Some Considerations on the Appropriate Dimension in the Numerical Analysis of Geoengineering Structures

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Abstract

The selection of the appropriate dimension in numerical analyses of geo-engineering structures is always important issue. The first author realized the importance of this issue when he was doing some numerical analyses of an advancing tunnel in 1986, during which the data-preparation and visualization were extremely cumbersome. Although the memory size and processing speed of computers increased and pre-post processing of computational results become more convenient and less laboring since then, it is still a major issue how to select the appropriate dimension in numerical analyses of structures. In this article, the authors address this issue and compare several hypothetical and actual case history examples involving tunnel face advance, man-made and natural geoengineering structures

Keywords: Geoengineering, structures, dimension, numerical analysis, analytical.

Introduction

How to select the appropriate dimension in numerical analyses of geo-engineering structures is always important issue. Every structure is three-dimensional in physical space. If time is considered, the problem becomes four-dimensional. When the first author did numerical analysis of an advancing tunnel in 1986, the data-preparation and visualization were extremely difficult at that time (Aydan et al. 1988). Furthermore, the memory size and computation speed were also severe problems. In 1986, the memory capacity of the super computer of Nagoya University was only 10 GB. The first author vividly remembers that he was asked by the computation center to have hardcopy outputs of the result of computation with a memory size of 2.5 GB and delete the computed output data files, immediately.

The present tiny notebook computers have now storage capacity of several TBs. Although the memory size and computation speed of computers increased and pre-post processing of computational results has become much more convenient and less laboring, it is still a major issue how to select the appropriate dimension in numerical analyses of structures. It is also fact that decisions in engineering design are still based on the rule of thumbs and/or one-dimensional analytical or numerical analysis of structures. In this article, the authors address this issue and compare several hypothetical and actual case histories involving tunnel face advance, man-made and natural geoengineering structures. The specific examples involve hypothetical tunnels, abandoned lignite mines, a karstic cave beneath Himeyuri Monument in Okinawa islands and steep cliffs and foundations.

Underground Structures

Tunnels

Advancing tunnels utilizing support systems consisting of rockbolts, shotcrete, steel ribs and concrete lining are three-dimensional complex structures and it is a dynamic process. However, tunnels are often modeled as a one-dimensional axisymmetric structure subjected to hydrostatic initial stress state as a static problem. The effect of tunnel face advance on the response and design of support systems is often replaced through an excavation stress release factor determined from pseudo three dimensional (axisymmetric) or pure three-dimensional analyses as given below

$$f = \frac{e^{-bx/D}}{1/B + e^{-bx/D}} \tag{1}$$

where x is distance from tunnel face and the values for coefficients B and b suggested by Aydan (2011) are 2.33 and 1.7, respectively.

Figure 1a illustrates an unsupported circular tunnel subjected to an axisymmetric initial stress state. The variation of displacement and stresses along the tunnel axis were computed using the elastic finite element method. The radial displacement at tunnel wall is normalized by the largest displacement and is shown in Figure 1b. As seen from the figure, the radial displacement takes place in front of the tunnel face. The displacement is about 28-30% of the final displacement. Its variation terminates when the face advance is about +2D. Almost 80% of the total displacement takes place when the tunnel face is about +1D. The effect of the initial axial stress on the radial displacement is almost negligible.

Figure 1c shows the variation of radial, tangential and axial stress around the tunnel at a depth of 0.125R. As noted from the figure, the tangential stress gradually increases as the distance increases from the tunnel face. The effect of the initial axial stress on the tangential stress is almost negligible. The radial stress rapidly decreases in the close vicinity of the tunnel face and the effect of the initial axial stress on the radial stress is also negligible. The most interesting variation is associated with the axial stress distribution. The axial stress increase as the face approaches, and then it gradually decreases to its initial value as the face effect disappears. This variation is limited to a length of 1R(0.5D) from the tunnel face. It is also interesting to note that if the initial axial stress is nil, even some tensile axial stresses may occur in the vicinity of tunnel face.

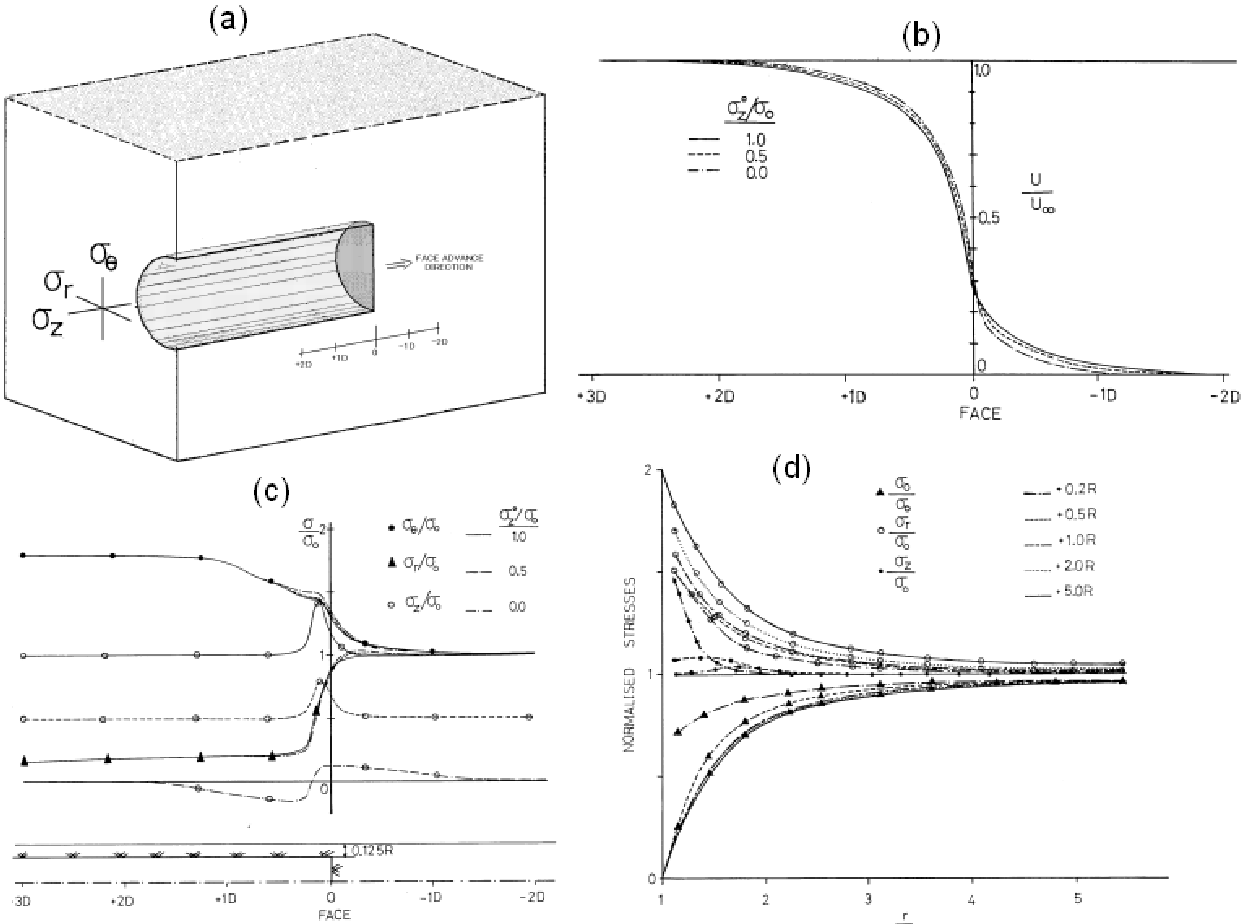
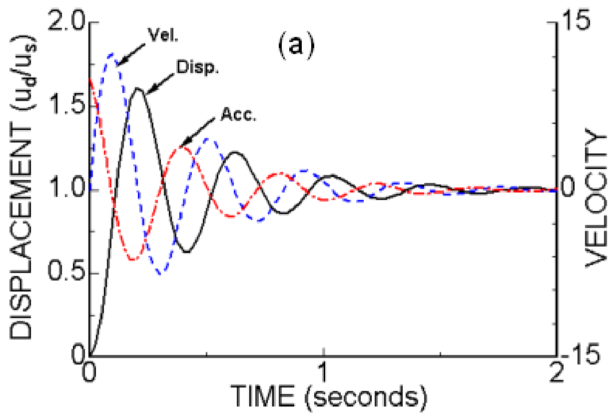


Figure 1. (a) Computational model for elastic finite element analysis; (b) Normalized radial displacement of the tunnel surface; (c) Normalized stress components along tunnel axis at a distance of 0.125R; (d): The variation of stresses along r-direction at various distances from tunnel face

Figure 1d shows the stress distributions along r-axis of the tunnel at various distances from the face when the initial axial stress is equal to initial radial and tangential stresses. As noted from the figure the maximum tangential stress is 1.5 times the initial hydrostatic stress and it becomes twice as the distance from the tunnel face is +5R, which is almost equal to theoretical estimations for tunnels subjected to hydrostatic initial stress state. The stress state near the tunnel face is also close to that of spherical opening subjected to hydrostatic stress state. The stress state seems to change from spherical state to the cylindrical state (Aydan 2011). It should be noted that it would be almost impossible to simulate exactly the same displacement and stress changes of 3D analyses in the vicinity of tunnels by 2D simulations using the stress-release approach irrespective of constitutive law of surrounding rock as a function of distance from tunnel (Aydan et al. 1988; Aydan and Geniş, 2010).

The effect of impulsive application of excavation force is evaluated for an axisymmetric cylindrical tunnel under initial hydrostatic stress by a dynamic visco-elastic finite element method. The responses of displacement, velocity and acceleration of the tunnel surface with a radius of 5m are plotted in Figure 2a. As noted from the figure, the sudden application of the excavation force, in other words, sudden release of ground pressure results 1.6 times the static ground displacement at the tunnel perimetry and shaking disappears almost at 2 seconds. As time progress, it becomes asymtoptic to the static value and velocity and acceleration disappear.

The resulting tangential and radial stress components nearby the tunnel perimetry (25cm from the opening surface) are plotted in Figure 2b as a function of time. It is of great interest that the tangential stress is greater than that under static condition. Furthermore, very high radial stress of tensile character occur nearby the tunnel perimetry. This implies that the tunnel may be subjected to transient stress state, which is quite different than that under static conditions. However, if the surrounding rock behaves visco-elasticallly, they will become asymptotic to their static equivalents. In other words, the surrounding rock may become plastic even though the static condition may imply otherwise.



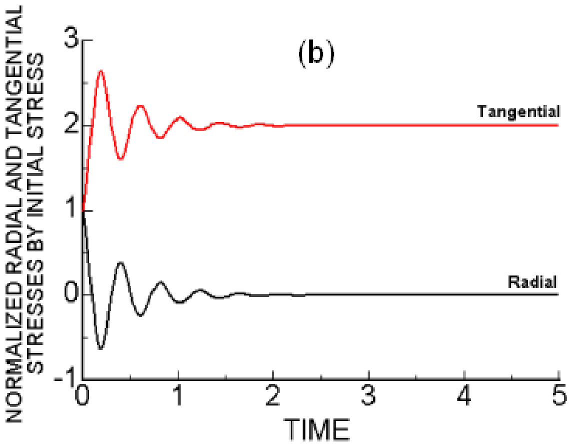
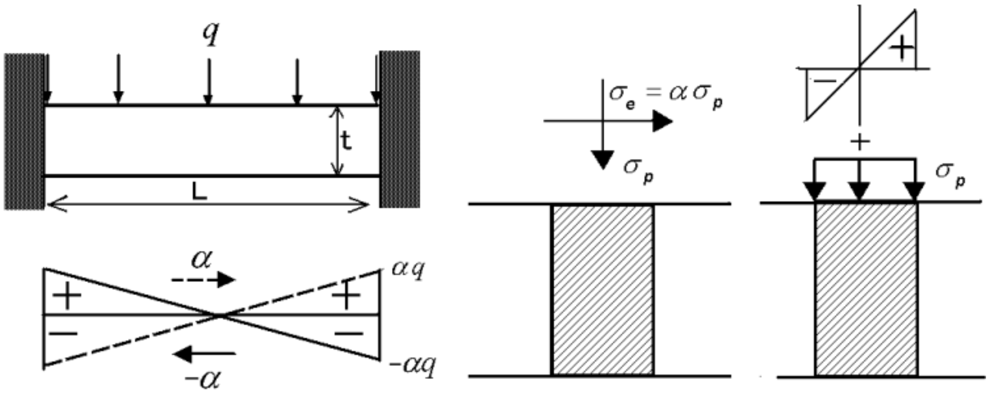


Figure 2. (a) Responses of displacement, velocity and acceleration of the tunnel surface; (b) Responses of radial and tangential stress components nearby the tunnel surface

Abandoned Lignite Mines

When abandoned lignite mines and quarries are of room and pillar type, their stability in short-term and long-term may be evaluated using some simple analytical techniques. Roof stability is generally evaluated using beam theory and/or arching theory under gravitational, earthquake and point loading (i.e. Coates, 1965; Obert and Duvall, 1967; Aydan 1989, 1994; Aydan and Tokashiki 2011). The tributary area method is quite widely used in mining engineering for assessing the pillar stability. Aydan and his co-workers (Aydan et al. 2008; Aydan and Geniş, 2008; Aydan and Tokashiki, 2011; Geniş and Aydan, 2008) extended to cover the effects of earthquake and point loading in addition to gravitational loading, creep and degradation of geomaterials to evaluate the stability of roof and pillars (Figure 3).

As shown by Aydan and Tokashiki (2011), simple analytical models and computations from two dimensional elastic finite element method yield very similar results. Nevertheless, stresses computed from the FEM in roof are less than those computed from the beam theory with built conditions. On the other hand, stresses computed from the FEM in pillars are slightly higher than those computed from the tributary area method. However, the stress state in the roof would be quite different if the opening depth increases. In such cases, the effects of gravitational load in the stress state of roof should be also taken into account. Nevertheless, the stability of pillars become more important than roof itself under such conditions and the tributary area method would yield quite reasonable values for the stress state in pillars for stability assessment.



(a) Roof model (b) Pillar model

Figure 3. Models for roof and pillars (from Aydan and Geniş, 2008).

Figure 4 shows the distribution of minimum principal stress (tenison is positive) for an abandoned room and pillar mine beneath Kyowa Secondary School in Mitake Town of Gifu Prefecture, Japan. Although the maximum pillar stresses are slightly higher than those computed from the tributary area method, the quick stability assessment using the tributary area method should be quite acceptable.

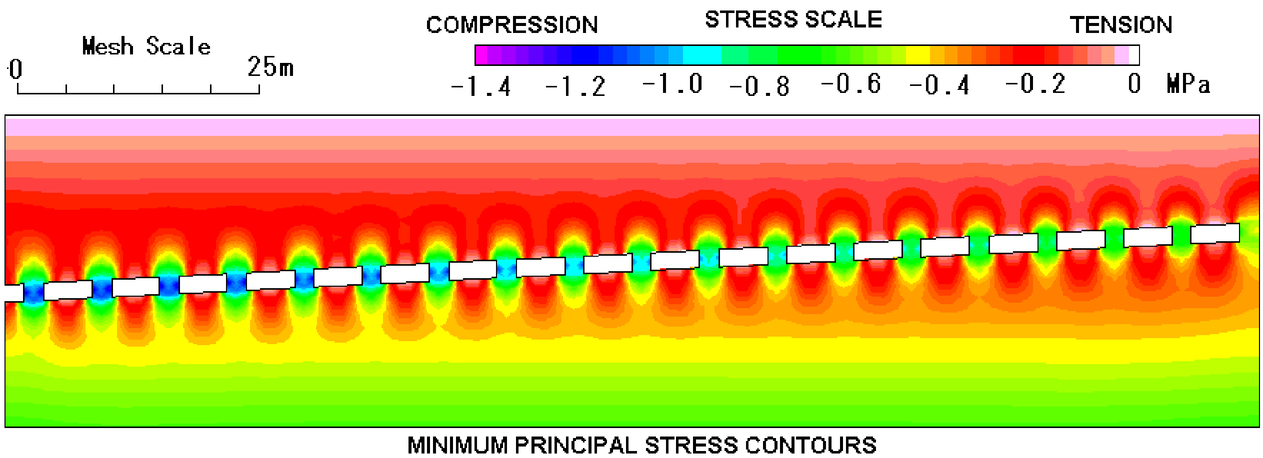


Figure 4. Contours of minimum principal stress beneath the Kyowa Secondary School.

This area would be subjected to the anticipated Nankai-Tonankai-Tokai earthquake in future and there is a great concern about it. Tha authors have been involved with the stability assessment of the abandoned lignite mine beneath Kyowa Secondary School during the anticipated Nankai-Tonankai-Tokai earthquake (Aydan et al. 2012; Geniş and Aydan, 2013). The authors carried out 1D, 2D and 3D dynamic simulations for an estimated base ground motion data at Mitake Town obtained from the method of Sugito et al. (2001). Figure 5 illustrates the numerical model of the ground and abandoned lignite mine beneath the Kyowa Secondary School. Figure 6 shows the computed responses from 1D and 3D numerical analyses. It is interesting to note that responses from 1D and 3D analyses are quite similar to each other.

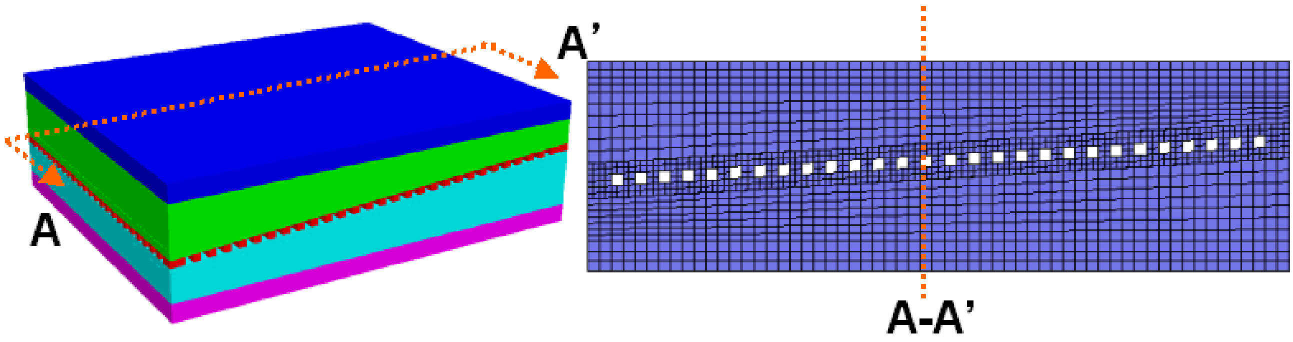
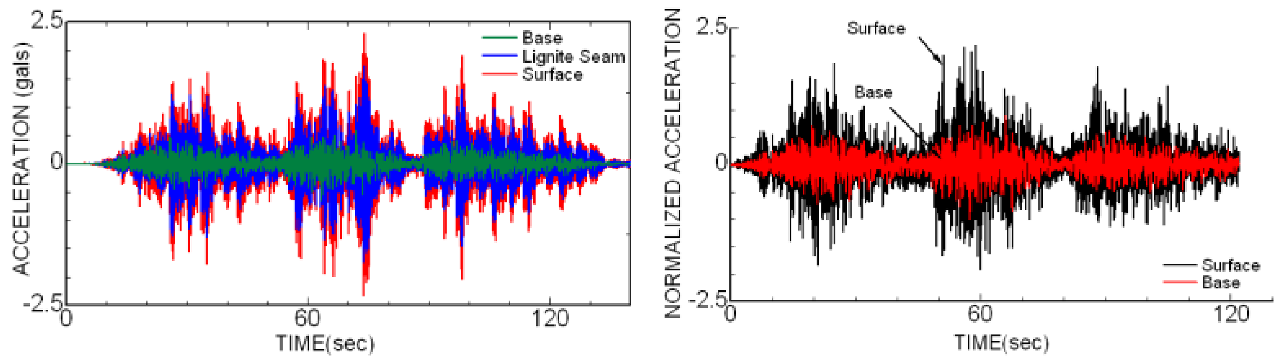
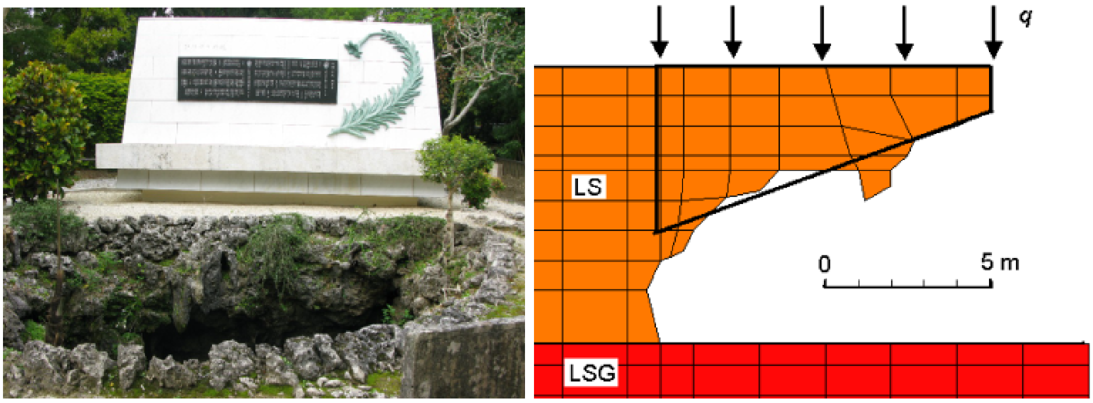


Figure 5. Illustration of models used in numerical analyses and selected section.



(a) 1D Analysis (b) 3D Analysis

Figure 6. Acceleration responses at selected section from 1D and 3D numerical analyses.



(a) View of the monument (b) Beam modeling of overhanging part

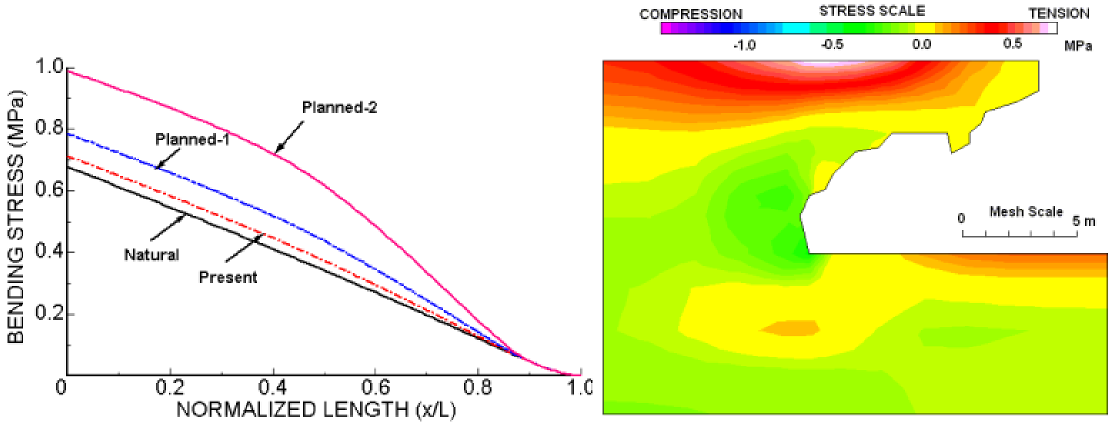
Figure 7. View and beam modeling of overhanging part.

Karstic Caves

Karstic caves are quite common worldwide whenever limestone and evaporates deposits exist. In the coral limestone formation in Ryukyu Islands of Japan, there are many karstic caves, which present many geo-engineering problems. The authors are involved with the stability assessment of some karstic caves in relation to some engineering projects or preservation of some monumental structures (i.e. Tokashiki, 2011; Aydan and Tokashiki, 2011; Geniş et al. 2009). There is a huge karstic cave beneath the Himeyuri monument in Okinawa Island. The enlargement of the monument was considered and the authors were consulted if the karstic cave would be stable upon the enlargement. Figure 7 shows a view of the monument and the beam modelling of overhanging part. Table 1 and Figure 8 compares the maximum tensile and compressive stresses computed from beam theory and FEM. Despite some slight differences, the results are quite similar.

Table 1. Comparison of maximum compressive and tensile stresses from FEM and bending theory.

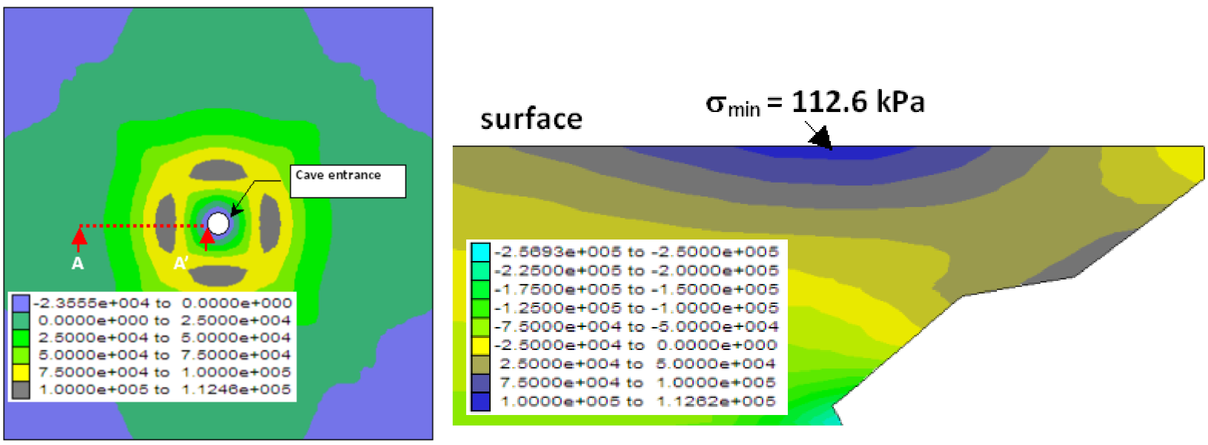
Loading Condition	Max. Tensile Stress (MPa)		Max. Compressive Stress (MPa)	
	FEM	THEORY	FEM	THEORY
Natural	0.557	0.677	−1.363	-0.677
Present	0.631	0.713	−1.402	-0.713
Planned-2	0.770	0.991	−1.478	-0.991



(a) Bending stress distribution from bending theory (b) Maximum principal stress from FEM

Figure 8. Comparison of stresses obtained from the bending theory and FEM.

A 3D analysis of the vicinity of Himeyuri monument and the cave beneath was carried out with the consideration of surface loading due to the dead weight of the monument structure (Aydan et al. 2011). The cave was considered to be circular in plan view. The maximum tensile stress was much smaller than that computed from the bending theory and 2D FEM analysis. Additional axisymmetric FEM analysis was also performed and it yielded similar results. However, the cave has an ovaloid shape in plan and the actual stress state is expected to be closer to that of 2D analyses. Furthermore, there are some cross-joints in rock mass so that the actual stress state should be quite close to that of 2D-FEM analyses.



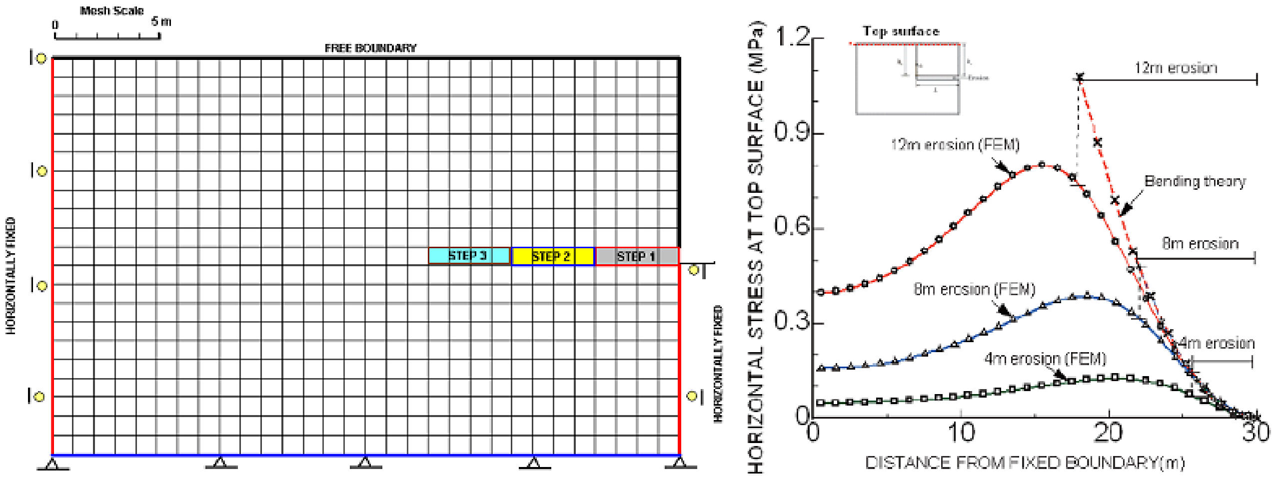
(a) Surface stresses (b) Maximum principal stress contours along A-A'

Figure 9. Computed maximum principal stress distributions from 3D numerical analysis.

Surface Structures

Cliffs

Very steep cliffs are observed along the shorelines of Ryukyu Islands. The toe of the cliffs is often eroded by sea waves and they result in overhanging configurations. When the erosion depth reaches to a certain distance, overhanging cliffs topple. Figure 10a shows the mesh used in FEM analyses for simulating erosion process while Figure 10b shows the distribution of tensile stress at top surface of the cliff for different erosion depths. The bending theory yields higher tensile stresses compared to those from the FEM analyses for different erosion depths (Tokashiki and Aydan, 2010). The maximum value of tensile stress obtained from the FEM analysis is about 75% of that computed from bending theory for the same erosion depth. Despite this slight difference, it may be quite acceptable to utilize the bending theory for quick assessment of the stability of cliffs.

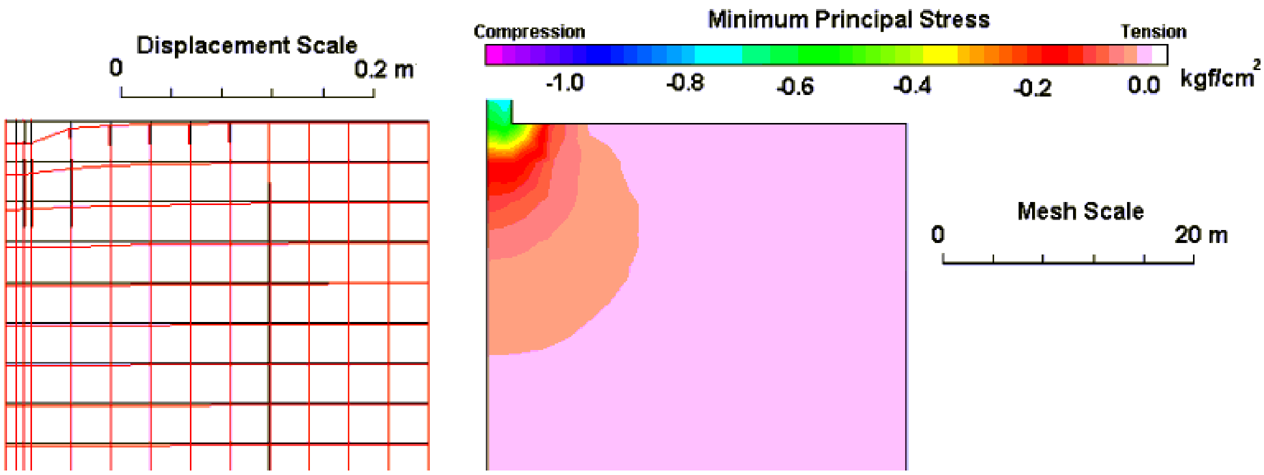


(a) 2D FEM mesh and simulation of toe erosion (b) Computed tensile stress at cliff surface

Figure 10. Mesh used in simulation of the toe erosion of steep cliffs and computed stresses at the upper surface of the model.

Foundations

Final example is concerned with settlement and stress state beneath foundations subjected to surcharge loads through relatively rigid foundation with a diameter of 3m, and it is modeled as an axisymmetric problem. Figure 11 shows the computed settlement and pressure contours beneath the foundation. The estimated settlement and pressure contours are generally in agreement with theoretical solutions by Timoshenko and Goodier (1951).



(a) Deformed configuration (b) Minimum principal stress contours

Figure 11. Settlement and pressure contours beneath a circular foundation.

Conclusions

The authors discussed how to select the appropriate dimension in numerical analyses of geo-engineering structures by considering some typical geo-engineering examples. There is no doubt that 3D simulation of geo-engineering structures is desirable. Although the pre-post processing of 3D mesh generation and visualization of computed results for geo-engineering structures have become much easier with the advance of computer technology and computational techniques, it is still useful to utilize 1D or 2D simulations for quick assessments and decision making. Therefore, researchers and engineers should explore the power of 1D and 2D simulations as much as possible. If such simulations yield some uneconomical results, then 3D simulations should be implemented. Furthermore, if numerical simulations involve time dependent problems, particularly, hyperbolic type solution schemes, it may be imperative to utilize first 1D and later 2D numerical simulations before exploring the power of 3D simulations.

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