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Three-Dimensional Modelling and Sensitivity Analysis for the Stability Assessment of Deep Underground Repository

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Abstract

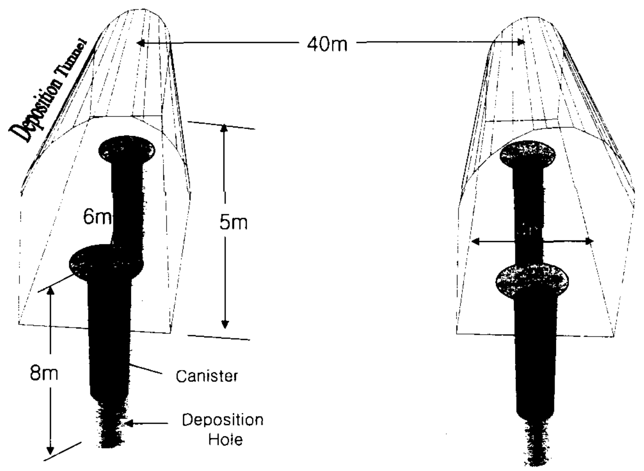
For the mechanical stability assessment of a deep underground high-level waste repository, computer simulations using FLAC3D were carried out and important parameters including stress ratio, depth, tunnel size, joint spacing, and joint properties were chosen from sensitivity analysis. The main effect as well as the interaction effect between the important parameters could be investigated effectively using fractional factorial design. In order to analyze the stability of the disposal tunnel and deposition hole in a discontinuous rock mass, different modelings were performed under different conditions using 3DEC and the influence of joint distribution and properties, rock properties, and stress ratio could be determined. From the three-dimensional modelings, it was concluded that the conceptual repository design was mechanically stable even in a discontinuous rock mass.

Key Words : high-level waste, underground repository, sensitivity analysis, mechanical stability, fractional factorial design, FLAC3D, 3DEC

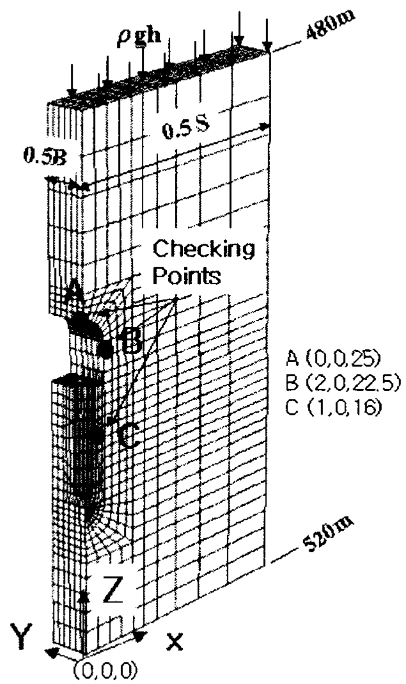
1. Introduction

Deep geological disposal is generally accepted as the most promising method for the permanent disposal of high-level waste. For the safe disposal of nuclear waste in the deep underground, the repository should be designed to be mechanically stable during the construction, operation, and monitoring periods. To confirm whether an underground repository design is mechanically stable or not, it is necessary to evaluate the stability of the repository under different conditions. There are many parameters, which can affect the stability. If we know which parameters are more important than others, we can spend more time and effort on the important parameters from the laboratory and/or in situ tests. In this study, a sensitivity analysis was carried out for the design-parameters as well as site-parameters to determine the important parameters from the mechanical stability point of view. It was assumed that the repository is constructed in a deep underground granite body, which is considered as one of the host rocks in Korea.

In this study, the three-dimensional finite difference code, FLAC3D, was used to assess the



(a) Disposal tunnel and deposition hole



(b) Boundary conditions and checking points

Fig. 1. Repository Layout and Model Mesh for FLAC3D Analysis

influence of the parameters. From the modeling for different conditions, important parameters could be chosen and also the interaction effect between those parameters could be investigated using fractional factorial design. Furthermore, mechanical analysis for a discontinuous rock mass was carried out to investigate the stability of the conceptual repository design in a discontinuous rock mass using the three-dimensional distinct element code, 3DEC.

2. Selection of Important Parameters from Mechanical Analysis

2.1. Mechanical Analysis Using FLAC3D

The underground nuclear waste repository is supposed to be located at 500 meters below surface in crystalline granite. The deposition holes with a diameter of 2 m are excavated vertically with spacing of 6 m in the tunnel floor. Figure 1 shows the dimensions of deposal tunnels and deposition holes.

Since the properties of rock mass in the deep underground were quite site dependent, the upper and lower bounds of each parameter need to be defined. In this study, the upper and lower bound values of the mechanical properties of granite were determined from literature review [1,2] and listed in Table 1. Granite is assumed to be isotropic homogeneous and a Mohr-Coulomb

Table 1. Material Properties and Their Range

	Parameter	Unit	Min.	Mean	Max.
Variables	Elastic modulus (E)	GPa	50	60	70
	Poisson' s ratio (ν)		0.2	0.25	0.3
	Density (ρ)	Kg/m^3	2600	2700	2800
	Stress ratio (σ_h/σ_v)(K)	0.5	1.0	2.0	
	Friction angle (ϕ)	Degree	20	30	40
	Cohesion and tensile strength	MPa	10	15	20
	Repository depth (Z)	m	300	500	700
	Tunnel width (W)	m	-	4	5
	Tunnel height (H)	m	4.5	5	5.5
	Tunnel spacing (S)	m	30	40	50
	Borehole spacing (B)	m	5	6	7
	Joint spacing (J)	m	0.2	0.6	1.0
	Joint stiffness (Kn&Ks)	GPa/m	50 5	100 10	150 15
Constants	Model size	m		40	
	Borehole diameter	m		2	

plastic material. In order to consider the influence of joints in the model, the following equations, developed by Fossum [3] for calculating the bulk modulus, Km, and shear modulus, Gm, of a randomly jointed rock mass, were used.

$$K_m = \frac{1}{9}E\left[\frac{3(1+\nu)sk_n + 2E}{(1+\nu)(1-2\nu)sk_n + (1-\nu)E}\right] \tag{1}$$

$$G_m = \frac{1}{30}\left[\frac{E}{(1+\nu)}\,\frac{9(1+\nu)(1-2\nu)sk_n + (7-5\nu)E}{(1+\nu)(1-2\nu)sk_n + (1-\nu)E}\right] + \frac{2}{5}\,\frac{EsK_s}{2(1+\nu)sk_s + E} \tag{2}$$

where, ν is Poisson' s ratio of intact rock, E is the elastic modulus of intact rock, s is joint spacing, and Kn and Ks are the normal and shear stiffness of joints. For the case with mean values listed in Table 1, Km and Gm are calculated as 51 GPa and 15 GPa, respectively.

2.2. Selection of Important Parameters

For comparison, the displacements, maximum principal stresses, and von-mises stresses at three different locations in the roof, wall, and borehole center were checked. Figure 1.b shows the three checking locations, the model mesh, and the boundary conditions. The coordinates of the three checking locations are from the case of tunnel dimension of 4m×5m. The vertical stress is assumed to be the same as the overburden pressure, which can be calculated by multiplying density (ρ), gravity acceleration (g), and depth (z). A preliminary sensitivity analysis with single parameter variation were carried out for the parameters and the results are listed in Table 2. In the sentitivity analysis, each parameter was changed from

Table 2. Variation of the Results when Each Parameter Changes from the Minimum to the Maximum Value

Variables	Location	Variation Disp. ($\times 10^{-1}$ mm)	σ_1 (MPa)	Von-Mises (MPa)
E	Roof	-1.8	0.0	0.1
	Wall	-2.7	0.0	-0.1
	Borehole	-0.8	0.1	0.1
υ	Roof	0.7	0.0	-0.1
	Wall	0.5	-0.5	-0.2
	Borehole	0.2	-0.1	-0.1
ρ	Roof	0.4	1.9	1.4
	Wall	0.6	1.2	0.9
	Borehole	0.2	1.9	1.5
K	Roof	-3.5	43.3	33.1
	Wall	12.2	1.0	0.3
	Borehole	2.5	36.8	29.9
ϕ	Roof	0.0	0.0	0.0
	Wall	0.0	0.0	0.0
	Borehole	0.0	0.0	0.0
C	Roof	0.0	0.0	0.0
	Wall	0.0	0.0	0.0
	Borehole	0.0	0.0	0.0
Z	Roof	4.5	20.4	15.0
	Wall	6.4	12.1	10.1
	Borehole	1.9	20.8	15.9
W	Roof	1.5	-2.3	-0.9
	Wall	0.0	2.7	3.1
	Borehole	0.4	0.5	0.5
H	Roof	0.0	1.5	1.1
	Wall	1.5	1.7	0.7
	Borehole	0.0	0.8	0.6
S	Roof	-0.5	0.0	0.0
	Wall	-0.2	-0.5	-0.4
	Borehole	-0.1	-0.3	-0.3
B	Roof	-0.1	-0.2	-0.1
	Wall	-0.1	0.0	0.1
	Borehole	0.3	-1.4	-1.2
J	Roof	-1.6	0.0	0.0
	Wall	-2.5	-0.2	0.0
	Borehole	-0.7	0.0	0.0
Kn &Ks	Roof	-1.1	0.0	-0.1
	Wall	-1.8	0.0	0.0
	Borehole	-0.5	0.0	-0.1

minimum to maximum values while mean values were used for the other parameters. From Table 2 the following conclusions could be drawn:

(a) Elastic modulus : With a decrease in E from 60 GPa to 50 GPa, the displacement increases about 20 %. In an elastic model, the variation of elastic modulus does not change the stress distribution. The small variation of stress values with variation of elastic modulus is a kind of calculation fluctuation due to various reasons such as the difference of iteration steps until getting equilibrium conditions in the elasto-plastic model. In the case of a tunnel wall, the influence of E is 0.013 mm/GPa while it is 0.004 mm/GPa in the deposition hole.

(b) Poisson' s ratio : When Poisson' s ratio increases from 0.2 to 0.3, there is about a 2 % variation on the maximum principal stress and von-mises stress and a 4% increase of displacement.

(c) Density : With the variation of rock density from 2600 to 2800 kg/m^3, maximum principal stress, von-mises stress, and displacement increased about 4 % due to the increase of in situ stress.

(d) Stress ratio : The stress ratio K of 2 means that horizontal stress is twice vertical stress. Similarly, when K is 0.5, horizontal stress is half of vertical stress. With an increase of K from 1 to 2, the maximum principal stress on the tunnel wall increases from 15 MPa to 19 MPa. Interestingly, the maximum principal stress on the tunnel wall also increases to 18 MPa when K decreases from 1 to 0.5. The influence of K on the displacements is shown in Figure 2. With an increase of K from 1 to 2, the displacement in the tunnel wall increases about 85%. The roof displacement decreases with an increase of K from 1 to 2, because of the confining effect by the increased horizontal stress. Since the stress ratio is quite site specific

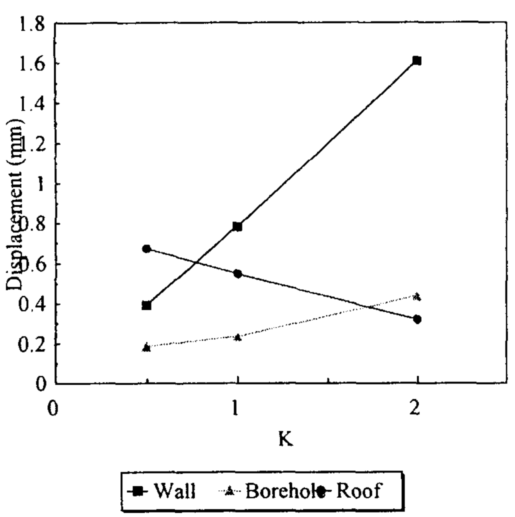


Fig. 2. Influence of Stress Ratio on Displacement

and usually shows serious fluctuation, it is highly recommended to determine the in situ stresses carefully and use them for the site selection as well as for the design, construction, and operation of the repository.

(e) Friction angle and rock strength : In the modeling with intact rock strengths, a plastic zone was not developed around the tunnel and deposition hole and thus it was not possible to observe the influence of the friction angle and rock strength. It should, however, be kept in mind that in situ rock strength is normally much lower than intact rock strength because of scale effect, saturation effect, and time effect. If in situ rock properties instead of intact rock properties are used, a plastic zone would be developed and the influence of friction angle and rock strength could be observed.

(f) Tunnel spacing : The variation of displacement at the checking points was only about 2%-5%, when the tunnel spacing increases from 30m to 50m. The variations of maximum principal stresses and von-mises stresses were also small. From the results, it was possible to conclude that the mechanical stability of the tunnel would not be damaged with the decrease of tunnel spacing from 50m to 30m.

(g) Borehole spacing : The increase on the deposition hole spacing reduces the maximum principal stress in the deposition hole. However, there is no significant influence of hole spacing on the displacement and stress distribution in the roof and wall.

(h) Joint spacing : The influence of joint spacing could be determined by changing it from 0.2m to 1m while keeping the joint stiffness Kn and Ks as constant as 100 GPa/m and 10 GPa/m, respectively. When the joint spacing is 0.2m, the displacement increases about 78% compared to the case of homogeneous rock. The influence of joint spacing on stress distribution could not be observed. This is due to that the influence of joints was indirectly implemented by adjusting elastic modulus, which cannot influence on the stress. If the joints are directly included in the model, the influence of joint spacing would be more significant.

(i) Joint stiffness : When Kn and Ks are are 50 GPa/m and 5 GPa/m, respectively, with joint spacing of 0.6m, the displacements around the tunnel and deposition hole are about 70 % higher than those for homogeneous rock.

(j) Opening width : When the opening width increases 25% from 4 m to 5 m, the stresses in the wall increase about 2.7 MPa. In contrast, the stresses in the roof decrease about 2.3 MPa. The displacements in the roof and deposition hole increase 18% - 27%, but that in the tunnel wall does not show any change.

(k) Opening height : With an increase of opening height from 5 m to 5.5 m, the maximum principal stress and displacement in the wall increase 17% and 6 %, respectively, but their

Table 3. Parameters Used for the Fractional Factorial Design

Parameter	a	b	c	d	e	f	g
	K	depth(m)	tunnel size (m)	Joint spacing(m)	Kn & Ks (GPa/m)	Friction angle(°)	Rock strength (MPa)
Max.	0.5	300	4 & 5	0.2	50 & 5	20	5
Min.	2	700	6 & 7	1	150 & 15	40	10

variation in the roof and deposition hole is not significant.

(1) Depth : Modeling was carried out for the cases with different repository depths of 300m, 500m, and 700m. When the repository depth increases 40% from 500m to 700m, the stress level and displacement increase about 40%. Similarly the stress and displacement decrease about 40% when the repository depth decreases from 500m to 300m. Such a linear relationship between repository depth and stress and displacement is due to the fact that there is no development of a plastic zone. From the result, it could be concluded that the variation of stress and displacement can be accurately estimated from repository depth, if a plastic zone is not developed in the rock mass.

3. Investigation of the Influence of Important Parameters

3.1. Fractional Factorial Design

From the preliminary analysis with a single parameter variation, the parameters listed in Table 3 were chosen for further sensitivity study, which is for deriving the influence of each parameter as well as the interactions between them. For effective modeling, fractional factorial design, which is widely used in experiments involving several parameters where it is necessary to study

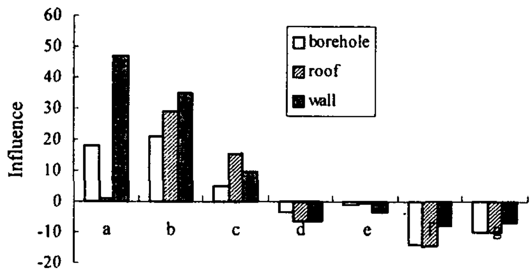


Fig. 3. Influence of the Parameters on Displacemen

the interaction effect of the parameters on a response, was used.

As the number of parameters in a factorial design increase, the number of runs required for a complete replicate of the design rapidly increases. For instance, when there are 7 parameters with 2 levels each, a complete replicate of the 2^7 design requires 128 runs. If it can be reasonably assumed that certain high-order interactions are negligible, then only a fraction of the complete factorial experiment is required [4]. In this study, 2^{7-2} (= 32) were runs instead of 128 with the assumption that the higher-order interactions of more than a second are negligible.

As mentioned earlier, there is no development of a plastic zone around the underground excavations, when intact rock strengths are used. For the fractional factorial design, the rock strength was reduced to half to investigate the influence of the parameters under the condition of plastic zone development. Such an assumption is

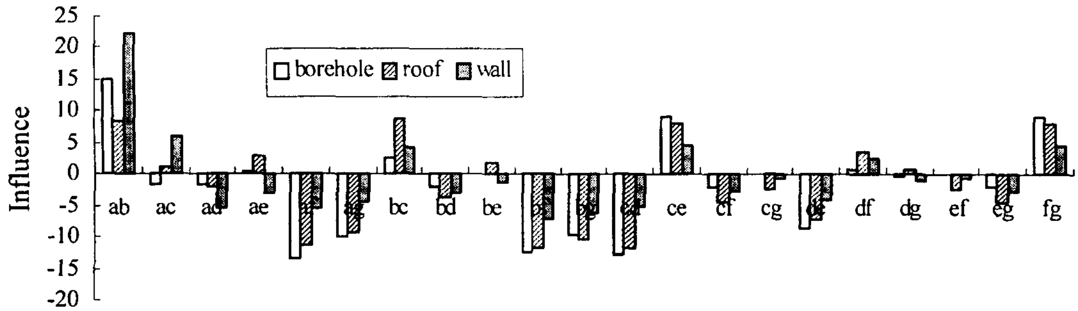


Fig. 4. Interaction Between the Parameters on Displacement

reasonable because in situ rock strength is much lower than intact rock strength.

3.2. Discussion of the Analysis

3.2.1. Influence of Parameters on Displacement

The influence of each parameter on the displacements in the borehole, roof, and wall was compared by plotting, as shown in Figure 3. In the figure, positive influence means that the displacement increases with an increase of the parameter. For instance, the influence of the parameter b, which represents depth, is positive for all displacements. This means that the displacements increase with increase of depth. As shown in Figure 3, the influence of parameters a, b, and c are positive while it is negative for f and g, which represent rock strength and friction angle. The general results from the preliminary analysis using single parameter variation and the analysis based on fractional factorial design are more or less the same.

Figure 4 shows the two-factor interactions between the parameters. Positive influence means the interaction relationship is proportional while negative influence represents reverse proportional. The interaction effect between the parameters a and b is strongly positive. This means the influence of parameter a increases with an increase of parameter b. From this it can be concluded that the influence of the stress ratio on displacement especially in the wall increases with an increase of depth. The other interaction effects are insignificant compared to the interaction between the parameters a and b.

3.2.2. Influence on Stress

Figure 5 shows the influence of the parameters on the maximum principal stresses at the checking points. The influence of parameter a is the strongest. When the stress ratio increases from 0.5 to 2, the maximum principal stresses in the roof and borehole increase significantly while the stress in the wall decreases slightly. It is also possible to conclude that the increase of rock strength results in the increase of the maximum principal stresses at the checking points. This can be explained by the fact that the maximum principal stress decreases with the development of a plastic zone, which is definitely dependent on rock strength.

Figure 6 shows the two-factor interactions of the parameters on the maximum principal stresses. The interaction effect between the parameters a and f is the strongest which means that the influence of stress ratio on the maximum principal

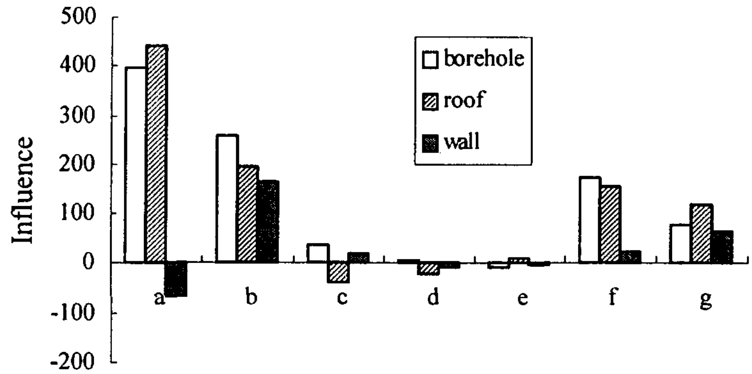


Fig. 5. Influence of the Parameters on Stress

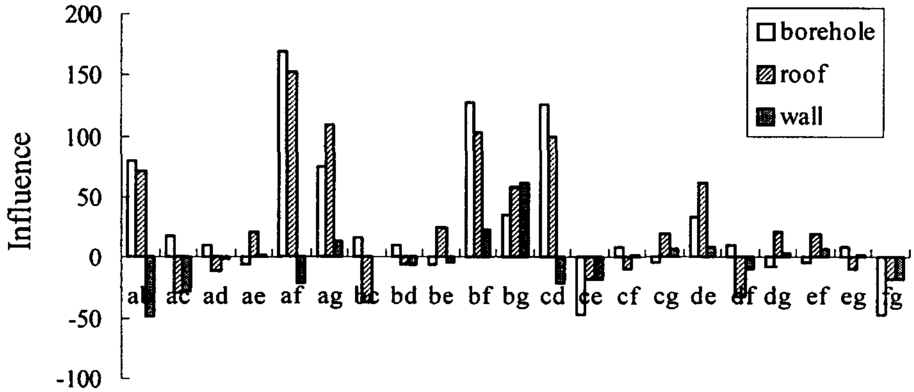


Fig. 6. Interaction Between the Parameters on Principal Stress

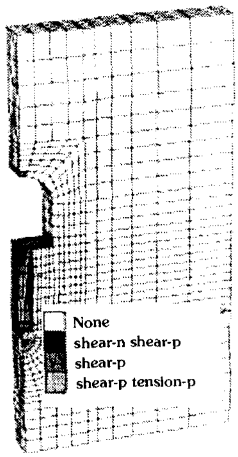
stresses in the roof and borehole increases with an increase of friction angle.

3.2.3. Influence on Failure Pattern

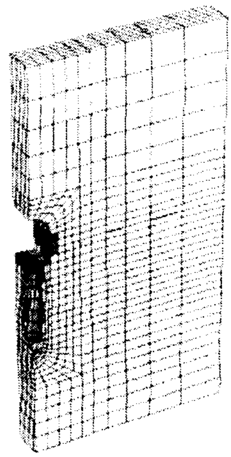
For each running, the development of a plastic zone was checked and classified into 4 categories as shown in Figure 7. In order to evaluate the influence of each parameter on the development of a plastic zone quantitatively, points from 0 to 4 were given depending on the size of the plastic zone. If there was no plastic zone, point 0 was given while point 4 was given when the plastic zone was developed deep into the rock mass as shown in Figure 7.d.

The influence of each parameter on the development of a plastic zone was plotted in Figure 8. The parameters a and b are strongly positive influences, while the parameters f and g showed strongly negative influences.

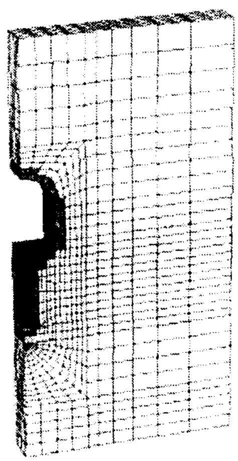
Figure 9 shows the interaction effect of the parameters on the development of plastic zone. The interaction effect of the parameters d and e is the strongest which means that the influence of joint stiffness on the development of plastic zone increases with an increase of joint spacing.



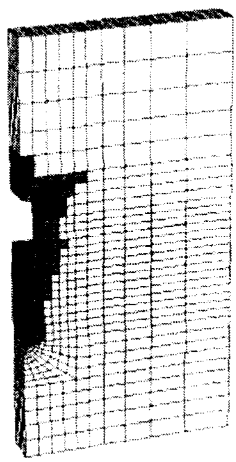
(a) A few plastic zones(1point)



(b) Several plastic zones(2point)



(c) Plastic zone all around the tunnel and borehole(3point)



(d) Plastic zone deep into the rock(4point)

Figure 7. Description of the Four Cases of Plastic Zone Development

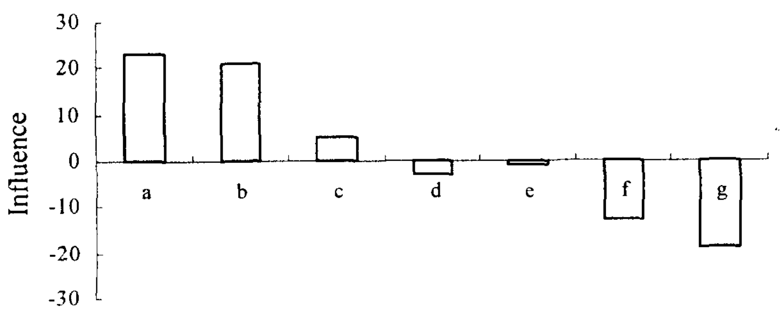


Figure 8. Influence of the Parameters on the Development of Plastic Zone

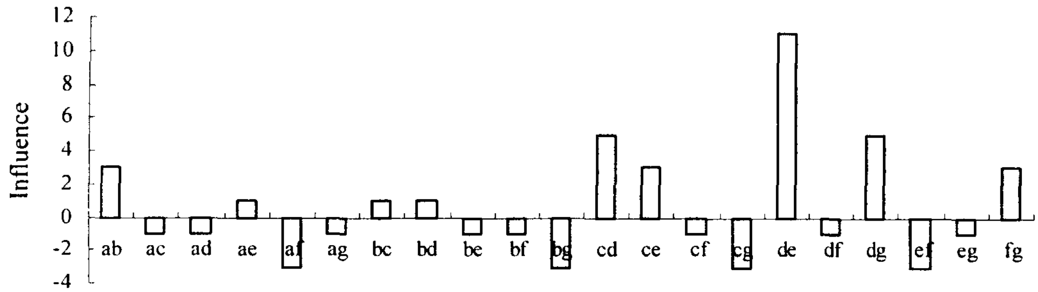


Figure 9. Interaction Between the Parameters on the Development of Plastic Zone

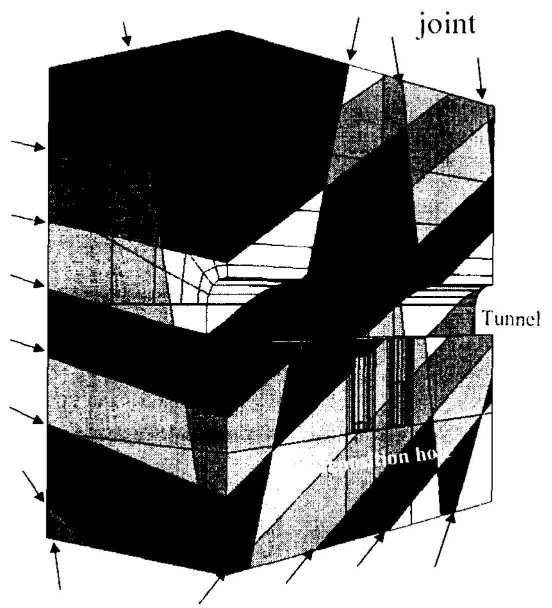


Figure 10. Model Mesh for 3DEC Analysis

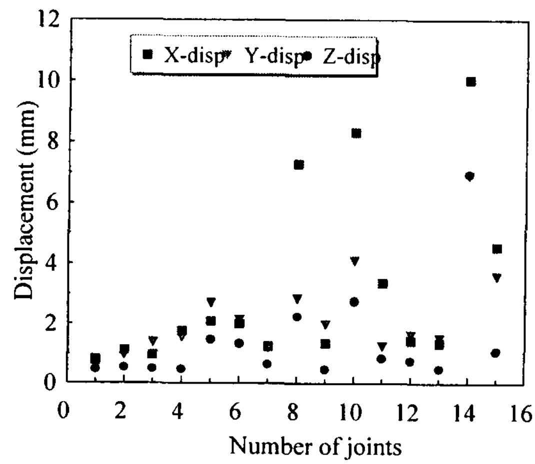


Fig. 11. Influence of Joint Number on the Maximum Displacement

4. Mechanical Stability Analysis for Discontinuous Rock Using 3DEC

3DEC is a three-dimensional numerical program based on distinct element method for discontinum modeling and was developed by Itasca Consulting Company. In the code, discontinuities are treated as boundary conditions between blocks and large displacements along discontinuities and rotation of blocks are allowed [5]. Because it is based on the Lagrangian calculation scheme, 3DEC is well-suited to model the large movements and deformations of a blocky rock mass. Since 3DEC can model discontinuities such as joint, fault, and fracture, it was applied to various areas including civil and mining engineering as well as waste repository projects and army projects. In this study, the following various conditions were considered to investigate the mechanical stability of an underground repository constructed in a discontinuous rock mass as shown in Figure 11.

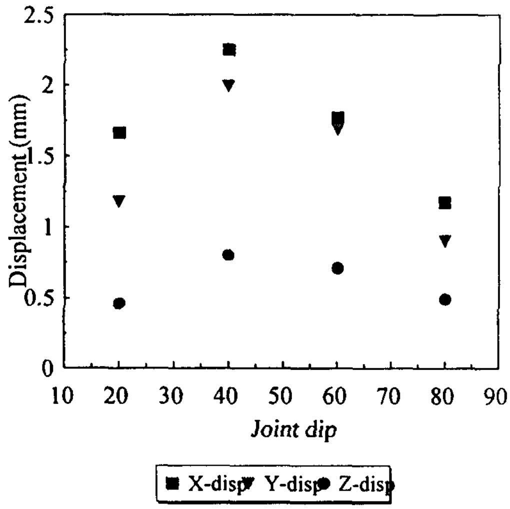


Fig. 12. Influence of Joint Dip on the Maximum Displacement

a. Number of joints in the model : In order to check the influence of joint number in a model, the number of joints was changed from 1 to 15. For each case, the dip and dip direction of the joints were selected randomly. Figure 11 shows the variation of X, Y, and Z maximum displacements in the model with an increase of joint number. It was found that the maximum displacement did not show a significant increase with an increase of joint number until the joint number is 7. The maximum displacement along the X axis varies more significantly than the others. The maximum displacement along the tunnel direction, Z direction, was not seriously influenced by the joint number. The highest maximum displacement of 10 mm, which is still insignificant compared to the tunnel size, was recorded when there are 14 joints in the model. We must, however, keep in mind that the displacements will increase continuously with an increase of in joint number, as shown in Figure 11.

b. Joint dip : To observe the influence of joint dip, different joint dips ranging from 20 degree to 80 were used. In the cases, joint locations and dip directions were chosen randomly. Figure 12 shows the variation of maximum displacement with the variation of dip angle of the joints. When the joint dip is 40 degrees, the displacements are the highest. The smallest displacement from the case of 80 degrees can be explained with the lower possibility of intersecting the tunnel and deposition hole.

c. Size of deposition tunnel : Different tunnel sizes of 4m × 5m, 5m × 6m, and 6m × 7m in continuous rock as well as discontinuous rock were modelled to investigate the influence of tunnel size on the mechanical stability of the repository. For discontinuous rock models, 5 joints were included. Figures 13. a and b show the influence of tunnel size on the maximum displacements around the tunnel in continuous rock and discontinuous rock. The displacements in discontinuous rock are larger than those in continuous rock. When the tunnel size increases from 4mx5m to 6mx7m in discontinuous rock, the amount of displacement change is more than the cases for continuous rock. This implies that the influence of tunnel size is more significant in discontinuous rock than in continuous rock.

d. Properties of joints : The influence of normal stiffness (Kn) and shear stiffness (Ks), which are usually considered as important joint properties, was analyzed. When Kn was increased from 50 GPa/m to 150 GPa/m, while Ks was kept constant as 10 GPa/m, the maximum Y displacement decreases from 4.5 mm to 4.1 mm. When Ks was increased from 5 GPa/m to 15 GPa/m, the displacement change was not significant. Even though the stiffness increased 3 times, the displacement changed only about 10 %.

e. Rock mass properties : Figure 14 shows the principal stress distribution at the locations where the factor of safety is less than 5 in the rock masses with different RMR values of 65

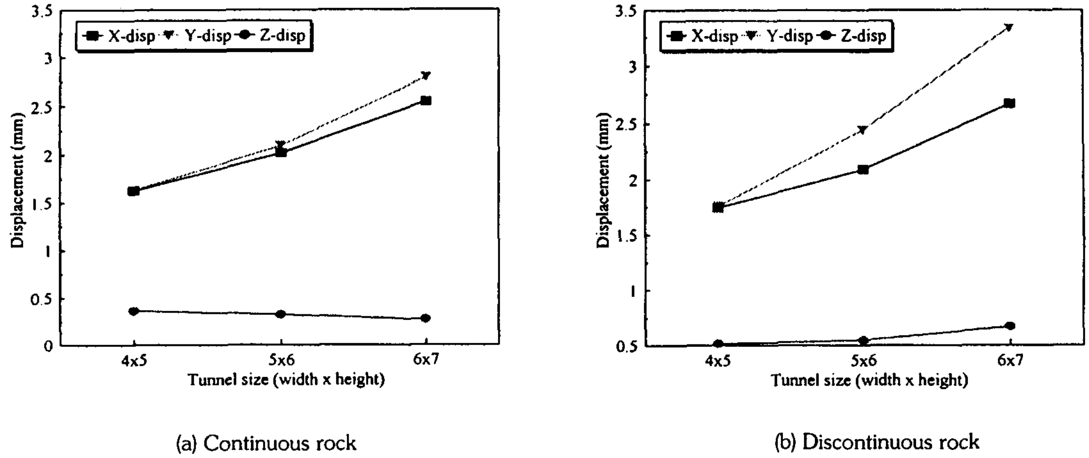


Fig. 13. Influence of Deposition Tunnel Size on the Maximum Displacement

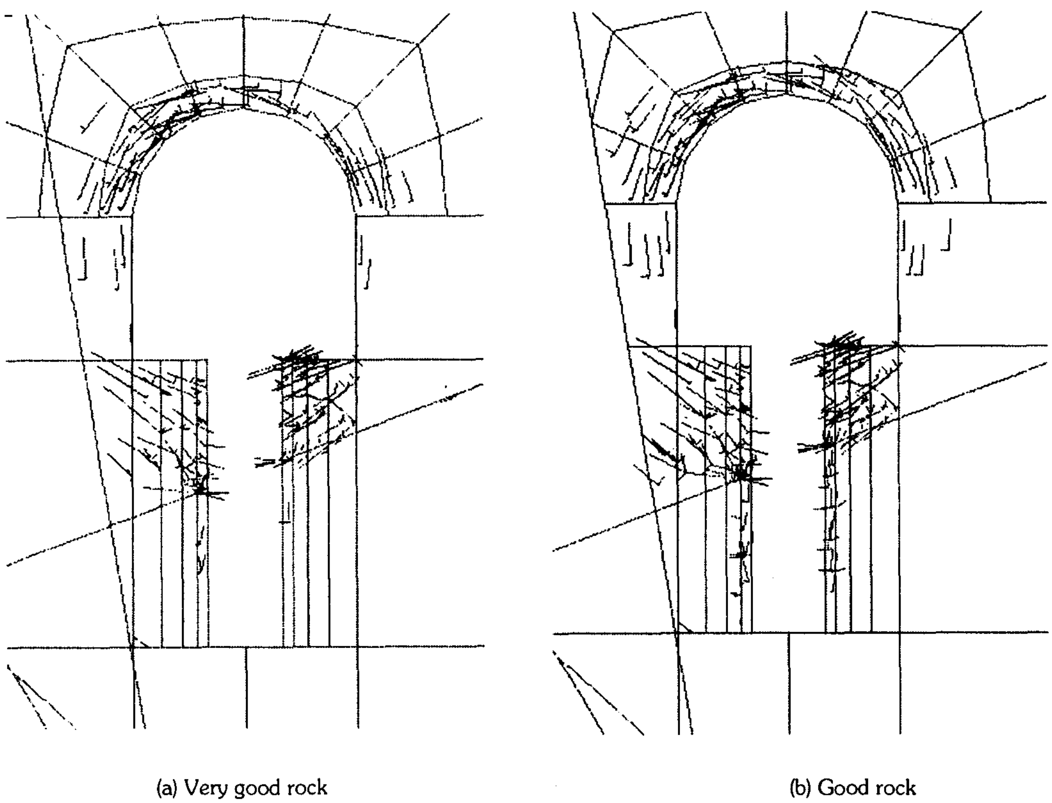


Fig. 14. Distribution of Factor of Safety Around the Tunnel and Deposition Hole in Different Rock Masses (Factor of safety < 5.0)

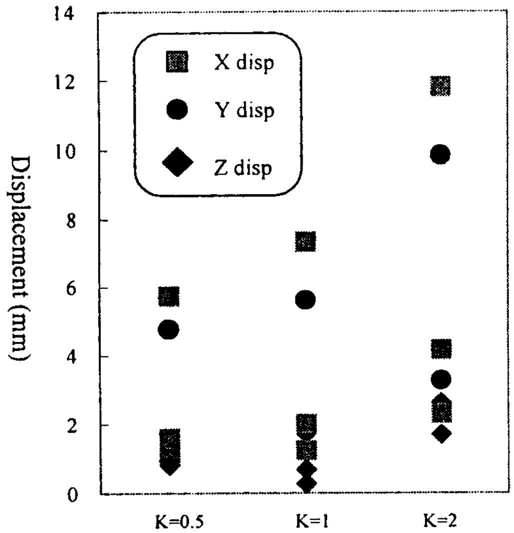


Fig. 15. Influence of Stress Ratio on Displacement

and 85. In both cases, uniaxial compression strength was assumed to be 110 MPa. RMR is a rock mass classification method and is widely used for evaluating rock mass quality. If RMR is 65, it is classified as good rock. It is very good rock if RMR is 85. Compared to the case of very good rock, the possible plastic zone in good rock is a little wider. The plastic zone is expected to be developed in the tunnel roof and especially close to the joints. In the deposition hole, the plastic zone is expected to be concentrated at above the joints intersecting the deposition hole.

f. Stress ratio : To evaluate the influence of stress ratio, K, which represents the ratio of horizontal to vertical stress, was changed from 0.5 to 2 in the model with a 6m×7m tunnel and 5 joints. For each case, modeling was carried out three times with different joint distributions. Figure 15 shows the variation of the maximum displacement with the variation of K. When K is 2, the displacement is almost twice the case of when K equals 1. However, there is no significant change in displacement when the horizontal stress became half. This might be due to the increase of deviatoric stresses, which depends on the difference between the maximum and minimum principal stresses.

5. Conclusions

Seven important parameters for the mechanical stability of an underground repository -- stress ratio, repository depth, tunnel size, joint spacing, joint properties, friction angle, and rock strength -- were chosen from the single parameter variation modeling using FLAC3D. For the selected seven parameters, fractional factorial design was applied for a detailed investigation of the influence of the parameters as well as the interactions between them. In order to analyze the stability of the tunnel and deposition hole in a discontinuous rock mass, a number of modelings were carried out using 3DEC. From the modelings, the influence of joint distribution and properties, rock properties, and stress ratio could be investigated.

From the sensitivity analysis using FLAC3D and 3DEC, the following conclusions could be drawn:

a. The influence of the in situ stress ratio and repository depth was found to be the most important. It was found that elastic modulus, Poisson' s ratio, rock density, borehole spacing, and tunnel spacing were not important parameters from the mechanical stability point of view. It was possible to see the influence of friction angle and rock strength, only when in situ rock strength was used.

b. When the stress ratio increased from 1 to 2, which means the horizontal stress becomes double, the overall stability of the repository was damaged. In the opposite case, when K was reduced to 0.5, the stability in the roof and borehole was improved, but the tunnel wall became less stable. From the 3DEC modelings, it was found that the displacements around the excavation increase when K increases from 1 to 2. It is, however, not possible to observe the influence of K on displacement when K decreases from 1 to 0.5. Such a result can be explained with the fact that deviatoric stress increases with the variation of K from 1 to 0.5 or 2.

c. The linear relationship between the repository depth and stress and displacement was found. When the repository depth increases 40% from 500 m to 700, the displacement and stress also increases 40%.

d. Two-factor interactions between the parameters could be effectively investigated using fractional factorial design. For example, it was possible to observe that the influence of stress ratio on the displacements especially in the wall increases with an increase of depth.

e. A strongly positive influence of the in situ stress ratio and depth on the development of plastic zone was found, while strongly negative influence was found for friction angle and rock strength, as expected.

f. Since the influence of tunnel size was found to be more significant in discontinuous rock than in continuous rock, it was recommended to consider the rock and joint conditions around the repository to determine the tunnel size.

g. If the rock mass can be classified as good quality rock, the locations, in which the factor of safety is less than 5, are distributed only close to the underground excavation and especially where the joints intersect the excavation surface. From the result that the factors of safety in the zones more than 1 m deep into the rock mass are over 5, it could be concluded that the conceptual repository design is mechanically safe even in discontinuous rock. Since the influence of discontinuities would be increased with an increase of joints, further research for investigating the relationships between the parameters and joint number is recommended to confirm the above conclusions

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