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INSTRUMENTATION AND ANALYSIS OF A DEEP SHAFT IN QUARTZITE

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ABSTRACT

Terra Tek Engineering instrumented and monitored a concrete shaft lining and the surrounding quartzite at levels 2412 ft, 4063 ft, and 5191 ft during sinking of the Silver Shaft at Hecla Mining Company's Lucky Friday Mine in Wallace, Idaho. Rock displacements were monitored with three 50 ft long multiple position borehole extensometers at each level. Stress build-up in the concrete lining was monitored with four pressure cells at each level, and the corresponding strains were monitored with embedded strain gages. Data from each level are compared, and a detailed analysis of rock and concrete behavior at the 5191 ft level is given. The results of the rock displacement and concrete lining stress monitoring have thrown light on several important aspects of deep shaft liner design, and on the behavior of jointed ground at great depth.

INTRODUCTION

Hecla Mining Company is further developing its Lucky Friday Mine in Idaho with the construction of a 6000 ft deep shaft. J. S. Redpath Corporation has sunk the shaft at record rates, achieving 473 ft of advance in one month, early in 1981. The concrete lined 18 ft diameter shaft is the first circular shaft ever sunk in nearly 100 years of mining in the Coeur d'Alene district, where rectangular, vertical or inclined timbered access ways have been the rule.

It is generally assumed that geological structure controls rock displacement near the surface, while high stress levels dominate behavior at depth. However, even at the 5191 ft level, the marked bedding joints in the quartzite were found to dominate behavior, causing distinctly anisotropic and non-elastic deformation of the rock, and correspondingly anisotropic concrete liner stresses. The principal stress direction trends NW-SE, parallel to the steeply dipping bedding joints at this level.

INSTRUMENT INSTALLATION SEQUENCE

Figures 1 and 2 illustrate the installation sequence followed for the 5191 ft level. The three fifty foot long MPB extensometers were installed close to the face, with the instrument heads protected by dog-hole recesses The shaft sinking then continued while continuously monitoring the three MPBX instruments, which had grouted anchors at radial depths of 50, 30, 15, 10, 5 and 3 feet. When the base of the shaft was some 23 feet below the extensometers following five bench blasts, the kerb ring shuttering was lowered ready for the kerb ring pour (see Figure 2). Concrete pressure cells (P.C.) and strain gages (S.G.) were installed in this pour at the locations shown in section in Figure 3.

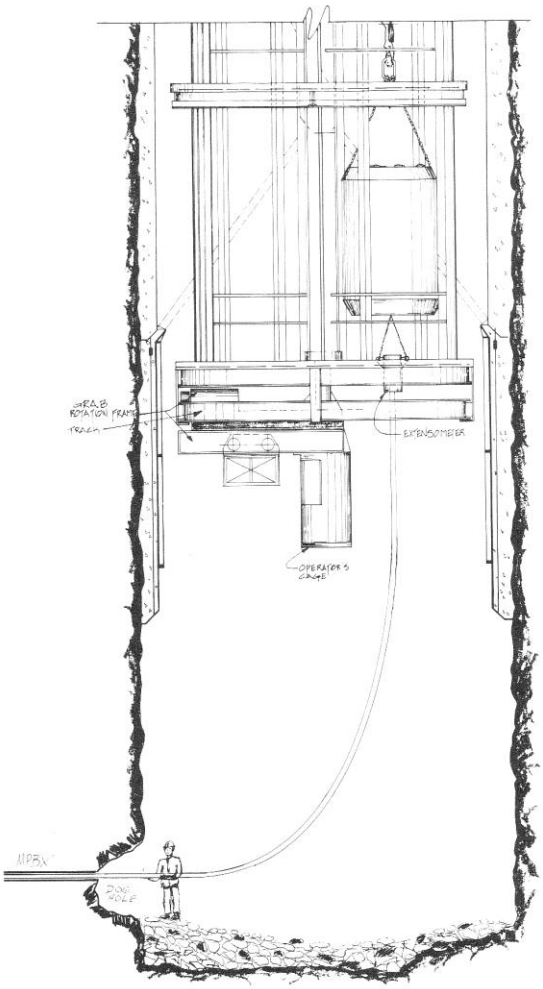

Figure 1. Installation of MPBX extensometers within ½ radius of the face.

A similar distribution of instruments was made at the other instrumented levels. However, the concrete lining was closer to the face on the first two installations, and all the instruments were installed during the same kerb ring pour. The 5191 ft level was instrumented in a manner more typical of construction sequences, with the lining lagging the face advance by 1-1½ diameters, i.e. 20 to 30 ft.

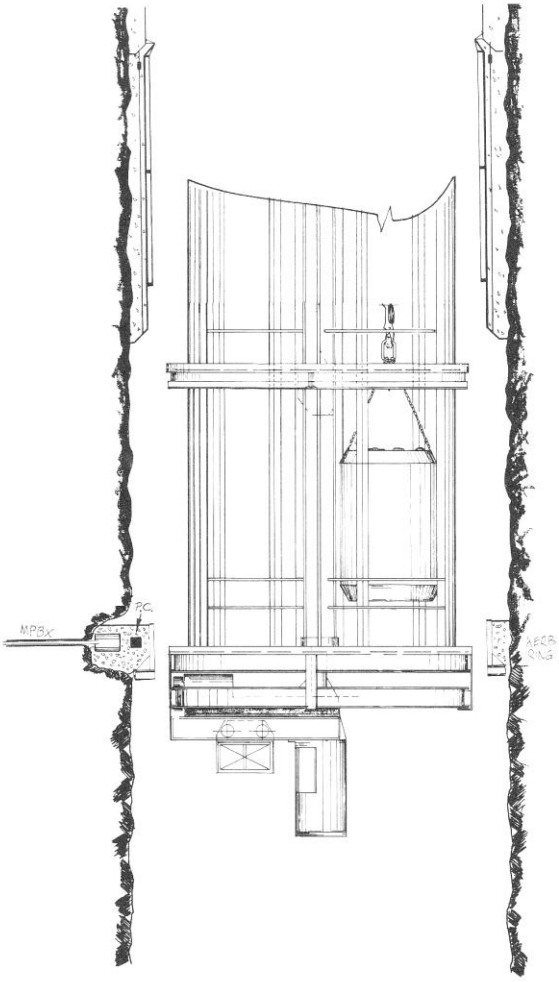

Figure 2. Installation of pressure cells (P.C.) and strain gages (S.G.) in the kerb-ring pour, approximately 23 feet (5 blasts) behind the face.

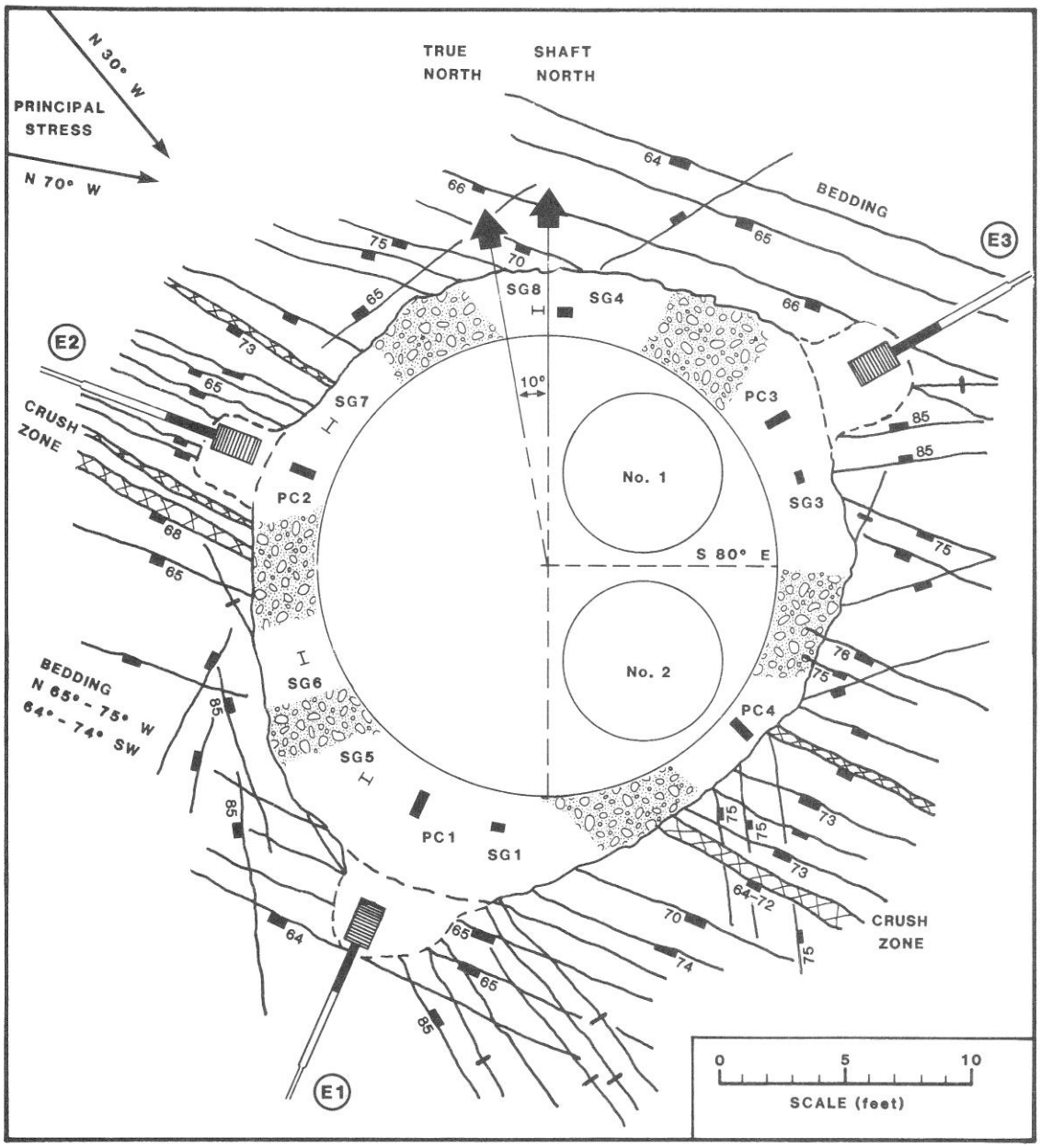

Figure 3. Location of instruments with respect to the shaft geology and the variable lining thickness. 5191 ft level.

DESCRIPTION OF INSTRUMENTS

Multiple Position Borehole Extensometers

The radial displacement of the rock mass and the coupled tangential stress and strain in the liner were recorded by four types of remote reading instruments as illustrated in Figure 3. The multiple position borehole extensometers were Model A3, with groutable anchors, manufactured by Geokon.

Model A-100 linear potentiometers were used for monitoring the rock displacements relative to the instrument head. These have a resolution of 0.001 inch. The extensometer cables were protected from blast damage by heavy bull-hose, and the recessed instrument heads were protected by ¼ inch steel plates bolted over the dog holes. The deepest anchors at 50 ft depth provided assumed zero motion references, to which the other measurements were compared.

All three extensometers were assembled at the surface, then transported one vertical mile down the shaft, hanging beneath the can, as seen in Figure 1. A heavy weight hanging in parallel with the 50 ft bundle of tubes minimized pendular motion.

Concrete Pressure Cells

Flatjack type pressure cells manufactured by Geokon were used for monitoring the tangential stress build-up in the concrete lining. The 9 inch diameter cells are filled with mercury. Pressure changes are monitored by resistive type pressure transducers. In an effort to reduce performance uncertainties associated with possible water migration when the steel cells are placed in a large volume concrete pour, the cells were pre-encapsulated in fine aggregate concrete briquettes and then calibrated in uniaxial compression in the laboratory. The thin plate-like voids between the steel and concrete interface could then be closed by crimping the secondary filling tube. The existence of these voids is apparent if the cells do not respond to initial stress build-up

Strain Gages

Tangential strain in the concrete liner was monitored using Geokon's 4 inch strain gages, Model CG4-128-11-2-6 ppm. Four of the gages were placed directly in the kerb ring concrete pour (SG5-SG8, Figure 3), the other four (SG1-SG4) were pre-encapsulated in fine aggregate concrete briquettes.

DATA ACQUISITION SYSTEM

Thirty-two channels of instrumentation were used to monitor the performance at each level. Signal cables were run from the instruments near the face up to stainless steel junction boxes located at appropriate mine stations 100-200 feet above the instruments. Each junction box contained the signal conditioning and a regulated 5 volts DC power supply for the 32 channels. The junction boxes also contained two 16-channel data telemetry modules for transmitting signal voltage levels to the surface. Each module contained a multiplexer and encoding circuitry so that 16 channels could be transmitted uphole via one twisted pair, which also provided the telemetry modules with power.

Micromux transmitters in the junction boxes transmit a sequence of frequency modulated pulses uphole, where the Micromux receiver digitizes the signals, each channel producing an integer from 000 to 999. These numbers are stored in the receiver and are accessible to a minicomputer on a continuous basis.

An HP-85 desktop computer and HP 7470A plotter were used for reducing and plotting the data. MPBX data were plotted as net displacements relative to the deepest anchor. The displacement of the collar and of the rock at 3, 5, 10, 15 and 30 feet was plotted as a function of time after installation, expressed in Julian days. Pressure and strain data were also plotted as a function of time after installation, with zero readings taken at the time of installation.

BEHAVIOR AT DIFFERENT LEVELS

The table below shows the depths and estimated major principle stresses for the three installations performed thus far.

Installation	Depth	σ_1
1	2412'	4010 psi (27.7 MPa)
2	4063'	6600 psi (45.5 MPa)
3	5191'	8250 psi (56.9 MPa)

Installation at 2412 Feet

In installation No. 1 where all instruments were installed within ½ radius of the face, a maximum displacement of about 0.6 inch was recorded by the extensometer (E3) located in the argillite in the south side of the

shaft, while E1 (located in the north side quartzite) recorded a maximum displacement of about 0.1 inch. Displacements stabilized after a shaft advance of about 2½ diameters. The tangential stresses developed in the liner reflected the closeness of the pour to the face of the shaft. PC3 located near E3 (in argillite) recorded a maximum liner stress of about 1600 psi, while two of the three pressure cells in the sector of the shaft adjacent to quartzite recorded only about 300 psi for the majority of the monitoring time.

The closeness of the pour to the face resulted in step-wise increases in tangential stress with the initial blasts. A major readjustment of liner stress occurred after eleven days, with the side at 1600 psi falling about 400 psi, and the side at 300 psi rising by about 400 psi.

Installation at 4063 Feet

Installation No. 2 was also made in a pour located close to the face. On this occasion all the shaft perimeter was in quartzite. Extensometer E1 which was perpendicular to bedding registered a maximum collar displacement of about 0.4 inch, while the other two extensometers recorded maxima of about 0.1 and 0.15 inch. Pressure cells PC1 and PC3 which were located where the shaft was tangent to the bedding recorded maximum tangential liner stresses of about 600 and 300 psi, respectively. This was consistent with the location of maximum displacement. Once again, the influence of face advance (blasting) was evident in the initial step-wise increases in liner stress. The strain gages installed in this second installation showed highest liner strain in locations consistent with the above stress maxima.

Installation at 5191 Feet

The present installation No. 3 was made in similar geology to No. 2, but the depth and stress levels were higher, causing larger displacements. Furthermore, the face was advanced prior to concreting, so initial rock deformation was not resisted by a lining.

Displacements

On this occasion the maximum displacement registered by E1 (perpendicular to bedding) was about 0.75 inch, while the other two extensometers registered maxima of about 0.2 and 0.15 inch. The displacement-time record for the three extensometers is shown in Figure 4. The time of occurrence of blast numbers 1 to 5 are indicated in the lower diagram.

Figure 5 shows the displacement record for extensometer E2, together with the shaft deepening that occurred with each bench blast. This extensometer was close to the E-W division between bench blasts, and registered distinct displacement steps with both the north blasts (2, 4, 6) and the south blasts (1, 3, 5). However, extensometer E1 located on the south side of the shaft hardly registered the north side bench blasts (#2 and #4), as can be seen in Figure 4.

It will be noticed that the side of the shaft showing maximum displacement (E1) registered half this displacement somewhere in the outer 3 ft of rock. In the position of E2 installed perpendicular to bedding, the behavior was more elastic.

Liner Stresses

Figure 6 illustrates the build-up of tangential stress in the lining, both as a function of position and as a function of time. The timing of concrete pours is indicated by the letters A, B, C, etc. Each pour extends the lining by a total of 15 feet. It is interesting to note the anisotropic nature of the stress build-up, with gages PC-3 and PC-1 tangential to bedding showing the greatest response to shaft sinking. This is consistent with the position of maximum displacement, as recorded by extensometer E1.

Stress continued to build in the lining even when the shaft was sunk to 5 diameters below the instrumented level (pour G). This was presumeably a function of both concrete curing and rock mass creep effects. The shaft remained at the 5300 ft level for several weeks while a mine level was developed.

Liner Strains

Figure 7 illustrates the build-up of tangential strain in the liner. Only four of the eight strain gages operated after installation. The unencapsulated gages were particularly vulnerable to damage from the vibrators used to consolidate the kerb ring pour.

The general shape of the curves shows similarity to the stress records, which, of course, is to be expected if the concrete displays linear stress-strain behavior.

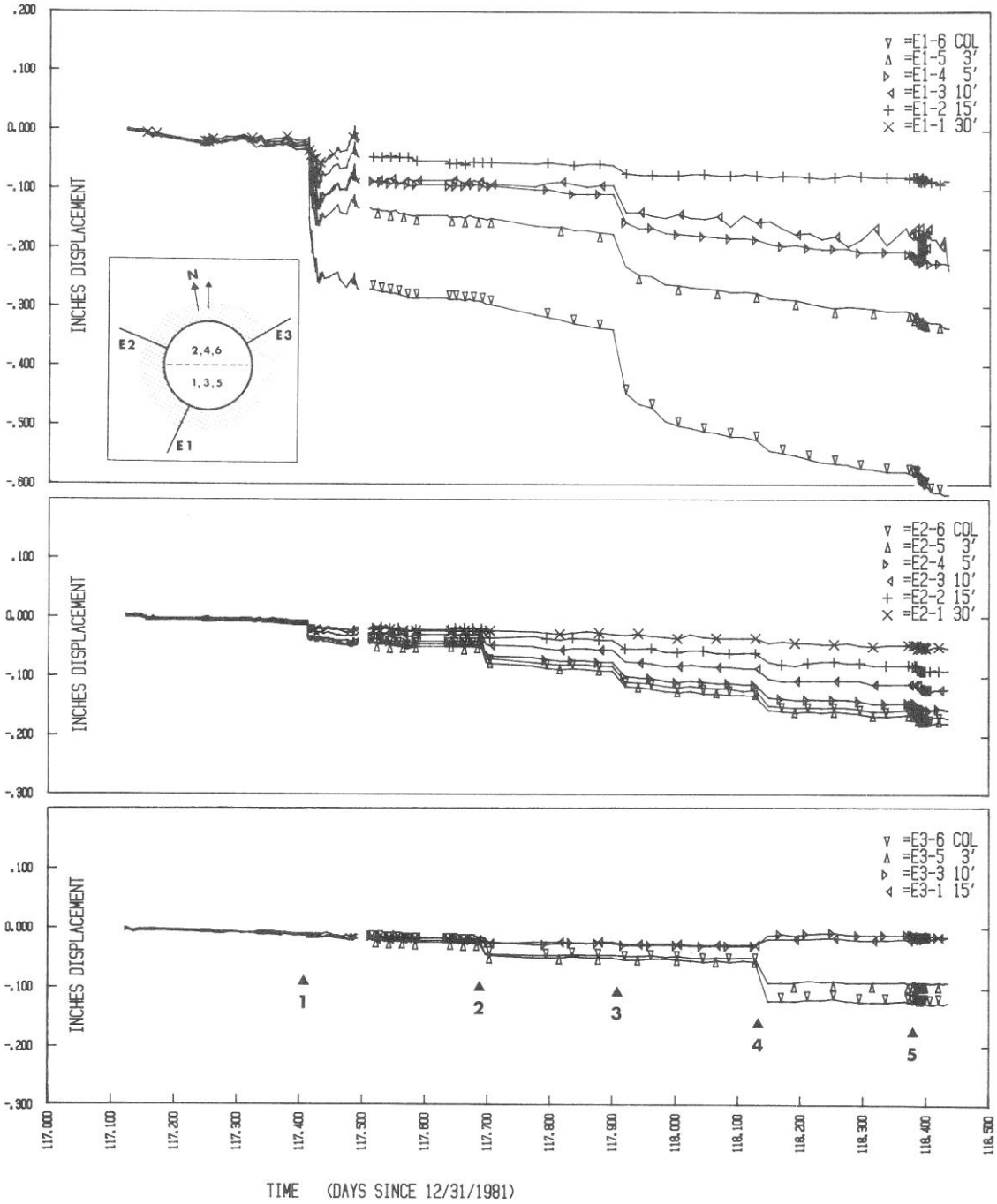

Figure 4. Comparison of rock deformation in the three orientations round the shaft. E1 is perpendicular to bedding, E2 parallel to bedding, and E3 intersects bedding at about 45°. 5191 ft level.

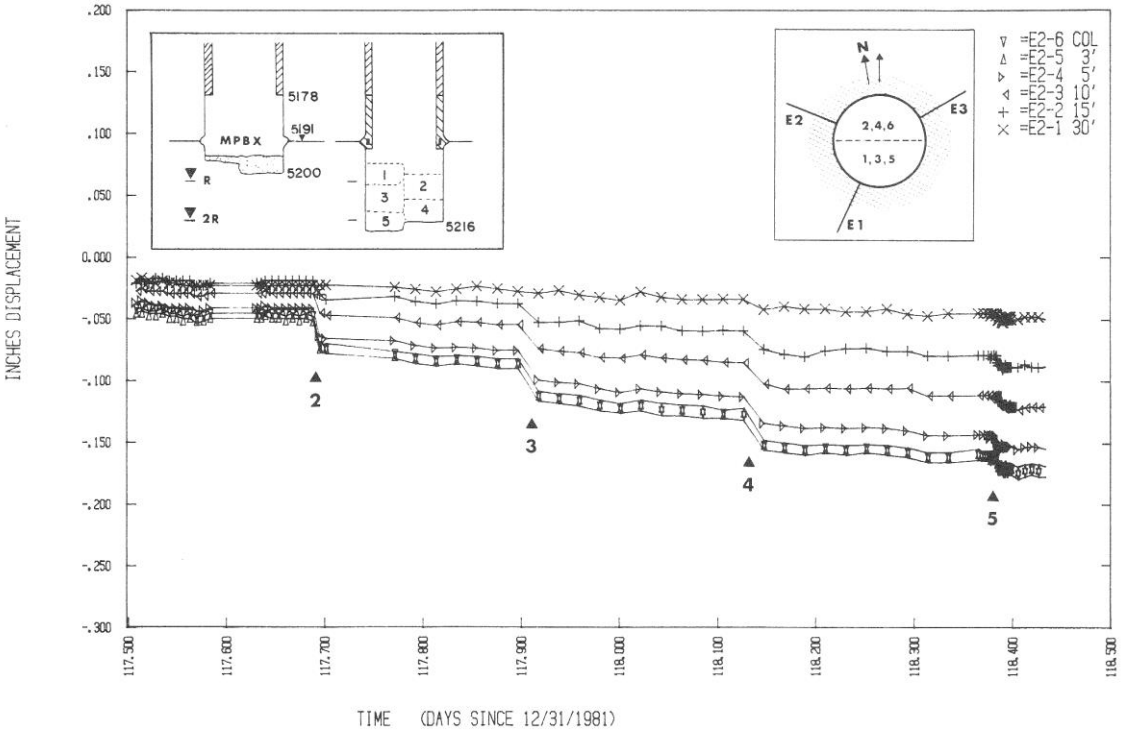

Figure 5. Response of MPBX E2 to South (3, 5) and North (2, 4) bench blasts. 5191 ft level.

Concrete Modulus

At the 4063 ft level, the implied full-scale concrete deformation moduli were 1.0×10^6 psi from the stress/strain pair PC1/SG1, and 0.4×10^6 psi from the pair PC3/SG3. At the present 5191 ft level, the behavior showed some inconsistencies. Figure 8 shows that the stress/strain pair PC3/SG3 give a derived modulus that increases from an initial value of 0.8×10^6 psi to about 1.5×10^6 psi in the long-term (28-52 days). However, the second stress/strain pair PC1/SG1 indicated inconsistent behavior after the second concrete pour, with PC1 sub-sensitive, and SG1 over-sensitive to changes with time. The explanation may be the variable liner thickness caused by the dog-holes.

Rock-liner response is a complex phenomenon, as illustrated by the fact that the thinnest part of the lining (at PC-4) showed the lowest level of tangential stress. The particulars of the jointing adjacent to the lining appear all-important in determining the stresses and strains experienced by the lining.

ANALYSIS OF ROCK DISPLACEMENTS

Closed-form two dimensional elastic solutions for an unlined and a lined opening, and a three dimensional analysis by Tanimoto (1980), were compared with the measured rock deformations. The comparison illustrated that the side of the shaft that was tangential to the bedding (position E1) was effectively loosened, and demonstrated a significantly lower modulus in the outer five feet.

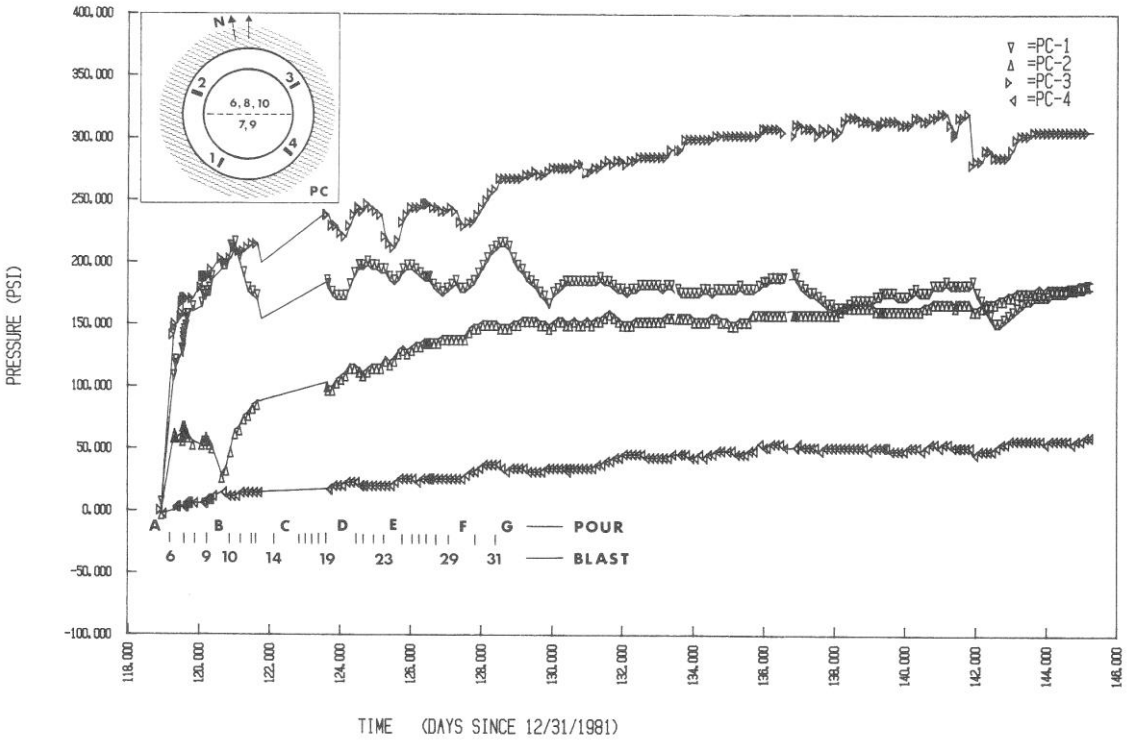

Figure 6. Pressure cell response during the first 26 days of monitoring. 5191 ft level.

Modulus Variation with Radial Distance

Figures 9 and 10 show the results of two of the closed-form two-dimensional elastic solutions for radial displacement versus depth, for the case of an unlined opening. The assumed anisotropic horizontal stress distribution is based on an extrapolation of measured data in the Coeur d'Alene mining district, which was reviewed by Board, et al. (1981). The major principal horizontal stress of about 8250 psi (56.9 MPa) has a NW-SE orientation, approximately parallel to the strike of the bedding in the quartzite.

In view of the two-dimensional solution, the displacements shown in Figures 9 and 10 have been arbitrarily reduced by a factor 0.4, to take approximate account of the difference between a 2D and 3D analysis.

The inferred variation of modulus with depth perpendicular to bedding is striking. The effective modulus appears to increase from about 0.5×10^6 psi near the shaft wall to 2×10^6 psi at a depth of 20 feet. Behavior appears to be more elastic in the direction parallel to bedding, and the inferred modulus is also higher at 3×10^6 - 4×10^6 psi. There is also an indication that the modulus closest to the shaft wall is favorably influenced by the tangential stress concentration for the sector of the shaft where jointing is perpendicular to the shaft wall. The opposite is true where bedding is tangential to the lining.

By way of comparison with the above inferred results, borehole deformation tests reported by Patricio and Beus (1976) and USBM (1980) using a CSM cell, indicate small-scale moduli for bedded quartzite in the range $7\text{-}11 \times 10^6$ psi, with one value as low as 2.6×10^6 psi for tests performed at 45° to the bedding.

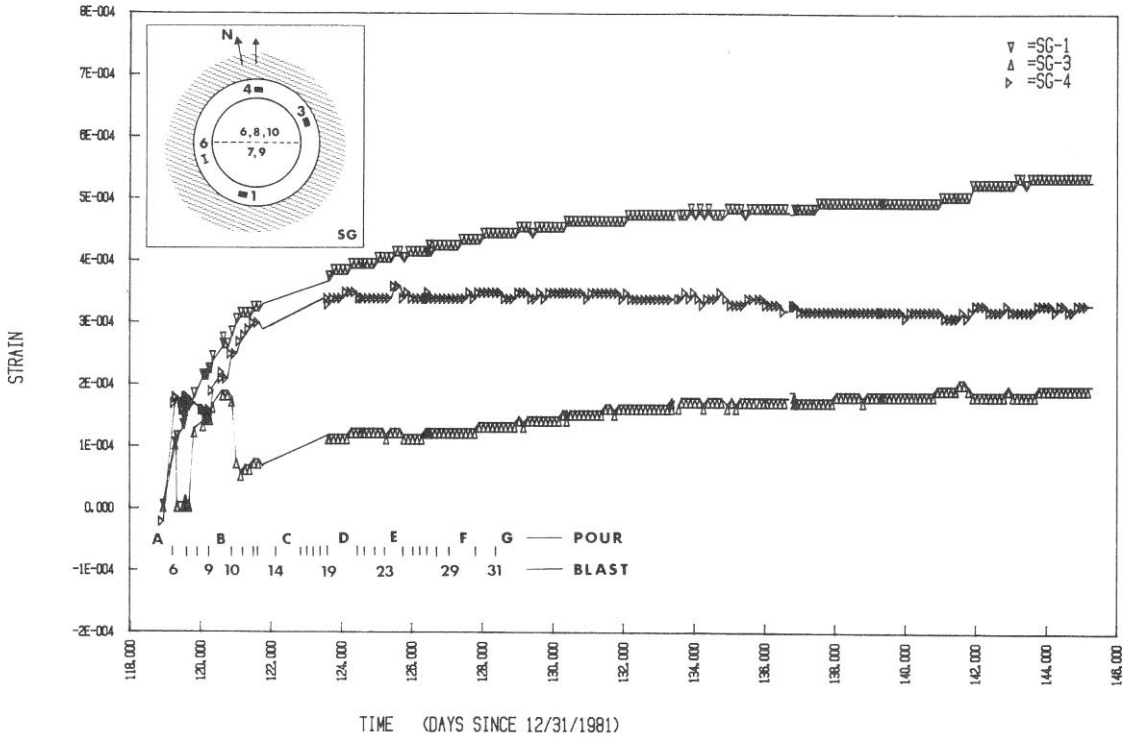

Figure 7. Strain gage response during the first 26 days of monitoring. 5191 ft level.

The lower inferred deformation modulus at position E1 resulted in an interesting liner-rock interaction some three weeks after installing instruments at the 5191 ft level. The outer five feet of rock at E1, which exhibited the above low modulus response, suddenly moved backwards away from the opening, and there was a corresondingly sudden reduction of tangential stress on pressure cells PC1 and PC3. Apparently the rock that yields most initially is also least resistant to the build-up of stress in the lining, and may yield outwards later, if sufficiently stressed by the lining.

Modeling a Damaged Zone

It is of interest to compare the closed-form solutions for unlined and lined openings, where the lining is represented by a lower modulus zone. This lining could equally well represent the lower modulus of the blast damaged zone, which often approximates a thickness of 2-3 feet, as shown by numerous measurements in a tunnel driven in a hard jointed gneiss (Chitombo, 1982).

When using a deformation modulus of $2x10^6$ psi and a Poisson's ratio of 0.25 in each case, and a damage zone modulus of $1x10^6$ psi for the analysis of the "lined" opening, it is found that the largest displacement occurs in the case of the "lined" opening. This is to be expected.

However, the most significant difference between the "undamaged" and the "damage-zone" analyses is the deformation occurring at depth. The opening with a lower modulus "damage zone" shows 0.15 inch displacement even seven radii into the shaft wall, while the "undamaged" case shows only 0.05 inch displacement at this depth. This has important implications for rock support techniques as discussed later.

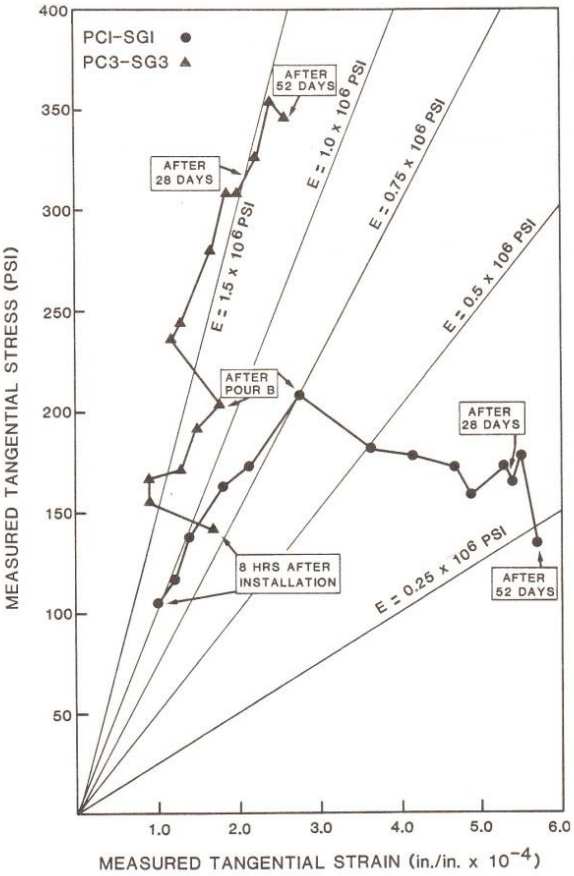

Figure 8. Comparison of measured stress and strain in the concrete lining. 5191 ft level.

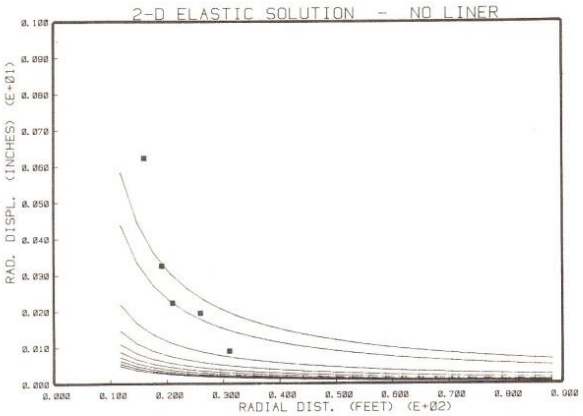

Figure 9. Calculated and observed radial displacements versus radial distance from shaft center for different rock moduli (from 0.75×10^6 psi to 9×10^6 psi). Orientation corresponds to that of E1. Assumed horizontal principal stresses = 8250 psi and 5700 psi (=0.35).

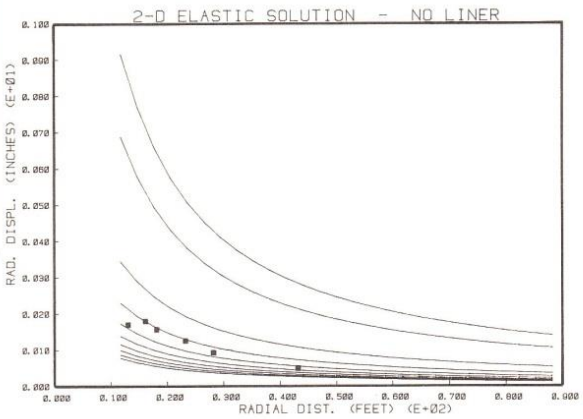

Figure 10. Calculated and observed radial displacements versus radial distance from shaft center for different rock moduli (from 0.75×10^6 psi to 9×10^6 psi). Orientation corresponds to that of E2. Assumed horizontal principal stress = 8250 psi and 5700 psi (=0.35).

THREE-DIMENSIONAL FACE ADVANCE EFFECTS

The data shown in Figure 11 indicates the amount of displacement measured by MPBX E1 for four increments of face advance. The data for one radius (1R) is the displacement that occurs for a face advance from ½R (at installation) to 1R. The data for two radii (2R) is the displacement that occurs for a face advance from 1R to 2R, and so on. The gradual loss of "half-dome" action is apparent in these results.

The three dimensional elastic analysis reproduced in Figure 12 shows the development of radial displacement with face advance, for different depths from the wall of a 10 meters diameter excavation. In the particular example shown, Tanimoto chose a modulus of 0.1 GPa (14,500 psi) and a principal stress of 1 MPa (145 psi), which gave a ratio of E/σ_1 of 100:1.

Analysis of the present Silver Shaft installation suggests that the rock mass modulus can be approximated to $2x10^6$ psi outside the immediate zone of blast damaged rock. The ratio of this modulus to the assumed principal stresses of 5700 and 8250 psi is of the order 350-240 respectively. We might therefore expect a maximum possible (3D) radial displacement of about one third that shown in Figure 12 due to the 100:300 ratio of the dimensionless ratio E/σ_1.

However, Tanimoto's analyses were performed for the case of an excavation with 5 meters radius, in contrast to the unlined Silver Shaft radius of 3.58 m (141 inches). Reducing the displacement data by a further reduction factor of 3.58/5.0 we obtain the results listed in Table 1.

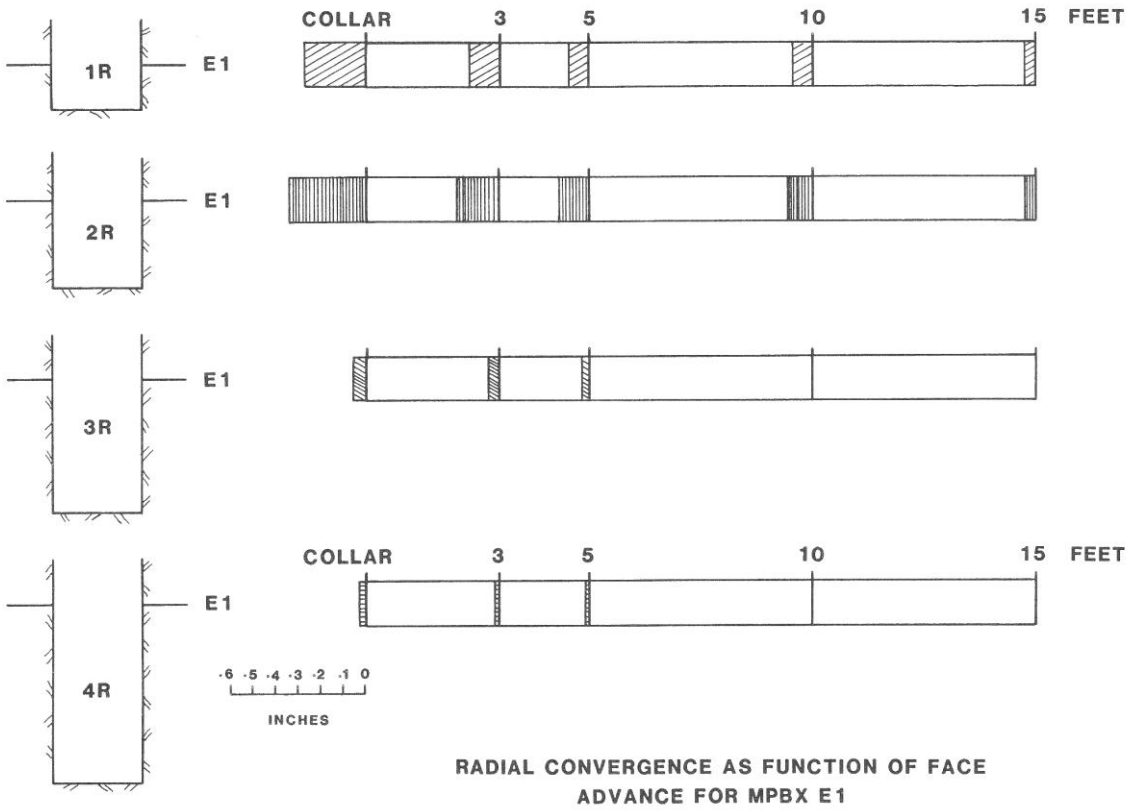

Figure 11. Relative depth-displacement data as a function of face advance for E1.

Table 1

Measured and Predicted Displacements
for Shaft Advance from D=½R to D=2R
(From Installation to Blast No. 5)

Radial Depth	Depth in Feet	Predicted Elastic Displacement (inches)	Measured Displacements E1 (inches)	E2 (inches)	E3 (inches)
r=R	collar	0.073-0.107	0.616	0.168	0.129
r=1.5R	5.9 ft	0.071-0.103	0.205	0.152	0.019
r=2R	11.7 ft	0.069-0.101	0.119	0.109	0.014
r=3R	23.5 ft	0.063-0.092	~0.08	0.060	---

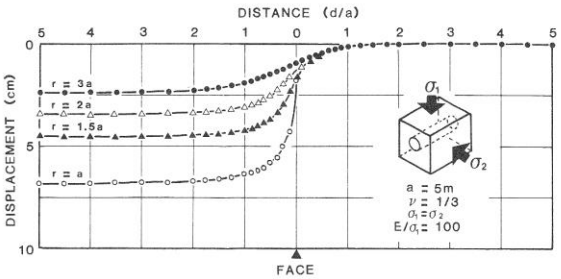

Figure 12. Theoretical displacements obtained from a 3D elastic analysis, with an E/σ_1 ratio of 100/1, after Tanimoto (1980). Excavation radius (a) = 5 m.

The most obviously non-elastic behavior (or behavior governed by a much lower modulus) is the outer 6 feet of rock at the location of E1 and, to a lesser extent E2.

The magnitude of the deformation moduli in the "damage-zone" can be back-calculated from the data shown in Figure 12 and in Table 1. The measured displacements of 0.61 inch at E1 and 0.19 inch at E2 following five blasts (D/R = 2) imply E/σ_1 values of 43 and 137 respectively. When the estimated value of σ_1 = 8250 psi for the 5191' level is substituted, "damage-zone" moduli of 0.35×10^6 and 1.1×10^6 psi are obtained for the rock at E1 (perpendicular to bedding) and for the rock at E2 (parallel to bedding). The arbitrary factor of 0.4 applied to the 2D data plotted in Figures 9 and 10 clearly underestimates the radical differences between 2D and 3D analyses.

The three-dimensional and two-dimensional elastic analyses (with and without lining) may also give quite similar distributions of radial displacement, if the 3D analysis is interpreted at an appropriate distance behind the face. Our 2D analysis for E = 2×10^6 psi, ν = 0.35 with and without lining were non-dimensionalized by dividing the displacements by shaft radius. This data is compared with Tanimoto (1980) 3D data, taking into account the 1:3 ratio of E/σ_1, in the two sets of analyses.

It is seen from Figure 13 that the 2D analysis with no liner falls exactly between the 3D results for face positions between one half and one radius beyond the plane of measurement. All three analyses predict larger displacements than those measured due to the practical limit on instrument installation. The extensometers cannot be installed ahead of the advancing face to register the complete record of advance.

EVALUATION OF SUPPORT REQUIREMENTS

The concrete lining adopted as final support for the entire Silver Shaft project was designed on the basis of 2D analyses, with the concrete assumed poured to the face. Quite high liner stress levels were apparently expected according to this conservative design philosophy.

In practice the lining was seldom closer than 25-30 ft from the face. The low levels of tangential stress (100-350 psi) are well below design levels due to the discrepancy between an elastic 2D analysis with the concrete assumed poured to the face, and the three-dimensional reality of concrete at least one diameter behind the face.

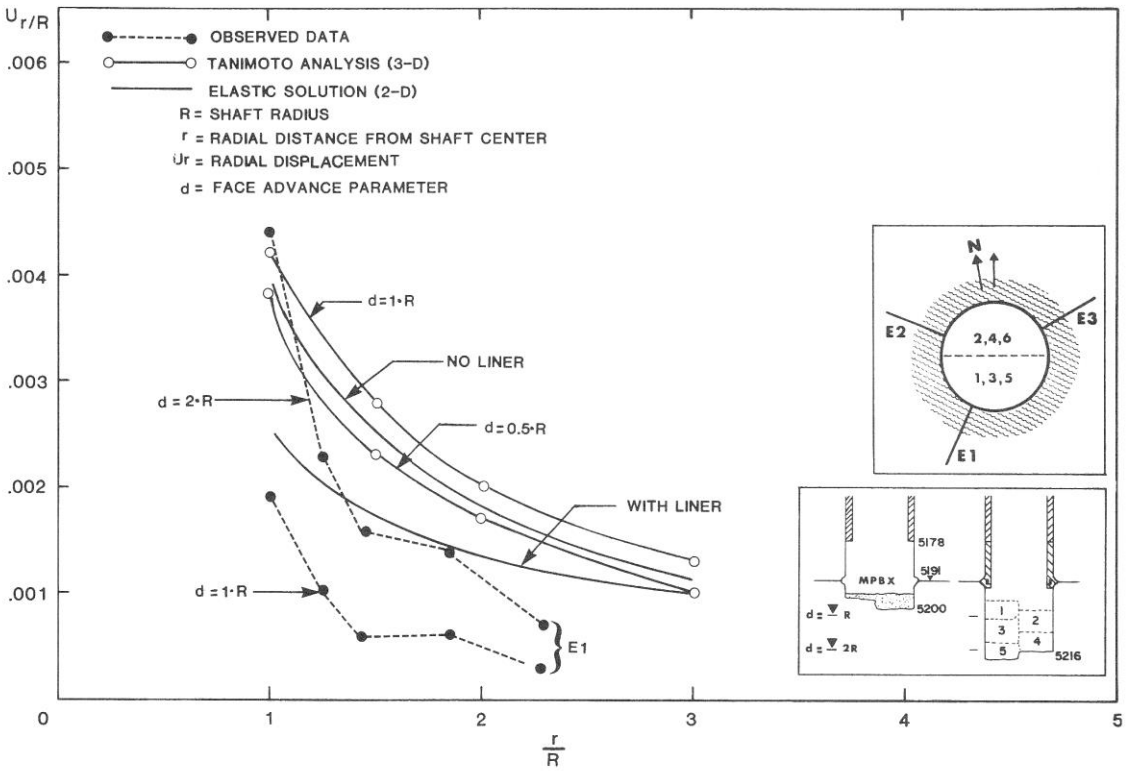

Figure 13. Comparison of non-dimensionalized data from 2D analysis (lined and unlined) and 3D analysis (at d = 0.5R and 1.0R). Also shown is the measured data from MPBX E1 when the face is 1.0R and 2.0R beyond the instrument location.

The limited build-up of stress in the concrete at least to the 5200 ft level suggests stable final conditions. However, the temporary stability of the 25-30 ft of unlined shaft wall has an important role in this limited stress level. It is important that the rock is bolted (and mesh covered for worker safety) to limit the amount of yield prior to concrete lining.

It is apparent that measures to maintain the in-situ modulus of the rock such as bolt reinforcement, will be most successful in limiting the volume of rock affected by the excavation. If increased stress levels at greater depth cause more marked deformation, slabbing and "popping", it may be advisable to increase the bearing capacity of the present bolting, by replacing the split sets with fully grouted bolts. From a rock mechanics stability point of view, experience has shown that fully grouted bolting combined with mesh reinforced shotcrete would provide adequate permanent stability for the shaft. However, it would obviously not be so convenient for erection of the internal shaft steelwork.

The bolting/shotcrete combination seeks to limit displacements away from the wall of the shaft, thereby reducing the load on any lining. Silver Shaft experiences thus far have demonstrated that lining loads can also be minimized by delaying the final lining. This philosophy works well provided that temporary support is adequate.

With MPBX installation one half radius from the face, displacements as large as 0.6 inch were registered prior to concrete lining. If uniform elastic behavior had occurred, the three dimensional analysis shown in Figure 12 would have predicted something closer to 0.1 inch during this face advance interval. However, the same figure indicates that displacements at least six times larger than this will already have occurred prior to MPBX installation. If the non-elastic (or low modulus)

response were to occur right from the start, it is possible that we would see up to 3½ inches of displacement due to excavation of the shaft. This magnitude of displacement could only be tolerated in a stable manner when high displacement gradients were prevented by suitable bolting.

ACKNOWLEDGEMENTS

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