A New Approach to Large-Scale Stress Reconstructor

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Abstract: The construction of stable structures in rock masses requires knowledge of the in-situ stresses at the scale of excavations. However, the measurements obtained by the conventional overcoring technique are related to a small scale (centimetres). To extrapolate them to the scales of interest to Rock Mechanics (from meters to kilometres) requires a large number of individual stress measurements followed, by statistical analysis if one is to avoid a considerable scatter of the measured values. In this paper, a method is proposed based on (a) large-scale surface stress and modulus measurements using the cylindrical jack method complemented by a special measuring scheme and then (b) back analysis for a given excavation shape. The method allows the simultaneous reconstruction of the stress components at the scale of excavation. The numerical simulation for a cylindrical excavation in an isotropic rock mass demonstrated the high accuracy and robustness of the method. The presence of a fractured zone surrounding the excavation can hamper the stress reconstruction, hence special measures should be taken to conduct the measurements in competent rock.

Key Words: in-situ stress, in-situ moduli, cylindrical jack, cylindrical excavation, back analysis

Introduction

Knowledge of the stress fields acting in the rock mass corresponding to both the original (pre-mined) regional stress state and the local stress concentrations caused by mining is necessary for the control the fracture processes in existing excavations and for the design of new ones.

Currently the most popular method of in-situ stress measurements is the overcoring technique. However, any one particular stress measurement can be non-representative with considerable scatter obtained from different measuring locations (eg. Cuisiat and Haimson, 1992; see also Galybin, et al., 1997b for more details). A major drawback is the irrelevance of extrapolating the results obtained by a small measuring device (tens of centimetres) to the scales of mining operations (from meters to kilometres).

There are two ways to overcome this problem. The first is to undertake a number of measurements spread over the area in question from which the mean value of the stress components may be determined. However the number of measurements required can be rather large. Indeed, if $\Delta$ is the range of the stress variation within the area and $N$ measurements are conducted in random and at independently chosen points, then the standard error of the stress reconstruction will be $\Delta N^{-1/2}$. In particular, if the local stresses vary by 100% (which is not unusual given that at a contour of a circular hole under uniaxial compression $p$, the stress concentration may vary from $-p$ to $3p$, i.e., by 400%), measurements at $N=100$ points are required to reach an accuracy of 10%. Thus reaching the proper scale requires a large number of point measurements and becomes a very expensive exercise.

The second is to use larger measuring devices (as illustrated by Cuisiat and Haimson, 1992, the scatter reduces when the diameter of the drilling holes in the overcoring method increases). However the possibility of enlarging the overcoring diameter is restricted. Other existing methods of stress measurements at larger scales have limitations such as the flat-jack (slot) method which provides only one stress component.
A potentially attractive method of obtaining a representative value for stresses at a larger scale is the under-excavation technique (Wiles & Kaiser 1994), in which the driven excavation itself is used as a measuring device. In this situation the scale of stress determination corresponds to the dimensions of the excavation. The method is based on displacement monitoring as the excavation advances and then the recalculation of the actual in situ stresses by back analysis based on a 3-D elastic solution.

However, the under-excavation method also has some shortcomings. First of all it requires the knowledge of the rock mass deformation characteristics which are obviously different from the ones obtained in the laboratory. Thus, the determination of the deformation moduli at the relevant scale remains a problem in this method. This could be overcome by complementing the displacement monitoring with stress measurements around the excavation. However, if they were to be obtained by the overcoring technique, the stresses determined would be at a scale much less than those involved in the elastic solution. This would still necessitate a considerable number of measurements. On top of that, the displacement measurements themselves would require drilling of measuring boreholes ahead of the excavation face which may interfere with the mining operations.

A new method of large-scale surface stress measurements has been proposed by Galybin, et al. (1997a, b). The method is based on Dean & Beatty’s (1968) cylindrical jack method complemented with a special measuring scheme ensuring the relevant size of the measurements. This scheme also allows reconstruction of the moduli at the same scale.

This new method is still restricted by the scale of measurement (possibly 1-2 m) and also in that only surface measurements may be undertaken. Nevertheless, in combination with back analysis of the stresses around the excavation in which the stress measurements are performed, the method will allow a true large scale stress reconstruction. It should be emphasised that this is not an under-excavation method, thus no deformation measurements are required ahead of the advancing excavation face.

This paper outlines the method of large-scale surface measurements; analyses the proposed combined method; investigates the error of the large-scale stress reconstruction as a function of the number of stress measurements and investigates the possibility of employing this method (as well as the under-excavation technique) for the case when a fractured zone developed during the mining operations surrounds the excavation.

**Surface stress and modulus measurements**

Three characteristics of an in-situ stress field can be determined by measurements near a free excavation surface. They are two principal stresses, \( \sigma_1, \sigma_2 \) (\( \sigma_3=0 \) on the free surface) and their orientation \( \theta_p \) at the surface. There are also deformation characteristics of the rock, which generally need to be determined. For the case of isotropic elastic rock there are only two: Young’s modulus, \( E \), and Poisson’s ratio, \( \nu \). Therefore, in this case, five parameters of the stress-strain state have to be determined. Three of them, \( \sigma_1, \sigma_2, E \), have the units of stress, the other two, \( \theta_p \) and \( \nu \), are dimensionless. This means, that in an ideal situation five independent measurements should be performed and at least one of them must have the units of stress.

The cylindrical jack method indeed provides an independent measurement of stress: the jack pressure. However, the variant employed in the 1960's by Dean & Beatty (1968) had some shortcomings. These shortcomings mainly came from the measuring scheme adopted which was (1) based on monitoring only radial displacements, which is not
sufficient to recover Poisson's ratio (since it requires an extra measurement in a perpendicular direction); and (2) the measuring pins were located close to the borehole contour, hence the scale of measurement was only of the order of the borehole diameter.

A new measurement system proposed by Galybin et al. (1997a, b) described below has been developed to overcome these shortcomings and thus enables in-situ stress and full moduli determination over a larger scale. The key innovations are the proposed arrangement of measuring points and the adoption of modern displacement measurement transducers with the level of resolution necessary to enable the system to work.

The measurement system is based on a number of extensometers (e.g., vibrating wire extensometers) placed as shown in Figure 1 to monitor the displacement (elongation/contraction) in both radial and circumferential directions. Since the measuring pins in this arrangement are spread over a large area, the scale of the stress and moduli reconstruction is determined by the size of this area rather than the borehole diameter. In addition, the extensometers do not obstruct the drilling and pressurising operations and do not have to be removed during drilling.

![Figure 1. The measuring scheme for the cylindrical jack method.](image)

In this arrangement, $N$ measurements are obtained in radial directions (lines 1-1', 2-2', ..., $N-N'$ denote the radially placed extensometers of base length $d$) and $N$ measurements in the circumferential directions (lines 1-2, 2-3, ..., $(N-1)-N$ denote the circumferentially placed extensometers of base length $2r_0\sin(\pi/N)$). The number of measuring pairs, $N$, is to be chosen to satisfy the redundancy requirement (4 minimum) and to be sufficient for the desired accuracy of the stress determination, while the choice of the extensometer base lengths, $d$ and $2r_0\sin(\pi/N)$ will be a compromise between the necessity to ensure appreciable scale of measurements and the sensitivity of the extensometers.

The mathematics of the stress and moduli reconstruction in elastic isotropic situation is described by Galybin et al. (1997a). It has however been found that the accuracy of the
reconstruction is not uniform: the most accurate are the reconstruction of the hydrostatic component of the stress state \( (\sigma_1 + \sigma_2)/2 \) and the shear modulus \( G=E/2(1+\nu) \). There are cases in which the reconstruction of Poisson's ratio is not accurate, e.g. when the stress state is close to the hydrostatic one. However, in this case the influence of Poisson's ratio is minor anyway. For the stress states close to the hydrostatic, the deviatoric parts \( (\sigma_1 - \sigma_2)/2 \) being small, reconstructed with high relative errors. Nevertheless, the orientation of the principal stresses can still be recovered accurately.

Table 1 illustrates the accuracy of the reconstruction of the original stress state and the Poisson's ratio obtained from computer simulations where the moduli and the stress states have been specified; the ideal readings of the extensometers computed and then distorted by introducing random errors uniformly distributed within the range of \((-10\%, 10\%)\). The reconstruction was undertaken with \( N=3, 4, ..., 8 \) measuring pairs.

Analysis of the variation coefficients for the reconstructed parameters for different number of measuring pairs has shown that \( N=6 \) measuring pairs are sufficient for reasonable accuracy. The accuracy of the determination of the shear modulus is not considered here, for it will obviously be the best one since it uses all \( 2N \) measurements for determination of this single parameter. It should be noted that by monitoring the displacements during the process of pressurising, the non-linear behaviour of the rock mass and the dependence of the shear modulus on the pressure can also be explored.

Table 1. Comparison of the average values of the reconstructed principal stresses \( \sigma_{1\text{rec}}, \sigma_{2\text{rec}}, \) the principal direction \( \theta_{\text{prec}} \) and the elastic modulus \( \kappa_{\text{rec}} = 3 - 4\nu \) with the original ones (shown in the table head, in all cases \( \kappa = 1.5 \)) for isotropic rocks. The stresses are in MPa.

<table>
<thead>
<tr>
<th>( N )</th>
<th>( \sigma_{1\text{rec}} )</th>
<th>( \sigma_{2\text{rec}} )</th>
<th>( \theta_{\text{prec}} )</th>
<th>( \kappa_{\text{rec}} )</th>
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<th>( \sigma_{2\text{rec}} )</th>
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**Full in-situ stress reconstruction**

The technique proposed for measurements of surface stress and moduli values can form the basis for complete, large scale in-situ stress reconstruction. Since a single installation can recover only a 2-D stress tensor related to the surface of measurements, measurements from at least three installations on differently oriented surfaces are required to compute all stress components. In order to extend these data and recover the original stress state in the rock mass itself, further computations are required.

This can be achieved by combining the results of the stress measurements at different locations on the excavation walls by means of back analysis of the solution for the excavation. In order to do this in the case of elastic rock mass, the deformation characteristics of the rock mass are required (for an elastic isotropic rock mass, only the Poisson's ratio is needed) and they are obtained simultaneously with the stress.
measurements when the proposed method is used. In the case of a non-elastic rock mass a number of additional moduli measurements might be necessary.

The method will be considered for the simple case of a cylindrical tunnel arbitrarily oriented with respect to the principal directions of the original (natural) stress. It is convenient to introduce a Cartesian coordinate frame $Ox_1x_2x_3$ with $x_3$-axis directed along the generatrix of the tunnel and axes $x_1$ and $x_2$ coinciding with the secondary principal directions. Then the original stress tensor has the following form

$$
\begin{pmatrix}
\sigma_{11} & 0 & \sigma_{13} \\
0 & \sigma_{22} & \sigma_{23} \\
\sigma_{13} & \sigma_{23} & \sigma_{33}
\end{pmatrix}
$$

(1)

In this case the stress field around the excavation can be presented as a superposition of the stress fields obtained in plane strain (the response to compressions $\sigma_{11}$ and $\sigma_{22}$), anti-plane (out-of-plane) strain (the response to shear loads $\sigma_{13}$ and $\sigma_{33}$) and the compression along the excavation axis, $\sigma_{33}$.

It is also convenient to introduce cylindrical coordinates $(r, \theta, z)$ such that $x_1 = r \cos \theta$, $x_2 = r \sin \theta$, $x_3 = z$. Stress components on the excavation wall can then be expressed as

$$
\begin{align*}
\sigma_\theta &= \frac{\sigma_{11} + \sigma_{22}}{2} + \frac{\sigma_{13} \sigma_{23}}{2} \cos(2\theta) \\
\sigma_z &= \sigma_{33} + \nu \sigma_\theta \\
\tau_{r\theta} &= -2(\sigma_{13} \cos \theta + \sigma_{23} \sin \theta) \\
\sigma_r &= \tau_{r\theta} = \tau_{z\theta} = 0
\end{align*}
$$

(2)

where $\nu$ is the Poisson’s ratio.

The components $\sigma_r$, $\sigma_\theta$, and $\sigma_z$, as well as Poisson’s ratio of the rock mass, $\nu$, can be measured in different points on the excavation wall by the method described in the previous section. The components of the stress tensor (1) as well as the coordinate angle of, say, the first measuring point, $\Theta_1$, serve as a reference point; the coordinate angles of other measuring points can be then determined with respect to the first one.

Let non-zero stress components be determined at $M$ different locations given by $0_k$, $k=1,\ldots,M$.

$$
\begin{align*}
\sigma_\theta^k &= \frac{\sigma_{11} + \sigma_{22}}{2} + \frac{\sigma_{13} \sigma_{23}}{2} \cos(2\theta_k) \\
\sigma_z^k &= \sigma_{33} + \nu \sigma_\theta^k \\
\tau_{r\theta}^k &= -2(\sigma_{13} \cos \theta_k + \sigma_{23} \sin \theta_k)
\end{align*}
$$

(3)

The coordinate angles $\theta_k$ are unknown, but their differences are known and can be denoted

$$
\Lambda_k = \theta_k - \theta_1
$$

(4)

The reference angle $\Theta_1 - \Theta$ remains unknown.

It is now convenient to introduce new parameters characterising the “plane strain” part of the original stress tensor.
\[ P = \frac{\sigma_{33} + \sigma_{11}}{2}, \quad D = \frac{\sigma_{22} - \sigma_{11}}{2} \]  

(5)

System (3) then assumes the form

\[ \sigma^k_x = P + D \cos(2\Delta_k + 2\Theta) \]
\[ \sigma^k_y = \sigma_{33} + \nu P + D \cos(2\Delta_k + 2\Theta) \]
\[ \tau_{13}^k = -2(\sigma_{13} \cos(\Delta_k + \Theta) + \sigma_{33} \sin(\Delta_k + \Theta)) \]

(6)

Let the surface stresses be determined with sufficient redundancy, i.e. \( M > 3 \). The problem of stress determination can then be reduced to the problem of minimisation of a proper functional in the space of 6 variables

\[ F(P, D, \sigma_{13}, \sigma_{23}, \sigma_{33}, \Theta) \rightarrow \min \]

(7)

In particular, one can use the following functional

\[ F(P, D, \sigma_{33}, \sigma_{13}, \sigma_{23}, \Theta) = \sum_{k=1}^{N} \left( \sigma_{13}^k - P - D \cos(2\Delta_k + 2\Theta) \right)^2 + \]
\[ \sum_{k=1}^{N} \left( \sigma_{23}^k - \sigma_{33} - \nu P - D \cos(2\Delta_k + 2\Theta) \right)^2 + \]
\[ \sum_{k=1}^{N} \left( \tau_{33}^k + 2(\sigma_{13} \cos(\Delta_k + \Theta) + \sigma_{33} \sin(\Delta_k + \Theta)) \right)^2 \]

(8)

A numerical experiment has been undertaken to estimate the accuracy of the stress reconstruction. The following values for stresses were used: \( \sigma_{11} = -2 \text{MPa} \), \( \sigma_{22} = -3 \text{MPa} \), (corresponds to \( P = -2.5 \text{MPa} \) and \( D = 0.5 \text{MPa} \)), \( \sigma_{33} = 1 \text{MPa} \), \( \sigma_{13} = 0.5 \text{MPa} \), \( \sigma_{23} = -0.2 \text{MPa} \) and the reference angle chosen was \( \Theta = 22^\circ \).

The surface stresses at points \( \Theta_k \) are modelled as the ideal ones computed for the above stress state using the formulae (3)-(6) and then distorted by adding to each of them an independently generated random error uniformly distributed within the range of \( \pm 20\% \). The Poisson’s ratio chosen was \( \nu = 0.3 \) and was supposed to be known in this simulation. In real situations the Poisson’s ratio can be determined as an average of the values reconstructed simultaneously with the stresses at the measuring points.

Functional (8) was minimised using a procedure built into the package MATHCAD 6 PLUS. The initial guess necessary for the numerical minimisation has been computed by using the first three sets of measurements \((k=1,2,3)\) to find parameters \( P, D, \Theta, \), and then the obtained value of the angle \( \Theta \) was used to find parameters \( \sigma_{33}, \sigma_{13}, \sigma_{23} \) from the first two sets of stress measurements.

For each number of surface measurements, \( M = 3 \ldots 16 \), 200 combinations of readings were independently generated, and the values of \( P \) were reconstructed. The averages over 200 sets of readings and the variation coefficients, \( \delta \), (the standard deviation over the absolute value of the mean, e.g., for a parameter \( X, \delta = \frac{\text{Var}(X)}{\text{E}(X)} \)) were then computed.

Figure 2 shows the mean values and variation coefficients representing the relative errors (\%) of the reconstruction for all 6 unknown parameters for different numbers of measuring points.
Figure 2. Reconstruction of the initial stress state vs. the number of measurements. Each broken line is the average value of the reconstructed parameter, while horizontal dashed line shows the exact value. Solid lines show the coefficients of variation which represents the relative error of reconstruction: (a) the hydrostatic pressure, $P$; (b) the deviatoric component, $D$; (c) normal component, $\sigma_{33}$; (d) shear component, $\sigma_{13}$; (e) shear component, $\sigma_{23}$; (f) the reference angle, $\Theta$. 
It is seen from the plots that the accuracy of reconstruction is quite high given the error specified in the stress measurements was very large, ±20%. Thus, 7 points of measurement are normally enough to bring the error of the stress reconstruction below ±20%, while some components are restored with even higher accuracy. Further increase in the number of measuring points can improve the accuracy even beyond the accuracy of the original measurements.

Thus, the proposed combined method allows the accurate reconstruction of the full stress state at the level of the excavation without the shortcomings characteristic for the under-excavation technique, viz the necessity to conduct displacement measurements in front of the advancing excavation face.

**Effect of the presence of a fractured zone around the excavation**

The method has to have been tested based on the assumption that the rock mass is homogeneous. However, in some cases the excavation is surrounded by a heavily fractured zone created either due to the excavation process, or due to deterioration of the rock adjacent to the excavation walls. In this case the non-homogeneity of the rock mass can no longer be neglected and its effect should be investigated.

In the simplest case the fractured zone can be modelled by an elastic ring possessing moduli lower than those of the rock mass. In order to analyse the possibility of using the method for the stress reconstruction in this case, a simple example will be considered in which the ambient stress field satisfies the plane strain conditions and is purely hydrostatic (i.e, $\sigma_{11}=\sigma_{22}=P$, $\sigma_{12}=\sigma_{13}=\sigma_{23}=0$). The excavation radius is taken as equal to unity and the external radius of the fracture zone be $R_d$ as illustrated in Figure 3. Then by taking the axial symmetry into account and introducing the unknown contact pressure, $p_d$ on the boundary between the fractured zone and the rock mass, the corresponding complex potentials can be expressed in the form (e.g., Muskhelishvili, 1953):

$$\Phi(z) = -\frac{P}{2} z, \quad \Psi(z) = -(P - p_d) \frac{R_d^2}{z^2}$$  \quad (9)

$$\varphi(z) = -\frac{P}{2} z + C_1, \quad \psi(z) = (P - p_d) \frac{R_d^2}{z} + C_2$$ \quad (10)

- outside the fractured zone, and

$$\Phi(z) = -p_d \frac{R_d^2}{2} \frac{R_d^2}{R_d^2 - 1}, \quad \Psi(z) = -p_d \frac{R_d^2}{R_d^2 - 1} \frac{1}{z}$$ \quad (11)

$$\varphi(z) = -\frac{p_d}{2} \frac{R_d^2}{R_d^2 - 1} z + C_3, \quad \psi(z) = p_d \frac{R_d^2}{R_d^2 - 1} \frac{1}{z} + C_4$$ \quad (12)

- inside the fractured zone.

The displacements outside the fractured zone can be obtained from (10) by acknowledging that the constants $C_1$ and $C_2$ vanish after neglecting the rigid body movements, i.e.

$$u_r - iu_\theta = \frac{1}{2\mu} \frac{z}{r} \left[ (1-\kappa) \frac{P}{2} z - (P - p_d) \frac{R_d^2}{z} \right]$$ \quad (13)
The displacements inside the fractured zone are obtained from (12) by taking into account that the terms containing constants \( C_3 \) and \( C_4 \) should be zero due to the axial symmetry, i.e.

\[
u_e - u_e = \frac{1}{2\mu_d} R_d^3 \frac{z}{r} \left[ (1 - \kappa_d) \frac{P}{2} z - (P - p_d) \frac{R_d^2}{z} \right]
\]  

(14)

Here \( \kappa_d = 3 - 4\kappa_\varepsilon \), \( \mu_d \) and \( \nu_\varepsilon \) are the shear modulus and Poisson's ratio of the fractured zone respectively.

![Diagram of a fractured zone](image)

Figure 3. An example: the elastic model of the fractured zone.

Continuity of the displacements allows the determination of the dimensionless interface pressure \( q_d = p_d/P \)

\[
q_d = \frac{(1 + \kappa)(R_d^2 - 1)}{2R_d^2 - 2 - mR_d^2 \kappa_d + 2m}
\]  

(15)

where \( m = \mu/\mu_d > 1 \).

By substituting (15) into (11) and (14) and using the Kolosov formulae, one can determine the non-zero stress and displacement components at the excavation wall:

\[
s_\theta^{\text{contact}} = -\frac{p}{2 - m\kappa_d + 2(m - 1)R_d^2}, \quad u_r^{\text{contact}} = \frac{1 + \kappa_d}{4\mu_d} s_\theta^{\text{contact}}
\]  

(16)

It can be seen that the proposed method based on the surface stress and modulus measurements can only recover a certain combination of the unknown ambient stress \( p \), the fractured zone radius, \( R_d \) and the moduli of the rock mass (the surface measurements can only provide the moduli of the fractured zone). The surface
displacement is proportional to the stress, hence measuring it will not contribute to the separation of the unknown parameters. Therefore, in the case considered the underexcavation method will not enable the stress reconstruction, so using it in combination with the proposed method can only increase the accuracy of the determination of the fractured zone moduli.

This example demonstrates that the fractured zone becomes a serious obstacle for the stress reconstruction which is why the installation of the measuring pins and the cylindrical jack deep enough to reach the competent rock (Figure 1b) is essential.

Conclusions

The proposed method based on (a) large-scale surface stress and modulus measurements at a number of points in the excavation and then (b) the back analysis for a given excavation shape allows the simultaneous reconstruction of the stress components at the scale of excavation. The numerical simulation for a cylindrical excavation in an isotropic rock mass demonstrated the high accuracy and robustness of the method.

An advantage of the proposed method over the under-excavation technique is that it does not require the prior determination of the rock mass moduli and does not interfere with the mining operations and therefore can be used for continuous stress monitoring.

The presence of a fractured zone surrounding the excavation can hamper the stress reconstruction, hence special measures should be taken to conduct the measurements in competent rock.

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