



# On the accuracy and evaluation of rock stress measurements in the Kanto-Tokai region, Japan

Y. NISHIMATSU  
H. KOIDE  
S. KOIZUMI

*University of Tokyo, Department of Mineral Development, Tokyo, Japan*

## ABSTRACT

The rock stress is measured by means of borehole deformation gauge in several test sites in the western region of Tokyo. The accuracy of rock stress measurement with this gauge is discussed from the technical point of view.

The state of rock stress determined by this gauge is compared with those determined by other principles of rock stress measurements, and discussed from the view point of plate tectonics.

It is indicated that results of the present measurement agree with those of several other measurements in a reasonable range of deviation from the view points of measurement engineering and plate tectonics.

## ZUSAMMENFASSUNG

In unterschiedlichen Orte im Westen von Tokyo, wird die Gebirgsspannung mittels eines Verformungsmessgerätes des Bohrloch gemessen. Es behandelt die Messgenauigkeit des Gebirgsspannungen aus dem messtechnischen Gesichtspunkt.

Der Zustand der Gebirgsspannungen in dieser Region die ist bestimmt mittels der Verformungsmessgerätes wird mit die bestimmt mittels anderes Messprinzip verglichen und aus dem Gesichtspunkt der Plattentektonik bewertet.

Es wird gezeigt dass die Abweichung zwischen die Ergebnisse dieser Messungen und die einiger anderen Messungen ist vernachlässigt in Hinsicht auf den Messtechnik wie auch Plattentektonik.

## RESUME

Les contraintes dans les roches s'ont mesurés à une cellule extensométrique dans les trous de forage à l'ouest de Tokio. La précision au mesure des contraintes par la cellule extensométrique sont en discussion au point de vue technique.

L'état de contrainte dans les roches mesuré par la cellule extensométrique se compare aux résultats dérivés par les principes différentes au mesure. Nous mettons le champ de contrainte en discussion en vue de la tectonique de plaque.

Il s'indique que les résultats aux présentes mesures sont en accord avec les résultats aux méthodes différentes à la portée acceptable de deviation aux points de vue de l'art de la mesure et de la tectonique de plaque.

## 1. INTRODUCTION.

One of the most reliable and popular principles of rock stress measurement would be the stress relief principle by overcoring among several principles proposed for rock stress measurement. Several techniques or gauges have been developed and used

in practice as applications of the stress relief principle.

For examples, it is reported that the complete state of stress has been determined in a single borehole by means of CSIR triaxial strain cell (Leeman 1968) as well as the doorstopper type strain cell (Hiramatsu, et al 1979). However, they have an inher-

nt difficulty to be applied in the borehole in which the wall is wet and/or some water inflows occur, because the strain cell must be glued to the rock wall of boerhole.

As another technique of the stress relief principle, the borehole deformation gauge (Obert 1962, Suzuki et al 1970) can be applied in the wet or submerged borehole, although this technique needs three boreholes at least, for determination of the complete state of stress in the rock mass.

The accuracy of rock stress measurement has been discussed by several authors for each technique, and the coefficient of variance more than 20% has been sometimes reported (Gray et al 1975, Van Heerden 1976). A few authors have measured the rock stress at the same test site by two different techniques, and reported the deviation of about 20% of the major principal stress measured by doorstopper strain cell from that measured by borehole deformation gauge (Van Heerden et al 1967, De La Cruz et al 1972).

As well known, all of techniques of rock stress measurement by overcoring becomes more difficult to be conducted as deeper becomes the borehole. This is a reason that the hydrofracturing technique is frequently used for rock stress measurement in the deep borehole (Haimson 1978). However, it is very difficult to determine exactly the direction or azimuth of generated crack. In order to reveal the accuracy of hydrofracturing technique, the rock stress is measured by means of hydrofracturing in the test site where the rock stress has been determined by the borehole deformation gauge in short boreholes drilled from an underground roadway (Haimson et al 1974). The discrepancies between

the results by these two techniques have been given as 9% and 30% for the major and minor principal stresses in the horizontal plane, respectively.

Since 1978, the authors have conducted the rock stress measurement in several test sites in Kanto-Tokai region, Japan. In this series of rock stress measurements which have been conducted as a part of the national project for prediction of large scale earthquake, the substantial interest concerns to the state of stress in the horizontal plane in the rock mass. As more data of rock stress measurements have been accumulated and compared with each other, as more necessities for evaluating the accuracy of measurement have been revealed.

In this paper, the authors describe the procedure and result of rock stress measurements conducted by means of borehole deformation gauge. The accuracy and error sources of rock stress measurement are evaluated, and the revealed state of stress in Kanto-Tokai region is discussed from the viewpoint of plate tectonics.

## 2. ROCK STRESS MEASUREMENTS NEAR SURFACE

### 2.1. Measuring instruments, divices and procedure.

There are neither tunnel nor mine in the vicinity of selected test site, because the test site has been selected from viewpoint of plate tectonics which is concerned with prediction of earthquake.

The borehole must be drilled vertically from ground surface, and a three components borehole deformation gauge must be set in the submerged borehole at the depth more than 10m. The gauge has been developed by K.Suzuki:

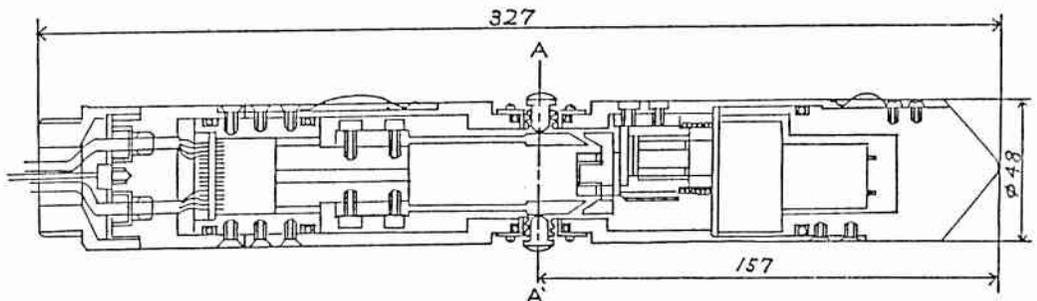


Fig.1. The Three Components Borehole Deformation Gauge.

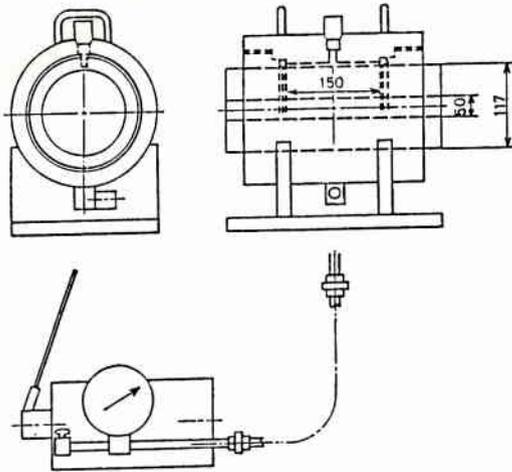


Fig.2. The External Hydraulic Pressure Cell

(Suzuki et al 1970), and modified by authors to be water-tight. It is provided with six elements to detect the change of diameter in three directions at every 60° around its periphery as shown in Fig.1. The stud or piston of each six element is glued into the bellows and isolated from the water-tight main part. The connector of electric cable at the end of main part is molded by rubber. The electric cable is drawn out through a packer, lower and upper ends of which are flexibly connected to the upper

end of the gauge and lower end of pipe for inserting the gauge, respectively. The packer is expanded by Nitrogen gas and fixes the gauge together with spring plates mounted on the periphery of gauge. An electric motor is provided for driving the contracting disc which is drawn back and releases six studs, when the gauge has been set in the borehole. Considering the possible irregularity of rock stress caused from various topographical and geological irregularities near ground surface as well as the technical restriction of measuring procedure, the rock stress measurement has been conducted at the depth between 10 and 20m from ground surface.

The recovered hollow cylindrical rock core is covered by the hot-shrinkable polyolefin seat and loaded radially in the external hydraulic pressure cell (see Fig.2). The change in diameter of centre hole is measured again by the borehole deformation gauge to determine Young's modulus of the rock cylinder.

## 2.2. Results of rock stress measurements

The rock stress measurements are conducted at several test sites where the outcrop of intrusive rock mass has been found, in the vicinity

Table 1 The Result of Rock Stress Measurements in Kanto-Tokai Region

Test Site	Latitude	Longitude	Altitude (m. S.L.)	Depth from Surface (m)	Maximum Horizontal Compression (MPa)	Minimum Horizontal Compression (MPa)	Azimuth of Maximum Horizontal Compression	Rock Type	Time of Measurement
Tanzawa A	35°28' N	139°4' E	500	5.73	11.7	3.2	N 7° E	Quartz Diorite	1978.11
Tanzawa D	35°28' N	139°4' E	500	14.89	7.5	2.6	N 9° E	"	1979.12*
Tanzawa E	35°28.5' N	139°5.5' E	900	11.25	12.3	4.5	N 27° E	"	1979.11
Shimoda	34°40.5' E	138°56.5' E	12	19.0	4.0	2.0	N 20° W	Porphyrite	1981. 3
Tsukuba	36°11' N	140°5.5' E	40	16.26	1.0	0.0	N 66.5°W	Granite	1982. 3
Tanzawa F	35°28' N	139°4' E	500	12.78	6.9	1.8	N 30° E	Quartz Diorite	1983. 2
Toi	34°54' N	139°50' E	12.50	350	4.7	2.1	N 8° W	Diorite	1983.12
	"	"	"	350	7.6	5.1	N 8° W	"	**
Kanbara	35°07' N	138°36' E	20	14.25	1.1	0.0	N 65° W	Andesite	1984. 3
Shibakawa	35°17' N	138°33' E	299	10.93	2.3	0.0	N 21° W	"	1985. 3

\* measured by a soft inclusion type gauge

\*\* measured by hydrofracturing technique

of boundary zone between Phillipine Sea, Eurasian, and North American plates. The state of stress in the horizontal plane is determined for each test site on the assumption of the plane state of strain.

The extension U of borehole diameter due to stress relief is given by (Panek 1966)

$$U = f_1 \sigma_u + f_2 \sigma_v + f_3 \tau_{uv} \quad \text{---(1)}$$

where

$$\left. \begin{aligned} f_1 &= \frac{d}{E}(1 - \nu^2)(1 + 2\cos 2\theta) \\ f_2 &= \frac{d}{E}(1 - \nu^2)(1 - 2\cos 2\theta) \\ f_3 &= 4 \frac{d}{E}(1 - \nu^2)\sin 2\theta \end{aligned} \right\} \text{---(2)}$$

d is the diameter of borehole, E and  $\nu$  are Young's modulus and Poisson's ratio of rock cylinder, respectively,  $\theta$  is the angle of the diameter measured with respect to u axis of the u-v coordinate system in the plane perpendicular to the axis of borehole.

The observed changes of borehole diameter in three directions at every 60° are substituted into Eq.(1) to determine three components of rock

stress.

The results of rock stress measurements conducted by the present authors are summarized in Table 1. They are compared with the results of several other measurements which have been conducted by means of hydrofracturing (Tsukahara et al 1984) and other measuring methods (Koide et al 1986). On the basis of all of those results of measurements, the azimuth of major principal stress in the horizontal plane (horizontal maximum compression) is plotted in Fig.3.

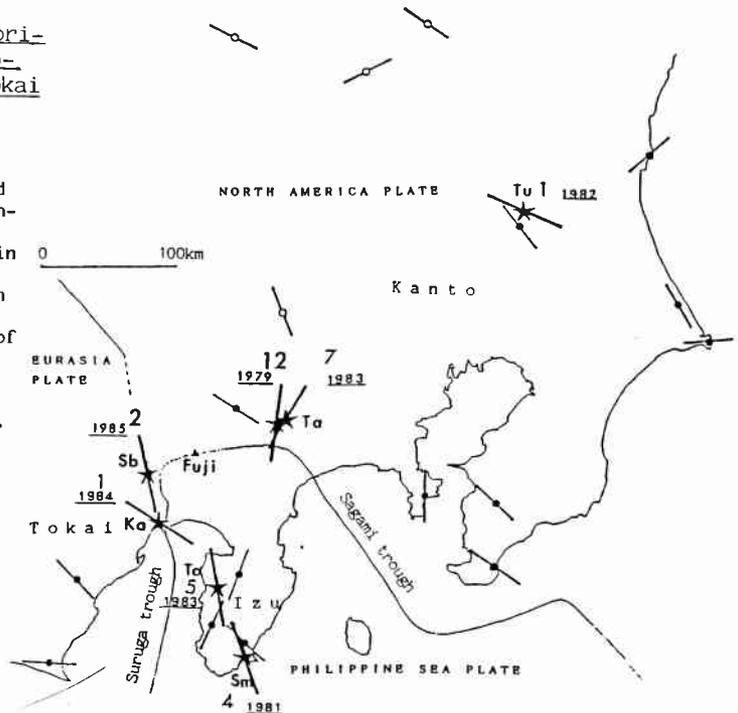
It has been revealed that the azimuth as well as magnitude of the principal stress show an evident discrepancy each other. Although a part of those discrepancies or fluctuations would reflect the real fluctuation of the state of stress in this region, it is desired to verify the accuracy of rock stress measurement. Furthermore the knowledge on the complete state of stress in the rock mass has been necessary to evaluate the accuracy of rock stress measurement.

### 3. ROCK STRESS MEASUREMENT IN UNDERGROUND MINE.

Fig.3. The Direction of Horizontal Maximum Compression in Kanto-Tokai Region.

Short bars indicate the measured directions of the maximum horizontal compression. Large stars are results of stress relief method in this study with the value of the maximum horizontal compression (in MPa) and the measured years.

Open circles are other results of stress relief method (Koide et al 1986). Closed circles are results of hydrofracturing technique by N.R.C.D.P. (Tsukahara et al 1983). Tu: Tsukuba, Ta: Tanzawa, To: Tol, Sm: Shimoda, Sb: Shibakawa, Ka: Kanbara.



### 3.1. Location and geology of the test site.

As well known, at least 3 boreholes in 3 different directions are necessary for determining the complete state of stress by means of borehole deformation gauge (Hiramatsu et al 1968). They would be easily drilled from the roadway of underground mine.

Considering geographical and geological conditions, it is decided to locate the test site in Seigoshi gold mine (34°54'N, 138°50'E) near Toi at the North-West coast of Izu peninsula. Ore deposits are shallow hydrothermal fissure filling type quartz veins including gold and silver. The veins occur in the country rocks which consist of basaltic andesite, tuff and pyroclastic rocks, and belong to Yugashima formation group deposited in Miocene of the Tertiary. The ore deposit consists of main veins of N30°-45°E in strike and 70°-75°SE in dip, and branch veins of N-S in strike.

The test site is located at the access roadway of mine water processing plant and distant about 150m North-West from the main shaft at 0m level (+12,5m S.L.) i.e. the depth of 350m from surface (see Fig.4).

The horizontal boreholes for measurement are drilled to East and North, respectively. Another borehole for measurement is drilled down vertically. All of three boreholes are drilled in the large intrusive rock mass consisted of andesite and diorite, which include few fractures and show low permeability.

The mechanical properties of

Table 2 Mechanical Properties of Boring Core

Borehole	Compressive strength (MPa)	Tensile strength (MPa)	Young's modulus (50% secant) ( $\times 10^3$ MPa)	Poisson's ratio (50% secant)
A	225 (38.9)*	19.1 (1.52)	71.1 (0.45)	0.280 (0.019)
B	281 (57.1)	18.3 (4.08)	77.2 (1.82)	0.280 (0.005)
C	259 (31.7)	19.9 (1.59)	67.9 (5.07)	0.313 (0.016)

\* Standard deviation is given in ( )

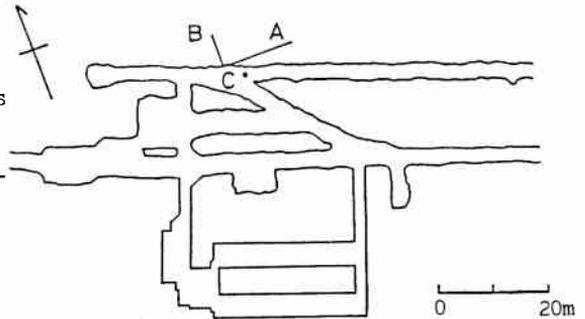


Fig.4. The Location of Test Site in Om Level in Seigoshi Gold Mine.

rock sample which are obtained by core boring of 50mm in diameter (the diameter of core is 32mm) and tested in uniaxial compression are shown in Table 2. On the other hand, the deformation of hollow cylindrical rock core (external and internal diameters are 117 and 50mm, respectively) recovered by overcoring are measured by means of the external hydraulic pressure cell, and Young's modulus is determined as secant value of stress-strain diagram up to 20 MPa. The results are shown in Table 3, and used for determining the rock stress. However, they are remarkably different from those determined from uniaxial compression test of centre core and shown in Table 2.

### 3.2. Measurement by borehole deformation gauge.

The gauge and procedure are same as those used for the measurement

Table 3 Young's Modulus of Rocks Measured by the Radial Pressure Test

Position of measurement	Mean ( $\times 10^3$ MPa)	Standard deviation ( $\times 10^3$ MPa)
A-1	63.8	1.08
A-2	63.4	1.71
B-1	79.8	13.78
B-2	62.2	0.10
C-1	60.6	2.82
C-2	65.1	2.24

near surface which is described in § 2. The instrumental error is verified as  $\pm 2 \mu\text{m}$  for measuring the extension of borehole diameter.

In this case, the borehole of 50mm in diameter is firstly drilled up to about 10m deep, then the overcoring is conducted step by step. However, the rock stress measurement is not conducted up to 3m deep. For every 3 boreholes, the rock stress measurements are conducted at two different depths, and the test results are shown in Table 4.

Assuming the plane state of strain, the state of stress in the plane perpendicular to each borehole axis is determined by substituting the test result shown in Table 4 into Eq.(1), and illustrated in Fig.5, which indicates a remarkable difference between the states of stress at different depth in the same borehole.

However, it is difficult to evaluate the accuracy of rock stress measurement, because the difference might be caused from some stress disturbance around the roadway. Furthermore, the effect of the axial stress  $\sigma_w$  in each borehole would not be ignored in this test site, because the overburden pressure is estimated as about 9MPa. Thus, considering to minimize the effect of stress disturbance around the roadway, three data sets obtained at the deeper measuring points in each three boreholes are used for determining the complete state of stress.

Table 4 Diametrical Deformation in the Borehole U(i)

Position	Distance from the mouth of borehole (m)	Extension of diameter		
		U (1)* ( $\mu\text{m}$ )	U (2) ( $\mu\text{m}$ )	U (3) ( $\mu\text{m}$ )
A-1	3.59	30.0	5.0	42.7
A-2	6.00	26.4	1.7	20.9
B-1	3.46	30.5	19.4	-4.8
B-2	4.21	40.5	2.4	3.5
C-1	3.62	-1.7	0.3	8.9
C-2	4.31	1.8	1.7	0

\* U(1) is measured in the direction from top to bottom, or North to South. U(2) and U(3) are measured in the direction of 60° and 120° counterclockwise from the direction of U(1), respectively.

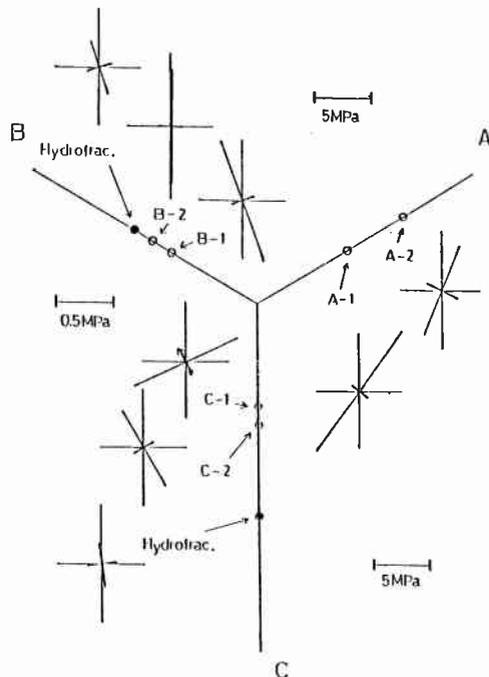


Fig.5. The State of Stress in the plane Normal to the Axis of Each Borehole

In this case, the observation equation is given by (Panek 1966)

$$U = f_1 \sigma_u + f_2 \sigma_v + f_3 \tau_{uv} + f_4 \sigma_w \quad (3)$$

where

$$\left. \begin{aligned} f_1 &= \frac{d}{E}(1 - \nu^2)(1 + 2\cos 2\theta) + \frac{d}{E}\nu^2 \\ f_2 &= \frac{d}{E}(1 - \nu^2)(1 - 2\cos 2\theta) + \frac{d}{E}\nu^2 \\ f_3 &= 4 \frac{d}{E}(1 - \nu^2)\sin 2\theta \\ f_4 &= -\frac{d}{E}\nu \end{aligned} \right\} (4)$$

the w axis of local coordinate system of u, v and w coincides with the axis of each borehole.

Let assume the same state of stress at these three measuring points and the common coordinate system of x, y and z, directions of which are selected as East, North and upwards vertically, respectively. Then, every axis of local coordinates of three boreholes coincides with any appropriate one of the common coordinate. Substituting three data sets of three boreholes into Eq.(3), we have 9 observation

equations to determine 6 components of rock stress in three dimensions.

The most probable values of rock stresses and their estimated standard deviations are determined by means of the least square method. As shown in Table 5, the result shows a remarkable standard deviation for each stress component.

### 3.3. Measurement by Hydrofracturing.

In order to verify the accuracy of measurement by hydrofracturing, the rock stress is measured by hydrofracturing in the same three boreholes used for measurement by borehole deformation gauge. The measuring point in each boreholes are shown in Fig.5 and Table 6.

After finishing the measurement by borehole deformation gauge, the borehole of 66mm in diameter is drilled more than 1m in depth from the bottom of overcored borehole of 167mm in diameter, because the double packer system of 66mm in diameter are used for pressurizing the borehole. The plunger pump of 0.5 l/min in capacity and 50MPa in maximum pressure is used to supply the hydraulic pressure. The flow rate of water for hydrofracturing is kept constant in 0.3 l/min, and shut down to zero, after keeping constant pressure for about 15min.

The procedure described above is repeated several times to determine the break-down pressure  $P_b$ , the crack reopening pressure  $P_b'$  and the shut-in pressure  $P_s$  from each record of test result.

Avoiding many technical difficulties concerned with the impression packer for fracture delineation, the

Table 5 The state of stress in three dimensions determined by means of the least square method

Stress	Borehole deformation method (MPa)	Hydrofracturing method (MPa)
$\sigma_x$	2.17 (4.55)*	5.30 (4.05)
$\sigma_y$	4.67 (3.74)	7.70 (6.52)
$\sigma_z$	16.17 (3.35)	8.86
$\tau_{xy}$	-0.36 (1.94)	-1.81 (6.96)
$\tau_{yz}$	-3.82 (1.89)	-3.92
$\tau_{zx}$	0.22 (1.89)	-1.31

\* Standard deviation is given in ( )

overcoring in the same diameter of 167mm as it for stress relief overcoring is conducted to recover the hollow cylindrical rock core including the fractures generated by hydrofracturing.

In order to observe the orientation of recovered rock core as well as generated fracture cracks, the plunge line is marked on the drill pipe immediately after finishing the overcoring and the core is recovered without any rotation of core barrel for two horizontal borehole, and a borehole compass provided with time-set clamp is fixed in the centre hole after recovering the packer system for a vertical borehole.

The test results including the orientation of generated fracture cracks are shown in Table 6.

The major and minor principal stresses in the plane normal to the axis of each borehole can be determined from the following equations (Haimson 1978):

for the break-down pressure  $P_b$

$$P_b - P_0 = \frac{T + 3\sigma_{Hmin} - \sigma_{Hmax} - 2P_0}{K} \quad \text{---(5)}$$

for the shut-in pressure  $P_s$

$$P_s - P_0 = \sigma_{Hmin} \quad \text{---(6)}$$

and for the crack reopening pressure  $P_b'$

$$P_b' - P_0 = \frac{3\sigma_{Hmin} - \sigma_{Hmax} - 2P_0}{K} \quad \text{---(5')}$$

where  $T$  is the hydrofracturing tensile strength,  $P_0$  is the pore pressure, and  $K$  is the constant concerned with effect of permeability. In the present measurement,  $K$  and  $P_0$  are assumed as zero, because the rock mass at the test site is intact and shows little water inflow. Considering that the hydrofracturing tensile strength does not coincide with the tensile strength of the core sample (Haimson 1973), Eq.(5') for the crack reopening pressure is used to determine the rock stress together with Eq.(6).

It has to be indicated that Eq.(5') and Eq.(6) are valid when the deviation of direction of a principal stress from the axis of borehole is not evident and the plane of generated fracture

Table 6 Observed Critical Pressures in Hydrofracturing

Position of measurement	Break-down pressure Pb(MPa)	Crack reopening pressure Pb'(MPa)	shut-in pressure Ps(MPa)	Orientation of crack
9.88-10.20m deep in borehole A	12.65		4.06 (0.12)*	see Fig.6
4.84-5.16m deep in borehole B	8.46	5.19 (0.04)	4.82 (0.13)	16-20° from plumb line to west
7.22-7.54m deep in borehole C		7.14 (0.12)	4.90 (0.0)	5-10° from North to west

\* Standard deviation is given in parentheses

crack includes the axis of borehole.

Substituting data shown in Table 6 into Eq.(5') and (6), we can determine the major and minor principal stresses in the plane normal to the axis of borehole B and C respectively, because the generated cracks are nearly parallel to the axis of hollow cylindrical rock core for both of borehole B and C. The results are shown in Fig.5.

However, the generated fracture crack of borehole A is nearly vertical, but intersects obliquely the axis as illustrated in Fig.6. In this case, Eq.(5') would not be valid, but we can apply Eq.(6) to determine the minor principal stress, under the assumption that the generated crack is normal to the minor principal stress.

Let coincide x' axis of local coordinate system to the direction normal to the generated crack, then we have

$$\tau_{x'y'} = \tau_{z'x'} = 0 \quad \text{---(7)}$$

regardless of the direction of y' and z' axis. When z' axis coincides with z axis of the common coordinate system, applying Eqs.(6),(7), and the formula for transformation of coordinates, we have

$$\left. \begin{aligned} \sigma_{x'} &= \sigma_x \cos^2 \theta + \sigma_y \sin^2 \theta - 2 \tau_{xy} \sin \theta \cdot \cos \theta = P_s \\ \tau_{x'y'} &= (\sigma_x - \sigma_y) \sin \theta \cdot \cos \theta + \tau_{xy} (\cos^2 \theta - \sin^2 \theta) = 0 \\ \tau_{z'x'} &= -\tau_{yz} \sin \theta + \tau_{zx} \cos \theta = 0 \end{aligned} \right\} \quad \text{---(8)}$$

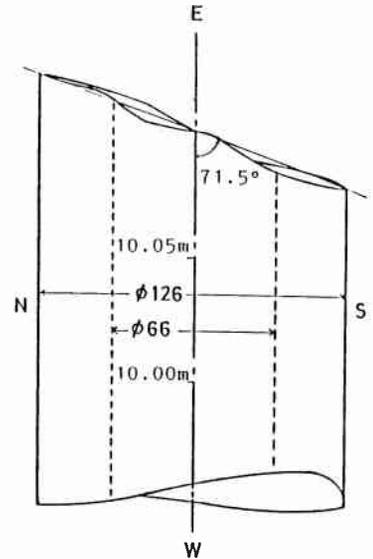


Fig. 6. Schematic View of Oblique Crack in the Borehole A.

where  $\theta$  is the angle between x and x' axes.

On the basis of the major and minor principal stresses determined for borehole B and C, we can determine 3 components of the rock stress as

$$\left. \begin{aligned} 2 \tau_{zx} &= (\sigma_{Hmin} - \sigma_{Hmax}) \cdot \sin 2\theta \\ 2 \sigma_{xx} &= (\sigma_{Hmin} + \sigma_{Hmax}) + (\sigma_{Hmin} - \sigma_{Hmax}) \cos 2\theta \\ 2 \sigma_{zz} &= (\sigma_{Hmin} + \sigma_{Hmax}) - (\sigma_{Hmin} - \sigma_{Hmax}) \cos 2\theta \end{aligned} \right\} \quad \text{---(9)}$$

for the borehole B, and

$$\left. \begin{aligned} 2 \tau_{xy} &= (\sigma_{Hmin} - \sigma_{Hmax}) \sin 2\theta \\ 2 \sigma_{xx} &= (\sigma_{Hmin} + \sigma_{Hmax}) + (\sigma_{Hmin} - \sigma_{Hmax}) \cos 2\theta \\ 2 \sigma_{yy} &= (\sigma_{Hmin} + \sigma_{Hmax}) - (\sigma_{Hmin} - \sigma_{Hmax}) \cos 2\theta \end{aligned} \right\} \quad \text{---(10)}$$

for the borehole C, respectively, where  $\theta$  is the angle between x axis and the normal to the generated crack i.e. the direction of minor principal stress  $\sigma_{Hmin}$ .

These 9 equations in Eqs.(8) (9) and (10) are the observation equations for determining the complete state of stress.

$\sigma_{zz}$ ,  $\tau_{zx}$  and  $\tau_{yz}$  can be definitively determined, because 6 among 9 equations don't include these 3 components. However,  $\sigma_{xx}$ ,  $\sigma_{yy}$ ,  $\tau_{xy}$  should be determined by means of the least square method. The results are shown in Table 5.

#### 4. DISCUSSION ON VARIOUS SOURCES OF THE ERROR OF MEASUREMENT.

Most of differences between each component of rock stress determined by borehole deformation gauge and hydrofracturing are smaller than their standard deviations. It means that the test results shown in Table 5 are not satisfactory but reasonable.

Such a low accuracy of rock stress measurement as suggested by the standard deviation of several MPa would be caused from the following reasons:

(1) The distance between the gauge position and rock wall of the roadway is about 2.5m fore borehole A. Considering the cross-section of the roadway of 2.5m x 2.5m, the gauge position would be affected by the stress concentration around the underground opening.

(2) Although the accuracy of borehole deformation gauge is  $2\mu\text{m}$ , the borehole deformations induced by the stress relief are sometimes smaller than  $2\mu\text{m}$ , as shown in Table 4. This would be the most important reason of the low accuracy of the present measurement.

(3) Boreholes A, B and C have been drilled from the same position of the roadway in three different directions perpendicular to each other by some technical reasons. The gauge position in three different boreholes is distant more than 10m each other. It means that the mechanical properties of rock mass as well as the state of stress might be different from a gauge position to other gauge positions. The remarkable difference between each mean value and the standard deviation of Young's moduli shown in Table 3 suggest the fluctuation and/or anisotropy of mechanical properties of rock mass.

#### 5. STATE OF STRESS IN KANTO-TOKAI REGION

The stress field derived from rock stress measurements in Kanto-Tokai region (Fig.3) is generally consistent with the plate tectonics model in spite of all possible measurement errors and topographic disturbances. The horizontal maximum compressions in the generally east-west direction in the northern part of Kanto area are likely to be the effect of pinching between the Pacific plate and the Eurasia continent. The horizontal maximum compressions in the north-west-southeast direction and in the nearly north-south directions are distinguished in the southern Kanto and Tokai areas.

The nearly north-south compressions are observed in some measurement points relatively close to the plate boundaries. The measured state of stress is rather abnormal in many sites under the nearly north-south compression. The abnormally high horizontal compression in the nearly north-south direction which was measured in the shallow borehole in the Tanzawa area coincides with geologic features such as the east-west trending anticlines and thrust faults. The rapidly uprising Tanzawa mountain is the area of stress concentration due to the thrust of the Philippine Sea plate.

On the other hand, the extremely low horizontal maximum compression in the east-west direction indicates the horizontal extensional state of stress at the Toi area in the north-western Izu peninsula. The east-west horizontal extension in the northwestern Izu peninsula, which is also indicated by north-south trending normal faults and tension fractures in young gold-bearing vein systems, is the effect of bending of the Philippine Sea plate which is subducting under the Eurasia plate along the Suruga trough (Nakamura, 1981).

The stress is rather low at the Shibakawa site, west of Mt. Fuji., which locates on the northeast-southwest trending thrust fault with left-slip component, which is one of northern extensions of the Suruga trough.

The results of rock stress measurements suggest that the general orientation of horizontal maximum compression is northwest-southeast in the southern

Kanto and Tokai areas, but that the stress field is disturbed to form areas of stress concentration or stress release, where the horizontal maximum compression tends nearly north-south.

Disastrous major earthquakes occur along the plate boundaries in the southern Kanto and Tokai areas such as the Great Kanto earthquake along the Sagami trough and the forthcoming Tokai earthquake along the Suruga trough. The areas of stress disturbance, where the direction of horizontal maximum compression is nearly north-south, are directly affected by the plate interaction and possibly make the most sensitive and critical areas for prediction of major earthquakes in Kanto-Tokai region.

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