CONFERENCE 1988
ON
DEFORMATION SURVEYS
EXECUTIVE SUMMARIES

A. CHRZANOWSKI
(Chairman)

June 1988
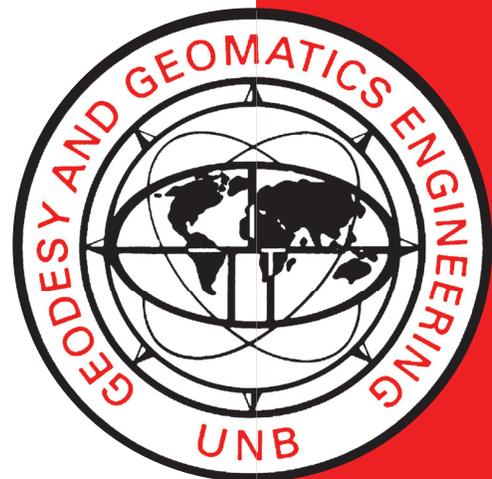
PREFACE

In order to make our extensive series of technical reports more readily available, we have scanned the old master copies and produced electronic versions in Portable Document Format. The quality of the images varies depending on the quality of the originals. The images have not been converted to searchable text.

EXECUTIVE SUMMARIES

5th INTERNATIONAL (FIG) SYMPOSIUM ON DEFORMATION MEASUREMENT

5th CANADIAN SYMPOSIUM ON MINING SURVEYING AND ROCK DEFORMATION MEASUREMENTS

6-9 June 1988
Fredericton, New Brunswick
Canada

A. Chrzanowski
(Chairman)

PREFACE

This volume contains executive summaries of 59 out of a total of 78 papers accepted for presentation at the international conference on deformation monitoring, analysis, and prediction which has been organized and hosted by the Department of Surveying Engineering, University of New Brunswick, from 6 to 9 June 1988.

The main goal of the conference has been to bring together surveying, structural, geotechnical, and mining engineers, as well as geodesists and geophysicists, to exchange information and to develop a closer cooperation in areas of common concern. In the spirit of the inderdisciplinary approach to deformation measurements and analysis, two symposia have been joined together and dubbed 'Conference 1988': namely, the 5th International Symposium on Deformation Measurement of Commission VI of the International Federation of Surveyors (FIG), and the 5th Canadian Symposium on Mining Surveying and Rock Deformation Measurements sponsored by the Canadian Institute of Surveying and Mapping (CISM).

In addition to FIG, CISM, and UNB as the main sponsors, the Natural Sciences and Engineering Research Council of Canada and Energy, Mines and Resources Canada have sponsored and financially supported Conference 1988. The New Brunswick Electric Power Commission, The University of Calgary, the Canadian Institute of Mining and Metallurgy, the Canadian Society for Civil Engineering, and the Province of New Brunswick are among the other sponsors.

The organizing committee of Conference 1988 is very grateful to all the authors who have contributed papers to the conference. Full proceedings of Conference 1988 will be published jointly by the University of New Brunswick and the Canadian Institute of Surveying and Mapping. These should appear within six months of the conference. More information is available from:

> Conference 1988
> c/o Department of Surveying Engineering
> University of New Brunswick
> P.O. Box 4400
> Fredericton, N.B.
> Canada
> E3B 5A3

The organizing committee sincerely hopes that Conference 1988 will bring a new interdisciplinary approach to solving the extremely important problems of deformation surveys within the fields of science and engineering.

Dr. Adam Chrzanowski, P.Eng
Chairman, Conference 1988

TABLE OF CONTENTS

Page

Table of Contents

Table of Contents

Table of Contents

DESIGN AND ANALYSIS OF STRAIN MEASUREMENTS OF COSTA BOLIVAR DYKE IN VENEZUELA

J. Abi Saab, Chief of Dykes and Drainage, Maraven, S.A., Maracaibo, Venezuela

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Oil extraction from sand strata at depths varying between 300 m and 1000 m along the Bolivar Coast of Lake Maracaibo causes a considerable ground subsidence which in some places reaches 20 cm/year. Since 1926, several subsidence basins have been created with a total maximum subsidence of about 5 m in some of them. As the original terrain consisted of swamps hardly above lake level, dykes and drainage systems had to be built to protect these areas against flooding. Presently, the total length of earth dyking is over 40 km with heights exceeding 7 m in the areas of maximum subsidence.

Due to the progressing subsidence which has been monitored by levellings since 1926, strains in the material of the dykes are being developed at the places of maximum curvature of the subsidence basins. In 1984, a survey scheme was designed and implemented by consultants from the University of New Brunswick (UNB) in cooperation with the petroleum company, Maraven, to monitor the strain accumulation in selected portions of the dyke. The survey scheme was designed as conventional geodetic micro-networks of high precision. Figure 1 shows one of the networks, established on a 2.5 km portion of the Tia Juana Dyke. As expected from the subsidence profiles, the network was designed to detect strain rates of 10 ppm/year at 95% probability, with the surveys repeated every 6 months. The well-known UNB generalized method of deformation analysis has been used in the design and analysis of the deformation surveys on the dykes. In order to detect the above-mentioned strain rate, the design required 2×10^{-6} accuracy (relative standard error ellipses) of relative positioning in the micro-networks. This has been achieved by using a Kern DKM3 and a Wild T2000 electronic theodolite and precision electronic instruments for distance measurements (Tellurometer model MA-100 and a specially selected and calibrated Wild DI4S).

Figure 2 shows an example of relative displacements developed at the Tia Juana Dyke over a period of 18 months between July 1985 and January 1987. The displacements have been calculated using the iterative weighted transformation of the UNB generalized method.

The strains obtained from the least-squares fitted displacement field agree very well with the strain values predicted earlier by the UNB consultants. This indicates that the deformation of the dykes is well under control.
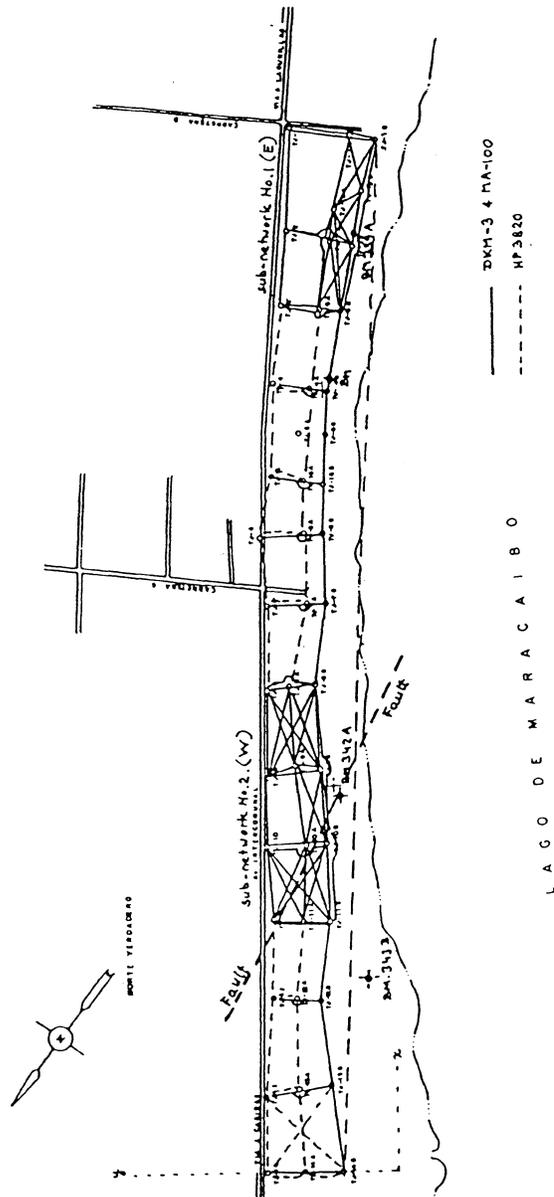
Figure 1. The study area—The total network and the subnetworks along the Tia Juana dyke.

Tia Juana Dyke Network Pillar Benchmark Movement from 1985 07 to 1987 01

".." row near Av. Intercomunal   "▲" row 1A to 14A (centre)   "o" row 1B to 13B (dyke)

ε ≅ 22 ppm/yr

ε ≅ -26 ppm/yr

Scale

0   500 m

0   10 mm
displacements
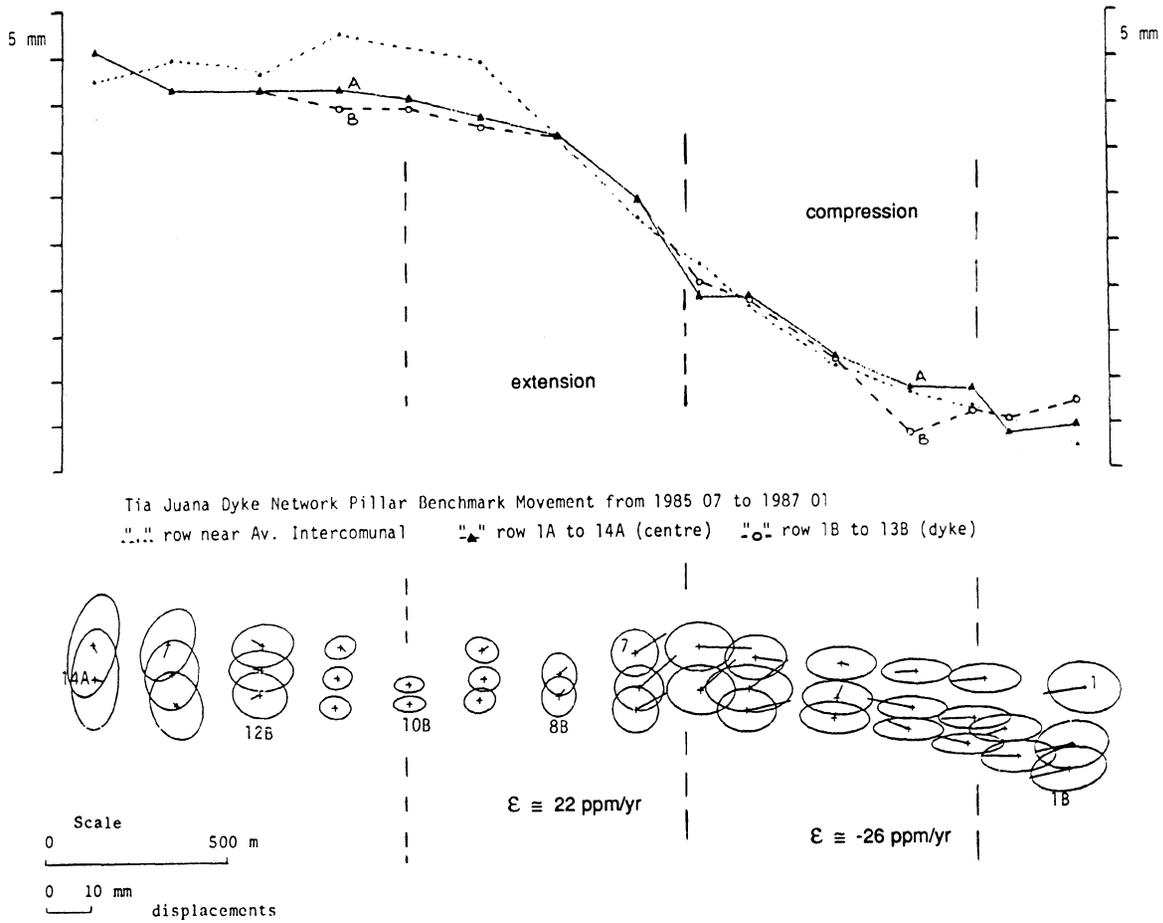
Figure 2. Tia Juana total network—Relative displacements of points after weighted transformation between July 1985 and January 1987 with ellipses at 95%.

PREDICTION OF MINING INDUCED SUBSIDENCE OVER LONGWALL PANELS IN THE NORTHERN APPALACHIAN COAL REGION.

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Specific lithological conditions in the northern Appalachian coal field prevent the use of existing predictive methodologies based either on influence or profile function. The presence of highly resistive limestone and sandstone layers within the relatively thin (122 m - 305 m) overburden is the most probable cause of discrepancies between measured surface deformations over the longwall panels and deformations computed by methods developed mainly for European conditions.

Field data used for the development of the new predictive model were collected by the Bureau in 11 longwall panel studies. The width of panels ranged from 140 m to 287 m and overburden thickness from 122 m to 305 m.

As a first step, the problem was approached by trying to establish the effect of lithology on subsidence characteristics for each test site and also to define relative differences of this effect between individual test sites. This cannot be done by simple comparison of subsidence profiles. Different underground geometry and mining conditions, including depth of cover, must be taken into account.

The principle of this idea is to establish the difference in subsidence characteristics between hypothetically homogeneous overburden, i.e., overburden without the presence of resistive hard rock units, and existing lithological conditions. At the same time, one must provide for given mining conditions.

For computation of subsidence for homogeneous overburden, Bals' theory was used. Since one cannot define the exact value of the angle of draw, the process of computation was repeated for several angle-of-draw values. The results have indicated that the most appropriate angle-of-draw value is 25°. Figure 1 shows a typical subsidence profile from the northern Appalachian coalfield in comparison with calculated profiles by Bals, using 15° and 25° angles of draw as functional parameters.

It is evident that a reasonable congruency could not be reached between the measured and computed profiles using Bals' theory for any angle of draw from 15° to 90°. However, Bals' theory was used as a helping tool to separate the effect of lithology from the effect of different mining conditions on subsidence characteristics.

To obtain congruency between computed and measured data, it was necessary to introduce the subsidence coefficient (a_v) as a variable:

$$a_{vi} = \frac{sF_i}{me_i},$$

where sF_i = measured subsidence,
m = extracted coal seam thickness,
e_i = efficiency coefficient for each point.

The definition of the efficiency coefficient is based on Newton's law governing the attraction of masses. Bals assumes that each differential part of mined-out area within the full area of influence exerts a force on the surface point inversely proportional to the square of its distance from it. Using the computer algorithm developed by the Bureau, it was possible to compute and tabulate the values of e for different mining conditions, i.e., underground geometry and overburden thickness.

For each of the 189 surface points, at all test sites, the value of a_{v1} has been defined. Figure 2 shows the characteristics of this coefficient along individual profiles adjusted to the edge of the panel. Each of these curves represents a separated effect of lithology on subsidence characteristics, expressed in the form of the variable subsidence coefficient. The dispersion of individual curves shows the differences of lithological effect between individual test sites.

Regression analysis of the variable subsidence coefficients from all test sites on the location relative to the edge of the panel have yielded a third-degree polynomial equation for 25° angle of draw:

$$a_v = -3.587 * 10^{-8} X^3 + 1.628 * 10^{-5} X^2 - 9.105 * 10^{-5} X + 1.359 * 10^{-1},$$

where X is the distance in feel from the edge of the panel toward the centerline.

For points located outwards from the edge of the panel, $X=0$. Then, the subsidence of any point toward the centerline will be

$$s_i = m * e_i * a_{vi},$$

and outwards from panel edge

$$s_i = m * e_i * 0.1359.$$

The validity of the prediction model has been proven not only by sensitivity tests but also by comparison of predicted values with field data gathered at test sites not included in the regression analysis.

Inclinations and curvatures can be computed as functions of vertical displacements. The prediction of horizontal strains from the curvature gives the best results. Coefficients were empirically defined for local conditions. The predicted results approximate the mean values for measured ground strains.

To facilitate the use of this precalculation methodology, a computer program was written in BASIC for use on a personal computer.
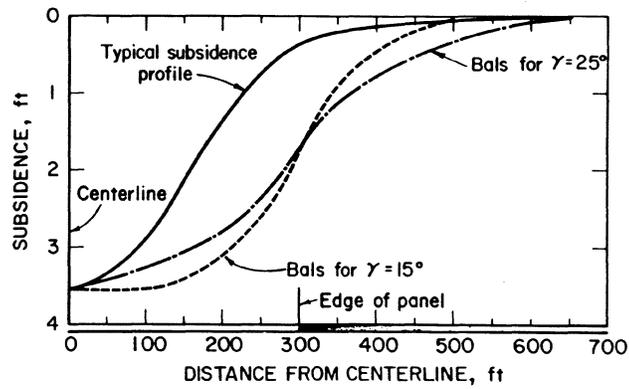
Figure 1. Comparison of measured profile with
Bals' predictive method.
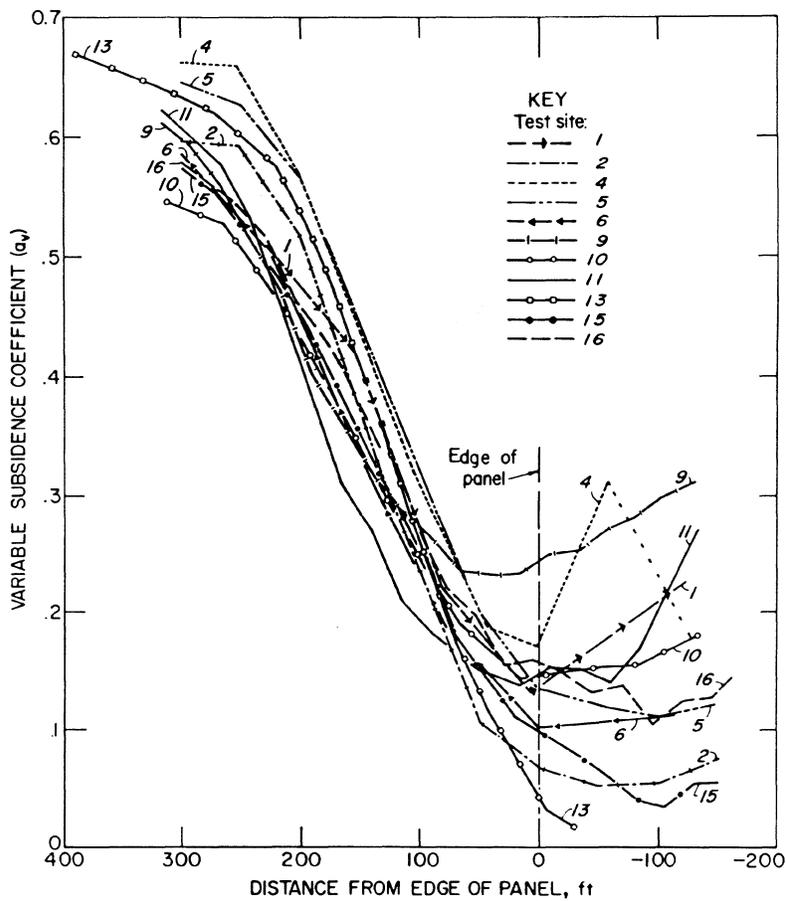
Figure 2. Variable subsidence coefficient for
25° draw angle.

SLOPE MONITORING BY INTEGRATING PHOTOGRAMMETRY WITH PREDICTED DATA

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Deformation models are generally of a static type, aiming for a statistical statement confirming the existence or non-existence of movements in the space domain. Recent developments focus on dynamic models, where displacements are studied with respect to time and frequency of occurrence and also as functions of a cause.

Photogrammetry has been successfully utilized for monitoring displacements in a static mode. The position vectors at times t and $(t+1)$ are determined solely from individual photogrammetric campaigns without any interrelation between the two epochs of observations.

In a time-varying situation, however, object points change their positions progressively as a reaction to a cause. Thus the parameter vector changes not only as new observations become available but also as a function of time.

If the functional relationship between points at successive epochs is adequately known, it is possible to predict a preliminary estimate of the position vector at time t based on its previous spatial position at $(t-1)$. By directly incorporating this dynamic information into the photogrammetric evaluation model, deformation monitoring is now performed in a sequential mode.

On this basis, a computer program, SPDM (Sequential Photogrammetric Displacement Monitoring) was developed at the University of New Brunswick. The program combines a prediction model, which determines the propagation of the parameter vector and its errors through time, plus a photogrammetric model, which includes the extended collinearity equations in photo-variant mode where the interior orientation elements and the object coordinates are treated as weighted parameter constraints.

The sequential estimation of the static information (object coordinates) involves three steps, executed in an iterative manner, namely:
- solution for interior and exterior orientation parameters by extended space resection;
- solution for object coordinates;
- determination of optimal position and accuracy estimates, using prediction and reduced observation models.

After successful tests of this procedure were carried out with simulated and laboratory data, it was applied to a slope monitoring project in a mining subsidence area in British Columbia. Although the control point distribution was not ideal, the

combined sequential method provided significantly better absolute accuracies than the conventional static photogrammetric approach, which was applied for comparison purposes. Furthermore, the redundant information about the object space contributes to the partial or complete removal of datum deficiency and/or configuration defects.

Thus the combination of photogrammetric and prediction methods contributes to a better estimation of position and accuracy, with each method complementing and safeguarding the other.

VALIDITY OF THE APPLICATION OF VIBRATING WIRE STRAIN AND STRESS METERS IN CONCRETE DAMS ON A LONG-TERM BASIS

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Most of the problems associated with concrete dams can be described as being due to aging or to deficiencies. Aging is due to a drop in mechanical properties of the material resulting from thermal fatigue, excessive shrinkage, freeze-thaw, infiltration, leaching, and from related swelling effects. The deficiencies can be related to different factors involving geological conditions, design, construction, operation, or maintenance. In all cases, aging or deficiencies, a redistribution of stresses and strains takes place.

Stress-Strain Measurements

The following methods for measuring stress and strain were developed in rock mechanics. Strains are usually determined on a short-term basis, using either relaxation techniques or strain field restitution by overcoring. On a long-term basis, strains are determined using inclusion techniques. Stresses are determined either directly or indirectly from strains, or by using methods of correlation.

The main methods used for strain measurement on a short-term basis are doorstoppers, modified USBM, and the CSIR-Leeman triaxial cell. On a long-term basis, the methods used are vibrating wire extensometers, strain rings, and various strain gauges. All these methods are difficult to achieve. The passage from strains to stresses demands a knowledge of the short- and long-term rheological properties of the material.

Direct stress measurements by strain restitution with the help of flat jacks are limited to a certain depth. Indirect stress measurements by correlative methods (hydraulic fracturing, acoustic, ultrasonic conductivity, resistivity, photoelasticity, etc.), are easy to carry out, but present calibration and interpretation difficulties. Besides, some of the methods are not easily applicable to concrete dams.

Vibrating Wire Extensometer

The vibrating wire extensometer is made up of a wire string fixed at its ends inside a metal tube (see Figure 1). The frequency of vibration of the stretched wire is directly related to its extension. The excitation and measurement of the vibrating frequency of the wire are carried out with the aid of one or two electrical coils. The measured frequency is affected neither by the length of the electrical wires nor by the quality of the electrical connections. This instrumentation is usually placed in the concrete during the construction stage of the dam. The vibrating wire measures strains but not stresses.

This extensometer is reputed durable and very stable with time. In fact, 90% of of those installed in dams in Quebec are still functioning 25 years after their installation.

It is generally admitted that this extensometer deforms in the same way as the concrete. Many studies have shown, however, that the rigidity of the protecting tube can reduce the measured strains. Also, a finite element study has shown that, for an extensometer placed in ordinary concrete, measured strains can be about 20% less than that of the concrete.

Though concrete is generally considered to be homogeneous, the size of the coarse aggregates can be of the same order of magnitude as that of the extensometer. This leads to measuring strains in a non-uniform field. Also, the anisotropy of the material is rarely taken into consideration.

Owing to the difference in coefficients of thermal expansion of steel and concrete, a correction is usually necessary. This correction does not take into account the rigidity of the extensometer.

It is generally considered that the behaviour of concrete is elastic, or in fact the rheology of the material is elasto-viscoplastic. The strains measured by the extensometer are then composed of an instantaneous strain, a creep strain, a thermal strain, and a hygrometric expansion strain. The strains measured are not independent of the stress history (hysteresis, residual strain). These quantities are difficult to evaluate and this complicates the conversion of strains into stresses. Also, a plane stress state is normally assumed whereas the state of stress is in reality, most of the time, triaxial and this further introduces additional errors into the calculations.

Laboratory Tests

Some laboratory tests carried out have permitted quantifying the order of the strains imposed on stressed and stress-free concrete by the conditioning. Strains were measured by vibrating wire extensometer. The study showed that strains generated by differences in conditioning can, in certain cases, be of the same order as the strains to be measured in the dam. A study was also conducted to explore the possibility of including the vibrating wire extensometers inside the concrete after the dam's construction. Some of the results are presented.

Statistical Modelling

The main objective of measuring strains and stresses is to monitor the behaviour of the dam over the long term. A method for detecting faults consists of a statistical modelling of the dam's behaviour with strain; this is possible with the aid of time series analysis.

Conclusions and Recommendations

It is difficult to determine on the long term the absolute stress inside a concrete dam, even with a vibrating wire extensometer. On a short-term basis, stress variations can be determined based on certain simplifying hypotheses. On the other hand, the vibrating wire extensometer, with its qualities of durability, reliability, and sensitivity, can be used to detect working abnormalities through statistical analysis using time series. The detection of these faults is thus possible in certain cases before water infiltration gets worse.

TYPE	A	B	C
C90	90	122	⌀ 5
C60	60		

CAOUTCHOUC MOULE
SURMOULAGE PRC
ELECTRO (SORTIE DE FILS SUR PERLES)
BOUCHON D EXTREMITE
JOINT TORIQUE
VIS DE BLOCAGE
PINCE DE CORDE
CORDE VIBRANTE ⌀ 0,25mm

TRESSE DE BLINDAGE
CABLE ETANCHE 5 COND.

50 mm
⌀ 20mm
C
25 mm
épaisseur 12 mm
A
B
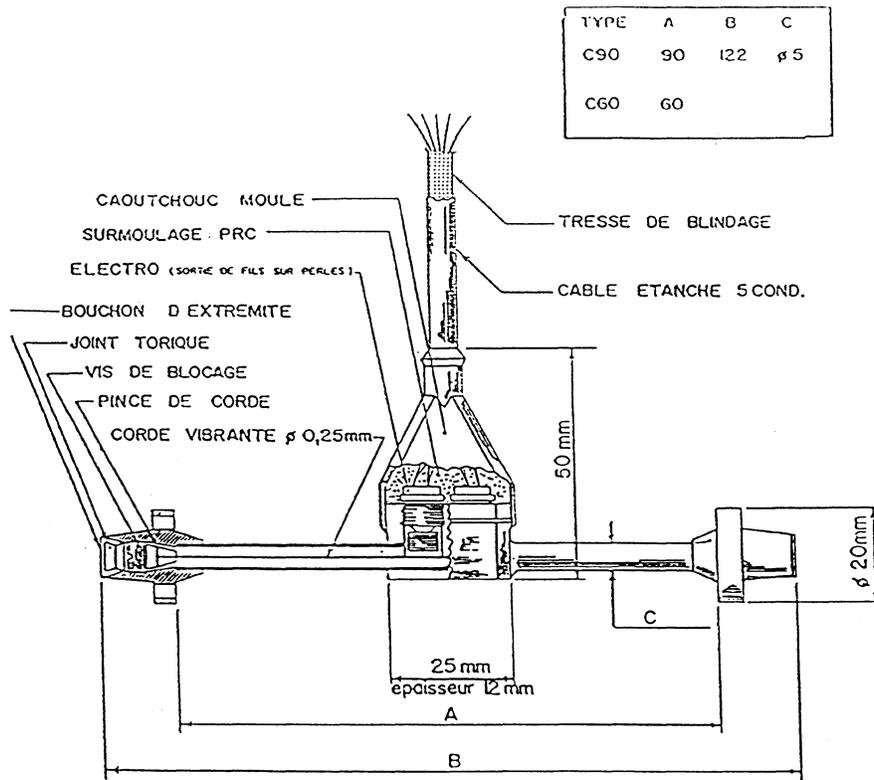
Figure 1. Telemac vibrating wire strain meter.

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 11

DEFORMATION SURVEYS OF LANDSLIDES USING TERRESTRIAL MEASUREMENTS AND SPACE TECHNIQUES

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The Savena river basin situated in the Appennines, between the towns of Florence and Bologna, has been repeatedly involved in landslides due either to the erosive action of numerous tributaries of the river or to the poor geomechanic properties of the ground. Landslides have lately blocked the course of the river in 1870, 1909, and 1951 producing lakes of various extent. The first two natural reservoirs disappeared in a short time, while the third one has remained without considerable changes until today thanks also to several hydraulic settlements.

In 1986, a control network was set up in order to monitor the movement of the slopes of the mountains above the lake. Five reference points situated in steady ground were permanently established by means of concrete pillars with forced centring devices for instruments (Figure 1). All angles and sides were measured using a Wild T3 theodolite and a Wild DI 20 EDM. The m.s.e. of coordinates of reference points resulting from the adjustment of the network did not exceed 3 mm (Figure 2). Seventeen control points were permanently set up just in the landslide body, and another seven were placed in buildings or in check dams. Measurements of angles and distances were carried out with the same instruments used for the reference network, while the adjustment was made assuming the coordinates of reference points as invariable.

The positions of control points have been determined every 6 months. The results of the last set of measurements have pointed out displacements of some control points of about 5-6 cm, which is a significant value if compared to the m.s.e. of coordinates. In order to control the stability of reference points, a new global measurement campaign was carried out in the autumn of 1987. The results of this survey indicate that two points of the reference network have moved 4-5 cm just in the direction of the maximum slope of the ground.

A photogrammetric survey of the whole area under control has also been made to produce a map at a scale adequate for planning hydraulic settlements and for the evaluation of natural and man induced modification of the territory by means of successive repetition of the survey itself. An area of 700 m x 2000 m has been covered keeping a relative flying height of 450 m in order to have photograms at a scale of about 1:3000, suitable for a map at a scale of 1:500. Any control point established on the ground has been pointed out using a circular p.v.c. white panel with a diameter of 1.2 m which has produced a clearly visible image on the photograms. Some reference and control points have been used as check points. The average deviations between coordinates have resulted in less than 5 cm for x and y, and less than 11 cm for z.

A bathymetric survey to determine the lake bottom topography and the thickness of sediments has also been performed. Measurements of depth have been carried out by a Honeywell Elac Laz 4700 sonic depth finder mounted on a catamaran equipped for this purpose. The boat position during measurements has been determined by an integrated Spider EDM-theodolite placed on the end of the bathymetric sections previously positioned with respect to the reference network. The thickness of sediments has been measured by an instrument based on the reflection shooting method. The bathymetric survey has pointed out a progressive silting up of the lake, which has produced a reduction of the lake volume to half its initial value.

The displacement of two points of the reference network has suggested that we place them at other sites, more distant from the lake, characterized by better geological stability as, for example, the ridges of the river valley. This solution would bring about remarkable problems of non-visibility between existing and projected reference points. It has been decided to verify the possibility of applying the Global Positioning System (GPS) to the determination of the relative positions of such points also in this area, where problems might arise from the presence of mountains, trees, and other obstacles. It must be considered in fact that the reference network had been designed for terrestrial measurements only.

For this purpose, a GPS campaign was carried out at the same time as the last survey of the existing reference network. Terrestrial measurements were made with the same instruments used in the first surveys. Two different types of GPS receivers were employed, namely, the Wild-Magnavox WM-101 and the Trimble 4000 SX. Vertical displacements of reference points have been also artificially produced. The results of GPS measurements have been seriously affected by the local environmental conditions. Anyhow, the differences between EDM and GPS measured lengths of some sides do not exceed one centimetre.
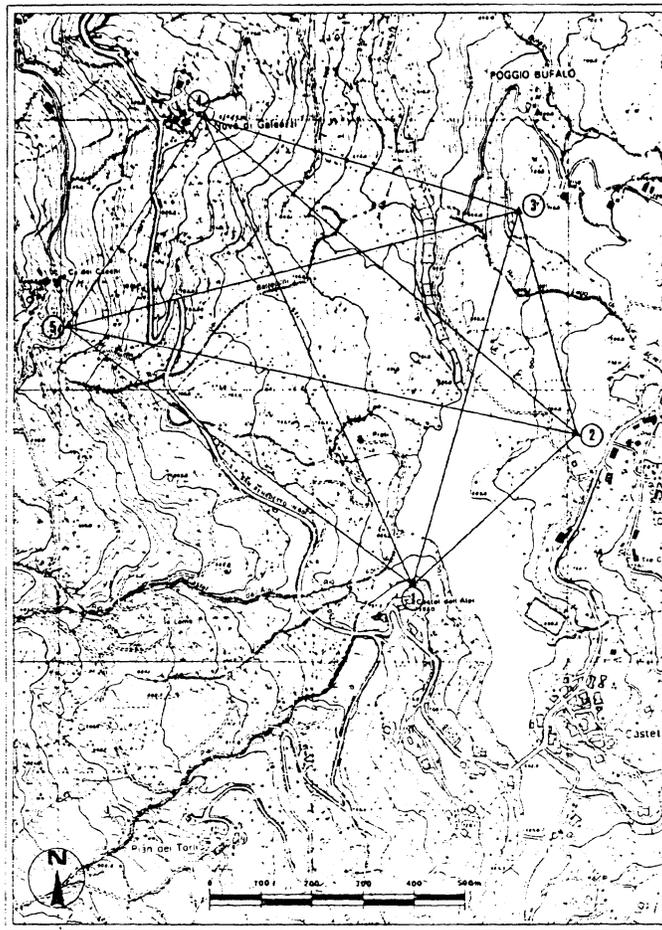
Figure 1. Scheme of the reference network.
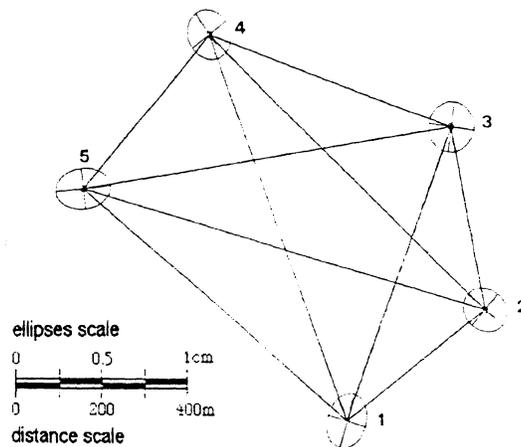
Figure 2. Standard error ellipses of the reference points.

EXPERIENCES WITH ON-LINE MEASUREMENTS OF REDUNDANT DIRECTIONS AND ANGLES

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Introduction

With modern electronic tacheometer instruments, the only human task is to aim at the targets and to record the results by pressing a switch. It is not necessary to note the results by hand, thus any possible errors can be avoided. For standard measurements, there is a serial data collector, which offers the ability to afterwards directly process the field data. This is not the case if you wish measurements of higher precision especially those of redundant directions and angles. With regard to precise observations, there are fewer problems with angles than with distances.

Therefore, we developed software to carry out the method of 'measuring the directions in sets' and 'angle measurements in all combinations' with a standard electronic theodolite of high precision, such as the Kern E2, Wild T 2000, and Zeiss Elta 2. At the end of the observations of a station, adjusted results without gross errors should be presented. In order to keep the operation time short, one should have a well functioning dialogue program.

Demands and Questions to a Program

In developing software, one has to pay attention to some program-specific things, e.g., fixing the storage area. If you wish to observe on several stations, you need a management of data including the elimination of the storage. For the target points at every station as well as for the different stations, it is necessary to test the marking (name or number): 'Was it perhaps used earlier?'

Before starting the observations, you need the sequence of the target points and the whole number of the data! In the case of repeated measurements, e.g., deformation measurements, one can shorten the time of aiming by an approximate value of the angle.

Before starting the programming, there are different kinds of questions to be answered:

- storage of only one station or the work of a whole day (several stations)?
- do you wish a protocol with the mini-printer?
- do you wish a print of a larger size?
- do you wish an on-line data transfer to a larger computer?
- do you wish the possibility of interruption or breaking-off in the process of measurements?
- what has to be done if suddenly a target point is not visible any more?
- reaction to gross errors: elimination and/or repetition of the observation?
- repetition of whole parts of the measurements?
- is it necessary to introduce different weights?

- what has to be done with a big difference between telescope position I and II?

Some Remarks on a Field Calculator

A so-called hand-held computer with a display and a connection to other bigger computers and peripherals can be adapted as a field calculator. In the most cases, the storage capacity of such computers is 32 kilobytes, but now more and more usual are 8-bit processors with 64 kilobytes.

If you have a lot of data, e.g., measurements of angles or directions of several stations, it is better and safer to use external storage. This can be either magnetic tape or disk. In our case it is a so-called RAM-disk. In reality it is not a disk but a plate-storage with access directly to extend the internal memory.

The computer used was an EPSON HX-20 which has a normal storage outfit of 16 kilobytes. It can be enlarged to 32 kilobytes by an additional RAM. The micro-tape takes up to 150 kilobytes. The storage of the RAM-disk is 256 kilobytes. The dimensions of this computer are 29 x 21.5 x 4 cm; its weight including the mini-printer station is 1.7 kg.

BASIC Routines for Adjustment Calculation

For establishing the design matrix automatically, an algorithm is necessary. For adjusting directions in complete sets as well as angle measurements in all combinations, the A matrix consists only of the number '0' and the number '1'. The heart of our BASIC algorithm is a double loop, running along lines and rows to mark the '1' at the right place.

On a hand-held computer, the matrix inversion should be quick without squandering storage. Therefore, we developed a direct inversion routine using only the upper triangle matrix. The algorithm works by exchanging and rotating; it needs only 18 BASIC statements. Algorithms to establish the design matrix for 'measuring directions in sets' and 'angle measurement in all combinations' as well as the mentioned inversion routine shall be given in the final version of the paper.

Gross Errors in Redundant Measurements of Angles

To find out gross errors in redundant systems, the weight coefficients of the adjusted observations $Q_{\overline{ll}}$ are needed. The diagonal elements of the matrix $R = Q_{vv}P$, where $Q_{vv} = Q_{ll} - Q_{\overline{ll}}$, $(Q_{ll} = P_{ll}^{-1})$, show the redundancy contribution of each observation. The matrix of weight coefficients of our measurements consists—in the case of equal accuracy $P = E$ and complete systems—of simple expressions as a result of symmetry. Therefore it is not necessary to calculate the large matrix $Q_{\overline{ll}}$. In this way, it is possible to calculate the so-called normalized residuals with a simple formula for the redundancy contribution and, on the other hand, the manifold of observation accuracy if a gross error is 'possible' or 'rather probable.' (We shall give tables for this in our final version of the paper.)

Some Results

We shall present a comparison of the above-mentioned instruments with regard to handling, time, and accuracy. Another comparison looks at the difference between our

two methods of observation. Also we will show an observation of 9 sets at 10 target points, where a rotation of the measuring pillar appears when we compare the results with those of the angles.

THE USE OF SCATTERED RELEVELLED SEGMENTS AND RELEVELLED NETWORKS IN DETERMINING GROUND VERTICAL MOVEMENTS: A CASE STUDY

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The most commonly used method for determining vertical ground movements in subsiding areas is based on the comparison between adjusted heights at the same bench mark at distinct epochs. In this case, the input of the mathematical model to obtain the representation of ground movements is the displacement at any control point.

This approach requires that a levelling network or different levelling networks, on the condition that they have common bench marks set up in the same area and are linked to the same reference point, are measured at various epochs. If the total length of levelling lines exceeds some hundreds of kilometres, then a long time may be required for the whole survey. In consequence, the results of measurements can be seriously influenced by ground movements, especially in areas where ground subsidence is taking place with great velocity. In order to reduce such influence, levellings must be carried out as quickly as possible, employing several levelling teams acting at the same time. Such a solution, however, is either expensive or difficult to organize. Another way to solve the problem can be based on the use of opportune mathematical models.

Vaníček et al. [1979] have proposed an analytical technique based on the fitting of a time varying algebraic surface $u = u(x, y, t)$ of prescribed degree to geodetic data consisting of relevelled segments and tide gauge records. Relevelled segments, which may be connected or disconnected in space and randomly levelled in time, are treated as tilt elements, while tide gauge readings are regarded as point movements. The solution of the problem is obtained through Vaníček's V.C.M. computer program which computes least-squares vertical displacement surfaces; these and associated standard deviation surfaces can be produced and plotted for n specified dates. Among the inputs to be specified there are: episodes of changing of vertical velocities, degrees of surface in x and y, spacing of prediction points in longitude and latitude, vertical displacements at tide stations, weight factors for relevelled segments, and so on. This program has given very interesting results for the determination of the uplift of wide regions, using data from existing networks levelled at different epochs with various accuracies.

The purpose of this work was to test the capabilities of such a technique also in small subsiding areas. The territory of Modena, a little town situated in the Po river valley not far from the Appennines, was selected. A subsidence phenomenon caused mainly by groundwater withdrawal is taking place in the town and in the surrounding plain, with

serious effects on some buildings. A vertical control network was established in 1981 to monitor ground movements. The whole network has been levelled twice, in 1981 and in 1985; a partial survey, concerning only some lines situated in the urban area, was carried out in 1982. A territory of about 400 km^2 is kept under control and comprises about 225 bench marks; the length of sections ranges from 0.25 km in the centre of the town to 1 km in the external area. The reference bench mark, the height of which has been assumed invariable in time, is situated in the Appennines about 40 km from the town centre. Levellings carried out in 1981 and 1985 have been adjusted using classical methods, and the variation of height of single control points has been computed (Figures 1 and 2).

For testing the V.C.M. program, different sets of data from the Modena network have been considered; disconnected segments have been selected and treated as tilt elements, while no displacement has been assumed for the reference bench mark. Experiments with various areal distributions of relevelled segments and different powers in x and y have been made. Not all the capabilities of the program have been tested as, for example, the option concerning episodes of change in the vertical velocity.

The results obtained indicate that the method requires a previous analysis to select input parameters for the specified area to be controlled. When the model is defined, then the relevelling of a certain number of segments opportunely distributed in the area may be sufficient to portray the distribution of displacements. Such a technique, making use of segments relevelled at different times, should reduce the influence of ground movements on the results of measurements; a continuous survey, carried out by a single team, might provide a monitoring of subsidence phenomenon.

The original V.C.M. program has been adapted to be used with a Macintosh SE personal computer; commercial general purpose software has been employed for the management of data and the plotting of results.

Displacement surfaces have been determined at the epoch of the last relevelling. For this kind of subsidence, in fact, only a short-term prediction of movements should be possible because the physical factors producing the ground sinking may vary quickly in time and in consequence also affect the trend of movements.

Reference
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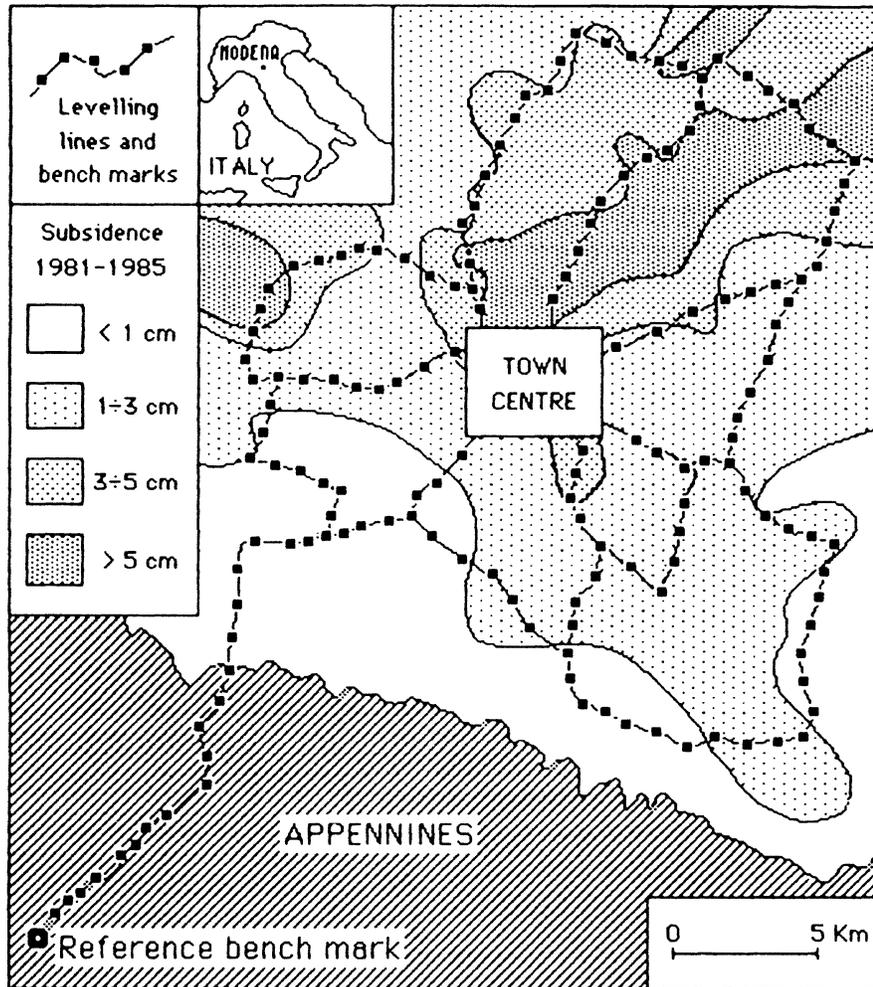
Figure 1. Modena levelling control network and areal distribution of vertical ground movements in the period 1981-1985.
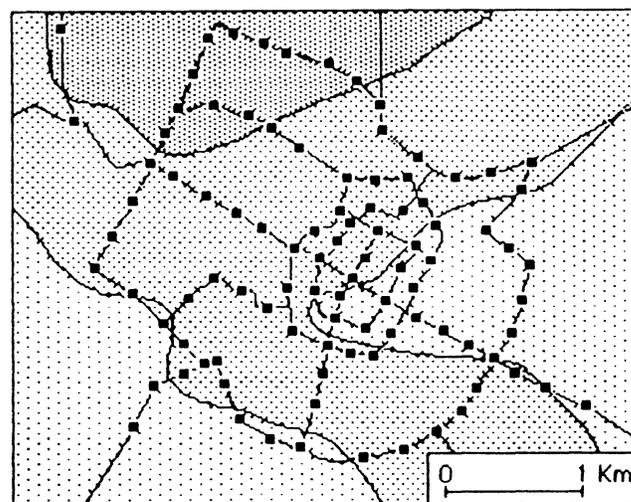
Figure 2. Levelling lines and vertical ground movements (1981-1985) in the town centre.

ASSESSING THE LONG-TERM REPEATABILITY AND ACCURACY OF GPS: ANALYSIS OF THREE CAMPAIGNS IN CALIFORNIA

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Introduction

Relative tectonic motion between the North American and Pacific plates is accommodated across a broad and complex crustal deformation zone in the western U.S. A discrepancy between the tectonic motions predicted by global plate tectonic models and local geodetic observations suggests that significant deformation (~1 cm/yr horizontally) may be occurring west of the San Andreas fault system along the California margin. In order to provide stronger kinematic constraints on the nature of the present-day crustal deformation in this region, M.I.T. and several other universities began a five-year series of geodetic experiments using Global Positioning System (GPS) measurements in central and southern California in December 1986; portions of the resulting geodetic network have been re-occupied in May 1987, and again in September and October, 1987. The design of these ongoing experiments takes into consideration three primary objectives: to assess and improve the accuracy of GPS measurements for studying crustal deformation; to compare the measurements with existing triangulation and trilateration data; and to establish a network for long-term monitoring of deformations along the California margin.

GPS offers several advantages over other geodetic techniques which makes it suitable for the study of crustal deformation. First, a network composed of GPS receivers can span the entire zone of deformation since it is not restricted by the need for line-of-sight visibility between sites, as required by terrestrial techniques. Also, the relative inexpense and mobility of GPS compared to other space-geodetic techniques, such as very long baseline interferometry (VLBI) and satellite laser ranging (SLR), make observing comparatively dense networks feasible. Both of these capabilities are often necessary to adequately characterize complex deformation zones. However, to obtain meaningful results, the GPS-derived baselines must be sufficiently accurate and temporally stable with respect to the tectonic motions. In this paper, we will consider the dominant error sources of the GPS phase observable, and assess the long-term repeatability and accuracy of the estimated network baselines using data acquired during three of the recent campaigns; these results will provide a basis for evaluating the appropriateness of the GPS technique for crustal deformation studies.

Dominant Error Sources of the GPS Observable

The GPS observables commonly used for high-precision geodesy are carrier beat-phase measurements acquired at two frequencies, which are ambiguous by virtue of an

unknown number of cycles between the satellite and receiver. In general, error sources associated with these observables can be separated into source, path, and receiver effects: they are susceptible to long-term instabilities in both the satellite and receiver clocks, as well as poorly-modeled satellite ephemerides and propagation delays of the signal due to refraction in the Earth's atmosphere. Suitable linear combinations of the observables can eliminate errors due to clock instabilities, and can minimize the first-order ionospheric effects which are frequency dependent, although the ionosphere still must be considered in resolving the carrier-phase integer cycle ambiguities. Currently, therefore, the dominant error sources are the satellite orbits, propagation delays of the signal due to the troposphere, and signal multipath near the ground antenna. The relative importance of these sources of error depends on the length of the baselines, availability of orbital tracking data, ionospheric and tropospheric conditions, and antenna design.

The California GPS Campaigns

The California campaigns were designed to study different sources of error, and baseline repeatability and accuracy at a wide range of spatial scales (30 km to 4200 km). In the first experiment, 24 sites of the network were occupied during 15 days in December 1986 and January 1987 along with other sites distributed across North America: Algonquin, Churchill (Manitoba), Platteville, and Westford. Most of the sites were occupied for 5 days in succession, with observations spanning over 7 hours each day. In the second experiment, conducted in May 1987, a subset of the network was occupied for 5 days at 6 sites in California and at Austin, Westford, and Yellowknife (NWT). Finally, in September and October, 1987, observations were made for 5 days at 10 sites in California and at Churchill, Platteville, Westford, and Yellowknife.

The four-month interval between campaigns allowed us to observe at different times of day, thus sampling different ionospheric and tropospheric conditions. The December and January observations were entirely at night, the May observations in the afternoon and evening, and the September observations from sunrise to mid-afternoon, the time of peak ionospheric gradients. We deployed up to five regional fiducial sites in California as well as the continental sites to perform orbit determination on fiducial networks over several scales, ranging from 500 km regionally to 4000 km continentally. The TI 4100 receiver antennas used in the campaigns are subject to signal multipath which may amount to several centimetres in equivalent phase path-length. These effects tend to repeat from day-to-day, due to the repetition of the satellite geometry, and therefore separation of multipath and atmospheric effects may be possible. We have also varied the antenna setup at a few sites with large suspected multipath effects, in order to better characterize their magnitude. Included in the regional network are 6 sites (Mojave, Vandenberg, Fort Ord, Owens Valley, Palos Verdes, and Santa Paula) whose coordinates have been previously determined from VLBI observations; comparisons between the two techniques should provide an independent assessment of the accuracy of our baseline determinations.

MULTIFUNCTIONAL SPATIAL NETWORK IN OPEN PIT MINING

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Recent progress in the use of precise satellite techniques, such as GPS, in engineering geodesy has encouraged some circles to anticipate a prompt decrease of traditional survey technologies. However, what seems more realistic is to be searching for a solution combining successfully the two methods. For this reason, it is required that traditional methods and techniques of surveys be supplemented with precise satellite measurements.

The purpose of the present study is to discuss a concept of a multifunctional topocentric spatial network exemplified by its use in open pit mining.

The multifunctional character of this network is determined by its suitability for cartographic works, designing purposes (realization by construction of buildings and engineering systems), and measurements of deformations. The parameter of its multifunctionality has been identified for the following relationship:

$$M_c = \prod_{i=1}^{n} [P \mid f_i \leq t_{1-\alpha,h} \cdot \sigma_{f_i} \mid = \alpha], \tag{1}$$

where f_i = factors characterising multifunctionality of network,
        $t_{1-\alpha,h}$ = limiting value of Student' t,
        σ_{f_i} = standard deviation of f_i,
        α = probability.

It should be noted that the value of the parameter (at $i=5$) can be considered as high for $M_c > 0.85$, moderate for $0.85 > M_c > 0.70$, and low for $M_c < 0.70$.

Stabilization of the network points is possible to attain by using concrete blocks provided with heads for forced centring of the measuring instruments. Sometimes inclinometric observations under the network points are helpful, e.g., in the measurements of rock mass deformations. For such purposes, a mechanical construction enabling connection of the pipe of an inclinometer with the sign head for forced centring of measuring instruments has been worked out.

The weakest element in the measurements of a local spatial network (horizontal angles, ω; zenithal angles, β; spatial distance, D) is vertical refraction. This problem can be solved on the basis of the results obtained in the test measurements of a given object, with respect to any factors affecting the refraction value. Analysis of the results by the method of multiple regression leads to an empirical equation. This equation enables the calculation of the vertical refraction angle for each sight line of the object

being analysed. The following pattern was applied for the spatial network of a lignite mine 'Turów' in Poland:

$$r'' = -3865.4 + 0.143w + 10.532p - 0.011D + 0.039h - 0.074V -$$
$$- 0.169\delta t + 0.690z - 0.012r^{-1}, -0.336H_s - 0.001w^2 - 0.007p^2 +$$
$$+ 0.001h^2 + 0.021V^2, \tag{2}$$

where w = humidity (%),
p = pressure (kP),
D = distance (m),
h = difference in height (m),
V = wind velocity (m/s),
δt = temperature gradient (°C/100 m),
z = cloudiness,
r^1 = reverse temperature,
H_s = mean height of the sight line above the area.

All these factors are considered significant. Moreover, factors such as temperature of air, time of day, and inclination of the sight line were analysed statistically ($\alpha = 0.95$), but these proved to be insignificant.

Another independent method tending to eliminate an adverse effect of refraction on the measurements of a zenithal angle is the use of light constructions of elevated sight line (to 20 m) installed on spatial network points. A target signal (Polish patent) is made up of light segments (1 m) which by backstay can be set up to the height of 20 m.

Accuracy of the signal's height survey (target point and reflectors) averages ±1 mm. The spatial network 'Turów' (45 points per 80 km^2, length of sides 270 m - 3445 m), measured periodically, using a T-3 theodolite and Wild DI-4L distance meter, is calculated according to:

$$\Sigma p_\omega V_\omega^2 + \Sigma p_\beta V_\beta^2 + \Sigma p_D V_D^2 = \min . \tag{3}$$

The mean errors obtained for location of the network points in a local system of X, Y, H coordinates are found within the range of $m_X = \pm(3.5 - 16.1)$ mm, $m_Y = \pm(3.1 - 20.6)$ mm, $m_H = \pm(9.8 - 26.6)$ mm.

Apart from technological advantages and high accuracy, the network 'Turów' presented above proves to be also very economical due to its high parameter of multifunctionality ($M_C = 0.95$). Such a network can fulfill the requirements of three types of network used for the following operations: map drawing, erecting buildings, and also for testing purposes. In addition, a spatial network is a combined construction; vertical and horizontal. The method of stabilization of the spatial network points also enables the use of satellite measuring instruments.

PLANNING OF DEFORMATION MEASUREMENTS AS THE BASIS OF RELIABLE DATA FOR SPECIFIC INTERPRETATIONS

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Planning of deformation measurements is very important in the process of providing the reliable quantitative data for specific interpretations and prediction of deformations on a given object.

Reliability of the data is affected by spatial and time factors. Figure 1 shows a relationship between these factors.

High reliability of the results ensures adequate evaluation of safety of a given object. It is known that geodetic observations provide quantitative data illustrating deformation effects. The commencement of surveys is a primary factor determining successful interpretations in a causal aspect. However, the measurements are usually delayed because, in the case of engineering objects, they usually coincide with exploitation and not with the initial stages of investing operations.

The following functions can be used for the interpretations, assuming that distribution of deformations is normal:

- exponential function $y = e^x$,   (1)
- linear regression plus sinusoid, $y = ax + A \sin(2\pi T + T_0)$.   (2)

The above functions were used for interpretations of the results obtained in the studies on vertical and horizontal deformations of a water dam. Calculations were made by IBM-PC microcomputer. It is worth noting that the interpretations proved high reliability of the results.

The data obtained in the study were analysed statistically, for the probability $\alpha = 0.95$. They included the following factors (x) affecting displacement (y) of the measured points (22 observations for 15 years); time (T), the level of ground waters in piezometers (H_g), the level of dammed up water (H_p), crustal movements Δh_c), and other factors (z):

$$x = [T, H_g, H_p, \Delta h_c, z] \ . \tag{3}$$

Approximation resulted in the determination of the arithmetic mean of the observed and calculated displacements. It was 1.2 mm for the exponential function and 0.9 mm for the linear regression with sinusoid.

It has been found that displacements of the majority of points are a consequence of centenary and technogenic crustal movements (Δh_c) and time (T). It should be noted

that the reservoir is situated in a non-seismic belt. All other factors proved to be either insignificant or slightly significant.

The equation of linear regression with sinusoid enabled the prediction of the behaviour of the reservoir, being stipulated by augmented levels of water. The data of prediction, whose mean error was found to be $m_0 = \pm 1.5$ mm, should be considered as satisfactory.

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┌──────────────┐    ┌──────────────┐    ┌──────────────┐    ┌──────────────┐
│ Location     │    │ Stabilization│    │ Stability of │    │ Accuracy  of │
│ of  points   │───▶│ of points    │───▶│ reference    │◀───│ measurements │
└──────────────┘    └──────────────┘    │ systems      │    └──────────────┘
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               ┌─────────────────────────────────────────┐
               │         RELIABILITY OF                  │◀──
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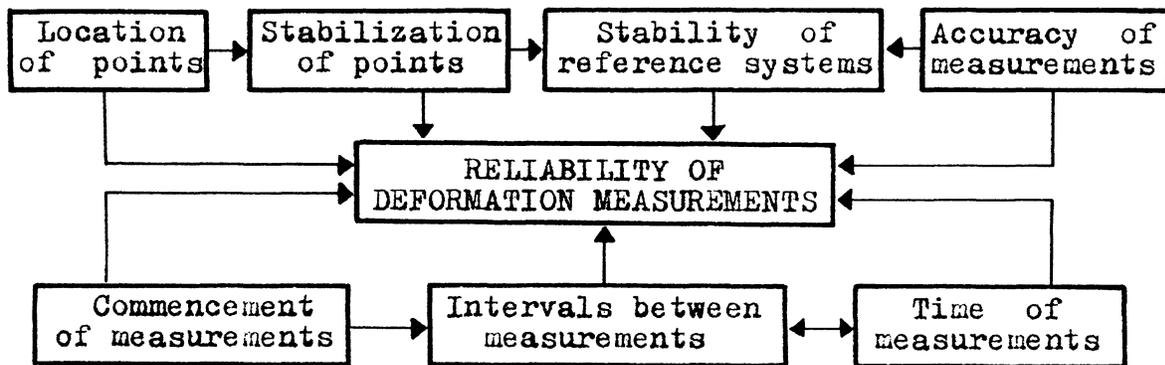
Figure 1. Relationship between spatial and time factors.

DEFORMATION SURVEYS IN P.R. CHINA

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Introduction

Great attention has been paid to deformation surveys in China for the following reasons:

- with a high population density any failure of engineering or geological structures would cause heavy losses;
- the country is in a tectonically active area, e.g., 648 large earthquakes (M>6) took place in this century; and
- a large number of deformation-sensitive engineering structures have been built, e.g., 18,595 dams had been completed up to 1984.

Deformation surveys of large engineering structures started in the mid-1950s. Repeat observations provided full evaluation and safety control of structures. Monitoring of ground subsidences in mining areas began in the latter part of the 1950s. Based on the observation data, subsidence prediction methods have been developed which are especially useful for mining activities under cities, railways, and water. An earthquake in 1966 further pushed the study of tectonic movements. Since then, 120 horizontal monitoring networks of 1300 points in total covering 60,000 km^2 and 20 vertical monitoring networks with 70,000 km of levelling lines covering 1,900,000 km^2 have been established.

Organization of Deformation Surveys

According to the state regulations, any large engineering structure must be monitored. Monitoring schemes are planned in the design stage of projects. Construction companies are in charge of establishing the monitoring systems and conducting the surveys during the construction stage. After the project has been completed, the tasks are taken on by the operation agencies.

The monitoring accuracies, frequencies, and procedures follow specifications which are issued by state ministries. Examples will be given.

Generally, a team of 5-10 persons is assigned to monitor the deformations of a dam of medium or large size and its vicinity. Mining subsidence observations are performed by the local mining bureaus. The State Bureau of Seismology is responsible for the determination of crustal movements. Most engineers in charge of deformation surveys have Bachelor degrees in surveying or geodesy. In the past few years, a number of graduate students have chosen deformation surveys as their research topics. The speciality of surveying in universities offers several courses on the subject, e.g., study of rock movements and ground subsidences in mining areas, deformation surveys of engineering projects, monitoring of crustal movements.

Monitoring Methods and Instruments Developed

Conventional geodetic surveys are still widely used. Precise EDM has become an important tool. More than 20 Mekometer ME3000 instruments have been purchased. Long-range EDMI was developed mainly for the study of crustal movements, e.g., model JCY2 has measuring ranges up to 30 km with an accuracy of 5 mm + 1 ppm.

Specialized instruments have been developed. Tensioned wire alignment and suspended and inverted plumblines have found wide application in the determination of dam deformations. Home-made telemetric coordinatometers with capacitance type sensors and displacement gauges with inductive transducers have been marketed since 1980 and have gradually replaced the optical-mechanical reading devices for the wire alignment and plumbline. The longest wire alignment and inverted plumbline in China are 960 m and 130 m, respectively.

Aligning telescopes with large magnification, e.g., SD-65, have been manufactured. In the mid-1970s laser alignment systems were developed and successfully applied to measure the displacements of linear structures. An accuracy of 1 ppm in open air has been reported. A system with zone plates being remotely controlled was developed at Wuhan Technical University of Surveying and Mapping (WTUSM).

Several tiltmeter types have been developed, e.g., a high-precision hydrostatic level with automatic recording FSQ. Some monitoring schemes used in China will be demonstrated.

Processing of Survey Data and Analysis of Deformations

Compilation of a recent vertical crustal movement map has been recently completed. Three hundred thousand kilometres of precise levelling, done between 1951 and 1982, were utilized. The compilation of horizontal movements is under preparation. The isostrain lines will be employed to represent horizontal deformations.

Free network adjustment and identification of unstable points in reference networks have been of great interest. Different methods, including some based on the theories of robust estimation and fuzzy sets, have been proposed. They can be generalized as weighted S-transformations. The difference between them lies in the way with which the weights are assigned to the different reference points in the datum transformation.

Physical interpretation of dam deformation surveys has been one of the main research topics. The deformation prediction models for all the medium and large dams have been established using statistical methods. Included in the models are three causative factors: temperature (air or concrete), water level in the reservoir, and non-elastic phenomena. Recently, a combination of statistical method and deterministical method using the FEM was tested, and the results were promising.

For prediction of ground subsidences in mining areas, the empirical method is widely applied. The theoretical method and the FEM have also been investigated.

INTEGRATION OF THE GLOBAL POSITIONING SYSTEM WITH GEODETIC LEVELLING SURVEYS IN GROUND SUBSIDENCE STUDIES

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The use of GPS in ground subsidence measurements is a part of a broader international research programme at the University of New Brunswick (UNB) on the use of the Global Positioning System (GPS) in engineering deformation surveys. As a part of the programme, two types of GPS receivers, the Wild WM101 and the Trimble 4000SX, have been evaluated on a test network (see Figure 1) established by UNB near Fredericton in an area of the Mactaquac power dam. Using IMINQE (Iterative MInimum Norm Quadratic Estimation), the evaluation indicates that with the present constellation of GPS satellites the determination of GPS height differences Δh is about twice less accurate than determination of the horizontal components $\Delta\phi$ and $\Delta\lambda$.

For instance, for the Trimble 4000SX, it has been obtained (for distances up to 30 km)

$$\sigma^2_{\Delta\lambda} = \sigma^2_{\Delta\phi} = 4^2 \text{ mm} + (1 \times 10^{-6})^2 \, S^2$$

and

$$\sigma^2_{\Delta h} = 10^2 \text{ mm} + (2.4 \times 10^{-6}) \, S^2$$

with coefficients of correlation equal to -0.72 between variances of $\Delta\lambda$ and $\Delta\phi$ and 0.25 between variances of Δh and the horizontal components.

Similar results have been obtained with the WM101 receivers. Nevertheless, misclosures of height traversing loops with both receivers are well below 2 ppm as is shown, for example, in results obtained with the WM101 receivers (Figure 1).

The GPS surveys have also been tested by the authors in oil fields near Maracaibo, Venezuela, where ground subsidence of up to 20 cm/year is taking place due to the oil extraction in an area of about 50 km x 50 km. Until the present, monitoring of the subsidence has been done by conventional geodetic levelling of high precision with over 1500 bench marks and long line of first-order levelling connecting to deep bench marks outside of the subsidence influence. GPS surveys with WM101 receivers have been tested in the area to replace the levelling frame net measurements which are slow and expensive. Figure 2 shows the test GPS network over a portion of the levelling scheme. Two GPS campaigns at 6 month intervals revealed some difficulties in processing the GPS data gathered in those extreme climatic conditions of high

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 29

temperature (30°C to 40°C) and large relative humidity even that comparatively short baselines (maximum of 18 km) have been used. The difficulties are most probably attributed to the tropospheric effects and lack of knowledge of the actual horizontal gradients in the tropospheric model. Investigations in this area continue, including measurements of meteorological data with balloons.

Despite the difficulties, very encouraging results have been obtained (Figure 2) from the two test campaigns when comparing subsidence values obtained from GPS and levelling surveys. At the time of writing, the GPS surveys are being implemented in the whole subsidence area (Figure 3). The UNB generalized method of deformation analysis will be used in the integration of the GPS measurements with levelling surveys as will be discussed in more detail during the conference.
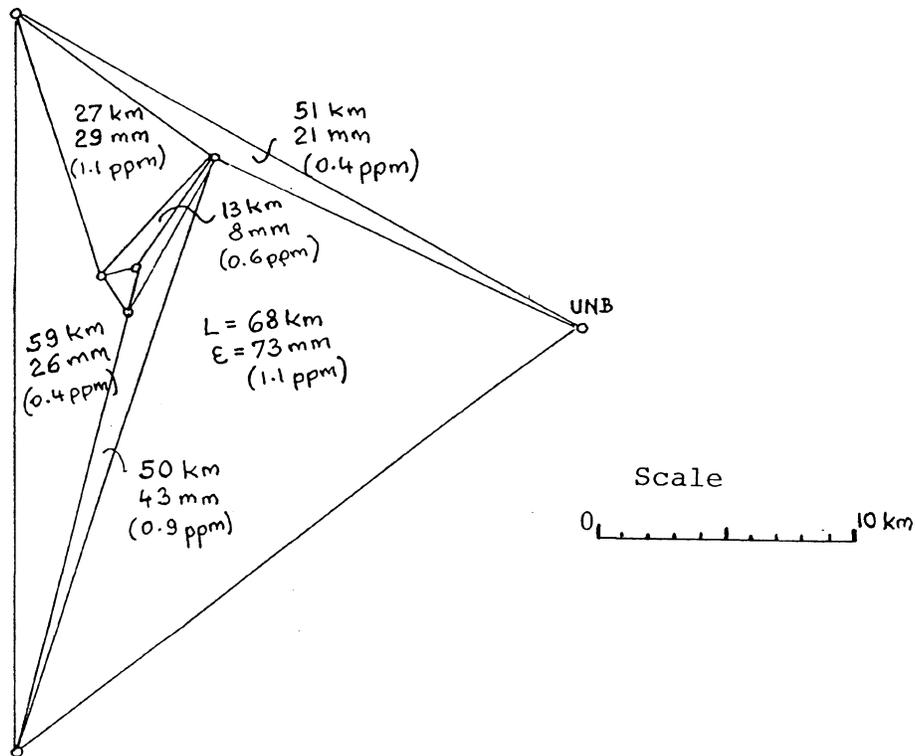
Figure 1. UNB test network. GPS loop misclosures with WM101 (October 1987).
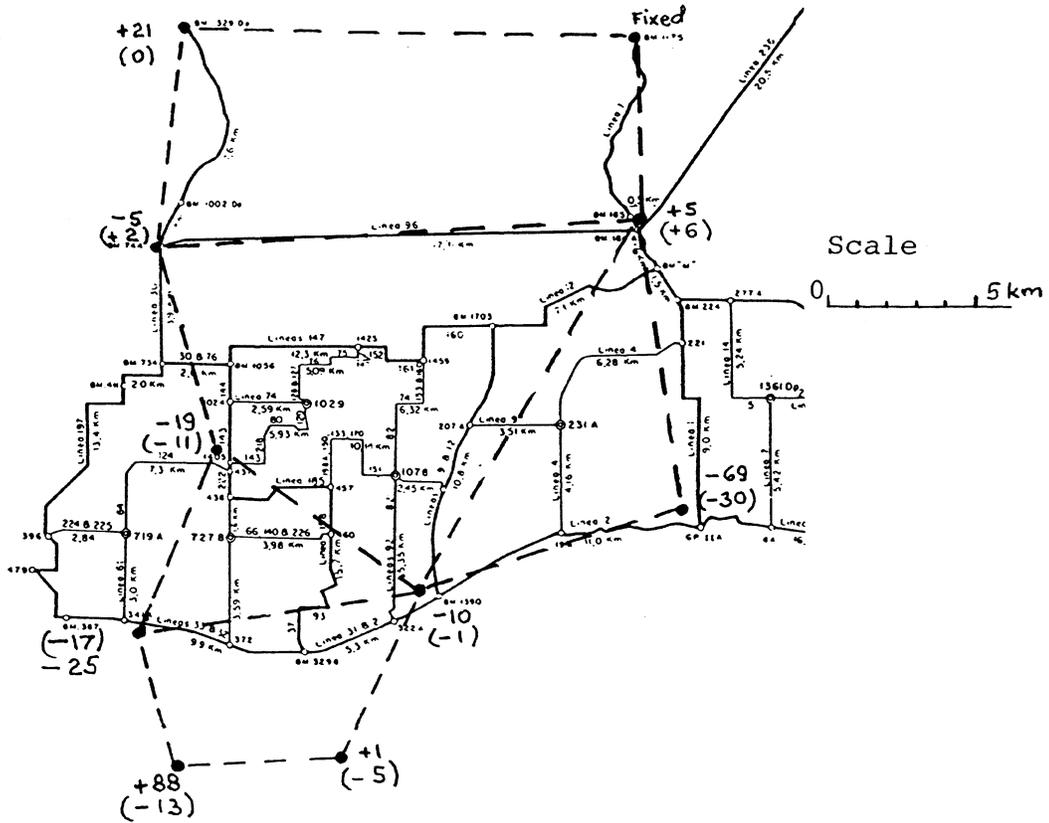
Figure 2. Test network in Venezuela. GPS subsidence in millimetres versus geodetic levelling (in millimetres).

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 31

INTEGRATION OF GEOTECHNICAL AND GEODETIC OBSERVATIONS IN THE GEOMETRICAL ANALYSIS OF DEFORMATIONS AT THE MACTAQUAC GENERATING STATION

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The Mactaquac Generating Station consists of a gravity concrete dam (intake and spillway structures), earth fill dam, and powerhouse located just below the intake. A few years ago, some irregular deformations of the intake and powerhouse were noticed in addition to predicted seasonal expansions and contractions of the structures. In order to find the source of the deformations, an extensive monitoring scheme of several hundreds of observables has been established within and outside of the generating station making it, perhaps, one of the best instrumented hydro-power structures. The observation scheme includes high precision geodetic surveys of horizontal and vertical displacements, inclination measurements with numerous suspended and inverted pendula, expansion measurements with multiple rod borehole and invar tape extensometers, and various types of jointmeters and tell tales. Figure 1 shows just one of the instrumented cross sections of the powerhouse with some average results (in millimetres per year) of repeated observations.

Several thousands of observations have been collected over the last few years. A preliminary evaluation of the repeated individual observations has indicated that in order to obtain an overall picture and an understanding of the apparently complex deformation mechanism at Mactaquac, a simultaneous integrated analysis of all types of observations in the cross sections of the structures would have to be performed. Such an integrated analysis is possible when using the recently developed UNB Generalized Method [Chen, 1983; Chrzanowski et al, 1983; Chrzanowski et al., 1986].

According to the UNB Generalized Method of deformation analysis, the deformation of an object is fully described in three-dimensional space if nine deformation parameters (six strain components and three differential rotation components) can be determined at each point of the object. In addition, components of relative rigid body motion between blocks should also be determined if discontinuities exist in the body. These deformation parameters can be calculated from well-known strain-displacement relations (theory of elasticity) if a displacement function representing the deformation of the object is known. Thus the main task of deformation analysis is to obtain a displacement function

which characterizes the deformation in space and time. Since, in practice, deformation surveys are made only at discrete points, then the displacement function must be approximated through some selected model which fits into the observation data in the best possible way.

If redundant observations are made, i.e., if more observations are available than the number of unknown coefficients of the selected model, the coefficients are determined through least-squares approximation and their statistical significance can be calculated. Of course, the larger the number of redundant observations, the better or more reliable the determination of the deformation parameters. Therefore, one tries to find the simplest possible displacement function, with as few unknown parameters as possible, which would fit into the observations in the best way statistically.

With regard to the analysis at Mactaquac, one has to be aware that the structures are not homogeneous. In the case of the powerhouse, for instance, there is a massive concrete foundation, a number of galleries and other empty spaces, turbines, dilatation gaps, and finally, the practically empty upper shell with the roof structure. The foundation bedrock, which may also demonstrate a deformation, must also be considered in the deformation modelling.

In each area one may expect a different deformation model. Therefore, the selection of displacement functions which would fit the observation data in the best (statistically) way has not been an easy process. For instance, in the case of the cross section shown in Figure 1, about 20 different functions (full or partial polynomial functions) have been attempted in fitting the observation data. In selecting the displacement functions, various possibilities have been considered as far as the rigid body movements of individual zones are concerned: relative rotation of single zone, rotation and rigid body translation of the whole structure, etc. Higher-order polynomials were attempted over the entire structure instead of subdividing it into zones.

Finally, the best model for the description of the displacement field in the whole cross section has been found to be:

$$dx = a_1 x + a_2 y + a_3 xy + a_4 x^2 + a_8 xy^2$$

$$dy = b_2 y + b_3 xy \, ,$$

with $dx = 0$ and $dy = 0$ for the bedrock zone, where x and y are coordinates of points in a local coordinate system (y-axis being vertical). The model has only 7 parameters with a redundancy of 24 (31 observations minus 7 parameters).

In the above model, all the parameters have been found to have probabilities larger than 92%. Figure 2 gives a graphical depiction of the displacement field described by the above model from which the strain field could also easily be found. A preliminary interpretation of the results of the analysis indicates that the structure of the powerhouse is expanding (perhaps due to concrete swelling). The analysis, using the UNB Generalized Method, will be repeated during the summer of 1988 after an evaluation of observations collected over an additional year. In progress, the final interpretation of the deformations will utilize the finite element method in combination with the determined strain field.

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Figure 1. Observation results in millimetres per year, powerhouse, cross section 1 and 2.

Figure 2. Annual rate of displacement versus ellipses at 95%
in a cross section of the powerhouse.

INTEGRATION OF AEROMAGNETIC AND LANDSAT DIGITAL DATA IN MINERAL EXPLORATION

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Collection, storage, analysis, and display of spatial data are common tasks in a wide range of disciplines. The tasks involved are often complex and the past process of 'map-human interpreter analysis' becomes insufficient for projects involving decision making conclusions.

This is particularly evident in geological investigations, where costly operations (e.g., drilling) are designed on a relatively thin layer of information—the geological map. Other sources of information, such as geophysical and geochemical data, can improve the reliability of the geological map.

Computer-aided digital spatial data acquisition and processing have brought a new aspect to data integration. Various data types can be registered to a common grid and operated simultaneously.

The main considerations in data selection include compatible resolution, sufficient density, and suitability for the given problem. Aeromagnetic maps and LANDSAT digital image data of the Lake George Mine area, New Brunswick, fulfill the above criteria in this project.

Data Entry
The CARIS system was used to digitize contour aeromagnetic data and store it in vector format. Vector-raster conversion, reformatting, and scaling was performed to conform to the ARIES II digital image analysis system. Various interpolation algorithms, grid size, and magnifying window were tested. The output was scaled to 356 levels comparable to the displayable range of LANDSAT images.

Data Registration
All data were registered to the UTM projection and 50 m (LANDSAT MSS) or 25 m (LANDSAT TM) grid. The accuracy of geometric correction was evaluated on the basis of segments surrounding each selected ground control point (GCP) (reference point identified on both the topographic map and satellite image). The deviation of the calculated polynomial transform function from the actual location of the GCP was expressed in pixel shift units. Aeromagnetic data were then registered to the geometrically correct satellite images.

Tonal Enhancements of LANDSAT Data

Image segments not pertinent to geomorphology (water bodies, forest clear cuts, etc.) were excluded from the analysed area. This important step compensates for statistical bias caused by usually bimodal or skewed distribution of reflectance values. These 'area specific' images were entered into enhancement procedures such as principal component generation, band ratioing, biomass index calculation, and band normalisation. The relevance of enhanced tonal and texture patterns to geological structures was examined.

Structural Enhancements of LANDSAT

Multi-step lineament extraction procedures designed by the authors were used to delineate tonal gradients possibly indicative of tectonic structures (faults). Water and chlorophyll-sensitive infrared spectral bands were used in multi-directional filtering procedures. Various filter values, sizes, and threshold limits were tested. Different weighting factors were used to compensate for system bias resulting from preferential enhancement of lineaments perpendicular to the sun azimuth direction.

Enhancements of Aeromagnetic Data

Useful image processing techniques included local contrast stretch, artificial illumination, calculation of residuals, second-order derivatives, and lineament extraction.

Simultaneous LANDSAT/Aeromagnetic Processing

Overlay of original data and enhancements in a variety of combinations highlighted the coincidence of subtle patterns on both types of information lineaments detected on both data types were correlated both visually and digitally.

Digital image processing of aeromagnetic and LANDSAT data significantly improved the existing structural interpretation of the Lake George area. Lineaments possibly indicative of tectonic structures, curvilinears pertinent to buried igneous bodies, and tonal patterns defining geobotanical anomalies were studied. Coincidence of these image patterns in different data types improves our confidence in the interpretation of these features.

INTEGRATING LONG-BASELINE ACOUSTICS AND GPS TO MONITOR SEAFLOOR SUBSIDENCE DUE TO SUBMARINE COAL MINING

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This paper proposes a solution to a very real and complex problem: Monitoring seafloor subsidence due to undersea mining. As an example, there currently exist off the east coast of Cape Breton, Nova Scotia, submarine coal mines (see Figure 1) whose undersea mine working are generating conditions of seafloor subsidence. For the efficient and economic removal of the coal, prediction models for subsidence must be used. However, no conclusive methods currently exist to predict seafloor subsidence. This is primarily due to the absence of referenced seafloor subsidence monitoring data, hence, specific empirical models have not been derived nor used. A recent study of methods to monitor seafloor subsidence over longwall workings in submarine coalfields stated that a useful movement detection threshold should be in the order of 30 to 300 centimetres [Jacques, Whiteford and Associates et al., 1984]. Various methods have been proposed to obtain these data. However, none of these methods seem to have the ability to connect seafloor positions to shore control with a high level of relative accuracy. It is proposed that an integrated system of long-baseline acoustics with the Global Positioning System (GPS) can achieve this goal.

Stated simply, there is a current need to make precision measurements on the seafloor at a geodetic level. If one is to work towards a solution to this problem, using the aforementioned approach, one should break the general task of high precision seafloor referenced positioning into three segments: (1) space-based positioning of sea-surface points; (2) acoustic positioning of seafloor points; and (3) modelling and predicting of seafloor subsidence. The major emphasis of this paper is on the space-based positioning segment. This segment involves the use of GPS in a relative kinematic mode (see Figure 2). Only a brief discussion of the acoustic positioning of seafloor points using high precision acoustic transponders will be given. Research involving the modelling and prediction of seafloor subsidence is beyond the scope of this paper.

As with any positioning system used offshore, the dynamics of the environment precludes the ability to obtain multiple measurement epochs at a specific point. Therefore, constrained by the environment, offshore positioning, by some degree, is considered kinematic. The requirement for precise relative kinematic positioning in the context of this paper means the continuous relative, positioning of a point to sub-metre relative accuracy with respect to another point whose location is known in a specified coordinate system. The phrase "continuous...positioning" is qualified to be a continuum which is constrained by the speed with which a measurement process acquires data sufficient for position determination.

To achieve sub-metre levels of relative position accuracy at the sea-surface one must preclude the use of conventional hydrographic positioning systems. Currently, positioning systems can achieve accuracies no better than the metre level for distances in the order of several kilometres offshore. To address the problem in a realistic manner one can turn to the use of the Global Positioning System.

Many methods for processing relative kinematic GPS data currently exist. However, the concept of integrating the two fundamental GPS signal components—carrier beat phase and pseudorange—in a postmission **position** filter and smoother algorithm is relatively new. To date, published test results using this approach have used only static GPS baseline data (simulating kinematic data). Nonetheless, relative positioning accuracies in the order of a few decimetres have been obtained for distances of 40 kilometres. This paper discusses and analyses the results from a test using this integrated observation position filter and smoother approach with real hydrographic relative kinematic GPS data.

If it can be shown that sub-metre levels of relative accuracy can be obtained with real kinematic GPS data in an offshore environment, then the concept of integrating long baseline acoustics and GPS to monitor seafloor subsidence to the same level of relative accuracy will come closer to reality.

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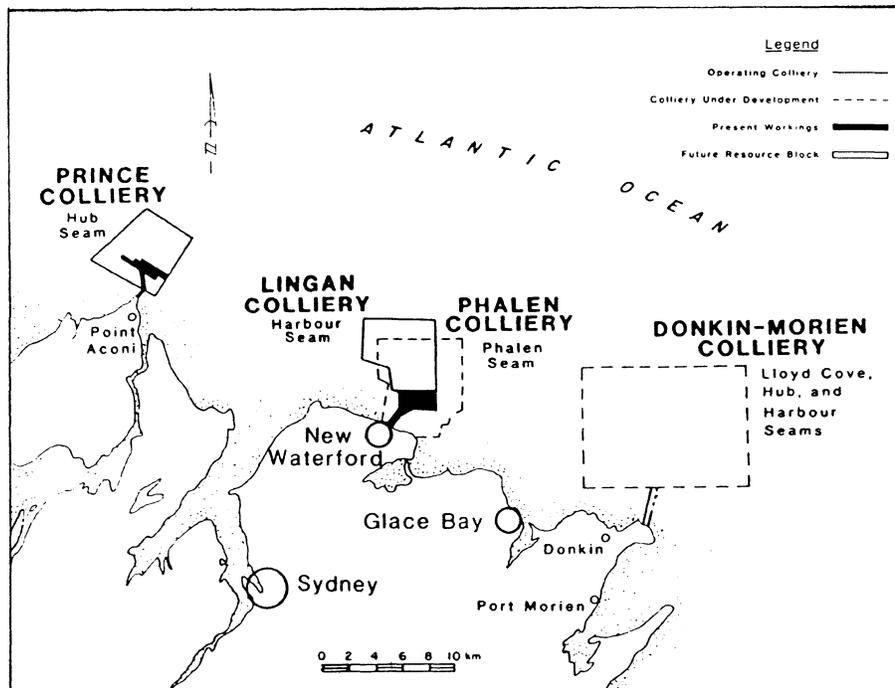
Figure 1. Location of Cape Breton Development Corporation mines.
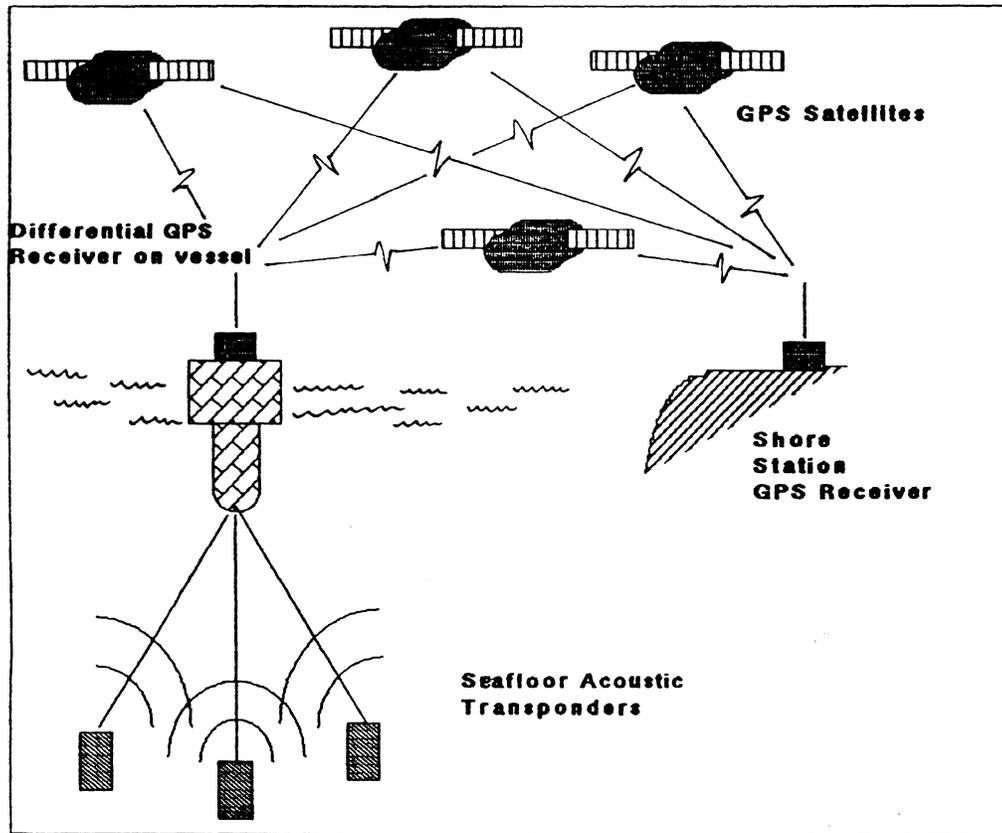
Figure 2. Acoustics and relative kinematic GPS.

LASER ALIGNMENT OF VERY DEEP SHAFTS

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In the recent and coming years, a few pairs of very deep shafts are being dug in China. Their depths are 1000 m or more. For the shaft construction, three types of vertical guide methods or instruments have been used in China. They are plummet point by suspended weight, optical plummet devices, and laser plummet instruments. However, the laser method is the best way for very deep shafts only. In the past 10 years, several types of He-Ne laser guide instruments have been used for the construction of some shafts to depths of about 700 m. But they are difficult for the shafts with depths of 1000 m or more, especially for the shafts with strong drop water. To solve this problem, a type of Ar+ laser plummet instrument has been developed by the China University of Mining and Technology.

Laser physics and our experiments have shown that the attenuance of blue-green laser energy is lower in water or in air with strong drop water, therefore the Ar+ laser, with wavelength of 0.49 - 0.51 μm is an available light supply.

The basic technical data of the developed instruments are as follows:

Effective guide range	> 1000 m
Diameter and deviation of laser beam at 1000 m	40 mm
Telescope system	
Focussing range	3 m - ∞
Magnification	56 $\times$
Laser	
Wavelength	0.458 μm
Power	> 5 mW
Divergent angle	< 2 x 10^{-3} arc degree
Sensitivity of level	10"/2 mm

The deviation of the laser beam will mainly be caused by the following factors:
* non-verticality of the vertical axis of the instrument;
* non-coincidence between the laser beam and the vertical axis; and
* the fluctuation of the laser beam in air.

The first deviation depends primarily on the sensitivity of the level, and will be ±3.3". The second error can be limited to within ±10". It has been shown by our experiments and the theory of laser transmission that the last factor will maximally reach about ±7". The total angle deviation θ will be ±12.6". Consequently, the total plummet error can be calculated as follows:

$$E = \pm \frac{\theta'' \cdot H}{\rho''} = \frac{12.6'' \times (1 \times 10^6)}{206265''} = \pm 63 \text{ mm} ,$$

in which E is the plummet error, and H is the depth of the shafts.

We take the balance point of three plummet points as the final positioning point. As a result, the final plummet error should be

$$M_e = \pm \frac{63}{\sqrt{3}} = \pm 36.4 \text{ mm} ,$$

which meets the requirement of the shaft construction.

At present the instrument has been installed on the uphole of a driving shaft in Shandong Province. Its depth had reached about 750 m when writing this summary. The practical application shows that the potentiality of laser energy will be enough for shaft construction of over 1200 m, because the effective range of the laser can cover 1800 m on the surface under poor climatic conditions. The plummet accuracies are also good.

Figure 1 shows the optical system of the instrument telescope.
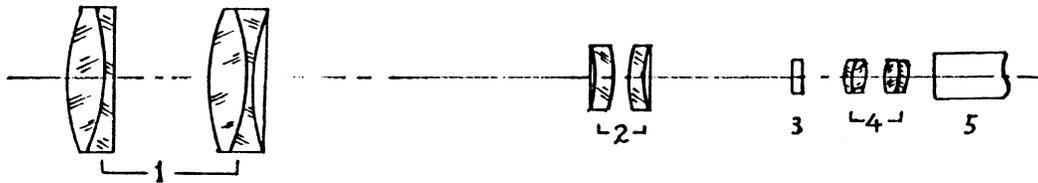
Figure 1. Optical system. 1—objective lens; 2—focussing lens, 3—diaphragm, 4—eyepiece, 5—laser.

MONITORING OF MINING INDUCED STRESSES AT NIOBEC MINE

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The need for monitoring the effect of changes in ground stresses due to mining in hard rock has existed for many years. The Mining Research Laboratories of CANMET, Energy, Mines and Resources Canada (EMR), have developed a new tool which monitors with high precision the radial deformation in boreholes. The result is the Mining Research Laboratories' strain monitoring system (MSMS) which uses the highly stable and very sensitive vibration wire principle to monitor deformations in relatively large boreholes, e.g., 100 to 153 mm/diameter.

Some similarities can be seen to the IRAD vibrating wire gauge, however, the CANMET unit, which is shown in Figure 1, incorporates significant changes. This unit consists of a relatively large calibrated proving ring across which a vibrating wire is strung which vibrates at a low frequency of between 600 and 1800 Hz.

The ring is installed with a hydraulic tool and brought into contact with the surrounding rock with the aid of precisely machined and guided wedges. The resolution of the system is in the neighborhood of 4×10^{-4} mm/Hz. The data collection can be carried out with a portable readout unit but provisions are available for continuous data collection.

The system was tested in cooperation with the Centre de recherche minéral du Québec at the Niobec Mine in Quebec for the purpose of monitoring the effects of drill drift development on surrounding ground stresses.

After access was available to the site, 153 mm diameter holes were drilled for the determination of in situ ground stresses and these holes were then used to install the strain monitoring system. During the development of the drill drift and due to production blasting in stope C20313, the effect on the ground stress distribution could be observed clearly for this particular site.

As an example, Figure 2 shows at three sites in the centre of the pillar the horizontal relaxation which occurred due to mining. This was between 0.082 mm and 0.111 mm. Calculations of stresses from these deformations indicate that the rock material is still in an elastic condition.

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 43
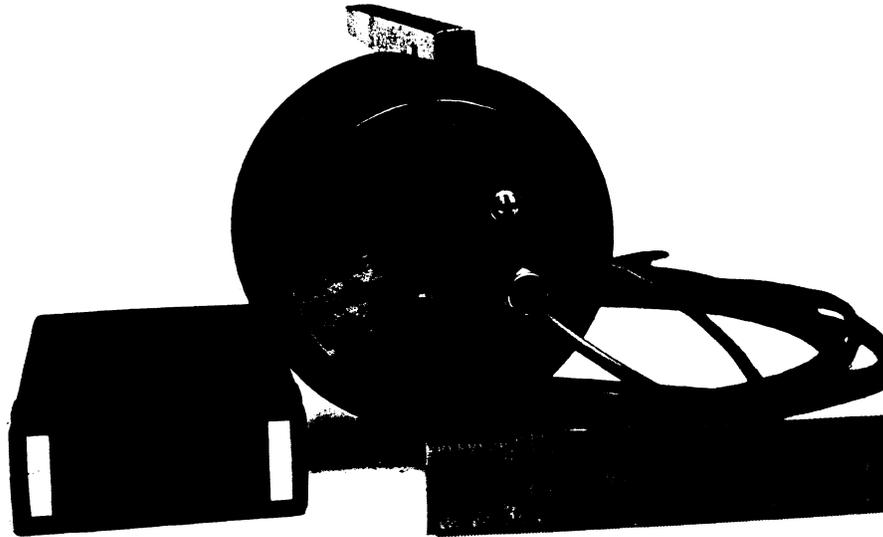
Figure 1. MRL strain monitoring system.
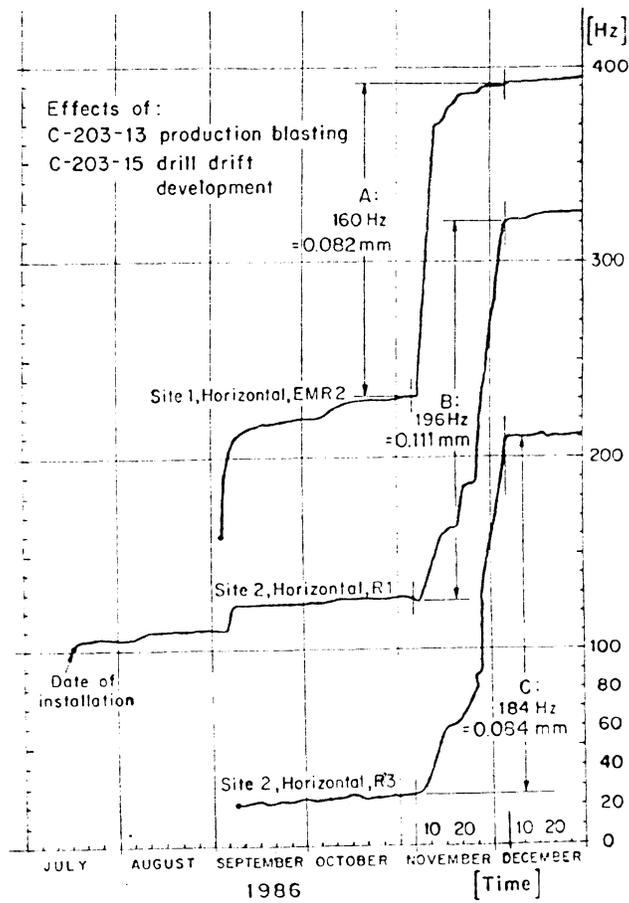
Figure 2. Change of borehole diameter due to mining induced stresses.

AN INDUSTRY-UNIVERSITY COLLABORATIVE DEVELOPMENT IN PRECISE ENGINEERING AND DEFORMATION SURVEYS

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In January 1988, Usher Canada Limited (UCL) and the departments of Surveying Engineering and Electrical Engineering of the University of New Brunswick (UNB) entered into a comprehensive 2.5 year program of collaborative research and development in precise engineering and deformation surveys. The scope of activities addressed under the terms of the program encompass aspects of deformation measurement, analysis, and prediction appropriate to both structural and crustal monitoring applications, and include specific developments in each of the following areas:

- the use and integration of structural and terrestrial geodetic instrumentation and measurement methods in structural engineering and deformation surveys;

- the use and integration of geotechnical, terrestrial geodetic, and GPS relative positioning instrumentation and measurement methods in geotechnical engineering and crustal deformation studies;

- GPS antenna design and data processing methods in surveys requiring high precision and reliability;

- the optimal design and preanalysis of integrated engineering and deformation monitoring networks combining both geodetic and non-geodetic observables;

- the geometrical analysis of deformations;

- the physical interpretation of deformations by empirical and deterministic methods.

Three software packages developed at UNB will figure prominantly in the proposed developments. These are:

- DIPOP 2.0, for the reduction and network adjustment of GPS double difference carrier phase data;

- DEFNAN, for the geometrical analysis of deformations based upon the UNB Generalized Method;

- FEMMA, for the non-linear Finite Element Analysis of ground subsidence in mining applications.

The full paper describes the developments planned in each of the above areas. In addition, an overview of related activities, including the application of GPS relative positioning to the monitoring of oilfield ground subsidence in Venezuela, and the precise alignment of optical beam elements in the construction of a nuclear accelerator at the Chalk River Nuclear Laboratories facility in Chalk River, Ontario, is also presented.

SUBSIDENCE DUE TO UNDERMINING OF SLOPING TERRAIN

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The topography over the northern Appalachian Coal Basin of southwestern Pennsylvania, northern West Virginia, and eastern Ohio ranges from nearly level flood plain to narrow ridges. Locally, slopes of up to 40 degrees may occur. In general, most of the terrain is sloping. The undermining of this type of topography causes subsidence and the effect of local slope on the resulting ground movement has not been resolved.

The U.S. Bureau of Mines has monitored subsidence movements over several longwall mining operations in this region. The comparison of subsidence data from these locales, each with its own topographic characteristics, indicates differences in the ground movement. Unfortunately, data from areas with similar slope characteristics can also exhibit differing ground movements. This may be due to local geologic differences in the strata between the surface and the coal beds being mined.

Others have reported that ground movements due to subsidence caused by the undermining of sloping surfaces can differ from those observed on more level surfaces. Khair has recently reported that, at a site in the northern Appalachian region under less than 91.4 m of overburden, slopes affected the horizontal component of surface subsidence but not the vertical.

The Bureau has developed a predictive model of vertical subsidence movements based upon 11 Bureau longwall panel studies. A BASIC program, for use on a personal computer, has been written to facilitate the use of the model by the mining industry. This study was undertaken to obtain data to verify the application of the Bureau's model to sloping terrain. The study area is located in southwestern Pennsylvania. A set of 4 contiguous longwall panels, each 190.5 m wide, comprised the study area. The average reported extracted thickness was 1.8 m. The Pittsburgh coal bed is the only coal mined in the study area. Overburden within the study area ranged from 207 m to 308 m.

The topography over this set of panels ranged from stream floodplain over the first panel to a narrow ridge over the third panel. Comparison of the ground movement data from the individual panels allows some insight into the effect of slope on subsidence ground movements.

The study site was sufficiently remote from prior mining so as to preclude any subsidence induced surface movements before monitoring commenced. The project lasted 26 months and a total of 65 data sets were collected. For this report, only the

data collected at the completion of each panel are used as the Bureau's model predicts final subsidence. During the study, 4 contiguous longwall panels were mined. Topographic relief in the study area is over 91.4 m.

Two core holes were drilled on the centerlines of the first and second panels, within the study area, to verify the lateral continuity of strata between panels. The information from these holes coupled with core hole data obtained from the mine operator verified that the overburden within the study area is typical of the northern Appalachian coal basin and free from stratigraphic anomalies. Therefore, any differences in observed subsidence movements could not be attributed to local stratigraphic changes. Soil cover in the study area ranges from a few inches on the slopes to a few feet on the stream flood plain. This is underlain by weathered rock to varying depths generally not exceeding 6.1 m.

Control points were established on permanent structures outside the influence of subsidence and used throughout the study to maintain vertical and horizontal control. The survey lines were installed in three phases as mining progressed. Survey pins were driven flush to the ground or to refusal at 7.6 m intervals along all survey lines. Initial position data were established prior to any movement caused by mining. Monitoring was begun when the face was a distance equal to the overburden thickness (213 m - 305 m) in front of the array and continued at weekly intervals during the active movement. A final survey of all points was made at the completion of mining of the fourth panel.

The initial array covered the centerline over the first panel and included a profile. Phase 2 included the centerline of the second panel and two half profiles toward panel 3, one of which extended the profile used in Phase 1. Phase 3 included the centerline of panel 3. Further extension of the monitoring was precluded by restricted surface access. At each survey, the vertical and horizontal position of each pin located on the centerline of the panel being mined and between the centerlines of the adjacent longwall panels was measured and recorded. The location of the face position during each survey was also recorded.

As previously mentioned, one purpose of this project was to verify the Bureau's developed vertical subsidence prediction model, particularly for varying overburden. The model is set up for profiles and predictions were made using the panel widths and overburdens for each panel across a continuous profile. Comparison with the field data shows that the model closely predicted the vertical movement over the three panels. Subsidence over panel 1 was less than predicted, but discussions with the mine operator indicated that the extracted thickness was less than 1.87 m in this area. For the same width panel, increasing overburden thickness will yield decreasing maximum subsidence for critical to subcritical geometries. The model predicted lesser maximum subsidence as the overburden increased and the field data are in agreement. It can therefore be concluded that we are dealing with critical to subcritical geometries at this site.

Jeran et al. have noted that the subsidence over the chain pillars separating two adjacent longwall panels can be predicted using the model and the principle of superposition. This assumes that the chain pillars have not deformed. The field data indicate there has been some deformation of the chain pillars as evidenced by the greater than predicted subsidence observed between the panels. The surface over the

chain pillars between panels 2 and 3 subsided more than that over the chain pillars between panels 1 and 2. The difference in overburden thickness is a probable cause.

The process of subsidence causes the surface to move downward and toward the area of evacuation. The direction of horizontal displacements should therefore be toward the center of the subsidence trough. The horizontal movements measured over panel 1 (level, stream flood plain), after the panel had been mined, generally followed this pattern. This was not so over panels 2 and 3 where the local slope materially affected the direction and magnitude of the horizontal movement. Figure 1 shows the horizontal movements measured at the completion of each panel. Contour lines are omitted for the sake of clarity. There appears to be a continuing horizontal adjustment of the surface after it is disturbed by subsidence. The horizontal movement directly attributable to subsidence should stop with the cessation of vertical movement. The continued horizontal adjustments without measurable vertical movement may represent a mass movement phenomena in which the surface is adjusting to the changes in slope induced by subsidence. Additional data will be gathered as program monies permit.

The physical act of moving does not cause damage to the surface. Damage occurs when differential movements impart strain to the surface. The distribution of horizontal strains computed from the measured differential horizontal movements between adjacent stations at the completion of each panel shows that the horizontal strains imposed during the undermining do not change after mining is completed. Also, subsequent adjacent mining does not alter the strain distribution. At all 3 panels, a zone of compression was developed over the center of the panel flanked by zones of tension inside each rib. There is a marked asymmetry in the magnitude of tensions developed over panel 2; the upslope zone exhibits a larger magnitude of tension than the downslope zone. This difference is to be expected from the implied mass movement of the surface downslope. At this site, the greatest imposition of strain is to the surface by the process of subsidence occurring during undermining and is essentially completed with the mining of the following panel.
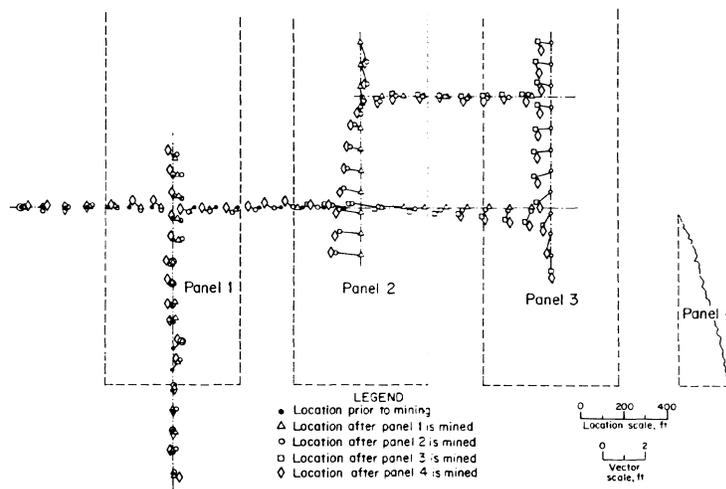
Figure 1. Progressive horizontal movement during study.

SLOPE STABILITY MONITORING USING PRECISION AND RELIABILITY MEASURES DERIVED FROM LEAST SQUARES

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The detection and measurement of slope displacement in western Canadian open pit mines remains primarily the domain of terrestrial surveying methods and equipment. Unlike geotechnical instruments which yield only relative, local information, a survey network defines an absolute reference frame enabling the determination of extent, rate, and direction of slope movement.

The procedure for determining slope movements involves comparing target coordinates determined for each epoch of the survey network. The significance of the movements depends on the precision of the observations and their assumed normal distribution. Thus outliers in the observations, or systematic effects on the observations, could be incorrectly interpreted as significant movements. Results from many slope monitoring campaigns illustrate large fluctuations in the movements of points (in both magnitude and direction) between recent epochs and confirm these incorrect interpretations. Further, these inconsistent results illustrate the unreliability of present observation methods and the need for enhanced mathematical processing of the observations.

Through investigation of slope stability monitoring campaigns in many western Canadian mines, a number of weaknesses are evident in the observation methods used in acquiring the data. Firstly, almost all mines investigated incorporate a two-point datum, utilizing only one station for monitoring, the other simply as a backsight. The monitoring station is often temporary, located on rock which eventually becomes mined itself. Mine surveyors, however, continue to trust the unreliable datum, observing only a direction observation to the backsight and having no outlet to check the stability of the datum. This major weakness must certainly be overcome through inclusion of a slope distance and zenith distance observation to the backsight to enable datum assessment. A second backsight with similar three-dimensional observations or ultimately a second monitoring station is highly recommended.

Observation methods to the target points themselves are also unreliable. Many mine surveyors are neglecting to observe both faces of the theodolite to each target. This is a very simple task which reduces both horizontal and vertical collimation error while providing a quick check for the surveyor against obvious blunders. Research has proven that surprisingly more rounds (mean face-left face-right) are necessary to attain a desired standard deviation.

Presently, almost all western Canadian mines apply a simple geometric fit to their meaned or individual observations to obtain target coordinates. It is a unique solution. A well-designed survey network is generally over-determined. That is, more

observations are taken than are necessary to uniquely define the values for the unknown coordinates. A least-squares adjustment provides the means to solve an over-determined system of linear equations and minimizes the corrections to the observables. Further, the method yields precision estimates for the unknowns, reliability measures which give insight to the controllability and sensitivity of the survey network, and enable statistical analysis of the observations for blunders.

To adequately assess the quality of the determination of the unknown coordinates, statements pertaining to the precision of their determination are required. The covariance matrix resulting from a least-squares adjustment contains a complete statistical statement about the unknown coordinates. From this matrix, confidence regions may be determined which reflect the amount of trust that can be placed on the estimated values of the unknowns.

Unlike precision estimates obtained through random error propagation, reliability describes the effect of true error propagation, both random and non-stochastic, on the accuracy determination of the network. Internal reliability measures give insight to the weak areas of the network. Thus, if weak areas of the network cannot be improved, due to set-up location limitations, etc., then the surveyor has the knowledge that extra care must be used in these areas to protect oneself against the presence of observational errors.

From the concept of non-stochastic error propagation, redundancy numbers may be computed for each observation which give insight to the controllability of the observation. Redundancy numbers reflect the amount of observational error recovered by the least-squares process, as represented by the estimated residual.

Another useful quantity in analysing survey networks is the value of the minimally detectable blunder. This value represents the size a blunder in an observation must be before leading to the rejection of the test on its standardized residual. They give insight to the sensitivity of the network to the presence of non-stochastic errors.

To further enhance one's ability to analyse the survey network, a simple statistical test may be applied to the value of the estimated residuals for the detection of outliers. This test on the standardized residual may be rejected for the following reasons:

(a)   the observation is indeed a blunder;
(b)   the observation is not from a normally distributed population; or
(c)   the wrong (too small) standard deviation was assigned to the observation.

To ensure consistent, precise, and reliable results from any slope stability monitoring campaign, mine surveyors must certainly practise proper procedures and methods for acquiring data. Simple procedures, such as datum assessment, must be performed to reduce incorrect results interpretation and ensure that target coordinates are determined with respect to a stable reference frame. Enhanced mathematical processing through least squares is seen as a very powerful tool in analysing survey monitoring networks. Presently, precision estimates for the unknowns are non-existent in most monitoring surveys. Reliability measures are very useful in located weak areas in the network and can assist in designing subsequent networks to meet precision requirements. Further, geotechnical engineers and others involved in assessing the results of slope monitoring

campaigns must be aware of the limitations and weaknesses inherent in present monitoring procedures and mathematical processing.

EFFECTS OF COLLAPSED EXCAVATIONS ON GROUND SUBSIDENCE IN GREATER DISTANCES

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Introduction
Frequently, deformations caused by tectonic and other displacements within the earth's crust can be observed on the surface. The paper presents a dynamic model which connects these displacements and the subsequent deformations. If both the deformations and the displacements can be measured, the model can be tested by comparing the real deformations with those predicted by the model. This comparison is, for instance, possible in mining areas where the deformations are known from repeated levellings and the displacements from the known dimensions of the excavation.

The Model
The model is applied to many geophysical and tectonic problems. It is based on the assumption of the earth's crust being a homogeneous isotropic elastic medium. In the case of subsidence in mining areas, however, these assumptions cannot be made in full.

The elasticity theory of dislocation [Steketee, 1958] on which the model is based considers a discontinuity of movements, a dislocation, within an elastic half-space which causes a displacement field within the half-space. There are many closed analytical formulas to describe these relations especially when only the displacements on the free surface are of interest. They are, however, restricted to a three-dimensional translation over a rectangular plane with two sides parallel to the surface, e.g., Okada [1985].

The displacements u at a point P are functions of the position, orientation, depth, width, dip, and length of the plane of dislocation and of the components of the burgers vector b describing the relative motion of the two sides of the plane (Figure 1). These formulas have been used successfully by geophysicists and geodesists, e.g., for the analysis of crustal deformations due to ruptures of the earth's crust during an earthquake. The analysis of the uplift of the Long Valley caldera, California, is another example. It could be interpreted by a magma inflation which increased the spacing of the two sides of a dipping plane.

The inverse case occurs when a mining excavation closes (Figure 2). An application is shown in the following.

Application
The model has been applied to a mined coal deposit. The excavation is located 800 m below the surface as a rectangle of 300 m × 500 m dipping 9°. The thickness of the coal seam is 3 m. Levelling lines were run 9 times from the beginning of the exploitation until the stagnation of the subsidence (Figure 3).

A comparison of the observed and predicted subsidence is shown in Figure 4a and b. Obviously the results do not agree at all directly above the excavation. This could be expected since very often deformations in the central region of subsidence cannot be described well by means of elasticity theory. For this region, deformation models have been developed empirically taking into account the fact that rock in the neighbourhood of the excavation does not behave in an elastic but in a brittle manner.

This phenomenon, however, disappears in a sufficient distance from the excavation (Figure 4a). Outside the central region, the predicted and the real deformations agree well. Here the deformations are usually too small to do damage; therefore, the ground is often assumed to be stable. But obviously rock reacts elastically if the distance from the excavation is great compared with the movement at the source of deformation.

Conclusions

The formulas of elasticity theory of dislocation can be applied to deformations in mining areas outside a central region above the excavation. The combination of empirical models as usually applied in mining and the models of elasticity theory of dislocation allows us to draw a complete picture of the deformations due to mining even in areas far away from the source. Further investigations have to be made to separate the areas of validity of both models.

$$u_i = f_i(E_o, N_o, \alpha, D, \theta, W, L, b_1, b_2, b_3)$$

$$i = 1,2,3$$
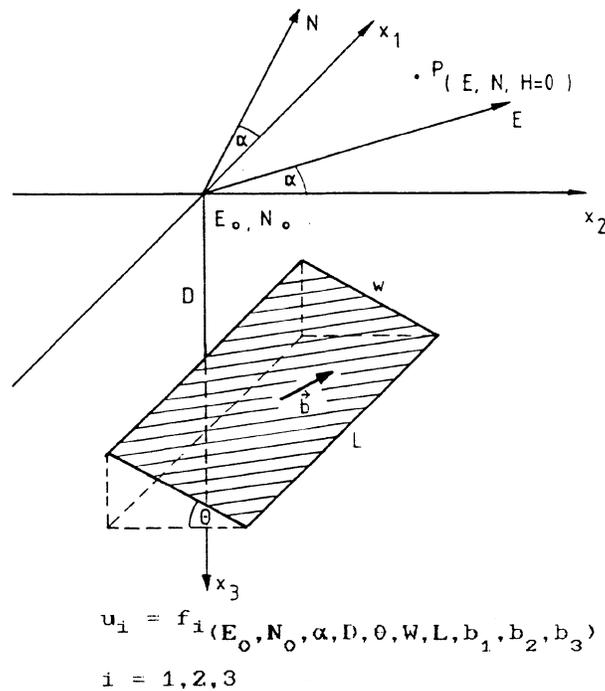
Figure 1. Geometry of a dislocation.
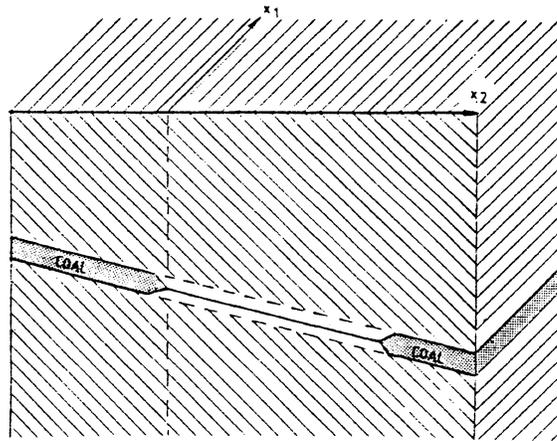
Figure 2. Closed excavation.
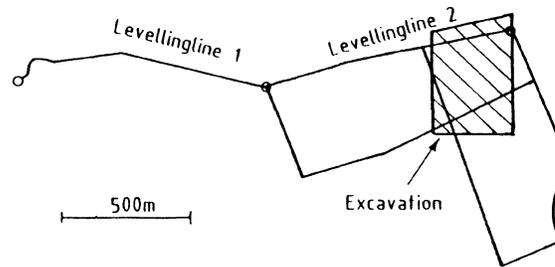
Figure 3. Excavation and subsidence.
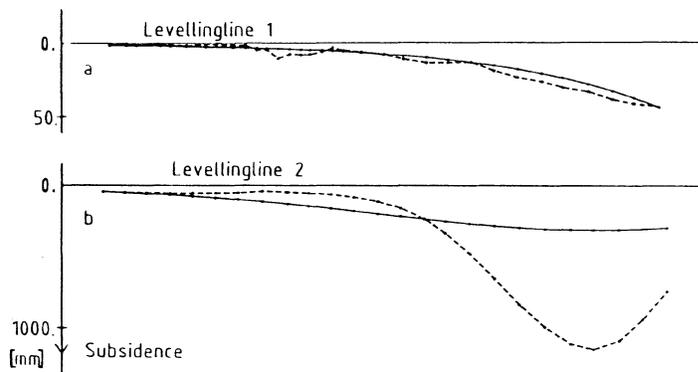
Figure 4. Subsidence of two levelling lines.

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 55

PREDICTION OF GROUND SUBSIDENCE IN LONGWALL MINING AREAS

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When an underground excavation becomes large enough, the surrounding strata is disturbed and displaced. The most severe of these displacements take place vertically and horizontally in the immediate strata above the excavation and gradually decrease as the distance from the excavation increases. Accordingly, the volumetric expansion due to the void spaces in the broken strata is highest in the excavated zone and gradually decreases as the distance from this zone increases. Based on these characteristics, the bulking characteristics of the overburden can be divided into three zones: namely, the caved zone, the fractured zone, and the highly deformed zone. Assuming a unit thickness of the strata cross section, the different areas, which include these three zones and the excavated and subsided areas, are shown in Figure 1, where A1, A2, A3, A4, and A5 are the excavated, caved, fractured, deformed, and subsided areas, respectively. An expansion factor is the percent increase of volume in a particular zone and is denoted by K.

$$\text{Area, A1} = W \times h_s$$

where h_s is the height of excavation and W is the width of excavation.

Based on abutment pressure [Peng, 1978], area A2, the caved zone, was hypothesized as the area under an elliptical shape arch of the side abutment pressure, from the rib (solid coal) to the middle of the panel (gob). As the pressure builds up in the gob, it develops into an arch of stress with a thickness of 1.5 percent of the depth of overburden. The caved zone is assumed to develop two arches from the rib to the center leaving a crush zone of $2(0.015\ h_o)$ in the center of the excavated area, as shown in Figure 1. The width d of these arches will then be:

$$d = \frac{w - 2(0.015\ h_o)}{2}$$

where w is the width of the panel and h_o is the height of the overburden. The height of these arches will range 2 to 8 times the excavation height [Peng, 1983]. It depends on the geology (mechanical properties) and the stratigraphic sequence of the strata immediately above the coal seam. It has been reported that the residual bulking factor of sandstone (hard rock) is 3 to 4 times that of shale (soft rock). Therefore, it can be logically assumed that the height of the caved zone decreases proportionally to the percentage of hard material in the immediate roof. The height of the caving zone, h_c, can be assumed to be:

$$h_c = [2 - 8 \times P_i]\ h_s$$

where P_i is the percent of hard material in the immediate roof.

Prediction of ground subsidence in longwall mining areas

$$\text{Area, A2} = 2 \times (\frac{d}{2} \times \Pi \times (h_c + h_s)/2) - \text{A1} \ .$$

The area of the fractured zone is that area, A3, which falls within the pressure arch above the caved zone. As soon as an opening is made, the vertical load directly above the opening shifts outward to both sides of the ribs, leaving a distressed or relaxed zone in the roof strata. The shape of the distressed zone resembles a parabolic arch and thus it is called a pressure arch. The width, W_p, of this pressure arch will be:

$$W_p = 0.15 \, h_o + 60 \ .$$

Observations in the U.K. coal fields indicated that the maximum height of the pressure arch, h_f, could be as high as twice the width of the excavation [Stefanko, 1980]. However, since the width of the excavation in longwall mining is very large, the height of the fracture zone may not be as high as twice the excavation but it greatly depends on the percent of hard material in the main roof. Therefore, the height of the fractured zone, h_f, could range between 30 to 50 times the excavation height depending on the expansion factors of the main roof material [Peng, 1983]. It has been cited that overburden strata in this zone act as a force-transmitting beam resulting in void space between each beam. Effective bridging of each stratum depends on their physical and mechanical properties that can sustain pressure. The stronger (harder) the rock, the less possibility of the beam strata collapsing. Consequently, the gap between the broken beam remains wide open, hence relatively higher expansion voids in the fractured zone can be expected. The height of the fractured zone, h_f, was assumed to be:

$$h_f = [50 - 20 \times P_m] \, h_s \ ,$$

where P_m is the percent of hard material in the main roof.

$$\text{Area, A3} = (2/3)W_f \times (h_f + h_s) - \text{A1} + \text{A2}$$

where W_f is the width of the fractured zone: $W_f = W + 2(0.015 \, h_o)$.

The area of the deformed zone, A4, is that area which falls between the pressure arch and the line that connects the points of the limits of the surface ground movement and the intact pillars on either side of the excavation (beyond the yield zone from the pillar ribs). The extent of the deformed zone depends on the angle of draw, θ, which further determines the limits of the subsidence profile. The angle of draw, however, depends on

$$\text{Area, A4} = (W_f + (W + 2 \times h_o \tan \theta))(h_o + h_s)/2 - (\text{A1} + \text{A2} + \text{A5})$$

the geology and the topography of an area and, hence, it is site specific.

Area A5 is the surface subsided area which can be represented as an area within the curve defined by a probability function such as:

$$\text{A5} = 2 \times y_{max} \int_{x_o}^{x_{max}} e^{-a(\frac{x}{l})^b} \, dx \ ,$$

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 57

where I is the horizontal distance between the edge of the pillar rib and the center of the panel. The constant, a, was determined from the field data and a relationship between b, I, and related parameters was established. A5 was calculated numerically after determining the expansion factors using the back calculation technique:

$$A5 = A1 - \sum_{i=2}^{4} A_i K_i \ .$$

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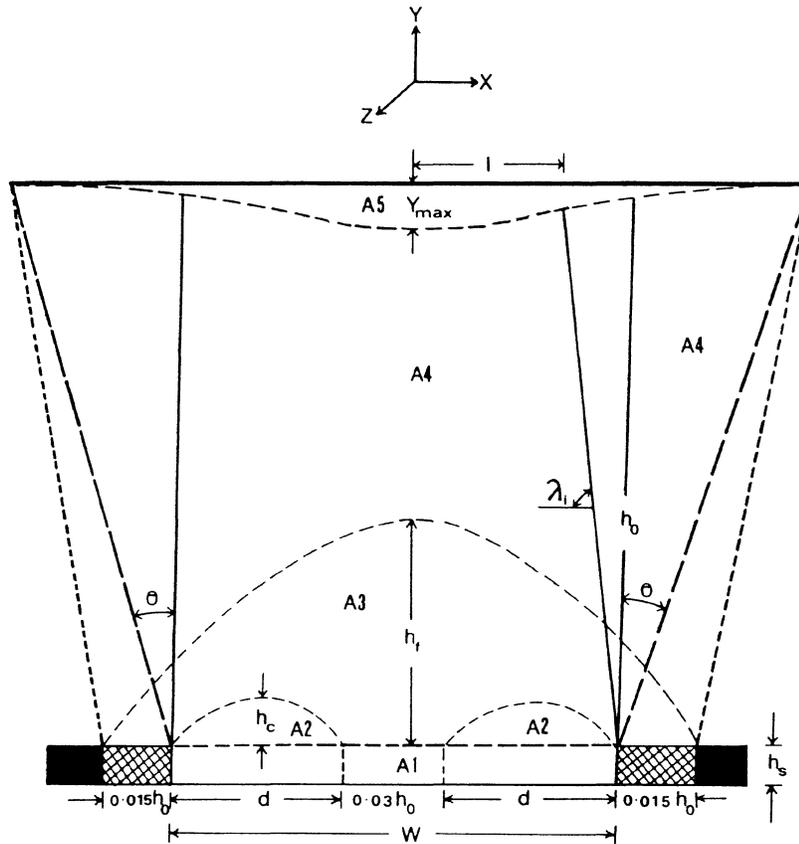
Figure 1. Schematics of overburden deformation characteristics over a longwall excavation.

DETERMINATION OF DRILLHOLE LOCALIZATION FROM CAVE SPACE

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There is a shortage of drinking water in the protected region of the Moravian Karst. The moving water quickly breaks through natural holes into karst cavities, and local brooks frequently disappear owing to sinking into earth and continue flowing as subterranean rivers. The discovery of such a source at the depth of about 135 m in proximity to a municipality gave us the idea of using it as a natural source of drinking water by drawing it through a borehole.

Irrespective of the fact that the cave was inaccessible to the public, the measurements were performed under difficult conditions, the line of stream flow was mapped and a vertical borehole of 300 m diameter was made from the surface downwards to the subterranean river at an appropriate place. Nevertheless, the borehole missed the mark and even the cave was not reached.

The inclinometric measurements did not give reliable results concerning the localization of the borehole with respect to the cave. More extensive blasting work also did not come into consideration as a means of looking for it because the stability of the rock might be impaired and the cave could collapse frustrating the whole plan and financial expenses.

We tried to apply the results of the methods developed by us on the basis of an induced magnetic field in order to localize the position of the borehole. A permanent rod magnet or electromagnet with a big magnetic moment may be used for this location work. The effective range of measurement is, to a great deal, dependent on these factors. In the scope of our research we developed, verified, and successfully employed a few methods on the basis of the vertical (Z) and horizontal (H) components of the total vector (T) of intensity (induction) of the magnetic field of the magnet. The use of individual methods is dependent on the kind of magnet and the position of this axis in space with respect to the measuring position.

An electromagnet was lowered by means of a cable to the cave level. To locate its position, we applied two methods based upon measuring the Z component of its field induction vector. By means of a flat induction coil and a ballistic galvanometer we measured the Z component of its magnetic field induction at points 1 to 7 of a horizontal line approximately at the level of the magnet's centre (Figure 1). By plotting the measured values to the profile status, a curve analogous to Gauss's error curve was obtained. The peak of this curve shows the place in the profile nearest to the magnet. The vertical line running through this point perpendicular to the profile lines is the line crossing the magnet axis. For the determination of the direction oriented to the magnet, i.e., the half-line from the profile to the magnet, a measurement at points 11 to 17 of the line of the profile is to be done in the given situation, at least as to orientation. If this profile is parallel with the original one, then the peak point of

the corresponding curve will be situated in the place of the original perpendicular line and the direction to the magnet will be given by the half-line oriented to the side of greater peak value of the corresponding curves. In case the other profile is oriented parallel then the direction to the magnet is the profile of corresponding perpendicular lines at the convergence side.

The distance l to the magnet can be determined from the peak of the induction B and by its comparison with the calibration curve $B = f(l)$ in the case of the first method. In the other case it can also be evaluated from the results of measurement at two points A and B by solution of a triangle given by the length of their connecting line $\overline{AB}$ and by the angles α, β closing this connecting line and the directions to the magnet.

Based on the measurement results, the place, direction, and distance to the magnet in the cave were determined. By means of these elements, the position of the borehole was located exactly by cutting. Nevertheless, it was in a place about 10 m distant from the expected position.
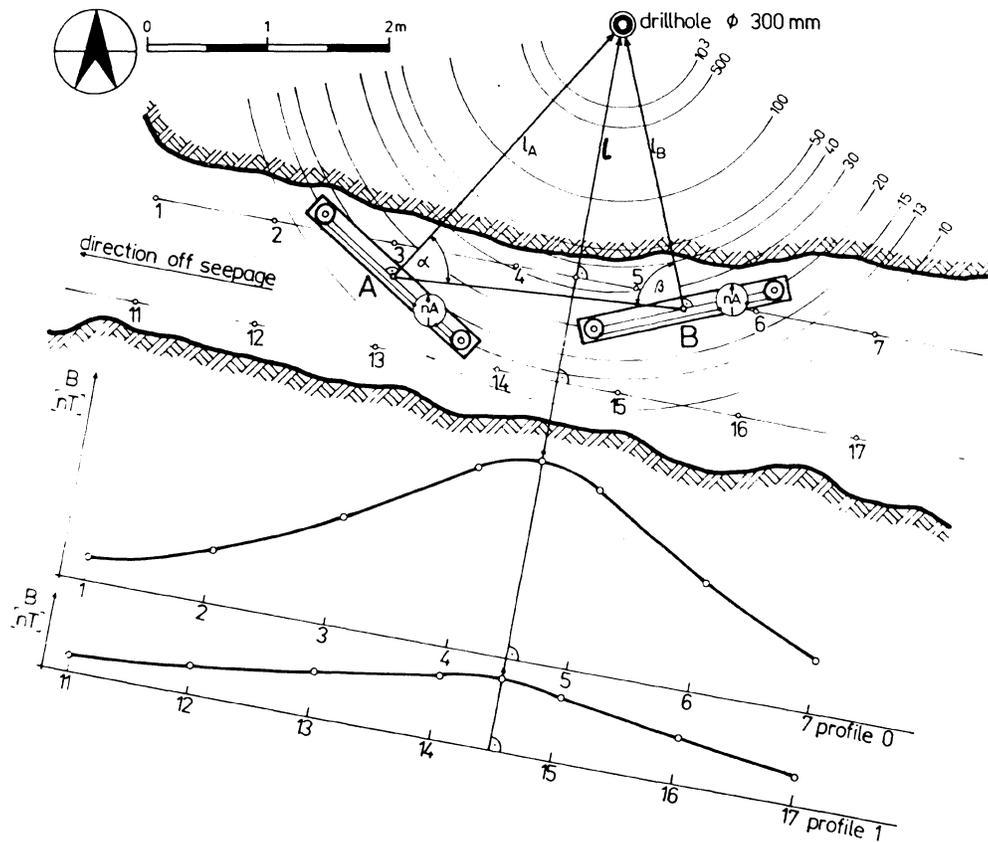
Figure 1. Diagram of locating the borehole from the cave.

MAPPING OF LEACHED SPACES IN A SALT DEPOSIT AND DETERMINATION OF THE UNDERMINED GROUND DEFORMATIONS

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In eastern Czechoslovakia there is a salt deposit at a depth of about 300 m which is extracted through boreholes by leaching. The salt is removed from the brine, which has the concentration of about 300 g.l^{-1} NaCl, by evaporation.

Extraction holes, about 40 m to 80 m from each other, are situated in an irregular network. Due to a high level of brine extraction through holes, wide leached spaces have been formed and continuously interconnected. This has resulted in the danger of collapse of the overlying strata into these spaces, flushing of unmanageable brine volumes, and consequently devastation in the form of contamination of soil, the local stream, as well as all other streams in this drainage system.

Above these larger complex interconnected leached spaces we set up lines of sight fields for examination of collapse effect. Based on the results of the last 10 years of systematic and precise site and level surveying in spring and autumn, spot displacement and deformation vectors in the surveyed areas were evaluated. The characteristic development of depressions above such a complex area is shown in Figure 1, also being suitable for deduction of deformation development, i.e., compression, expansion, and inclination. In the left part of the figure apart from drops also rises in the profile are registered corresponding to the situation behind the boundary of the extraction area, whereas in the right part of the figure the profile proceeds in the collapse area and registered drops are caused by adjoining extractions.

Our prediction of possible collapse of overlying strata into such complex areas was also confirmed. Fortunately, only minor collapses occurred there little by little causing sinking of two cased boreholes whereby the casings and collapsed material formed a compensatory ceiling support.

For the quantitative evaluation of extraction we do not yet have data on the areal extent of the leached spaces for a reliable determination of limit angles.

For mapping of leached spaces, an echo location measurement apparatus was purchased, the measuring activity of which is shown in Figure 2. The apparatus has two vibrators of equal parameters; one is stable in the downward direction, and the other rotates about a vertical axis and may be horizontally oriented in any direction. Location data are transferred by means of a cable to the surface and along with the apparatus depth registered on the chart paper. Vertical profiles can be made by both vibrators by sinking and raising the apparatus in the borehole. By means of the

vertically active vibrator, mainly calibration data can be acquired for the correction of location distances in view of the physical conditions of the environment. While the apparatus is in a rigid position in the borehole the vertically active vibrator is capable of locating the magnetically oriented horizontal profile through a leached chamber in one revolution of 360°. Successively, the echo location mapping of leached spaces was done in all boreholes providing the picture of the status, development, and range of extractions in the above mentioned mine area.

These echo location measurements are highly significant from several points of view. Knowledge of leached space development is important for distance optimization in situating new extraction holes and control of the leaching process in individual leached chambers. Detailed maps or models of leached spaces are of an extraordinary meaning for determination of influence limit angles, estimation of overlying strata stability, preliminary calculation of shift and deformation parameters in a collapse area, and estimation of mining damage.
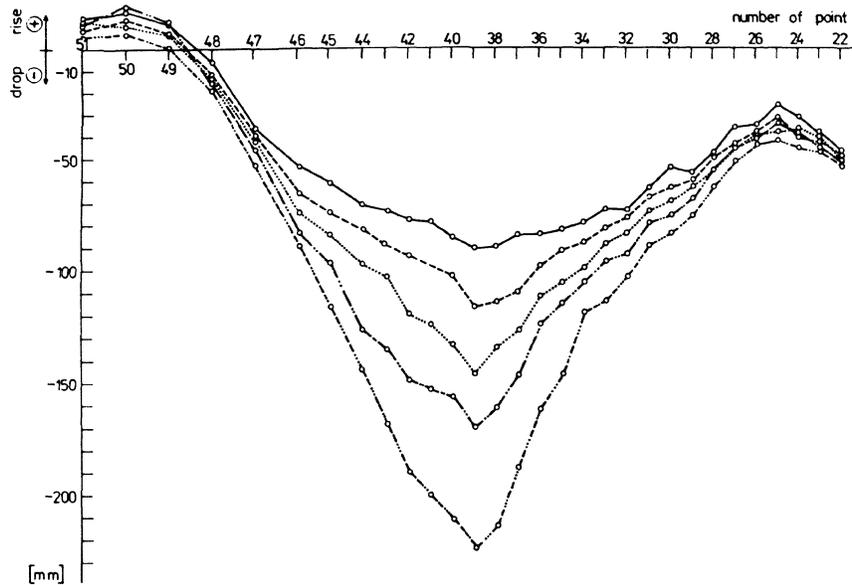
Figure 1. Development of drops in the profile of the collapse area.
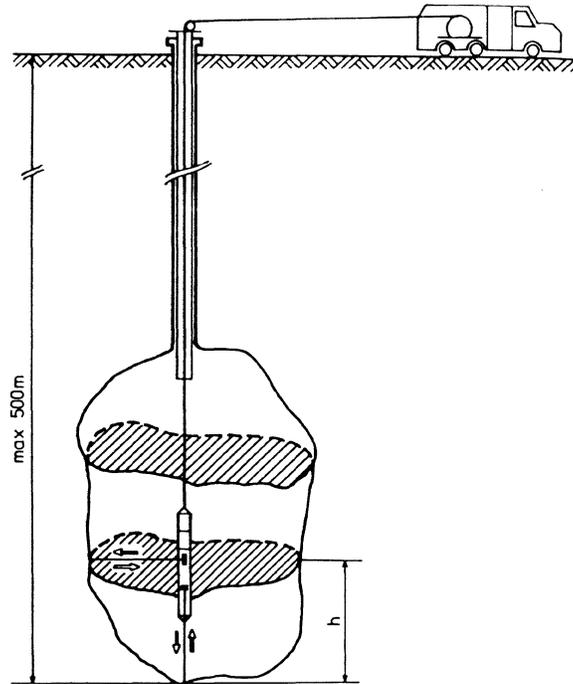
Figure 2. Diagram of echolocation of leached spaces
in the salt deposit.

LONGWALL MINE SUBSIDENCE SURVEYING—
AN ENGINEERING TECHNOLOGY COMPARISON

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This investigation evaluates and compares five surveying methodologies available for use in surface longwall subsidence monitoring. They include EDM-theodolite, tacheometer, and photogrammetry surveying. Inertial surveying systems are discussed, but an in-depth evaluation was impossible owing to on-site system failure.

Geodetic survey work is conducted to establish time-based deformational characteristics of the ground surface as the longwall mine face advances beneath. Surface monuments installed above the mine panel must be accurately and repetitively measured to determine the dynamic vertical and horizontal movements.

The purpose of this investigation was to compare conventional and high-technology geodetic surveying systems using a survey grid with physical characteristics typical of current longwall mine subsidence study sites. Since continued aberrant surface movement during the study would have masked the comparative measurements, a grid of monuments was installed on the Bureau of Mines', Pittsburgh Research Center, Pittsburgh, PA, U.S.A., grounds over a stable surface. The grid was developed over gently rolling, well-drained topography with slopes approaching 30 pct and a maximum relief of approximately 45 m. The grid base line, 533.4 m long, was oriented with two perpendicular cross profiles. One profile was 365.8 m long; the other profile was 198.1 m long. This configuration was typical of actual subsidence grids since it allows cross section capability both parallel and perpendicular to the direction of mining. The monuments were spaced every 25 ft (7.62 m) and consisted of 0.6 m sections of No. 5 reinforcement bars driven into the ground. Although the center of the grid was located in an open field, the ends of the baseline and profiles encountered thick vegetation. Property boundaries forced control monument locations much closer to the grid than normal, but since the site was not being undermined, there was no danger of subsidence alteration. The control monument coordinates were determined to a first-order accuracy horizontally, and to a second-order accuracy vertically. The results and conclusions of this report should be interpreted in light of the prerequisites and requirements of surveying for this particular application. Highlights of the systems follow.

Inertial: Extremely portable in that it can be placed in surface vehicles and aircraft. Sources of error include accelerometer measurement caused by thermal effects, platform drift rate caused by vibration variations and thermal transients, and environmental effects such as variations in the earth's gravitational field and

temperature variations. Survey data could not be used in this study owing to computer malfunction.

GPS: When complete, it will be able to determine receiver position instantaneously. Constraints are the high amperage required to power the field system and sensitivity to temperature variations and site selection. This system was used to accurately determine the position of the control monuments, but the expense and time required to survey precluded its use over the entire grid.

EDM-theodolite: Portable, rugged, and relatively simple to operate. Sensitive to windy and/or rainy conditions and subject to errors due to hand tabulations and computations. Utilized as the base for this comparison because it is the most commonly used method for making subsidence measurements.

Tacheometer: Completely automated for rapid recording, computing, and data generation. Subject to the same environmental constraints as the theodolite system.

Photogrammetry: Can be used for inaccessible areas. All data are gathered simultaneously, providing a permanent photographic record that can be re-evaluated. Surveys cannot be conducted during inclement weather, data for control points must be available, and targets must be visible on the photographs.

A statistical analysis of the resultant data indicated that the three-dimensional displacements from the base (EDM-theodolite) were almost identical for the tacheometer and photogrammetry.

DIGITAL DATA INTEGRATION IN THE SEISMOTECTONIC STUDY OF 1982 MIRAMICHI EARTHQUAKE EPICENTRAL AREA

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The 1982, 5.7 magnitude, Miramichi, New Brunswick, earthquake and subsequent aftershocks remain unsolved in terms of seismotectonic settling. To date, no surface evidence of a fault can be associated with the conjugate 'V' shaped, north trending fault plane solution. The seismic data also indicate a 7 km depth and thrust faulting in an overall east-west horizontal stress.

Digital processing of satellite and airborne imagery complemented by digitized geophysical, topographic, and geological data defined patterns possibly related to structural features. Detailed lineament analysis of NOAA AVHRR, LANDSAT MSS, LANDSAT TM, and airborne MSS coverage revealed a regional lineament in the vicinity of the epicentre (NOAA) and five north-south lineaments in the epicentral block (LANDSAT). As well, it extended the trace of a mapped WNW-SES striking shear zone. All north trending lineaments were evaluated according to their mode of expression on the various types of data. The most dominant, NNW trending lineament is evident on a regional scale and shows offset by the previously mapped shear zone. Ground VLF survey over three of the five lineaments shows the presence of conductors.

Microseismic data obtained from onsite readings in 1985 show that the Miramichi area remained seismically active. Aftershock locations were plotted onto a series of east-west cross sections. Various interpolation algorithms were used to fit a line within the aftershock locations. Surface intersections of this 'simulated fault plane' were correlated with the mapped lineaments. Good correlation was found in the northern portion of the epicentral area, north of the mapped shear zone. The extent of this shear zone was found to be greater than mapped.

Interpolated aeromagnetic data was found useful for lineament correlation and definition of diorite extent in the epicentral area. Spectral and magnetic properties of diorite over a known outcrop were used as criteria to search for similar characteristics in the study area. The results show areas potentially underlain by diorite.

In summary, integrated data support the existence of lineaments in the epicentral area. Five different data types correlate in a NNW trending, overburden covered lineament, which perhaps played an active role in the past and current seismic activity in central New Brunswick.

DISTORTION DURING LIFTING OF A LARGE STEEL STRUCTURE

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Background

This test was devised and carried out by Harland and Wolff/Queen's University of Belfast Teaching Company in August, 1987, to determine the magnitude of distortion which occurs when a large prefabricated steel block is lifted. Distortion can affect the alignment of blocks when they are lifted into position onto the ship, which can cause unnecessary rectification work and delay erection.

The block chosen was a superstructure block, 37 m × 32 m × 10 m high, shown in Figure 1, weighing 640 Tonnes. This block was selected because of its similarity of construction with blocks on future vessels.

Measurement of Distortion

It was decided that photogrammetry would be used to measure the distortion because of the expected movement of the block when supported by the crane. BKS, a British company which specialises in photogrammetry, was contracted to perform this task. Stereo pair photographs were taken of each external face using Wild P31 cameras, while P32 cameras were used to monitor interior movements.

Test Details

The test comprised the following:
- All faces of the block were surveyed while sitting on supports.
- The block was lifted approximately 800 mm and the forward face, port face, and internal bulkhead were photographed.
- The block was lowered back onto its supports and the forward and port faces photographed.
- The block was lifted approximately 500 mm and the forward and port faces photographed.
- The block was moved 40 metres sideways and the aft and starboard faces photographed.

Unfortunately, access to the aft face of the block was restricted by another superstructure block, and this face was surveyed using theodolite intersection while the block was sitting on its supports. It was not possible to measure this face during the first lift, but it was photographed when the block was moved sideways during the second lift.

Dimensional Control

Dimensional control was provided by targets fixed to the ground in front of the block and to stools at each corner. The positions of these targets were located by theodolite intersection using two Kern E2 theodolites and a Husky Hunter portable computer. Approximately 500 paper bull's-eye targets were stuck onto the structure at various positions so that the movements of these points could be measured from the photographs.

Analysis of Results

BKS supplied the coordinates of each target, related to the ground control. They claimed that these were accurate to within 2 mm and this was confirmed by check dimensions. In order to determine distortion it was necessary to transform these coordinates onto the structure. This was achieved using a spread sheet on an IBM PC-AT microcomputer. The origin for each face was translated onto the bottom left-hand target on that face and the X-axis was rotated to pass through the bottom right-hand target. This was performed for each of the lift and rest positions. The relative deformation of each face was determined using the initial rest position as the reference value.

Results

The results show some quite large lateral deviations from a straight line of the bottom edge of the forward face, +17 mm to -16 mm at two adjacent points, approximately 1.8 m apart, even before the block was lifted. The maximum lateral variation in the lifted position was +24 mm. The relative vertical deflections for all faces, except the aft face, were generally small, varying from +4 mm to -6 mm for the bottom edge. Other interesting features were:

- The block did not appear to assume the same shape during each of the lift and rest positions.
- The block distorts about all three axes when lifted but there is no evidence that this is permanent.
- Distortion due to the fabrication process can contribute significantly to the deformation when lifted.

Finite Element Analysis
A finite element analysis was performed on a simplified model of the block. A general purpose Finite Element Analysis suite of programs was used on the VAX minicomputer at Queen»s University, Belfast. The data was input using an interactive graphics program.

Modelling of Block

The block itself is a complex structure comprising many bulkheads and openings (plan view in Figure 2). Bulkheads and decks are formed using stiffening bars welded to the flat plates.

The initial model of the block comprised an upturned, open box with two horizontal panels (decks) and a large opening surrounded by bulkheads—see Figure 3. Intermediate cross bulkheads were also included to corespond with the lifting positions as shown.

The applied loading was due to the self weight of the block, 640 Tonnes, distributed over the members detailed in the analysis. Panels were given an effective thickness because of the presence of the stiffeners.

Comparison of Results

Differences between the results of the experimental and theoretical work can be explained;

- The greatly simplified model does not include the stiffening effects of the bulkheads omitted.
- The model did not include curved surfaces.
- Welding distortion introduced during fabrication, greatly reduces the buckling strength of panels and this could increase the lifting distortion.

Conclusions

The test was useful to Harland and Wolff:

- The initial surveys of the block on its supports highlighted that fabrication welding distortion was present before lifting. Much of this had previously been attributed to the lifting operation.
- The distortion due to lifting was less than anticipated and did not appear to be permanent. It was not of a sufficient magnitude to cause serious fit-up problems.
- The different shape of the block when returned to its supports highlighted that the method and accuracy of setting the block down was very important.

Recommendations

The main recommendations were:

- Adjustable supports should be considered during fabrication to aid fabrication and reduce internal stresses.
- Temporary stiffening beams should be fitted at the fabrication stage to the free edges of bulkheads based on panel size to reduce distortion.
- Blocks should be supported along their edges at the ship, i.e., released from the crane in order to determine alignment problems and rectification work required.
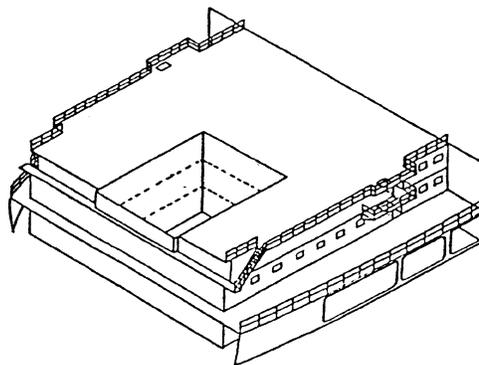
Figure 1. Isometric view.
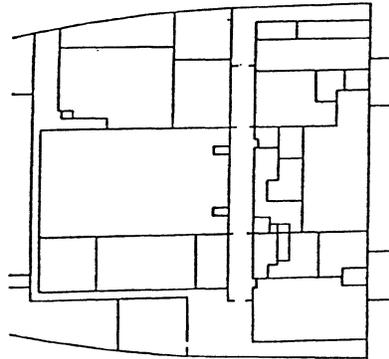
Figure 2. Plan view.
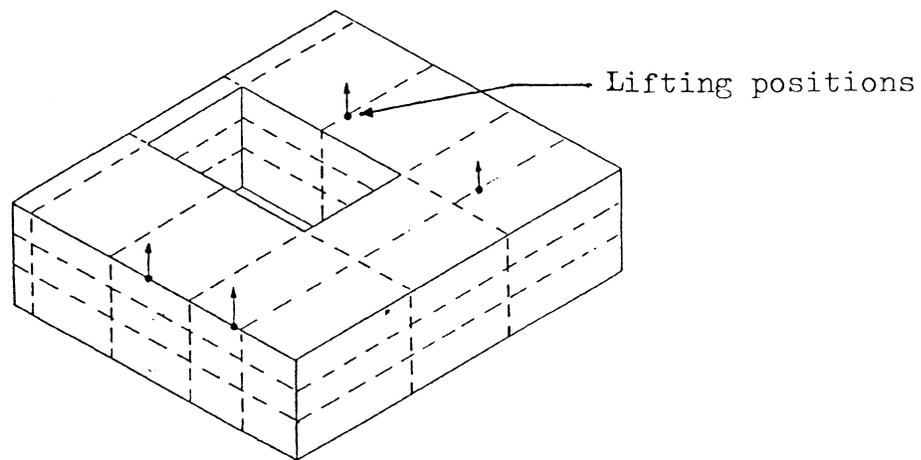
Figure 3. Initial FEA model.

DIGITAL MAPPING FOR MINING APPLICATIONS

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Introduction

One reason why computers are used to map resources is that they are flexible tools. They can be used to store a large volume of data, and allow fast retrieval of information from the database. Such capabilities have been demonstrated by the use of digital map databases to store topographic, hydrographic, and cadastral information. The proliferation of digital mapping systems on the market during recent years is an indication of the success of this concept.

Spatial information is inherently three dimensional. Traditional cartography based on the flat map has emphasized the depiction of special types of three-dimensional objects: points in space, lines in space, and single-valued surfaces. The surface of the terrain, with exceptions such as overhanging cliffs and caves, is single valued. The representation of solid objects, such as ore bodies, is difficult on a flat map and is therefore relatively ignored by traditional cartography.

The representation of solid objects in a computer is easier than on paper, but the problem is not trivial as we shall discuss in the next section. The modelling of solid objects has been studied in computer graphics and the results have been applied successfully in computer-aided design (CAD) and computer-aided manufacturing (CAM). Solid object modelling for mine planning and design address similar problems as in CAD/CAM applications. However, the much larger data volume and the higher complexity of the objects involved create unique problems. This paper will discuss these problems and describe a prototype system.

Three-Dimensional Modelling

The problem of three-dimensional modelling is to represent a solid object in space using digital techniques. It is not a trivial task because most computers today are serial in organization. Data are stored and accessed in a linear sense. This kind of organization is natural for storing features such as points and lines which are not higher than one dimension. The representation of two-dimensional objects, such as polygons, is not impossible but requires careful consideration. For example, the on-going debate on the relative merits of vector versus raster data is mainly focussed on two-dimensional objects.

Solid object modelling presents new challenges. It not only needs more complex data structures, but also requires special manipulating functions to create, update, and display the objects.

There are basically four approaches to represent solid objects: point based, line, based, surface based, and solid based. The point-based scheme uses discrete points to represent a surface. An example is the digital elevation model where a surface is represented by a collection of spot heights. The line-based scheme uses a wire-frame diagram making use of edges and faces to define a solid. The surface-based scheme uses a composition of mathematically defined surfaces to form the 'skin' of a solid object. The solid-based scheme uses solid primitives, such as cubes, to form the object. This last scheme is the three-dimensional equivalent of the raster approach to represent polygons. A data structure for organizing these solid primitives is called the octree which is a method of consolidating smaller cubes into larger cubes, and of indexing these cubes.

Design of a System

The prototype system is based on the octree structure to model irregular solid objects. There are a number of advantages offered by this structure. Among them are its capability to represent irregular objects, and to model full and void spaces with equal ease. This means the same structure can be used to represent ore bodies, mining tunnels, surface mining objects, and cross sections. This uniformity in structure is highly desirable both in terms of computation and organization.

For convenience, other representations are used in the system. Points can be represented by coordinates, lines in vector form, and single valued surfaces in digital terrain models. It is possible to convert these representations to the octree format.

The architecture of the system is composed of four layers. At the lowest two layers are the data structures and the associated data files. Together they contain the necessary mechanisms to store and retrieve the primitive objects. Above these two levels are the modelling operations which are used to create and manipulate the mining objects. Complex queries on geometric and non-geometric attribute information are performed at the highest level.

Implementation of the prototype system was done on a VAX 11/750 computer running under the VMS operating system. The software was written in Fortran. Hardware devices associated with the system include a Tektronix 4125 colour graphics terminal and an Altek digitizing table. The system was tested with simulated and real data.

Conclusion

In this paper we have discussed the problems of representing complex, three-dimensional spatial objects in digital form. We have also described a prototype system which is based on a hybrid representation for point, line, area, and solid objects. This system allows the geometric modelling of irregular geo-solids (such as ore bodies) from other types of input data (such as drill hole information). The project has demonstrated the feasibility of integrating conventional vector and raster data with octree representation to model a wide range of three-dimensional objects.

DETERMINATION OF PERIODIC DISPLACEMENTS OF BUILDINGS AND MACHINES WITH THE AID OF A LASER-INTERFEROMETER

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Fairly often, the dynamic behaviour of buildings or machines is to be investigated. The amount of the displacements and/or the inherent frequencies can be of interest. With the aid of a laser-interferometer, a direct measurement of the displacements is possible. The parameters of the periodic motion, amplitudes, and frequencies of harmonic functions are calculated on-line with a connected small PC.

Measuring System

For the observations, an HP 5526A Doppler laser-interferometer is used. Based on Michelson's interferometric principle, the HeNe laser light is split into a reference and a measuring beam. The measuring beam is reflected by a cube corner mounted at the object to be investigated. Displacements in direction of the light cause a Doppler frequency shift Δf which is read 2000 times per second. After an integration of Δf with respect to time, the displacements of the prism are gained in units of the wavelength of the light or in millimetres. The resolution of the system is about 0.1 μm. The distance between interferometer and object may vary between practically 0 and a couple of hundred metres.

The speed of displacement may not exceed 300 mm/s. Thus, maximum amplitudes, b, result according to the respective frequency f of the displacement (Table 1).

Table 1					
f [Hz]	0.1	1	10	100	1000
b [mm]	477	47.7	4.77	0.477	0.048

In practice, a constant observation rate of 2000 values per second is of little use. Therefore, the display of the interferometer can be read with a varying frequency between 0.01 and 2000 Hz. This is done with the aid of a 100 kHz quartz within the connected computer. This computer, a Commodore CBM 8032, is also used to register and analyse the observations. It is able to analyse about 1200 observations on-line. The results, amplitudes and frequencies of the detected harmonic functions, are printed and graphically shown on a plotter. The whole system is movable and can easily be brought to the object to be investigated.

Spectral Analysis

A periodic function $y = f(x)$ may be expressed by the Fourier series

$$s_n(x) = \frac{a_0}{2} + \sum_{m=1}^{n} a_m \cos(m \frac{2\pi}{T} x) + \sum_{m=1}^{n} b_m \sin(m \frac{2\pi}{T} x) \tag{1}$$

where T represents the period, and the coefficients, a_m and b_m, can be evaluated by the following expressions:

$$a_m = \frac{2}{T} \int_0^T f(x) \cos(m \frac{2\pi}{T} x) \, dx, \qquad b_m = \frac{2}{T} \int_0^T f(x) \sin(m \frac{2\pi}{T} x) \, dx . \tag{2}$$

Equation (1) can also be written as the sum of sine functions:

$$s_n(x) = \frac{a_0}{2} + \sum_{m=1}^{n} A_m \sin(m \frac{2\pi}{T} x + \phi_m) , \tag{3}$$

with amplitudes

$$A_m = \sqrt{a_m^2 + b_m^2} \tag{4}$$

and the phases

$$\phi_m = \arctan \frac{a_m}{b_m} . \tag{5}$$

The spectral analysis consists of calculating the coefficients a_m and b_m by equation (2) and therewith amplitudes A_m and phases ϕ_m of the respective frequencies.

Since in our case $f(x)$ is given in discrete form, that is, observed in equal time intervals ΔT, the integrals (2) have to be calculated by sums. Then, if the number of observations is a power of 2, full advantage may be taken of the resulting duplication of values within the cosine and sine functions. The respective computer procedure is called fast Fourier transform (FFT) and is very efficient. The small Commodore CBM 8032 analyses 512 observations in about 3 minutes.

Now, if T is the duration of the observation
$$f_T = 1/T$$
is the lowest frequency to be detected. On the contrary, if $2N$ is the number of observations in T,
$$f_c = N/T$$
is the highest frequency to be found. Frequencies higher than f_c in the datamay disturb the results (aliasing effect). In addition, we have
$$\Delta T = T/2N$$
for the time interval between two successive observations. Therefore a raw knowledge of the expected frequencies is necessary in advance for a right choice of a priori T and N or ΔT.

Applications

The first example deals with the frequency analysis for a machine with 3750 revolutions per minute or 62.5 Hz. The prism was attached instead of the worked article. With ΔT = 1/500 s, frequencies of 25.4 Hz and 60.5 Hz were found with amplitudes of 0.9 μm and 2.2 μm (Figure 1).

As a second example, measurements in the famous 'Wieskirche' are to be mentioned. Here it was suspected that the bells or deep sounds of the organ made the cupola with

the ancient painting oscillate. The prism was attached to the cupola and the laser system stood on the ground in the church observing directly vertical through a distance of 15 m. The results show that there were no critical amplitudes. While the bells caused oscillations of ±6 μm (Figure 2) and frequencies of 2-8 Hz, the music itself was without any influence.

For the last example, it was suspected that ringing the bells of a church tower leads to a resonant response of the tower. The prism was fixed on the tower near the bells and the laser system stood on the ground at a distance of 60 m (Figure 3). The displacements ran up to ±4 mm (Figure 4). The spectral analysis confirmed that the trinomial exciting frequencies of 0.45 Hz and 0.52 Hz correspond with the natural frequency of the tower.

A laser interferometer together with a connected small computer provide a powerful tool to determine amplitudes and/or frequencies of periodic displacements. The frequency range from 0 Hz to 1000 Hz is wide enough for practically all applications.

GLOCKE 6

$f_{\Delta T} = 500$ Hz
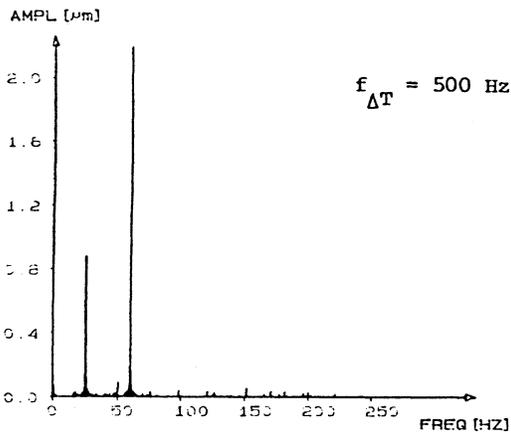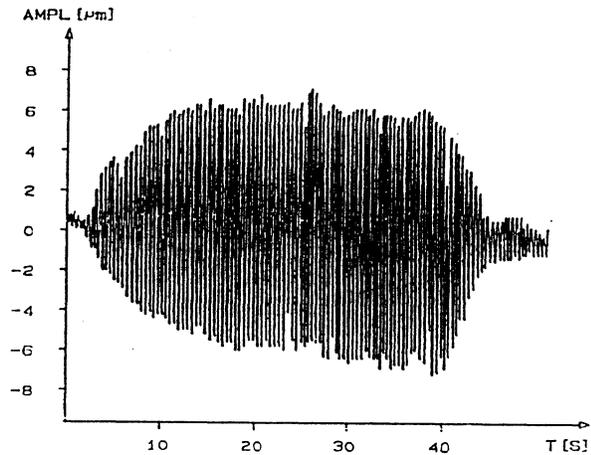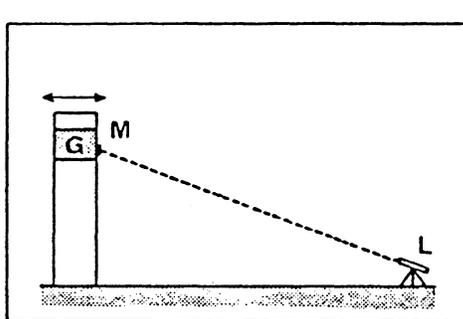
Figure 1.

Figure 2.

Figure 3. G = bell room; L = laser head; M = prism
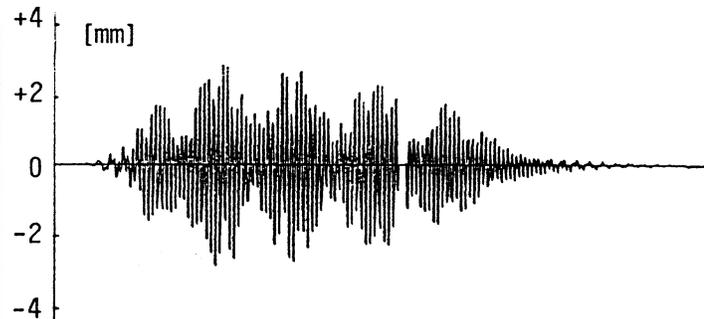
Figure 4.

WIRELESS ROCK MECHANICS DATA TRANSMISSION SYSTEM

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Rock mechanics instrumentation sites are generally located in remote, inaccessible, and often dangerous areas of mines. It is often undesirable for mine personnel to access such sites to monitor instruments such as shear strips, extensometers, and vibrating wire stressmeters. In addition, recording measurements manually is both tedious and time-consuming, often requiring a full-time technician. For these reasons, it may be desirable to have remotely monitored rock mechanics instrumentation, where data from several instrumentation sites is gathered centrally, preferably on the surface, for subsequent analysis. Conventional hard-wired remote instrumentation is vulnerable to line damage, and the installation of cabling suitable for data transmission is expensive.

If a mine-wide radio communication system already exists, it can be utilized to transmit the remotely-recorded rock mechanics data, eliminating expensive wiring and the duplication of existing communications systems.

IMS Electronics has developed a wireless Rock Mechanics Data Transmission System (RMDS) that is used in conjunction with the Montan-Forschung FM Voice Communications System or any other underground communication system. Monitored data is transmitted from the RMDS unit by radio to a base station, where it is then re-transmitted via twisted pair cable to a desktop computer. Each RMDS unit can monitor up to 16 channels of analogue (voltage or current) and/or contact sensor inputs. Typical applications are extensometer, shear strips, and warning devices, such as rock movement monitors. Work is underway to include frequency monitoring, such as vibrating wire strain gauges.

The RMDS is made up from the following:
(1)   Sixteen channel input, differential analogue and/or contact sensor, multi-plexed to a 12-bit resolution analogue/digital converter.
(2)   Microprocessor. An 8-bit Motorola 6802 microprocessor translates each channel into a 16-bit (2 byte) digit. The readings are then combined to form a data gram. The format of the data gram is:

Byte #	1	2	3	4	5		34	35	36	37	38	39	40
	▯	▯	▯	▯	▯	. . .	▯	▯	▯	▯	▯	▯	▯
	preamble		RMD ID#	Channel #1 data			Channel #16 data		Bit count	RMD ID#	Xor check sum	post-amble	

Three data grams are combined to form a complete telegram ready for transmission via a Universal Synchronous/Asynchronous Receiver/Transmitter (USART).

(3) Modem. A Bell 103 compatible modem transmits the complete telegram at 300 baud.

(4) RMDS Output. Data output is either via an on-demand serial port to a hand-held (local) computer or is transmitted over the Montan-Forschung radio communications system. Radio transmission is via the in-built antenna/transmitter.

(5) Data Transmission. Data is transmitted as an FM signal at 430 kHz (the standard MF voice transmission frequency) to the MF base station. Data transmission can be heard on the voice system as a one-second burst of 1 kHz noise. A line driver at the base station amplifies the signal and sends it to the surface via a twisted pair cable. The line driver incorporates both gain and signal limit controls.

(6) Data Recording. Data is received, via a modem, by a surface computer which stores, prints, and/or plots the measured data as required.

The frequency of monitoring is set at the RMDS unit using internal switches and can be varied from once every 20 seconds to once every 4 days. The unit identification number is also set internally, from 1-256.

Except when powering measurement devices and transmitting data, only a low power clock (0.1 microamp) drains power from the internal lithium batteries. Battery life is expected to be approximately 10 years at 10 readings per day.

At Ruttan Mine, the prototype Rock Mechanics Data Transmission System is monitoring a standard rod extensometer installed at 730 mL in stope 27J. The extensometer head requires a 12 volt unregulated power supply which is applied by the RMDS unit to activate rotary potentiometers which supply an output of 10 mV per millimetre of movement. On the surface, a Radio Shack TSR80, which has a built-in modem, is used to monitor and print the monitored data.

PANDA—A MENU DRIVEN SOFTWARE PACKAGE ON A PC FOR OPTIMIZATION, ADJUSTMENT, AND DEFORMATION ANALYSIS OF ENGINEERING NETWORKS

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PANDA is a new software package for handling networks in surveying engineering. Its objective is to serve for most applications of the surveying practice, such as project design and optimization, network adjustment and quality analysis, and to determine significant movements after at least two measuring epochs. The package allows handling of large data sets, represents the latest developments in network theory and deformation analysis, and includes modern observation techniques.

The programs are developed for use on personal computers, like PCs, PSs, or Laptops, consequently it is possible to make all computations and analyses in the field office. As the main routines are written in standard Fortran 77, the package can be used on workstations and mainframes as well. For MS-DOS computers, a special user surface has been developed which allows an easy understanding of the complete system, helps to handle and modify the various files, sets parameters, and starts the different main programs.

At the moment, the two-dimensional version is fully established, tested, and in use; the one-dimensional version is almost ready; and the three-dimensional version will be available about the middle of 1989. The description of the package here is restricted to the two-dimensional version. The software package PANDA consists of three main programs.

PAN: Program for the Adjustment of Networks (see Figure 1). This program was developed to optimize, adjust, and analyse typical survey measurements, including gyro azimuths and sets of relative GPS coordinates. To achieve greatest flexibility, various additional parameters can be introduced, like scale factors, gyro constants and orientation, rotation and scale parameters for sets of coordinates.

The datum of the adjustment can be defined within the concept of free networks (total or partial minimum constraints) or the weak datum (weighted given coordinates) or by fixing point coordinates. The quality of a projected or real network can be analysed using various criteria for precision and reliability. Additionally, outlier testing and variance — component — estimation can be carried out.

Two versions of this adjustment program exist. On computers with 640 KByte central memory, Version 4.1 is restricted to approximately 90 points and 800

observations, while Version 5.1 can handle approximately 900 points and about 5000 observations.

DEFANA: Program for Deformation Analysis (see Figure 2). This program can perform a rigorous or an approximate deformation analysis, using the results of two or more measuring epochs.

If the covariance matrix of the estimated parameter vectors is available, a rigorous analysis will be carried out. This analysis consists of the global congruency test and a localization of single point movements using repeated S-transformations. Due to the a priori knowledge of reference points, a forward or backward strategy can be applied, starting from a minimum or a maximum number of points, which are suspected to be stable. More sophisticated models for the analysis of movements, like strain, block movements, trend or specific predicted values, can be added to this program.

If the covariance matrix for at least one epoch is not available, an approximate technique is applied using, in principle, simple repeated similarity transformations and adequate testing.

A rigorous multi-epoch analysis, using the cumulative approach, is in preparation.

PANPL: Plot Program. This program was developed for the graphical representation of the results of PAN and DEFANA. This package is compatible with HPGL or CalComp plotters. As a result of PAN, the network configuration with or without the observations can be plotted, and the absolute or relative error ellipses can be added. As a result of DEFANA, the discrepancies after a similarity transformation or the significant point movements of the approximate or rigorous analysis can be plotted.

Typical Applications
From the wide range of applications, typical examples will be presented during the poster session:
(1)   Preanalysis and optimization of a tunnelling network using precision and reliability criteria.
(2)   Adjustment and analysis of real dam network.
(3)   Combined adjustment of GPS and terrestrial observations.
(4)   Deformation analysis for two epochs of the dam network.

Observations:

Directions/Angles
Distances
Azimuths
Coordinates

Adjustment Parameters

– Datum : free
fixed
weak

– Additional Parameters
– Simulation or
real observations

Adjustment:

Estimated Parameters $\hat{X}$

Cofactor Matrix $\hat{Q}_{xx}$

Variance factor $\hat{\sigma}_o^2$

Analysis:

Precision, e.g. confidence ellipses
Reliability, e.g. redundancy number
Outlier – testing
Variance – Component – Estimation
Significance additional parameters

Figure 1. Potential of the adjustment program PAN.

Epoch 1	Epoch 2		Epoch k		Epoch 1
$\hat{X}_1, \hat{\sigma}_{o1}^2, Q_{x1}$	$\hat{X}_2, \hat{\sigma}_{o2}^2, Q_{x2}$	● ● ●	$\hat{X}_K, \hat{\sigma}_{oK}^2, Q_{xK}$		$\hat{X}_1$

Selection of Analysis Technique :

– one or two steps
– selection of reference points
– two or more epoch analysis
– approximate technique

2-Epoch-Analysis

1-2	1-2
2-3	1-3
3-4	1-4
.	.
.	.
(k-1)-k	1-k

Cumulative More Epoch Analysis

1-2
(1+2)-3
(1+2+3)-4
.
.
(1+...(k-1))-k

Approximate Technique

For epoch 1

Q_{x1} and $\hat{\sigma}_{o1}^2$

is not known

k – 1

Numerical and Graphical Representation of Results

Figure 2. Potential of the program for deformation analysis DEFANA.

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 81

GEODETIC MONITORING OF DAMS—A COMPARISON OF VARIOUS CONCEPTS

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Introduction

The development of long-term monitoring systems for dams is a classical but still actual problem in engineering surveying. The introduction outlines what a priori knowledge of the deformation model, i.e., the magnitude and direction of expected movements, the area of influence, and the character of movements is needed to develop an optimal monitoring system.

Classical Concept for Dam Monitoring

The well-established classical concept for long-term monitoring of dams is a combined approach, comprising so-called relative and absolute methods. The instruments for relative monitoring are installed inside the dam structure. The devices for continuous monitoring can be regular and inverse plumbings, extensometers, tilt meters, etc. Apart from these, precise traversing and levelling are frequently performed in the control ways. For a major dam, a typical configuration for the combined use of all these relative methods is depicted. These techniques will not be emphasized in this paper; they are outlined to show the problems of connecting the relative inner measurements to the herein discussed absolute measurements outside the dam structure or the area of influence.

As reference for modern absolute techniques, the classical concept of absolute measurements by use of a monitoring network is worked out to some extent. The typical configuration and instrumentation of such networks will be discussed. The criteria for the quality analysis of monitoring networks will be presented, and often found problems will be evaluated.

Potential of Modern Techniques

During the last years, a number of modern techniques have been developed which might have the potential to be used for absolute dam monitoring now or in the future. These techniques are presented and discussed in this paper with respect to their applicability to monitoring problems.

For each method, the general hardware and software requirements are defined and a typical configuration for the application of the technique is given. Subsequently, the achievable quality of the information and the costs and efficiency of the method are discussed.

As modern method 1, the presently available, fully automatic theodolite/tacheometer systems are presented. Using, for example, two electronic theodolites with servodrives, a CCD camera for target identification, and sophisticated software, a huge number of points can be observed repeatedly, i.e., their coordinates can be determined. This technique has the main advantage of allowing continuous monitoring of the object points and a real-time computation of the results. The limitation and problems will be discussed in the paper.

As method 2, close-range photogrammetry and its potential for dam monitoring is considered. Using metric or even amateur cameras, good analytical mono-comparators, and applying the presently available software packages for terrestrial bundle adjustments, this technique starts to compete with classical techniques.

As method 3, the use of differential GPS for dam monitoring is discussed. Differential GPS gives three-dimensional coordinate differences with a relative high precision. The main advantage of GPS is its potential to determine coordinate differences between points without direct intervisibility. Thus the monitoring of a dam relative to reliable reference points further apart from the dam and the area of influence is possible.

Comparison of Classical and Modern Techniques

In this section, the authors make the attempt to compare the four presented methods with each other. This comparison is based at first on usual quality criteria like precision, reliability, sensitivity, costs, and efficiency. Apart from these, the criteria repeatability and long-term stability of the systems as well as the effort for field campaigns and their potential for real-time computations are thoroughly discussed. Special emphasis is given to specific problems associated with the individual techniques. For GPS, e.g., the height problem, the topographic restrictions of the visibility, and availability problems of the complete system will be considered.

Recommended Monitoring System

Looking into the future, considering new dams and neglecting most limitations of the real world, an optimum monitoring system will be proposed which fulfills all requirements from structural mechanics, geology, and surveying engineering.

Being more realistic and considering an improvement of established monitoring systems, certain proposals towards a relative optimum of a monitoring system will be given.

MONITORING AND SIMULATION OF MINING-INDUCED SUBSIDENCE

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A comprehensive instrumentation system was installed to monitor surface subsidence and subsurface displacements associated with longwall mining at a coal mine in West Virginia. In addition to a network of survey monuments on the surface, extensometers and inclinometers were installed within the overburden for the purpose of quantifying the influence which a vertical joint system, horizontal bedding planes, and an overlying abandoned mine had on rock mass behavior and subsidence.

To extend the usefulness of the data beyond this specific mine, a two-dimensional rigid block computer model was utilized to simulate rock mass displacements. The jointed rock block mesh was developed from stratigraphic and joint set data obtained from drilling logs, borehole geophysics, and mapping of rock exposures. Rigid block contact stiffnesses were estimated from intact rock properties. Simulation was performed with five different sets of rigid block contact properties and two meshes with different vertical joint spacings to evaluate the influence of joint stiffness and vertical joint density on mining-induced subsidence.

Subsurface Horizontal Displacements

The stratigraphy consists of shale, siltstone, coal, and arkosic sandstone which contains numerous planes of weakness parallel to bedding due to oriented clay minerals. The measured horizontal displacements indicate that shearing occurred not only at changes in strata (i.e., along bedding planes) but also at planes of weakness within each stratum. The rigid block model stratigraphy was developed to duplicate bedding plane discontinuities but there were not enough simulated horizontal discontinuities to faithfully duplicate the measured horizontal displacements.

Subsurface Vertical Displacements

The measured and simulated vertical displacements were approximately two orders of magnitude greater than the horizontal displacements over the longwall panel centerline. The vertical displacement progressively decreased from 1.8 m at mine level to 0.5 m at the ground surface. It is typical for subsurface caving to propagate up from the mined-out void to a height of 30 to 50 times the mined thickness. For this longwall panel, the mined thickness was 1.8 m so it would be expected that subsurface caving would propagate anywhere from 54 m to 90 m above the coal. The inclinometer casing was squeezed or sheared off at 55 m above the coal seam so this was consistent with expected response. Furthermore, this extent of block caving was simulated by the rigid block model.

Rigid block simulation indicates that displacements are controlled by the vertical joints in that a 'plug' of blocks subsided into the mined out area and then surface movement was controlled by orientation of bedding planes and by topography. This type of behavior deviates from the commonly accepted view of a cone of displaced strata above a mined out area.

Surface Displacements

For all cases, the simulated subsidence trough extended 150 m to either side of the centerline which translates into an angle of draw of 24° for an overburden thickness of 200 m. The maximum simulated subsidence was 293 mm. The maximum measured subsidence was 512 mm, and the measured subsidence trough extended 300 m to either side of the centerline which translates into an angle of draw of 36°. The NCB [National Coal Board, 1975] predicts a maximum subsidence of 850 mm and a trough extending 200 m to either side of the panel centerline. Based on the results of the rigid block simulation, it has been possible to gain some insight into the differences between measured and predicted subsidence.

Interpretation

It has been noted by other researchers that the maximum subsidence decreases and the angle of draw decreases as the percentage of high stiffness strata in the overburden increases. The simulated case in which a relatively low stiffness value was maintained constant for all contacts produced the best agreement between measured and calculated displacements. Variable, but overall higher, contact stiffness as well as slightly denser vertical jointing produced less agreement. Increasing the vertical joint density resulted in less subsidence as rigid blocks interlocked. Also, the measured response revealed more instances of horizontal displacement between strata than was possible with the rigid block simulation. The inference for overall rock mass behavior is that the density of bedding plane type discontinuities and the relative stiffness of the bedding plane discontinuities has a significant influence on subsidence associated with longwall coal mining. It was similarly noted by other researchers that increasing the number of bedding plane joint elements of reduced stiffness in a finite element mesh increased the magnitude of maximum subsidence.

THE DATUM ISSUE IN DEFORMATION ANALYSIS

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Assuming the existence of a stable reference frame, deformations are demonstrated by the varying positions of a network of object points. Deformation between any two epochs can be evaluated by integrating the time derivatives (velocities) of the object-point positions. Measurements performed in a sequence of epochs provide the raw material for estimating point-velocities. Frequently, however, all we have at our disposal are measurements, which pertain only to the geometry of the network. In such cases there is no alternative to adopting kinematic model for the analysis. In a kinematic model, point-velocities are modelled as linear or higher-order polynomial functions of time. The coefficients of those polynomial expressions are thus the only objectives of the analysis.

If the network which is being investigated can be described as a physical system, the above geometrical measurements can be analysed by a dynamical model. Such a model would depend normally on a set of physical parameters which may be known a priori or could be estimated as an integral part of the adjustment process. A dynamical model can be employed also when, in addition to the geometrical measurements, there are other auxiliary data available, which are relevant to the physical system. Examples of such data are: temperature, gravity, water level, etc.

When using a dynamical model, point-velocities are still the objectives of deformation analysis although we may have now the means for modelling their time derivatives as a function of the prevailing force field. The result is a set of second-order differential equations of point-positions whose analytical or numerical solution can be used directly for evaluating deformations. It is common practice to represent the solution of such a set of differential equations as a linear dynamic model.

In a linear dynamic model x—the state vector of the network (a vector of point-positions and velocities at a given epoch)—is represented as a linear function of two fixed vectors: y—the initial state vector—i.e., the state vector at the standard epoch, and z—the vector of physical parameters. The vector x is related to y and z through F—the state-transition—and G—the parameter-sensitivity matrices—by the following well-known equation:

$$x = F \cdot y + G \cdot z . \tag{1}$$

The F and G matrices, which are in general time variable, can be evaluated by various methods. One of those methods which is general and easy to apply is based on the numerical integration of simple variational equations. The initial values of F and G (at the standard epoch) are the identity and zero matrices, respectively. In order to evaluate deformations in the network between any two epochs, all we need are the y and z

vectors and the F and G matrices, which pertain to those two epochs. As F and G can be obtained for any epoch, deformations can be evaluated between epochs where no observations have been made.

The adjustment of measurements by a dynamical model is similar in form to adjustment by a kinematic model. The observation equations and, in particular, A—the design matrix—are identical:

$$l + v = A \cdot x \ . \tag{2}$$

The only difference is in the fact that x—the vector of parameters—is variable. We substitute equation (1) for x in equation (2) and thus transform the system of observation equations into a conventional form with fixed parameters as shown in the following equation:

$$l + v = A(F \cdot y + G \cdot z) \ . \tag{3}$$

We make the convenient assumption that in terms of quantity and distribution (in space and time) the measurements do not give rise to any configuration defect. Yet equation (3) is defective due to the column rank-defect of the A matrix. In a dynamical model, datum still needs to be defined. It can be shown that the (F, G) matrix is a full row-rank even at the standard epoch. As a result, equation (3) has the same defect as in a kinematic model. Thus the datum defect in a dynamical model can be resolved by means identical to those applied in a kinematic model. In a way similar to what we do in a kinematic model, we impose a minimum norm condition on the state vector x. The datum basis needed for the application of free net constraints is a nominal initial state vector y. As a result of the arbitrary choice of y, the estimate of z—the vector of physical parameters—is also conventional, i.e., datum dependent.

We performed two numerical tests with dynamical models. In the first experiment, measurements of a free falling body were analysed. The physical model of the system is very well known with g as the only physical parameter. In the second experiment, the subsidence of a multi-story structure was investigated. We had at our disposal spirit-levelling measurements made at intervals of a few months. We did not have, however, a suitable physical model for the behaviour of the structure. On the basis of auxiliary data which were collected in addition to elevation-differences, an empirical model was utilized. In both cases respective state-transition and parameter-sensitivity matrices were evaluated by numerical integration. Comparison with alternative solutions showed the clear advantage of our solution in terms of a much better fit to the measurements. We should point out that in the second case, in spite of the good fit to the measurements, the empirical model turned out unsatisfactory for prediction purposes. We should remember that an empirical model cannot reflect properly the unknown factors of the force field.

We regard the results of our study as reported in this paper as highly encouraging. We feel that further efforts should be invested in studying the application of dynamical models in deformation analysis.

MONITORING AND PREDICTION OF SURFACE DYNAMIC SUBSIDENCE DUE TO LONGWALL COAL MINING

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Instruction

When undermined by a longwall panel, the stability and integrity of a surface structure may be affected. The development of structural damage is directly related to the relative location of the longwall panel and the advancing longwall face, and the rate of the face advancement. To minimize possible damage to the surface structures induced in the subsidence process, accurate prediction of dynamic subsidence is needed.

Subsidence Monitoring Program

Ten longwall panels in two underground coal mines in the northern Appalachian coal field were monitored continuously for surface movement as the longwall faces moved from panel to panel. An electronic total survey station was used for field measurements. In each survey, both vertical subsidence and horizontal displacement were measured.

Dynamic Subsidence Prediction Model

Mathematical Model: It is assumed that the subsidence velocity is normally distributed with respect to the advancing longwall face. The subsidence at any point in the subsidence trough at a particular time is

$$S(x,y) = S_x \frac{S_y}{S_{max}} \tag{1}$$

where x,y =   coordinates of the prediction point in the coordinate system shown in Figure 1;

S_x =   subsidence along the panel centerline assuming the panel width is critical or supercritical

$$S_x = \sqrt{\frac{2}{\pi}} \frac{V_{max}}{l+l_1} \frac{1}{v} \int_x^\infty e^{-2[(x+l)/(l+l_1)]^2} dx \tag{2}$$

$$= \sqrt{\frac{2}{\pi}} \frac{S_{max}}{l+l_1} \int_x^\infty e^{-2[(x+l)/(l+l_1)]^2} dx$$

where S_{max} = maximum possible subsidence for critical or supercritical panel width;

V_{max} = maximum possible subsidence velocity for critical or supercritical panel width (Figure 2);

v = average daily face advance rate;

l = distance between V_{max} and the longwall face;

l_1 = distance between the longwall face and the location where the subsidence can be detected for the first time;

S_y = subsidence on the transverse direction assuming the dimension of the longitudinal direction has reached or exceeded the critical size (Figure 3),

$$S_y = \frac{S_{max}}{R} \int_{y1}^{y2} e^{-\pi[y'/R]^2} dy'$$ (3)

R = radius of major influence.

By differentiating equation (1) once and twice, the expressions for dynamic surface slope and curvature, resulting from the subsidence process can be obtained, respectively.

Computer Program DYNSUB: Based on the mathematical model, a computer program called DYNSUB has been written for predicting the dynamic subsidence over an active longwall face. The program can be used on IBM personal computers and PC compatibles. It is menu-driven and user-friendly.

Case Studies
Some case studies have been selected to validate the computer model. One of them is shown in Figure 4. A good agreement is seen between the predicted subsidences and the field measurements.

Summary
A model has been developed to predict the dynamic surface subsidence over an active longwall panel. The model assumes that the subsidence velocity is a normal probability distribution in relation to the advancing longwall face. It can predict subsidence, slope, and curvature at any point in the subsidence trough created by longwall mining at any time. Some case studies are used to demonstrate the validity of the model.
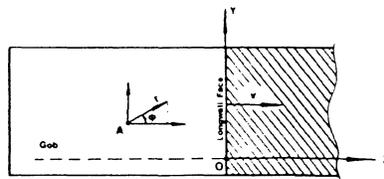
Figure 1. Coordinate system for dynamic subsidence prediction

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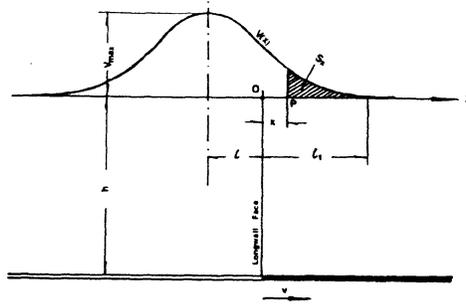
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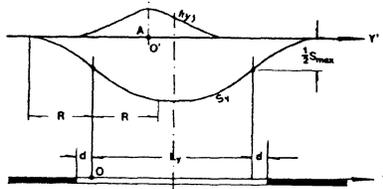Figure 2. Subsidence velocity distribution about an advancing longwall face.

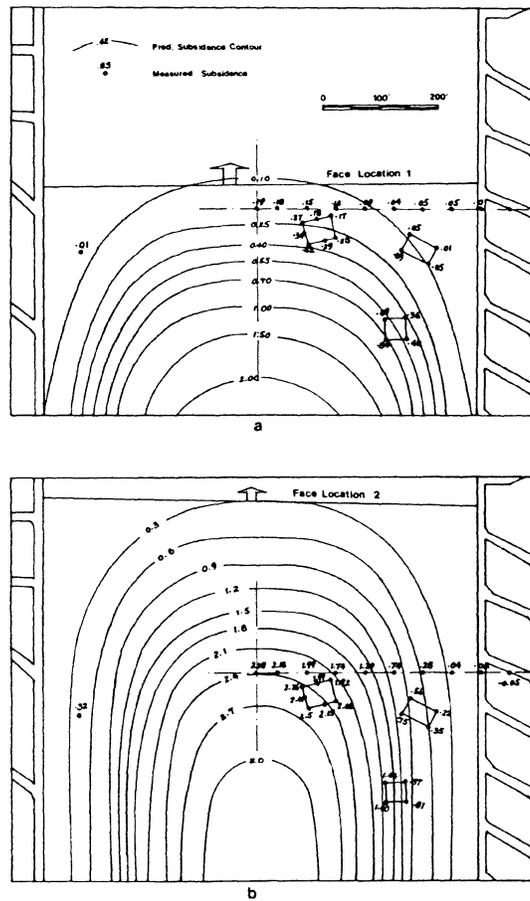
Figure 3. S_y distribution on transverse cross-section.

Figure 4. Predicted vs. measured subsidence over a longwall panel:
(a) longwall face at location 1; (b) face advanced to location 2.

APPLICATION OF AN INTEGRATED METHOD FOR MONITORING THE BEHAVIOR OF BURIED PIPELINES

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Buried pipelines are subjected to loadings due to changes in operating temperature and pressure, and geotechnical loads imposed by ground movement. This ground movement may be the result of freeze/thaw action or an unstable soil region through which the pipeline is routed. The pressure and temperature loadings can be modelled with reasonable accuracy, while the pipeline movement due to geotechnical forces cannot. In order to provide the most complete description possible of actual deformation behavior due to the geotechnical loadings, a method of integrating the measured deformations on buried pipelines with a structural finite element model of the pipeline has been developed. Measured deformations include displacements from a combined GPS-conventional survey network observed in two epochs, and measured strains from electronic strain gauges. Results and conclusions from the application of this method to an operating buried pipeline are presented.

Application

The monitoring of buried pipelines in geotechnically unstable areas poses some unique problems with respect to monumentation, measurement, and logistics for remote sites. Burial depths of the pipeline can range from 0.5 m to 3.0 m requiring excavation by heavy equipment to expose the pipeline, and specially designed monuments for attachment to the operating pipeline. The transmission of oil or gas at high pressures and controlled temperatures requires great care during installation of these monuments. These pipeline monuments are positioned at overbends and sidebends, according to the as-built and design plans, in order to detect the movement at these critical locations.

The design of the surrounding primary network must be configured to accommodate the measurement technique to be used. If conventional measurement techniques are used, a suitable number of primary network points must be established external to the pipeline, in stable areas, in order to achieve acceptable geometrical strength for position movement detection of the pipeline monuments. Intervisibility is also a critical design consideration. For GPS measurement techniques, a reduced number of primary points is required due to the observation of coordinate differences. Design criteria for GPS does not require intervisibility, but does require an unobstructed view of the horizon, or at least 15° above the horizon. Measurement of tilt may also be observed on the pipeline monuments about all three axes. Measurements of strain at critical locations may be used as verification, or to supplement the displacement measurements. This measurement information of displacement and strain is then coupled with the structural finite element model (FEM) in an integrated least-squares adjustment approach [Teskey,

1986] to provide a more detailed estimate of the structural response of the pipeline to these movements.

Results and Conclusions

The application of the integrated analysis approach to buried pipelines provided some interesting results, both expected and enlightening. (Details regarding the design of an integrated analysis system for application at a particular site are given in Price et al. [1988] and Wong et al. [1988].) It was found that the incorporation of strain measurements into the integrated analysis was of no benefit as indicated by the reliability statistics of the adjustment. This was of no surprise since strains are a secondary measurement, localized, and are not capable of accurately determining deformation behavior. Measurements of displacement are, on the other hand, a primary measurement for the determination of overall deformation behavior of a structure. Strains can be accurately determined from displacements, but displacements cannot be accurately determined from strains. The displacement measurements were comprised of translation (from the GPS survey), and rotations (from the precise conventional survey). The rotations contributed very little to the adjustment due to large observation variances and will most likely be shelved in future observation epochs. The GPS measurement technique was verified with precise conventional methods, and agreement was obtained at the 10-15 mm level proving the GPS survey suitable for this application. The integrated analysis provides, in addition to the deformation behavior analysis, reliability estimates for both the structural model and the measurements. This is a valuable tool of use in data acquisition and optimization strategies.

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ANALYSIS OF LEVEL SURVEY DATA OVER MINE SUBSIDENCE EVENTS IN ILLINOIS, U.S.A.— ABANDONED AND ACTIVE MINES

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Statement of the Problem

The U.S. Bureau of Mines and the Illinois Mine Subsidence Insurance Fund have cooperated since the early 1980s in instrumenting, monitoring, and analysing coal mine subsidence-induced ground movements in Illinois and the response of structural foundations to these ground movements. The coal mining industry in Illinois and the Illinois Mine Subsidence Insurance Fund are both responsible for repairing structures that have been damaged by mine subsidence. The former is responsible for structures over active mines and over mines in which the company did not own a subsidence waiver in the past. The Mine Subsidence Insurance Fund has over 300,000 policy holders in the State for which it is responsible for damage to structures due to abandoned coal mines. The major problems that need to be solved are:

(1) analyse surface movements as they correspond to failures in the underground operation,
(2) define the amount of ground movement that will cause the foundation of a structure to fail,
(3) develop cost-effective means of measuring ground and structural movement,
(4) define the size of the disturbed surface or the subsidence basin, and
(5) determine the time period in which damaging ground movements may occur.

Presented are research results related to these problems including an analysis of ground and structural movement data from three case studies, two mine subsidence events over abandoned operations, and one study of ground movements over a high-extraction operation and the structural response to the ground movements.

Methods of Dealing with the Problem

Level survey methodologies have proven to be the most cost-effective means of determining the size, amount, and duration of ground movements in the subsidence basin. The U.S. Department of Commerce has defined second-order class I leveling to be the standard of accuracy and general specifications for vertical control for local crustal movement in subsidence investigations. In Illinois, these standards are exceeded, with maximum closures on all survey loops maintained at 0.005 foot. Because many of the level survey nets are located in urban areas, innovative vandalproof benchmarks have been developed to maintain long-term accuracy and continuation of data.

The first case is a summation of ground movements in a mine subsidence event that began in February 1981. The event damaged 28 homes with a total property loss of approximately $750,000. Damage producing movements are still occurring in portions of the sag, while other areas of the sag show no measurable downward displacement. As a result of this event and the resulting structural damage, the Illinois Mine Subsidence Insurance Fund established the following standards for measurements:

(1) The outer edge of the subsidence basin has been established as vertical displacement of greater than 0.005 foot. The zones of influence from the ground movement are the fringe tension zone, which lies outside the maximum tension zone with the outer limits established as 0.005 foot of ground movement, the maximum slope zone, and the compression zone. Structural response and damage in these areas of ground movement differ considerably

(2) It has been established that portions of this 7-year-old active event stopped moving sooner than other areas and the center of maximum movement shifted from the original position to areas in the maximum slope zone and in the fringe tension zone. These cells of maximum movement are thought to be associated with continuing partial void closure at depth.

The second case is an analysis of a continuing, rapidly expanding series of adjacent mine subsidence events that occurred in southwestern Illinois. Some of the original benchmarks were in place prior to any mine subsidence occurring on the ground surface. Thus, total ground movements over an abandoned mine may be calculated. The succession of three dramatic events adjacent to each other have caused severe distress to occur to 30 structures. Downward displacement in the second event was quite dramatic with 1.1 feet of downward displacement occurring in a period of 48 hours. The mechanics of void failure in this event is unknown at this time, but illustrates that successive mine subsidence events may occur over abandoned operations because of uneven distributions of overburden pressure upon the pillars and floor of the mine.

The third case is a description of a monitoring program in which two foundations were built, instrumented, and monitored over a high-extraction retreat mine in southern Illinois. The objective of this monitoring program was to study the interaction between the ground surface and a foundation during a subsidence event to enhance the understanding of the mechanisms that produce subsidence effects in structures. This study measured the response of the foundations to subsidence, evaluated the capability of the instrumentation and monitoring program to detect and track foundation and ground movements, and provided data for developing mitigative techniques to reduce foundation damage and formulate economic repair strategies.

Conclusions and Recommendations

Monitoring programs in Illinois are ongoing and have reached the following conclusions:

(1) Minor differential movement of only 0.06 foot plus the horizontal movement on a foundation footing will cause failure. Thus, structures outside the maximum tension zone may be damaged because of mine subsidence events.

(2) Ground movements described in the first example are still ongoing after 7 years of monitoring.

(3)   The measurement of the horizontal component of ground movement is important and is the area of investigation in which the least amount of data exists.

(4)   Over abandoned mines in the mid-continent field, it appears that the area with the highest probability for future mine subsidence events is adjacent to those areas of the mine in which void failure has already occurred. Thus multiple adjacent events appear to be a common occurrence.

(5)   More monitoring is needed to determine the area of the draw zone over high-extraction operations.

DETERMINATION OF ATOMIC REACTOR VESSEL TILT

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Trouble-free and safe operation of the atomic power station supposes, except for other topics, verticality of the atomic reactor vessel, with respect to the flange horizontality, on which the vessel is suspended. Maximum height difference of two opposite points of a flange must not exceed the value of 1 mm. From this strong condition results the requirement to frequently check the horizontality of the reactor vessel/flange during the whole existence of the atomic power station to prevent a crash, loss of human life, and devastation of nature.

We have chosen three independent methods: hydrostatic and geometric levelling, and a method of a suspended plumb to observe reactor verticality (flange horizontality). Hydrostatic levelling was performed fully automatically by a new instrument—the 'NIVELOMAT'. The Nivelomat was designed by the Institute of Measuring and Measuring Techniques of the Electrophysical Research Center (UMMT, SAV) in Bratislava.

Sensors were situated on four opposite points in walls of the atomic reactor vessel room. Measured differences of height difference were related to the reactor flange. Mean-square error of height difference was 0.014 mm.

Geometric levelling was performed by levelling instrument with compensator 'CONI' using especially exact levelling. Observed points were identical with points of the hydrostatic levelling. In spite of lower accuracy, characterized by mean-square error of 0.14 mm, it was chosen as an independent method, which had to grant results corresponding to results of hydrostatic levelling.

The method of a suspended plumb was realized by a new instrument—the 'PENDAMETER,'—which was designed again in UMMT SAV. Plumbs were suspended on two points situated on the protective reactor cover. There was one-half of the arched distance between points.

Immediate position of the plumb suspension was sensored and performed in horizontal Cartesian coordinate differences to the reactor position before working as a base. By this means its tilt was determined with accuracy characterized by mean-square error 0.05 mm.

During this observed period of 8 months it was determined by hydrostatic levelling that the maximum height difference change in one (A-B) direction; it means that the flange was tilted 0.12 mm.

Height difference change in the vertical (*C-D*) direction was 1.23 mm, which means that the flange was tilted 0.17 mm. Geometric levelling determined values in the *A-B* direction of 0.01 mm, and in the *C-D* direction of 0.07 mm. It is not significant with regard to mean-square error.

It was determined by pendameters that the maximum tilt value in the *A-B* direction was 1.04 mm and in the *C-D* direction, 0.88 mm.

The discrepancy between tilt determined by hydrostatic (geometric) levelling and pendameters can be explained in the following: points, the positions of which were observed by hydrostatic (geometric) levelling, were stabilized in the wall of the atomic reactor vessel room, and points on which pendameters were suspended (plumb hinge and sensors) were directly on the protective cover of the reactor vessel. After beginning the reactor activity, temperature change caused the cover to deform; in the meantime walls of the atomic reactor vessel room (they were not exposed such temperature changes) did not change their position. We can come to the conclusion that measuring instruments should be placed (Nivelomat sensors, pendameter) directly on the reactor vessel during the atomic power station construction. It is not possible to do it without distortion after completion of the safety zone.

Coming out from the construction solve of the whole block, we can state that during the observed period the reactor vessel was not tilted, safety of the atomic power station was not endangered.

STRAIN DISTRIBUTION AROUND TUNNELS DETERMINED BY DISPLACEMENT MEASUREMENTS

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Field measurements are commonly performed to monitor the stability of tunnels during and after construction. Although various types of field instruments are presently available, displacement measurements such as convergence and multi-rod borehole extensometers are preferable in particular because they are easy to take and provide reliable results. Direct stress measurements, on the other hand, are often questionable and less reliable. Nevertheless, more attention is generally paid to stress, because of the fact that the failure criterion of soil and rocks is commonly given in terms of stress. Therefore, even in measuring displacement, stress is derived from the measured displacements by introducing the stress-strain relationship. It is obvious, however, that it is extremely difficult to evaluate the stress-strain relationship of in situ rock masses properly.

Considering the difficulty involved with monitoring in terms of stress, Sakurai [1981] proposed the 'Direct Strain Evaluation Technique (DSET),' which is applied to monitor tunnel stability in terms of strain rather than stress. The method is based on a direct comparison of the strain occurring in the ground around a tunnel with the allowable strain of soil and rocks. If the occurring strain is greater than the allowable strain, the stability is questionable, and additional support structures such as shotcrete, rock bolts, and steel ribs must be installed to stabilize the ground. The advantages of this method are not only the simplicity and reliability of the measurements of occurring strain, but also the easy determination of the allowable strain of soil and rocks. The allowable strain of in situ rock masses seems to be much easier to evaluate than the allowable stress, because the allowable strain is not influenced much by the existence of joints.

It should be emphasized, however, that the monitoring method in terms of strain is only usable for ground which can be represented by a continuous or pseudo-continuous model, that is, for either soils or highly fractured rocks. When the ground consists of discontinuous jointed rocks and their joint space is almost in the same order of magnitude as the tunnel diameter, strain is unlikely to be definable, and each joint must be taken into consideration individually.

When the number of displacement measuring points is sufficiently large, strain can be determined directly from measured displacements using only the kinematic relationship. In this case, no information is needed with respect to the initial stress and material constants of the soil and rocks. In practice, however, the number of extensometers being installed around the tunnels is generally limited, and not sufficient enough to obtain an overall view of the strain distributions.

In order to overcome this difficulty, Sakurai and Takeuchi [1983] proposed another method which is based on 'back analysis,' a reverse calculation of ordinary stress analysis. According to this method, the initial stress and material constants are first back calculated from measured displacements and then are used as input data for an ordinary analysis by the finite element method or boundary element method, to determine the strain distribution around the tunnels. The mathematical formulation of the proposed method is based on the linear theory of elasticity so that only Young's modulus and Poisson's ratio are the material constants which should be back analysed. If, therefore, the rock and soil around the tunnel behave as non-elastic materials, the back-analysed Young's modulus and Poisson's ratio are not real, but equivalent values for the materials.

This paper presents a case study where the proposed back-analysis method has been successfully used. In this study, four adjacent tunnels are constructed at a shallow depth; two upper and two lower, parallel to each other. A plane view of the tunnel construction site and cross section of the tunnels are shown in Figure 1. An intensive field measurement system is carefully planned and executed. Displacement measurements are highlighted in this measurement system, in which sliding micrometers, TRIVEC, multi-rod extensometers, and convergence meters are used. One of the measurement sections is shown in Figure 2.

The tunnel excavation is carried out step-by-step starting from the lower tunnels, thus requiring that the back analysis be performed considering this excavation sequence. Special care has been made, therefore, to formulate a back analysis capable of analysing the displacements which occur as each tunnel is excavated. The systematic measurements and back analyses were conducted throughout the excavation. One of the back analysis results is shown in Figure 3, compared with the measured displacements used as input data for the back analysis. The strain displacement around the tunnels is then calculated. The maximum shear strain distribution at the final stage of excavation is illustrated in Figure 4. Considering this strain distribution, the stability of the tunnels was monitored and the construction was completed successfully.

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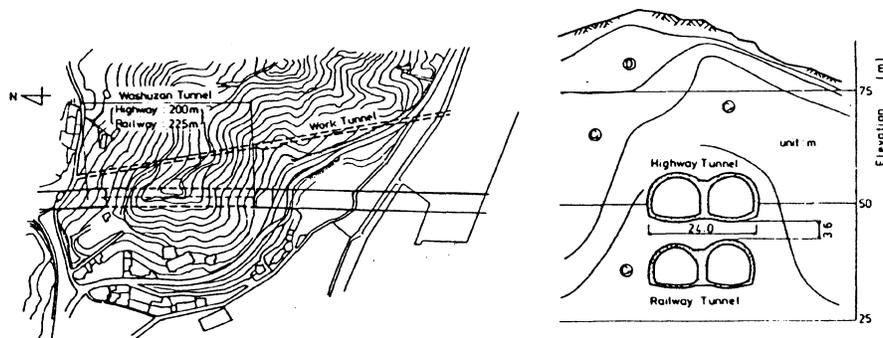
Figure 1. Plane view of tunnel and its cross-section.

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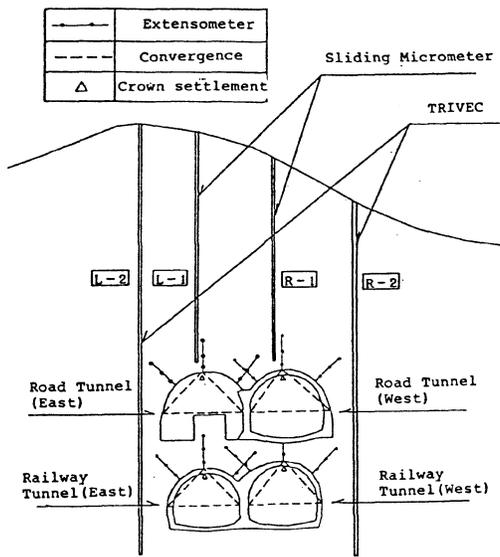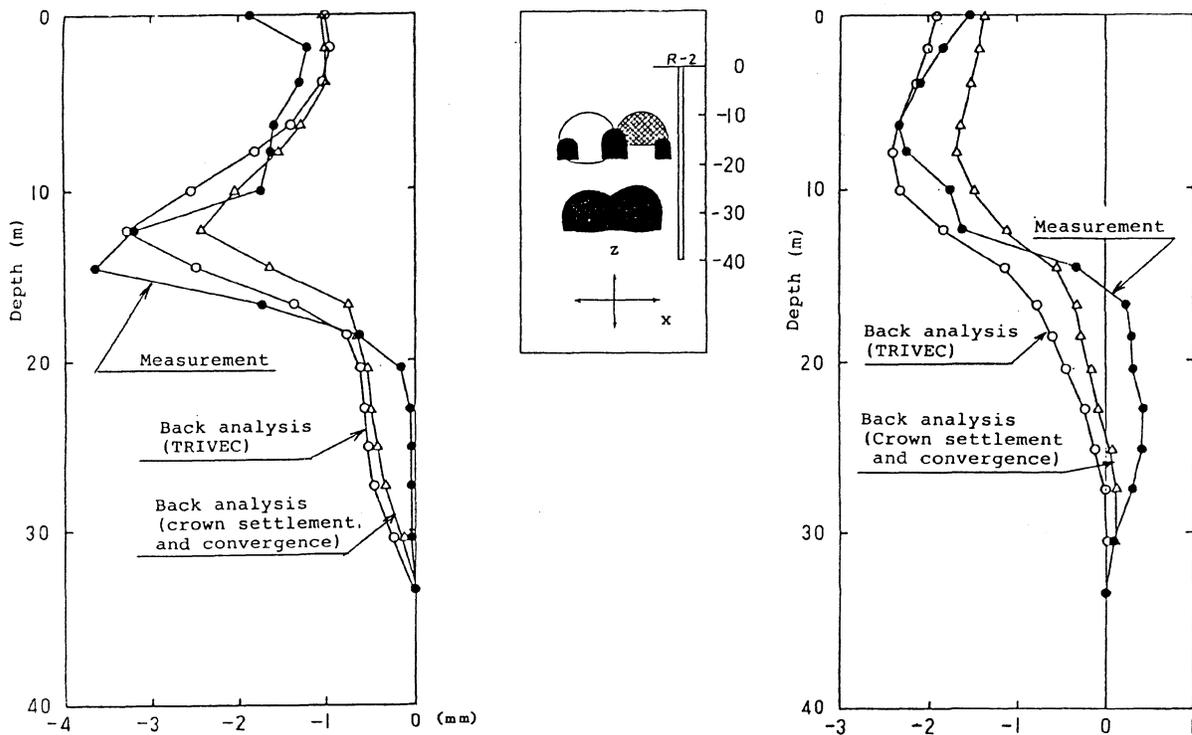
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Figure 2. Displacements obtained by measurements
and back-analysis.

Figure 3. Displacement distribution obtained from measurements and back-analysis.
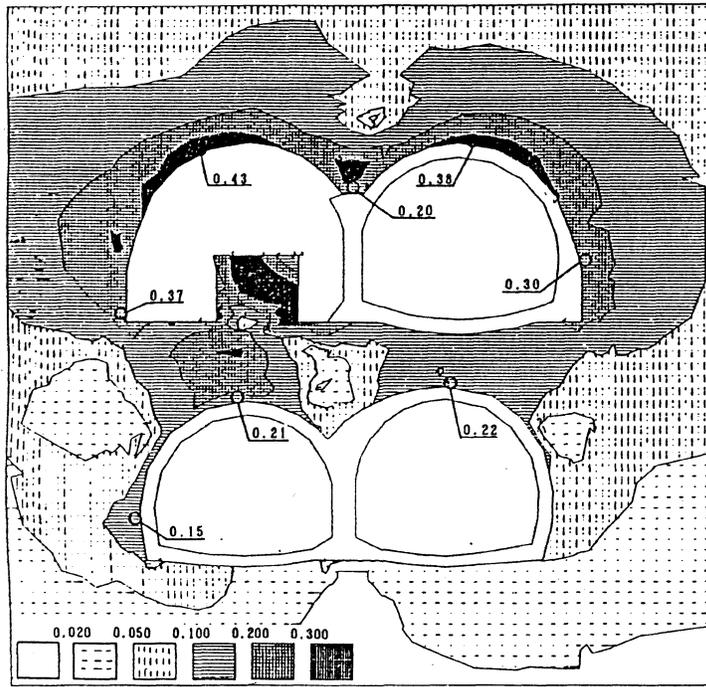
Figure 4. Maximum shear strain distribution
obtained by back-analysis.

PREDICTION OF SUBSIDENCE MOVEMENTS IN COAL MINING AREAS IN INDIAN COAL FIELDS

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A sizeable percentage of a total of about 158.46 billion tonne of coal reserves in the Indian coal fields lies underneath and in the vicinity of surface, sub-surface, and underground properties. On a rough estimate, the quantity of coal locked up underneath railway properties alone may be more than 600 million tonne.

Underground extraction of coal seams tends to damage the surface, sub-surface, and underground properties due to subsidence movements. Therefore, the extraction of coal seams under such conditions presupposes knowledge of pre-estimation/prediction of subsidence movements which can be obtained after extensive subsidence investigations in coal mining areas.

In coal mining areas, subsidence movements can also take place due to other natural and man-made causes, such as solution of rocks and minerals, drainage of sub-soil, frosting and defrosting, tectonic movements, pumping of water from underground, etc.

In order to develop the know-how of subsidence behaviour of coal measures in Indian coal fields, systematic subsidence investigations were started by the Central Mining Research Station (CMRS), Dhanbad, in 1964, and up to December 1987 the investigations has been completed over 34 caved and 21 hydraulically sand stowed workings/panels and were in progress over 45 caved and 42 stowed panels. The investigations have shown that:
- In undisturbed conditions, the non-effective width, the width which can be extracted without causing practically any subsidence, varied between 0.3 and 1.17 times the depth of the workings and depended upon the percentage of sandstone in the rock mass overlying coal seams.
- Over caved workings, the maximum subsidence was generally not more than 60% of the extraction thickness. Figure 1 shows a plot between width-depth ratio and maximum subsidence over caved workings.
- Over hydraulically sand stowed workings, the maximum was generally not more than 5% of extraction thickness.
- The subsidence profiles in the coal fields were in general asymmetrical in shape and were continuous as well as discontinuous. This was mainly due to extraction of the coal seams at shallow to medium depths and the brittle nature of the rock mass overlying the coal seams.
- About 70% to 90% of subsidence over the workings had taken place almost immediately after completion of underground extraction and the remaining subsidence continued up to about 200 to 1000 days afterwards.
- It was earlier believed that the angle of draw in Indian coal fields is negative, that is, the area influenced on the surface was less than that extracted underground. The

investigations have shown that the area influenced on the surface was always more than that extracted underground and hence the angle of draw is positive. In undisturbed conditions, its maximum value was between 20 and 25 degrees.

• The volume of subsidence trough over 7 caved workings varied between 3.3% and 40% of that extracted underground.
• Relationships for anticipation of maximum slope and strains for different width-depth ratios are shown in Figure 2.

Based on the experience of extraction of coal seams underneath and in the vicinity of surface properties, the following safe limits of subsidence movements have been defined for different categories of the properties:

• Railway lines of jointed construction can be subjected to a maximum strain of 3 mm/m and its long gradient should not be steeper than 1 in 100.
• In water bodies, the maximum permissible tensile strain is 4.5 mm/m.
• Buildings can be subjected to a total maximum compression or elongation of 60 mm which can be expected to cause slight repairable damage.
• In the case of high tension pylons and aerial ropeway trestles, the maximum permissible strain is 3 mm/m and maximum displacement of the top-most point due to slope should not exceed one-third of the radius of the base.

The above results of the subsidence investigations have been used to study more than 90 problems involving extraction of coal seams underneath and in the vicinity of surface properties and protection thereof. As a result, it has been possible to help the coal mining industry to extract more than 7 million tonne of coal from underneath and in the vicinity of surface properties and many more million tonne may be extracted in the near future. A main railway line has been gradually made to subside by a maximum of 631.6 mm in a span of over 17 years without any disturbance in its operation.
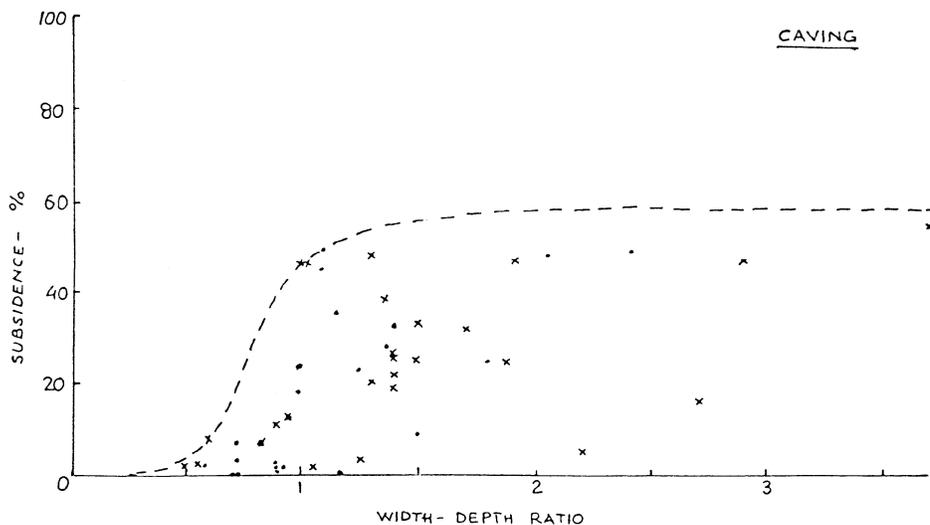
Figure 1. Relationship between width-depth ratio and maximum subsidence over caved panels
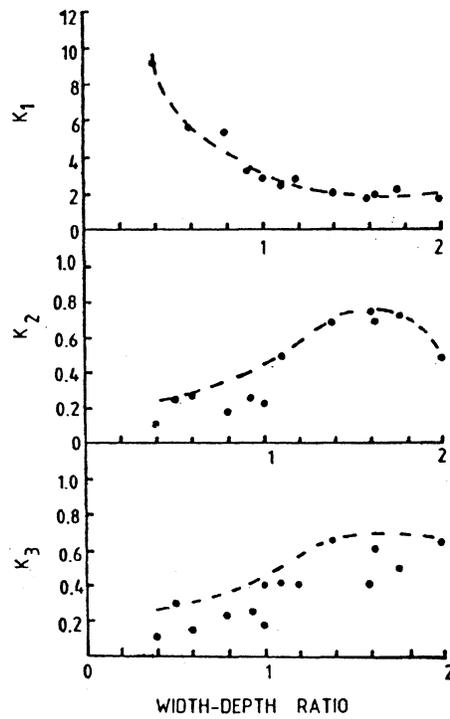
$$G = K_1 \frac{S}{h}$$

$$E_{(-)} = K_2 \, G$$

$$E'_{(+)} = K_3 \, G$$

Where
G = Max. slope in mm/m
S = Max. subsidence in mm
h = Avg. depth in m
$E_{(-)}$ = Max. compressive strain in mm/m

$E_{(+)}$ = Max. tensile strain in mm/m

K_1, K_2 & K_3 = Constants

Figure 2. Relationship between k_1, k_2, and k_3 and width-depth ratio.

STABILISATION OF UNAPPROACHABLE OLD UNDERGROUND WORKINGS IN INDIAN COAL FIELDS—PILOT TRIALS

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Coal mining in India has a history of more than 200 years. The old mining practices have left a legacy of unapproachable underground workings which are generally partially or fully waterlogged. With the increase in population and industrial development in coal mining areas, surface properties were made, many of them over the unapproachable and unknown underground workings. The coal pillars, stooks, chowkidars, fenders, barriers, etc., left underground in the unknown workings became weak with the passage of time and resulted in collapses of a large number of such workings damaging surface properties. At many locations, more prominently in Jharia and Ranigunj coal fields, collapses and thereby damage to surface properties is imminent, demanding urgent attention.

The unknown and unapproachable waterlogged workings are not only a threat to the surface properties but are in the way of effective exploitation of underlying seams.

Potentially dangerous areas in Jharia and Ranigunj coal fields have been identified and the concerned departments are contemplating suitable remedial measures to make such areas reasonably safe. It was found that the best measure would be filling underground cavities with sand or some other suitable material. Conventional methods and systems used elsewhere, like normal stowing, borehole stowing (open gravity feed system), pumped slurry backfilling, and a point support system, were not found suitable for conditions in Indian coal fields. Therefore it was necessary to study and develop system(s) suitable for Indian conditions.

Considering the prime requirement of an effective backfilling system that the cone of solids under the borehole should not be allowed to form and a dynamic suspension of solids is maintained for their transportation in underground workings, use of compressed air and water jet was contemplated in conjunction with an open gravity feed system. In order to develop the systems called 'hydro-pneumatic' and 'hydro-hydro-jet' laboratory scale model studies were conducted at the Central Mining Research Station (CMRS), Dhanbad.

A leak-proof model depicting bord-and-pillar workings, of 800 mm x 1050 mm size with opening size of 20 mm x 50 mm was made with a total void volume of 9000 cubic centimetres and with provision of changing the gradient, the feed position and exit

position, and monitoring the spread of solids, air quantity and pressure, and water quantity was used to study the two systems in the laboratory.

Encouraging results were obtained in the model studies. It was possible to backfill about 75% to 95% of the voids. The flow of solids can be regulated by regulating the direction of travel of compressed air. With the 'hydro-pneumatic' system, the flow of solids was predominantly on the rise side compared to all around the borehole in the 'hydro-hydro-jet' system. The systems were equally well adaptable to seam gradients up to 30°. These results led to the first field trial of the 'hydro-pneumatic' system at Jogta Fire Project of Bharat Coking Coal Limited in Jharia coal field for backfilling of unapproachable workings in XIV seam underneath a jore bed.

The workings in XIV seam underneath a jore bed at Jogta were supposed to be dry with the height of stooks estimated as 6.9 m at depths ranging from 35 m to 45 m. The colliery had planned to put about 90 boreholes at about 3 m intervals for backfilling the area by the 'open gravity feed system.' With the hydro-pneumatic system, the work of backfilling about 9000 cubic metres of solids (sand, crushed stone, washery reject, and murram) could be finished from two injection boreholes only. A schematic diagram of the system is shown in Figure 1. In this trial it was noticed that the solids had travelled to about 36 m distance towards the rise side of the first injection borehole.

The second trial of the 'hydro-pneumatic' system was conducted in an area in a 2.9 m thick BD seam at depths ranging from 17.5 m to 41.3 m. The area developed on the bord-and-pillar system was isolated by erecting 6 dams underground for the purpose of the experiment. The size of the area in the seam dipping at 1 in 7 was about 100 m x 108 m and the total void volume was about 25 000 cubic metres. In the trial in the area at Ramjeebanpur colliery of Eastern Coalfields Limited in Ranigunj coal field, it was possible to inject/backfill about 28 000 tonne (16 500 cubic metres) of solids (sand) from one injection borehole, which was about 66% of the total void volume. Figure 2 shows the mixing and feeding arrangement.

The two field trials and laboratory experiments have shown promising results based on which preparations are being made to study the systems in a large-scale model having roadway section as 1000 mm x 1000 mm for further studies and development of the systems. More field trials are also contemplated.
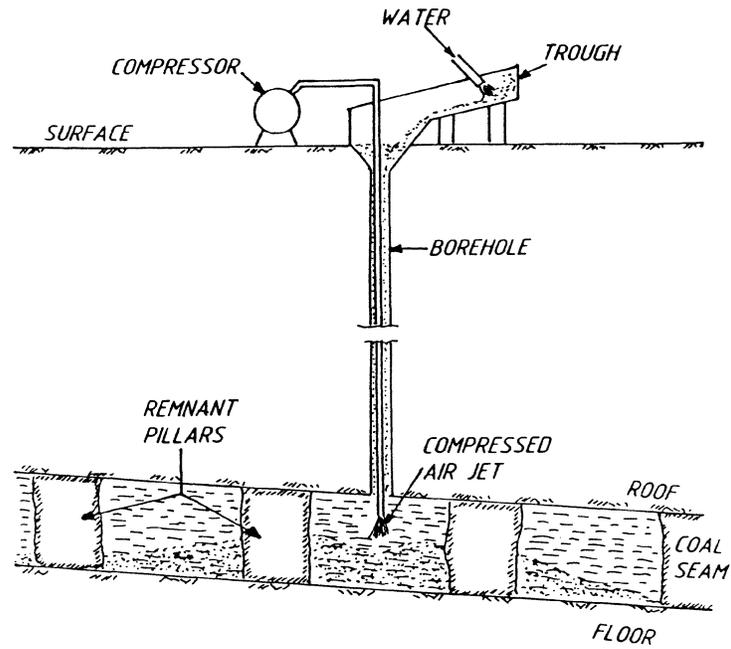
Figure 1. Schematic diagram of hydro-pneumatic system.

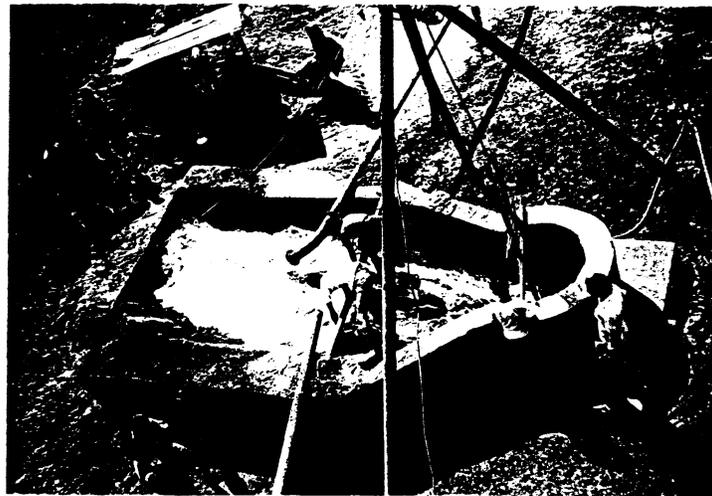Figure 2. Mixing and feeding arrangements at Ramjeebanpur colliery.

EXTRACTION OF THREE THICK SEAMS UNDERNEATH A MAIN RAILWAY LINE AND UNAPPROACHABLE OLD WORKINGS AT SUDAMDIH IN JHARIA COAL FIELD

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Sudamdih shaft and incline mines of Bharat Coking Coal Limited in Jharia coal field lie on the southern flank of the coal field. Almost all the coal seams existing in the coal field outcrop in the property of the two mines and the seams dip from about 25° to 55°. In the beginning, the coal mining in the area was done by opencast system along the outcrops of the seams and later by underground bord-and-pillar system from the opencast mines or from the surface during a period from about 1910 to 1940. Due to difficult working conditions, the mines were abandoned and became waterlogged with the passage of time.

In order to exploit vast coking coal reserves available in the area, Sudamdih shaft and incline mines were opened on the basis of planning done by Polish consultants. After the initial development, depillaring was started in XI/XII seam at the incline mine in 1969 and later extraction in VIIIA, IX/X, and XI/XII seams was done at the shaft mine. The two mines were separated by a vertical barrier of 50 m. The shaft mine workings are below 200 m depth, while the workings of the include mine are at 150 m depth. The extraction of these seams was planned with hydraulic sand stowing due to

- availability of a large number of thick to very thick seams in close proximity in the area,
- existence of waterlogged old workings in a large number of coal seams under which the extraction was planned,
- presence of surface features and structures like the Damodar river, Adra-Gomoh main railway line of South Eastern Railway, an assisted railway siding, a washery, villages, buildings, etc.

In the 7.5 m thick XI/XII seam, extraction was done by ascending slicing system with hydraulic sand stowing at depths ranging from 35 m to 135 m at Sudamdih incline mine. At the shaft mine, the seam was similarly extracted at depths ranging from 200 m to 400 m. The seam was extracted in two slices.

The 20 m thick IX/X seam was extracted at Sudamdih shaft mine between 200 m and 300 m deep. Initially the extraction in the seam was started by horizontal slicing system in ascending order with hydraulic sand stowing, and after extraction of eight slices of about 3 m thickness each the method was abandoned due to difficulties faced in roof control. Thereafter extraction in the seam was done by ascending slicing and a

maximum of 3 slices of 3 m thickness each were extracted. In general, the extraction thickness was about 6 m.

The 3 m to 5.5 m thick VIIIA seam was extracted in one and two ascending slices with hydraulic sand stowing according to the thickness at depths varying from 200 m to 400 m at the shaft mine.

Before starting the extraction of XI/XII seam underneath and in the vicinity of the main railway line of the South Eastern Railway in 1971 from the incline mine, feasibility of extraction of the seam was studied by the Central Mining Research Station (CMRS), Dhanbad, on the basis of subsidence studies carried out over two panels at the mine. The following safe limits of subsidence movements were taken into consideration:
- maximum strain   = 3 mm/m
- limiting operating gradient = 1 in 100.

The limiting operating gradient of the railway line was defined by the Chief Mining Advisor to the Railway Board.

In a span of about 17 years, from 1971 to 1987, more than 3 million tonne of coking coal has been extracted from the three seams from underneath and in the vicinity of the main railway line and other surface properties and waterlogged old workings in XIV, XIVA, XV, and XVI seams. As a result of the extraction of the three coal seams (Figure 1), the main railway line has been gradually made to subside by a maximum of 631.6 mm in December 1987 without any disturbance in its operation. The maximum slope due to subsidence along the railway line was 7.5 mm/m. The railway line has been subjected to a maximum strain of 2.9 mm/m. Figure 2 shows the subsidence profile observed along the railway line.

In some parts, the railway line was laid initially at a gradient of 1 in 105 and after the subsidence in December 1987 the gradient in the corresponding part was 1 in 104. During the period of investigation, the steepest gradient in this length was 1 in 102.4.

The experience gained at Sudamdih is being used for extraction of coal seams underneath railway lines/sidings at other collieries in Indian coal fields. At Surakachhar colliery of South Eastern Coalfields Limited in the Korba coal field, a 3 m thickness in the G-III seam has been successfully extracted by longwall system with hydraulic sand stowing underneath an assisted railway siding. At Salanpur and Jamodoba collieries in Jharia coal field, coal seams are being extracted with hydraulic sand stowing underneath railway lines.
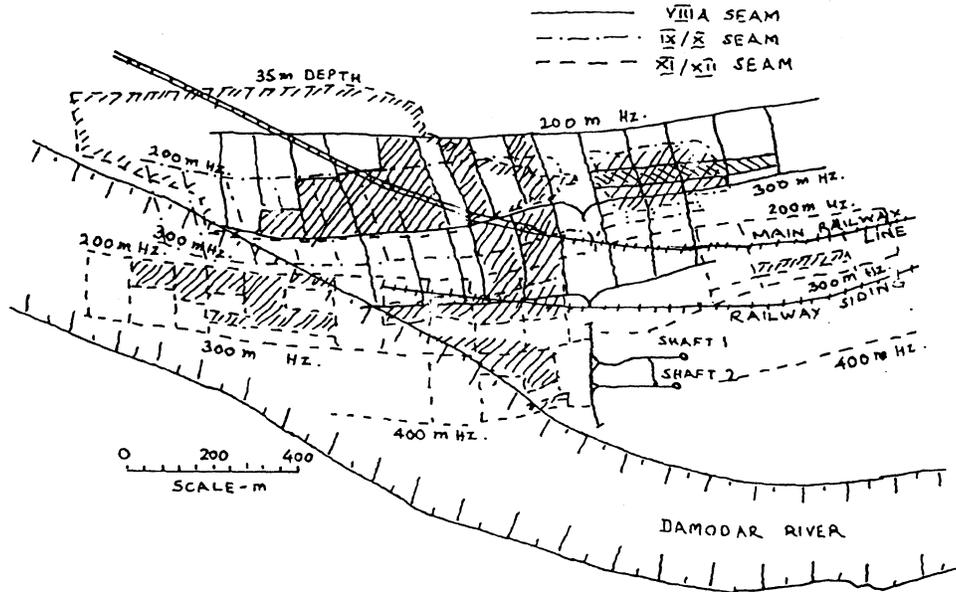
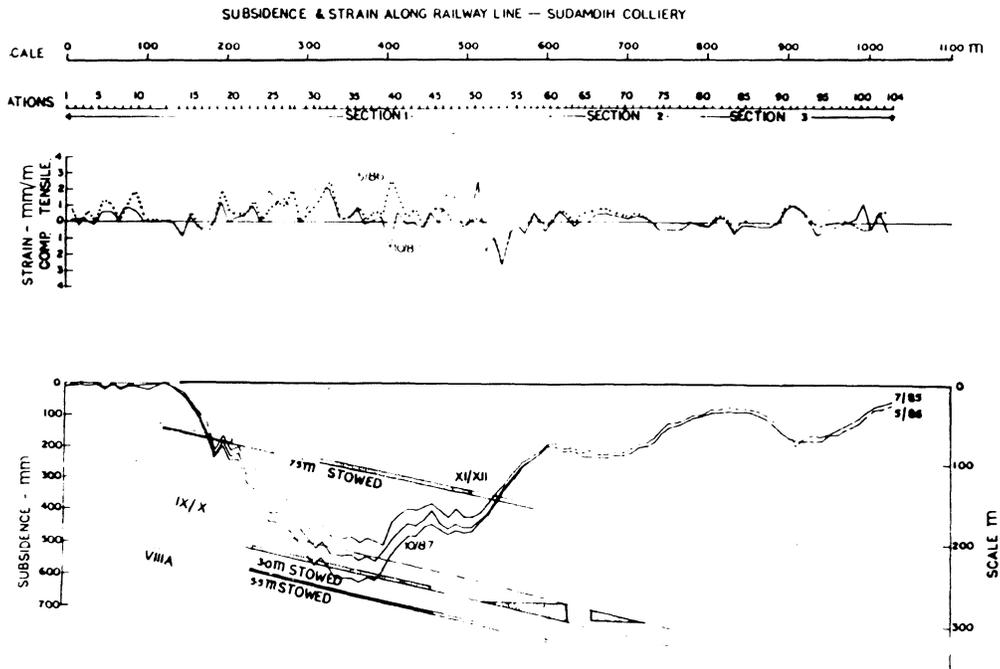Figure 1. Plan showing areas extracted in VIIIA, IX/X, and XI/XII seams.

Figure 2. Subsidence and strain profiles along the railway line.

SUBSIDENCE RELATED EARTH MOVEMENTS IN THE TIA JUANA FIELD, STATE OF ZULIA, VENEZUELA

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Subsidence due to oil exploitation in the Bolivar Coast Fields in western Venezuela has been monitored since 1926. Related horizontal earth movements have been measured by geodetic methods during the last 10 years in the northern Tia Juana Field area in and around the Ulé tank farm and the GLP gas liquefaction plant. Due to damage to storage tanks and the increasing appearance of fractures in the ground in the vicinity of the coastal dyke protection system, a detailed study was undertaken by INTEVEP.

A surface geological survey of the Ulé area, based initially on aerophotographic interpretation of infrared photographs, flown at a 1:5000 scale, showed the area to be divided into blocks by several major faults. These faults were confirmed on the ground by visual inspection, shallow seismic refraction, geo-electric surveys, and correlation of geotechnical borings.

A geodetic survey of the area, with a precision of 5 mm + 32 mm/km for distances, showed significant horizontal displacement of ground markers during an 8-year period. Accordingly, a measuring plan of high precision (0.015 mm) electro-mechanic extensometer lines was designed for critical zones across known fault traces and fracture zones. The extensometers were installed in trenches 1 m below ground level, anchored and cemented in place and covered with earth. Periodic measurements were taken after a 6-month period of settlement; initially measurements were recorded every 2 weeks and later in 2-month intervals. The extensometers crossing the Tia Juana fault were monitored for a time in 4-hour intervals, showing the influence of lake and moon tides on these extensometer measurements.

This study proposes a conceptual model integrating the geodetic and extensometer measurements of horizontal movements in the Ulé area with the geologic and neotectonic frame of subsurface faults, joint systems, and tensional surface cracks. Rates of Holocene postglacial isostatic adjustments and natural pre-industry subsidence due to degassing of reservoirs, determined by C-14 radiocarbon dating, were used to calculate subsidence rates due to oil exploitation. It was found that subsidence occurs in the area, not as a continuous function of compaction as previously mapped, but as discrete, fault and crack limited blocks, governed as much by reactivation of fault slippage as by subsurface compaction.

Oil storage tank shell deformation measured over the same 10-year period, observed tank settlements and surface fractures in the tank farm area, were correlated with strain measurements, earth movements, theoretical settlement due to geotechnical

characteristics of foundation layers, and centrifuge experiments of scale models, in order to isolate the effect of soil deformation and cracking due to subsidence. From these data a statistical horizontal strain-subsidence model was developed for the Tia Juana-Ulé area.

SELECTED PROBLEMS IN PREDICTING INFLUENCE OF MINING-INDUCED GROUND SUBSIDENCE AND ROCK DEFORMATIONS

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For the past few years, the problem of defining a general model of the influence of underground mining exploitation on the ground surface has been a subject of intensive research at several industrial and university institutes in the countries with a significantly developed mining industry.

Within a research programme at the Institute of Mine Surveying and Mining Surface Protection of the Technical University in Clausthal, complex research is underway to describe the deformation phenomena as a function of such elements as:
- non-homogeneity and anisotropy of rock masses from the point of view of their physio-mechanical properties,
- non-linearity of the rock masses from the point of view of their geometry (tectonics),
- non-horizontality of exploitation and geological strata,
- convergence progress in the mining openings in space and time,
- delay in the activation of the overburden strata,
- influence of a multi-time mining exploitation on the parameters and behaviour of the rock masses (anisotropy in time), and
- activation of the previously excavated mining areas.

It has been assumed that the general functional relationship between the cause (convergence of the mining opening, Q) and the result (for example, the vertical displacement-subsidence, S) can be expressed in the form:

$$S = Q * \varphi \tag{1}$$

where symbol $*$ has the meaning of the juncture of the function Q and φ.

The function Q pertains to the boundary condition and describes the displacement at the source of deformation (the mining opening). The function φ pertains to the transformation function (influence function) which allows for a mathematical formulation of the causative relationship between Q and S.

From equation (1) one can see that the precision of the calculated displacements, S, depends not only on the influence function φ, but first of all on the correct description of the convergence progress Q of the mining opening in space and time. The source function Q must consider the type of the mining extraction which may, for example, be a longwall exploitation with the roof caving or with the backfill, the room and pillar

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 113

exploitation of deposits of coal, ore, or salt, or dissolving creation of salt chambers and caverns, as well as extraction of oil.

The influence function φ characterizes the reaction of the overlying rock masses. Considering the fact that the displacements are caused by a smoothed out reaction of the rock masses to the disturbed state of the equilibrium, and considering that the number of degrees of freedom of the whole system of 'rock mass' in relation to the possibilities of an accurate description of its geometry and its physio-mechanical characteristics is very significant, the behaviour of the system can be described by a statistical distribution function from which the simplest one relates to the normal distribution.

In the proposed solution, the influence function φ relates to an expanded and parameterized Gaussian function. Such a defined influence function φ allows us to consider non-linearities and anisotropy of the rock mass, the inclination of the strata, the influence of geometrical non-linearities, and also the delayed reaction of the overburden strata.

The general theoretical solution obtained has been compared with in situ measurements for different cases of exploitation of coal, ore, salt, and oil deposits. The obtained results show a large potential of the proposed solution in its applications at various stages of mining planning and production in order to design a safe mining method and, later on, to modify the mining specifications according to a comparison between the designed and actual (measured) displacements during the progressing mining extraction.

SURVEY OF VERTICAL DEFORMATIONS BY GPS AND PRECISE LEVELLING

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By virtue of their speed, convenience, and accuracy, GPS measurements seem to be extremely well suited to deformation analysis and in particular for monitoring vertical deformations of the earth's crust. The fact that accurate elevation differences between two distant points can be obtained overnight without having to build and maintain an extensive vertical network, makes monitoring deformations in seismically active areas an attractive alternative to classical spirit levelling. There is, however, a certain pitfall in using GPS as the only data source for monitoring vertical deformations: possible variations in the GPS datum may appear as components of vertical deformation. The only way to avoid misinterpretation of GPS datum variations as vertical deformations is to use GPS in combination with classical levelling. A combination of GPS and classical levelling is natural and necessary nowadays if we regard the GPS measurements as a complete or as a partial substitute for classical levelling in monitoring existing vertical control networks.

Processing the two types of data together requires the identification of a common set of parameters. In order to process GPS and spirit levelling measurements with a kinematic model of adjustment we need three groups of parameters as follows:
- orthometric heights at a standard epoch (ortho-heights);
- time derivatives of the orthometric heights (ortho-velocities);
- undulations of the geoid (undulations).

In deformation analysis by a kinematic model, the only parameters of interest are the ortho-velocities of the object points. The differences between GPS heights and orthometric heights, i.e., the undulations of the geoid, which are perceived as invariant quantities, are thus irrelevant. The undulations, however, being dependent on GPS measurements, contain a systematic component which is due to possible variations in the GPS datum and thus depend on time and on the relative positions of the points in the network. Two rotational parameters of the GPS datum are needed to model this systematic effect. Those additional parameters can be estimated from the same measurements by the technique of extended free net adjustment constraints. We make a necessary assumption that those additional parameters are invariant for the duration of one GPS measurements session which may last anywhere between a few days and up to several weeks or even months. Thinking of the probable causes of GPS datum variations, this assumption seems reasonable.

There are now four parameter groups to be estimated in our adjustment. We point out that the GPS orientation parameters are modelled as different quantities for each

session, while ortho-heights, ortho-velocities, and undulations are the same parameters for all GPS sessions.

Table 1 describes schematically the estimability of the four parameter groups as a function of the combination of data types and the number of repeated measurement sessions. As is well known, if the only measurements are GPS, there is no basis for separating ortho-heights from undulations. In the table, this is indicated by the liaison symbol (+————+). On a similar basis, it can be shown that there is no way to separate effectively the ortho-velocities from the GPS orientation unknowns. If we insist on solving the two groups ((2) and (4)) separately, we would actually get for (2) deviations of the individual point-velocities from a global behaviour of the network (a tilting plane). The number of GPS-borne elevation differences per session has to be in excess of two. This is a necessary condition for making a contribution to the estimation of ortho-velocities. At least two of the GPS elevation differences have to be non-parallel.

Table 1 Estimability of: (1) ortho-heights, (2) ortho-velocities, (3) undulations, (4) GPS orientation unknowns.							
Parameters	**A single GPS session** (1) (2) (3) (4)				**Two or more GPS sessions** (1) (2) (3) (4)			
Spirit levelling	+				+	+		
GPS levelling	+———————+				+————+————+————————+			
GPS + spirit levelling	+		+		+	+	+	+

If the only available measurements are GPS coordinate differences and spirit levelling elevation differences, the system of observation equations is defective and has to be minimally constrained in order to resolve the inherent rank defect. The defect is the size 3, same as the number of arbitrary datum parameters. In deformation analysis, we define the motion of all object points with respect to a unique and stable reference frame. This frame is based on a subset of the net points—the reference or the datum points. This is why the above rank defect is resolved by applying free net constraints to the velocities of those datum points.

Three datum bases are needed for the free net solution of groups (1), (2), and (3). The basis for the velocities (2) is trivial, i.e., the nominal (datum) velocities of the reference points are usually taken to be zeroes. The datum basis for group (1) can be any, more or less, consistent set of ortho-heights referred through a mareograph to mean sea level. The datum basis for group (3) is of no importance in deformation analysis and can be a zero set. Another choice which seems 'closer to nature' may be formed by undulations computed from a consistent set of spherical harmonic coefficients.

The above principles for combining GPS with spirit levelling were tested on a small vertical network. Measurements of both types were simulated and processed by an 'ad hoc' program written in APL. For each session of GPS measurements, different biases (rotations) were simulated in the data. The results of the adjustment of the simulated measurements confirmed the effectiveness of our method for removing the GPS orientation biases. In addition, we obtained some insight into the accuracies which could be expected.

In conclusion, we would like to stress the following points:
- GPS measurements alone are insufficient for deformation analysis of a vertical network.
- In a combination of GPS and spirit levelling, the biases in the GPS data have to be filtered out in order to have an unbiased contribution to the evaluation of vertical deformations.
- In spite of all our reservations with respect to the usefulness of GPS in deformation analysis, its clear advantages over conventional spirit levelling are such that every effort should be made, and newer and better mathematical devices should be developed, in order to make the best possible use of its potentials.
- Consistent and widespread application of our method may provide GPS analysts with an additional data source for studying the effective stability in orientation of the GPS datum.

DEVELOPMENTS IN MINE MONITORING — A WESTERN CANADIAN PERSPECTIVE

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The significant changes in mine monitoring technology are outlined based on the
experience of the Canada Centre for Mineral and Energy Technology (CANMET) in
western Canada. The changes can be divided into hardware changes,
techniques/software changes, and changes in the application of the technology.

Hardware has evolved enormously in some areas, especially in digital electronics. In
other areas, however, the advances have been less spectacular. The requirements of
monitoring are rather specific, however, the increasing power of microprocessors will
do little to further ease the development of these systems. It is other factors, such as the
increasing ease of interfacing components, the reduced number of chips required to
build systems, and the greater number of chips available with extended temperature
operating ranges, that make the difference.

The techniques used in monitoring for the transmission of information can be classed
into five generations:
(1) **Non-electrical methods,** such as tube bundle systems for gas sampling. Non-
    electrical methods get around the problem of intrinsic safety in underground
    mines, a problem that slowed the introduction of automatic systems because of the
    extra expense and limited markets.
(2) **Analogue techniques,** such as current loop techniques. Transmitting signal
    values such as voltages, currents, or field strength is not suitable for moderate or
    long distances at d.c. However, d.c. is used over short distances, e.g., over the
    length of boreholes. The effects of losses and noise are important.
(3) **Frequency coded techniques,** where signals are converted into frequencies to
    be transmitted. The a.c. signal is generated by a voltage controlled oscillator
    (VCO). These are available as a single chip. The signal is read by a frequency
    counter, another standard device. This technique has good immunity to noise and
    attenuation. Several mine monitoring systems were built on this principal
    although it is now largely displaced by digital methods. It can be thought of as
    bridging the gap between digital and analogue technologies.
(4) **Digital transmission.** All quantities are digitized for transmission. This is
    performed by a digital to analogue converter (ADC) generally a single component,

though until recently quite expensive. The digital output from the converter will generally be manipulated by a microprocessor before modulation and transmission. Generally 'error detection' or 'error correction' is used. The data is transmitted in such a manner that should a transmission error occur, it will be detected and either directly corrected or retransmitted.

(5) **Advanced digital techniques.** These allow communications strategies similar to computer local area networks and permit the use of multiple repeater stations without loss of data or extra carrier frequencies. In terms of hardware, there is little difference from (4).

For the transmission of digital data over radio links and over long cable links, it is generally modulated by the frequency shift mechanism (FSM). This has been common practice in mining systems known to the authors. Its advantages are that it is easy to implement and compatible with voice channels. This means that it can be used over radio links with normal voice licensing. It is not the fastest way to transmit data in a given channel.

Originally, monitoring systems were designed to address single problems such as mine atmosphere monitoring. Increasingly, one is dealing with communications between intelligent devices with the consequence that the communication system can be used for multiple purposes.

There are a number of mine monitoring areas that have particular characteristics from a data communication point of view. Equipment monitoring and truck dispatch tend to involve high amounts of data traffic. Some geotechnical monitoring problems have severe requirements in terms of temperature of operation and power consumption. The all-encompassing mine communication system must try to anticipate a wide variety of objectives and problems in terms of the operations and the environment found at mines. There is an increase in the use of telemetry led by operational monitoring and control systems. There is thus a need to make good use of channel time and have standards of compatibility between systems.

Some of the systems used and planned in western Canada will be described, including plans for a general purpose mine system.

CLOSURE MEASUREMENTS IN DEEP UNDERGROUND OPENINGS IN BEDDED SALT, WIPP FACILITY, NEW MEXICO, U.S.A.

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The Waste Isolation Pilot Plant (WIPP), located in southeastern New Mexico at a depth of 2150 feet in thick deposits of bedded salt, is being developed by the U.S. Department of Energy (DOE) as a research and development facility to demonstrate the safe disposal of low and medium level radioactive wastes. The design must provide for the access and storage openings to remain accessible and stable during the emplacement and retrieval period, even though these openings will eventually close due to salt creep. The facility consists of four shafts leading to a single underground facility level. The facility consists mostly of horizontal openings of rectangular cross section. The drifts were excavated 8 feet to 12 feet high and 12 feet to 25 feet wide. The four test rooms were each 13 feet high and 33 feet wide. The test rooms were designed to permit an assessment of deformation and closure rates due to salt creep.

Geomechanical Instrumentation
As part of the design validation process, a geomechanical instrumentation program was implemented to gather geotechnical data from the underground openings. Instruments for measuring the geotechnical behaviour of the shafts and drifts consisted of convergence points, convergence meters, multiple-point and single-point borehole extensometers, load cells, pressure cells, stressmeters, strain gauges, inclinometers, piezometers, and lateral movement gauges. Data from these instruments were collected manually as well as read remotely by an automatic datalogger system at the surface. All data were entered on magnetic tapes for data reduction, tabulation, analyses, and archiving. The frequency of data collection was determined on a per instrument basis and was dependent on instrument location, method of instrument reading, and the number of days elapsed since excavation at the instrument location.

Analysis
Statistical and numerical methods were used to analyse in situ data and to evaluate the physical behaviour of salt. Numerical models were used to compute empirical creep parameters from the in situ data.

Since closure behaviour as a function of time is an important requirement for design validation, in situ measurement data for the drifts and test rooms were fitted with empirical equations using regression procedures. Since early data were lacking for most of the instruments, one approach was to calculate the closure rates and fit an equation to the closure rate versus elapsed time relationship. An estimate of the early closure not

measured by the instruments was independently derived. More than three years of measurement data and data analysis have provided an adequate data base for predicting future behaviour.

Closure Behaviour of Shafts

The radial convergence of the vertical shafts was monitored by convergence points as well as radially located multiple borehole extensometers (MPBX) which are each 36 feet long. Analysis of the MPBX readings indicated that the instrument was affected by seasonal changes in temperature in the ventilation shaft. Comparison of convergence point readings with the collar movement readings suggested that the radial extent of the participating salt in the creep process could be greater than 36 feet.

Stability of Shaft Station

The station below the Construction and Salt Handling shaft (CaSH shaft) exhibited spalling from both roof and wall surfaces that required continuous monitoring and maintenance. Analysis of periodic measurements of the roof MPBX indicated that the relative percentages of movements between intermediate anchors have remained essentially constant over a period of three years of measurement indicating that there is no accelerating trend in the rate of opening of a clay seam some distance above the roof.

Analysis of Closure Rates in Drifts and Test Rooms

An estimate of the early closure immediately after excavation and before the instrument was installed was made by fitting a cubic equation to the early data points taken after the installation of the instrument. If $R(t)$ is the instrument reading at time t since the end of the excavation, and R_1 is the regression parameter equal to the total closure at elapsed time t_1, then

$$t = t_1(1 + R(t)/R_1)^3 \ . \tag{1}$$

A non-linear statistical analysis enabled an estimate of the value R_1.

Closure rates in salt generally decrease with time except for occasional increases due to disturbances from nearby excavations. Thus they may be assumed to reach a steady state value until the onset of tertiary creep. It is possible to get an estimate of the steady state closure rate using either a phenomenological model or an empirical model that is essentially a curve-fitting technique.

Closure rates were used in the empirical approach because measured closures were not absolute. Since the closure rates will be initially relatively high and then decrease to a steady state, the following model was used:

$$C(t) = (A/t^n) + C_c(1 - e^{-at}) \ , \tag{2}$$

where $C(t)$ is the closure rate at elapsed time t after excavation; C_c is the steady state closure rate; and A, a, and n are constants.

The values of $C(t)$ were determined by arbitrarily choosing a time interval of not less than 7 days.

$$C(t) = \frac{R_{i+1} - R_i}{t_{i+1} - t_i} \ , \tag{3}$$

where R_{i+1} is the instrument reading at time t_{i+1}, and R_i is the reading at time t_i. The value of t was determined as the average of the time intervals t_i and t_{i+1}.

The non-linear regression analysis was done using iterative methods such as Gauss-Newton or the Marquardt compromise technique. However, non-linear regression analysis did not always give good estimates of the value of C_c. Correlations of estimated steady-state closure rates with room dimensions were made.

Instrument monitoring of the underground openings is very valuable in assessing their closure behaviour. In conjunction with periodic, in situ observations, they are useful in predicting the future closure behaviour of the openings and in particular the onset of their potential failure.

ANALYSIS OF THE DEFORMATION OF NETWORKS WITH ABSOLUTE ORIENTATION

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In deformation measurements, it is usual to repeat the measurements of reference points in networks. The displacements of the points representing the object under observation are then derived from the computed positions. The stability of the reference network points is threatened by many factors, for example, the sinking of observation pillars, geotechnical reasons, the traffic of heavy building machines in the neighborhood, or the changes due to deformation of the building constructions.

As for as the reference network or its part which lies inside the deformation zone, it may be difficult to identify the stable points in mutual transformation of the network structure. In linking the particular measuring epochs, congruence analysis with various statistical identity tests are used currently for single points or for groups of points. Considerable correlations in the results of the network computation presents a problem that is caused by a common adjustment of homogeneous data in fulfilling the chosen abstract geometrical conditions. That is why every possibility for independent checking of the network position or orientation is a significant contribution to solve the problem.

A possibility is the absolute (astronomic or gyroscopic) orientation of a certain number of sides in the deformation reference network. Both the methods in question have their advantages and disadvantages. Thanks to the recent development of gyroscopic instrumentation and measuring technology, the precision reached in using both methods can be compared and makes some seconds in an absolute azimuth.

For detecting the displacements of reference network points, it is not necessary to determine the absolute azimuths of the sides, but only their azimuthal differences between the particular measuring epochs. The differences can be determined with the accuracy better than one second by an adequate astronomical method, without any special instrumental equipment. If the Polaris (or sun) hour angle method is employed, an ordinary one second theodolite can be used, as it is in angular measurement of the network, and a relatively simple time measuring equipment, for example, a good quartz stopwatch. The following processing of measured data is quite simple too.

When the indirect azimuthal method is applied, the resulting azimuth is obtained as the sum of the azimuth of an arbitrary celestial target and the intermediary horizontal angle including all relevant corrections. In azimuthal differences, only the corrections that vary with time are to be taken into account; other corrections need not be calculated. The corrections in question concern the influence of the polar motion and the difference between the rotation and radio signal time. Steep sight lines on a celestial target are always influenced by non-verticality of the alidade axis and must therefore always be

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 123

corrected. The uncertainty in determining this correction is one limiting factor as far as the accuracy of the azimuthal differences method is concerned. The construction of modern theodolites now enables us to determine the inclination of the alidade axis with higher reliability, as it was formerly by means of a striding level. For example, very convenient for the purpose is Baumann's method which applies the automatic index of the vertical circle. Other systematic instrumental errors that influence the resulting relative azimuth are alidade fluctuation and changes in the sight line when focusing on near targets. The influence of these errors can be reduced in application of suitable measuring procedure, attention should also be given to the centring of both the instrument and target in the transversal direction.

External accuracy of the relative azimuth or azimuthal difference is dependent above all on the accuracy in timing and on the accuracy in determining the intermediary horizontal angle. On the basis of the author's experience, accuracy of better than one second can be reached in azimuthal differences in about one hour's time (6 measuring sets) by means of Polaris, and accuracy of about two seconds by means of the Sun. The accuracies mentioned are results of long-term measurements on a test basis.

A relatively very precise determination of azimuthal differences can be carried out economically and with currently used instrumental equipment. For the purpose, the Polaris hour angle method is especially suitable. The extension of observation time is possible in regard to the visibility of Polaris both in the early morning and late afternoon hours.

Changes in directions of the deformation reference network sides can be used for rapid detection of unstable points in the first stages of data processing as well as for introducing complementary conditions in the network adjustment. The main contribution of the absolute orientation in deformation measurements is the independent check that increases the reliability in determining the resulting displacements.

AN ITERATIVE MODELLING OF GROUND SUBSIDENCE USING NON-LINEAR ELASTIC FINITE ELEMENT ANALYSIS

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Ground subsidence in particular or deformation of any deformable body in general may be modelled by using either empirical (statistical) or deterministic (mechanistic) methods, depending on the type of information available.

Most empirical theories for predicting ground subsidence have been developed in central Europe, where systematic and accurate monitoring surveys have been conducted in mining areas for several decades. The empirical prediction theories are applicable only to the areas where the mining, geologic, and tectonic conditions are the same or very similar to the area where the empirical data for the theory had been collected. Since the conditions in different areas are never the same, any attempts to adapt, for instance, the European empirical models for ground subsidence prediction in North America require a calibration of the model parameters through many years of comparisons with the observed deformations in the new area. Usually this approach is unrealistic because very few, if any, mines in North America have a systematic program of monitoring surveys. One should also stress that the empirical theories are generally not reliable in cases of complicated geometry of the mined deposits, in the presence of faulting, and in the areas of previous extensive mining operations.

The deterministic models of ground subsidence are obtained from the a priori known properties of the material, from the mechanical relations between the loads and internal stresses, and from the physical laws governing the stress-strain relation. The deterministic methods are definitely more universal than the empirical theories because they can be applied in any geological and mining conditions and they provide information not only on the surface subsidence but also on deformations within the rock masses and within the mine workings. They require, however, reliable information on the in situ properties of rocks, initial stresses, and tectonics of the area. Unfortunately, the information on the rock's behaviour and their properties is usually incomplete or based only on laboratory determination of the mechanical parameters which may considerably differ from the in situ values. Therefore development of a reliable deterministic methodology for modelling the ground subsidence is not an easy task. It has taken the author several years to develop such a method and supporting software which are reported in this paper.

Basic Principles of Szostak-Chrzanowski Method
The method is based on an iterative non-linear elastic finite element analysis of deformations coupled with empirically known behaviour of rocks above mining openings. The analysis is performed in the following steps.

Step 1. Elastic solution without mining opening with given properties of the material (including anisotropy) and with initial tectonic stresses if their influence is known. If in situ elasticity parameters are unknown, then the laboratory values of E are reduced for the analysis by a factor of 3 to 5 depending on the qualitative properties of the rocks and using as guidelines some comparisons given by Bieniawski [1984].

Step 2. Elastic solution with the mining opening and determination of:
- 'Weak' zone which is delineated by the elements with maximum shearing stresses between the opening and the surface. If obvious discontinuities (e.g., faults) are present in the vicinity of the opening, then the 'weak' zone is extended to the discontinuities.
- Critical tensional zone of the elements in which tensional stresses exceed critical values.
- Strains and displacements by combining Step 1 and Step 2 solutions.

Step 3. Non-linear elastic solution with the mining opening by introducing:
- $E/3$ in the 'weak' zone.
- $E=0$ (isotropic) in the critical tensional zone immediately above the opening (up to 100 m).
- $E=0$ in the critical tensional elements in the direction (anisotropy) of the tensional stresses.

A new state of stresses and displacements is obtained and new elements with critical stresses may be obtained.

Step 4. Iterative repetition of Step 3 until no additional elements with critical stresses are found.

The above procedures are handled by program FEMMA (Finite Element Method for Mining Applications) developed by the author at the University of New Brunswick. The program uses quadrilateral elements with an automatic mesh generation. The method has been successfully applied to coal, copper, and zinc mines in Canada, the U.S.A., the People's Republic of China, and Poland. Due to the limited space, only two examples are given below. More examples will be given in the full paper. See also Chrzanowski and Szostak-Chrzanowski [1987].

Example 1: Shoemaker Coal Mine, West Virginia, U.S.A.
The described methodology has been employed in calculations of ground subsidence produced by longwall mining of a 1.7 m thick and 183 m wide coal panel at an average depth of 180 m. Geological profiles (mainly shales and limestone) and observed subsidence values were taken from Adamek and Jeran [1981]. The mechanical parameters of the rocks had to be taken as average values published in Bieniawski [1984] for the given types of rocks. Figures 1 and 2 show the FEM mesh and comparison between the calculated and measured subsidence profiles respectively. The agreement is astonishingly good.

Example 2: 'T' Mine of Lead and Zinc, southern Poland
Figure 3 shows the FEM mesh and the geometry of a two-layer extraction of a lead-zinc deposit with a thickness of 4 m and 5 m of the lower and upper extracted panels respectively, separated by a 4 m thick unexcavated layer. A hydraulic backfill was used in both excavated layers with an estimated 15% consolidation. The mechanical parameters of the rocks (dolomite domination) have been taken as average values

published in Kidybinski [1982]. Figure 4 shows the comparison between the calculated and measured subsidence.

Conclusions

The developed methodology for deterministic modelling of ground subsidence provides a universal tool for predicting purposes in any geological and mining conditions. The presented examples show excellent agreement between the calculated and observed deformations, though the data on the mechanical properties of the rocks had to be approximated from publications related to other mining regions. The program FEMMA, in its present form, is applicable only to a two-dimensional analysis of cross sections of elongated shapes of excavations. Its adaptation to three-dimensional analysis is in progress.

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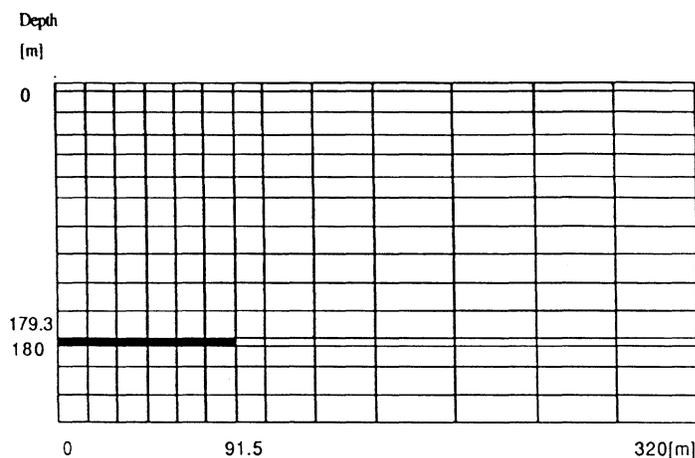
Figure 1. Shoemaker Coal Mine. FEM model.

5th Int. Symp. Deformation Measurement
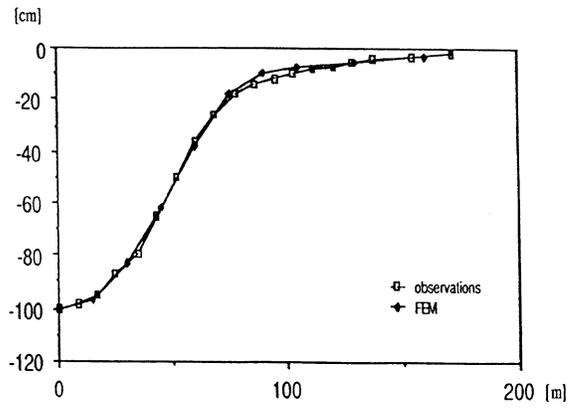5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 127

Figure 2. Shoemaker Coal Mine. Subsidence profiles.
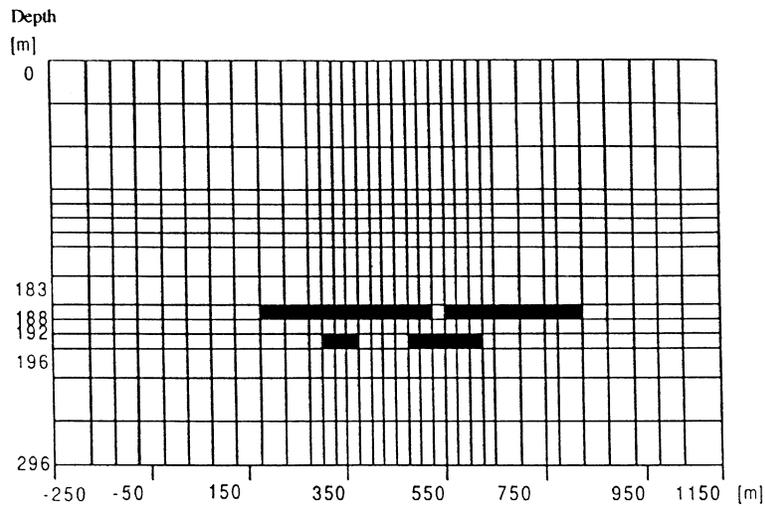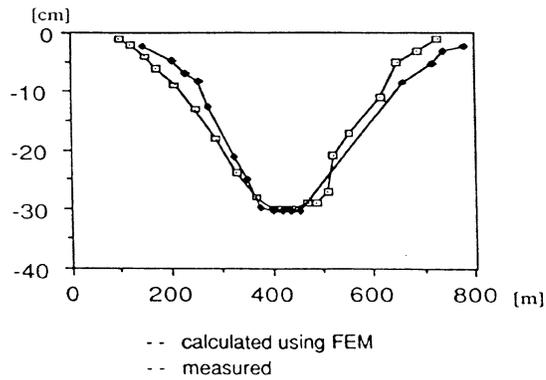

Figure 3. 'T' Lead-Zinc Mine. FEM model.

Figure 4. 'T' Lead-Zinc Mine. Subsidence profiles.

GROUND AND ROCK DISPLACEMENT IN REN FENG WOLFRAMITE

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This paper introduces research on ground and rock displacement in Ren Feng Wolframite in southern Jiang-xi (P.R.China), known worldwide for its production of tungsten. This research by the Surveying Group of the Jiang-xi Institute of Metallurgy, completed in January 1986, was part of an overall research on ground pressure activity and its control in deep level mining.

A comprehensive geodetic network of 31 lines, over a total length of about 6 km, was established with connections at several levels to 865 m below the surface. Data was processed by and stored in a Sharp PC1500 and was found to be acceptable, e.g., with a subsidence constant of $a = 0.1$, the standard error per levelling phase was less than $\pm$ 1 mm, achieved with additional care in the measurements.

From the analysis of the various observations, the centres and areas of ground pressure were defined, the spatial distribution and expression of rock displacement in the ore areas, and the displacement relationship with rock structure were determined. Also, the relationship between rock displacement and mining activity and some slip faces were discovered.

The total research programme involved specialists in rock mechanics and engineering geology as well as surveyors. Comparison of the several approaches showed consistent conclusions.

Deformation surveys have been shown to be very useful in rock deformation research and this should be emphasized to other specialists and to surveyors who are not fully aware of the potential of their involvement. Surveyors should become better acquainted with the theory of deformation and stress analysis in order to progress in their research work.

5th Int. Symp. Deformation Measurement
5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 129

AN INTEGRATED METHOD FOR MONITORING THE BEHAVIOUR OF ENGINEERING STRUCTURES

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Deformations of engineering structures are often measured in order to ensure that the structure is exhibiting a safe deformation behaviour. These measurements are usually most critical early in the life of the structure when full loadings are first applied.

Methods of Measuring Deformations

It is possible to measure deformations by a number of different methods. These methods can be conveniently categorized as:

- survey methods (including both direct measurement methods and survey network methods);
- geotechnical methods; and
- structural methods.

Displacements are usually measured by the survey methods, although rotations and strains can also be measured. The accuracy attainable using the survey network method (conventional, photogrammetric, or GPS (Global Positioning System) network, or some combination) is now about 4 ppm (parts per million) of the extent of the network.

The use of devices such as extensometers (which measure change in length to an accuracy of about 0.5 mm) are typical of geotechnical methods of measuring deformations, while the use of devices such as strain gauges (which measure strain to an accuracy of about 10 microstrain) are typical of structural methods of measuring deformations.

Integrated Analysis

It is obvious that for all but very simple structures, deformation measurements at discrete points on or in the structure will not provide a complete solution for the deformation behaviour throughout the structure. To accomplish this, a deformation model must be used. The best available choice for a deformation model is an appropriate structural finite element model. It is the best choice because it takes into account the geometry and boundary conditions of the structure, the stress-strain relationships of the materials in the structure, and the forces applied to the structure (see Bathe [1982]; Zienkiewicz [1977], and other standard references).

Integrated analysis, which was developed by Teskey [1986], can be considered an extension of the structural finite element method that is applied to integrate all possible types of actual measured deformation data in a deforming structure into the appropriate finite element model for the structure. When integrated analysis is applied, an optimum

solution for actual deformation behaviour is produced, along with the estimated precision of the solution, and the redundancies contributed by the structural finite element method and the measured deformations.

Applications of Integrated Analysis

Earthfill Dam. The first example of integrated analysis is its application to an earthfill dam in West Germany. Shown in Figure 1 are the deformations of the structure, at the measurement points, due to the dead weight load plus hydrostatic pressure and porewater pressure load. The displacement measurements were all made using the survey network method with conventional survey observations. The integrated analysis results were computed using a program system developed by Teskey.

The integrated analysis solution for this structure shows that up to the completion of the first filling of the reservoir it was exhibiting a very safe deformation behaviour. In addition to this, the solution was shown to be reasonably precise (estimated precisions of displacements and material parameters were in the 5% to 10% range) and the redundancy contributed by all the measured displacements was about 50% of the total redundancy.

Olympic Speedskating Oval. The second example of integrated analysis is its application to the Olympic Speedskating Oval in Calgary. Shown in Figure 2 are the deformations in the middle cross section of the structure due to the dead weight load of the roof. The displacement and rotation measurements were all made using direct survey measurements. The integrated analysis results were computed using program system STFN [Desai, 1979] to provide the three-dimensional linear elastic structural finite element solution and program system DEFINA II, written by the authors, to perform the integrated analysis.

The results show that for the loading due to the dead weight load of the roof, the structure is exhibiting a very safe deformation behaviour. The solution was also shown to be precise (estimated precisions of displacements approximately 5%) and the redundancy contributed by the measured deformations (displacements and rotations) was approximately 70% of the total redundancy.

Conclusion

Both the structural finite element method and the method of integrated analysis can be applied to any deforming structure. Using the method of integrated analysis, one can compute an optimal solution for actual deformation behaviour. The method also makes it possible to design better monitoring schemes. (This would be done by deleting planned deformation measurements having high (>80%) redundancy numbers and adding additional or more precise deformation measurements in areas where planned deformation measurements have low (<20%) redundancy numbers.) The final advantage of the method of integrated analysis is that in the case of unsafe deformations, the method would aid in the design of remedial works by identifying the problem areas.

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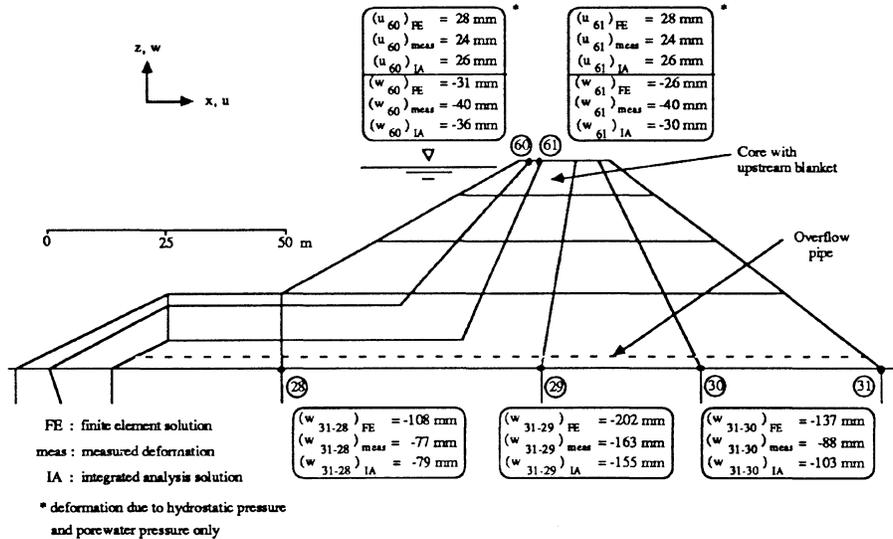
Figure 1. Deformation due to dead weight load plus hydrostatic pressure and porewater pressure load, earthfill dam.
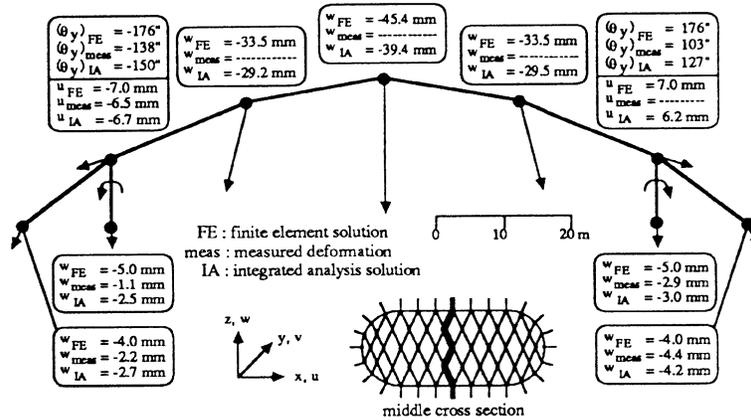

Figure 2. Deformation due to dead weight load of roof, middle cross-section, Olympic Oval.

CALIBRATING A DIPSTICK

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The large road tankers which deliver to petrol stations usually have 6 compartments, each with a capacity of 7000 litres. In practice, approximately 40 000 litres of petrol or 32 000 litres of diesel are the maximum amounts carried because of weight restrictions. The traditional method of measuring volume involves checking the depth in each compartment using a dipstick in the form of a long wooden rod, 25 mm square, which is dipped into the tank by the driver standing on the top and noting where the liquid level is in a similar way to checking the oil level in a car engine. In practice, this method of depth measurement has several disadvantages, as summarised below.

(1) From a safety point of view, it is dangerous to have people walking around on the top of road tankers, particularly if the catwalks become covered in petrol or diesel, making them very slippery.

(2) In the traditional method, false volumes can be delivered by an unscrupulous driver simply holding the dipstick at some level in the tank and claiming that the contracted amount had been delivered. The staff at petrol stations are often part-timers who are not very keen to scramble about on top of greasy tankers and, consequently, they are quite prepared to accept the word of the driver.

(3) An accurate measurement of genuine deliveries is difficult to achieve. A difference of 1 mm in a dipstick depth reading corresponds to approximately 1/1000th of the volume of the compartment, i.e., approximately 7 litres. Since the profit margin on petrol and diesel is not that large, inaccurate depth readings can erode profits. In practice, the wooden dipsticks are only accurate to approximately ±20 mm.

(4) With traditional methods, deliveries are restricted to working hours since, officially, someone other than the driver of the tanker has to check the delivery.

The New Measuring System

A new system of measuring volumes inside the tanker compartments has been developed by Drum Instrumentation who are part of the Drum Engineering Group based in Bradford, England. The technique involves an electronic dipstick and overcomes all the problems outlined above, in particular:

(1) The new system has an onboard computer which measures the volume automatically and eliminates the need for anyone to walk on top of the tanker.

(2) The computer displays the volume delivered on a control panel mounted on the side of the tanker at a convenient level which ensures that all volumes can be accurately monitored by the garage staff.

(3) The new dipstick measures depth to a precision within ± 1 mm.

(4) With the new system, if a small computer is installed at the garage, then the tanker computer can simply be plugged into the garage computer through a socket on the forecourt. The tanker computer checks that it is the correct garage and confirms the amount to be delivered. The delivery is then carried out automatically under the supervision of the tanker computer, even when the garage is closed.

The new system works on the 'added mass principle.' The dipstick is vibrated and, depending on the depth of the liquid, it will have a certain frequency. If the relationship between frequency and depth is known, then knowing one enables the other to be calculated. Once depth is known, volume is obtained from the internal dimensions of the tanker. In practice, when deliveries are being made the reverse of this process is carried out; the frequency is measured and knowing the depth/frequency relationship, depth can be calculated and hence volume determined.

Calibration

Each dipstick manufactured by the company is calibrated before being fitted into the tankers. This is done by attaching each dipstick to a calibration rig and then completely immersing it in a control tank of liquid to its fullest depth. The dipstick is then slowly withdrawn, 10 mm vertically at a time, and its frequency is measured at each point. This enables a graph of depth versus frequency to be plotted for the dipstick in question which is then used to program the computer which will control the delivery monitored by that dipstick.

The Survey Section of the Civil Engineering Department at the University of Leeds, England, were called in to check the rig used to calibrate each dipstick because Drum Instrumentation were uneasy about the reliability of the depth measurements it gave. They had built their calibration rig around a rolled steel joist (RSJ) which they had thought would be perfectly straight. This was mounted vertically, and they had assumed that the dipstick support, which was mounted on a screw drive, would travel exactly vertically as it moved up through its complete range. However, their initial tests revealed that the RSJ was considerably distorted as far as their accuracy was concerned, and, for example, a 10 mm movement registered on the screw drive control was not in fact a vertical movement of 10 mm.

In order to enable their system to be accepted by the Weights and Measures Section of their local Trading Standards Service, Drum Instrumentation required to know the accuracy of the vertical movement of the dipstick support to ± 1 mm. Consequently, the Survey Section was asked to check the vertical readings obtained from the screw drive control to determine its accuracy, i.e., if it read 2080 mm vertically above the zero point, for example, was this correct?

Using the Wild RMS 2000 mini system owned by the Survey Section, the calibration rig was checked as follows:
(1) A suitable target was attached to the dipstick support.
(2) The two theodolites were set up in a good configuration approximately 3 m from the rig.
(3) The dipstick support was lowered to its zero point.
(4) The target was sighted and its z, y, and z coordinates set at x = 10 m, y = 10 m, and z = 10 m to avoid negative values. The reading given on the screw drive control for this point was also recorded.
(5) The support was raised vertically using the screw drive by what was thought to be 10 mm, the target resighted, and new x, y, and z coordinates were measured. The screw drive control reading was again measured.
(6) This was repeated over the full 4.5 m range of the screw drive.

A computer program was used to calculate the x, y, and z coordinates of the target as it moved vertically through the full range of the screw drive. Drum Instrumentation

were supplied with a computer file containing the x, y, and z coordinates of the rig support together with a measure of the error in the depth (z) values displayed on the screw control drive. The results were used to calibrate the screw drive control and they have since received approval for their new dipstick which is now in full production.

The Wild RMS 2000 mini system used to check the rig consisted of two T2000 electronic theodolites (reading accuracy 0.1"), a carbon fibre scaling bar of very precisely known length (1.305676 m), and a Wild GRE3 data recorder with its associated software. The target used was a black cross on a yellow background, the centre of which, to the naked eye, appeared to give a very distinct point for viewing. However, under the 35X magnification of the theodolite telescopes, the centre of the target appeared very irregular. Fortunately, there were several very well-defined minute black marks within this irregular image and one of these was used as the exact sighting point. Subsequent projects with this RMS system have revealed that well-defined targets are absolutely crucial to the accuracy of the technique.

An interesting footnote to this problem is that a previous attempt to calibrate the rig by another organization using a laser interferometer had not been completely successful. The system used involved mounting a mirror on the dipstick support and a prism directly underneath it at the base of the rig. The laser beam was then fired into the prism which deflected it upwards to the mirror where it was reflected back to the prism and returned to the laser instrument. Unfortunately, as the support moved upwards, it began to drift further and further off vertical in the x and y directions due to the distortions in the RSJ and eventually, at slightly more than 2 m, the returning laser beam was so far off line with the outgoing laser beam that the interferometer could not cope and no further measurement was possible.

TRANSFER OF MINING-INDUCED SUBSIDENCE TO SURFACE STRUCTURES

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Damage to surface structures as a result of mining-induced subsidence has been a concern of coal operators and land owners for many years. In the past, most mine subsidence events were unplanned, occurring over abandoned mines long after coal removal. However, with the increasing use of full extraction mining methods, which cause surface ground movements to occur immediately after mining, there is a growing public concern about the effect of these movements on surface structures and features. With more and more longwall mine operators planning to undermine populated areas, it is expected that this issue will become one of the most important considerations for mine planners and regulatory officials. At present, mine operators establish the layout of their mines to avoid longwall mining under surface structures whenever possible. Instead, they utilize more costly room-and-pillar methods under populated areas so that support pillars can be left in these areas without severely altering production. However, as the number of mining areas with few surface structures decreases and the economic need for high extraction mining increases, it becomes obvious that methods to protect surface structures while allowing the use of complete extraction mining methods need to be developed.

The Bureau of Mines, as part of its research to delineate ground movements associated with subsidence, has initiated a program to assist the mining community in the prediction of mining-induced stresses that may be transferred into a surface structure. Very little information is presently available concerning the surface effects and amount of damage that can be expected as a result of mining. What information is available is most likely based on European methods and, like European subsidence prediction models, may be inappropriate for conditions existing in the United States. The concept of the Bureau's program is to monitor the movements of both existing and specifically designed structures in response to mining-induced subsidence in an effort to determine the mechanism for the translation of ground strains into a structure. Once an understanding of this mechanism is established, it may be possible to modify existing subsidence prediction models to also predict mining-induced strains in surface structures. Accordingly, a pilot project was initiated to record the movements of surface structures and the adjacent ground.

A study site was selected in Barbour County, WV, because the topography was typical of the Appalachian Coal Region. The surface area consisted of fairly level land over a longwall panel. The depth to mining was approximately 198 m and approximately 1.8 m of coal was extracted. The overburden lithology consisted of fine-grained sedimentary rock with numerous massive sandstone units, representing 37% of the overall thickness.

Four masonry walls were constructed over the longwall panel in areas where maximum subsidence effects, such as compression, tension, and inclination, were predicted by the Bureau's subsidence prediction model. The longest wall, termed the 'Dynamic wall,' was located along the centerline of the panel parallel to the long axis. This wall was specifically placed so that it would flex in response to the advancing subsidence wave. The remaining three walls, termed 'Static walls,' were placed perpendicular to the advance of the longwall face. The specific locations of the walls relative to the panel were calculated, prior to mining, to be in areas of maximum compression, maximum curvature, and maximum tension.

This foundation response project was part of a larger study to monitor subsidence over two adjacent longwall panels. For the larger study, subsidence monitoring grids, consisting of a 457 m baseline parallel to the centerline of the longwall panel and two perpendicular profile lines, were established above each panel. During the 60-day period when the walls were undermined, survey data were collected from each wall and ground surface arrays on a regular basis to detect both vertical and horizontal displacement. The dynamic wall included additional instrumentation to continuously measure the inclination during critical portions of the study period.

The results of the field measurements showed that the three static walls were not damaged in any way by the subsidence process. The foundations remained intact, and no cracks were observed in the blocks or mortar joints. The final subsidence-induced ground movements measured along the profile where the walls were located were similar to the values predicted by the Bureau's subsidence model. As predicted and designed, the walls were correctly located in the areas of maximum compression, maximum inclination, and maximum tension.

The dynamic wall was located along the centerline of the panel where it would be subjected to the most extreme of the various stresses associated with the advancing subsidence wave. As a result of the rapidly changing ground surface due to undermining, the wall broke into three approximately equal pieces.

The direction and timing of the cracks in the wall with respect to the location of the face gives some insight as to when damage will occur to structures. As expected, the portion of the wall nearest to the advancing face failed first. The exact time of failure was not observed, but it is estimated that the failures occurred when the face was between 0 m and 30.5 m beyond the surface point. Failure cracks occurred in the footing and traveled vertically through the first course of block. From above the first course of block to the top of the wall, the cracks traveled in a stepwise fashion upward along the mortar joints. The width of the failure cracks increased until the ground reached maximum measured subsidence. These ground cracks closed within one week.

The orientations of the failure cracks give some indication of how the wall responded to the subsidence-induced ground distortion. The leading edge of the wall dropped as the longwall face passed directly beneath it. This section of the wall broke away from the remainder of the wall and moved as an independent unit by rotating at the point of failure in the footing. As the face advanced further, the second portion of the wall began to rotate near its midpoint and lifted the extreme end. It is unclear what caused the second failure to occur in a direction opposite that of the first. However, at the time when the first crack was observed, hairline cracks were also observed in the upper

mortar joints in the section where the second crack was later observed. No cracking of the footing was observed at this time.

Tiltmeters installed in the wall and the ground continuously measured inclination for 30 days during the period in which the wall was undermined. (Note that for the purpose of this paper, tilt and inclination will be considered synonymous.) The tiltmeters indicated that the advancing subsidence did in fact pass through both the wall and the ground in a wavelike motion. However, the inclinations observed in the wall and the ground were not identical. In general, more inclination was measured in the ground than in the wall. The maximum inclination measured at the wall was 11.92 mm/m (0.683°), and the maximum inclination observed in the ground was 12.43 mm/m (0.712°). These maximums occurred at a tiltmeter adjacent to one of the cracks when the face had passed the tiltmeter by approximately 310 feet. After the completion of the vertical displacement, the tiltmeters did not return to a neutral position. Residual inclinations of approximately 1.92 mm/m (0.11°) and 2.44 mm/m (0.14°) were measured for the wall and the ground respectively.

This case study is a first step toward understanding the mechanism that transfers mining-induced ground stresses to surface structures. Based on the initial analysis, it appears that surface structures, such as walls, move in a manner similar to the adjacent ground. Because of this indication, it is now worthwhile to closely analyse the more than 6000 measurement records of 3 different parameters for each of the 8 paired sets of tiltmeters. However, before any significant conclusions can be drawn from the measured differences between mining-induced ground movements in structures and in the ground surface itself, many more studies must be completed. Once the transfer mechanism is defined through further field and laboratory research, prediction models can be developed that will allow accurate estimation of the effect of mining on surface structures.

NORTECH EXPERIENCE WITH GPS FOR PRECISE ENGINEERING AND SUBSIDENCE APPLICATIONS

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Nortech Surveys has utilized Global Positioning System (GPS) technology for numerous precise engineering and subsidence applications. The experience and results achieved on three projects will be presented. The projects include confirmation surveys on the precise relative positioning of stations used for alignment of tunnel excavations. In one case, the initial positioning was performed using conventional survey techniques; in the second case, GPS techniques were employed on the original survey. The third project was performed to test achievable accuracy for offshore platform subsidence monitoring. This test involved long baselines of 100 km to 200 km in length, and long-term repeatability over a 2.5 year period. The results of the three projects demonstrate repeatability of baseline components of 1 part per million (ppm) and better in most instances.

Objective

The primary objective for the three projects discussed is to establish precise relative positional control. In the first two projects involving tunnel alignment, precise three-dimensional control in a local datum is required in order that tunnel excavations, initiated at both ends of the tunnel, will connect within established design specifications. The third project requires the precise determination of relative height displacements to monitor possible offshore platform subsidence.

Methodology

The precise relative positioning was achieved using dual frequency carrier phase observations from GPS. These measurements were processed using NOVAS, a software package developed at Nortech Surveys [Wanless and Lachapelle, 1987]. NOVAS employs a standard double difference observation model in a baseline mode, and a sequential adjustment algorithm using adaptive Kalman filtering techniques.

Project 1: Rogers Pass Tunnel Alignment. The Rogers Pass alignment survey was carried out for Canadian Pacific Railway to construct a new tunnel through the Selkirk mountains in British Columbia, Canada. The original control was established using conventional survey techniques, including precise astronomic observations [Lachapelle et al., 1984]. This survey established the relative positions of each tunnel portal, approximately 14.5 km apart, to a precision of better than 4.5 cm horizontally at a 95% confidence level. The orthometric height difference was established using precise levelling to a precision of 1 cm.

A subsequent GPS survey was performed between the two portals to confirm the conventionally determined relative position. The agreement between the GPS and

conventional surveys was 1.0 cm and 3.7 cm in easting and northing respectively. The ellipsoid height difference determined from GPS observations, compared with the orthometric height difference, yielded an undulation of 54 cm, which is considered normal for such a mountainous region.

Project 2: English Channel Tunnel Alignment. A control survey was performed to establish primary control for the alignment of a tunnel to be constructed under the English Channel, connecting England and France. GPS surveys were conducted using single frequency receivers by the Ordnance Survey (OS) and Institut Geographique National (IGN) on respective sides of the channel. The GPS survey across the channel, connecting these individual networks, was performed by Nortech Surveys using dual frequency receivers. The final network adjustment was carried out by IGN and reduced to a local mapping grid established specifically for tunnel layout procedures.

The adjustment of the dual frequency cross-channel network indicated an average precision of 1 ppm horizontally and 3.8 cm vertically was achieved.

Project 3: Subsidence Monitoring over Long Baselines. A project was carried out for Shell Internationale Petroleum Maatschappij (SIPM) in The Netherlands to investigate the applicability of GPS for monitoring the subsidence of offshore oil platforms. The project was conducted between 1985 and 1987, during which three measurement campaigns were performed. The requirement was to achieve long term repeatability at the centimetre level on baselines 100 km to 200 km in length, primarily in the height component.

The test project included dual frequency observations of a triangle of three stations, approximately 100 km to 212 km apart. The results, using broadcast ephemerides, showed repeatabilities better than 1 ppm in all three components of the baseline, and 0.5 ppm or better in height repeatability over the 2.5 year period. Baseline length repeatability was 1 cm plus 0.5 or better.

Conclusions

The three projects discussed above demonstrate the accuracy and reliability of GPS surveys for establishing precise relative control. The use of appropriate equipment and techniques will yield accuracies of 1 ppm or better. These projects also illustrate the cost effectiveness of using GPS techniques.

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RELIABILITY THEORY OF THE GENERALIZED GAUSS-MARKOFF MODEL AND ITS APPLICATION IN THE OPTIMAL DESIGN OF DEFORMATION NETWORKS

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From both theory and practice, reasons for expanding the Gauss-Markoff model are given with both the design matrix, A, and the weight matrix, P, possibly being rank deficient:

$$\underline{L} + \underline{V} = A\underline{X}, P$$
$$H_0 : E\{B\underline{X}\} = \underline{W}$$
$$H_1 : E\{B\underline{X}\} = B\hat{\underline{X}} \ .$$

The relationship between the generalized Gauss-Markoff (GGM) model and the traditional model with constraints is discussed giving methods of statistically checking the GGM.

Reliability theory for the GGM is developed for internal and external reliability. The internal reliability can be found from the error of closure:

$$\nabla_0 S = \frac{S \cdot \sigma_0 \cdot \delta_0(S)}{\sqrt{S^T M_B^- S}}$$

$$M_B = B \, Q_{\hat{X}} \, B^T \ ,$$

and since rank deficiency may be involved in M, the concept of 'covered space' is defined. External reliability is expressed for the unknowns through

$$\nabla_0 \underline{X}(S) = - \frac{Q_{\hat{X}} B^T M_B^- S \, \sigma_0 \cdot \delta_0(S)}{\sqrt{S^T M_B^- S}}$$

$$\bar{\delta}_0^{\,\prime}(S) = \delta_0(S) \ ,$$

and is extended for a subset, $\underline{x}_1$, of the unknowns by

$$\overline{\delta}'_0(S) = \sqrt{\frac{S^T M S}{S^T M_B S}} \cdot \delta_0(S)$$

$$M = M_B^- B (Q_{\hat{X}}^{\wedge})_1^T N_{11} (Q_{\hat{X}}^{\wedge})_1 B^T M_B^- \ ,$$

in which $(Q_{\hat{X}}^{\wedge})_1$ is a matrix constructed of the rows of $Q_{\hat{X}}^{\wedge}$ corresponding to the unknowns, $\underline{x}_1$, under consideration. The influence on a function of the unknowns is given by

$$\nabla_0 f(S) \leq |e| \ \sigma_0 \ \delta_0(S) \ .$$

Distinguishability theory is developed for the GGM with two alternate hypotheses:

$$\underline{L} + \underline{V} = A\underline{X}, P$$
$$D\underline{X} = \underline{C}$$
$$H_0 : E\{B\underline{X}\} = \underline{W}$$
$$H_1 : E\{B_1 B\underline{X}\} = B_1 B_{\hat{X}_1}^{'\wedge} \ , \ E\{B_1^{'} B\underline{X}\} = B_1^{'} \underline{W}$$

$$H_2 : E\{B_2 B\underline{X}\} = B_2 B_{\hat{X}_2}^{'\wedge} \ , \ E\{B_2^{'} B\underline{X}\} = B_2^{'} \underline{W} \ ,$$

and with expressions for the total correlation through

$$p_T = \sqrt{p_{T_1,T_2}}$$

$$p_{T_1,T_2} = \frac{1}{\sqrt{p_1 \, p_2}} SP(M_{B21} \, M_{B11}^- \, M_{B12} \, M_{B22}^-)$$

$$M_{Bij} = B_i(I - M_B \underline{W}_{ii}) \, M_B (I - M_B \underline{W}_{jj})^T \, B_j^T$$

$$\underline{W}_{ii} = B_i^{'T} \, M_{Bi}^{'} \, B_i^{'}$$

$$M_{Bi}^{'} = B_i^{'} \, M_B \, B_i^{'T} \qquad\qquad j,i = 1,2$$

and for the correlation between pairs of directions by

$$p_{1,2}(S1, S2) = \frac{S_1^T \, M_{B12} \, S_2}{\sqrt{S_1^T \, M_{B11} \, S_1} \cdot \sqrt{S_2^T \, M_{B22} \, S_2}}$$

as well as for the maximum correlation coefficient,

$$(p_{1,2})_{max} = \sqrt{\lambda_{max}(M_{B21} M_{B11}^- M_{B12} M_{B22}^-)}$$

These expressions are reduced to the traditional forms as special cases.

The above is applied to the optimal design of deformation surveys resulting in a standard system including resistance to model error, discovery in the deformation model, distinguishability of model error, and distinguishability between deformation models. The algorithm and an example using a developed program are given.

RELATIONS AMONG FIVE QUANTITIES IN GROUND MOVEMENT

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The traditional relations among five quantities of ground movement, W, T, K, U, ε are really one among the three quantities, W, T, and K. The curvature is the basis for the five quantities since it includes the relationship between subsidence and inclination.

The curvature of the subsidence surface, K_n, is obtained through differential geometry and compared to the curvature derived from surface measurements. This reveals a contradiction between the traditional method of calculating among the five quantities from observations and the theoretical formula. In the traditional method, the inclination, T, is related to the length of the side, l, and the subsidence, W_1 and W_2, at the two ends of the side by

$$T = \frac{W_1 - W_2}{l} \tag{1}$$

Since equation (1) is not taken to the limit, there is not the differential relationship between T and W. Thus T cannot be the derivative of W, and the curvature, K, cannot be the second derivative of W.

A suitable diagram depicting the five quantities has always been a puzzle—whether the subsidence surface can be shown as continuous or as a broken curve. According to the traditional methods, as shown in Figure 1, the left and right extrema are not equal. The curvature, K_m, of the observed curve is continuous beyond the boundary of the mining area but is divergent above the area being mined. This is true along both the major profiles of strike and that of dip.

In order to overcome the divergence of curvature of the observation line, the curvature, K_c, obtained from a fitting to the subsidence curve, is used to replace that of the observation line in the traditional method of calculation. Figure 2 shows that this reduces the extremum although the form of K_c is similar to that of K_n. The negative extremum of K_c is 30% less than that of K_n and the positive extremum of K_c is 52% less than that of K_n. This method of calculating the curvature still does not remove the limitations of the traditional method, and the difference in dip angles of the two sides is automatically reduced when it is divided by the average side length. But, this is not necessary since K_n can be determined for any ppoint on the curve.

In Figure 2, K_n increases by 150%, while K_c increases by only 20% in the range to 20 m from maximum subsidence. The main defect of the traditional method is to decrease the peak value and to slowly reach the bending of the curve. The loss of end

values means that the value of curvature at the end of the observation line cannot be used in the traditional method and is unreasonable when the extremum is at the end of the observation line. The loss of effectiveness is implied by the inability of the traditional method to reveal the many great changes in curvature along the line for an average side length of 10 m to 40 m. Even when this can be revealed, the degree of true curveness cannot be accurate as it is an average over the 10 m to 40 m and this is called loss of trueness.

The values of K for the line S are calculated with a dense distribution of ordinates and this curve, shown in Figure 2, is the closest to K_n. The positive extremum of K and of K_n are both equal to 1 but K is slightly larger than K_n, as at the peak negative value where it is larger by 20%. The positive peak values of K and K_c are equal but the negative peak value of K is 20% larger than that of K_c. Thus, K is more accurate than K_c. The bending of the curve is described point by point through K, but K_c approximates the average bending of each side.

In the traditional five quantities, dip and curvature are not the first and second derivatives of subsidence. Especially for curvature and horizontal deformation, the traditional method has some defects, e.g., insensitivity, loss of effectiveness, loss of trueness, and loss of end value. Before accepting the use of K_n or K_m, it would be practical and reasonable to use the second derivative, K, of the subsidence curve (obtained from the best fit). When using observed data, it is suggested that a subsidence curve be fit to the data first and then the dip and curvature be replaced by the first and second derivative of this fitted curve. The method is unified in the sense that horizontal deformation can be described in the same manner with respect to the fitted curve of horizontal movement.
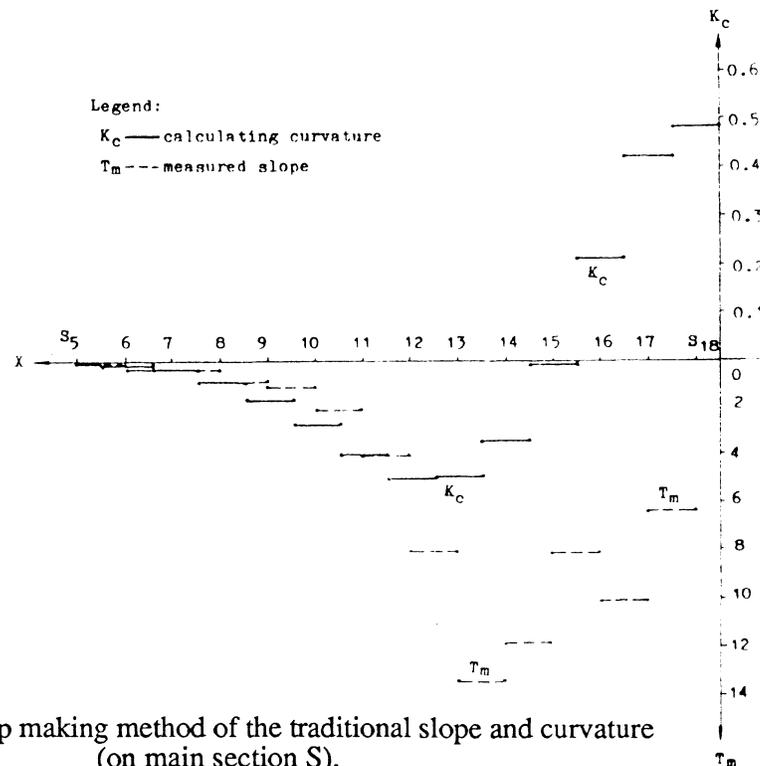
Figure 1. The map making method of the traditional slope and curvature (on main section S).
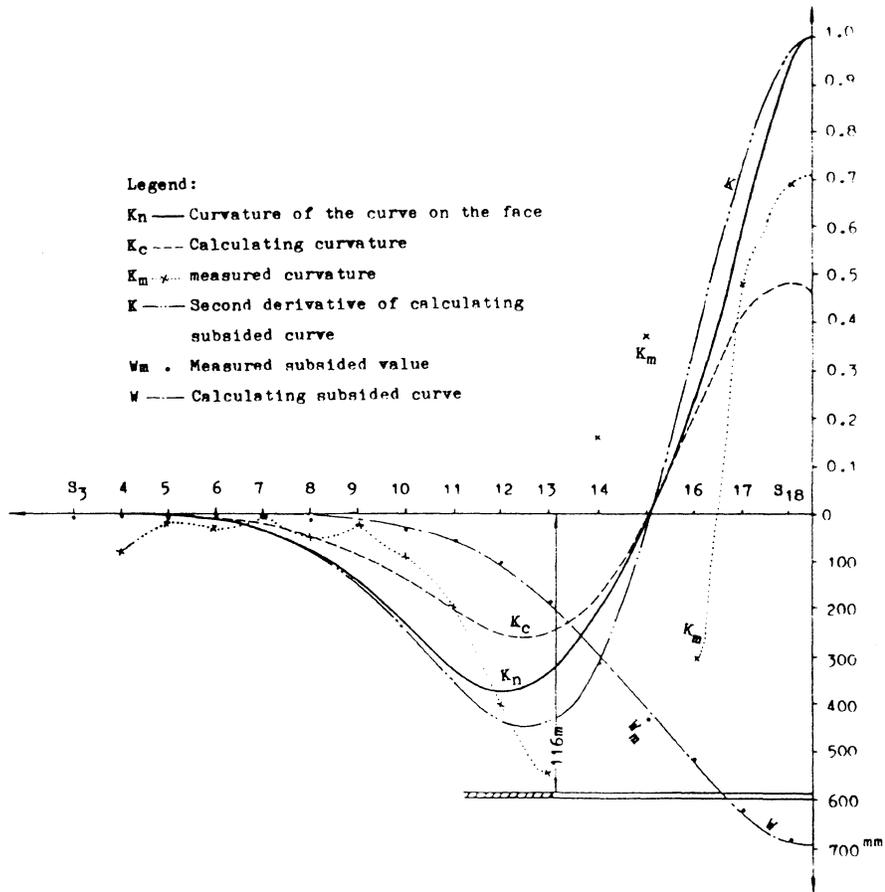
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5th Cdn. Symp. Mining Surveying & Rock Deformation Measurements

Page: 145

Figure 2. Comparison between the curvature of curve on the face and curvature of the measured curvature on main section line S.

AN ANALYTICAL PROJECTIVE METHOD FOR DEFORMATION MEASUREMENTS

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An accurate knowledge of the deformations of structures under static and dynamic loads is of great value to civil engineering for stress analysis. During a test, especially on a model of structure, deformations are measured using mechanical devices such as dial gauges, strain gauges, or electrical devices. The use of mechanical devices is hindered by their inability to maintain a fixed reference point. Another alternative involves using surveying methods, but these methods are unsatisfactory in many of the more stringent applications. The solution of this problem can be provided by photogrammetric approaches, especially by analytical photogrammetry.

The objective of this paper is to present an analytical photogrammetric approach and the laboratory experiments to evaluate the use of photogrammetric methods for measuring structural deformations of a curved precast concrete bridge model at 1/5 scale.

The deflections were measured using surveying and photogrammetry, dial gauges, and strain gauges. Dial and strain gauges as well as 60 targets were attached to the bridge model. The bridge model was fixed on three pillars and deflected at the top by applying different known forces and the deflections at each increment of forces were measured by the above methods.

In order to study the feasibility of non-metric systems for deformation measurements, two different non-metric large format cameras, $f=180$ mm, and $f=400$ mm were tested.

Mathematical Model of Analytical Stereotriangulation

The photogrammetric theory developed in our study for the least-squares adjustment and error propagation of analytical photogrammetric triangulation is based on direct linear transformation (DLT).

The fundamental projective relations for DLT can be put in the form:

$$f(x) = y + \Delta x - \frac{m}{q} = 0$$

$$f(y) = y + \Delta y - \frac{n}{q} = 0 \ ,$$

(1)

where

$$\Delta x = \bar{x} + \bar{x}\,(k_1\bar{r}^2 + k_2\bar{r}^4 + k_3\bar{r}^6 + ...) + p_1\,(\bar{r}^2 + 2\bar{x}^2) + 2p_2\overline{xy}$$

$$\Delta y = \bar{y} + \bar{y}\,(k_1\bar{r}^2 + k_2\bar{r}^4 + k_3\bar{r}^6 + ...) + 2p_1\,\overline{xy} + p_2(\bar{r}^2 + 2\bar{y}^2)$$

$$\begin{array}{l} \bar{x} = x - x_{\mathrm{p}} \\[2mm] \bar{y} = y - y_{\mathrm{p}} \\[2mm] \bar{r}^2 = \bar{x}^2 + \bar{y}^2 \end{array} \qquad \begin{bmatrix} m \\ n \\ q \end{bmatrix} = \begin{bmatrix} a_1 & a_2 & a_3 & a_4 \\ a_5 & a_6 & a_7 & a_8 \\ a_9 & a_{10} & a_{11} & 1 \end{bmatrix} \begin{bmatrix} X \\ Y \\ Z \\ 1 \end{bmatrix}$$

The above relations are the basic equations for the DLT, where the 11 parameters are considered as being independent. In order to obtain an exact solution for the calibration it is necessary to establish two constraint equations, which must be enforced for each station:

$$f_1 = Q_1 - Q_2 + (Q_5^2 - Q_4^2)\,/\,Q_3 = 0$$

(2)

$$f_2 = Q_6 - Q_4 Q_5\,/\,Q_3 = 0$$
in which

$$Q_1 = a_1^2 + a_2^2 + a_3^2 \qquad\qquad Q_4 = a_1 a_9 + a_2 a_{10} + a_3 a_{11}$$

$$Q_2 = a_5^2 + a_6^2 + a_7^2 \qquad\qquad Q_5 = a_5 a_9 + a_6 a_{10} + a_7 a_{11}$$

$$Q_3 = a_9^2 + a_{10}^2 + a_{11}^2 \qquad\qquad Q_6 = a_1 a_5 + a_2 a_6 + a_3 a_7 \ .$$

When two or more cameras photograph common points it is possible to go further by considering as unknowns not only the DLT coefficients, but also the coordinates corresponding to some of the measured images. In what follows, the unknown coordinates of the relative control points are not eliminated from the original condition equations, but instead are carried through the solution as unknowns along with the unknown DLT coefficients.

As a result of the determination afforded by convergent photographs, a complete calibration of inner cone can be recovered. Since x_{p}, y_{p}, c are incorporated in DLT, the model makes provision for coefficients k_1, k_2, k_3 for Gaussian symmetric radial distortion and for coefficients p_1, p_2 for decentring distortion, both assumed constant over all photographs.

Geodetic measurements, in our case ground distances and microcomparator observations, provide excellent constraints to the general adjustment. For calibration and stereotriangulation, the all constraint equations are interpreted as additional observations with a suitable high weight which enforces zero variances.

The complete mathematical model is obtained by combining the fundamental projective equations (1), the constraint equations (2), and the constraint equations arising from distance measurements. The system of normal equations formed from consistently weighted independent observations and operating on a common parametric vector are additive. This property of independent systems of normal equations cannot be applied directly because each operates on a different vector. This difficulty is easily

overcome by a process of zero augmentation to force each system to operate on the total parametric vector to be recovered.

A computer program, DEFORM, was developed by the author and its formulation is based on the principle of the observation equations as described in the above paragraph.

Test and Practical Application
The DEFORM program was tested with fictitious data and under operational conditions. The primary purpose of these series of tests was to evaluate the computational accuracy of the mathematical model. Hence unperturbed image coordinates were used as input data in all tests and they were assigned a standard deviation of ±2 micrometres. Unperturbed data were also used for distance measurements, but they were assumed to have first-order accuracy and they were weighted correspondingly. Because the exact object coordinates of all control points were known, the accuracy of the DEFORM solution can be evaluated directly by comparing the computed point coordinates with the corresponding known values.

Various configurations of controls were used for the photography which was generated at various scales. The results show that the DEFORM solution has computational accuracy and sensitivity relative to the accuracy of the measured data.

The standard errors of the computed coordinates are involved between ±0.09 mm and ±0.16 mm in X, Y coordinates, and ±0.07 mm to ±0.10 mm in Z coordinate. A rejection technique was established and was proved quite adequate.

The above method was performed under operational conditions to determine the deformations of the model of a concrete bridge. Three camera sites were established at a distance of 12 m in front of the model. The photos were measured on a linear spectrophotometric comparator and the photocoordinates with their weight matrices were obtained by a least-squares solution using a trilaterated photocoordinates method. The entire set of observations for 72 points was processed through a simultaneous least-squares adjustment. Four iterations were necessary before the adjustment converged to a sufficiently stable solution. The root mean square error of the residuals from the final solution was ±2.3 micrometres and the residual vectors were random from point to point. To determine the deflections during the experiment, a similarity transformation of the coordinates from each loading step to the initial position was applied.

Conclusions and Recommendations
It was proved theoretically and also by the above practical application that DEFORM solution combined with precise surveying and comparator distance measurements and processed by free net adjustment technique is an extremely powerful tool which can be applied to a wide range of problems. The obvious advantage of this method is that only a few control data are necessary to compute the DLT coefficients. No internal camera or external exposure station parameters are needed as input data.

From the comparison between the photogrammetric and the mechanical comparator measurements, it can be concluded that the proposed method is accurate within 0.2 mm. The analysis of the results indicates that a large format camera (180 x 130 mm) with longer focal length (f=400 mm) gives better accuracy of coordinates in all directions. The coordinates of points external to the model can readily be determined with the same accuracy as the points situated on the model.

Based on the general linear transformation, an exact solution for orientation and calibration is derived by a simultaneous least-squares adjustment of projective transformation parameters and of control point coordinates.

COMPILATION OF A NEW RECENT VERTICAL CRUSTAL MOVEMENTS MAP FOR CANADA

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The first map of recent vertical crustal movements for Canada was compiled nine years ago by Vaníček and Nagy [1980]. Three kinds of data had been used for the first compilation: relevelled segments of first- and second-order levelling with practically no U.S. data included, uplift rates as determined from tide-gauge records, and tilts across the Great Lakes as determined from pairs of lake level records.

The technique used for the compilation was dictated by the character of the available data: both the relevelled segments and lake level gauges give information only about tilt, while tide gauges register point vertical rates. A mathematical model for the postulated spatially continuous vertical movement rates had to be adopted and an algebraic surface was selected. Because of the size of the country and the irregular density of data coverage, it was necessary to divide the country into several zones. The solution in each zone was sought separately while a certain amount of overlap between the zones was used to give a smooth transition between zones.

The resulting maps reflect the broad features of crustal dynamics in Canada as known from a variety of geodetic and geophysical studies. Spot checks with carbon dated uplift rates valid for geological time periods show an excellent point agreement in Eastern Canada. The internal estimates of relative accuracy seem to be quite realistic.

Two aspects of the maps are mostly criticized: the fact that no correction for refraction effects had been applied to the levelling data, and that the lack of U.S. data makes for a poor fit with some American studies covering regions adjacent to Canada.

To eliminate these problems and to take advantage of the vast increase in the amount of data now available, the compilation of a new map has been commissioned of the University of New Brunswick by the Surveys and Mapping Division of the Canadian federal government. In addition to the increase in the spatial data coverage, the time coverage will now be significantly expanded. The existence of a complete vertical data base is going to make data selection and correction a much less arduous task. An inclusion of levelling data from the northern most parts of the U.S. is also envisaged.

Concerning the technique to be used, several changes to the originally used approach are planned. It should be possible to use surfaces of higher order, now supported by denser spatial distribution of relevelled segments. Subject to confirmation through further studies, the tide-gauge indicated uplift rates will be differenced to increase the signal-to-noise ratio by exploiting the high degree of spatial coherence of adjacent tide gauge records. Thus tide-gauge pair tilts will be used in the model evaluation instead of

individual gauge uplift rates. A rigorous, geographically sequential treatment of levelling data will be attempted to replace the heuristic treatment of zone overlaps.

Finally, a serious effort will be made to model all the predictable systematic effects acting on levelling. These are: thermal rod expansion, rod mis-graduation, residual refraction, and magnetic effect on automatic compensator. They will be expressed in the form of corrective surfaces to be added to the computed uplift surface.

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DISPLACEMENTS VERSUS STRAIN

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When one is interested in measuring/monitoring earth's deformations on either a local scale (engineering applications), or regional and global scales (geophysical applications), one automatically rules out a whole host of geodetic techniques which do not have sufficient accuracy. The usable techniques, with accuracies on at least the centimetre level, are all — with the notable exception of satellite laser ranging (SLR) — of the 'relative positioning' variety. These usable techniques include all the useful terrestrial measurements (levelled height differences and distances, angles of various guises) as well as very long baseline interferometry (VLBI), relative (differential) mode of GPS positioning, and relative mode of SLR positioning. It is our thesis that consequently no geodetic technique, again with the exception of SLR point positioning, can be used to compute displacements.

Let us discuss this thesis using a simple example of a completely reobserved network. For this example, the system of observation equations reads:

$$\underline{\Delta l} - \underline{A}\,\underline{\Delta x} \tag{1}$$

where $\underline{\Delta l}$ is the vector of observation differences, $\underline{A}$ is the design matrix identical to the design matrix for network point positions, and $\underline{\Delta x}$ is the vector of displacements (in one, two, or three dimensions). As we know from scores of studies, the defect of $\underline{A}$ (def $\underline{A}$) equals to the number of indeterminacies in network point positions, e.g., def $\underline{A}$ = 2 for a two-dimensional network with known orientation, def $\underline{A}$ = 3 for a two-dimensional network without an orientation. More complicated cases of incompletely reobserved networks display the same behaviour. The incompleteness only obscures the issue of basic indeterminacies but contributes nothing to its solution.

Attempting now to solve equation (1) using the least-squares technique (preferred for statistical reasons) one obtains

$$\underline{N}\underline{\Delta\hat{x}} = \underline{A}^{\mathrm{T}}\,\underline{C}_{\Delta l}^{-1}\,\underline{A}\,\underline{\Delta\hat{x}} = \underline{A}^{\mathrm{T}}\,\underline{C}_{\Delta l}^{-1}\,\underline{\Delta l} \;, \tag{2}$$

where

$$\underline{C}_{\Delta l} = \underline{C}_{l_1} + \underline{C}_{l_2} - 2\underline{C}_{l_1 l_2} \;. \tag{3}$$

(We note 'en passant' that the side issue of the temporal cross-covariance matrix, $\underline{C}_{l_1 l_2}$, is an important one and should be seriously addressed.) It is again well known that

$$\text{def } \underline{N} = \text{def } \underline{A} \ , \tag{4}$$

and $\underline{N}$ cannot be inverted in the ordinary sense to yield a unique solution $\underline{\Delta \hat{x}}$.

In practice, a justifiable **particular** solution can be obtained by imposing the minimum number of constraints which equals to def $\underline{A}$. (The imposition of more than the minimum number of constraints is ordinarily not justifiable at all.) Infinitely many such particular solutions may be produced which, in the absence of some additional information external to our problem, will all be equally acceptable. One such minimum constraint solution would be, for instance, to hold one point 'fixed,' i.e., declare one point to have no displacement. Clearly, any such particular solution gives only displacements relative to the 'fixed' point and says nothing about the displacement of the 'fixed' point itself. A particular solution that uses any of the generalized inversion procedures (minimizing the norm of displacements, minimizing the estimated errors of displacements, etc.) is also justifiable on the grounds of being equivalent to any of the minimum constraint particular solutions. However, a case cannot be made that in the absence of additional information, g-inverse solutions should be preferred. Thus only particular relative displacements can be obtained from geodetic observations.

Under these circumstances, it appears to us that strain, as the tensor gradient of the displacement field, is a superior measure of deformation to compute. It can be easily shown that strain is completely blind to translational indeterminacies. Moreover, the symmetrical part of the tensor gradient, called usually the strain tensor, is blind also to orientation indeterminacies. Put more simply, the strain tensor is the only measure of deformation fully determinable from geodetic observations.