DESTRESS BLAST TESTING AT SIGMA MINE:
Experimentation and results

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DESTRESS BLAST TESTING AT SIGMA MINE:
Experimentation and results[1]

by

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SUMMARY

An experimental destress blast was carried out at Sigma Mine during January 1996, in order to find a way to decrease the stresses induced by mining in loaded areas. The blast was performed in the sill pillar of a stope located on level 34, about 1500 metres underground. Geophysical and geomechanical surveys were made in the pillar and at the ends of the stope, before and after the blast, in order to determine the efficiency of the blast.

This report presents the experiment and the blast parameters. The blast pattern and the explosives used are specified. The results are presented and discussed.

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1 KEYWORDS: Rockburst, violent failure, destress blast, geophysical survey, structural surveys, stress determination, stress monitoring, dilatometer testing, in situ modulus of deformation, laboratory testing, compressive strength, modulus of elasticity, scale effect.

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ESSAI DE DYNAMITAGE DE PRÉFRACTURATION À LA MINE SIGMA:

CONTEXTE DE L'EXPÉRIMENTATION ET RÉSULTATS OBTENUS[1]

par

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SOMMAIRE

Un essai de dynamitage de préfracturation a été effectué à la mine Sigma au mois de janvier 1996, pour examiner une façon de réduire les contraintes induites par les excavations dans des secteurs fortement chargés. Le sautage a été effectué dans le pilier de protection d'un chantier situé au niveau 34, à environ 1 500 mètres sous terre. Des relevés géophysiques et des essais géomécaniques ont été effectués à l'intérieur du pilier et aux extrémités du chantier avant et après le tir pour déterminer l'efficacité du sautage.

Cette présentation fait le point sur le contexte de l'expérimentation et les paramètres du tir. Le patron de forage et les types d'explosif et de chargement utilisés sont spécifiés. Les résultats de mesure obtenus sont présentés et discutés.

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1 MOTS CLÉS: Coup de terrain, rupture violente, tir de préfracturation, levé géophysique, relevés structuraux, détermination de contrainte, suivi de contrainte, essai au dilatomètre, module de déformation en place, essai en laboratoire, résistance en compression, modules de déformation élastique, effet d'échelle.

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SOMMAIRE

Un essai de dynamitage de préfracturation a été effectué à la mine Sigma au mois de janvier 1996, pour examiner une façon de réduire les contraintes induites par les excavations dans des secteurs fortement chargés. Le sautage a été effectué dans le pilier de protection d'un chantier situé au niveau 34, à environ 1 500 mètres sous terre. Des relevés géophysiques et des essais géomécaniques ont été effectués à l'intérieur du pilier et aux extrémités du chantier avant et après le tir pour déterminer l'efficacité du sautage.

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INTRODUCTION

A destress blast test was carried out at the Sigma Mine, property of Placer Dome Canada Ltd., on January 20, 1996. The objective of the test was to take stock of that technique, as a means of reducing the risk of rockbursting in areas susceptible to violent failure.

This violent failure phenomenon in deep hard-rock mines is a well known phenomenon. It is present in many mines in Québec and Ontario (Hedley, 1992 [1]). However, this phenomenon does not limit itself only to deep mines. More generally, this phenomenon occurs in competent ground when the level of induced stresses exceeds the *in situ* strength of the rock. While it is possible to affirm, without too much fear of contradiction, that all rockbursts are the result of excess stresses, the nature of the phenomenon is nevertheless very diversified.

Rockbursts are grouped two ways: those associated to slippage along natural structural planes, and those associated with high strain energies inside less fractured rockmasses. The damage resulting from rockbursts can be very severe, depending of the vicinity of the hypocentre compared to the openings and the energy released by it.

The rockbursts observed at Sigma Mine are usually part of the second group, those associated with high strain energies. One of the factors generally invoked to explain them is the geometry of mine openings - the presence of isolated structures inside mining zones increase the risk of rockbursts. Isolated structures concentrate stresses and are subject to fail violently -. The problem is compounded by the different properties of geological units of the mining area. The most rigid units accumulate energy which adjacent units are unable to absorb, when released. The seismic events produced by the release of energy create most of the damage observed on the face of openings.

There are not many ways of reducing the potential of violent failure in heavily loaded mining structures. The simplest solution could consist in avoiding these difficult situations, by mining uniform faces and towards the outside of the orebody, to get away from the main infrastructures. Destress blasting could also be the only alternative left within these conditions. The objectives of destress blasting is to reduce the level of stresses observed in loaded areas, and to move these outside the mining zone by modifying the bulk properties of these areas.

Indeed, the increase of fracturation produced by blasting allows a reduction of rigidity and of fragility of the material. As a consequence, the capacity for the material to concentrate stresses will be reduced, its ability to fail violently as well. Its strength will be reduced, and stresses the material can support as well. Should the occasion arise, the excess stresses will be transfered to the ends of the stope, where the level of stress will be lower and the damage resulting of an eventual failure will be less critical. Experiments of that nature were conducted in the past, some succeeded, others were less successful (Hedley, 1992 [1], Makuch et al., 1987 [2], ROCTEST, 1980 [3]).

THE EXPERIMENT

The test carried out in January 1996 was to verify the efficiency of this technique. This technique could then be used to reduce the potential of violent failure prevalent in loaded mining structures, and to limit damage associated to stronger bursts. The test was carried out at approximately 1500 metres underground, on the 34 level, in the sill pillar of the abandoned 3420E stope. The site is located within the P-Zone, about 60 metres from the main cross-cut and 150 metres from No. 3 shaft. Longitudinal sections of the P-Zone and of the 3420E stope are shown on Figures 1 and 2.

The 3420E stope is located inside a sheared zone orientated east-west. The zone dips at 55 degrees towards the south, typical of mineralized zones at Sigma Mine. The walls are porphyritic diorite relatively homogenous, more or less fractured at the contact with the mineralization. The zone contains some quartz-tourmaline veins. Some secondary minerals, such as pyrite, chalcopyrite, chlorite and carbonatite, are also present with the mineralisation. The contact with the walls is concordant. Further details on the mine geology and the mineralization at Sigma Mine are given in Robert et al. (1983 [4]).

Structural surveys and geocharacterization have been carried out in the 3416E and 3420E drifts, to study the fracturation in the vicinity of the stope. The mean orientations and the average spacing of discontinuities are shown on Table 1. Generally, two families of discontinuities are present, with an orientation parallel and perpendicular to the ore zone. A third family, intermediate to the first two, is sometimes present. The spacing of discontinuities varies between 0.35 and 3.0 metres. The indices of quality of the rockmass have been calculated. Results achieved are shown on Table 2. The RMR is slightly above 80, the Q index varies between 1.6 and 3.3. The discontinuities for the 3416E drift have been plotted on stereonet which is shown on Figure 3. A photograph of the fracturation is shown on Figure 4.

THE DESTRESS BLAST

The destress blast was limited to the western portion of the sill pillar located at the bottom of the 3420E stope, between the sections 5 335 E and 5 390 E (see Figure 2). The volume to fracture was about 450 m^3, or 16 000 ft^3. The drilling pattern, the types of explosive and loading, and the blasting sequence used are detailed in the following paragraphs.

The drilling pattern

Fifteen (15) holes of 38.1 mm (1½ inch) diameter and 8.5 m (28 ft) long were drilled in the back of the 3420E drift, inside the sill pillar. The holes were drilled along the vein at an angle of 55 degrees towards the top. The holes were alterned on two lines spaced 1.2 m (4 ft) apart. One line was drilled near the contact with the north wall, the other at the centre of the drift. Holes were drilled every 0.9 m (3 ft) in the direction of the drift. The drilling pattern was selected to create an optimal angle for fractures, at 45 degrees with the orientation of discontinuities. The location of drillholes is shown on Figure 5.

The loading of holes and their detonation

The holes were loaded pneumatically with AN/FO bulk explosive, for a length of 5.5 m (18 ft). The choice of explosive was based on its high gas energetic partition. The remaining 3 m (10 ft) were filled with fast grip cement cartridges, in order to maximize the time of gas retention. The cartridges were inserted into the holes with a stuffer. A plastic cover and a jute stemming were put in place prior to the insertion of cement, to avoid any reaction between the ANFO and the cement. The distance with cement was deeper than the damage zone created by the advance of the drift. This zone is usually less than 1 m deep. The powder factor of the blast was 0.2 kg/m^3 or 0.15 lb/t.

Two 15 metre long and zero delay non-electric detonators were inserted in each hole. This allowed an almost instantaneous initiation of the blast. Cartridges of dynamite of 25*200 mm were used to ensure an effective detonation of the ANFO. The detonators were connected to a detonating string fired electrically. The pattern of holes loading is shown on Figure 6.

The blast sequence and the damage observed

All the holes were initiated at the same time in order to obtain the maximum power from blasting. A recording of the blast was made with far-field sensors installed to monitor the seismic activity at Sigma Mine. The recording is shown on Figure 7. No attempt has been made to correlate the recording with the power of the blast. The factors to do so are unknown or too imprecise to conclude in a valid way on the subject. The magnitude of the event determined from the analysis of waveforms was estimated at 1.0 m_N, or in terms of energy, at 1.84 kJoules at 350 m from the blast. It is deemed more appropriate to examine the signature of the blast and the length of the event. Only one P wave and one S wave were recorded. The pulse was relatively short, less than 150 msec. This demonstrates that all the charges went at the same time, and that the test utilized the maximum energy available from blasting.

Inspection of the site after the blast showed that the cement had been blown from the holes with the explosion. Therefore, some energy might have been lost although the amount is hard to evaluate. Support was reenforced before the blast. All production and haulage drifts at Sigma Mine are supported with standard rock bolts and resin bolts. Metals straps and 12 gauge screen are used in stopes area. A view of the 3420E drift before the blast is shown on Figure 8a.
The 3420E drift after the blast is shown on Figure 8b. More important damages were observed around the 5 360 E section, where a bay had been made for drilling. Elsewhere, the damages were relatively minor, the power of blast and the proximity of the walls being considered. The damage was limited to the screen cut up by the blast, and to a light spalling of the walls.

GEOPHYSICAL SURVEYS AND GEOMECHANICAL TESTING

Technical surveys were carried out before and after the blast to measure the success of the experiment and the impact of the blast on the properties of the rock in the area. Properties investigated were: 1) the velocity of the seismic waves, affected by the level of stresses and the character of fractures in the milieu; 2) the size of total stresses and their evolution with time, inside the pillar and at each end of the stope; and 3) the *in situ* deformability of the rock material, determined at the same location. The strength and the elastic moduli of the rock material were determined in the laboratory. The surveys carried out and the results obtained are presented in the following paragraphs.

The geotomographic survey

Geotomography is a geophysical technique to estimate the properties of a milieu by analysing the velocity profiles of waves propagating that milieu (Young et al., 1987 [5]). The homogeneity of the milieu, the modulus of elasticity, the level of stress, the fracturation of the rock, or all other properties that could be correlated to velocity are among properties estimated usually with this technique. The sensitivity of the method and the difficulty to calibrate the profiles determined with the properties of the rock constitute the main limitations of the technique. Nevertheless, even if the absolute value of rock properties cannot be be determined with accuracy with this technique, the method could be very useful to investigate changes in properties or their evolution over time. This was attempted within the present test program.

Two (2) series of holes were drilled in the vicinity of the pillar. A first series of holes were drilled at an angle of 50 degrees upwards on the north wall of the drift to insert the transmitters. A second series were drilled straight up in the back near the south wall to fix the receivers. The holes were spaced 0,9 m (3 ft) apart. Their length were 6,1 and 2,1 m (20 and 7 ft) respectively, for angle and vertical holes. The position of the transmitters and the receivers defined a plane almost horizontal located at about 2 m (6,6 ft) over the drift, inside the pillar. An isometric view of the 3420E drift and the holes drilled are shown on Figure 9.

The transmitters were electric seismic detonators. They were placed in the holes of the north wall at the desired depth. Only one detonator was initiated at a time. The resulting wave was received by all the sensors located on the opposite line. The time of arrival of the wave at each receiver was recorded. A series of shots were carried out from each transmitting hole to cover all the surface of the plane. The velocity of the waves in the milieu was calculated. The velocity profiles determined before and after the blast are shown on Figures 10a and 10b.

The determination and monitoring of *in situ* stresses

The stresses were determined in three (3) specific locations, in the middle of the pillar and at both ends of the stope. The method used was the *doorstopper* method, carried out inside B holes. The determination of the complete stress tensor requires drilling three holes at each location, converging towards the measuring point. This point was located at a depth of about 4 m (13 ft) from the back of the drift, following the dip of the orebody. A hole layout is shown on Figure 11.

The rock samples recovered from the tests were reloaded in laboratory to determine the specific elastic moduli of the samples. The results of the stress measurements were interpreted by refering to the linear elasticity theory for homogenous materials. The size and orientations of the stresses, determined before and after the blast, are shown on Tables 3a and 3b. The magnitude of the mean and the deviatoric stresses at each location were also calculated. The results are also shown.

The maximum stress was determined before the blast, with a value of 104 MPa, and was located at the centre of the pillar . The stress values at the ends of the stope were 34 and 64 MPa, at west and east end respectively. After the blast, the average stress determined inside the pillar decreased to 59 MPa. At both west and east ends, the stresses were 42 and 19 MPa respectively. These values have to be examined in their real perspective, keeping in mind the experimental nature of the measurement and the variability that could be observed on the rock in place.

Three (3) triaxial stress cells were installed inside N holes drilled close to the B holes. These cells allowed the monitoring of stress changes in the sector during the whole period of experimentation. A slight modification of the stress regime was recorded during that period, as a consequence of a major rockburst that occurred in November 1995 at a lower level. A change of 1 or 2 MPa, mainly along the borehole axis, was observed then. Also, even if results are mitigated, the cells showed a slight change in the stress level following the destress blast. A reduction of 2.3 MPa was observed along the borehole axis at the east end of the site. The cell located inside the pillar was destroyed by the blast. As for the unit installed at the west end of the pillar, its position was far from optimal and no significant change was recorded. This unit was installed inside the damaged zone of the drift, less than one meter inside the borehole. A crack with lateral movement prevented the installation of the unit beyond that distance. The results obtained are shown on Table 4. The reaction curves of the cell installed at the east end of the site - borehole N4 - are shown on Figure 12.

The measurement of ***in situ*** deformability

Dilatometer tests were carried out before and after the blast in N size holes, to determine the *in situ* modulus of deformation of the rock material and to verify the effect of blasting on its value. The N size holes had been drilled at the beginning of the field campaign, to provide samples for testing in the laboratory and to allow the installation of the triaxial sensors. The boreholes were inspected with a geocamera to determine their general conditions. The inspection revealed the presence of many breakouts within the boreholes - these breakouts are an indication of a high stress level, higher than 50% the nominal strength of the rock -. The borehole N1 had a major displacement at the collar and then could not be used for testing. A second hole - borehole N1a - was drilled next to the original one after the blast. Borehole N2 was destroyed by the blast. A second hole - borehole N2a - was also drilled in replacement of the previous one after the blast. Borehole N3 showed severe damage - breakouts, spalling, etc - too. Some tests were carried out in this borehole only after the blast.

Nevertheless, when conditions proved to be acceptable for testing, according to the conditions of the walls, dilatometer tests were attempted in boreholes. A chart of the dilatometer probe is shown on Figure 13 to illustrate its working. The probe is introduced in the borehole at a chosen depth, and the membrane is inflated to the desired pressure. The volume of fluid injected to inflate the membrane is measured. The volume of fluid is a measure of the deformability of the rock material. Pressure is increased and the volume of fluid injected is measured accordingly. The complete curve pressure versus deformation of the rock material for the depth chosen is then known. The interpretation of results is done in accordance with the theory of linear elasticity for isotropic and homogenous material. Testing was repeated twice, to verify the effect of reloading on rock behavior. The rock showned no hardening. This confirms the purely elastic behavior of the rock. The results are shown on Table 5.

The *in situ* modulus of deformation averages 55 GPa. The results achieved after the blast do not show any major change with those achieved before the blast. The difference is about 10%. A difference of 7% was observed between boreholes N2 and N2a, and an increase of 11% within borehole N4. The deformability profiles plotted for boreholes N2 and N2a are shown on Figure 14.

The properties of the rock material

Compressive tests were made in the laboratory on samples recovered from the N size boreholes, to determine the strength and the elastic moduli of the rock material. The results were extrapolated to the scale of rockmasses, to take into account the effect of volume on rock properties. Extrapolation was made by using correction factors suggested usually in literature. Results are shown on Tables 6 and 7. Interested readers will see references given at the bottom of tables for details on the significance and the determination of the parameters displayed on the tables. More important are the values of strength and moduli determined for the rock material. The uniaxial compressive strength varies between 110 to 230 MPa, depending on the characteristics of the material tested. The modulus of elasticity of the rock generally ranges from 50 to 90 GPa, for the same rock types. This value decreases with the scale of testing, or with the volume of rock affected by the test or considered in the interpretation. Results of dilatometer tests show intermediate values, included between those determined in the laboratory on rock samples and those determined empirically for the rockmass.

THE NUMERICAL ANALYSIS

The surveys made and the results presented would be incomplete without reiteration of the work objective and the reasons of using destress blasting. It has been said previously that this technique could be used successfully to limit or reduce stresses in mine structures located in heavily loaded ore zones. Also, the risk of violent failure will be reduced or limited. However, the use of this technique also implies that to limit or reduce stresses in a given structure, adjacent structures will have to be loaded beyond their actual level. Ideally, this technique should not be used before having made a stability analysis of the zone investigated, and that the changes produced in a given structure will not produce an excess of stress in the adjacent ones.

Simon et al.,1995 [9], have proposed a methodology to estimate the potential of rockbursting of a structure in relation with the characteristics of the zone including that strucure. The authors suggest to perform an analysis of stability of the structure - using one of the softwares available on the market per se, based on numerical analysis techniques -. If stresses exceed the level allowed for that structure, the stiffness of the structure is compared with the stiffness of the zone and the potential of violent failure is rated. This methodology has shown promising results in the past. This methodology was used in parallel with the preparation of the present experiment. The results of analysis will be published soon.

DISCUSSION AND CONCLUSION

This document describes an experiment carried out at Sigma Mine to investigate a technique for reducing or limiting stresses in some heavily loaded mine structures. Not constrained, stresses most often lead to a violent failure phenomenon. This phenomenon called rockburst constitutes a risk for serious accident in deep hard rock mines. It has to be controlled, if not eliminated. The risk is high both for personnel and for operations. It causes loss of production and damage that need costly rehabilitation.

A destress blast was carried out on January 20, 1996, within the 3420E stope sill pillar, at approximately 1500 m underground. The volume of rock involved by the experiment was about 450 m^3. The drilling pattern and the types of explosive and loading were selected to maximize the energy available for the blast, and to increase the degree of fracture of the rock while maintaining the integrity of the mine structure. Geophysical surveys and geomechanical testing were conducted prior to and after the blast to evaluate its efficiency. The results are presented.

The results show the difficulty in measuring the efficiency and the degree of success of an experiment carried out in a practical context. The objective of destress blasting is to reduce the risk of violent failure in mine structures by reducing their rigidity - or stiffness - and their fragility - or brittleness -, and by redistributing the stresses in the adjacent zones. Unfortunately, the precision of the instruments used and the nature or the variability of the rock properties - these are intrinsic characteristics of rock materials - do not allow a decision to be made on the efficiency of the blast. The monitoring of seismic activity in the sector during the next few months could add more information. Some compressive tests on the rock material that would be sampled after the blast, to characterize the behaviour of the fractured material, could also provide interesting results.

Destress blasting does not constitute a solution to all cases of heavily loaded structures prone to violent failure. Once all possibilities of adjustment by changing stope design or mining sequence are exhausted, one of the remaining alternatives left to mine engineers is to try to reduce the potential of violent failure of heavily loaded mine structures by relocating excess stresses towards areas less loaded or less critical for the safety of workers and operations.

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TABLES

Table 1. Mean orientation and average spacing of discontinuities.

DRIFT	SCANLINE	MEAN ORIENTATION			
		Number	Direction (°)	Dip (°)	Average spacing (m)
3416 E	All	1	014	86	1.33
		2	103	74	0.78
		3	248	78	0.32
3420 E	All	1	033	90	2.82
		2	094	79	0.35

Table 2. Rockmass quality indices (surveys done on Level 34).

IDENTIFICATION			QUALITY INDICES				
Drift	Scanline	Geological Unit	λ^* (#/m)	RQD	RMR	Q	Q'
3416 E	All	Type C	1.54	99	81	1.6-2.2	16.5-22.0
3420 E	All	Type C	1.33	99	82	1.6-3.3	16.5-33.0

* Number of fractures per metre.

Table 3a. Principal stresses and average stress level before the blast.

SITE	σ_1		σ_2		σ_3		σ_{moy} [1] (MPa)	$\tau_{dév}$ [2] (MPa)
	Mag. (MPa)	Az/Dip [3] (°/°)	Mag. (MPa)	Az/Dip (°/°)	Mag. (MPa)	Az/Dip (°/°)		
Site 1	55.0	057/14	46.9	146/-05	1.8	036/-74	34.5	23.4
Site 2	134.0	002/-56	92.3	018/-38	86.6	101/09	104.3	21.1
Site 3	77.7	162/-24	60.7	081/20	54.6	025/-57	64.3	9.7

Notes
[1] Mean stress.
[2] Deviatoric stress.
[3] Negative dip (-) towards down.

Table 3b. Principal stresses and average stress level after the blast.

SITE	σ_1		σ_2		σ_3		σ_{moy} [1] (MPa)	$\tau_{dév}$ [2] (MPa)
	Mag. (MPa)	Az/Dip [3] (°/°)	Mag. (MPa)	Az/Dip (°/°)	Mag. (MPa)	Az/Dip (°/°)		
Site 1	64.4	158/-30	38.4	050/-28	24.8	106/46	42.5	16.4
Site 2	77.6	173/-86	54.1	029/38	47.6	107/-14	59.7	12.8
Site 3	32.6	022/21	21.2	115/07	4.4	042/-67	19.4	11.5

Notes
[1] Mean stress.
[2] Deviatoric stress.
[3] Negative dip (-) towards down.

Table 4. Stress changes determined from the triaxial cells recordings.

BOREHOLE	SENSOR[1] (Axis)	VARIATION		STRESS CHANGE[2] (MPa)
		Frequency (Hz)	Displacement (mm)	
N1	NS	-14	-0.00113	-0.32
	V	0	0.0	0.00
	EW	-4	-0.00035	-0.19
N2	NS	-	-	-
	V	-	-	-
	EW	-	-	-
N4	NS	0	0.0	-0.35
	V	2	-0.00797	-2.27
	EW	2	0.00016	-0.32

Notes

[1] Sensor orientation **NS** Axis, **V**ertical, **EW** Axis.

[2] Young's modulus$_{N1}$: 50.34 GPa, Poisson's ratio$_{N1}$: 0.19.
Young's modulus$_{N2}$: 39.54 GPa, Poisson's ratio$_{N2}$: 0.26.
Young's modulus$_{N4}$: 51.32 GPa, Poisson's ratio$_{N4}$: 0.32.

Table 5. Modulus of deformation values determined from dilatometer tests.

IDENTIFICATION		MODULUS of DEFORMATION			
Test	Depth (m)	Before the blast		After the blast	
		E (GPa)	ν	E (GPa)	ν
N1a.1	9.02	-	-	63.27	0.30
N1a.2	6.77	-	-	-	-
N1a.3	5.02	-	-	-	-
N1a.4	3.82	-	-	67.08	0.30
N2.1	8.37	54.18	0.26	-	-
N2.2	7.72	68.47	0.26	37.69 *	0.26
N2.3	6.97	-	-	62.52 *	0.26
N2.4	5.42	57.62	0.26	44.72 *	0.26
N2.5	4.62	39.54	0.26	-	-
N2.6	4.02	65.95	0.26	61.52 *	0.26
N2.7	2.32	54.54	0.26	58.08 *	0.26
N2.8	1.87	-	-	-	-
N3.1	7.77	-	-	59.60	0.30
N3.2	5.87	-	-	55.47	0.30
N3.3	4.02	-	-	66.60	0.30
N4.1	7.82	-	-	-	-
N4.2	7.07	55.11	0.32	-	-
N4.3	6.47	-	-	72.04	0.32
N4.4	5.77	47.53	0.32	50.24	0.32
N4.5	4.27	53.25	0.32	66.11	0.32
N4.6	3.52	67.59	0.32	-	-
N4.7	2.77	58.23	0.32	-	-

Notes
E *In situ* modulus of deformation.
ν Poisson's ratio, from laboratory.
* Tests carried out in borehole N2a.

Table 6. Rock and rockmass strength according to the Hoek and Brown criterion.

SAMPLE	MATE-RIAL	RMR	PARAMETERS of the CRITERION[1]				
			s_c[2] (MPa)	Rock material		Rock mass	
				m_i[3]	s_i[3]	m_m[3]	s_m[3]
N1.1	C alt.	73	113.80	5.96	1.0000	2.27	0.0498
N1.2	Type C	82	137.22	12.07	1.0000	6.34	0.1353
N1a.1	Type C	80	205.68	16.12	1.0000	7.89	0.1084
N1a.2	Type C	85	282.46	26.66	1.0000	15.60	0.1889
N2.1	Type C	84	229.72	27.33	1.0000	15.43	0.1690
N2a.1	Type C	83	228.43	11.74	1.0000	6.39	0.1512
N2a.2	Type C	78	146.68	18.21	1.0000	8.30	0.0868
N3.1	Qz-To	79	179.58	17.77	1.0000	8.38	0.0970
N4.1	Qz-To	77	122.81	19.89	1.0000	8.74	0.0776
N4.2	Type C	79	224.58	16.07	1.0000	7.59	0.0970

Notes
1 See Hoek and Brown, 1988 [6], for more information on constants.
2 Average unconfined compressive strength.
3 Constants of the Hoek and Brown criterion, $_i$ for rock material, $_m$ for rock mass.

Table 7. Moduli of deformation determined for the rock material and the rockmass .

IDENTIFICATION				MODULUS of DEFORMATION				
Sample	Material	RQD	RMR	E_l[1] (GPa)	E_d[2] (GPa)	$E_{m.RQD}$[3] (GPa)	$E_{m.RMR}$[4] (GPa)	ν[5]
N1.1	C alt.	69	73	45.66	-	9.01	16.88	0.16
N1.2	Type C	100	82	50.34	-	50.34	25.94	0.19
N1a.1	Type C	94	80	60.18	67.08	50.55	28.82	0.31
N1a.2	Type C	97	85	78.97	87.30	72.65	45.44	0.36
N2.1	Type C	98	84	90.18	65.95	85.36	50.02	0.25
N2a.1	Type C	89	83	71.36	58.08	50.42	38.16	0.31
N2a.2	Type C	90	78	56.01	44.72	41.07	24.92	0.21
N3.1	Qz-To	90	79	81.78	55.47	59.97	37.75	0.46
N4.1	Qz-To	84	77	92.19	58.23	52.85	39.53	0.33
N4.2	Type C	84	79	74.85	67.59	42.91	34.55	0.30

Notes
[1] Young's modulus, from the laboratory.
[2] *In situ* modulus of deformation, from dilatometer tests.
[3] *In situ* modulus of deformation, function of RQD (Bieniawski, 1978 [7]).
[4] *In situ* modulus of deformation, function of RMR (Nicholson and Bieniawski, 1990 [8]).
[5] Poisson's ratio, from the laboratory.

FIGURES

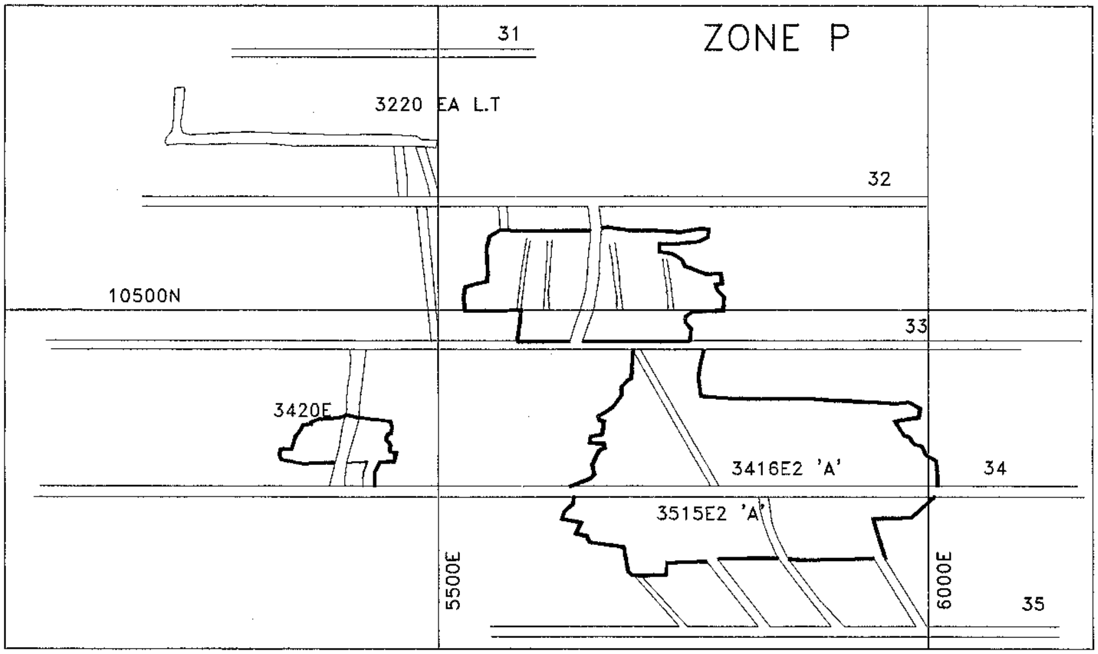


Figure 1. Longitudinal section of the zone P.

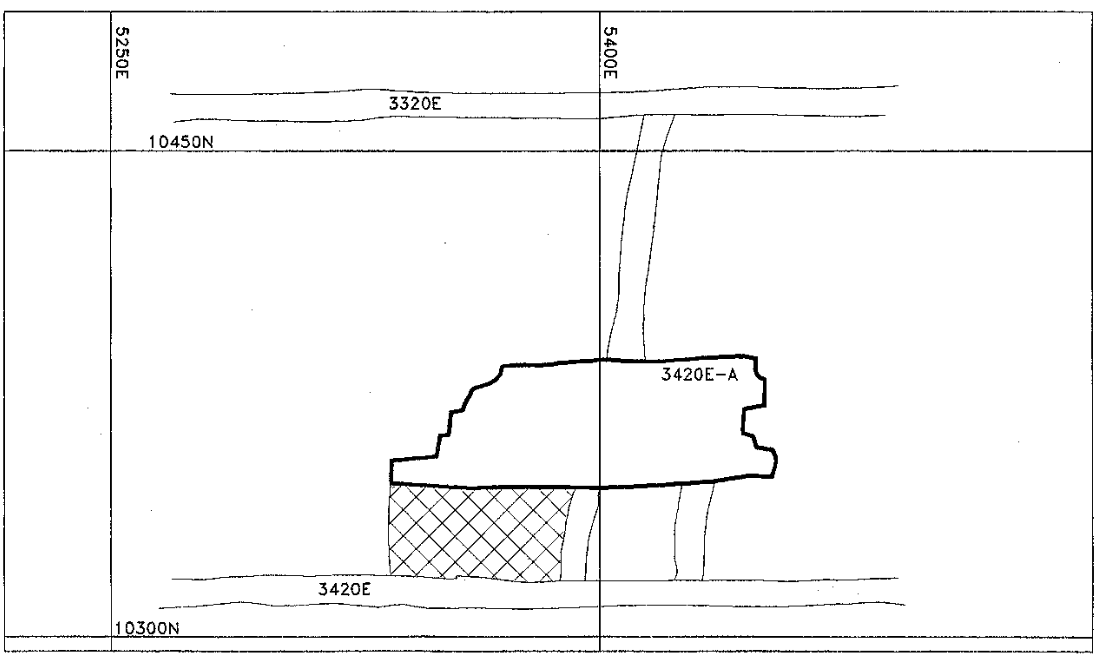


Figure 2. Longitudinal section of the stope 3420E.

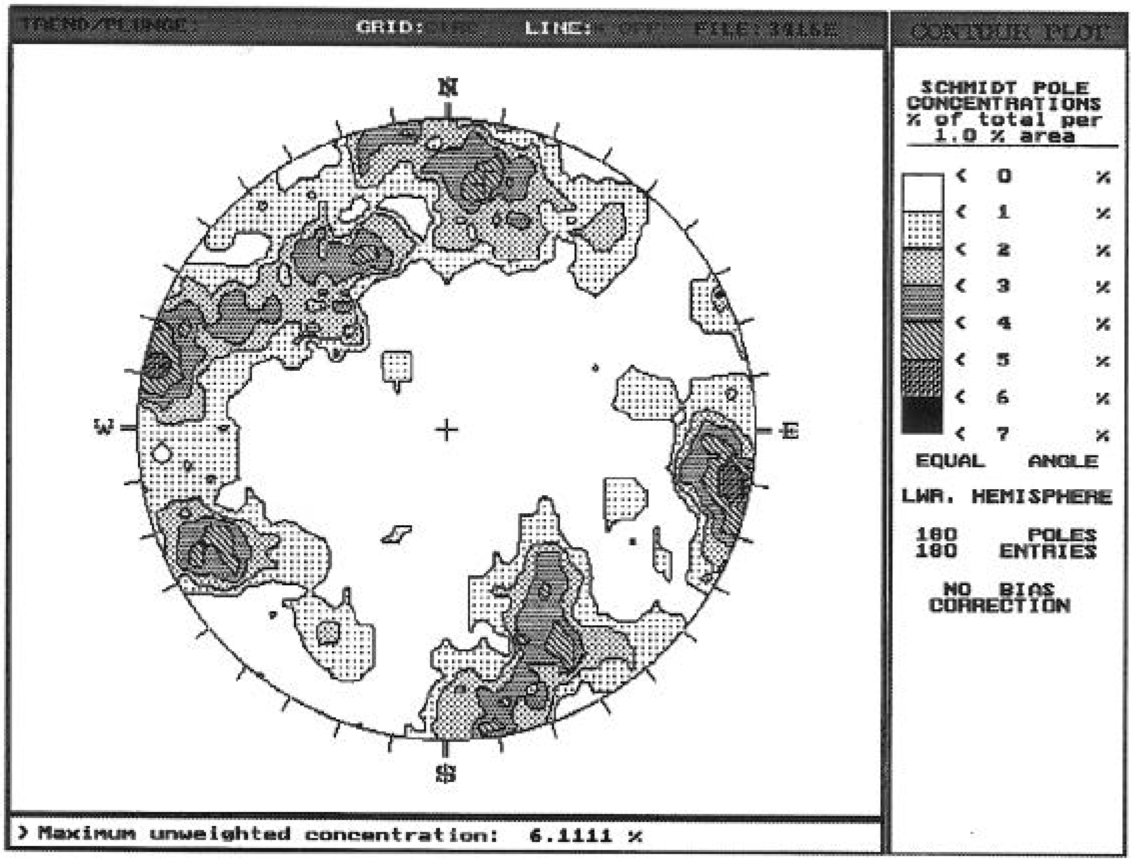


Figure 3. Graphical projection of the discontinuities from the drift 3416E.

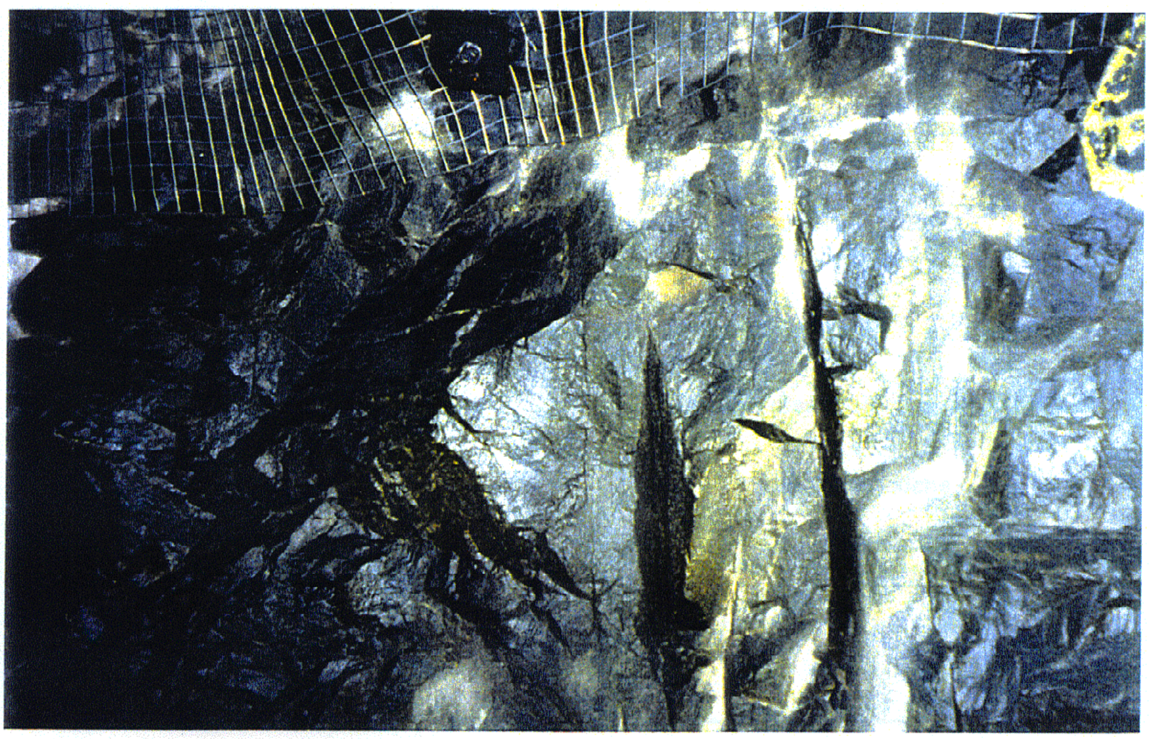


Figure 4. Fracturation observed in the drift 3416E.

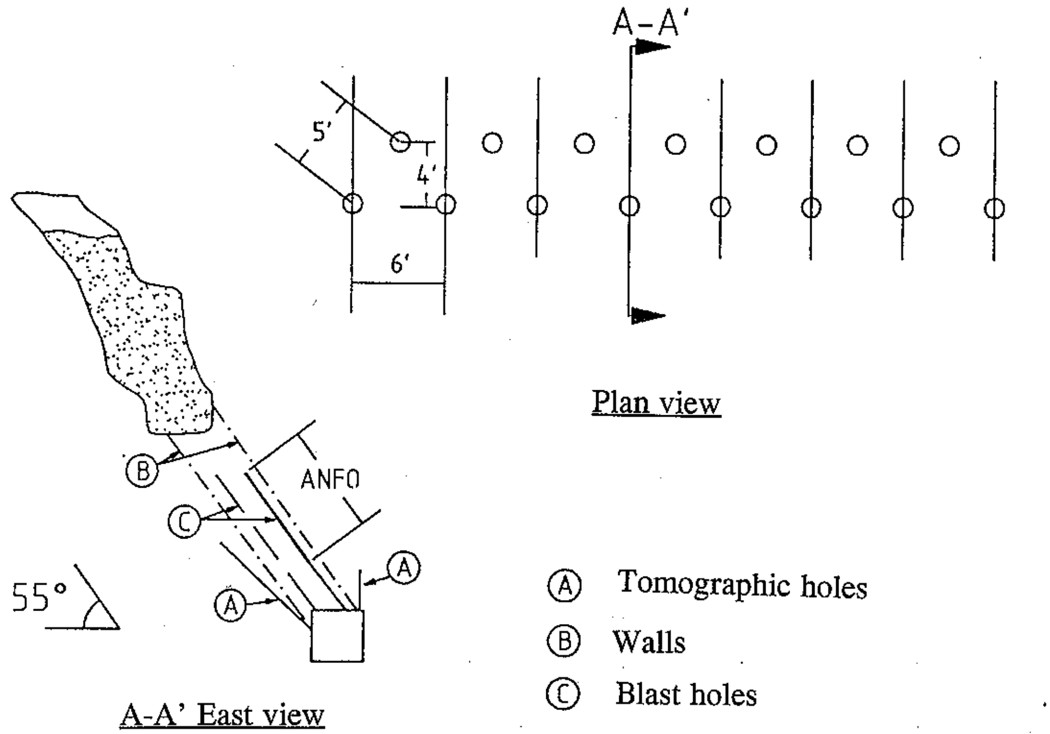


Figure 5. Pattern of holes drilled for the destress blast.

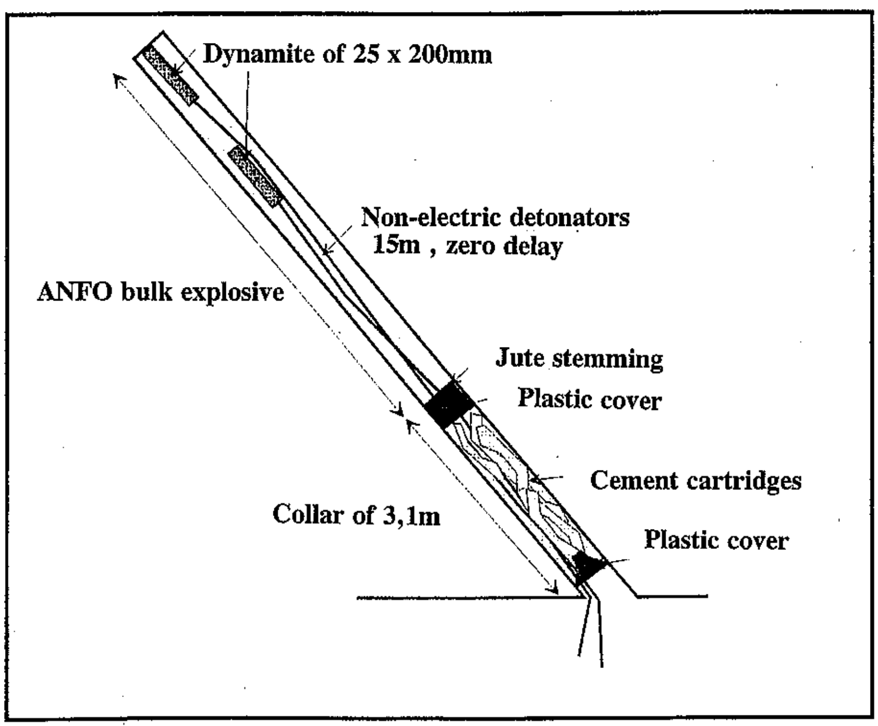


Figure 6. Schematics of the loading of the drillholes.

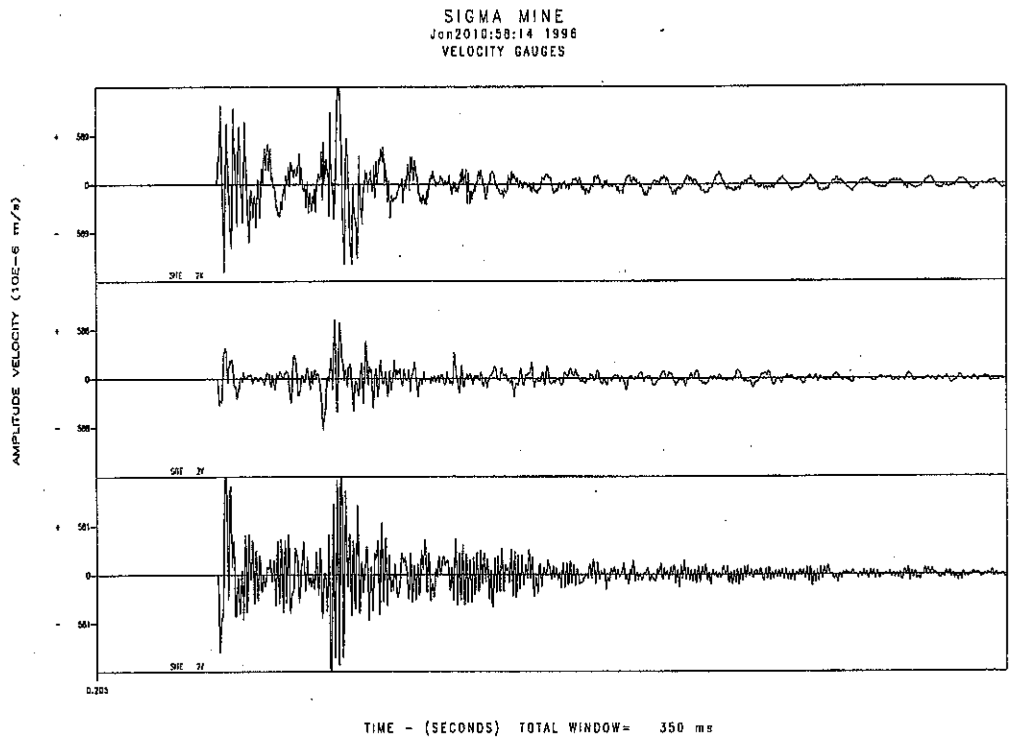


Figure 7. Far-field recording of the blast.

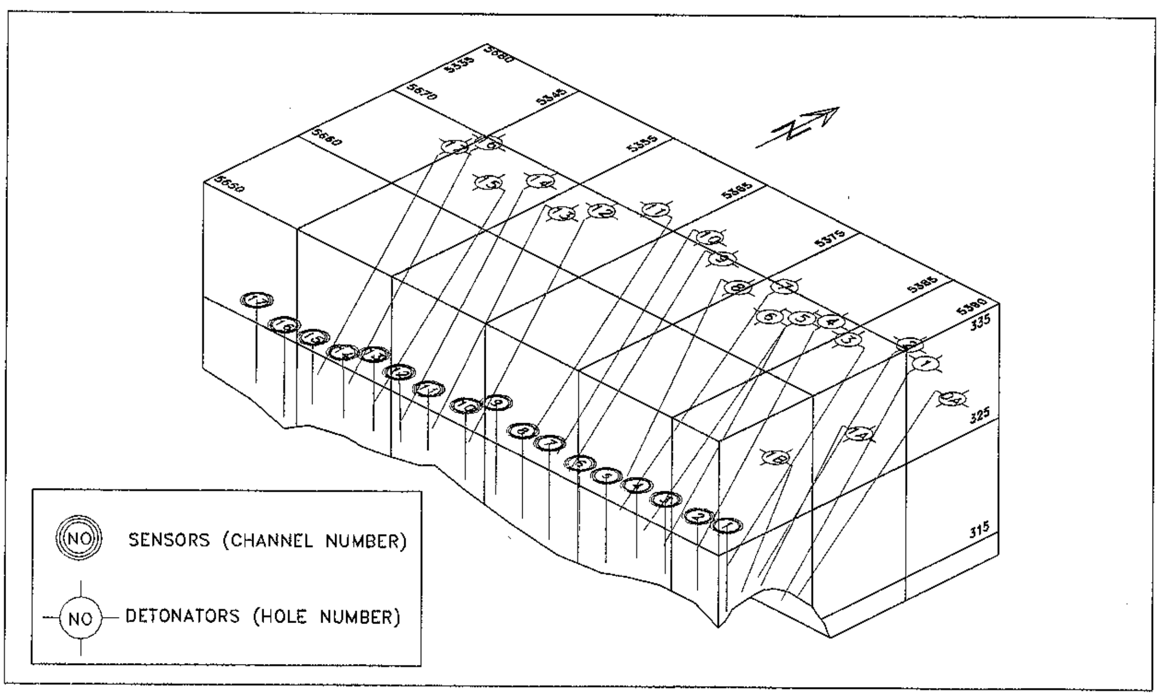


Figure 9. Location of the transmitters and the receivers for the tomography survey.

Figure 8a. The drift 3420E before the blast - Section 5 350 E (east view).

Figure 8b. The drift 3420E after the blast - Section 5 350 E (east view).

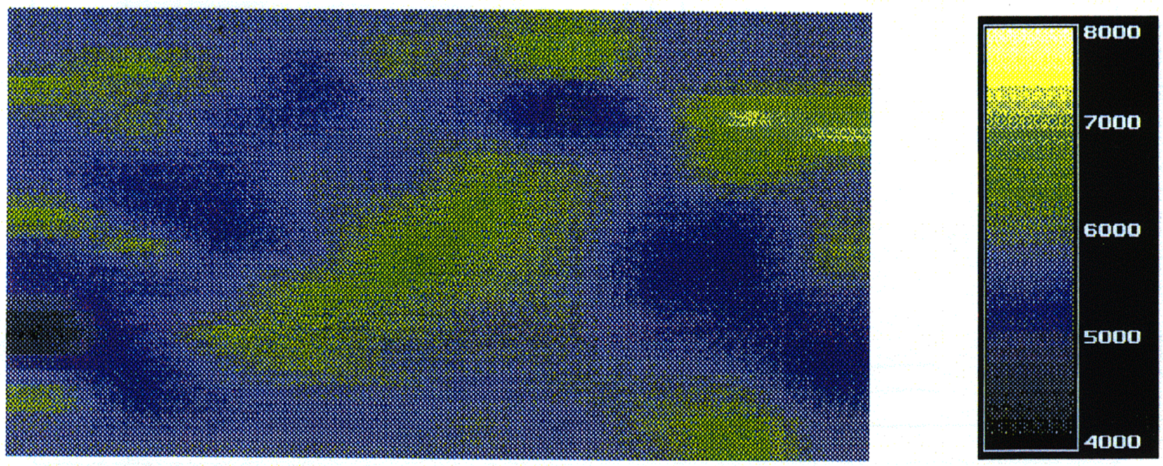


Figure 10a. Tomographic profile determined before the blast.

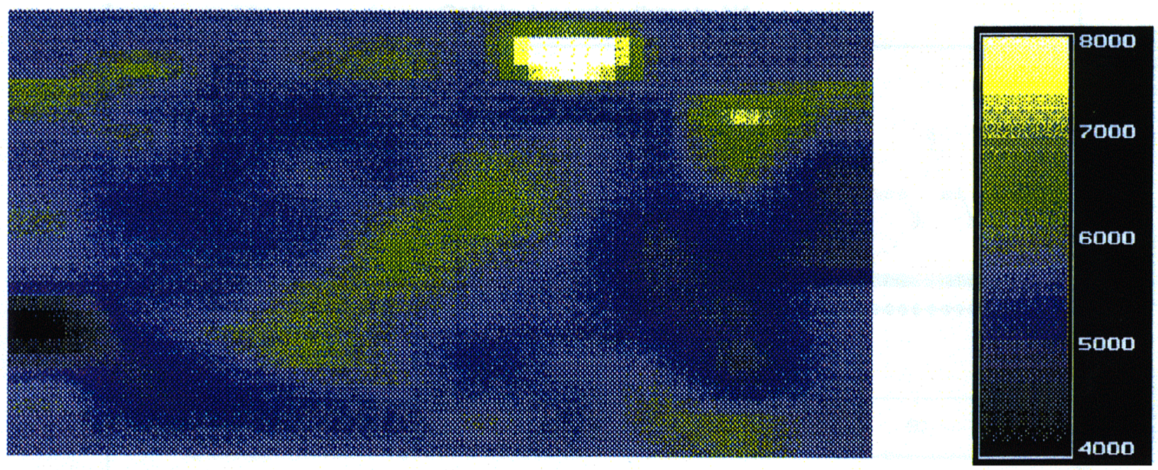


Figure 10b. Tomographic profile determined after the blast.

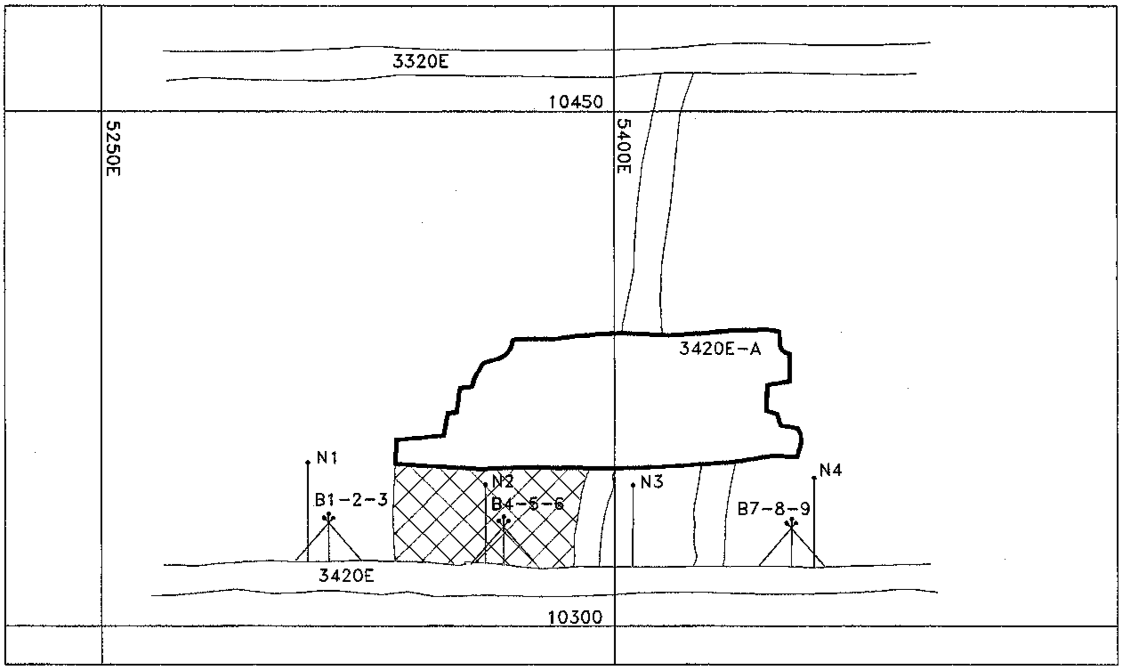


Figure 11. Pattern of boreholes drilled for the stress measurements.

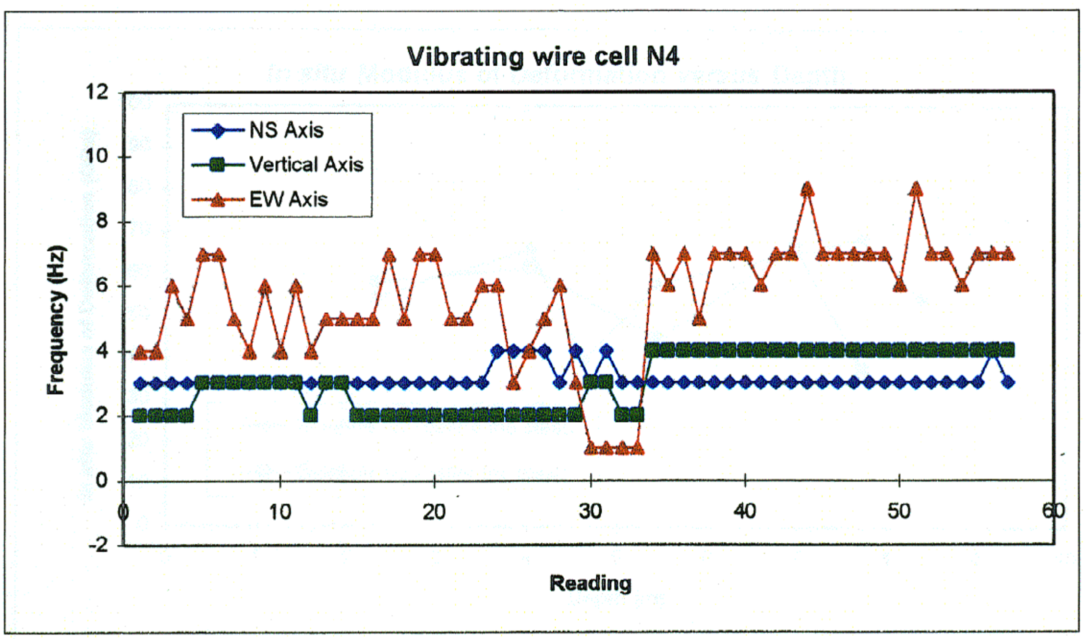
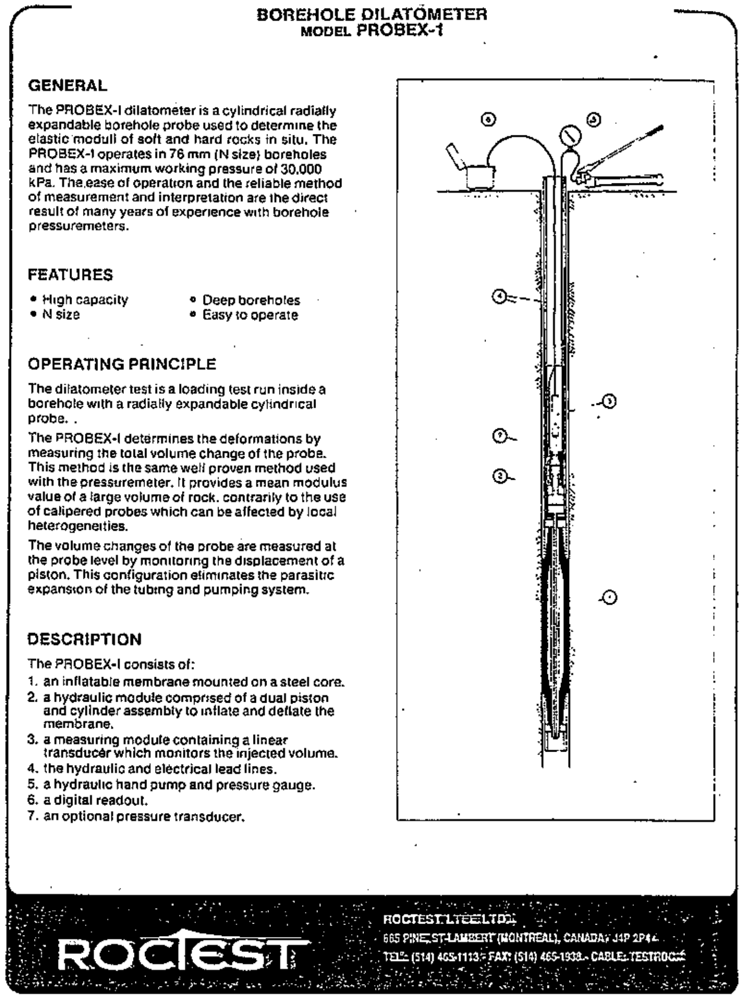


Figure 12. Variation of frequency shown by the vibrating wire cell N4.



BOREHOLE DILATOMETER
MODEL PROBEX-1

GENERAL

The PROBEX-I dilatometer is a cylindrical radially expandable borehole probe used to determine the elastic moduli of soft and hard rocks in situ. The PROBEX-1 operates in 76 mm (N size) boreholes and has a maximum working pressure of 30,000 kPa. The ease of operation and the reliable method of measurement and interpretation are the direct result of many years of experience with borehole pressuremeters.

FEATURES

- High capacity
- N size
- Deep boreholes
- Easy to operate

OPERATING PRINCIPLE

The dilatometer test is a loading test run inside a borehole with a radially expandable cylindrical probe.

The PROBEX-I determines the deformations by measuring the total volume change of the probe. This method is the same well proven method used with the pressuremeter. It provides a mean modulus value of a large volume of rock, contrarily to the use of calipered probes which can be affected by local heterogeneities.

The volume changes of the probe are measured at the probe level by monitoring the displacement of a piston. This configuration eliminates the parasitic expansion of the tubing and pumping system.

DESCRIPTION

The PROBEX-I consists of:

1. an inflatable membrane mounted on a steel core.
2. a hydraulic module comprised of a dual piston and cylinder assembly to inflate and deflate the membrane.
3. a measuring module containing a linear transducer which monitors the injected volume.
4. the hydraulic and electrical lead lines.
5. a hydraulic hand pump and pressure gauge.
6. a digital readout.
7. an optional pressure transducer.

ROCTEST

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Figure 13. Determination of *in situ* rock deformability with dilatometer (Courtesy of Roctest).

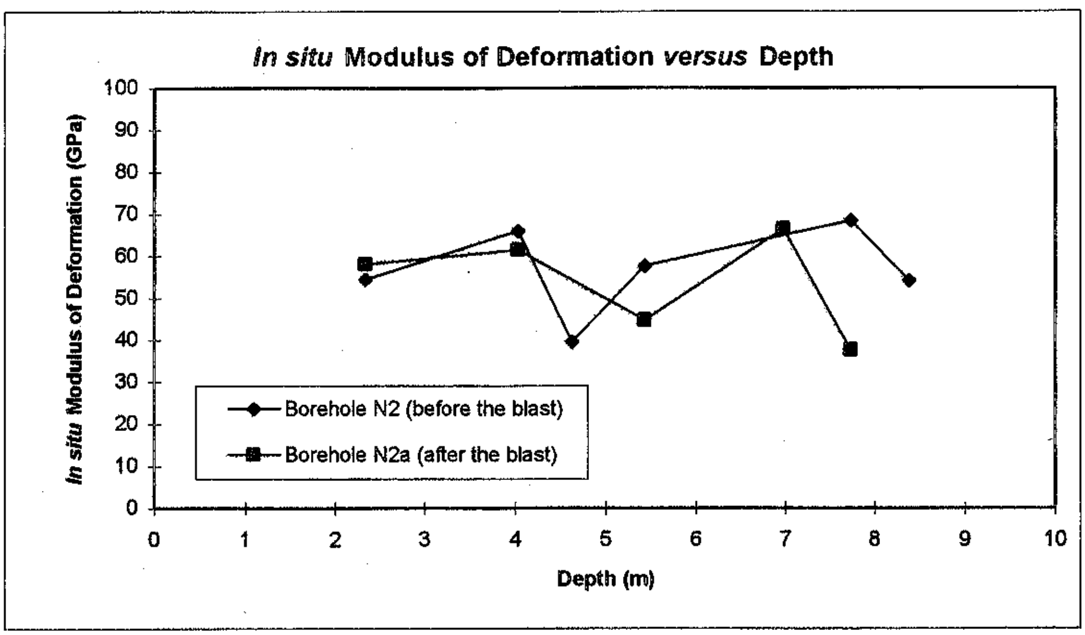

Figure 14. Profiles of deformability determined for boreholes N2 and N2a.