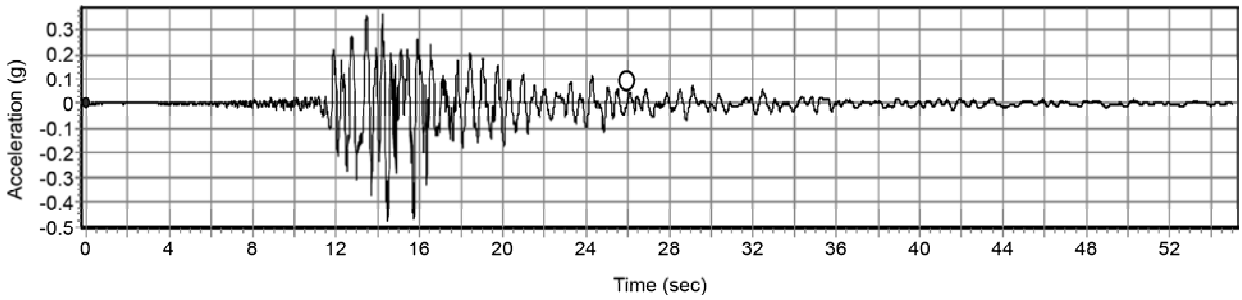

Figure 2. Avaj acceleration record [9].

The project with a generating capacity of 1000 MW of hydroelectricity aims to balance the country's electricity production network [10], which represents a vital link at the time of acute domestic power crisis. Figure (1) highlights topography of the project and historical earthquakes there. The dimension of the caverns is as follows:

Power House Cavern:
Length: 130 m; Width: 22 m; Height: 42

Transformer House Cavern:
Length: 182 m; Width: 13 m; Height: 22

Guard Gate Cavern:
Length: 182 m; Width: 13 m; Height: 22

3.1. Geomechanical Parameters

Rock mass investigation is to create not only some order out of the chaos in site investigation procedures, but also to identify the most significant parameter influencing the behavior of rock mass. Here, to determine the cohesion, friction angle and the rock mass deformation modulus, investigations were conducted. Particularly, to determine the modulus of deformation at several locations, flat jack tests have been conducted. For an in-situ stress, hydraulic jacking has been utilized where the maximum stress was found to be 6 MPa [11]. Since the in-situ stress ratio (K) had not cleared yet, the values 1, 1.1 and 1.5 were chosen for K. A comparison between the static result and instrumentation data indicated that $K = 1$ is the proper one [11]. Further, to evaluate the mechanical properties (elasticity density and modulus, unconfined compressive strength, tensile strength and Poisson's ratio), laboratory tests were also carried out on several rock samples collected from different locations.

The acquired results of the above in-situ and laboratory tests indicate that the correlation coefficients between rock mass and intact rock properties are close to 1.4 [11]; although, for the discontinuum analysis, laboratories values are used. Such a correlation helps in determining the rock mass properties (deformation modulus and cohesion), which could not be firmed by in-situ tests. Based on the assessment of the project consultant engineering, the rock layers are divided into six groups: three related to certain observed rock types and the other three with strong, weak and shear zones represented by the equivalent continuum properties [11]. Geotechnical properties of intact rocks are presented in Table (1). The rock mass exhibits an average unit weight of 2500 kg/m^3.

3.2. Discontinuity Parameters

In the basis of experimental studies on joint rock-like specimen, model studies as well as field observation, it is found that the joint deformation

Table 1. Geotechnical properties of intact rocks [11].

Rock Type	ν	E (GPa)	σ_t (MPa)	C (MPa)	$\Phi(\circ)$	σ_c (MPa)
Quartzite-Sandstone	0.2	15	0.12	1	53	85
Shale - Siltstone	0.25	7.5	0.11	0.68	41	50
Mellafir	0.2	20	0.5	2	58	100
Shear Zone	0.28	5	0.073	0.65	27.3	-
Weak Zone	0.2	15	0	0	30	-
Strong Zone	0.2	16.5	0.122	1.76	55	-

is a critical aspect of the response to the dynamic loading of excavations in jointed rock [12].

As such, the current study also tries to evaluate the frictional resistance of discontinuities in jointed rock masses. The studies indicate that there are five major joint sets in the area and bedding [11]. At the proposed area, the bedding crosses the three caverns and has nearly constant dip and dip direction. Figure (3) depicts the bedding and the joints.

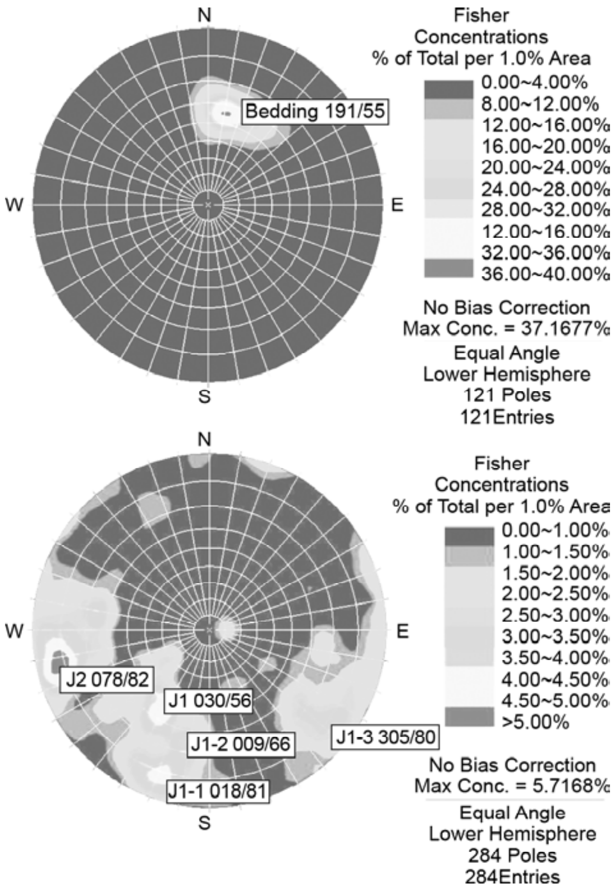

Figure 3. Bedding and joints plot in the proposed area [11].

Following conditions are met once two-dimensional analyses are preferred [13]:

I. Plane strain condition;
II. Geological condition of cross-section has to be general feature of the opening; and
III. Cross section must be close enough to one in strumentation ring so that result could be compared.

It must be noted that the geological condition was very different along the caverns; hence, the typical section could not be selected. The most critical section -as a matter of stability- was chosen at the point of conservativeness. The cross-section was located in chainage 70 of cavern and 90 of transformer. The overall dip for bedding measured as 40°. The joints, in discontinuity model, are derived from geological maps of each cavern and according to their apparent dips inserted in the model; this application leads to wedges, see Figure (4). Here, the joints parallel to the bedding are ignored. In continuum analysis, the bedding layer contact surfaces are treated like joints, see Figure (5).

All joint properties assume like each other which are:

K_n = 20000 MPa/m, K_s = 7692 MPa/m, Tensile Strength = 0, C = 0.025 MPa and Φ = 27°.

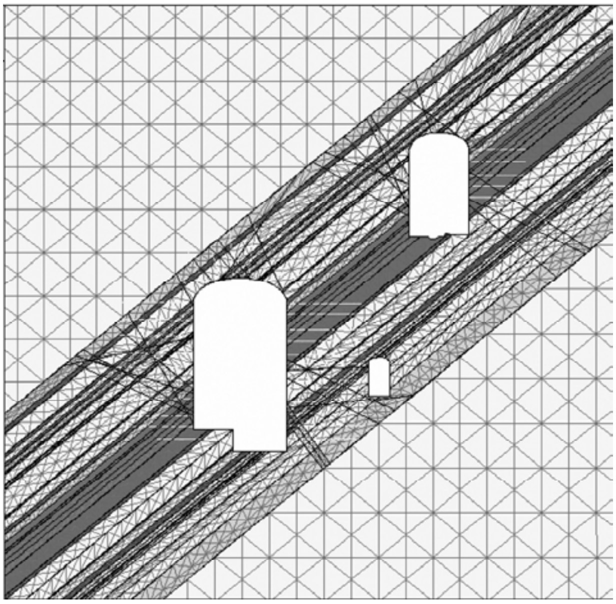

Figure 4. Discreet element model.

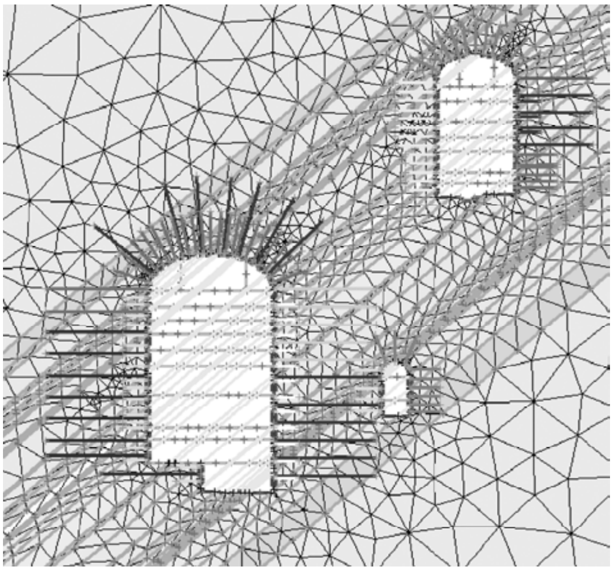

Figure 5. Continuum and semi-continuum model.

4. Rock Mass Behavior

The analysis of rock mass behavior without taking into account the existing discontinuities does not often show the actual behavior of the rock mass. In other words, under applied forces, the presence of cracks and discontinuities has an essential role in the behavior of rock mass [14]. The intact rock may deform elastically, or may undergo significant plastic deformation, causing joints to open, close or slip, resulting in instability of an opening [2]. The current study has resorted to Mohr-Coulomb yield criterion in order to represent elasto-plastic material behavior of rock mass. The Mohr-Coulomb is the simplest model for joint strength and deformation that also includes a shear failure criterion for a rock joint, given by [1]:

$$\tau_o = C + \sigma_n \tan(\phi) \tag{1}$$

where τ_o is the shear strength along the joint, σ_n is the normal stress across the joint, C is the cohesion and ϕ is the friction angle. Once reached, the joint deformation is perfectly plastic. Here, the joint shear response is governed by constant shear stiffness k_s [1]:

$$\Delta\tau = K_s \Delta u_s^e \tag{2}$$

where $\Delta\tau$ is the incremental shear stress and Δu_s^e is an elastic compound of the incremental shear displacement. In the basis of above two equations, $\Delta\tau$ becomes zero reaching to the condition $|\Delta\tau| = \tau_o$, where τ_o is the shear stress on the joint surface. Although, Mohr-Coulomb joint model does not consider joint wear and dilation behavior in its basic form; however, such behavior may be added readily. For example, the dilatation may be restricted until shear stress reaches to the shear strength of the joint, i.e. the joint dilatation starts after the joint begins to deform plastically. Since there is no wear of the joint, the dilatation should remain constant with shear displacement [1]. The dilation angle, in the present research, is considered to be 11 degrees.

5. Rock Support

There are three types of support elements in the caverns: shotcrete, rock bolts and tendons. Shotcrete lining is comprised of "beam" elements, corresponding to the edges of the elements. According to Timoshenko, the beam formulation is used to allow transverse shear deformation effects. The Young's Modulus is 15 GPa and ν is 0.2. The liners are installed up to the final surface after excavation of each stage. The rock bolts are modeled fully bonded. Once axial force exceeds on the bolt element, the axial capacity failure of a fully bonded bolt element occurs in tension. The rock bolts are not pre-stressed. Bolts with 25 mm diameter and 500/550 steel quality are considered for the support design and analysis. Here, each anchor behaves as a single element, and interaction with the element mesh is through five meter of bound length. When the peak capacity exceeds or the bounded area fails, an end-bounded anchor fails in tension as well. Another support element, i.e. tendons can be mono-bar or stranded anchors. Based on their allocation, tendons are pretended differently. Normally, tendons with 47 mm diameter and a steel quality of 900/950 are considered.

6. Numerical Modeling

6.1. Static Analysis

The use of numerical modeling has begun a revolution in the existing methods to study geomechanics. However, before numerical modeling, it is essential to know the type of media ought to model in th course of research. Since, there are five sets of joint and cross bedding in Siah Bisheh region; it is required to reflect on the discontinuum media. Meanwhile, the present study has, altogether, selected three types of media, i.e. continuum, semi-continuum and discontinuum.

PHASE2 V.5 software was used for numerical analysis of the first two media where the local joints which might form wedges were ignored, and bedding was considered alone. In the first type of media, the shear behavior of joints was not taken into account and thus, the joints are completely elastic. In the second type, the joint shear parameters were established with help of "thin element" in PHASE2 [15]. For the third media, UDEC (Universal Distinct Element Code) software was used, and all joints and bedding were considered. Tables (2) and (3) show the execution of static modeling and results of its analysis. Since, the continuum media with elastic behavior of the joints is far away from reality, see Table (3), it was not considered for further analysis

Table 2. Comparison of loads in rock support system in different numerical modeling with instrumentation data (KN).

Load Cell Point	Instrument	Semi-Continuum	Discontinuum	Continuum-Elastic
Tendon in PHC Roof	840	800	830	680
First Tendon between PHC & GGC	1003	800	836	650
Up Steam-Upper One	850	850	920	600
The Lowest Transformer Tendon	950	700	830	500

Table 3. Comparison between displacements in different numerical models and. instrumentation data (mm).

Data Acquisition Point (Displacement)	Instrument	Semi- Continuum	Discontinuum	Continuum-Elastic
PHC Roof	17	21	16.1	8
Up-Stream - Upper Level	18	30	18.38	18
Down-Stream - Upper Level	11	42	15.3	24
Up-Stream - Middle	56	33	48	22
Down-Stream - Middle	45	57	44	36

and concluded that this media is not valid. Based on the results, discontinuum media had the best agreement with instrumentation data, and therefore, it can be concluded as the proper media for the analysis of Siah Bisheh caverns.

6.2. Dynamic Analysis

6.2.1. Semi-Continuum Modeling

PHASE2 software considers dynamic loading as a coefficient of weight no matter in horizontal or vertical direction. The process is quite simple with no damping and boundary condition different from static analysis is required. It can estimate the structural reaction through dynamic loading. Here, the coefficient for both vertical and horizontal loading is equal to 0.3 [16]. Figures (6) and (7) highlight the plastic zones whereas; Figures (8) and (9) make comparison of displacements in static and dynamic models.

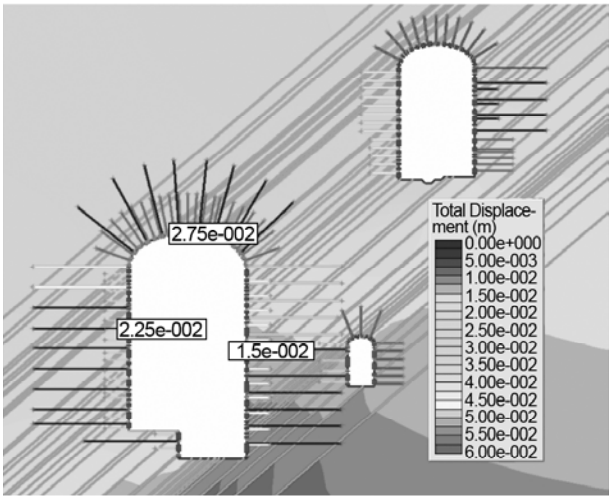

Figure 7. Plastic zone in static analyze (semi-continuum).

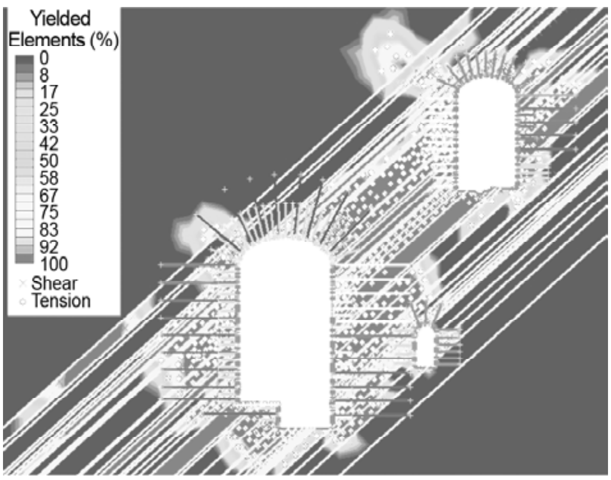

Figure 6. Plastic zone in dynamic analyze (semi-continuum).

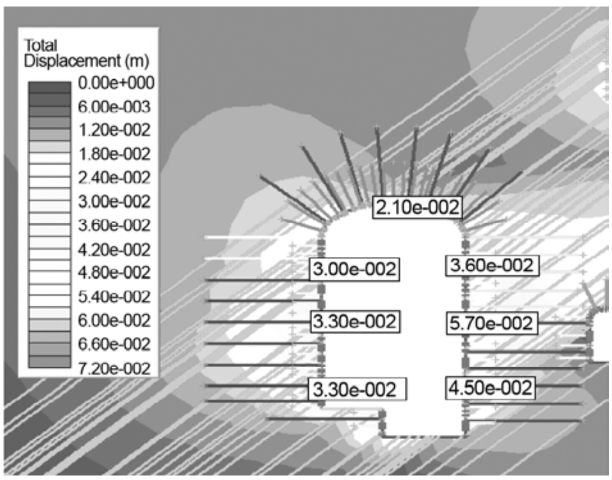

Figure 8. Displacement in dynamic analysis (semi-continuum).

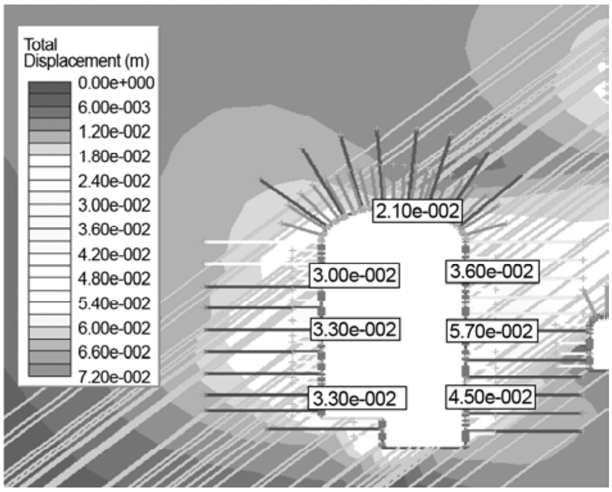

Figure 9. Displacement in static analysis (semi-continuum).

6.2.2. Discontinuum Modeling

For discontinuum analysis, the UDEC software with capability of discrete element modeling has been utilized. This software allows two-dimensional, plane-strain or plane-stress as well as fully dynamic analysis. To solve the full equations of motion, the calculation is based on the explicit finite differential scheme, using real rigid block masses, or lumped grid point masses derived from the real density of surrounding zones. This formulation can be coupled to the structural element model, thereby permitting analysis of rock structure interaction brought about by ground shaking [17]. The dynamic loading applied from the base boundary is in stress form. The velocity is converted to stress time history by equation [13]:

$$\tau = 2(\rho C_s) \times V_s \tag{3}$$

where τ is applied shear stress, V_s is input shear particle velocity, ρ is mass density and C_s is given by [13]:

$$C_s = \sqrt{G/P} \tag{4}$$

In order to the absorb wave and to stop reflection, the viscous boundaries are applied across the model. The two damping modes available in UDEC programs are: Rayleigh damping and Local damping. Rayleigh damping is usually used in the dynamic analysis of structures, to damp the natural oscillation modes of a system. The damping equations, therefore, are expressed in matrix form, i.e. matrix D is used, with components proportional to mass (M) and stiffness (K) matrices [13]:

$$D = \alpha M + \beta K \tag{5}$$

where α and β are the mass-proportional and the stiffness proportional damping constants respectively. While calculating α and β, they obtain values of following equations [13]:

$$\alpha = \zeta_{\min} \times \omega_{\min} \tag{6}$$

$$\beta = \zeta_{\min} / \omega_{\min} \tag{7}$$

where ζ is the critical damping ratio, $\omega_{\min}$ angular frequency of the system, $\zeta_{\min}$ is a combination of predominant frequency of model and input wave [17]. For predominant frequency, the model oscillates with damping zero under its own weight, and thus, the natural frequency (f) 2 Hz is obtained. Since the response spectra for input wave have the same result, so the value 2 Hz is chosen for predominant frequency of input load, Figure (10). To determine $\zeta_{\min}$ [17] by considering its results, the value 0.005 was chosen.

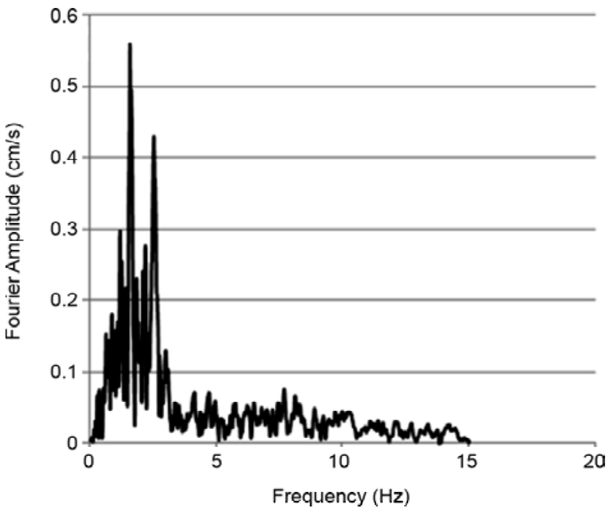

Figure 10. Fourier amplitude spectrum of Avaj time histories.

Finally, α and β are calculated with values 0.0628 and 3.97×10^{-4}, respectively. For faster convergence, the β value, which is related to the stiffness matrix, could be ignored without having erroneous results. Again, for accurate representation of wave transmission through a model, the spatial element size (Δl) must be smaller than approximately one-tenth to one-eighth of the wavelength associated with the highest frequency component of the input wave i.e.:

$$\Delta l \leq \lambda / 10 \tag{8}$$

Here, λ is wave length associated with the highest frequency component that contains appreciable energy. For discontinuum analysis involving rigid blocks, this applies to joint spacing (or block size) as well. In this model, the value $\lambda/10$ is equal to 40 and the largest element size is 12 meters; hence, the wave propagation is secure and now the dynamic load can be applied. However, it must be noted that the dynamic load is applied for two seconds from lower boundary to the shear direction. Figures (11) and (12) highlight the results for displacement and plastic zone while Figure (13) indicates the input stress time history.

Figure (14) depicts peak particle velocity in lining structures. As shown in this figure, the maximum particle velocity is less than 50 cm/s, which means shotcrete does not expose to any serious damage. However, the cohesion parameter for shotcrete should be checked with caution because it may detach from the rock or could pose some problems for tendons. As Figure (14) indicates, compare with static mode, the plastic zone has expanded; however, this does not create any problem for the proposed model. Also, the particle velocity has nothing to do with intact rock.

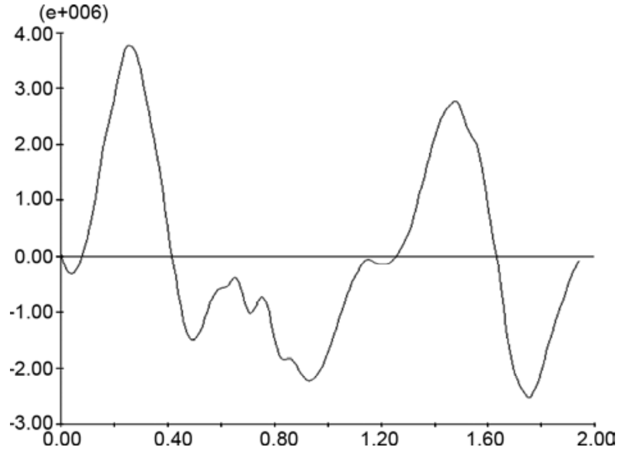

Figure 13. Input stress time history (MPa) into discontinuum model.

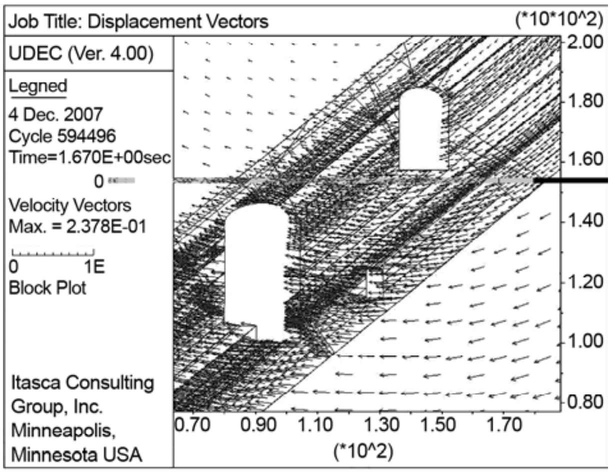

Figure 11. Displacement vector around the opening.

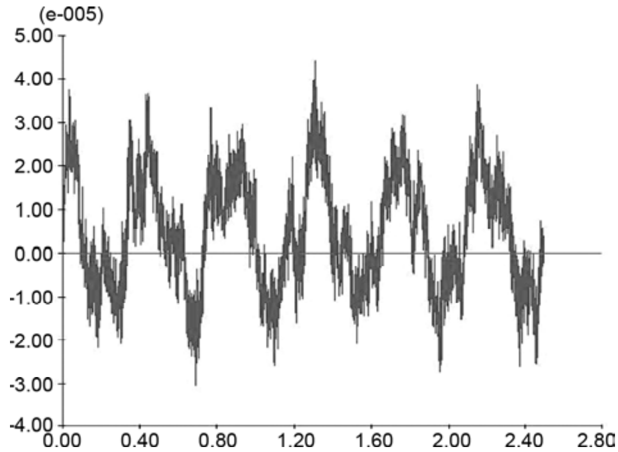

Figure 14. Peak particle velocity in shotcrete.

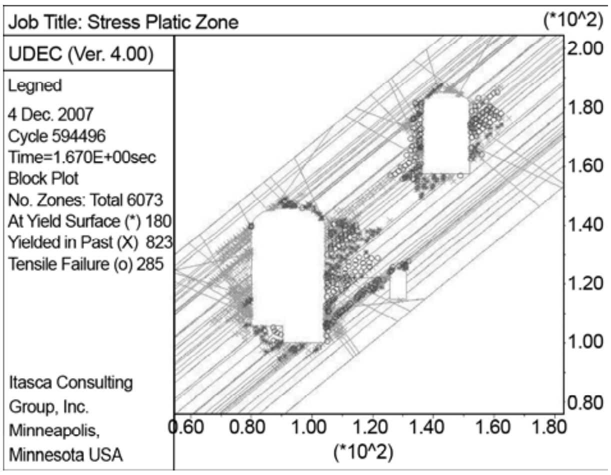

Figure 12. Plastic zones in discontinuum model.

7. Conclusion

Dynamic response of three caverns in Siah Bisheh power plan was considered in this study. The results of numerical analysis indicated that it's vital to check the model response in static mode with continuum and discontinuum methods, before going with the dynamic analysis.

During the course of present study in dynamic mode, no collapse was observed at the roof of Guard Gate cavern and PHC downstream despite the failure of the rock bolts there. The maximum shear displacement of joints seems very low and hence; confirms the stability of openings. For Siah

Bisheh rock mass, the elastic continuum assumption is not valid rather bedding is the main factor of instability. At the proposed site, the most susceptible area for collapsing is the wall between Guard Gate and Power House caverns, and it requires some tendons for an improvement.

In the basis of the monitoring data and result of numerical modeling, it can be inferred that the proposed geomechanical parameter and rock mass constitutive model are close to the reality; however, the discrete element method is the best suited for stability analysis because joints, as compared to bending, have more influence on the instability. This aspect is also visible in the dynamic analysis with discrete method where the displacement is nearly four times more than the pseudo-static analysis.

With regard to plastic zone in dynamic loading, it expanded during both analysis methods. However, the expansion is less in the discrete element method than the semi-continuum method. It is also observed that an extra displacement in the discreet element method is due to joints' movement that is restricted in the semi-continuum method.

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