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Existing Resources, Standards, and Procedures for Precise Monitoring and Analysis of Structural Deformations Volume II - Appendices

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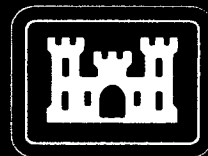
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13. ABSTRACT (Maximum 200 words)

This report is based on a review of literature, international reports, and results of a questionnaire sent to national representatives of about 70 countries to the International Commission on Large Dams. The study focused on monitoring and analysis of deformations of large dams. The main conclusions of the study are: (1) There are no available standards and specifications in any of the reviewed countries which could be recommended for direct adaptation to dam deformation monitoring in the United States; (2) With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing; (3) Over the past 10 years there has been significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. However, due to a lack of an interdisciplinary cooperation and insufficient exchange of information, the developments have not yet been widely adapted in practice; and (4) Generally, the overall qualifications and educational background of the personnel placed in charge of monitoring surveys in the U.S. seem to be inadequate, particularly in the areas of data processing and analyses.

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APPENDICES

DEFORMATION MONITORING, ANALYSIS, AND PREDICTION - STATUS REPORT

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DEFORMATION MONITORING, ANALYSIS, AND PREDICTION — STATUS REPORT

LA SURVEILLANCE, L'ANALYSE, ET LA PRÉDICTION DE DÉFORMATIONS — UN RAPPORT DE STATUS

DEFORMATIONSÜBERWACHUNG, -ANALYSE UND -PRÄDIKTION — EIN STATUSREPORT

A. Chrzanowski and Y.Q. Chen — Canada

SUMMARY

New space techniques, coordinating systems with electronic theodolites, as well as high precision geotechnical and other non-geodetic instrumentation, close the gap between the regional and very local deformation surveys. Integration of various monitoring techniques calls for an interdisciplinary approach to the deformation surveys and analysis. All basic problems of geometrical analysis of deformation surveys in the static and kinematic modes have been solved. Additional research is needed for a proper design of the integrated monitoring surveys, for monitoring and analysis of dynamic deformations, and for physical interpretation and prediction of deformations.

RÉSUMÉ

Le désaccord entre les études de déformation régionale et de déformation locale se diminue grâce à des nouvelles techniques spatiales, à des systèmes de coordonation et de mesurage électroniques de distance avec les théodolites électroniques, ainsi qu'à l'instrumentation géotechnique (très précise) et non-géotechnique. L'intégration de plusieurs techniques de surveillance demande une approche interdisciplinaire à l'étude et l'analyse de la déformation. Tous les problèmes fondamentaux de l'analyse géométrique des études de déformation kinétique et statique furent résolus. Il faut de la recherche supplémentaire dans les domaines de la conception des études de surveillance intégrées, de la surveillance et l'analyse de déformations dynamiques, ainsi que de l'interprétation physique et la prédiction de déformations.

ZUSAMMENFASUNG

Neue Raummeßverfahren, Koordinierungssysteme mit elektronischen Theodoliten, sowie präzise geotechnische und andere nicht-geodätische Instrumente schließen die Lücke zwischen regionalen und sehr lokalen Deformationsvermessungen. Die Integration verschiedenartiger Über- wachungstechnologien verlangt einen interdisziplinären Zugang zu Deformations- messung und -analyse. Alle prinzipiellen Probleme der geometrischen Analyse von Deformationsmessungen sowohl im statischen als auch im kinematischen Modus sind als gelöst zu betrachten. Weitere Forschung ist notwendig sowohl zur Entwicklung eines geeigneten Designs für integrierte Deformationsmessungen und -analysen als auch in der physikalischen Interpretation und Prädiktion von Deformationen.

1. INTRODUCTION

Study Group C, dealing with the problems of monitoring and analysis of deformation surveys, has always been one of the most vital groups of FIG Commission 6. This is not surprising because deformation surveys have always been of major importance in engineering surveys. Due to the ever growing technological progress in all fields of engineering and, connected with it, the growing demand for higher accuracy, efficiency, and sophistication of deformation measurements, survey engineers must continuously search for new monitoring techniques and refine their methods of deformation analysis. Study Group C has played a very important role in providing a forum for the exchange of information in the new developments by organizing technical sessions during FIG Congresses and, more importantly, by organizing specialized international symposia on deformation surveys, such as, in 1975 in Krakow, Poland; in 1978 in Bonn, West Germany; in 1982 in Budapest, Hungary; in 1985 in Katowice, Poland; and in 1988 in Fredericton, Canada. The next (6th) symposium is to be held in Hannover, West Germany, in 1991. The published proceedings of these symposia provide an enormous wealth of information on the development of new techniques and new methods in monitoring and analysis of deformations. One should commend the organizers of the symposia and the chairmen of Commission 6, most of whom have personally contributed to the generally excellent contents of the technical sessions on deformation surveys; namely, Professor T. Lazzarini of Poland, Professor L. Hallermann of West Germany, Dr. A. Detreköi of Hungary, and Dr. G. Milev of Bulgaria, just to mention a few from the long list of many important contributors. Special appreciation is owed to Dr. A. Platek of Poland, who until 1988 led Study Group C, for his efforts and many technical contributions which have laid a foundation for the present approaches to integrated deformation analysis.(e.g. Platek [1974]).

Since 1978, the main activity of Group C has concentrated on the geometrical analysis of deformation surveys within an *ad hoc* Committee on Deformation Analysis which was created at the 2nd symposium in Bonn. Therefore, in this brief report on the status of deformation surveys, emphasis is placed on the analysis, including physical interpretation, rather than on the status of the monitoring techniques which are reviewed only very briefly in Section 3 below.

2. INTERDISCIPLINARY STATUS OF DEFORMATION SURVEYS

One should point out that FIG is not the only international organization concerned with deformation surveys. For instance, Commission 4 of the International Society for Mine Surveying (ISM) deals with deformation studies in mining areas. At the last congress of ISM in Leningrad in 1988, more than half of the several hundred presented papers dealt with monitoring techniques, analysis, and interpretation of ground movements and related deformations. The International Commission on Recent Crustal Movements of the International Association of Geodesy (IAG) is very active. During their last symposium in Edinburgh in 1989, several sessions dealt with the use of new techniques, particularly the Global Positioning System (GPS) and its integration with terrestrial geodetic surveys in the regional and continental studies of the earth's crustal movements. Commission 5 of the International Society for Photogrammetry and Remote Sensing (ISPRS) deals with aspects of photogrammetric deformation surveys and uses of new techniques such as solid

state (CCD) cameras and integrated measurements. Besides the surveying and geodetic organizations, practically all engineering organizations which deal with geomechanics and structural engineering, such as the International Society of Rock Mechanics, the International Commission on Large Dams (ICOLD), the International Society of Soil Mechanics and Foundation Engineering, and several others, have working groups which deal with deformation studies.

There is little or no exchange of information and, certainly, there is no coordination of the work performed by the individual organizations in the field of deformation surveys. This leads to an unnecessary duplication of effort and outdated "discoveries" (development of "new" methods or concepts which have already been used by other specialists for many years). Hopefully, the recently organized International Union of Surveying and Mapping (IUSM) will take care of the coordination of work, at least, among the surveying and geodetic organizations. However, more work is required to create a truly interdisciplinary approach to the design, analysis, and interpretation of deformation surveys. The need for an interdisciplinary approach and for the integration of various techniques, such as those used by surveyors and geodesists with those used by geotechnical and structural engineers, has long been recognized by many authors (e.g. Janusz [1971]; Lazzarini and others [1977]; Platek [1974]; Chrzanowski [1986]). The first major step in correcting the situation, however, was not made until 1988 when the aforementioned 5th FIG Symposium on Deformation Measurements was organized together with the 5th Canadian Symposium on Mining Surveying and Rock Deformation Measurements as an interdisciplinary conference on deformation monitoring, analysis, and prediction. The main goal of the conference was 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. Among the resolutions of the conference we find [Chrzanowski and Wells, 1988, p. 614]:

> Recognizing that the 5th International (FIG) Symposium on Deformation Measurements and the 5th Canadian Symposium on Mining Surveying and Rock Deformation Measurements brought together experts from different fields with the common concern regarding deformations, be it resolved that FIG seek better communication with:
> the International Society of Rock Mechanics,
> the International Society of Soil Mechanics and Foundation Engineering,
> the International Society of Mining Surveying,
> the International Congress on Large Dams,
> the International Union of Geodesy and Geophysics,
> with the aim of holding a truly interdisciplinary symposium on deformations and promoting local communication on the subject matter.

Thus, the organizers of the 6th FIG Symposium (Hannover 1991) have a task to organize it in a truly interdisciplinary spirit.

3. STATUS OF MONITORING TECHNIQUES

As far as new survey techniques are concerned, the technological progress of the last few years in electronics and in automation of measuring systems has gone beyond the control of surveyors who are becoming simply the users of the new techniques, learning about their potentials and possible applications. The impact

of the surveying community on the development of the new techniques has been minimal and has been reduced to only the adaptation of existing technology to particular applications. As far as the instrumental precision, automation in the measuring procedures (robotics), and continuous and remotely controlled data acquisition are concerned, the only limitations are economical not technical. At a cost, one may achieve almost any, practically needed instrumental resolution and precision, full automation, and almost real-time data processing.

As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrumental errors but the environmental influences. Atmospheric refraction (tropospheric and ionospheric) is still the number one factor in any electromagnetic and electro-optical geodetic surveys despite the application of dual frequency measuring systems which are still too expensive for engineering applications. Thermal influences, affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of the stations involved in the monitoring scheme, are another environmental source of errors. In long-term measurements, the instrumental repeatability (precision) may be affected by aging of the electronic components and resultant drift of the instrumental readout. Proper calibration techniques may reduce the last two effects.

It is not a purpose of this report to discuss and list all the new types and models of instruments used in monitoring surveys because there is such a vast technical and commercial literature available on the subject that all participants of this congress certainly are familiar with all the new developments. Only general comments on the different techniques are given below.

Traditionally, the monitoring techniques have been categorized into three broad groups, according to the main groups of professionals using the technique:
*   geodetic surveying (space and terrestrial) and photogrammetric measurements (aerial and close range terrestrial)
*   geotechnical and structural engineering measuring techniques (direct measurements of deformations using strainmeters, various types of extensometers, suspended and inverted pendula, tiltmeters, inclinometers, various deformeters, etc.)
*   special non-geodetic (industrial metrology) techniques of high precision (interferometric measurements, hydrostatic levelling, precision alignment, new holographic methods, etc.)

The above classification has also been connected with the historically different purposes and different accuracies produced by the three groups of instrumentation in the past. Even within the geodetic surveying methods, the space techniques used to be classified separately from the terrestrial and photogrammetric techniques because to their different applications. Due to the technological progress of the last decade, however, this differentiation of techniques, at least from the accuracy point of view, becomes artificial. For example, the recently developed 3-D coordinating systems with electronic theodolites which employ the traditional terrestrial surveying methodology, currently replace the opto-mechanical techniques of the highest precision in industrial metrology measurements [Wilkins et al., 1988]. Also, close-range, real-time photogrammetric techniques with solid state cameras (CCD cameras) compete successfully with traditional industrial metrology techniques [Gruen and Kahmen, 1989]. New space techniques like the Global Positioning System (GPS), even before the system is fully operational, give

the accuracy which is compatible with precision terrestrial surveys over medium distances (several kilometres), and much higher relative accuracy over long ranges. For instance, GPS measurements have been successfully combined with first-order geodetic levelling in ground subsidence studies in oil fields in Venezuela [Leal, 1989]. Thus, the technological gap between the regional and very local deformation measurements is narrowing and the array of different types of instruments available for any type of deformation studies has been significantly broaden. Surveyors should more than ever grasp the advantage of the variety of instrumentation available to design optimal monitoring schemes.

There are still some limitations to geodetic surveying methods in comparison with geotechnical techniques. Geodetic and photogrammetric surveys are limited only to open areas and cannot be used in detecting local deformations inside the material of the deformable body. Deformations in foundations of large engineering structures and relative movements of different soil and rock layers in slope stability studies are examples in which the geotechnical type of instruments must be used [Chrzanowski, 1986]. An excellent review of the geotechnical instrumentation is given by Dunnicliff [1988].

Geotechnical instrumentation can be easily adapted for continuous and telemetric data acquisition with an instantaneous display of deformations which is very advantageous in comparison with the slow, labour intensive, terrestrial surveys. The recently developed survey robots with motorized electronic theodolites and built-in vision system (e.g., Wild ATMS and Geodimeter servo-robot 140SR [Gruen and Kahmen, 1989]) cannot compete with the geotechnical instrumentation which does not require intervisibility between the stations and is operational practically in any environmental conditions. The geotechnical and particularly special non-geodetic instruments, such as invar wire extensometers, laser interferometers, diffraction aligning equipment, and laser holography [Takemoto, 1989], offer accuracies in the order of a few hundredths of a millimetre in localized measurements.

For the above reasons, the trend in the last few years has been to adopt geotechnical and special non-geodetic instrumentation for deformation surveys not only in the areas which are inaccessible for geodetic and photogrammetric surveys but also in studies where geodetic monitoring networks could have been used. If it continues, this tendency may lead to an unhealthy situation in which the usefulness of the geodetic and photogrammetric surveys will become underestimated by those who favour the geotechnical instrumentation.

Here, one should point out to those who advocate the use of geotechnical methods that the geotechnical instrumentation also has weak points despite the aforementioned and indisputable advantages. First of all, the measurements are very localized and they may be strongly affected by local disturbances (noise) which do not represent the actual deformations. Since the local observables are very often not geometrically connected with observables at other monitoring stations, any global trend analysis of the deformations is much more difficult than in the case of geodetic surveys unless the observing stations are very densely spaced. The geomechanical, rock mechanics, and structural engineers, who are the main users of the geotechnical instruments, are usually less acquainted than survey engineers and geodesists with the evaluation of the measuring data and

calibration of measuring instruments. On the other hand, most surveyors demonstrate complete ignorance in the use of other than geodetic techniques.

One could cite many examples of deformation analyses being conducted on the same object separately by both survey engineers and other engineers using only their own survey data obtained with their own instrumentation. Both groups have not fully recognized the advantages and disadvantages of various measuring techniques and, perhaps, they have not known how to combine geodetic and geotechnical results into an integrated analysis, which is briefly discussed in the next section.

4. ANALYSIS OF DEFORMATION SURVEYS

The analysis of deformation surveys includes:
* geometrical analysis which describes the geometrical status of the deformable body, its change in shape and dimensions, as well as rigid body movements (i.e. translations and rotations) of the whole deformable body with respect to a stable reference frame or of a block of the body with respect to other blocks, and
* physical interpretation which explains the relationship between the causative effects (external and internal forces) and deformations.

Until very recently, the surveying community concentrated mainly on geometrical analysis, particularly or the analysis of geodetic monitoring networks. At the last FIG Congress in Toronto, the aforementioned FIG ad hoc Committee on Deformation Analysis presented a detailed report [Chrzanowski and Chen, 1986] on the progress in geometrical analysis, and the members concluded that all basic problems of geometrical analysis had been solved and no further international activity in that area was needed. This was reflected in one of the resolutions of the 5th FIG Symposium in 1988 which reads [Chrzanowski and Wells, 1988, p. 614]:

Considering the history and actions of previous international symposia on deformation measurements and the work of the ad hoc committee that this symposium comes to the conclusion and complete agreement that the methods of geometric deformation analysis are well understood and have reached the standard where they should be made applicable to other engineering disciplines.

At the same time, the Committee decided to become more active in aspects of physical interpretation, particularly in the aspects of an optimal combination of geometrical analysis with the physical interpretation for the purpose of a better understanding of the mechanism of deformations and a better design of monitoring surveys. Extensive reviews of the up-to-date methods used in physical interpretation have been given in Chen and Chrzanowski [1986] and Chrzanowski et al. [1990]. A brief general review of the status of the deformation analysis is given below.

4.1. Geometrical Analysis of Deformation Surveys

When analysing geodetic monitoring surveys, the monitoring networks are classified as:
* absolute networks, comprised of reference points established outside the deformable body and of object points, and

- relative networks, in which all the survey stations and observed points are located on or within the investigated object.

The main problem in geometrical analysis of absolute networks is the identification of unstable reference points, while in the analysis of relative networks, determination of the deformation pattern in space and in time domains is a major concern.

The identification of unstable reference points, so important in defining the datum for the displacement calculations of the object points, has been a subject of intensive investigation over the past several decades. According to Lazzarini and others [1977], Zölly and Lang of Switzerland were among the first, in the 1920s, to advocate the inspection of differences of raw observations rather than differences of datum dependent coordinates in identifying unstable reference points. In the 1960s and 1970s, Lazzarini developed the whole school of thought in deformation analysis based on the estimation of the displacements directly from differences of observations rather then from single epoch adjusted coordinates. He developed a method [Lazzarini, 1975] to identify unstable reference points based on a minimization of the displacements through a transformation of the displacements. In the 1970s, another school of thought was initiated by Pelzer [1971; 1974] based on statistical testing (congruency test) of quadratic forms of the residuals of observations obtained from single epoch and simultaneous two-epoch adjustments of the monitoring network. The two schools of thought have led to the development (in most cases in an independent way) of several methods of the identification of unstable reference points which may be categorized into two groups:

- those based on the congruency test [Pelzer 1974; van Mierlo, 1978; Niemeier, 1981; Heck, 1983; Gründig et al., 1985], and
- those based on defining the datum which is robust to unstable reference points.

In the first case, a failure in the congruency test is followed by a search for the new congruency test which has the minimum statistic. The test statistics are calculated by in turn removing, one by one, points from the set of reference points until all the unstable points are identified. In the second case, one of the developed strategies is based on an iterative weighted similarity transformation of the displacements until the first norm of the vector of the displacements of the reference points is minimized [Chen, 1983; Chrzanowski et al., 1986b]. A similar method is used by Caspary [1984] by minimizing the summation of the lengths of the displacements for all the reference points. All the above methods give compatible results, and one may say that the problem of identifying unstable points has been solved and no more research is needed in this area.

Analysis of relative monitoring networks is more complicated. The deformation pattern and general trend of the deformation in the space domain may be identified from the relative displacements through the aforementioned iterative weighted transformation of the displacements. Welsch [1983] proposes to identify the trend by dividing the deformable body into smaller elements and by assuming a linear deformation in each element. The trend is then deduced from the differences in the deformation parameters (strains) in each element. The deformation trend in the time domain is obtained from the analysis of time series of observations for each observable [Prescott et al., 1981; Chrzanowski et al., 1989a]. Some authors assume the deformation between two epochs of observations to be linear in time and estimate just the displacement rate for each surveyed point (e.g., Papo and Perelmuter [1982]).

The deformation trend is used in selecting a deformation model which would describe the actual geometrical status of the deformed object. As accepted by the FIG *ad hoc* Committee, the deformation of a 3-D body is fully described if nine deformation parameters (six strain components and three differential rotation components) can be determined at each point of the object. These parameters can be calculated from well-known strain-displacement relationships if a displacement function representing the deformation of the object is known. In addition, components of relative rigid body motion between blocks should also be determined if discontinuities exist in the body. Thus, as suggested by some earlier authors (e.g. Czaja [1971]; Platek [1974]) and as refined and generalized within the activity of the FIG *ad hoc* Committee [Chrzanowski et al., 1982; Chen, 1983], the main task of geometrical analysis is to find a deformation model, expressed in terms of a displacement function, which characterizes the deformation in space and time. Since, in practice, deformation surveys involve only discrete points, the function describing the displacement field must be approximated through some selected model which fits into the observation data (e.g., the relative displacements in the case of the relative geodetic network) in the best possible way. If no a priori information (the deformation trend or a prediction model) of the expected deformation is available, a process based on fitting a general polynomial with an iterative elimination of the insignificant coefficients has been proposed by Chrzanowski et al. [1983] and Chen [1983] until the *best* (statistically) model is obtained. The least-squares criterion is commonly used for the estimation of the deformation parameters. The technique of robust estimation, however, which is less sensitive to model errors, has also been proposed [Caspary, 1988].

Recently, survey engineers have become more involved in the analyses of integrated monitoring schemes in which geodetic (space and terrestrial) and photogrammetric surveys are combined with geotechnical and other non-geodetic measurements (direct measurements of strain, tilt, inclination, etc.). The process of geometrical analyses of integrated surveys is basically the same as outlined above for geodetic monitoring networks as long as the approximate coordinates of all survey stations are known. The displacement field, in this case, is determined through fitting a selected deformation model (expressed in terms of displacement functions) directly to the raw observations. Functional relationships between any type of observables and displacements of involved survey stations are given in Chen [1983], Secord [1985], and Chrzanowski et al. [1986b].

Chrzanowski et al. [1989a] and Chrzanowski et al. [1986a] give two examples of the integrated geometrical analysis, which utilize the approach given above. The first example describes a complex integrated analysis of a power dam involving geodetic and various geotechnical measurements (tape and borehole rod extensometers, pendula, joint meters). The second example, involving geodetic, photogrammetric, and electronic tiltmeter measurements, is related to a slope stability study in a mining area. Both examples indicate that, practically, all the basic problems of integrated geometrical analyses have been solved. The only aspects remaining for possible further research are the choice of the statistically best deformation model, if more than one model seems to fit well into the observed deformations, and modelling of systematic errors of deformation measurements, if the systematic errors differ between survey campaigns. For instance, when combining GPS with levelling surveys in ground subsidence studies [Chrzanowski et al., 1988; Steinberg and Papo, 1988], the GPS-derived height differences may

be contaminated by some sources of bias which are a function of the geometry of the satellites. The geometry and the resulting bias may be different in each survey campaign. Thus, the systematic errors do not cancel out in the subsidence calculations. If the bias is included in the deformation model as an unknown parameter, however, its value and its significance may be estimated through the integrated analysis because levelling and GPS measurements are not affected by the same systematic errors [Chrzanowski et al., 1989b].

Several software packages for geometrical analysis have recently been developed, for example, DEFNAN [Chrzanowski et al., 1986b], PANDA [Niemeier and Tengen, 1988], and LOCAL [Gründig et al., 1985]. Some of them (e.g., DEFNAN) are applicable to the integrated analysis of any type of deformations, while others are limited to the analysis of reference geodetic networks only.

Generally speaking, all problems of static and kinematic geometrical analyses of deformation surveys have been solved. Geometrical analysis of dynamic deformations, e.g., vibration of machines, deformation of bridges due to dynamic loads, and dynamic behaviour of tall buildings under wind loading, has not been a subject of investigation of the FIG *ad hoc* Committee, and it may still require additional research. The main interest in data analysis of dynamic deformations is to define the frequencies of the oscillations and the corresponding amplitudes of the deformation. The Fourier transformation technique may be employed in these cases. Maurer et al. [1988] give some practical examples.

4.2. Physical Interpretation and Prediction of Deformations

Physical interpretation of deformations is performed to determine the physical status of the deformable body, the state of internal stresses and, generally, the load-deformation relationship. Once the relationship is established, the results of the physical interpretation may be used for a development of prediction models. Through a comparison of predicted deformations with the results of the geometrical analysis of the actual deformations, a better understanding of the mechanism of the deformations is achieved. Thus, the survey engineers, with the results of their integrated geometrical analysis, may significantly contribute to the physical interpretation. On the other hand, the prediction models which, in most cases, are developed by other specialists, supply information to surveyors on the expected deformation, facilitating the design of the monitoring scheme as well as the selection of the deformation model in the geometrical analysis. Unfortunately, the above scenario of the truly interdisciplinary approach to the design and analysis of deformation surveys has not yet been fully implemented in practice. The reasons are an inadequate understanding by surveyors of the methods of the physical interpretation and inadequate familiarization of other specialists with the comparatively new methods of geometrical analysis of integrated surveys. Within the last few years, one can see progress in the right direction, at least within the survey community, with more papers on the physical aspects of deformations being presented at the surveying and geodetic meetings and conferences (e.g., Chen and Chrzanowski [1986]; Teskey [1986]; Milev [1988]; Szostak-Chrzanowski [1988]; Chrzanowski and Chen [1990]). Only a few comments on the methods of physical interpretation are given in this presentation.

The determination of the load-deformation relationship may be obtained by using either of the two methods:

- a statistical method (regression analysis), which analyses the correlations between observed deformations and observed loads (external and internal causes producing the deformation) and
- a deterministic method, which utilizes information on the loads, properties of the materials, and physical laws governing the stress-strain relationship.

The statistical method is of an a posteriori nature because it utilizes the past data through a regressive analysis in establishing an empirical prediction model of deformations as a function of loads. The deterministic method is of an a priori (design) nature. Due to difficulties in solving usually complex differential equations, which describe the relationship between the external forces and internal stresses, the deterministic method is based on approximate solutions using numerical methods. The finite element and, to a lesser extent, boundary element methods are the most popular and, perhaps, the most powerful tools in numerical analysis.

The statistical method has been used in engineering, for instance, in dam deformation, for several decades, while the deterministic method could have been employed only after large computers became available. Therefore, there is still an open field for research in the applications of the deterministic method.

In a comparison of the two methods of physical interpretation, each has its advantages and drawbacks. The statistical method does not require knowledge of the material properties of the deformable body which, sometimes, may be difficult to determine. From the point of view of prediction, good agreement between the predicted deformations and the observed ones is usually obtained if a long series of observations of both the deformations and the causative effects are available. A separation of different effects is sometimes difficult in the statistical method when the different causative factors are strongly correlated. On the other hand, the deterministic method does not require any observations of the actual deformations. Therefore, it is the only method for predicting deformations at the design stage of a new engineering project. Due to many uncertainties in deterministic modelling, however, including imperfect knowledge of the material properties, wrong modelling of the behaviour of the material, and approximation in calculation, the accuracy of predicted deformations using the deterministic method is usually low, particularly in geotechnical projects dealing with non-linear behaviour of soil and rocks. Therefore, a combination of both methods is recommended. For instance, various empirical prediction models of ground subsidences in mining areas have been known since the 1920s. They could be successfully used, however, only in cases of a simple geometry of the mining extraction and only in the same mining areas and in the same geological conditions from which the observation data for the statistical modelling was available. Recently, a universal and successful method for ground subsidence prediction has been developed [Szostak-Chrzanowski, 1988] which is based on an iterative non-linear elastic finite element analysis. The method has not been developed as a purely deterministic method, however, because it uses some empirically determined general properties of rocks treated as non-tension material.

Many examples of a simplified combination of two methods of interpretation concerning dam deformations have been given in ENEL [1980]. They calibrated the constants of the material properties using the discrepancies between the displacements of a point at different epochs calculated from FEM and the

measured ones. One must be aware, however, that if the real discrepancy comes from other effects than the incorrect values of the constants, the model may be significantly distorted.

Recently, Chrzanowski et al. [1990] have developed a concept of global integration, where all three — the geometrical analysis of deformations and both methods of physical interpretation — are combined. Using the concept, deformation modelling and understanding of the deformation mechanism can be greatly enhanced. The research is in progress.

5. CONCLUSIONS

Summarizing this report on the status of deformation monitoring, analysis, and prediction, a general conclusion is that within the activity of the FIG group 6C over the last few years, most attention has been paid to and most significant progress has been made in the geometrical analysis of deformation surveys. Presently, attention is being shifted towards the physical interpretation, towards integration of geometrical analysis with the methods of physical interpretation, and towards an interdisciplinary approach to the design and analysis of deformation surveys. far as future activity is concerned, more research and international cooperatio still needed in the areas of:

* optimal design of the integrated monitoring schemes,
* adaptation of new technologies to integrated monitoring surveys, particularly to monitoring dynamic deformations,
* development of procedures and techniques for calibration of geotechnical and special non-geodetic instrumentation,
* further developments in the integration of geometrical analysis with the physical interpretation methods in a truly integrated and interdisciplinary analysis of deformation surveys.

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TERRESTRIAL SURVEY METHODS FOR PRECISION DEFORMATION MEASUREMENTS

TERRESTRIAL SURVEY METHODS
FOR
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Abstract

Modern electronic and sophisticated optical and mechanical instrumentation provide the survey engineer with enhanced accuracy, ease of operation, and facility in data collection. Submillimetre and subsecond accuracies have become a matter of routine. Measurements can now be done virtually at the press of a single button or automatically with the data being similarly stored and eventually processed. Nonetheless, most sources of error still persist and there must be an awareness of these errors in order to fully exploit modern equipment and to design an effective monitoring scheme. Even with modern technology, the limitation imposed by the environment, especially through refraction, dominates.

In the context of comparing repeated campaigns of observations, this paper pursues deformation measurements made by traditional geodetic instrumentation and the considerations that must be made in such adaptation.

1. Introduction

In the monitoring of deformations, terrestrial survey methods involve the measurement of geometric angular and linear relationships by either optical, mechanical, or electronic means. Many of these methods are the adaptation of conventional geodetic positioning methods and instrumentation but with some subtle differences in the philosophy of measurement and with stretching instrument performance to the limit of accuracy and precision.

Although the trend in modern technology is toward the utilization of elaborate and sophisticated extra-terrestrial positioning techniques and hence their repetition in the monitoring of deformations, the more conventional terrestrial techniques are still viable in isolated monitoring schemes, especially for economy and relative accuracy.

Modern electronic instrumentation provides enhanced accuracy, ease of operation, and facility in data collection; however, the main obstacle to precision and even higher accuracy is still the effect of the measuring environment, the earth's atmosphere. A change in the velocity of electromagnetic radiation, due to temperature gradients across and along the path, results in curvature of the line of sight when measuring angles or when levelling and in change in magnitude (scale) in electro-optical distance measurement (EODM).

The references and suggested readings cover well most of the aspects of the measurement of angular relationships (horizontal angles or directions, azimuths, and vertical or zenith angles) and of linear relationships (slope or horizontal distances and vertical height differences). The most salient of these will be presented here, including general considerations in using geodetic methods for monitoring as opposed to positioning and in the concern for station mark or pillar stability and specific considerations when measuring horizontal and vertical angles; when measuring distances by optical, mechanical, or electro-optical means; and when measuring height differences by geometric levelling or by trigonometric height traversing. Also discussed is the concept of integrated monitoring using readily available commercial systems or components for facility in observation, increase in speed, and enhancement of accuracy.

2. Using Geodetic Methods for Monitoring

The problem of deformation monitoring using conventional geodetic (geometric) observables may be divided into two intentions:

a) reference networks, or

b) relative networks

as illustrated in Figure 1. In order to describe the behaviour of a well defined object of interest, a reference network of stations (points occupied by instrumentation) is established with respect to which the behaviour is described as reflected in the movement of the object points. The stability of the reference stations is paramount to the description and the observation scheme must provide for this ensurance and allow for a contingent reference if instability were to arise. Also, the object points must be placed on the body of interest at appropriate locations and in sufficient number to resolve the behaviour. On the other hand is the relative network in which all the stations are situated on the body and the description of deformation involves all of the stations and their relative movement. In this case, the stability of the station is its attachment to the body and its movement properly reflecting the deformation as in the object points mentioned above. Hence, in both types of network, there should be concern for station mark or pillar stability as discussed in section 3 below.

The adoption of geodetic methods in monitoring requires some subtle changes in the philosophy of measurement. Relative positioning and repeatability over-ride the customary geodetic concern for positioning since, in monitoring, it is the change in position rather than the position itself which is of major interest.

In the comparison of a pair of campaigns, the random errors in each combine together while the systematic errors would cancel in the differencing provided that they were of the same nature in both campaigns. Two avenues of analysis are available, based on:

a) coordinate differencing, or

b) observation differencing.

The former involves single campaign adjustments and provides an assessment of the consistency of the observations together in a network at the cost of requiring a complete configuration as is required for geodetic positioning and of being contaminated by systematic errors of which there is no account. The latter, observation differencing, obviates the necessity of a complete configuration and removes the contamination except when the systematic effects do not persist through both campaigns as would be the case through seasonal variations, especially in air temperature and its gradient. However, differencing requires that the same observables are repeated in the subsequent campaigns

and that the same instrumentation and observer are employed. This is difficult to guarantee because of the nature of some observables, notably directions which have an orientation parameter for each bundle, and because of the usual logistical complications.

The differences in philosophy between monitoring and positioning stem from these approaches. Since the analysis is based on differencing, the absolute *scale* of a network does not have to be determined. It is only necessary to ensure that it does not change or that its change can be well determined. The former case would occur in a reference pure triangulation network with several stations proven to be stable. The latter would be the case in a relative trilateration network measured by well calibrated EODM instruments (EODMI) as discussed in section 7. If observation differences are taken, these two relaxations are allowed. Further, it is not necessary to maintain a complete *configuration* and isolated, but repeated, observations may be used, e.g., a solitary repeated distance may be used if its orientation can be sufficiently described for the design matrices. This is also true for some aspects of *datum* defects. If the network is isolated and yet there is at least one reference sight that is distant by about five times the aperture of the network [suggested by Chrzanowski (1981)], then the orientation of the network or subsections of it may be checked by several of these sights even though the distances to the sight may be known only approximately.

The requirement for a complete configuration can be further relaxed to having targets *eccentric* to the instrument stations as shown in Figure 2, for a simple triangular component of a network. None of the angles involves the other instrument station and the misclosure of the triangle cannot be determined; however, the relative displacement of the three stations can be determined from the changes in the angles since each target is firmly attached to its associated station. This would more likely apply to a reference network for which the supposition is that the stations are relatively stable and a significant change in the value of the angle would raise suspicion on the station triplet relative stability and would require investigation and further analysis. The same principle would hold for eccentric retroreflectors for EODMI.

One other aspect contrasting monitoring with positioning, the *duration* of a campaign, may be obvious but should be mentioned. Inherent in the monitoring process is the repetition of measurements. The *frequency* of repetition is closely related to the rate at which the body is deforming. Therefore, it is essential that the scheme of observing, especially when in the form of a "network" of measurements, endures only so long as the campaign would appear to be instantaneous relative to the deformation rate. Hence, the duration of a campaign is important not only economically but also with regard to capturing the whole of the deformable body in the same state. As a consequence, there is

then not likely any opportunity for remeasurement if a blunder is detected after the campaign or if measurements are to be added to strengthen the configuration - unless the approach to the analysis is capable of dealing with isolated measurements. A strong case is thus presented in favour of electronic instrumentation and data collection, as mentioned in a subsequent section, and of real-time assessment of the observations as they are being made.

3. Station Mark or Pillar Stability

In order for a measurement system (theodolite and targets; EODMI and retroreflectors) to be effective in its reflecting the relationships among the stations, it is necessary that the system be appropriately related to the station marks - this is commonly regarded as centering in horizontal networks. It should be extended to including the height of instrument, target, or retroreflector as the present trend is to measure horizontal circle, vertical circle, and distance simultaneously. This is further discussed as it applies to the various observables in the sections that follow.

It is also necessary that the station marks, whether tablets at ground level or concrete pillars, reflect the behaviour of the object on which they are situated. For either type, it is advisable to have several reference marks arranged about each station so that they would not likely move together or with the actual station.

Especially if at an appreciable height, concrete pillars can exhibit behaviour that could inadvertantly contaminate the measurements due to the movement of the plate on the top with respect to the stable base. As the concrete ages and dehydrates, it shrinks. After 13 months, a 2 m high pillar will have shrunk by 0.8 mm in height (Zwart, n.d.). As the ambient relative humidity changes, the concrete will swell or contract at a rate of 70% of the orginal shrinkage. This would be especially noticeable between summer and winter when the relative humidity may change from ~90% to nearly 0% , along with temperature dropping from ~30°C to <0°C, in some localities.

Even more dangerous is the lateral movement associated with the bending of the pillar as a consequence of its differential heating. A pillar of concrete with a coefficient of thermal expansion, C, and a width, W, a height, H, exposed to a temperature difference, ΔT, between its two faces, will bend with a lateral displacement at its top of (Zwart, n.d.)

$$\Delta x = H^2 \, \Delta T \, C \, / \, (2 \, W) \qquad (1)$$

Typically, solar radiation on a pillar in winter time could cause a temperature difference of ~20 C° between the exposed and shaded sides. With 1.5 m of a 0.25 m diameter exposed, a pillar with C = 11 ppm/C° will be displaced 1 mm at the top in the direction away from

the sun. Often pillars are constructed with an outer isolating cylinder or they may be loosely wrapped with 50 mm to 75 mm thick foam padding or totally shaded.

Very useful discussions may be found in Kobold (1961) and in Dearinger (1974).

4. Horizontal Angles, Directions, and Azimuths by Gyro-Attachment

Although the procedure for the measuring of angles or directions and azimuths is likely familiar, the extra care required in the monitoring context is revealled when regarding the expressions for the contribution to the variance in an angle measurement by the sources of random error. Considering the A station as being occupied by the theodolite with measurement of the horizontal angle (in one position of the telescope) clockwise from the F station, distant at s_F from the A station, to the T station, distant by s_T, leads to (Chrzanowski, 1977)

$$
\begin{aligned}
\sigma_\beta{}^2 = {}& \sigma_L{}^2 \left(\cot(\gamma_F)^2 + \cot(\gamma_T)^2 \right) \\
& + 2\sigma_P{}^2 \\
& + 2\sigma_R{}^2 \\
& + \left[\sigma_F{}^2/s_F{}^2 + \sigma_T{}^2/s_T{}^2 + \left(\sigma_A{}^2 / (s_F{}^2 s_T{}^2) \right) \left(s_F{}^2 + s_T{}^2 - 2 s_F s_T \cos(\beta) \right) \right]
\end{aligned}
$$

$$(2)$$

The four components are the contributions in variance by levelling, $\sigma_L{}^2$, pointing, $\sigma_P{}^2$, reading, $\sigma_R{}^2$, the theodolite and by the centering of the theodolite, $\sigma_A{}^2$, and of the two targets, $\sigma_F{}^2$ and $\sigma_T{}^2$.

The effect of mislevelment depends on the slope of the line of sight, described by the zenith angle, γ. The maximum effect on the line of sight as it is brought to the horizontal arises from the component of the error in levelling, ε_L, which is along the trunnion axis, transverse to the line of sight. The plane of the line of sight is inclined from the vertical by the amount ε_L. This skew projection results in an error in the horizontal plane of $\varepsilon_L \cot(\gamma)$. A similar consequence stems from the ability to level the instrument which is considered as a random error with the contribution in variance as $\sigma_L{}^2 \cot(\gamma)^2$ for each line of sight. As the line of sight deviates from the horizontal, the levelling of the instrument becomes more critical. This is well known in astronomical observations and precision theodolites are equiped with very sensitive striding levels. A trunnion axis micrometer was incorporated into the Kern DKM2-AM single second precision optical-mechanical theodolite and utilizes a biaxial liquid compensator. The Kern E2 is the electronic version of the DKM2-AM and has circle readings made with respect to the theodolite axes with the relationship to the vertical resolved with the same type of biaxial compensator giving mislevelment com-

ponents along and transverse to the line of sight. The output from the E2 can be either the raw circle readings and the mislevelment components or the compensated vertical and horizontal circle readings. The Wild T2002 and T3000 are electronic versions of the single second optical-mechanical automatic T2 and have biaxial compensators in the same manner as the E2. Their predecessors, the T2000 and T2000S, had compensation only along the line of sight, as found in the T2.

The variance in pointing, σ_P^2, may be expressed as a function of the magnification, M, of the telescope and may vary from 30"/M to 60"/M (Kissam, 1962; Chrzanowski, 1977; Nickerson, 1978) according to the quality of the sight which is affected by the design of the target, the length of sight, s, and the atmospheric conditions. For a single cross-hair width of w_c, which is commonly 2" to 3" or 1 ppm to 2 ppm of s, the judgement of symmetry and centering within the design requires a background that is at least $w_c + 1.2(s/M)/1000$ wider (Chrzanowski, 1977). To accommodate a variety of sight lengths, the design should incorporate a divergent pattern as is often done in traversing targets or should employ a pattern of concentric annulars which would allow both horizontal and vertical sighting simultaneously. High precision targets of the latter type are available from Kern or Wild for sights to about 40 m as encountered in gallery traverses. Kern also produces a small half cone for similar horizontal sights.

In the use of an optical micrometer with a least division of 1" or 0.5", the reading error may be estimated to be 2.5" or 1.3" respectively (Chrzanowski, 1977). Electronic circle reading has effectively reduced this error to where it is no longer easily recognized but is in the order of 0.3" (Chrzanowski, 1984).

Errors in centering affect the angle β according to the centering at each of the stations, σ_A, σ_F, and σ_T; the lengths of sight, s_F and s_T; and the horizontal angle between them. As the sight lengths become short, centering becomes critical. This is especially of concern when tripods and optical plummets are used instead of trivets on pillar plates. Earlier styles of tribrachs did not allow the checking of centering by its rotation at setting up; however, most modern instruments and target holders incorporate the plummet in the alidade portion allowing rotation. Commonly with a plummet in proper adjustment, for an instrument height of h_i, centering can be done to 0.0005 h_i [m]. On the other hand, trivets with their "forced" centering can be placed to within 0.0001 m.

If centering and levelling are done before each of β_i in n sets, then the variance, $\sigma_{\beta_m}^2$, of the mean, β_m, can be expected to reduce by a factor of 1/n, i.e. $\sigma_{\beta_m}^2 = \sigma_\beta^2/n$. However, in some situations, especially where lateral refraction or other adverse observing conditions persist, the variance associated with the estimation of the mean would likely be a better indication of the variance. But, this would require a statistically

amicable number of sets, at least 3 or 4, and measurement under some variety of those conditions.

A similar discussion can be made for the measurement of a round of directions in which each direction is considered individually. Now, Equation (2) simplifies to

$$\sigma_d^2 = \sigma_L^2 \cot(\gamma)^2 + \sigma_P^2 + \sigma_R^2 + (\sigma_A^2 + \sigma_T^2) / s_T^2 \qquad (3)$$

Futher, the effects are the same and the same case is held in favour of several sets. As the number of directions in a round increases, there may be less advantage in observing directions rather than angles if the stability of the "stations", i.e., the targets or the instrument especially if on tripods, degrades through the round enough to contaminate the observations. In such a case, the full round could be subdivided into several subrounds, the extreme case of which would be independent adjacent angles. If independent angles are observed, then the angle closing the horizon should be included to provide field checks and assessment. Each round of directions should be observed as clockwise in the direct aspect and counter-clockwise in the reverse, returning to the first sighting.

Table 1. Common Precision Theodolites

Make	Model	Trunnion Axis Levelling	M	σ_d	Notes
Kern	DKM3	striding level (1"-2.5"/2mm)	45x	0.5"	
	DKM2	striding level (5"-6"/2mm)	32x	1"	
	DKM2-AM	trunnion axis micrometer	32x	1"	
	E2	biaxial compensator	32x	<1"	a
Wild	T3	at 90° left and right	40x	<1"	
	T2	same	30x	1"	
	T2000	same	32x	<1"	b
	T2000S	same	42x	<1"	b
	T2002	biaxial compensator	32x	<1"	b
	T3000	biaxial compensator	43x	<1"	d
AGA	142	same	30x	<2"	c

Notes   a) electronic DKM2-AM, measures every 300 msec, add any DM500 series EODMI
b) electronic T2, measures on command, add any DI-- series EODMI
c) measures on command, integrated EODMI
d) note b) plus panfocal telescope (for industrial metrology)
see also Table 8

Table 1 gives an indication of what precision theodolites are commonly available in North America. The E2, T2002, T 3000, and Model 142 are electronic and each is capable of RS232C interfacing in addition to the marketed data collectors, Alphacord128, GRE3, and Geodat, respectively. These will be discussed further in the section on integrated

systems.

Although their accuracies would likely be inappropriate in most cases, the measurement of azimuth by gyro-attachment might still be considered and should be mentioned. The Swiss, Hungarian, and West German makes of relatively precise gyro-attachments yield azimuths with standard deviations in the order of 3" to 5", as shown in Table 2. The most common, the GAK1, would yield such a standard deviation only under carefully controlled conditions with considerable effort (Jeudy, 1986). The

Table 2. Common Gyro-theodolites

Make	Model	Notes
Wild	GAK1	attachment, +/- 30" in 20 minutes on a T2, +/- 4.4" in 15 determinations (Jeudy, 1986)
Fennel	TKW	equivalent to GAK1
MOM	Gi-B11	30x telescope, +/- 3" to +/- 5" in 30 minutes, 50 kg
WBK	Gyromat	+/- 3" in 7 minutes, 16 kg

GAK1 usually yields a standard deviation of 15" in an azimuth that is the mean from observation at each end of the line. The other two are integrated gyro/theodolites with the lightest and most rapidly measuring being the Gyromat. Both are quite expensive and not so readily attainable.

5. Vertical Angles or Zenith Distances

In a manner similar to that already done for horizontal angles, the random error associated with a single zenith distance, γ, may be expressed through

$$\sigma_\gamma^2 = \sigma_L^2 + \sigma_P^2 + \sigma_R^2$$
$$+ \sigma_T^2 \left(\sin(\gamma)^2/s_T^2 \right) + \sigma_k^2 \left(s_T^2/4R_E^2 \right) \tag{4}$$

in which the levelling, $\sigma_{L'}$, is now along the line of sight and the "centering" component is related to the knowledge of the target height, σ_T. An additional term is introduced to account for knowledge of the refractive index, k, which may be estimated from the atmospheric pressure, p [mb], and dry bulb temperature, t [K], and some knowledge of the vertical temperature gradient, dt/dh [Km^{-1}], by (Bomford, 1977)

$$k = 502 \left(p/t^2 \right) \left(0.0341 + dt/dh \right) \tag{5}$$

A more substantial approximation may be obtained utilizing a series of gradient determinations, derived from temperature gradient measurements, and the terrain profile (Greening, 1985).

Unless it is possible to observe the opposite zenith distance along the line simultaneously and therefore practically compensate for the curvature due to refraction, then the effects of this curvature can be quite substantial (Greening, 1985) and a great deal of caution should be exercised if three dimensional positioning and monitoring is attempted. This is particularly so if the campaigns are to occur under different atmospheric conditions. Differing of campaigns would not remove the systematic effects since they are no longer common.

6. Alignment

There are several ways of determining the alignment of object points with respect to some reference line created or defined by a pair of reference stations. Lateral displacement can be measured either indirectly using optical methods, by angular measurements, or directly using optical means or mechanical means of creating the reference line. The direct optical methods using alignment telescopes with displacement targets, using laser with centering detectors or with diffraction zone plates or with Koster prism and retroreflector, and the direct mechanical methods using steel or nylon wires and optical, mechanical, or electronic sensors are beyond the realm of geodetic methods and are covered in Chrzanowski (1987). On the other hand, indirect optical methods are specific applications of traditional horizontal angle measurement and are considered in this section. Vertical alignment is a special application of height differencing as covered in a subsequent section. Horizontal alignment is commonly regarded as "centering" which is not so much a measurement as a constraint on two dimensional displacement from a vertical reference line; however, with appropriate scales the amount of displacement can be measured.

6.1 Lateral Alignment by Indirect Optical Means

The choice among the four methods of angular measurement from which the lateral offset or displacement is derived may be made by considereing the lengths of sight, the number of angles to be measured, and the resultant accuracy in the offset determination. The four methods may be called (Chrzanowski, 1981)

      a. closed traverse,

      b. open or fitted traverse,

      c. single station small angle, and

      d. separate point included angle

and are illustrated in Figure 3 in which the reference line is defined by stations R_1 and R_2 (triangles) in between which are the object points P_i (circles), each with a transverse offset

of x_i (heavy lines). The offsets have been greatly exaggerated with respect to the actual separation of the stations and the angles are usually quite flat and whether the angles as shown or their explements are measured, the same effect is met.

The closed traverse consists of the measurement of each traverse angle β_i at the P_i plus the closing angles at R_1 and R_2. If the P_i are equally spaced so that each traverse leg has a length, s, with R_2 being $(n+1)s$ from R_1, then the variance σ_β^2, in radians squared, of angle measurement propogates into the variance of the offset, σ_x^2, at P_i as

$$\sigma_{xi}^2 = [i(i+1)(n+1-i)(n+2-i)] \; [(2i+1)n-(2i^2-2i-3)] \; s^2 \; \sigma_\beta^2$$
$$/ \; [6(n+1)(n+2)(n+3)] \qquad\qquad (6)$$

The open or fitted traverse is the same as for the closed except that the angles at R_1 and R_2 are not measured. Consequently, the variance of the offset at P_i is given by

$$\sigma_{xi}^2 = [i(n+1-i)] \; [2i(n+1-i)+1] \; s^2 \; \sigma_\beta^2 \; / \; [6(n+1)] \qquad\qquad (7)$$

The sacrifice in accuracy by neglecting the two end angles may be illustrated in an example considering $n=7$ points spaced at $s=100$ m so that R_2 is 800 m from R_1. At the centre point, P_4, $i=4$ so, with angles having $\sigma_\beta^2=(0.5")^2$, the closed traverse results in an offset variance of $\sigma_{x4}^2=(0.5 \text{ mm})^2$ and the open traverse, $\sigma_{x4}^2=(0.8 \text{ mm})^2$.

By occupying only one of the end stations, say R_1 as in Figure 3c, and by measuring only the small angle between the reference line and the object point P_i, which is s_i from R_1, the offset variance is a direct propogation as

$$\sigma_{xi}^2 = s^2 \; \sigma_\beta^2 \qquad\qquad (8)$$

with σ_β^2 in radians squared. As in the above example, at P_4 which is 400 m from R_1, σ_{xi}^2 is now $(1.0 \text{ mm})^2$.

If only the object point, P_i, were occupied and just the included angle, β_i, measured as shown in Figure 3d, then the offset becomes proportional to the deflection angle. If point P_i is s_i from R_1 and R_2 is s_T from R_1, the offset x_i may be obtained from

$$x_i = (\pi-\beta_i)(s_T-s_i)s_i \; / \; s_T \qquad\qquad (9)$$

with β_i in radians and the arrangement of R_1,P_i,R_2 in nearly a straight line. From this, the variance of the offset may be obtained by

$$\sigma_{xi}^2 = x_i^2 \; \sigma_{\beta i}^2 / (\pi-\beta_i)^2 \qquad\qquad (10)$$

For the above example, s_i is 400 m, s_T is 800 m, and now, with an angle of $180°59'31"$, the offset is 30.0 mm +/- 0.5 mm. If the offset were 60.0 mm, its variance would still be $(0.5 \text{ mm})^2$ since the deflection in the denominator of (10) compensates for the increase in offset.

Taking $\sigma_\beta^2 = (0.5")^2$ is typical of such alignment endeavours using a DKM3, E2, or T2000; however, the lengths of sight and particularly possible lateral refraction should be regarded when choosing which method to employ. In these expressions, only the random

error associated with the angle measurement has been considered and the method of centering will add directly to the variance of the offset. More serious is the effect of lateral refraction (Chrzanowski et al., 1976) that would be encountered in the longer sights associated with the small angle and included angle methods. Nonetheless, the curvature due to refraction is systematic and if the traverse were along a slope, for instance, where there would be a similar effect on each traverse leg, then the effect would be cumulative. This would be especially serious if the conditions resulting in refraction curvature were to vary with the time of year and thus be peculiar to each campaign. The difference in effect would contaminate the resultant differences in alignment in campaign comparisons.

Table 3. Precision Optical Plummets

Make	Model	Type	Range	Accuracy
Kern	OL	nadir and zenith	100 m	1 in 50 000
Wild	ZNL	nadir and zenith	100 m	1 in 30 000
Wild	ZL	zenith, automatic	100 m	1 in 200 000
Wild	NL	nadir, automatic	100 m	1 in 200 000
Zeiss Jena	PZL100	zenith, automatic	100 m	1 in 100 000

6.2 Horizontal Alignment or Centering

Precision optical plummets, a sample of which is given in Table 3, can also be used to measure small amounts of horizontal relative displacement. This would be applicable in the horizontal connection through a borehole, short shaft, or stairwell between two levels in a structure or two gallery traverses in a dam. The mechancial equivalent would be the suspended or inverted pendulum [see Chrzanowski (1987)]. The major restriction in the use of plummets would be visibility and sturdiness of the support for the plummet. The more modern models have automatic "compensation" or creation of the horizontal reference against which a perpendicular, vertical reference sight is projected. This has resulted in a tenfold increase in accuracy over the spirit level versions; however, the same cautions should be exercised as with similarly operating levels.

7. Distance Measurement

Because of the infusion of electronics, the surveying instrument industry provides a profusion of electro-optical distance measuring instruments (EODMI). Their convenience and ease of operation has left the mechanical measurement of distance virtually neglected. It is impressive how some of the modern, but more specialized, instruments come close to

rivalling the more laborious procedures of the past in both precision and accuracy. Many of the commonly found EODMI are capable of accuracies sufficient for all but the most particular cases of deformation measurement; however, this is not without some effort. Further to this, the following sections extend into what care is required to evaluate the performance of a particular EODMI and to achieve dependable measurements under a variety of conditions.

Both optical and mechanical methods still have application since very few EODMI are capable of measuring very short distances (tens of metres). The mechanical means, virtually portable extensometers, verge on the realm of geotechnical instrumentation and are a subtle indication of how far ranging is the the surveying engineer as an expert in measurement.

7.1 Distances by optical means - the subtense bar

Remarkably enough, the subtense bar has re-emerged as a useful means of measuring relatively short horizontal distances, to about 5 m, especially if the distance cannot be mechanically spanned. A precision 1 m bar of invar tubing is available from Kern and results in an accuracy of 40 ppm if the subtended angle can be measured to at least +/- 1.3" (Keller and Aeschlimann, 1976). Now, with electronic theodolites capable of subsecond accuracy with a minimum of effort, such an angle accuracy is readily attainable as opposed to the at least 4 sets required with a DKM2-A (Keller, 1974).

Table 4. Wire and Tape Extensometers

Make	Model	Δs	Range	Accuracy	Restriction
Using invar wire					
Kern	Distometer ISETH	100 mm	to 30 m	+/-0.02 mm	any orientation
CERN-SEPEM	Distinvar	50 mm	to 106 m	+/-0.03 mm	only horizontal
Rocktest	Distomatic	60 mm	to 24 m	+/-0.05 mm	any orientation
Using steel tape					
Rocktest	Convex		to 20 m	+/-0.125 mm	any orientation
Solinst	MKII		to 20 m	<+/-0.1 mm	any orientation

7.2 Distances by mechanical means - extensometers

Extensometers are generally regarded as geotechnical instruments but there are several models manufactured more specifically as surveying instruments, as summarized in Table 4. The accuracy of the first three is attainable through the use of invar wire in catenary. In the measurement of a distance, a specific tension in the wire is created using either a calibrated coil spring (Distometer ISETH, Distomatic) which allows the line to be

in any spatial orientation or a counter weight (Distinvar) which restricts the instrument to measuring in the horizontal but to over a much greater distance. Both the Distomatic and the Distinvar can be automated and adapted for continuous recording.

Although used for its low coefficient of thermal expansion (about 0.8 ppm/C°), invar does present some problems in its use, especially since submillimetre accuracy is sought. Even though it is aged somewhat by its manufacturer, invar is susceptible to catastrophic changes in length, by one or so millimetres, unless it is further aged artificially by coiling and recoiling in the opposite direction or sense several hundred times. Also the wire must be handled with a great deal of care, not only to prevent obvious physical damage but also to avoid sudden forces on the wire, e.g., hitting or dropping the coil of wire, which would likely also result in dimensional changes. It is advisable to have several duplicate lengths for multiple measurement and as a check against changes and also as a contingency for one section becoming unusable. As an additional check, a stable reference base should be regularly measured for comparison and possible "calibration" of the extensometer. The Distometer, for instance, has its own invar frame for calibrating the meter and a weight for calibrating the tensioning device (Kern & Co. Ltd., 1978). A constant air current crossing the wire will introduce an additional lateral sagging which introduces a false lengthening of the distance - a systematic effect that is difficult to model.

Commonly used, but less accurate, are tape extensometers, e.g., Convex and MKII, which operate in a manner similar to the Distometer ISETH. In contrast, they use steel tape perforated at 25 mm intervals for indexing to the meter portion. This makes them more convenient since the tape reel is an integral part of the instrument which then serves for measuring any distance to about 20 m. With its larger cross-section, tape is more susceptible to false lengthening by transverse air currents and, being made of steel, it is more sensitive to changes in temperature (11 ppm/C°).

7.3 Distances by electro-optical means

In simplified but practical terms, the concept of electro-optical distance measurement over short ranges may be represented by

$$s = mU + d + z + c + \varepsilon + \Delta s \tag{11}$$

in which the distance is an integral number, m, of unit lengths, U, plus the fractional part, d, of a unit length. To this three corrections (known systematic effects) are applied: z, the zero correction or additive constant; c, the cyclic correction having characteristics, amplitude and phase, depending on the distance or the noise to signal ratio; and ε, a correction for noncyclic effects, the magnitude of which depends on the distance. The final term, Δs, is whatever geometric correction that is applied to reduce the spatial distance to

the desired computational surface. Further, the unit length may be decomposed into

$$U = \lambda_{mod} / 2 = c_0 (1 - n_R - n_A) / (2 n_R v_{mod}) \tag{12}$$

in which the modulation wavelength, λ_{mod}, is described in terms of the speed of light in vacuo, c_0; reference refractive index, n_R; actual refractive index, n_A, at the time of measurement; and the fine pattern modulation frequency, v_{mod}.

The variance of such a distance measurement may be expressed as

$$\sigma_s^2 = a^2 + b^2 s^2 + \sigma_{\Delta s}^2 \tag{13}$$

which is comprised of a constant or distance independent component, a^2; a distant proportionate component, $b^2 s^2$; and the noninstrumental portion, $\sigma_{\Delta s}^2$, arising from the reductions applied to the instrument output. The first two terms are the instrumental portion that is commonly quoted by manufacturers as "+/- a +/- b" and a common misconception fails to recognize that the standard deviation of a distance is $\sigma_s = (\sigma_s^2)^{1/2}$ and erroneously combines the two by direct addition. The constant component, a, contains those error components of (11) and (12) which are not functions of the distance,

$$a^2 = \sigma_d^2 + \sigma_z^2 \tag{14}$$

The dependent component, $b^2 s^2$, contains all those errors, the amounts of which may vary with the magnitude of the distance, so

$$b^2 = \sigma_{co}^2 / c_0^2 + \sigma_n^2 / n^2 + \sigma_v^2 / v^2 + (\sigma_c^2 / s^2 + \sigma_E^2 / s^2) \tag{15}$$

In order to capture the ultimate accuracy of commonly found short range EODMI and to ensure its consistency, several tests should be performed routinely with the use of the instruments. Especially if repeated measurements are to be compared, each of the above influences must be regarded in the light of observing and reduction procedures and corrections from calibration. The pursuit of full details is strongly recommended from such sources as Dracup et al. (1982), Fronczek (1980), Greene (1977), and especially Rüeger (1980) and Rüeger (1990). In the measurement of a distance, some of the

Table 5. Errors Considered in the Performance of an EODMI

Instrumental Errors
    1. scale (by change in the frequency of the fine measuring pattern)
    2. zero error
    3. cyclic error
    4. phase inhomogeneities in the light emitting and photo diodes
Non-instrumental Errors
    1. centering error
    2. scale (by knowledge of n_A in the first velocity correction)
    3. spurious reflections, especially when weak return signal
    4. reductions to a computational surface

processes or characteristics are interrelated and their individual contributions cannot be

readily distinguished or isolated. Hence, they have been classified in the manner as given in Table 5. The non-instrumental errors are controlled by observational procedures including ancillary measurements. The characteristics of the instrumental errors may be determined through calibration or testing so that corrections may be applied.

7.3.1 Non-Instrumental Errors and Observation Procedures

The distance measuring system consists of the EODMI itself, possibly a theodolite upon which the instrument is mounted, the retroreflectors at the other station, the mechanical means of centering or relating the instrumentation to the station marks, and instrumentation for determining the atmospheric conditions at the time of measurement, e.g., an aspirating psychrometer for dry and wet bulb temperatures and an aneroid barometer for atmospheric pressure. Hence the operation of the system entails the use of much paraphernalia, each piece having its own errors.

The determinination by a distance measuring system can be no better than the means by which it is related to the station marks. If the instrumentation is not properly centered, the longitudinal component adds to the zero error. This is particularly influential when using tripods and tribrachs with non-rotatable optical plummets.

The centering is intimately related to the levelling of the tribrach and the more sensitive target vials should be used since they and their associated plummet can be rotated for checking through 90° and 180°. This should be checked frequently to ensure that the set-up was maintained during the mounting of the EODMI (e.g., the Tellurometer MA100 is very awkward at 17 kg) and has not drifted off level.

The first velocity correction, via the value of n_A, is determined by measuring wet and dry bulb temperatures (for the water vapour pressure) and atmospheric pressure along the line of sight during the measurement. For most short range applications, sampling at the two stations is likely sufficient unless they are not representative of the whole line. The ambient refractive index may be obtained from

$$(n_A-1) = (n_G-1) (273.16 \ p/(273.16+t)1013.25)$$
$$- 11.20 \times 10^{-6} e/(273.16+t) \tag{16}$$

in which n_G is the group refractive index which is a function of the effective wavelength in vacuo, p is the atmospheric pressure [mb], t is the dry bulb temperature of the air [°C] and e is the partial water vapour pressure [mb] which is a function of t, p, and t', the wet bulb temperature (Rüeger, 1990). The differentiation of equation 16 reveals the tolerances for these ancillary measurements. The error in n_A effects the same error in the distance. For an error of 1 ppm, the dry bulb temperature must be known to 1°C; the atmospheric pressure, to 3 mb (2.3 mmHg); or the water vapour pressure, to 25 mb. Nomograms and

short field formula often neglect the term for partial water vapour pressure. Rigorous formulae [e.g., Rüeger (1980), Rüeger (1990)] should be followed for high precision and on longer lines.

In order to enhance the determination of the fraction of the unit length, i.e., to lessen σ^2_d/n_s, a number of determinations should be made. As mentioned below regarding the phase inhomogeneities of the diodes, obtaining the maximum level of return signal strength by electronic pointing should be done several times. In order to obtain an indication of the repeatability of the system, three or four groups of five measures with fresh pointing at the start of each group is suggested. The noise of the resultant mean is a measure of the repeatability. The several pointings also help to reduce the effects of spurious reflections from vehicle reflectors, mirrors, windows, etc. that may occur when using less efficient reflectors at very short range.

The various geometric reductions are treated extensively in Rüeger (1990). Especially at short ranges, significant error could result from insufficient knowledge of the height differences between the instrument and the reflector, ΔH, which includes the station elevation difference and the apparatus heights. For short range, the total differential of the reduction formula simplifies to

$$\sigma^2_s = (\Delta H^2/s^2)\, \sigma^2_{\Delta H} \tag{17}$$

showing that as the distance shortens, the knowledge of the height difference becomes more critical.

7.3.2 Instrumental Errors and Calibration

Some characteristics of the behaviour of an EODMI system can be determined by certain types of controlled observations so that corrections to usual measurements might be made. Of the list in Table 5, only the last, phase inhomogeneities, is a test to illustrate the quality of the components, while the others yield correction values.

The frequency of the fine measuring pattern, v_{mod}, is generated by a crystal oscillator, the stability of which is sometimes controlled by its being kept at a constant temperature (e.g., MA100: 75°C) in contrast to the ambient temperature. Some instruments allow for the monitoring of v_{mod} as v_a, the actual frequency during measurement. The output is then scaled by the factor $k = v_{mod}/v_a$. However, most common EODMI do not have such facilities or they have such a mixture of signals that the separate frequency is not suitably measureable. Nonetheless, the scale may be determined by comparison with "known" distances. This has been done commonly on "calibration baselines", linear arrays of a series of supposedly stable concrete pillars with forced centering to ± 0.0001 m or better. It has been advocated to solve for both a scale factor

and the zero error (see below) simultaneously from the series of inter-pillar baseline distances. But, 1 ppm is only 0.0016 m in 1600 m and the lesser distances would not enhance the knowledge of the scale factor. Most modern reputable instruments are stable well within usual needs (e.g., Kern DM503 has oscillator stability that contributes less than 1 ppm). With the assumption that atmospheric effects and the zero and cyclic errors have been removed, the comparison of several long distances, at least to resolve 1 or 2 ppm, would serve as a routine diagnostic of consistent behaviour.

For each combination of EODMI and retro-reflector there is an amount by which the optical/electronic centres are longitudinally offset from the centres related to the station marks. When applied to the output to bring the distance to be between the mechanical centres, this amount becomes the zero correction which can be modelled along with the network adjustment provided the same combination of instrumentation was used throughout. This should be avoided in favour of analysing baseline measurements for the correction since it has been found to be unusually sensitive to configuration geometry. For a particular combination, the z may be determined using an ideal linear array of stations - the calibration baseline - which do not need to be concrete pillars, but at least forced centering on tripods that will remain stable throughout the campaign of measurement of 4 to 6 hours.

Considering the cumulative inter-station distances along a linear array as the abscissae along an x-axis, with zero at one end station, leads to the observation equation

$$s_i + v_i = x_{jj} - x_j + z \qquad\qquad i = 1, 2, ..., (p/2)(p-1) \qquad\qquad (18)$$

assuming that the value of z is constant over the range of s_i. With p stations, there are $(p/2)(p-1)$ possible one-way distances and $(p-1 + 1)$ unknowns - the $(p-1)$ abscissae and the z, the zero error. The redundancy of $[(p/2)(p-1)-p]$ allows least squares estimates for the abscissae and zero error and estimates of their accuracies. The zero correction is s_i+z and the variance of z is contained in the a^2 term of equation (13).

Any contaminating electrical signal from the transmitter to the receiver will perturb the phase with maximum effect when the two signals are relatively displaced by $\pi/2$ and will be null when by zero or π. The error as a function of distance is then sinusoidal with a period of $U = \lambda_{mod}/2$ and with an amplitude proportional to the ratio of the contaminating and received signals, i.e. inversely proportional to the return signal strength. Hence the error is described by

$$c(s) = a_s \sin[(2\pi/U)(s+\phi_c)] \qquad\qquad (19)$$

for which the amplitude a_s for a given return signal strength, and the phase of the sinusoid, may be determined through the following procedure.

A straight, reasonably level rail, graduated every 0.050 m (+/- 0.001 m or better)

through the unit length of the instrument (commonly 10 m, but 2 m for the MA100) serves to guide the movement of a retro-reflector through known increments. The instrument is aligned to the travel of the retro-reflector at a distance of at least 100 m from the least of the graduations that increase away from the instrument. The distance should be enough to allow sufficient attenuation of the signal by an iris diaphragm over the retro-reflector, but not so far that meteorological variation would affect the distances. The instrument is pointed electronically several times and the mean position is maintained so that the same portion of the diode is used. Distances are measured in five or ten repetitions at each position of the retro-reflector (e.g. every 0.250 m for 10 m, every 0.100 m for 2 m) from zero to U and from U back to zero. The variances of the means are a measure of the repeatability and steadiness of the atmosphere and setup

Each of the (k+1) measured distances may be modelled as

$$s_i = d_i - z + c = d_0' - z + x_i + c \tag{20}$$

The zero error cannot be resolved, but it does not have to be applied. The initial distance can be redefined as $d_0 = (d_0' - z)$ and with the definition of c from equation (19), the observation equation becomes

$$s_i + v_i = d_0 + x_i + a_s \sin [(2\pi/U)(d_0+x_i+\phi_c)] \tag{21}$$

Since the x_i are known well enough to be considered as constants, there are 3 unknowns: the initial distance, d_0; the amplitude, a_s, for a particular return signal strength; and the phase, ϕ_c, accompanying this value of the amplitude. The redundancy and the sampling through the whole of one period allows their least squares estimation. By controlling the aperture of the retro-reflector with a variable iris diaphragm, the return signal strength can be attenuated to simulate longer distances. Several determinations over the working range for the instrument will yield several amplitudes, hence a_s, to which a straight line may be estimated for interpolation.

Often expressed in terms of their effect on the zero error are the phase inhomogeneities across the diodes - pointing error for the light emitting diode (LED) of the transmitter and aperture phase error for the photo diode (PD) of the receiver. The former would remain constant provided that the mis-aiming were consistent. Both could vary non-linearly with distance depending on the nature of the measuring procedure.

As there is a variety of small time delays over the junction in the LED, there are consequently differences in phase across the modulation of the beam collimated by the transmitter optics. At any appreciable distance from the instrument, the beam width is considerably larger than the diameter of the retro-reflector. Accordingly, the retro-reflector samples only a portion of the radiation which may differ in phase from another portion sampled by another pointing. The internal reference signal will have a phase that is the

average of the emitted light. Unless the sampling and subsequent return signal is of a portion having the same phase, the discrepancy would be interpreted as part of the delay due to the distance to the retro-reflector. Also, contamination would result if it were different from the pointing or portion sampled in determining the zero error. This quality may be depicted through a pointing diagram which indicates the care necessary in electronic pointing rather than offering a correction (Greene, 1977; Covell, 1979).

Table 6. Precision EODMI

Make	Model	Range	a	b	Radiation	Note
Kern	DM502	2 km	0.003 m	2.0 ppm	infrared	
	DM503	5 km	0.003 m	2.0 ppm	infrared	
Wild	DI4S	700 m	0.001 m		infrared	
	DI4L	5 km	0.005 m	5.0 ppm	infrared	
	DI5	5 km	0.003 m	2.0 ppm	infrared	
	DI20	14 km	0.005 m	1.0 ppm	infrared	
	DI2002	2.5 km	0.001 m	1.0 ppm		
Kern	ME3000	2.5 km	0.0002 m	1.0 ppm	Xenon flash	a
	ME5000	8 km	0.0002 m	0.2 ppm	HeNe laser	
ComRad	Geomensor 204 DME	10 km	0.0001 m	0.1 ppm	Xenon flash	a,b
Tellurometer	MA100	2 km	0.0015 m	2.0 ppm	infrared	
	MA200	1.5 km	0.0003 m	0.5 ppm	laser	
TerraTechnol'y	Terrameter LDM2	20 km	0.0001 m	0.1 ppm	HeNe,HeCd	c
SpectraPhysics	Geodolite 3G	35 km	0.003 m	0.2 ppm	HeNe	d

Notes:
"b" is an instr···m· ·al value [actual values for equation (13) are usually 2.0 ppm or more].
a) sample of t ; . · .tive humidity by instrument
·) also remote ··· pling of t, p, and relative humidity with telemetric link to instrument
·) refractive index from comparison of two wave length behaviour. accuracy is a or b, whichever is the larger.
d) requires elaborate remote sampling of t, p, and relative humidity (aerial profiles)

This discussion has focussed on the more common EODMI. Precision instruments, such as the ones in Table 6 for example, and especially those which incorporate some means of sampling the atmosphere, e.g. the ME3000 and the Geomensor 204 DME, require much more elaborate formulation for reduction of the instrument output into actual spatial distances. The additive correction and geometric corrections are the same in essence but there is no cyclic error in either the ME3000 or the Geomensor since each utilizes optical-mechanical phase delay.

One other type of electronic distance measurement, the laser interferometer, has been adapted from its customary laboratory environment to the somewhat controlled atmosphere of accelerator setting out. The Hewlett-Packard HP5525B is capable of measuring distance

differences, between two tracable positions of the retroreflector moving at a speed of less than 0.3 ms^{-1}, to 60 m with a standard deviation of +/- 0.5 μm/m (Gervaise, 1981).

8. Height Differences

The measurement of height differences has been traditionally done by geometric or differential levelling. Over the past decade, developments in very compact EODMI and in precision electronic theodolites have directed interest toward trigonometric height traversing as a more than viable alternative. Even so, each method has its advantages and applications and both are considered in this section.

8.1 Geometric or differential levelling

The use of a precision tilting or automatic level with objective parallel plate micrometer extends from very close range applications in optical tooling through vertical networks of assorted aperture to successive setups in route levelling of tens of kilometres between junction points. The methods of measurement are relatively simple and well established. Presently available levels and their route levelling accuracies are given in Table 7.

Table 7. Precision Spirit and Automatic Levels

Make	Model	Accuracy	Micrometer
Spirit			
Wild	N3	+/- 0.2 mm(km)$^{-0.5}$	integral plane parallel plate (0.1 mm)
Automatic			
Wild	NA2	+/- 0.3	accessory
	NA2000	+/- 1.5	digital, bar coded staves
	NA3000	+/- 0.4	digital, bar coded staves
Kem	GK2-A	+/- 0.3	accessory
Zeiss Jena	Ni002	+/- 0.2	integral
Zeiss Oberkochen	Ni1	+/- 0.2	integral
Zeiss Oberkochen	Ni2	+/- 0.3	accessory

Within the galleries of the Gigerwald Dam in Switzerland, Keller (n.d.) was able to achieve +/- 0.04 mm per setup using a Ni1 with sight lengths of 16 m. Through repeated measurements, one setup would be capable of detecting a tilt (change in slope or relative vertical orientation) greater than 0.36". This is comparable to hydrostatic levelling which is usually +/- 0.03 mm over a 40 m separation of recording stations and which corresponds to a tilt of 0.22" [see Chrzanowski (1987)].

Even the measurement of a single height difference, as would be likely encountered

in an isolated monitoring network situated in rough terrain, becomes laborious if the lengths of sight are necessarily shortened to accomodate the slope of the terrain. Due to the effects of refraction, the restrictions on the lines of sight require that no staff reading be made below 0.5 m and that no line graze the terrain by less than 1.5 m. This can severely limit progress in rough terrain. If the setups are part of a longer route, the systematic errors that are relatively negligible in one setup accumulate to alarmingly significant amounts (Greening,1985). Most appreciable of these are the effects of refraction along steadily sloping routes and the settlement of staves and of the instrument tripod between and during setups.

In his considering the feasibility of trigonometric height traversing, Chrzanowski (1984) reports that current motorized efforts for geometric levelling by the United States National Geodetic Survey progress at a daily rate 7 km to 12 km one way with sight lengths of 40 m to 60 m for first and second order respectively, on flat terrain. Shortening the lengths of sight greatly impedes progress on sloping terrain: 3.5 km/day at 4° and only 1 km/day at 10°. Since the horizontal lines of sight are oblique to the isothermal layers that are usually parallel to the terrain, the systematic effect of refraction can accumulate to as much as 0.015 m for every 100 m change in elevation. Even the preventative measures, e.g., limiting the nearness of the line of sight to the ground, and the application of modelled corrections, from extra temperature measurements and terrain profiling, still do not remove 20% to 60% of the systematic effects (Chrzanowski, 1984). This is especially of concern when the height differencing is taken over long routes to beyond the zone of influence, e.g., to 5 km or 10 km from a reservoir, or where a vertical network has been established for monitoring tectonic movement, e.g., at Palmdale, California (Holdahl, 1983).

8.2 Trigonometric Height Traversing

The effects of refraction can be overcome to a large extent if the lines of sight are parallel to the terrain so that they do not obliquely intersect several isothermal layers. On the average, this is possible with trigonometric height traversing which has two methods - leapfrog and reciprocal.

The leapfrog method is a modification of the geometric levelling method by replacing the level with a theodolite/EODMI and the staves, by mutiple targets/retroreflector rods. The lengths of sight are balanced and the sloping of the terrain is no longer such a restriction except that the lines of sight should be kept at least 1.5 m above the terrain through their extent. Sighting to targets at several heights, e.g., 2.0 m and 3.5 m, helps to randomize the effect of refraction by using sights that pass at several angles through the

isothermal layers. With a motorized observer and two target rods and computerized data gathering and setup evaluations, the leap frog method can progress by 12 km to 14 km daily irrespective of the slope of the terrain with sight lengths of 200 m to 250 m. The major drawback to the method is the requirement of balanced sight lengths for foresight and backsight. This entails additional reconnaissance and is most troublesome on winding roadways along which most routes would be taken. An experienced crew of observer and two rod men could engage the rod transporting vehicle and driver for the initial reconnaissance for the next station during the measurements at the present setup which take about 10 to 12 minutes, involving four sets of zenith angles plus distance measurements to the fore and back targets and retroreflectors. A typical accuracy for leapfrog route traversing is better than +/- 2 mm(km)$^{-0.5}$.

In the reciprocal method, two theodolites sight each other simultaneously and the curvature of the line of sight due to refraction is removed in the averaging from the sighting at each end. Along with requiring a second theodolite, the reciprocal method is somewhat cumbersome since additional separate and sometimes laborious procedures are needed to connect the height traversing to the benchmarks. Also, the theodolites themselves are the turning points. This is in contrast to the leapfrog method in which the rods can be placed on temporary, intermediate or permanent bechmarks directly and recovery is more easily done if the instrument setup happens to be disturbed. In the reciprocal method, there is not the need for as extensive reconnaissance since the restrictions are that that the lines of sight are at least 1.5 m above the terrain and that the lengths do not exceed 250 m. Since data is collected at both instruments, the ability to check the observations in the field is hampered without some form of telemetric link between the second theodolite and the main controlling or "master" theodolite computer. In addition, a second experienced observer is required for the reciprocal method as the procedures for sighting simultaneously must be well conducted. For proper sighting, balanced symmetric targets must be attached to the theodolites and one, likely the master, would be equiped with an EODMI while the other, the retroreflector. The accuracy and rate of progress by the reciprocal method for route height traversing is comparable to that for the leapfrog method (Chrzanowski, 1984); however, further evaluation, especially of the behaviour of electronic theodolites and their compensators is necessary for both methods.

The reciprocal method of height differencing becomes attractive for three dimensional monitoring methods employing forced centering on concrete pillars. As mentioned above in the horizontal context (section 2), vertically eccentric sights could be used; however, no misclosure would be available to evaluate the observations. This would

place the onus on extensive knowledge of the environment and behaviour of the instrumentation if comparison is to be made of campaigns at various times of the year. The effects of refraction would predominate in a network of rough terrain with likely grazing lines and a variety of ground cover and ambient temperature.

9. Integrated Systems

As a consequence of contemporary technological developments, it has become increasingly easier to mate EODMI with theodolites, creating electronic total stations. Nonetheless, only a few are near single second precision (Table 1) and have proven themselves and have become the basis for a return to optical measurement in industry. Not only are the measurements easier to make and of higher precision, but also the capturing of data is virtually automatic via data recorders or direct interfacing to microcomputers. The interfacing capability has further lead to the development of practically real-time positioning and analysis in three dimensions in coordinating systems that are commercially available (Table 8) or that may be assembled from modules. Even further again is the development of a surveying robot, Wild T2000S MOT, positionable to +/- 2" horizontally and +/- 4" vertically; however, even by the early 1980's, one had been developed by modifying an AGA model 710 (Kahmen, 1984) with servomotors driving the tangent screws and an ocular quadrant laser centering detector and with a programmed expanding spiral search routine in addition to the angular orientation or positioning.

Table 8. Total Stations and Coordinating Systems

Total Stations

Make	Model	Angle	a	b	Data Collection	RS232C
AGA	142	<+/-2"	0.005 m	3.0 ppm	Geodat	accessory
Kern	E2/DM503	<+/-1"	0.003 m	2.0 ppm	Alphacord	integral
Wild	T2000(S)/DI4S	<+/-1"	0.001 m		GRE3	accessory
	T2002/DI2002	<+/-1"	0.001 m	1.0 ppm	GRE3	integral
	T3000/DI2002	<+/-1"	0.001 m	1.0 ppm	GRE3	integral

Three Dimensional Coordinating Systems

Kern	ECDS1	(E2 + E2 + DEC Micro/PDP-11) x,y,z +/- 5 ppm
Wild	TMS	(at least two T2002 or at least two T3000)
Wild	ATMS	(at least two TM3000V or at least two TM3000L)

Of the total stations given in Table 8, only the Kern provides for virtually direct single cable RS232C interfacing. The AGA and Wild require accessory connection boxes

as the interfacing is a modification of the data recorder interface and both require 120 V ac to maintain voltage levels for successful interfacing communication. The data recorders offer some opportunity to programme for field analyses but at the sacrifice of storage space for data. The simultaneous measurement of both circles and distance is tempting; however, proper EODMI use requires electronic pointing of the instrument and this cannot be assumed to coincide with the optical axis of the telescope. Besides, even if there is a target portion to the retroreflector, it is not suitable for a zenith angle measurement compatible with the horizontal sights. Also the consideration for a simultaneous three dimensional network must have regard for the ever present effects of refraction and also for what ever analysis package is available for proper reduction, weighting, and adjustment of the observations.

Very recently, the precision electronic theodolites have been further exploited in the creation of three dimensional coordinate determination and real-time analysing systems (Table 8). These have revolutionized the approach to industrial quality assurance by providing rapid non-contact measurements in the order of +/- 5 ppm in close range applications (to tens of metres).

It is very tempting to project these developments to the day when the whole survey might be conducted by robots telemetrically linked to a main computer at the push of a single button; but, still the major aggravation will be the effects of the environment, especially atmospheric refraction, that limit achievable accuracy.

References and Suggested Readings

ACSM   American Congress on Surveying and Mapping
ASCE   American Society of Civil Engineers
FIG   Fédération Internationale des Géomètres
UNB SE University of New Brunswick, Department of Surveying Engineering
US DC   United States Department of Commerce
NOAA   National Oceanic and Atmospheric Administration

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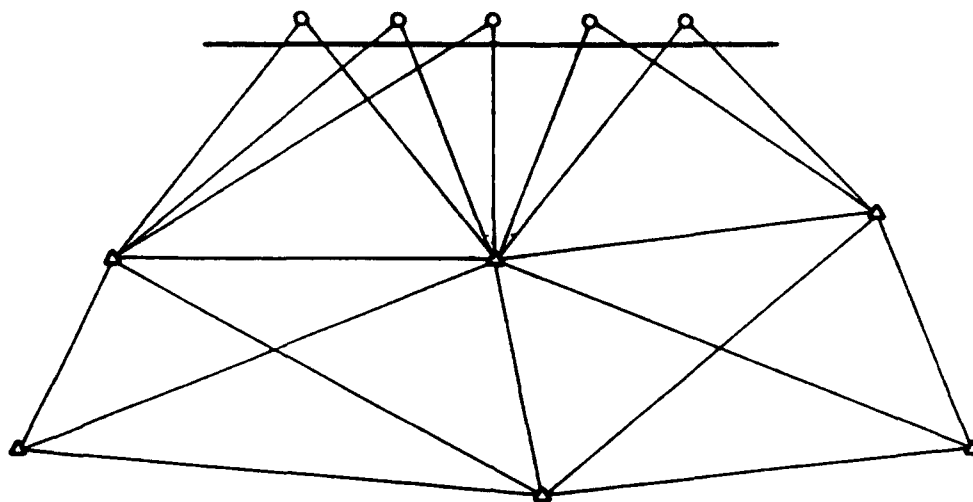
a) a reference or absolute monitoring network
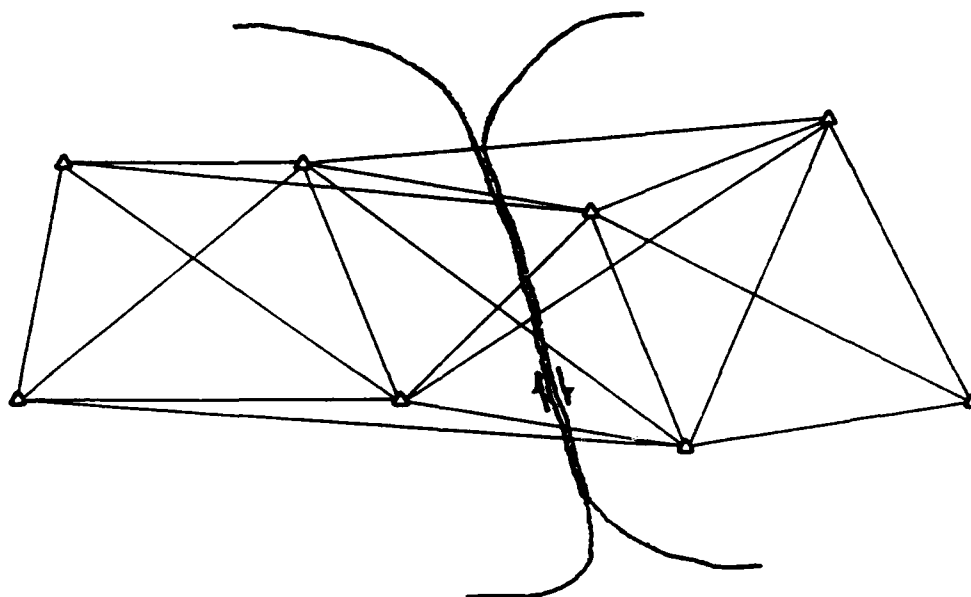
b) a relative monitoring network

Figure 1. The two types of intended monitoring networks

The occupied "stations" are denoted by triangles. Object points are denoted by circles.
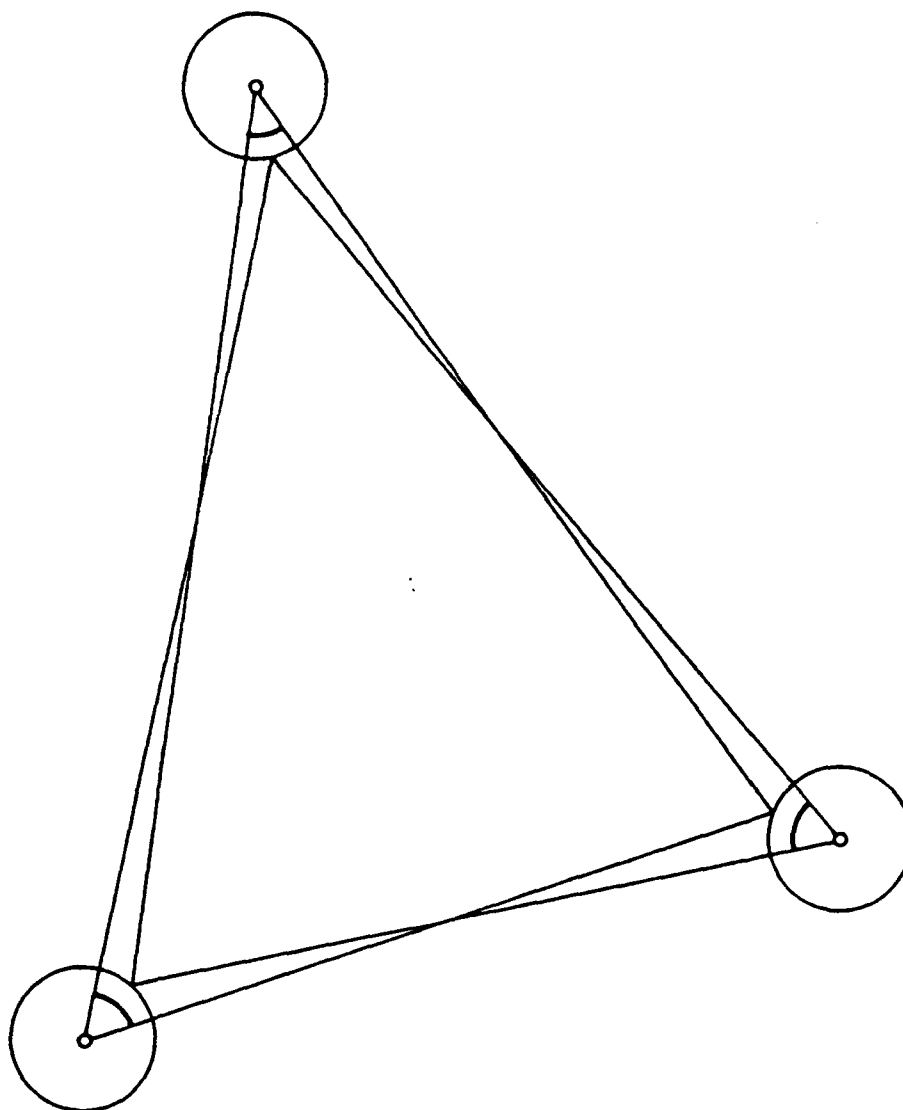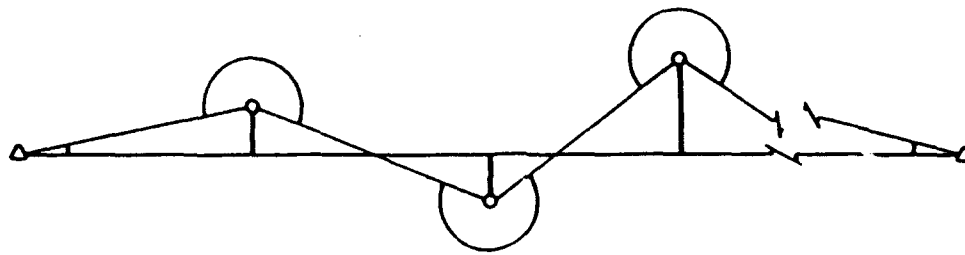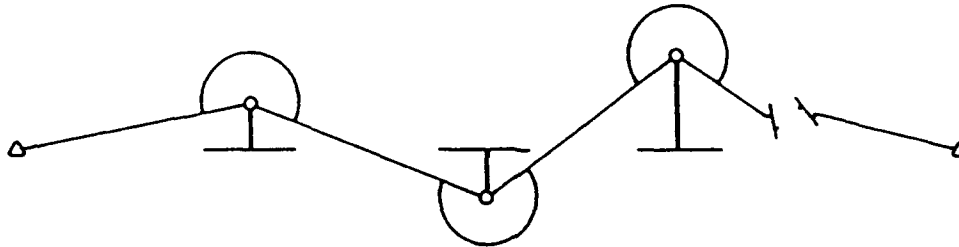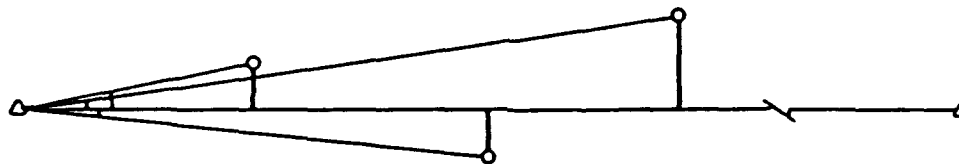
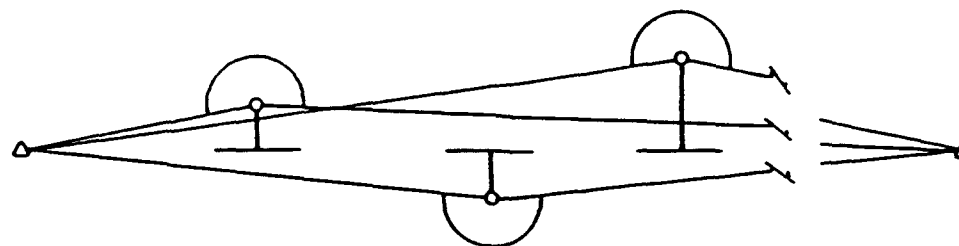**Figure 2. Permissible configuration defects in a deformation network**

a) closed traverse

b) open or fitted traverse

c) single station small angle

d) separate point included angle

Figure 3. Indirect lateral alignment by angular methods
Triangles denote reference stations. Circles denote object points.

GEOTECHNICAL AND OTHER NON-GEODETIC METHODS IN DEFORMATION MEASUREMENTS

GEOTECHNICAL AND OTHER NON-GEODETIC METHODS
IN DEFORMATION MEASUREMENTS

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Abstract

Instruments and methods of geodetic and photogrammetric surveys cannot satisfy all requirements of deformation measurements. Geotechnical and other non-geodetic instrumentation must be used when gathering information on local deformations inside the deformable body. The geotechnical instruments, such as extensometers, inclinometers, inverted pendulums, and various deformeters, can easily be adopted for continuous and telemetric data acquisition which makes them attractive also in the applications where geodetic surveys could have been used. There are hundreds of different types of geotechnical instruments available for deformation measurements. In this brief review, the instruments are grouped according to three main applications: measurements of extensions and strains; measurements of tilts and inclinations; and alignment surveys. Geotechnical methods for measurements of acting forces and properties of material are not included.

1. Introduction

The three earlier presentations in this Workshop gave a thorough review of the contemporary geodetic and photogrammetric techniques used in deformation surveys. Through interconnections among the monitoring stations, these techniques provide very useful information on the global deformation status of the monitored object and, in most cases, can also provide information on its rigid body translations and rotations with respect to reference points located outside the deformation area. Sub-millimetre accuracies over small areas and 10^{-6} relative accuracies over expanded bodies are achievable in the determination of relative displacements of the surveyed points. However, the instruments and methods for geodetic and photogrammetric surveys cannot satisfy all the requirements of deformation measurements.

Due to the necessity of having intervisibility between the survey stations, or between the monitoring stations and the satellites in the case of space techniques, the geodetic and photogrammetric surveys are limited only to open areas and cannot be used in detecting local deformations inside the material of the deformable body. Deformations in foundations or foundation rocks of large engineering structures and relative movements of different layers of soil or rock formations in slope stability studies are examples for which special measuring methods are needed. In these cases, geotechnical instruments, such as borehole inclinometers, various deformeters, extensometers, and inverted pendulums must be used. Geotechnical instrumentation can be easily adapted for continuous and telemetric data acquisition with an instantaneous display of the deformations which is very advantageous in comparison with the slow, labour intensive, terrestrial surveys. The expensive survey robots which were mentioned in the earlier presentation [Secord, 19876 cannot compete with the geotechnical instrumentation which does not require intervisibility between the stations and is operational practically in any environmental conditions. The geotechnical and particularly special non-geodetic instruments, such as hydrostatic levels, invar wire extensometers, laser interferometers, and diffraction aligning equipment, offer accuracies in the order of a few hundredths of a millimetre in localized measurements.

For the above reasons, the trend in the last few years has been to adopt geotechnical and special non-geodetic instrumentation for deformation surveys not only in the areas which are inaccessible for geodetic and photogrammetric surveys but also in studies where geodetic monitoring networks could have been used. If it continues, this tendency may lead to an unhealthy situation in which the usefulness of the geodetic and photogrammetric surveys will become underestimated by those who favour the geotechnical instrumentation.

Here, one should point out to those who advocate the use of geotechnical methods

that the geotechnical instrumentation also has weak points despite the aforementioned and indisputable advantages. First of all, the measurements are very localized and they may be strongly affected by local disturbances (noise) which do not represent the actual deformations. Since the local observables are not geometrically connected with observables at other monitoring stations, any global trend analysis of the deformations is much more difficult than in the case of geodetic surveys unless the observing stations are very densely spaced. The geomechanical, rock mechanics, and structural engineers, who are the main users of the geotechnical instruments, are usually less acquainted than survey engineers and geodesists with the evaluation of the measuring data and calibration of measuring instruments. On the other hand, most surveyors demonstrate complete ignorance in the use of other than geodetic techniques.

One could cite many examples of deformation analyses being conducted on the same object separately by both survey engineers and other engineers using only their own survey data obtained with their own instrumentation. Both groups have not fully recognized the advantages and disadvantages of various measuring techniques and, perhaps, they have not known how to combine geodetic and geotechnical results into an integrated analysis. The problem of integrated analysis is the subject of three other presentations of this Workshop which, hopefully, will help in the development of a better cooperation between the various engineering and scientific groups.

This presentation is directed mainly toward survey engineers to give an overview of basic geotechnical and other non-geodetic special techniques, their applications and compatibility with the surveying and geodetic methods.

2. Classification of Geotechnical and Special Non-Geodetic Instrumentation

The geotechnical instrumentation may be divided into three main groups, according to the three main purposes of the measurements:

(1) determination of physical properties of the deformable material,

(2) determination of acting forces (loads) and internal stresses,

(3) determinations of changes in dimensions and shape (geometrical deformations).

Since from the point of view of survey engineers only the third group of instruments is of direct interest, the two other groups will not be discussed in this brief review. However, from the point of view of the physical interpretation (see later presentation by Chen and Chrzanowski) and design of the geometrical measurements, the first two groups are just as important.

In the first group, the main problems of measurements is the laboratory and in situ determinations of the elastic parameters of the material (Young modulus, Poisson ratio), thermal coefficient of expansion, viscosity coefficient, and time-dependent behaviour. In the second group, various load cells are used to measure static and dynamic loads, various types of piezometers are employed to measure pore water pressure, and built-in temperature sensors help in the determination of the loads due to the thermal expansion of the material. Of special importance in the second group is measurement of in situ stresses (overcoring methods, various pressure meters) which similar to the determination of the in situ Young modulus still requires a development of better and more accurate methods than are presently available.

The third group of instrumentation which will be discussed in more detail in this review encompasses literally hundreds of different types and models of instruments produced by a large number of manufacturers. Appendix I lists some of the producers and distributors of the instruments which will be mentioned in this presentation. An excellent review of geotechnical instrumentation is given by Hanna [1985] who cites about 1200 references to his book which, in itself, is over 800 pages.

Very often, due to special requirements of each individual geomechanical problem, special instruments must be produced only for the given project. In many applications, the speed and simplicity of the measurements are of a higher priority than the high accuracy and also simple mechanical devices (e.g., convergence meters) may be developed in any local workshop depending on the ingenuity of the engineer in charge of the project. Thus, hundreds of various types of instruments are available for measurements of extensions and strains, tilts, deflections, curvings, bendings, etc. For a general review, only three basic types of measurements will be covered.

(1) Measurements of extensions (changes in distances) and strains with:
- (a) wire extensometers and strainmeters
- (b) rod extensometers
- (c) laser interferometers.

(2) Measurements of changes in tilts and inclinations with:
- (a) precision tiltmeters
- (b) borehole inclinometers
- (c) suspended and inverted plumb lines
- (d) hydrostatic levelling.

(3) Measurements of changes in alignment with:
- (a) mechanical methods
- (b) direct optical and laser alignment

(c)   laser diffraction method.

Many instruments used for the above measurements are purely mechanical with manual readout systems. In order to adapt them to automatic and continuous data recording, electric transducers can be employed using, for instance, linear potentiometers, differential transformers, and self-inductance resonant circuits. A detailed description of the transducers is beyond the scope of this presentation, however, any textbook on applied electronics gives the principles of various transducers. In general, when choosing the type of transducer for automatic data acquisition one should consult with an electronics specialist on which type would better suit the purpose of the measurements in the given environmental conditions. For instance, the output signals from linear potentiometers are a function of the length of the transmission cable, while the readout (frequency output) of the inductance types, is independent of the distance from the sensor. However, the inductance transducers are sensitive to changes of environmental temperature and require temperature compensation.

Londe [1982] gives interesting remarks on various sensors. He favours the self-inductance and vibrating wire transducers over any other types for their accuracy, linearity of the output, and durability. The inductance type is comprised of two electric circuits, for instance, two concentric coils which form a resonant circuit of frequency

$$f = 1/(2\pi\sqrt{LC})$$

where L is the self inductance and C is capacitance. Any change in the relative position of the two coils alters the inductance and hence the resonant frequency of the inner circuit. Linear displacements of a few micrometers may be detected.

The vibrating wire transducers work on the principle that the natural frequency of the wire varies with the square root of the tension of the wire. Some other types of transducers will be mentioned when discussing some particular instruments.

## 3.   Measurement of Changes in Distances

3.1 General Remarks

Various types of instruments, mainly mechanical and electro-mechanical, are used in geomechanics, rock mechanics, and structural engineering to measure changes in distances in order to determine settlement or upheaval of soil, convergence of walls in engineering structures, and underground excavations, strain in rocks and in man-made materials, separation between rock layers around driven tunnels, slope stability, and movements of structures with respect to the foundation rocks. Depending on its particular application, the same instrument may be named an extensometer, strainmeter, convergencemeter, or

1. Surface wire extensometer
2. Subsurface slope extensometer for localisation of slide planes
3. Mobile borehole inclinometer
4. Multi-point borehole deflectometer
5. Multiple point rod extensometer
6. Disk load cell to monitor pre-stressed anchors
7. Soil spy for surface monitoring of slide planes
8. Multi-point hydraulic level gauge
9. Invar wire extensometer
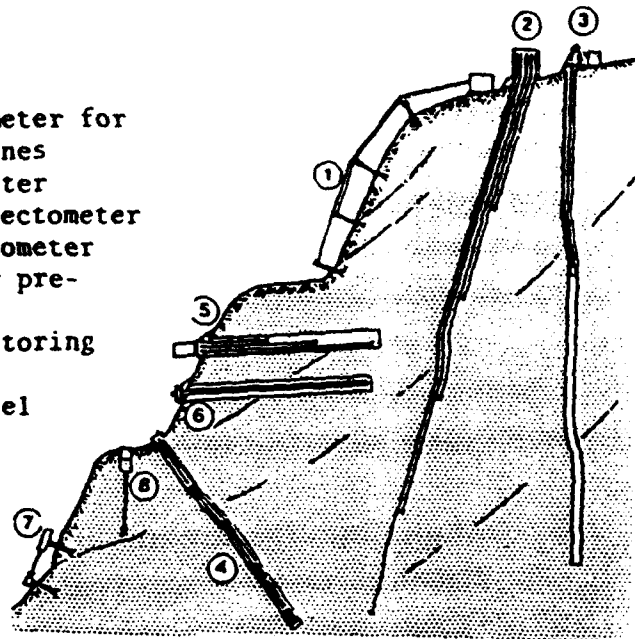
a) Slope monitoring instrumentation
   (Müller et al.,1977)
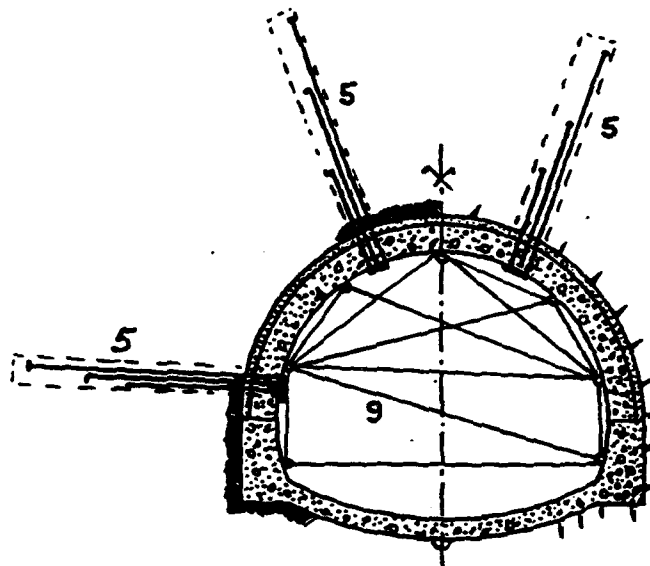
b) Tunnel monitoring

FIG.3.1. Examples of the use of wire and rod extensometers

fissuremeter. Perhaps a common name, distance extensometer or distance variometer, could be accepted for all of them. Figure 3.1 shows some typical applications of extensometers.

The various instruments differ from each other by the method of linking together the points between which the change in the distance is to be determined and type of sensor employed to measure the change. The links in most instruments are mechanical, such as wires, rods, or tubes. The sensors usually are either mechanical, such as calipers, dial gauges, or the aforementioned electric transducers.

A separate type of instrument, interferometers, are also discussed in this section because they belong to the category of extensometers and strainmeters although they are more popular in metrological measurements than in engineering deformation surveys.

3.2 Wire and Tape Extensometers

Figure 3.2 shows samples of various wire extensometers. Some of them, namely, ISETH Distometer, CERN Distinvar, and Roctest's Distomatic (not shown) have already been discussed in the earlier presentation on surveying methods [Secord, 1986]. All three instruments use invar wires and special constant tensioning devices which, if properly calibrated and properly used, can give accuracies of 0.05 mm or even better in measurements of changes of distances over lengths from about 1 m up to 100 m. Invar is a capricious alloy and must be handled very carefully to avoid sudden changes in the length of the wire. Several months before the wires are used for the first time they must be aged (to remove initial stresses) by numerous rewindings in different directions and preheating to 100°C and slowly cooled. During the measurements they must be calibrated before and after each use. Gervaise [1978] gives a good account of all the problems which may be encountered with invar wire measurements. When small changes in temperature are expected or a smaller precision (0.1 mm to 1 mm) is required, then steel wires or steel tapes are more comfortable to use.

The Rock Spy (Figure 3.3) also uses invar wire for measurements of small relative movements (up to 60 mm) across fissures, joints, and cracks. It employs the inductance transducer and a precision tension spring. However, the tension of the wire is not as constant as, for instance, in the Distometer. However, an accuracy between 0.02 mm to 0.2 mm is claimed by Telmac and Roctest Ltd. depending on the base length (up to about 20 m). A special arrangement (patent pending) is used for the temperature compensation.
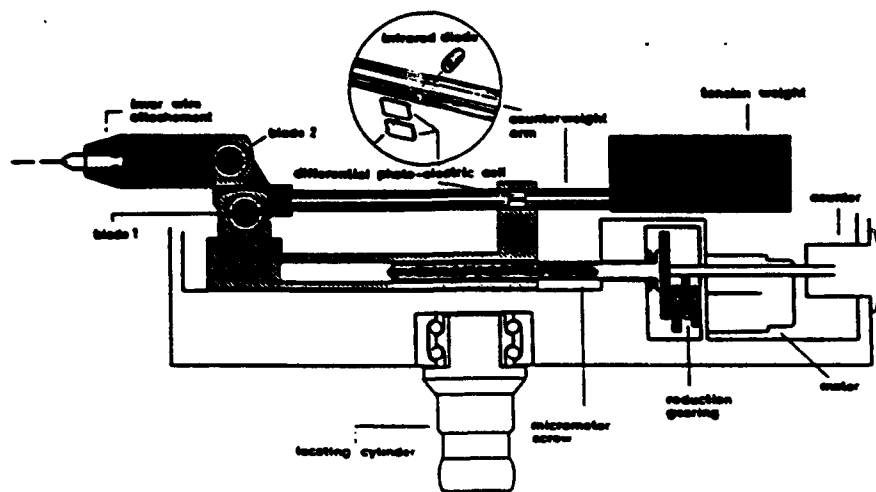
Tensioned wires have been used extensively in boreholes in ground movement studies. Figure 3.4 shows a multiple point wire extensometer for measurements of differential movements at different depths. Each wire is anchored at one end to the ground
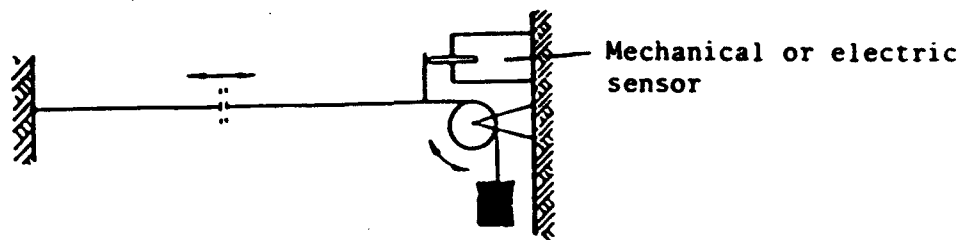
a) basic principle
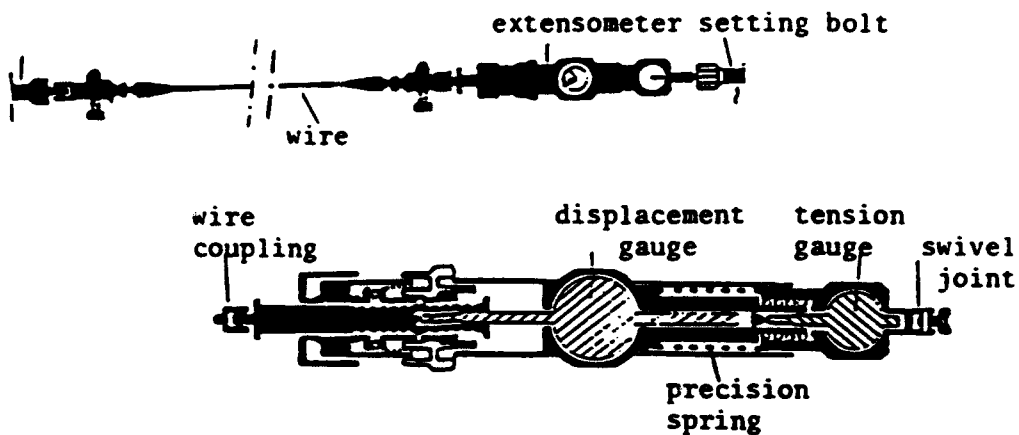
b) ISETH Distometer (Schnädelbach,1980)
   with a spring controlled constant tension

c) CERN Distinvar with a knife-edge constant tension
   (horizontal use only), (Gervaise,1978)
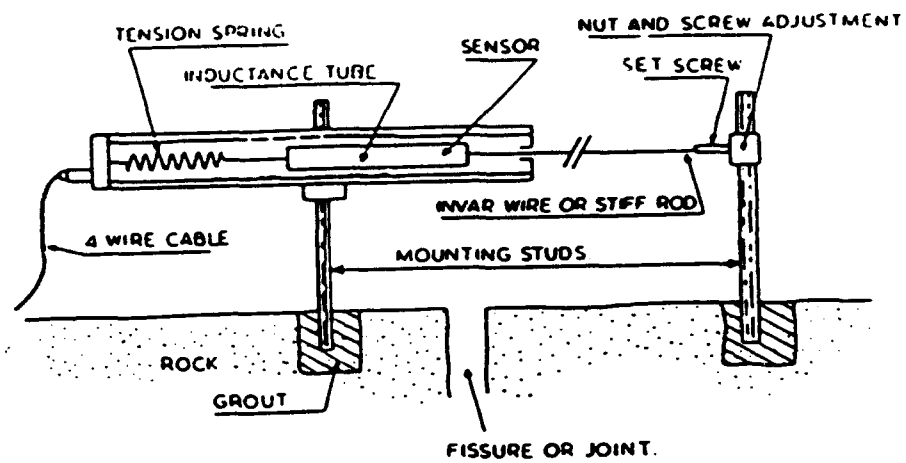
FIG.3.2. Invar wire extensometers

TENSION SPRING

INDUCTANCE TUBE

SENSOR

NUT AND SCREW ADJUSTMENT

SET SCREW

INVAR WIRE OR STIFF ROD

4 WIRE CABLE

MOUNTING STUDS

ROCK

GROUT

FISSURE OR JOINT.

FIG.3.3. Rock Spy (Telemac)

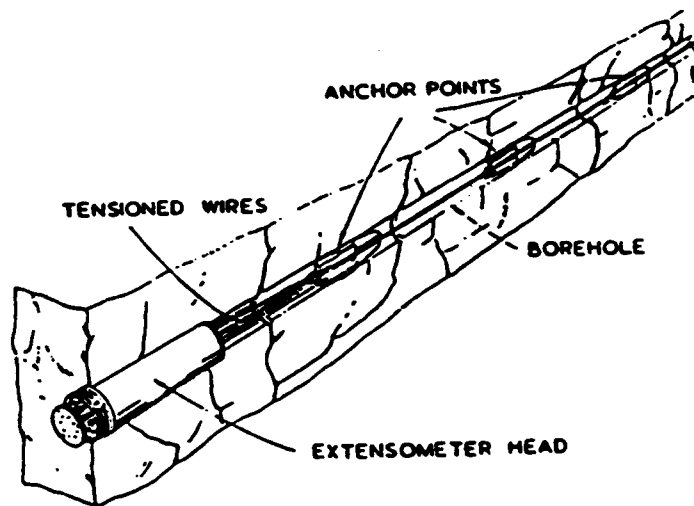ANCHOR POINTS

TENSIONED WIRES

BOREHOLE

EXTENSOMETER HEAD

FIG.3.4. Multiple point wire extensometer (Hanna,1985)

at different depths and tensioned at the measuring head on the surface by spring cantilevers. Any change in the distance produces cantilever deflections which can be sensed either manually or by using transducers. The installation of wires in deep boreholes is cumbersome and requires good experience. Use of rods in this case is preferred.

If an extensometer is installed in the material with a homogeneous strain field, then the measured change Δl of the distance l gives directly the strain component $\varepsilon = \Delta l/l$ in the direction of the measurement. In order to determine the total strain tensor in a plane (two normal strains and one shearing), a minimum of three extensometers must be installed in three different directions.

Special high precision strainmeters of a short base (up to a few decimetres) are available for strain measurements in structural material and in homogeneous rocks. The aforementioned vibrating wire transducers may directly be used for that purpose. An example is a vibrating wire strain gauge available from Irad Gage. The instrument employs a 150 mm steel wire in which the changeable resonant frequency is measured. An accuracy of one microstrain (10^{-6}) is claimed in the strain measurements which corresponds to 0.15 μm relative displacements of points over the distance of 150 mm.

3.3 Rod and Tube Extensometers

Figure 3.5 shows typical single-point and multiple-point rod extensometers. Steel, invar, aluminum, or fibreglass rods of various lengths may be used depending on the application. The multiple point measurements in boreholes or in trenches may be made using either a parallel arrangement (Figure 3.5b) of rods anchored at different distances from the sensing head, or a string (in series) arrangement (Figure 3.5) with the sensors mounted at different depths. The multiple-point rod extensometer model E-10 (Figure 3.7) from Irad Gage is an example of the parallel multi-rod instrument. Up to 6 rods may be used in a 75 mm borehole to a total length of 200 m and total range of 300 mm with a resolution of 0.01 mm. The actual accuracy depends on the temperature corrections. The dial indicator readout may be replaced by potentiometric or other transducers with digital readout systems. Several models are available from different companies, for example, from SINCO, Roctest, and others. The self-explanatory Figures 3.6 and 3.8 show schematically two models of the string type (in series) extensometers: BOF-EX (Roctest) with differential transformer (LVDT) transducers, and Distofor (Telemac) with the self-inductance transducers. In both models a precision of ±0.02 mm accuracy is claimed. The actual accuracy depends, of course, on the accuracy of calibration of the transducers and temperature compensation.
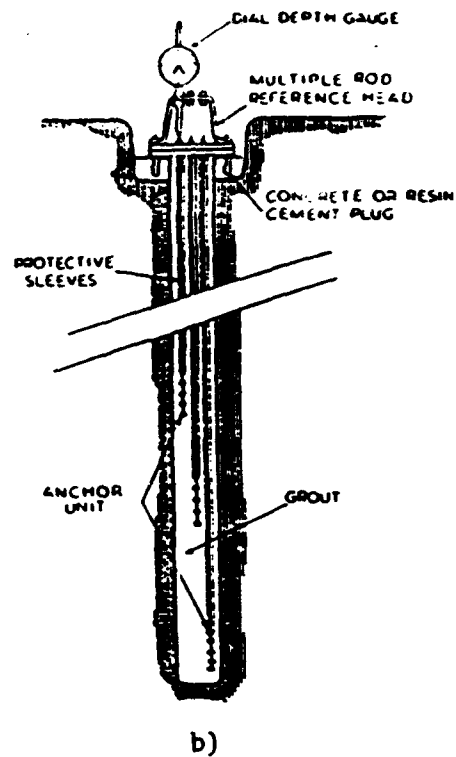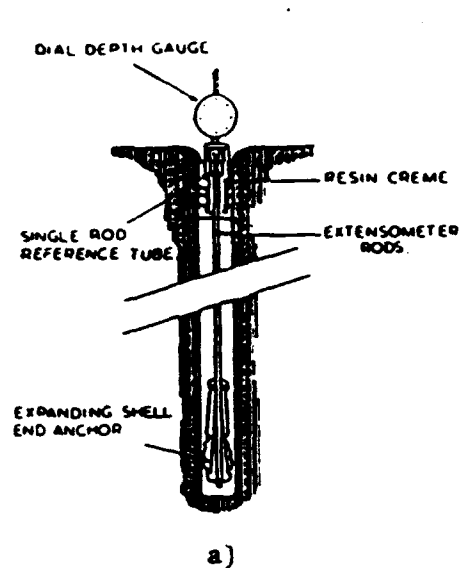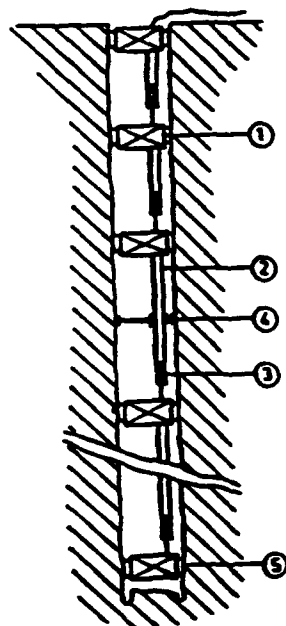
FIG.3.5. Rod extensometers: a) single-point type
b) multiple point.(Hanna,1985)
Note: in some models the dial depth gauges are
replaced by electric sensors.

1. Mechanical anchor
2. Extension rod
3. Electric transducer (LVDT)
4. Spacer ring
5. Bottom anchor

FIG.3.6. Multiple point (in series)
rod extensometer model
BOF-EX (Telemac,Roctest)

Flat Wedge Borehole Anchor

Dial Indicator Readout

Measurement Rods

Quick-setting Cement

Alternative Anchor (Groutable)

PVC Pipe Coupling

Extensometer Head

Protective Cap

Measurement Surface

The IRAD GAGE Multiple-Point Rod Extensometer is designed for the measurement of deformations in soil or rock.
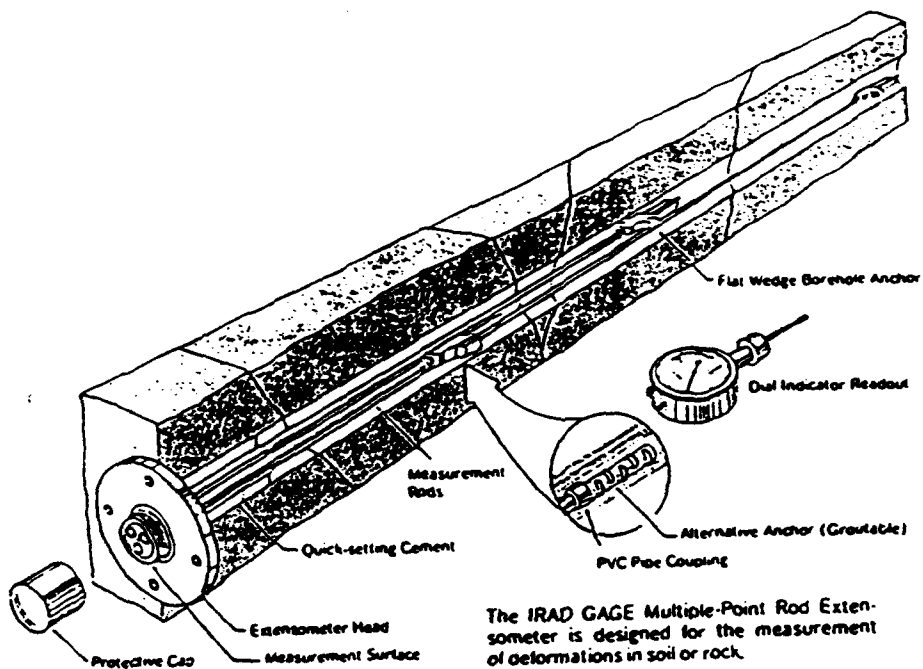
FIG.3.7. Multiple point rod (parallel) extensometer
model E-10 (Irad Gage)
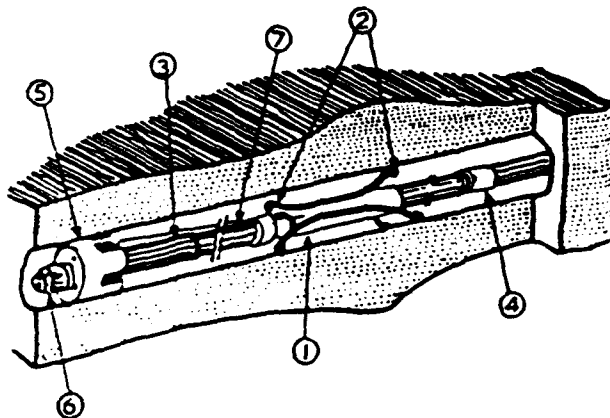
1 REFERENCE RING	5 FLUSH HEAD
2 ANCHOR SPRINGS	6 WATERTIGHT COUPLING
3 ROD	OR SEALED OUTLET
4 WATERTIGHT COUPLING	7 SPRING RELEASE

FIG.3.8. Multiple point rod (in series) extensometer
model DISTOFOR (Telemac)

A torpedo type of borehole extensometer, Extensofor, is available from Telmac and Roctest (Figure 3.9). The instrument consists of a 28 mm diameter torpedo of 1.55 m length with an inductance sensor at each end. Reference rings on the casing are spaced within the length of the torpedo. The sensors and reference rings form the inductance oscillating circuits. The torpedo is lowered in the borehole and stopped between the successive rings recording changes in distances between the pairs of rings with a claimed accuracy of 0.1 mm. Boreholes up to several hundred meters long can be scanned.

Several other models of rod extensometers with various transducers are available for mounting in boreholes or in trenches in soil or along slopes by connecting, for instance, a series of concrete monuments with rods equipped with transducers.

Telescopic tubes may replace rods in some special applications. For instance, a simple convergence meter COR-P (Roctest) for measuring deformations in underground mines has been developed in Canada in cooperation with the Canada Centre for Mineral and Energy Technology (CANMET). The instrument consists of two telescoping tubes maintained together by a friction ring. The relative movement of the tubes, spanned between the floor and roof of the opening, is sensed by a potentiometric transducer. The instrument can safely be used in gaseous mines, is very simple in operation, and gives resolution of 0.1 mm.

3.4 Interferometric Measurements of Linear Displacements

Michelson type (Figure 3.10) interferometers with laser as a source of the monochromatic radiation are becoming a common tool in precision displacement measurements. A light radiation of wavelength λ is split into s_1 signal to a fixed reference mirror and s_2 signal to a movable retro-reflector. The combined reflected signals s_1 and s_2 create a pattern of interference fringes. A movement of the retro-reflector by $\lambda/4$ causes a shift of the pattern by one fringe. Thus a change in the distance to the moving retro-reflector is determined by continuous counting of the fringes.

Theoretical limits of the resolution in the interferometric distance measurements is of the order of $10^{-4}\lambda$ because it is the limit of the accuracy with which the interference fringes may be interpolated using photoelectric sensors. One has to remember, however, that the interferometric distance measurements are affected by the atmospheric refractivity in the same way as all the EDM systems. Therefore, the accuracy depends on the accuracy of the determination of the density of air along the optical path of the distance measurements.

The continuous counting of fringes requires a very precise alignment of a rail on which the retro-reflector could smoothly move along the measured distance. Counting of the fringes is done electronically. Presently available counting systems allow for the

1. Sensors

2. Guide
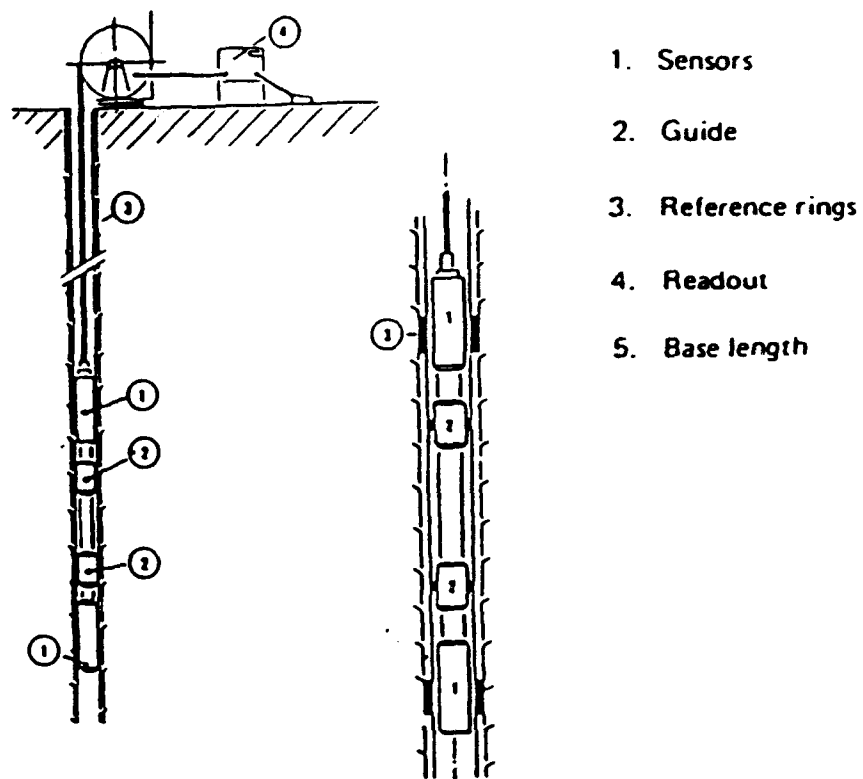
3. Reference rings

4. Readout

5. Base length

FIG.3.9. Continuous (torpedo type) borehole extensometer
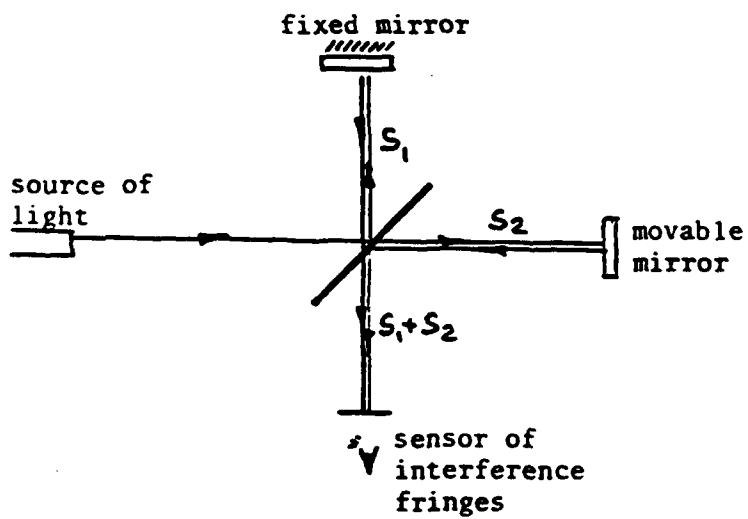model EXTENSOFOR (Telemac,Roctest)

FIG.3.10. Michelson type interferometer

maximum speed of 30 cm/s of the mirror motion.

Use of interferometry in long distance measurements is limited by two factors:

- frequency bandwidth of the radiation
- thermal turbulence of air.

The bandwidth of the narrowest spectral lines from conventional sources of radiation, such as obtained from gaseous electrical discharges, is about $10^{-6}\lambda$. Therefore, the interference fringes blur out at distances greater than $10^6\lambda$ due to the internal interference between individual rays of light having different wavelengths. For instance, the maximum distance of interferometric measurements with the radiation obtained from Kr^{86} (the primary standard of length) is only about 60 cm because $\lambda = 0.606$ μm. The bandwidth of the gaseous lasers may easily be controlled to $10^{-9}\lambda$ or even up to $10^{-12}\lambda$ [Baird, 1969] allowing for interferometric measurements over distances of several kilometres. However, due to the thermal turbulence of air which disturbs fringe counting, the practical range of the laser interferometry in open air is a maximum of 100 m only, unless the measurements are performed in an evacuated tube.

Accuracy of 4×10^{-10} in linear changes over a distance of 800 m (Figure 3.11) has been achieved in practice using a laser strain meter (LSM) developed at the University of California in San Diego [Berger and Loveberg, 1969]. This consists of a long arm Michelson type interferometer. The whole 800 metre optical path of the long arm, except two short 30 cm distances at both ends, is enclosed in an evacuated (10^{-3} Torr) pipe with quartz windows. The 30 cm distances and two end piers with the optical and electronic components of the interferometer are kept under highly controlled temperature and pressure conditions.

Development of lasers which can simultaneously produce two signals of different frequencies f_1 and f_2 allow for measurements of linear displacements by using the Doppler shift method [Dukes and Gordon, 1970]. Hewlett Packard Model 5525B laser interferometer is an example of the double frequency interferometer. Its principle of operation is shown in Figure 3.12.

The laser emits a coherent red laser beam comprised of two signals of frequency f_1 and f_2 of opposite polarizations. After being expanded and collimated, a portion of the beam is split into a fixed internal reflector and directed into a photodetector. An optical filter passes only f_2 along this path. The transmitted beam reflects from the movable external reflector, and is filtered to contain only f_1. Any axial movement of the reflector produces a Doppler shift of the signal frequency about f_1. This Doppler shifted signal combines with f_2 at the photodetector to generate "fringes". Denoted by the Δf the Doppler shift of the frequency, the velocity of the movement of the reflector can be calculated from:
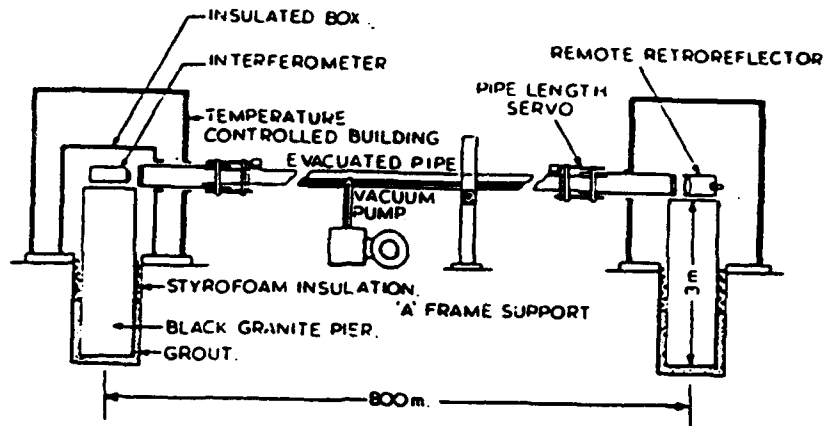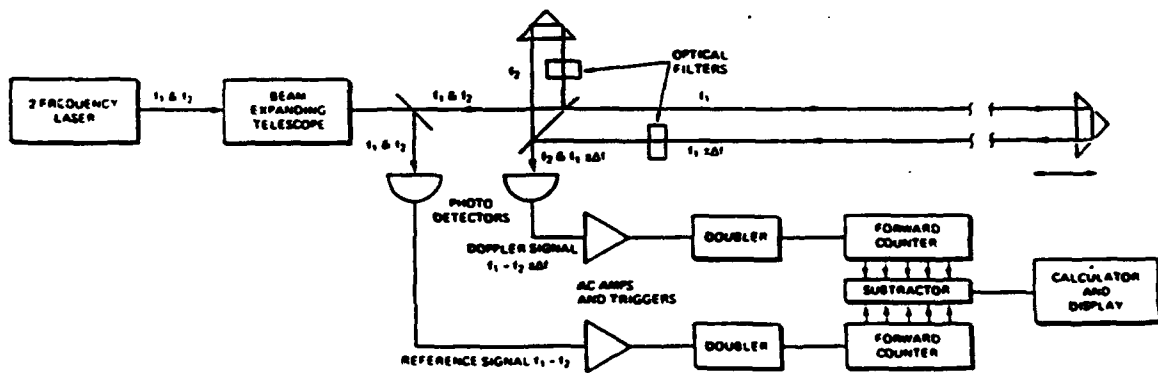
FIG.3.11. Laser Strain Meter (Berger and Loveberg,1969)

FIG.3.12. Double frequency industrial interferometer model HP 5525B
(Hewlett Packard)

$$v_f = \pm \, \Delta f \, \lambda/4 \; ,$$

and the distance travelled by the reflector is obtained through the integration:

$$\Delta s = \lambda/4 \int \Delta f \, dt \; .$$

Therefore, in this type of interferometer, the frequencies rather than the fringes of different intensity are counted.

One should emphasize that the influence of air turbulence on the performance of double frequency interferometers is much smaller than when using the conventional single frequency interferometry. The HP 5525B model has the following specifications:

accuracy:	0.5 μm per metre ±2 counts in the last digit
resolution:	0.1 μm in normal mode and 0.01 μm in the X10 mode of operation
range:	60 m, depending on the environment
maximum measuring velocity:	0.3 m/s
velocity measurement accuracy:	0.04 mm/s
operating temperature:	0°C to 55°C.

During the measurements, the temperature of the air and the instrument and barometric pressure must be recorded. Corrections for the velocity of light and for the expansion of the material must be calculated. As an option, an automatic compensator may be connected to the instrument. The instruments have outputs for printer, computer, Fourier analyser, etc.

A newer model HP 5526A of the laser Doppler interferometer [Kahmen, 1978] allows also for measurements of angular deflection with an accuracy of ±0.1" and alignment offsets with a resolution of 0.1 μm over 30 m.

4. Measurement of Tilt and Inclination

4.1 Tiltmeters and Inclinometers

The measurement of tilt is usually understood as a determination of a deviation from a horizontal plane, while inclination is interpreted as a deviation from vertical. Thus the same instrument which measures tilt at a point can be called either a tiltmeter or an inclinometer, depending on the interpretation of the results.

From the point of view of the basic principle of operation, the tiltmeters may be divided into:

- liquid (including spirit bubble type)
- vertical pendulum
- horizontal pendulum.

As far as the sensitivity of tilt is concerned, we have two distinct groups:

- geodetic or geophysical special tiltmeters of a resolution of 10^{-8} or even 10^{-9} radian
- engineering tiltmeters of resolutions between 0.1" to a few tens of seconds of arc, depending on the required range of tilt to be measured.

The first group includes instruments which are used mainly for geophysical studies of earth tide phenomena and tectonic movements. Here we have, for example, the Verbaander-Melchior [Melchior, 1983] horizontal pendulum tiltmeter, Rockwell Model TM-1 [Cooper and Schmars, 1974] liquid-bubble type, and many others. This category of instrument requires extremely stable monumentation and a controlled environment. There are very few engineering projects where instruments which are so sensitive may be needed. Deformation measurements of underground excavations for the storage of nuclear waste may be one of the few possible applications. Figure 4.1 shows an example of a tiltmeter developed for that purpose by the Auckland Nuclear Accessory Co. in New Zealand. In this instrument, the tilt is proportional to the change in capacitance between the electrodes and the mercury surface. This tiltmeter, with a total range of 15 minutes of arc, is claimed to give a resolution of 10^{-9} radian which corresponds to a relative vertical displacement of only 6×10^{-7} mm over the base length of 587 mm.

There are many models of liquid or pendulum types of tiltmeters of reasonable price ($2000 to $3000) which satisfy most the needs of engineering surveys. One of them is the Electrolevel with the spirit bubble principle (Figure 4.2) in which the movement of the bubble is sensed by three electrodes. The tilt produces change in differential resistivity between the electrodes which is measured by means of the Wheatstone bridge. The resolution of 0.25" is obtained over a total range of a few minutes of arc. Many other liquid types of tiltmeters with various ranges (up to 30°) are available from various companies.

The self-explanatory Figure 4.3 shows an example of a pendulum type electronic level, the Talyvel, which gives an accuracy of ±0.5" over a total range of ±8'.

Of particular popularity are servo-accelerometer types of tiltmeters with horizontal pendulum. They offer ruggedness, durability, and low temperature operation. The output signal (volts) is proportional to the sine of the angle of tilt. Figure 4.4 shows the Schaevitz servo-accelerometer. It employs a small-mass horizontal paddle (pendulum) which due to the force of gravity tries to move in the direction of tilt. Any resultant motion is converted by position sensors to a signal input to the electronic amplifier whose current output is applied to the torque motor. This develops a torque equal and opposed to the original. The torque motor current produces a voltage output which is proportional to the sine of the angle of tilt. The typical output voltage range for tiltmeters is ±5 V which corresponds to

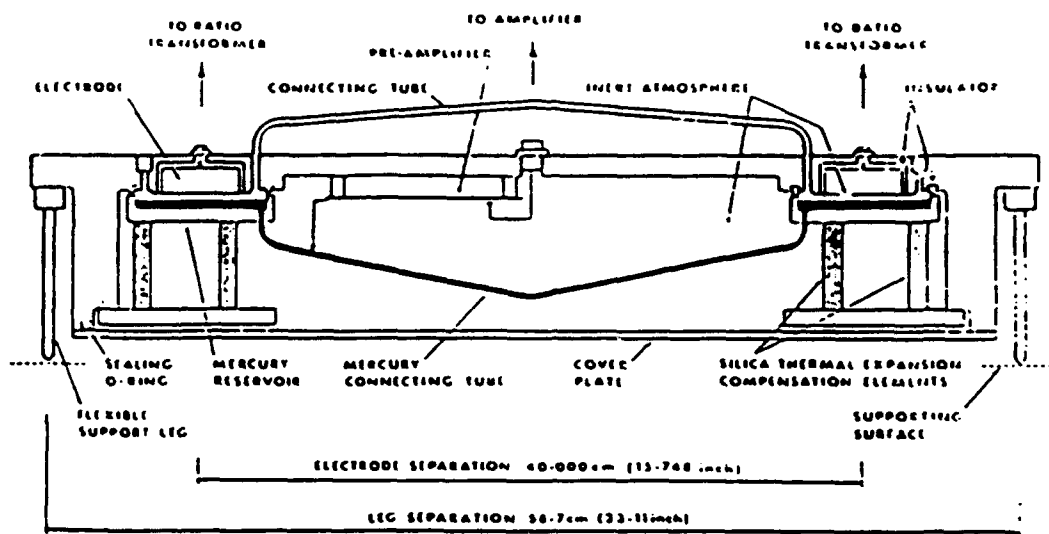FIG.4.1. High precision mercury tiltmeter (Auckland Nuclear
Accessory Co.)
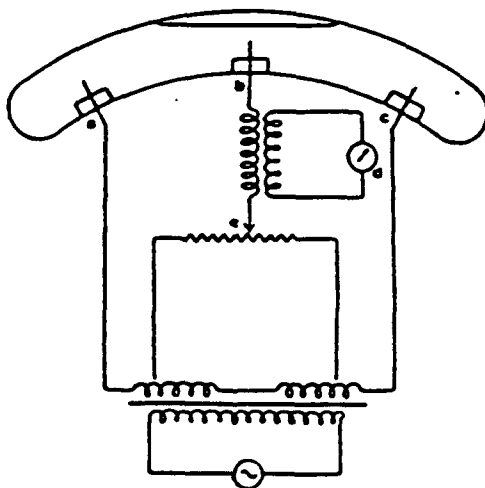

FIG.4.2. Electrolytic tiltmeter ELECTROLEVEL
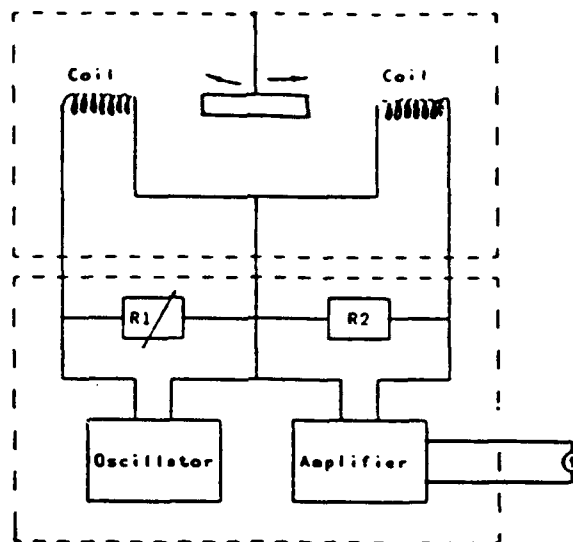(British Aircraft Co.,Cooper,1971)

FIG.4.3. TALYVEL pendulum tiltmeter (Rank Organization U.K.)
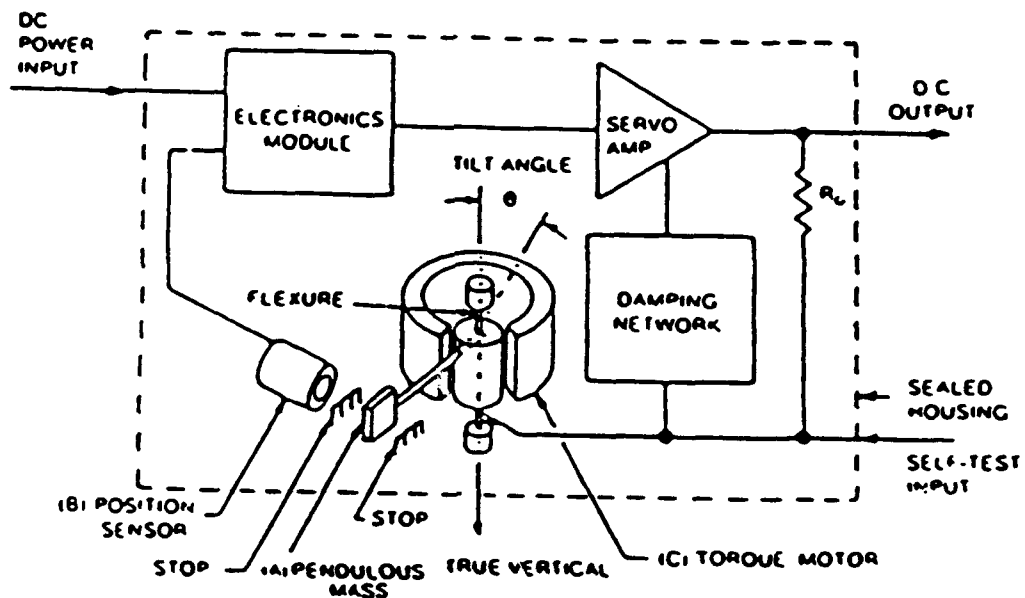(Caspary and Geiger,1978)

FIG.4.4. Servo-accelerometer type of tiltmeter (Schaevitz)

the maximum range of the tilt. Thus the angular resolution depends on the tilt range of the selected model of the tiltmeter.

There are many factors affecting the accuracy of tilt sensing, not just the resolution of the readout. A temperature change produces dimensional changes of the mechanical components, changes in the viscosity of liquid in the electrolytic tiltmeters and of the dampening oil in the pendulum type tiltmeters. Also the electric characteristics change with the temperature. Drifts of tilt indications and fluctuations of the readout may also occur. For a price, most of the errors may be compensated for, or their effect may be made linear thus allowing for an easy calibration.

Within the inexpensive models, the compensation for the aforementioned sources of error is not very sophisticated, and those tiltmeters may show non-linear output versus temperature, erratic drifts, etc. Therefore very thorough testing and calibration are required even when the accuracy requirement is not very high [Chrzanowski et al., 1980].

Compensators of vertical circles of precision theodolites work on the same principle as some engineering tiltmeters. The liquid compensator of the Kern E3 electronic theodolite gives repeatability better than 0.3". Therefore, the theodolite may also be used as a tiltmeter giving the same accuracy as, for instance, the Electrolevel.

Tiltmeters have a wide range of applications. A series of tiltmeters if arranged along a terrain profile in a mining area may replace geodetic levelling in the determination of ground subsidence [Chrzanowski et al., 1980] as is shown in Figure 4.5. For instance, the subsidence of point 4 with respect to point 1 may be calculated from the observed tilt angles and known distances between the points as:

$$h_4 = s_1(\alpha_1 + \alpha_2)/2 + s_2(\alpha_2 + \alpha_3)/2 + s_3(\alpha_3 + \alpha_4)/2 .$$

Similarly, deformation profiles of tall buildings may be determined by placing a series of tiltmeters at different levels of the structure [Kahmen, 1978].

In geomechanical engineering, the most popular application of tiltmeters is in slope stability studies and in earth dams. Torpedo type bi-axial inclinometers are used to scan boreholes drilled to the depth of an expected stable strata in the slope. By lowering the inclinometer on a cable with marked intervals and taking readings of the inclinometer at those intervals, a full profile of the borehole and its changes may be determined through repeated surveys. SINCO, Terra Technology, and other producers of tiltmeters provide special borehole inclinometers (50 cm or 2 feet long) of the torpedo type with wheels. Special borehole casing (vinyl or aluminum) with guiding groves for the wheels is available. Usually the servo-accelerometer type inclinometers are used with various ranges of inclination measurements, for instance, $\pm 6°$, $\pm 53°$, or even $\pm 90°$. A 40 m deep borehole if measured every 50 cm with an inclinometer of only 100" accuracy should allow for
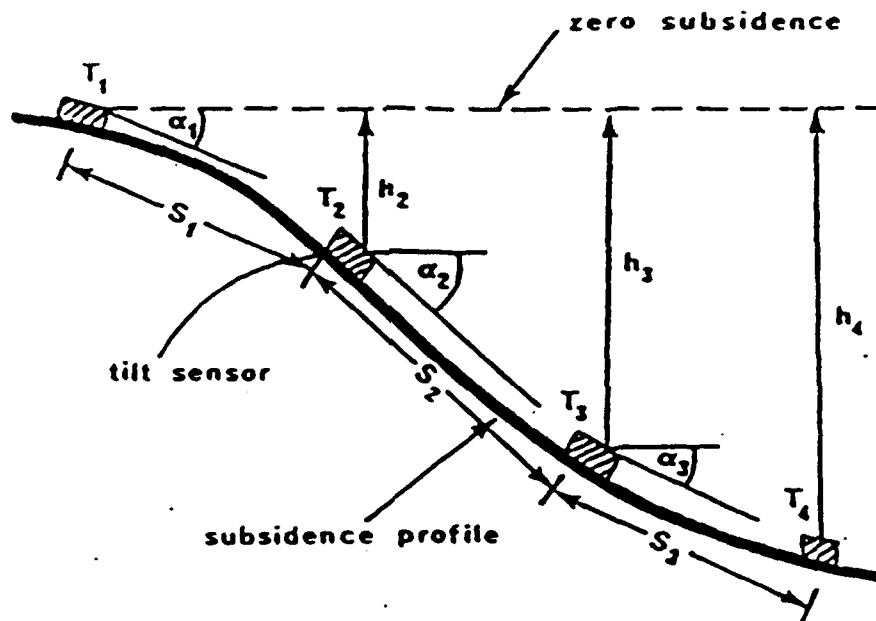
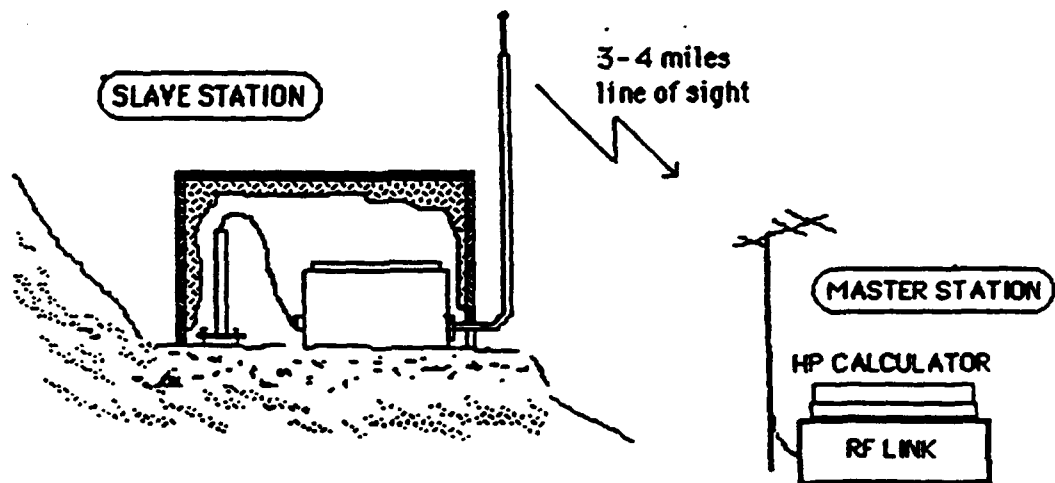FIG.4.5. Ground subsidence determination using tiltmeters

FIG.4.6. Telemetric monitoring of ground movement in Sparwood,B.C. using UNB/CANMET telemetry system and Terra Technology bi-axial tiltmeters (Chrzanowski et al.,1980)

determination of the linear lateral displacement of the collar of the borehole with an accuracy of 2 mm.

In cases of a difficult access to the monitored area and/or a need for continuous data acquisition, the tiltmeters or borehole inclinometers are left in place at the observing station with a telemetry monitoring system. Figure 4.6 shows an example of a station set-up of a telemetric monitoring of ground subsidence in a mining area near Sparwood, B.C., using a telemetry system developed for CANMET at the University of New Brunswick [Chrzanowski et al., 1980; Chrzanowski and Fisekci, 1982]. Terra Technology bi-axial servo-accelerometer tiltmeters of ±1° range were used in the study. The same telemetry system has been adapted for monitoring SINCO borehole inclinometers (Figure 4.7) at Syncrude Canada Ltd. oilsands mining in northern Alberta [Chrzanowski et al., 1986a]. The telemetry system may work with up to 256 field stations. Each station accepts up to 6 sensors (not only tiltmeters but any type of instrument with electric output).

4.2 Hydrostatic Levelling

If two connected containers (Figure 4.8) are partially filled with a liquid, then the heights h_1 and h_2 are related through the hydrostatic equation:

$$h_1 + P_1 / (g_1 \, \rho_1) = h_2 + P_2 / (g_2 \, \rho_2) = \text{const.}$$

where P is the barometric pressure, g is gravity, and ρ is the density of the liquid which is a function of temperature.

The above relationship has been employed in hydrostatic levelling, as schematically shown in Figure 4.9. The air tube connecting the two containers eliminates a possible error due to different air pressures at two stations. The temperature of the liquid should also be maintained constant because, for instance, a difference of 1.2°C at two containers may cause an error of 0.05 mm in Δh determination for an average h = 0.2 m and t = 20°C. Figures 4.10 and 4.11 show examples of two typical hydrostatic instruments which are used in precision levelling. The ELWAAG 001 is a fully automatic instrument with a travelling (by means of an electric stepping motor) sensor pin which closes the electric circuit upon touching the surface of the liquid. A standard deviation of 0.03 mm is claimed over distances of 40 m between the instruments [Schnödelbach, 1980]. Another automatic system (Figure 4.12), the Nivomatic Telenivelling System, is available from Telemac or Roctest Ltd. The Nivomatic uses the inductance transducers which translate the up and down movements of the floats into electric signals (frequency changes in the resonant circuit). An accuracy of ±0.1 mm is claimed over a 24 m length.

The hydrostatic levels may be used in a form of a network of permanently installed instruments to monitor tilts of large structures. Robotti and Rossini [1984] report on a
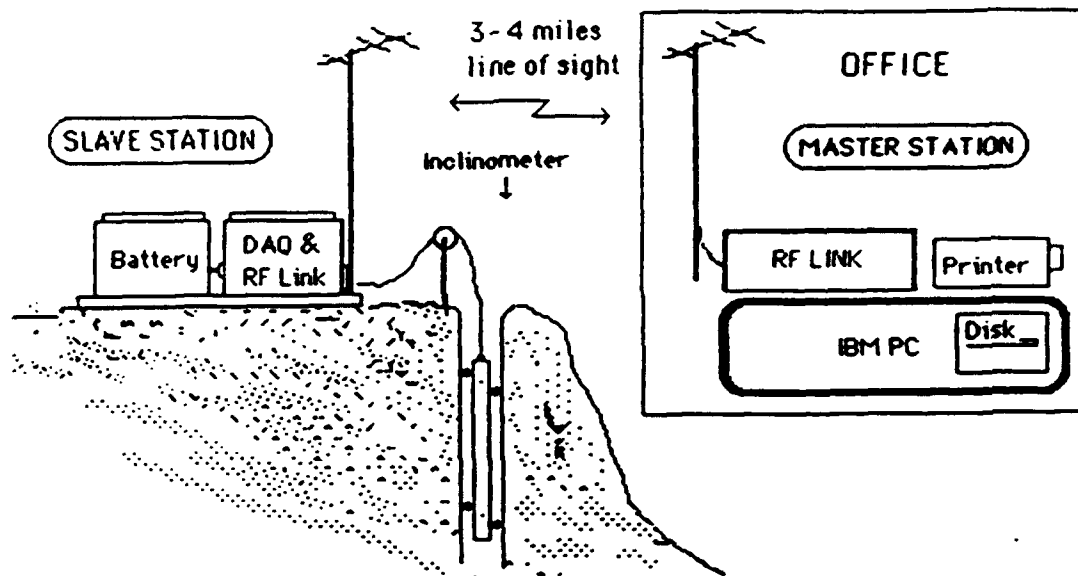
FIG.4.7. Telemetric monitoring of slope stability using
UNB/CANMET telemetry system and SINCO borehole
inclinometer (Chrzanowski et al.,1986)

$$h + P/g\rho = \text{const.}$$
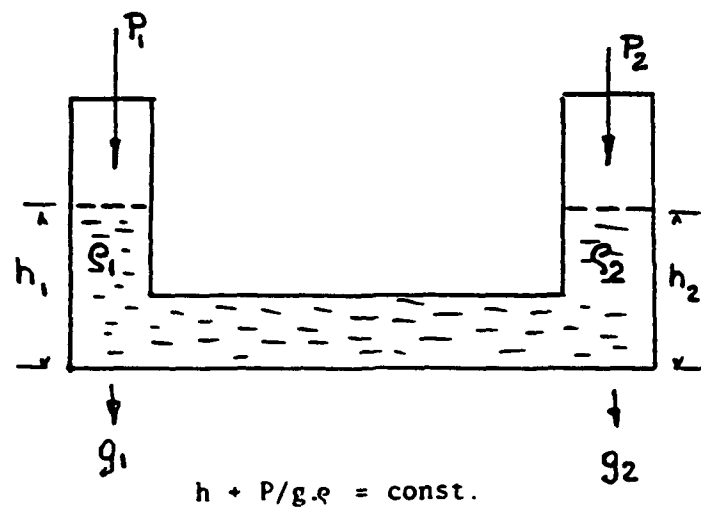
FIG.4.8. Hydrostatic equilibrium of connected vessels
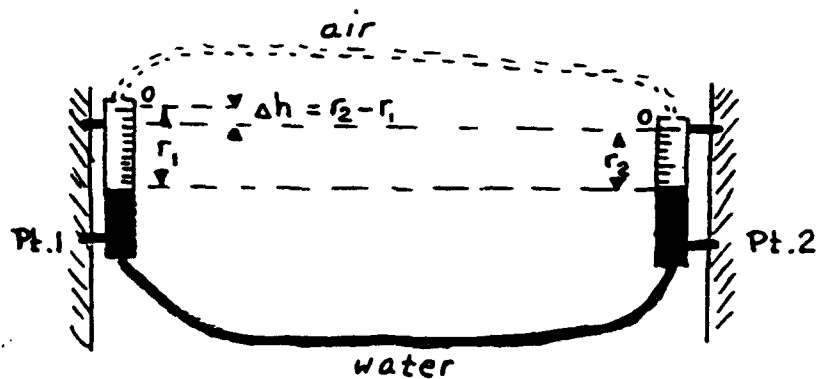
FIG.4.9. Principle of hydrostatic levelling
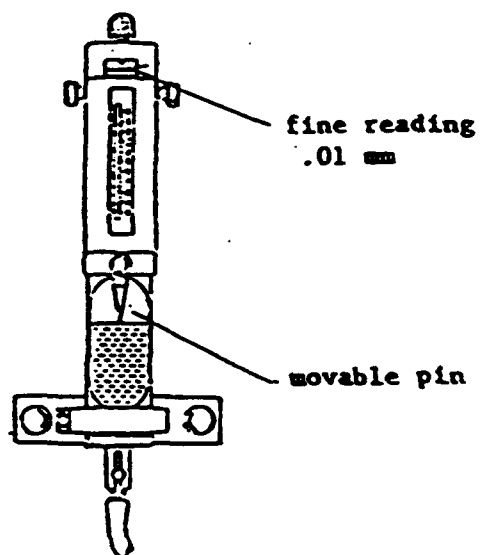
FIG.4.10. Hydrostatic level of Freiberger Prazisionmechanik
with a manual-optical readout system
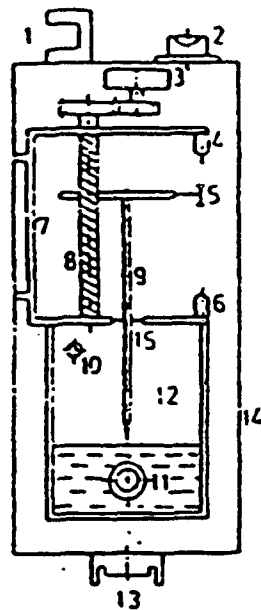
3 – Electric stepping motor (0.01 mm steps) with a counter

4 – Upper switch (by a contact with 5)

6 – Lower switch (by a contact with 5)

9 – Travelling pin

10 – Connector for an air pressure tube (to equilize air pressure in both instruments)

11 – Water tube connector

FIG.4.11. ELWAAG 001 - automatic hydrostatic level (Bayernwerke AG,Munich)
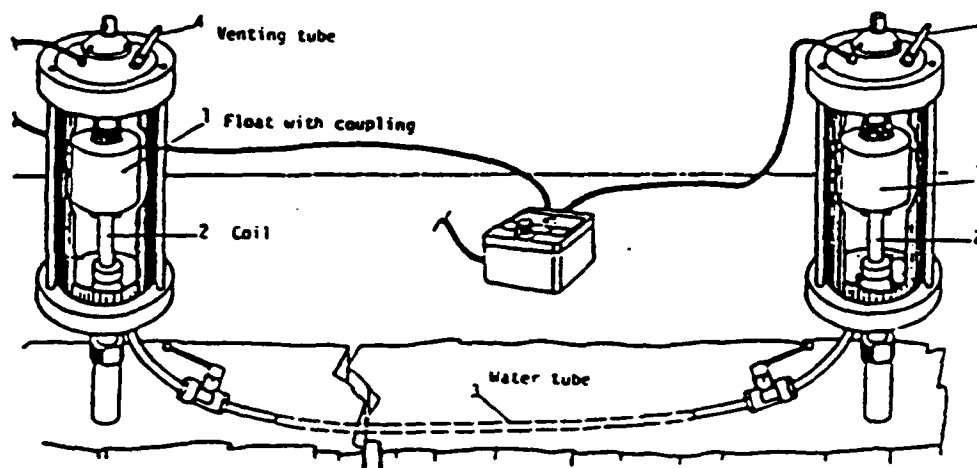
FIG.4.12. NIVOMATIC Telenivelling System with continuous data acquisition (Telemac,Roctest)

DAG network monitoring system available from SIS Geotecnica (Italy) which offers an accuracy of about 0.01 mm using also inductive transducers in the measurements of liquid levels.

Various systems of double liquid (e.g., water and mercury) settlement gauges based on the principle of hydrostatic levelling are used for monitoring power dams [Hanna 1985] with extended networks of connecting tubing.

The instruments with direct measurement of the liquid levels are limited in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in elevation of the water levels. Pneumatic pressure cells or pressure transducer cells may be used. Hanna [1985] gives numerous examples of various settlement gauges based on that principle.

4.3 Suspended and Inverted Plumb Lines

Two types of mechanical plumbing are used in controlling the stability of vertical structures (Figure 4.13):

(1)   suspended plumb lines
(2)   floating plumb lines also called inverted or reversed plumb lines.

Typical applications are in monitoring of power dams and monitoring of stability of reference survey pillars. The suspended plumb lines are also commonly used in mine orientation surveys and in monitoring the stability of mine shafts.

The floating plumb lines have become standard instrumentation in large dams (e.g., Hydro Quebec uses them routinely). Their advantage over the suspended plumb lines is in a possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rocks which may be considered as stable. In the case of power dams, the depth of the anchors must be 30 m or even more below the foundations in order to obtain the absolute displacements of the surface points. The main problem in the case of the floating plumb lines is drilling of vertical boreholes so that the vertical wire of the plumb line would have freedom of motion. A special technique for drilling the vertical holes has been developed at the Hydro Quebec Co. [Dubreuil and Hamelin, 1974].

Several types of recording devices which measure displacements of structural points with respect to the vertical plumb lines are produced by different companies. The simplest are mechanical or electro-mechanical micrometers with which the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table with an accuracy of ±0.2 mm or better. Travelling microscopes may give the same accuracy. Automatic sensing and recording is possible with a Telecoordinator from Huggenberger Ltd. Telemac Co. (France) developed a system, Telependulum (marketed in Canada by
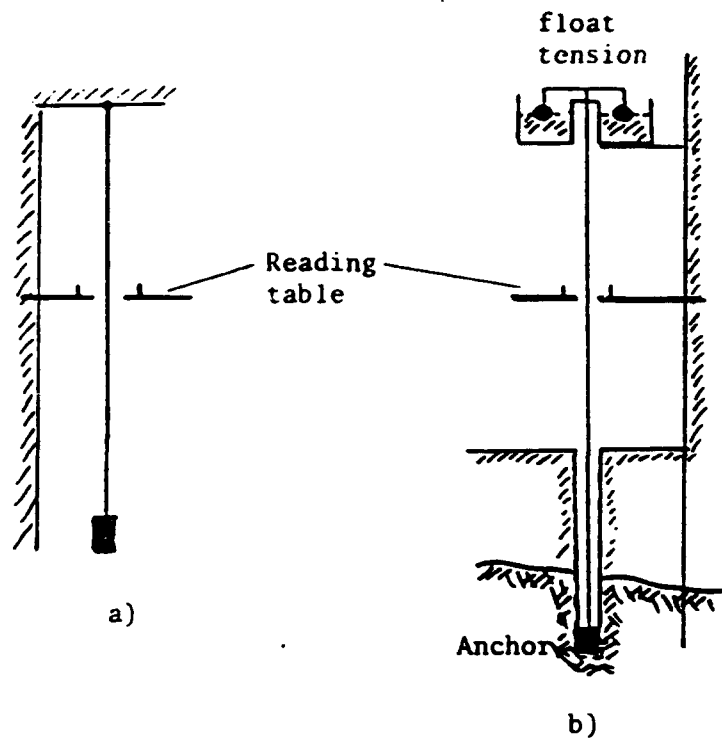
FIG.4.13. Inclination measurements with mechanical plumb lines
a) suspended plumb line b) floating (reversed) plumb-line
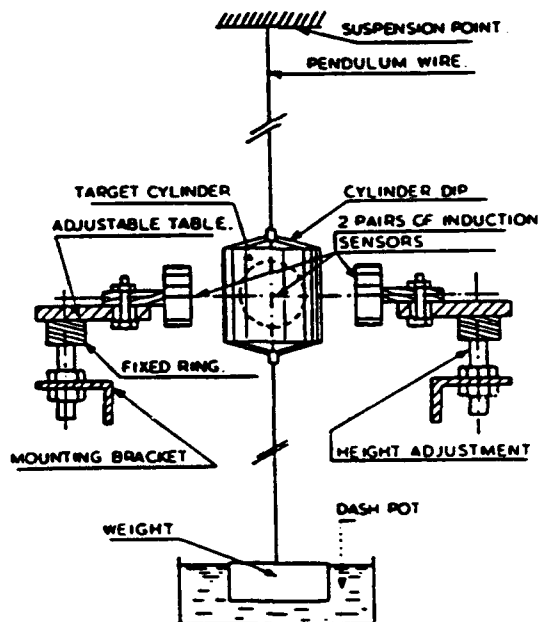
FIG.4.14. Inductive sensor TELEPENDULUM (Telemac) for remote
inclination measurements with mechanical plumb lines

Roctest), for continuous sensing of the position of the wire with remote reading and recording. Figure 4.14 illustrates the basic principle of the system. The rigidly mounted reading table supports two pairs of induction type proximity sensors arranged on two mutually perpendicular axes. A hollow cylinder is fixed on the pendulum wire at the appropriate level, passing through the centre of the table and between the sensors. Changes in the width of the gap (resolution ± 0.01 mm) between the target cylinder and the sensors are detected by the corresponding changes in the induction effect.

An interesting Automated Vision System has been developed by Spectron Engineering (Denver, Colorado). The system uses solid state electronic cameras to image the plumb line with a resolution of about 3 μm over a range of about 75 mm. Several plumb lines at Glen Canyon dam and at Monticello dam, near Sacramento, California, use the system.

Two sources of error, which may sometimes be underestimated by the user, may strongly affect the plumb line measurements:

    (1)   influence of air currents

    (2)   spiral shape of the wire.

If the wire of the plumb line with weight Q is exposed along a length h (Figure 4.15) to an air current of velocity v at a distance H from the anchored point, then the plumb line is deflected by an amount:

$$e = f_o \, h \, H/Q \, ,$$

where f_o is the acting force of air current per unit length of the wire. The value of f_o may approximately be calculated [Chrzanowski et al., 1967] from:

$$f_o = 0.08 \, d \, v^2/Q \, ,$$

where d is the diametre of the wire, v is in metres per second, and Q is in kilograms. Example: H = 50 m, h = 5 m, d = 1 mm, Q = 20 kg, and v = 1m/s (only 3.5 km/h):

$$e = 1 \text{ mm.}$$

The second source of error, which is usually underestimated in practice, is that the spiral shape of the wire (Figure 4.16) affects all wires unless they are specially straightened or suspended for a prolonged time (say, several months).

If the wire changes its position (rotates) between two campaigns of measurements, then the recorded displacements could have a maximum error of 2r. The value of r can be calculated [Chrzanowski, 1967] from

$$r = (\pi d^4 E) / (64 \, RQ) \, ,$$

where E is Young's modulus of elasticity ($\sim 2 \times 10^9$ kg/cm^2 for steel), and R is the radius of the free spirals of the unloaded wire which, typically, is about 15 cm for wires up to 1.5 mm diameter.
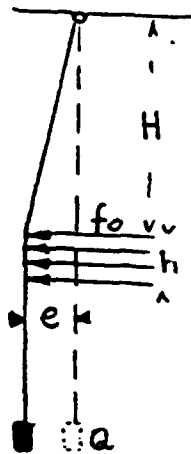
Fig.4.15. Influence of air current on mechanical plumb lines
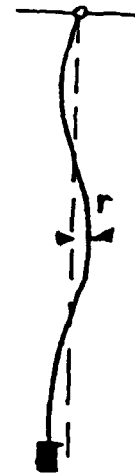
FIG.4.16. Error due to the spiral shape of the wire
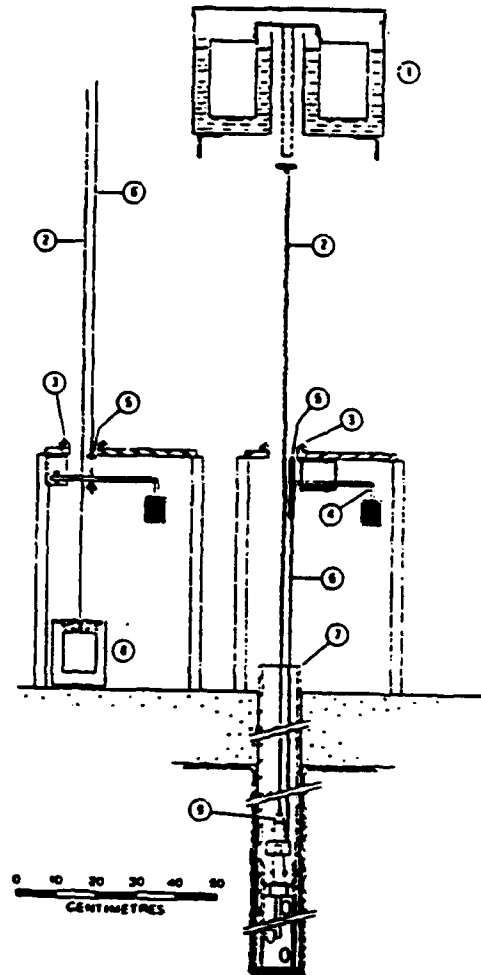
1. Tank with float
2. Wire
3. Reading table
4. counterweight tension
5. Bench mark
6. Levelling wire
7. Casing (60 mm diam.)
8. Tank and weight (11 kg)
9. Removable anchor-head

FIG.4.16. Double pendulum with wire bench marks at Hydro-Quebec (Boyer and Hamelin,1985)

Example: For a plumb wire with d = 1 mm, R = 15 cm, and Q = 20 kg,

r = 0.3 mm.

If one plumb line cannot be established through all levels of the monitored structures, then a combination of suspended and inverted plumb lines may be used (Figure 4.16) as long as they overlap at least at one level of the structure. At Hydro Quebec, the drill holes of the plumb lines are also used for monitoring height changes by installing tensioned invar wires, as shown in Figure 4.16 [Boyer and Hamdin, 1985].

5. Alignment Methods

5.1 General Remarks

The alignment surveys cover an extremely wide spectrum of engineering applications from tooling industry through measurements of amplitude of vibrations of engineering structures to deformation monitoring of several kilometres long nuclear accelerometers. Each application may require different specialized equipment for measuring offsets of monitored points from the reference line of alignment. Some previously described methods of deformation surveys, for instance, inclination measurements of engineering structures using mechanical plumb lines, could also be classified as the alignment surveys, because the plumb line serves as a reference line for offset measurements of selected points which is a basic procedure of the alignment measurements.

The alignment surveys may be classified according to the method of establishing the reference line:

*   geodetic methods in which the reference line is defined by coordinates of two reference points of the alignment line;

*   mechanical methods in which stretched wire (steel, nylon, etc.) establishes the reference line;

*   direct optical, in which the optical line of sight or a laser beam "mark" the line; and

*   diffraction method in which the reference line is created by projecting a pattern of diffraction slits.

There are also some other methods more suitable to a laboratory rather than for field applications, for instance, interferometric measurements of deflections [Chrzanowski, 1974] which are not discussed in this review. The geodetic methods (traversing, small angle deflections, etc.) which utilize the conventional surveying techniques with theodolite and fixed targets were described in the earlier presentation on surveying methods [Secord,

1986].

Both direct optical and diffraction methods are affected by the atmospheric refraction in the same way as any other surveying method based on optical instrumentation. In the case of alignment in a horizontal plane, the influence of lateral refraction in the open atmosphere is usually much smaller than in the vertical plane near the ground and very often of a random rather than systematic character. However, in industrial applications, for instance when using an alignment method in galleries of a hydro-electric power house, the heat from generators may produce large gradients of temperature across the optical reference line. When the constant gradient of temperature, dT/dy, is persistent across the length s of the alignment line then the maximum error of alignment can approximately be calculated from:

$$\Delta y_{max} = s^2 P(dT/dy) \, 10^{-5} / T^2 \,,$$

which is valid for an average wavelength of the visible light, where P is the barometric pressure in [mb], T is the temperature in [K], and dT/dy is in [°C/m]. The above equation has been derived on the basis of a more detailed discussion in Chrzanowski et al. [1976]. For example, if dT/dy = 0.3°C/m, s = 200 m, T = 300K, and P = 1000 mb, the expected maximum error of alignment (in the centre of the line) would be 1.3 mm or 6×10^{-6} of s.

In deformation surveys, the influence of refraction can, of course, be neglected if the same gradients of temperature appear in all repeated surveys.

The mechanical methods, though not affected by the refraction, may be influenced by other environmental conditions such as air flow [Chrzanowski, 1983] and the spiral shape of the wire similarly as already discussed in the inclination measurements with plumb lines.

Therefore, the choice of the alignment method requires a careful analysis of the environmental conditions.

5.2 Mechanical Methods

The methods with tensioned wires as the reference lines have found many applications due to their simplicity, high accuracy, and easy adaptation to continuous monitoring of deformations over distances up to a few hundred metres.

For example, Jakob [1969] and Zill [1970] describe an application of the method for deformation surveys in large dams. They used steel wires with 1 mm diameter suspended at both ends and stretched with a tension of 10 to 50 kg. The offsets at intermediate points were measured electrically, using inductive sensors. In order to dampen the vibrations of the wire and at the same time to suspend the wire, a float swimming in oil has been used. This type of equipment is appropriate for a permanent installation in the galleries of large
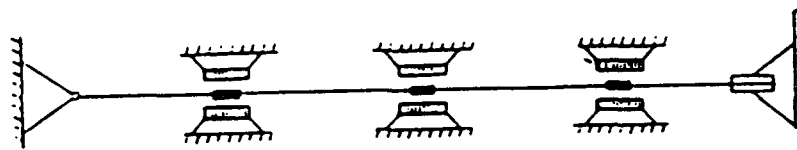
FIG.5.1. Mechanical alignment with inductive transducers
(Pelzer,1976)
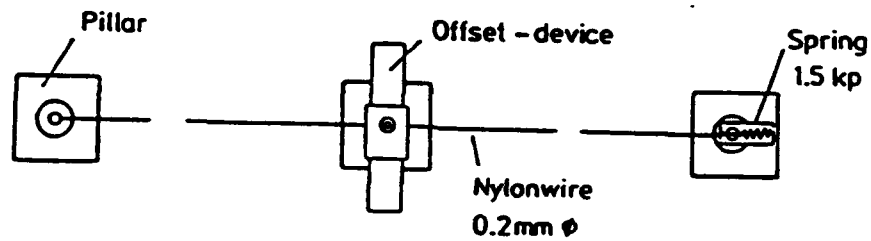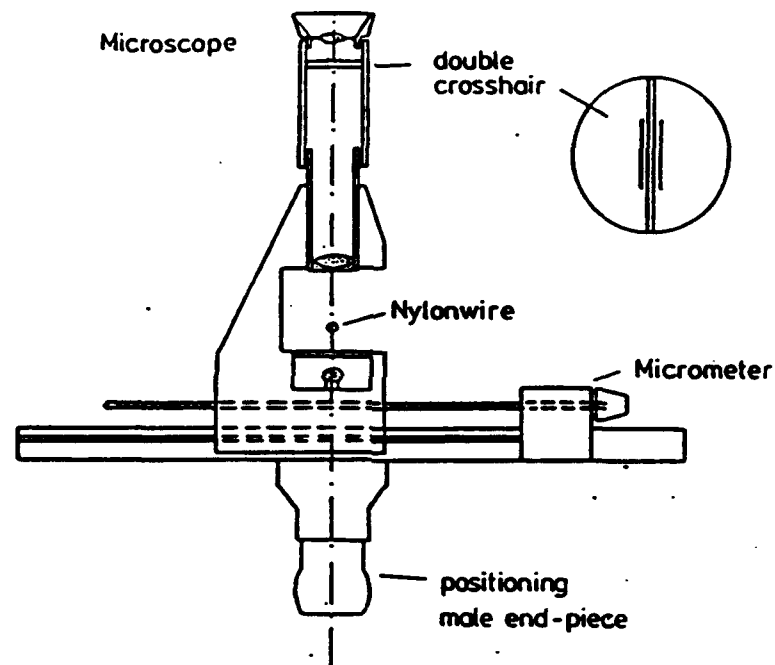
FIG.5.2. CERN alignment system with nylon lines
(Gervaise,1974)

dams or any other long structures. Accuracies of ±0.10 mm are reported. Pelzer [1976] describes a similar system as shown in Figure 5.1.

A high precision alignment system using a nylon line of 0.2 mm diameter with tension of 1.5 kg was developed [Gervaise, 1974] at the aforementioned European Centre for Nuclear Research (CERN). The system is composed of two centring devices for the suspension and tensioning of the nylon line and an offset measuring device consisting of a microscope and a micrometer, as shown in Figure 5.2. The system has successfully been used over distances up to 96 m with standard deviations between 0.035 mm to 0.070 mm in calm air conditions. Some difficulties were reported when measuring in air currents, and in those cases the nylon mechanical system had to be replaced with a laser aligning method [Gervaise, 1976].

The principle of the mechanical alignment has found applications in geomechanics for measuring ground movements transverse to the axis of a borehole [Hanna, 1985] using deflectometers. An example is shown in Figure 5.3. In this instrument, a tensioned wire passes over knife edges at various points along its length and transducers measure the angular change in direction of the wire at the knife edge. Thus relative movements or deflections are measured. Muller and Muller [1970] discuss the use of the deflectometer for earth dam instrumentation and quote an accuracy of ±0.04 mm with about 5 m spacing of the knife edges.

5.3 Direct Optical Alignment

The method utilizes either an optical telescope and movable targets with the micrometric sliding devices or a collimated (projected through the telescope) laser beam and movable photocentring targets. Besides the aforementioned influence of the atmospheric refraction, pointing and focussing are the main sources of error when using optical telescopes. The pointing error with properly designed targets [Chrzanowski, 1974] varies from 15"/M in the calm atmospheric conditions at night to 60"/M in daylight in average turbulent conditions, where M is the magnification of the telescope. The focussing error in precision telescopes may have an extreme value of one second of arc but it is practically eliminated if the survey is repeated from both ends of the alignment line in two positions (direct and reversed) of the telescope.

Special aligning telescopes with large magnification (up to 100x) are available from, among others, Fennel-Cassell (West Germany) and Zeiss-Jena (East Germany). Aligning telescopes for the tooling industry and machinery alignments are available in North America from Keuffel and Esser Company. When the optical line of sight is replaced by a collimated laser beam, then the accuracy of pointing may be considerably improved if

special self-centring laser detectors with a time integration of the laser beam energy are used [Chrzanowski, 1974]. The use of laser allows for an automation of the alignment procedure and for a continuous data acquisition. However, when using the laser beam directly as the reference line, one has to pay attention to the stability of the laser cavity. Due to thermal effects on the laser cavity, a directional drift of the laser beam as high as 4"/°C may occur [Chrzanowski and Ahmed, 1971]. The latter effect is decreased by a factor of M when projecting the laser through a telescope [Chrzanowski and Janssen, 1972].

5.4 Alignment with Diffraction Gratings

The method is illustrated in Figure 5.4. The pinhole source of a monochromatic (laser) light, the centre of the plate with diffraction slits, and the centre of the optical or photoelectric sensor are the three basic points of the alignment line. If two of the three points are fixed in their position, then the third may be aligned by centring the reticle on the interference pattern created by the diffraction grating.

It should be pointed out that movements of the laser and of its output do not influence the accuracy of this method of alignment because the laser serves only as a source of monochromatic light and not as the reference line. Therefore, any type of laser may be employed in this method, even the simplest and the least expensive ones, as long as the output power requirements are satisfied. Two types of diffraction gratings are commonly used: equidistant circular slits, and Fresnel zone plates (Figure 5.5). Both types have found a very wide use in precision displacement measurements.

Equidistant circular gratings are employed in a commercially available aligning system (the alignoscope) produced by Ahrend-Export in Holland. In this system, the diffraction plate produces a circular interference pattern which is observed by a telescope equipped with a circular reticle. The alignoscope has been used in several industrial projects and has given accuracies of the alignment close to 10^{-6} of the distance. Its range in the open atmosphere is limited, however, to a maximum of 100 m because of difficulty in identifying the centre of the interference pattern over longer distances in turbulent air.

The Fresnel zone plate employs such spacing and width of slits that all rays of the laser beam that produce negative interference after the diffraction are cut off by opaque circular zones. This can be done by designing the distances s between the centre of the plate and edges of the slits as equal to

$$s_m = \sqrt{m\lambda \, (ab \, / \, (a + b))} \, ,$$

where s_m is the radius to the mth edge as counted from the centre, a and b are distances from the source of light to the zone place and from the plate to the image-observing screen.

ELECTRICAL
INDUCTIVE   RIGID   KNIFE   TENSIONED
TRANSDUCER   HOUSING   EDGES   WIRE

FLEXIBLE PIPE   ORIFICE
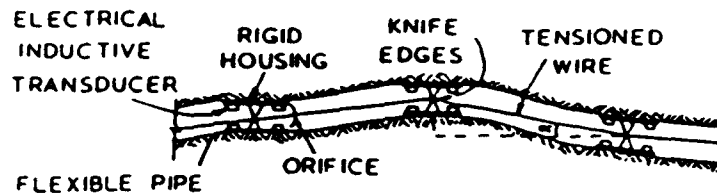
TRANSDUCERS SENSE ANGULAR VARIATION α
OF WIRE WITHIN ORIFICE . HENCE CONTINUOUS
ANGULAR TRAVERSE ALONG PIPE

$H = L \sin . \alpha$

FIG.5.3. Measurement of borehole deflections with
         wire and inductive sensors (Hanna,1985)

LASER   observer

diaphragm   coarse   reticle
                   grating or   or electro-
                   Fresnel zone   optic sensor
                   plate
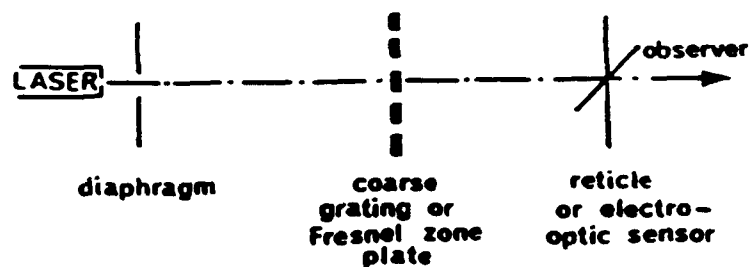
FIG.5.4. Principle of the diffraction method of
         laser alignment

a   b
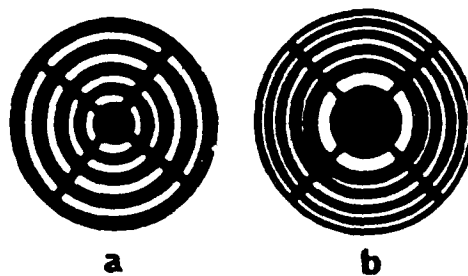
FIG.5.5. Diffraction zone plates a)equal spaced slits
         b)Fresnel zone plate

The image that is formed by the Fresnel zone plate contains a very bright spot at its centre with very faint rings around it. The zone plate acts, therefore, as a focussing lens with the focal length calculated from:

$$f = (s_m^2/m\lambda) = s_1^2/\lambda = (ab / (a + b)) \ .$$

Intensity of light in the central bright spot of the image is approximately m^2 larger than it would be without the zone plate between the source of light and the observing screen.

If a pinhole source of light is placed at distance a from the zone plate, then the bright, focussed spot is created at distance b. Therefore, when a number of points are to be aligned along one line with different a and b distances, different zone plates must be designed for each point. The method gives very high accuracy over long distances. For instance, rectangular Fresnel zone plates with an electro-optical centring device were used in alignment and deformation measurements of a 3 km long nuclear accelerator [Herrmannsfeldt et al., 1967] giving relative accuracy (in a vacuum) of 10^{-7} of the distance. In the open atmosphere, the effect of the thermal turbulence of air seems to have a smaller effect when using the Fresnel zone plates than in the case of the direct optical alignment [Chrzanowski et al. 1976].

6.   Closing Remarks

As indicated in the introduction, out of hundreds available, only a sample selection of models of various geotechnical instruments have been briefly discussed in this presentation. Many of them give compatible or better accuracy than geodetic methods. The decision on which instruments should be used and where they should be located must be in the hands of an experienced person who is very familiar with both geotechnical and geodetic methods and has a good understanding of the behaviour of the deformable object. This leads to a need for a proper preanalysis and optimization of a proposed measuring scheme which should be based on the best possible combination of all the available measuring instrumentation. The design of localization of the instruments should satisfy not only the best geometrical strength of the network of the observation stations, as is the case in geodetic positioning surveys, but should primarily satisfy the needs of the subsequent physical interpretation of the monitoring results [Chen and Chrzanowski, 1986], i.e., should give optimal results when solving for the deformation parameters of the selected deformation model [Chrzanowski et al. 1986b]. An integration of all available results of geotechnical and geodetic measurements should be incorporated into a

simultaneous analysis of deformations. This is possible nowadays by using the University of New Brunswick Generalized Method of deformation analysis [Chrzanowski et al., 1986b].

There is, very unfortunately, a wrong attitude among managers of large industrial entities (mines, power dams, etc.) toward deformation surveys; generally, the monitoring surveys are initiated only when some dangerous events occur, for instance, caving of the surface, or a fatal accident in the mining operation, or cracking of walls of a hydro-electric structure. This is usually too late to start precision monitoring surveys because there is no reference survey to which a monitoring survey can be referred. A proper interpretation of the surveys is impossible and the ad hoc organized survey in the emergency situation cannot be properly designed. Hundreds of expensive instruments are placed on the structure in hastily designed locations. Finally, no one knows subsequently what exactly to do with all the results.

In order to give an idea of the amount of instrumentation employed on some structures, an example is given for Tarbela dam in Pakistan [Hanna, 1985]. The dam is founded on deep alluvial deposits of boulders, gravels, and sands. The rock is weak with many folds, shear zones, and faults. The project (generating capacity of 2100 MW) includes a 3 km long and over 100 m high main embankment dam, 6 tunnels up to 15 m in diameter, spillways, power houses, and some auxiliary dams. Construction was completed in 1974 and some problems already started arising during initial filling of the reservoir; one tunnel was damaged, about 360 sink holes and about 140 cracks were observed in the upstream clay blanket. The instrumentation for movements monitoring includes about 300 multipoint cable and rod extensometers, 90 invar tape extensometers, 412 strain meters, 96 torpedo inclinometers, 24 in-place inclinometers, about 85 double fluid settlement gauges with a total of 1200 m of tubing, 84 joint meters, and a large amount of other geotechnical instrumentation for stress, water pressure, seppage, microseismicity, and earthquake peak measurements. The geodetic surveys are not mentioned in the description by Hanna [1985].

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APPENDIX I

SAMPLE LIST OF PRODUCERS AND/OR DISTRIBUTORS
OF GEOTECHNICAL INSTRUMENTS

Eastman Whipstock GmbH
Gutenbergstrasse 3
3005 Hannover-Westerfeld
West Germany

Geotechnical Instruments Ltd.
Station House Old Warwich Road
Leamington Spa, Warwickshire CV31 3HR
England

Huggenberger Ltd.
Holstrasse 176
CH-8040 Zürich
Switzerland

IRAD GAGE
Etna Road
Lebanon, NH 03766
U.S.A.

Maihak AG
Semperstrasse 38
D-2000 Hamburg 60
West Germany

Roctest Ltée Ltd.
665 Pine
Montreal, P.Q.
Canada J4P 2P4

RST Instruments Ltd.
1780 McLean Avenue
Port Coquitlam, B.C.
Canada V3C 4K9

Schaevitz Engineering
P.O. Box 505
Camden, NJ 08101
U.S.A.

Serata Geomechanics, Inc.
4124 Lakeside Drive
Richmond, CA 94806
U.S.A.

SINCO (Slope Indicator Co.)
3668 Albion Place N.
Seattle, WA 98103
U.S.A.

Soil Instruments Ltd.
Bell Lane, Uckfield
East Sussex TN22 1Ql
England

Solexperts AG
Postfach 274
CH-8034 Zürich
Switzerland

Solinst Canada Ltd.
2440 Industrial St.
Burlington, Ontario
Canada L7P 1A5

SIS Geotecnica
Via Roma, 15
20090 Segrate (Mi)
Italy

Telemac
2 Rue Auguste Thomas
92600 Asnieres
France

Terrametrics
16027 West 5th Avenue
Golden, CO 80401
U.S.A.

Terra Technology
3860 - 148th Avenue N.E.
Redmond, WA 98052
U.S.A.

A STRATEGY FOR THE ANALYSIS OF THE STABILITY OF REFERENCE
POINTS IN DEFROMATION SUREVYS

A STRATEGY FOR THE ANALYSIS OF THE STABILITY OF REFERENCE POINTS IN DEFORMATION SURVEYS

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Confirmation of the stability of reference points is one of the main problems in deformation analysis. The difficulty lies in the datum defects of monitoring networks. A strategy has been developed by the authors and successfully applied in a number of projects. The method leading to the minimization of the first norm of the vector of displacements of reference points has been designed for identifying unstable reference points. Having flagged the unstable reference points, estimation and statistical testing of their displacements are performed.

Two examples are given. A vertical reference network is analyzed step by step to illustrate the proposed strategy. Results of analyzing a horizontal reference network for monitoring a gravity dam are given in the second example.

La confirmation de la stabilité des points de référence constitue l'un des principaux problèmes dans l'analyse des déformations. La difficulté réside dans l'imperfection des données des réseaux de surveillance. Les auteurs ont mis au point une stratégie qu'ils ont appliquée avec succès à un certain nombre de projets. La méthode menant à la minimisation de la première norme du vecteur des déplacements des points de référence a été conçue pour identifier les points de référence instables. Après avoir ainsi identifié ces derniers, une estimation et une analyse statistique de leurs déplacements sont alors effectuées.

Deux exemples sont donnés. Un réseau de référence verticale est analysé de façon systématique pour illustrer la stratégie proposée. Les résultats de l'analyse du réseau de ré;.rence horizontale à des fins de surveillance d'un barrage figurent dans le second exemple.

Introduction

Most surveying schemes for monitoring deformations are comprised of several reference points against which the displacements of the object points are calculated. To obtain the absolute displacements of the object points, the stability of the reference points must be ensured and any unstable points identified. Otherwise, the calculated displacements of the object points and the subsequent analysis and interpretation of the deformation of the object may be significantly distorted. Figure 1 illustrates a situation where points A, B, C, and D are reference points and the others, object points. If point B has moved but is not identified and is used with point A as explicit minimal constraints in the adjustment for two campaigns of observations, then all the object points and reference points C and D will show significant movements (even when, in reality, they are stable). The reference points are supposed to be located outside the deformation area. However,

some of them may move due to, for instance, local forces and inappropriate monumentation. Even if the reference points are monumented on solid bedrock, the forces which cause the deformation of the object may also affect the surroundings over a large area. Therefore, the stability of reference points should always be carefully checked. Unfortunately, this problem is very often underestimated and neglected in surveying practice.

Over the past two decades several methods for the analysis of reference networks have been developed in various research centers [Pelzer 1974; van Mierlo 1978; Niemeier 1981; Koch and Fritsch 1981; Chrzanowski et al. 1983; Heck 1983; Janusz 1983; Gründig et al. 1985]. A conceptual review has been given by Chrzanowski and Chen [1986].

One method, developed by the authors, is a special case of the UNB generalized method for

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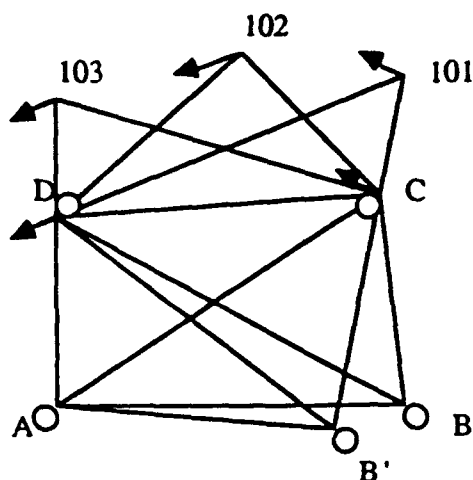
Figure 1: The influence of an unstable reference point (not to scale)

the analysis of deformation surveys. This method has been implemented in the computer package DEFNAN [*Chrzanowski et al.* 1983; *Chen* 1983; *Secord* 1985; *Chrzanowski et al.* 1986]. Since 1983, the method has been successfully applied to a number of monitoring networks. This paper discusses the basic principles of the method and the analysis strategy.

Adjustment of the Observations in a Free Monitoring Network

Since no reference point in a geodetic monitoring network can be accepted as stable until the analysis is performed, the network must be treated as a free network. It means that the network in itself does not contain enough information to be located in space. Examples are a leveling network without elevation information of any point, or a horizontal trilateration network without the known coordinates of any point and any known azimuth between a pair of points. Therefore, free networks can be freely translated or rotated or scaled in space, and can be considered as suffering from datum defects.

Consider the linearized parametric adjustment model of a free network as

$$l + v = Ax \quad , \quad \text{with } \sigma^2_0 Q \tag{1}$$

where l is the n-vector of observations, v is the n-vector of residuals, x is the vector of the corrections to the approximate coordinates of the survey points, A is the configuration matrix, σ^2_0 is the *a priori* variance factor, and Q is the cofactor matrix of the observations. The least squares criterion leads to the normal equations:

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$$Nx = w \tag{2}$$

where $N = A^T Q^{-1} A$, $w = A^T Q^{-1} l$. Due to datum defects in the network, the coefficient matrix, N, of the normal equations is singular, i.e., $\det(N) = 0$. Therefore, one must define a datum to solve for x, expressed by a system of constraints or datum equations as

$$D^T x = 0 \tag{3}$$

in which there is an equation for each datum defect of the network. For example, a leveling network of m points with point P_i held fixed, has the datum equation $\delta H_i = 0$ where δH_i is the correction to the approximate height of point P_i, and matrix D^T is of the order 1 by m and has 1 for the ith element and 0 elsewhere; a trilateration network of m points with, say, point P_1 and the azimuth from point P_1 to point P_3 held fixed, then the datum equations are $\delta x_1 = \delta y_1 = 0$ and $\sin(\alpha_{13})\delta y_3 - \cos(\alpha_{13})\delta x_3 = 0$, where δx_i and δy_i are the corrections to the approximate coordinates of point P_i, and matrix D^T (3 by $2m$) is written as

$$D^T = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 & \ldots & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & \ldots & 0 & 0 \\ 0 & 0 & 0 & 0 & -\cos(\alpha_{13}) & \sin(\alpha_{13}) & \ldots & 0 & 0 \end{bmatrix}$$

The solution of equation (2) with datum equations $D^T x = 0$ reads as [*Chen* 1983]

$$\hat{x} = (N + DD^T)^{-1} w \tag{4}$$

with a cofactor matrix

$$Q\hat{x} = (N + DD^T)^{-1} - H(H^T DD^T H)^{-1} H^T \tag{5}$$

in which the matrix H fulfills the conditions that rank(H) = rank(D) and $NH = 0$. For a vertical network $H = I$, a vector with all elements equal to 1. For a pure triangulation network of m points,

$$H^T = \begin{bmatrix} 1 & 0 & 1 & 0 & \ldots & 1 & 0 \\ 0 & 1 & 0 & 1 & \ldots & 0 & 1 \\ -y_1^0 & x_1^0 & -y_2^0 & x_2^0 & \ldots & -y_m^0 & x_m^0 \\ x_1^0 & y_1^0 & x_2^0 & y_2^0 & \ldots & -y_m^0 & x_m^0 \end{bmatrix} \tag{6}$$

where x_i^0, y_i^0 are the coordinate components of

point P_i with respect to the centroid of the network, i.e.,

$$x_i^0 = x_i - (\sum_1^m x_j) / m, \ y_i^0 = y_i - (\sum_1^m y_j) / m$$

with x_i, y_i the coordinate components of point P_i in the original coordinate system. The rows in H^T correspond to the translations in the x and y directions, rotation, and scale change of the network, respectively. For a trilateration or triangulateration network, the last row of H^T in equation 6 is not involved.

In practice, one could introduce some pseudo-observations with very small variances to remove the datum defects. For example, in a trilateration network, the observations of an azimuth and the coordinates of one point are introduced. The results will be pratically the same as that obtained from equations (4) and (5). However, one should be cautioned that ill conditioning of the normal equations may occur due to very small variances for the pseudo-observations.

The solution of equation (2) with respect to the datum equations $D^T x = 0$ can also be realized through a similarity transformation from any solution $\hat{x}_u$ as

$$\hat{x} = S\hat{x}_u, \ Q_{\hat{x}} = S Q_{\hat{x}_u} S^T \tag{7}$$

with

$$S = I - H(D^T H)^{-1} D^T = I - H(H^T W H)^{-1} H^T W \ , \tag{8}$$

where $W = D(D^T D)^{-1} D^T$. Matrix W in equation (8) can be interpreted as a "weight" matrix in the definition of the datum and therefore equation (7) is called a weighted similarity transformation. If all of the points in the network are of the same importance in defining the datum, then $W = I$ and the results become the "inner constraint" solution. If only some points are used to define the datum, these points are given unit weight and the others zero weight, i.e., $W = \text{diag } \{I, 0\}$. For more details on the adjustment of free networks, reference can be made to Chen [1983].

The a posteriori variance factor $\hat{\sigma}_0^2$ and its degrees of freedom, df, are computed from the estimated residuals $\hat{v}$ as:

$$\hat{\sigma}_0^2 = \frac{\hat{v}^T Q^{-1} \hat{v}}{df}, \ df = n - rank \ \{A\} \tag{9}$$

where the rank of A for a complete network (without configuration defects) is equal to the number of unknown parameters x minus the number of datum defects of the network.

Identification of Unstable Reference Points by Minimizing the First Norm of the Displacement Vector of Reference Points

When comparing two campaigns, the vector of displacements for all the surveyed points and its cofactor matrix are calculated as:

$$d = \hat{x}_2 - \hat{x}_1, \ Q_d = Q_{\hat{x}1} + Q_{\hat{x}2} \tag{10}$$

The pooled variance factor $\hat{\sigma}_{0p}^2$ and its degrees of freedom df_p are computable from

$$\hat{\sigma}_{0p}^2 = [df_1(\hat{\sigma}_{01}^2) + df_2(\hat{\sigma}_{02}^2)] / df_p, \ df_p = df_1 + df_2 \tag{11}$$

where the subscripts 1 and 2 refer to the first and second campaigns, if the a priori variance factor is not available and the statistical test on the null hypothesis $H_0: \hat{\sigma}_{0_1}^2 = \hat{\sigma}_{0_2}^2$, with significance level α,

$$[F(\alpha/2; df_2, df_1)]^{-1} < \hat{\sigma}_{01}^2 / \hat{\sigma}_{02}^2 < F(\alpha/2; df_1 df_2) \tag{12}$$

is true. Failure of the above test may be caused by incompatible weighting of the observations between the two adjustments or by incorrect weighting scheme.

As already mentioned, the displacements calculated from equation (10) may be biased by a pre-selected datum or by different datum definitions in the adjustment of two campaigns, which makes identification of unstable reference points difficult. To overcome this problem a strategy of minimizing the first norm of the displacement vector of the reference points has been developed by the authors [Chen 1983]. The strategy provides a datum which is robust to unstable reference points and gives less distorted displacements. For more on robust estimation, the readers are referred to Huber [1981], and Caspary [1988].

Let d_r and Q_{d_r} be the displacement vector and its cofactor matrix for the reference points, respectively, extracted from d and Q_d in equation (10). Transformation of them onto another datum is performed using equations (7) and (8) as

$$\tilde{d}_r = [I - H_r(H_r^T W_r H_r)^{-1} H_r^T W_r] d_r = S_r d_r, \tag{13a}$$

and

$$Q\tilde{d}_r = S_r Q d_r S_r^T \tag{13b}$$

Matrix H_r is constructed in the same manner as in the previous section and depends on the union

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of the datum defects in the two campaigns and on the number of reference points. For example, if the monitoring network in the first campaign is triangulation which has datum defects of two translations, one rotation and one scale and in the second campaign, a trilateration which has datum defects of two translations and one rotation, then the union of the datum defects is the same as in the first campaign.

The strategy presented here is to select such a weight matrix W_r in equation (13a) that the first norm of the displacement vector $\tilde{d}_r$ approaches a minimum, i.e., $\|\tilde{d}_r\|_1 = \min$. Let

$$t = (H_r^T W_r H_r)^{-1} H_r^T W_r d_r,$$

called the transformation parameters, then $\|\tilde{d}_r\|_1 = \sum |d_r(i)-h_i t|$, where $d_r(i)$ is the ith element of d_r and h_i the ith row vector of matrix H_r. The condition can be written as

$$\min_t \{\sum_i |d_r(i)-h_i t|\} \qquad (14)$$

Equation (14) may not always have a unique solution. This is, however, not a problem for the purposes of identification of unstable reference points.

For a vertical monitoring network, the datum parameter is a translation quantity t_z in the vertical direction. If w_i is the displacement of point P_i, then expression (14) becomes

$$\min_{t_z} \{\sum_i |w_i - t_z|\} \qquad (15)$$

The solution for t_z is straightforward. All the w_i are arranged in a sequence of their increasing algebraic values, and the middle value is the value t_z. If there is an even number of reference points, either value of the two middle displacements or their average can be used as t_z. In other words, the point or a pair of points whose displacement(s) is in the middle place has weight 1 and the rest, weight 0. The new vector of displacements and its cofactor matrix are calculated from equation (13).

For a two-dimensional network, a method of iterative weighted similarity transformation has been elaborated [Chen 1983; Secord 1985]. In this method, the weight matrix W_r in equation (13) is taken as identity at the outset, then in the $(k+1)^{th}$ transformation the weight matrix is defined as

$$W_r^{(k+1)} = diag \{1/|\tilde{d}_r^{(k)}(i)|\}. \qquad (16)$$

where $\tilde{d}_r^{(k)}(i)$ is the ith component of the vector $\tilde{d}_r$ after the kth iteration. The iterative procedure continues until the absolute differences between the

successive transformed displacement components are smaller than a tolerance δ (say, half of the average accuracy of the displacement components). During this procedure some $\tilde{d}_r^{(k)}(i)$ may approach zero causing numerical instabilities because $1/|\tilde{d}_r^{(k)}(i)|$ becomes very large. There are two ways to handle this. One is to replace the expression (16) by $W_r^{(k+1)} = diag \{1/(|\tilde{d}_r^{(k)}(i)|+\delta)\}$, and the other is to set a lower bound. When $|\tilde{d}_r^{(k)}(i)|$ is smaller than the lower bound, its weight is set to zero. If in the following iterations the $\tilde{d}_r^{(k)}(i)$ becomes significantly large again, the weights can be changed accordingly. The explanation for the second way is given in Schlossmacher [1973]. The above procedure provides an approximate solution to equation (14). In the final iteration, say $(k+1)^{th}$, the cofactor matrix is calculated from:

$$Q\tilde{d}_r = S_r^{(k+1)} Q d_r [S_r^{(k+1)}]^T \qquad (17)$$

By comparing the displacement of each point against its confidence region at a specified significance level α, one can identify the reference points which are most probably unstable.

Estimation and Statistical Testing of the Displacements of Unstable Reference Points and Object Points

The final displacements of the points identified as unstable and of all the object points are estimated by a least squares fitting of a deformation model Bc to the displacements d obtained from equation (10) as

$$d + v = Bc \qquad (18)$$

where v is the vector of residuals after fitting, c is the vector of the final displacements to be estimated and B is the design matrix. Explicitly, the deformation model for each unstable point and object point P_i in a two dimensional network is written as:

$$d_i + v_i = \begin{bmatrix} a_i \\ b_i \end{bmatrix} = c_i \qquad (19a)$$

and for each stable point P_j as:

$$d_j + v_j = \begin{bmatrix} 0 \\ 0 \end{bmatrix}. \qquad (19b)$$

Thus, the matrix B in equation (18) has unit elements corresponding to the unstable points and

For a two-dimensional network, a method of iterative weighted similarity transformation has been elaborated . . .

object points, and zeros elsewhere. Solution of equation (18) gives

$$\hat{c} = (B^T P_d B)^{-1} B^T P_d d \qquad (20a)$$

and its cofactor matrix

$$Q\hat{c} = (B^T P_d B)^{-1} \qquad (20b)$$

The weight matrix P_d can be calculated [Chen, 1983] either as

$$P_d = N_1 (N_1 + N_2)^{-} N_2 \qquad (21)$$

or

$$P_d = (SQ_d S)^+ = [SQ_d S + H(H^T H)^{-1} H^T]^{-1} - H(H^T H)^{-1} H^T \qquad (22)$$

In equation (21) N_i ($i=1,2$) is the coefficient matrix of the normal equations (see equation (3)). A generalized inverse, $(N_1+N_2)^-$, can be computed as $(N_1+N_2+HH^T)^{-1}$, where the column vectors of H correspond to the commom datum defects in the two campaigns. If two campaigns have the same survey scheme and measurement accuracy, i.e., $N_1 = N_2 = N$, then

$$P_d = N/2. \qquad (21')$$

In equation (22) matrix S is as expressed in equation (8) with $W = I$ and the column vectors of H correspond to the union of datum defects in the two campaigns. The reason for computing the weight matrix in such a way is so that the estimated parameters $\hat{c}$ will be independent of the datum used in the adjustments. If the datum defects are removed by the introduction of some pseudo-observations with small variances, then the weight matrix could be calculated from

$$P_d = Q_d^{-1} \qquad (22')$$

However, in this case, not only will numerical problems likely occur due to ill-conditioning of Q_d but also will complications arise in modelling of deformations. Some additional parameters have to be introduced, as is explained in the second example below. More details are given in [Chrzanowski et al. 1983].

The significance of the estimated displacement $\hat{c}_i$ for an unstable point P_i is indicated by

$$\hat{c}_i^T Q\hat{c}_i^{-1} \hat{c}_i / [m_{ci} (\hat{\sigma_0^2})_p] > F(\alpha ; m_{ci}, df_p) \qquad (23)$$

where m_{ci} is the dimension of $\hat{c}_i$, $Q\hat{c}_i$ is the submatrix of $Q\hat{c}$, and $\hat{\sigma_0^2}_p$ and df_p are the pooled variance factor and its degrees of freedom, respectively. To test the null hypothesis that no other unstable point exists, a quadratic function ΔR of the estimated residuals $\hat{v}$ is calculated as

$$\Delta R = \hat{v}^T P_d \hat{v} \qquad (24)$$

which follows a chi-squared distribution with degrees of freedom as

$$df_c = \text{rank}\{P_d\} - m_c \qquad (25)$$

where m_c is the dimension of unknown vector c and the rank defect of P_d is equal to the number in the union of datum defects in both campaigns. If the inequality

$$\Delta R / (df_c \hat{\sigma_0^2}) < F(\alpha ; df_c, df_p) \qquad 26)$$

holds, the null hypothesis is acceptable at the (1-α)% confidence level. Otherwise a search for other unstable reference points should be made. The latter case seldom occurs. When the a priori variance factor σ_0^2 is known, $\hat{\sigma_0^2}_p$ and df_p in the tests (23) and (26) are replaced by σ_0^2 and ∞, respectively.

The analysis procedures discussed above are summarized in Figure 2.
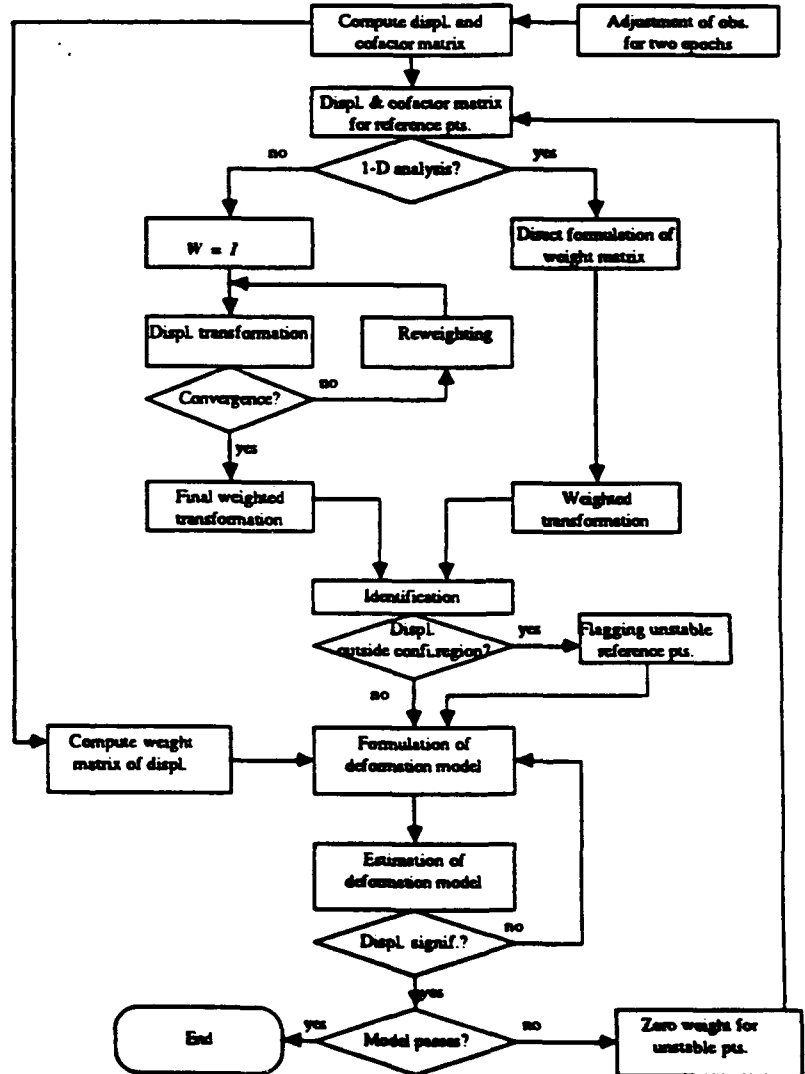
Figure 2: The flowchart of analysis procedures

Examples

Analysis of a Vertical Reference Network

Figure 3 is a leveling reference network with two survey campaigns. The observations are listed in Table 1.
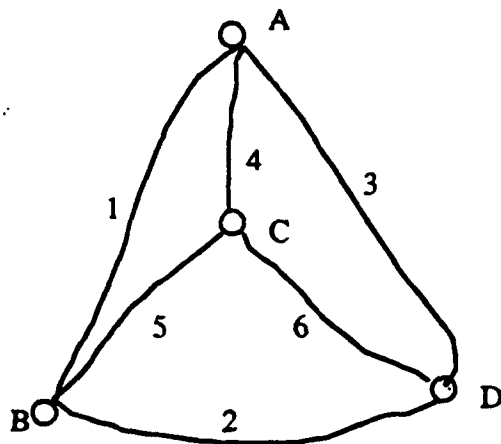
Figure 3: A leveling network

Table 1: The observations of the leveling network.

leveling line	observed height difference [mm] campaign 1	campaign 2	weight p_i
1	45.2	46.9	1
2	265.8	265.6	2
3	310.3	312.2	1
4	-26.2	-24.1	2
5	70.8	70.7	2
6	336.5	336.1	2

Step 1.   Adjustment of the leveling network and computation of the displacements

Point A is fixed with an elevation of 0.50000 m in the adjustment of the observations for each campaign. The adjusted heights [m] of points A, B, C, and D for both campaigns are

$$\hat{x}_1^T = (0.50000\ 0.54479\ 0.47392\ 0.81046)$$

and

$$\hat{x}_2^T = (0.50000\ 0.54673\ 0.47598\ 0.81220)$$

Since the *a priori* variance factor is not available, the *a posteriori* variance factors for both campaigns are estimated from the residuals as

$$\hat{\sigma}_{0_1}^2 = (\sum_1^6 P_i \hat{v}_i^2) / df_1 = 0.269 /3 = 0.0897$$

and

$$\hat{\sigma}_{0_1}^2 = (\sum_1^6 P_i \hat{v}_i^2) / df_2 = 0.109 /3 = 0.0363$$

The null hypothesis H_o: $\hat{\sigma}_{0_1}^2 = \hat{\sigma}_{0_2}^2$ is tested using expression (12):

$$1/F(0.025;3,3) < \hat{\sigma}_{0_1}^2/\hat{\sigma}_{0_2}^2 = 12.47 < F(0.025;3,3) = 15.4$$

Therefore, the pooled variance factor is calculated from equation (11) as

$$\hat{\sigma}_{0_p}^2 = 0.0630$$

with degrees of freedom $df_p = 6$.

The vector of displacements [mm] and its cofactor matrix read

$$d = \hat{x}_2 - \hat{x}_1 = (0.00\ 1.94\ 2.06\ 1.74)^T$$

and

$$Qd = \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 0.74 & 0.40 & 0.45 \\ 0 & 0.40 & 0.60 & 0.40 \\ 0 & 0.45 & 0.40 & 0.74 \end{bmatrix}$$

Step 2. Identification of unstable points

Using the method discussed above, the displacements are arranged in the sequence of their increasing algebraic values as (0.00 1.74 1.94 2.06). Thus points D and B are assigned unit weight and points A and C zero weight (translation parameter t_z in equation (15) is the mean of the two middle displacements, i.e., $t_z = 1.84$). After the weighted similarity transformation, the new vector of displacements [mm] and its cofactor matrix are

$$\tilde{d}^T = (-1.84\ 0.10\ 0.22\ -0.10)$$

and

$$Q\tilde{d} = \begin{bmatrix} 0.60 & 0.00 & 0.20 & 0.00 \\ 0.00 & 0.15 & 0.00 & -0.15 \\ 0.20 & 0.00 & 0.40 & 0.00 \\ 0.00 & -0.15 & 0.00 & 0.15 \end{bmatrix}$$

The displacement of each point is tested at a significance level of $\alpha = 0.05$, i.e.,

$$\overline{d}_A^2/(\hat{\sigma}_{\varphi}^2 q_{11}) = (-1.84)^2/[(0.60)(0.0630)] = 89.6 > F(0.05;1,6) = 6.0$$

$$\overline{d}_B^2/(\hat{\sigma}_{\varphi}^2 q_{22}) = \overline{d}_D^2/(\hat{\sigma}_{\varphi}^2 q_{44}) = 10.6 < F(0.05;1,6) = 6.0$$

$$\overline{d}_C^2/(\hat{\sigma}_{\varphi}^2 q_{33}) = 1.92 < F(0.05;1,6) = 6.0$$

It is clear that only point A can be strongly suspected as being unstable.

Step 3. Estimation of the displacement of the unstable point.

Using equation (18) with $d^T = (0.00 \quad 1.91 \quad 2.06 \quad 1.74)$, $B^T = (1 \quad 0 \quad 0 \quad 0)$ and

$$P_d = N_1(N_1 + N_2)N_2 = N/2 = \begin{pmatrix} 2 & -0.5 & -1 & -0.5 \\ -0.5 & 2.5 & -1 & -1 \\ -1 & -1 & 3 & -1 \\ -0.5 & -1 & -1 & 2.5 \end{pmatrix}$$

the estimated displacement of point A is

$$\hat{c} = -1.95 \text{ mm}$$

and its cofactor

$$q_{\hat{c}} = (B^T P_d B)^{-1} = 0.5$$

The displacement is significant due to the fact that

$$\hat{c}^2 / (\hat{\sigma}_{\varphi}^2 q_{\hat{c}}) = 126.1 > F(0.05; 1,6) = 6.0$$

The test on the deformation model is performed using the quadratic function (24) with the residuals equal to

$$\hat{v}^T = (-1.95 \quad -1.94 \quad -2.06 \quad -1.74)$$

and

$$\Delta R = \hat{v}^T P_d \hat{v} = 0.189$$

The test

$$\Delta R / (2\hat{\sigma}_{\varphi}^2) = 1.5 < F(0.05; 2,6) = 5.1$$

indicates that the deformation model is acceptable, i.e., the remaining points can be considered as stable at 95% confidence level.

One can also use the vector of displacements $\overline{d}$ after the weighted similarity transformation to estimate $\hat{c}$ and calculate the test statistic ΔR. The results will be identical. This indicates that whatever minimal constraint solution is used in the estimation and test processes, it will not affect the final results.

Analysis of the Reference Network for Monitoring a Gravity Dam

A pure triangulation network of 6 reference points and 10 uniquely intersected points on a
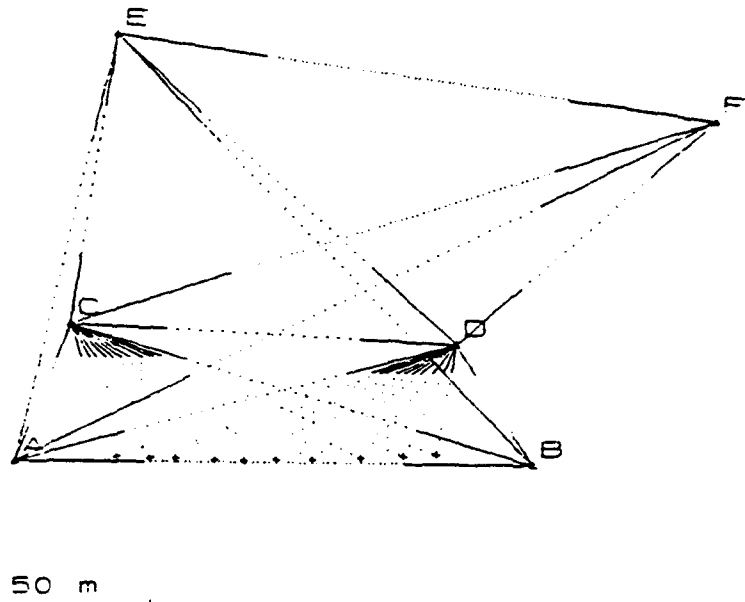
Figure 4: The monitoring network of a gravity dam

dam crest (Figure 4) was observed in two survey campaigns with 47 directions in the first campaign and 53 directions in the second. Least squares estimations of the coordinates $\hat{x}_1, \hat{x}_2$ were made under explicit minimal constraints involving points E and F (considered "fixed" and errorless). No observation in either campaign was detected as being an outlier at $\alpha = 0.05$ using τ-max test [Pope 1976; Vaniček and Krakiwsky 1982]. The pooled variance factor, $\hat{\sigma}_{\varphi}^2 = 0.95$, had $df_p = 31$ degrees of freedom. The vector of displacements d and the cofactor matrix Q_d were obtained using equation (10). Within the d and Q_d there are zero elements corresponding to points E and F.

The vector of displacements d_r and its cofactor matrix Q_{d_r} for the reference points A, B, C, D, E, and F were extracted from d and Q_d. The iterative weighted transformation resulted in the displacement pattern, coupled with the 95% confidence ellipses, for the reference points as shown in Figure 5. Obviously, reference point D has moved significantly while the others remain stable.

Having a_i and b_i as unknown parameters corresponding to the x- and y-components of the displacement for each object point and also for point D, the deformation model consisted of 22 parameters (i.e., 11 pairs of displacement components). The weight matrix P_d was calculated using equation (22) and the 22 unknown parameters were estimated using equation (20). A plot of these estimated displacements and their asso-
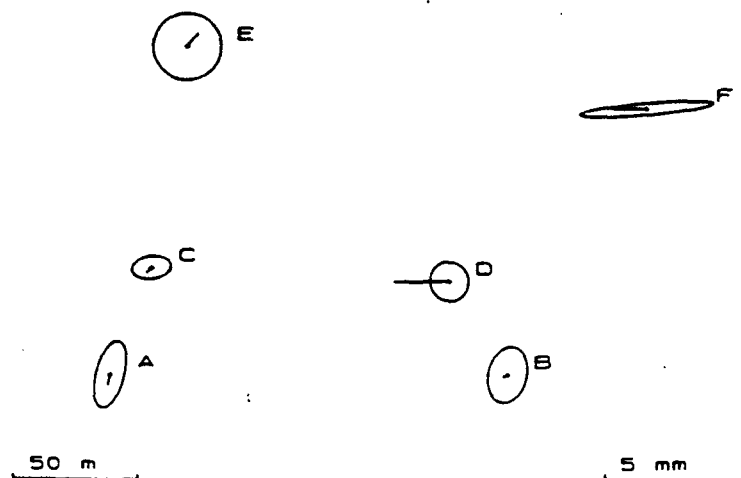
Figure 5: Displacements after the iterative weighted transformation

ciated 95% confidence ellipses is given in Figure 6. With 6 degrees of freedom in the modeling, the test on the adequacy of the model was not rejected at $\alpha=0.05$ since:

$$\Delta R / (df_c \hat{\sigma}_{op}^2) = 1.76 < F(0.05; 6, 31) = 2.41$$

where ΔR is the quadratic form of the residuals $\hat{v}$ after fitting, calculated from equation (24). Thus, the deformation model, in which the reference points A, B, C, E, and F are accepted as stable, is appropriate.

One could perform the above analysis using a subvector d_1 of d and submatrix Q_{d1} of Q_d, which do not include zero elements corresponding to the "fixed" points E and F. In this case, the weight matrix of d_1 would be calculated directly as $P_{d1} = Q_{d1}^{-1}$ rather than using equation (22). Since points E and F have been identified as stable, the deformation model would be formulated in the same way as the above, i.e., using equations (19a) and (19b) except that points E and F would not be involved in modeling. The same estimation and

testing procedures would be used. However, if either or both of points E and F would have been identified as unstable, additional parameters which are a function of the displacements of points E and F would have to be included in deformation modeling. If both points would have been identified as unstable reference points, the calculated displacements of other points would have been distorted by translation, rotation and scale change. To account for these effects equations (19a) and (19b) would have to be changed to

$$d_i + v_i = \begin{bmatrix} a_i + k_i(x_i-x_F) + k_2(y_i-y_F) + a_F \\ b_i - k_2(x_i-x_F) + k_1(y_i-y_F) + b_F \end{bmatrix}$$

and

$$d_j + v_j = \begin{bmatrix} k_1(x_j-x_F) + k_2(y_j-y_F) + a_F \\ k_2(x_j-x_F) + k_1(y_j-y_F) + b_F \end{bmatrix}$$

respectively, where k_1 and k_2 are the unknown scale change and rotation parameters. The final displacement components of point E can be then calculated from

$$\hat{a}_E = \hat{k}_1(x_E-x_F) + \hat{k}_2(x_E-x_F) + \hat{a}_F, \quad \hat{b}_E = \hat{k}_1(y_E-y_F) - \hat{k}_2(x_E-x_F) + \hat{b}_F$$

If only one of them, say point E, would have been identified as unstable, a_F and b_F would disappear in the above formulation. To avoid these problems the methodology suggested in this paper should be followed.

Acknowledgement

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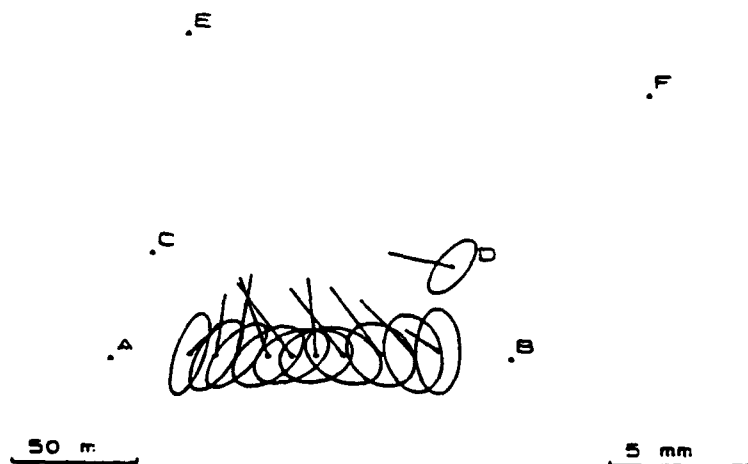Chrzanowski, A., Y.Q. Chen, and J.M. Secord. 1986. Geometrical analysis of deformation surveys,

Figure 6: Final displacements of the unstable reference point and the object points

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GEOMETRICAL ANALYSIS OF DEFORMATION SURVEYS

FIG
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COMBINATION OF GEOMETRICAL ANALYSIS WITH PHYSICAL INTERPRETATION FOR THE ENHANCEMENT OF DEFORMATION MODELLING

LA COMBINAISON DE L'ANALYSE GÉOMÉTRIQUE ET DE L'INTERPRÉTATION PHYSIQUE POUR LE REHAUSSEMENT DU MODELAGE DE DÉFORMATION

KOMBINATION VON GEOMETRISCHER ANALYSE UND PHYSIKALISCHER INTERPRETATION ZUR VERBESSERUNG VON DEFORMATIONSMODELLEN

A. Chrzanowski, Y.Q. Chen, A. Szostak-Chrzanowski, and J.M. Secord - Canada

SUMMARY

The geometrical analysis and physical interpretation of deformations serve different purposes, but distinction between them should not be taken as absolute. The physical interpretaion through the statistical or deterministic modelling may help in selecting a proper deformation model in the geometrical analysis, and the outcome from the geometrical analysis may reveal causes of the deformation, thus enhancing the further interpretaion. A proper combination of them may improve our knowledge on the mechanism of the deformation. This paper presents the basic idea and a conceptual formulation on the combination of geometrical analysis and physical interpretation. Two practical examples are given.

RÉSUMÉ

L'analyse géométrique et l'interprétation physique des déformations se prêtent aux buts distincts, pourtant, la différence entre les deux ne peut être considérée comme absolue. L'interprétation physique peut aider, en utilisant un modèle statistique ou déterministique, à choisir un modèle de déformation approprié pour l'analyse géométrique. Le résultat de ce dernier peut révéler les causes de la déformation, et donc, l'interprétation supplémentaire est améliorée. Une combinaison correcte de l'analyse et de l'interprétation peut améliorer notre connaissance du mécanisme de la déformation. Ce papier présente l'idée de base ainsi qu'une formulation conceptuelle de la combinaison de l'analyse géométrique et de l'interprétation physique. Deux exemples pratiques sont offerts.

ZUSAMMENFASSUNG

Die geometrische Analyse und die physikalische Interpretation dienen zwar unterschiedlichen Zwecken, aber diese Unterscheidung sollte nicht als absolut angesehen werden. Die physikalische Interpretation mit statistischen oder deterministischen Modellen kann dazu beitragen, ein geeignetes Deformationsmodell für die geometrische Analyse auszuwählen. Andererseits kann das Ergebnis der geometrischen Analyse die Ursache der Deformation aufdecken und damit eine weitergehende Interpretation ermöglichen. Eine sinnvolle Kombination von geometrischer Analyse und physikalischer Interpretation kann den Kenntnisstand bezüglich der Deformationsmechanik verbessern. In diesem Beitrag werden die grundsätzlichen Ideen und die konzeptuelle Formulierung dieser Kombination entwickelt. Zwei praktische Beispiele werden vorgestellt.

1. INTRODUCTION

Deformation analysis is comprised of two aspects - geometrical analysis and physical interpretation. Geometrical analysis gives information on the geometrical status of a deformable body - the changes in its position, shape, and dimension. Physical interpretation explains the reasons for the deformation and, possibly, establishes the load-deformation relationship. The outcome of the geometrical analysis may lead to a qualitative interpretation of the deformation. For example, in the case of crustal movement studies, survey engineers and geodesists perform geometrical analyses and indicate the deformation status of the area of interest both in space and in time domains. From this geophysists try to explain the deformation mechanism. If the causes of the deformation are well defined, the analysis of the load-deformation relationship can be performed quantitatively.

There are two basic methods of interpretation: the statistical method and the deterministic method [Chrzanowski et al. 1982]. In the statistical method, the correlations between observed deformations and observed loads, or causative factors, are analysed and a prediction model is developed by performing a statistical analysis on the past data. In the deterministic method, information on the loads, properties of the material, geometry of the body, and physical laws governing the strain-stress relationship are utilized. In contrast to the statistical method, the deterministic method does not require observation of the deformations and loads.

For several decades, there has been an obvious separation between the geometrical analysis and physical interpretation and between the two methods of interpretation. The geometrical analysis is usually done by survey scientists and engineers, but physical interpretation is done by other specialists. In the interpretation of some engineering structures, e.g., dams, survey engineers establish the prediction model using the statistical method, while civil engineers calculate the deformations at the design stage using the deterministic method. Survey engineers and other specialists have had little communication with each other, thus not taking advantage of a possible integrated analysis.

In recent years, more and more attention has been paid to an interdisciplinary approach to deformation analysis in surveying community. Several papers discussing the importance of the integrated analysis have been presented [e.g., Chrzanowski et al. 1983; Chen and Chrzanowski 1986; Teskey 1986].

In this paper, the authors, after briefly reviewing the geometrical analysis and the methods of physical interpretation, discuss the concept of the combination of geometrical analysis and physical interpretation. Two practical examples are given which illustrate the usefulness of the integrated analysis.

2. GEOMETRICAL ANALYSIS AND PHYSICAL INTERPRETATION

2.1. Geometrical Analysis of Deformation Surveys

The deformation of an object is well defined if a displacement function representing the deformation is known [Chrzanowski and Chen 1986]. Assume that the displacement function is written as

$$d(x,y,z; t-t_0) = B(x,y,z; t-t_0)c \qquad (1)$$

where d is the displacement of a point (x,y,z) at time t with respect to a reference time t_0, B is the deformation matrix whose elements are some selected base functions, and c is the vector of coefficients. The analysis of the deformation of an object is determining the function (1), including its analytic shape (model) and the coefficients, from the observations. Let $l_i(t)$ be an observation at time t. It is related to the deformation model (1) either by

1. expressing it in terms of the coordinates of the related points, and then transforming the coordinate differences (displacements) into the deformation model; or by
2. expressing it directly in terms of the deformation model.

The first case is symbolically represented in linear form by

$$E\{l_i(t)\} = a_i x_t = a_i x_0 + a_i(x_t - x_0) \tag{2}$$

where $E\{\ \}$ is the expectation operator, the row vector a_i contains the partial derivatives of the observation l_i with respect to the coordinates of the related points, x_t is the vector of the corrections to the approximate coordinates of the surveyed points at time t, and $(x_t - x_0)$ is the vector of the displacements. Substituting equation (1) into (2) results in

$$E\{l_i(t)\} = a_i x_0 + a_i \widetilde{B}_t c \tag{3}$$

where $\widetilde{B}_t$ is the matrix constructed when superimposing the deformation model Bc and is evaluated at the surveyed points and at time t. The second case is expressed as

$$E\{l_i(t)\} = E\{l_1(t_0)\} + a_i \widetilde{B}_t c \tag{4}$$

The use of equation (3) or (4) depends on the type of observation. For an observation in a geodetic network, expression (3) is better because it takes full advantage of the observations. For individual isolated geodetic observations and geotechnical measurements, equation (4) is employed. Complete comparison of the above two formulations has been given in Chen [1983]. The functional relationship between different types of observables and the deformation model have been derived in Chen [1983], Secord [1985], and Chrzanowski et al. [1986]. Combining all the observations made at different locations and at different time epochs results in:

$$l + v = Az + Gc \tag{5}$$

where l is the vector of the observations, which, in general, includes geodetic observations in a network, individual isolated observations, and geotechnical measurements; v is the vector of residuals; z is the vector of the coordinates of the surveyed points in a network and expected values of the individual observations and geotechnical measurements at reference epoch t_0, and A and G are the corresponding transformation matrices. By applying the least squares criterion, the vector of unknown coefficients c, its cofactor matrix Q_c, and the sum of the weighted squares of the residuals, ΔR, are calculated. The statistical tests on c and ΔR are performed to examine the significance of the estimated parameters and the appropriateness of the deformation model. If the tests are unacceptable, the model has to be refined and the estimation processes are repeated.

A method, called the UNB Generalized Method for Deformation Analysis, has been developed by the authors [Chrzanowski et al. 1983; Chen 1983; Secord 1985]. The method is applicable to any type of geometrical analysis, both in space and in time, including the detection of unstable reference points and the determination of strain components and relative rigid body motion within a deformed object. It allows utilization of different types of survey data and geotechnical measurements. In practical application, the approach consists of three basic processes: identification of deformation models; estimation of deformation parameters; diagnostic checking of the models and final selection of the "best" model. The method has been implemented through a software suite DEFNAN developed in FORTRAN 77 on an IBM 3090 mainframe and on an IBM PC AT. More details are presented in Chrzanowski et al. [1986].

2.2. Physical Interpretation of Deformations

2.2.1. Interpretation by the statistical method

Let $y(t)$ be an observed deformation response (e.g., displacement) at time t with respect to a reference time t_0, and $x_i(t)$, $i=1,2,....,$ m, be the observed value of the ith causative factor. Assume that the deformable body can be considered as a linear system, i.e., additivity and homogeneity can be applied [see Bonaldi et al. 1977; Fanelli 1979]. Then, the response is expressed as:

$$y(t) = f_1\{x_1(t)\} + f_2\{x_2(t)\} + \dots + f_m\{x_m(t)\} \tag{6}$$

where $f_i\{..\}$ is the function of a causative factor. Different causative factors may produce deformations in different manners. Some effects can be modelled by polynomial functions, but others may be more adequately expressed as trigonometric functions, and so forth. Take as an example the modelling of the response of a concrete dam. The horizontal upstream-downstream displacement $d(t)$ of a point can be expressed as [Chen and Chrzanowki 1986]

$$d(t) = d_1(t) + d_2(t) + d_3(t) \tag{7}$$

where d_1, d_2, and d_3 are the hydrostatic pressure component, the thermal component and the irreversible component due to non-elastic phenomena, respectively. Component d_1 may be modelled as a polynomial function of the water level $h(t)$ in the reservoir:

$$d_1(t) = a_0 + a_1 h(t) + a_2 h^2(t) + \dots + a_{k1} h^{k1}(t) \tag{8}$$

Component d_2 may be modelled in two ways. If some key temperatures T_j ($j = 1, 2, \dots, k_2$) in the dam are measured, then

$$d_2(t) = b_1 T_1(t) + b_2 T_2(t) + \dots + b_{k2} T_{k2}(t) \tag{9a}$$

If no temperature measurement is available, it can be expressed as a cyclic function of time:

$$d_2(t) = c_{11}\sin(\omega t) + c_{12}\cos(\omega t) + c_{21}\sin(2\omega t) + c_{22}\cos(2\omega t) + \dots \tag{9b}$$

where $\omega = 2\pi/yr$. Component d_3 is usually approximated by a time function, for instance, as

$$d_3(t) = \theta_1 t + \theta_2 t^r \ln(t) \qquad (0 < r < 1) \tag{10}$$

In the above, all the coefficients have to be estimated from the observed displacements. In general, equation (6) is expressed as

$$y(t) + v(t) = bc, \quad \forall t \tag{11}$$

where b is a row vector whose elements are some selected base functions of causative quantities, c is the vector of unknown coefficients, and $v(t)$ contains the residual to the fitting. Following the same procedures as in the geometrical analysis, the coefficients c are estimated and statistically tested. The final model suggests the response behaviour of the different causative factors and is used for prediction purposes.

2.2.2. Interpretation by the deterministic method

Deformation of an object will develop if an external force is applied to it. The external forces may be of two kinds: surface force, i.e., forces distributed over the surface of the body, and body forces, which are distributed over the volume of the body, such as gravitational forces and thermal stress. The relation between the acting forces and displacement is discussed in many textbooks on mechanics [e.g., Sokolnikoff 1956]. Let d be the displacement vector at a point and f be the acting force. They are related as

$$L^T DL d + f = 0 \tag{12}$$

where D is the constitutive matrix of the material whose elements are functions of the material properties and L is a differential operator transforming displacement to strain. If initial strain ε_0 and initial stress σ_0 exist, equation (12) becomes

$$L^T DL d + (L^T \sigma_0 - L^T D \varepsilon_0) + f = 0 \ . \tag{13}$$

In principle, when the boundary conditions, either in the form of displacements or in the form of acting forces, are given and the body forces are prescribed, the differential equation (12) or (13) are completely solved. However, direct solution may be difficult, and numerical methods have to be used and this is where the finite element method (FEM) provides a powerful tool.

The basic concept of the FEM is that the continuum of the body is replaced by an assemblage of small elements which are connected together only at the nodal points of the elements. Within each element a displacement function is postulated and the principle of minimum potential is applied, i.e., the difference between the work done by acting forces and the deformation energy is minimized. Therefore, the differential operator L is approximated by an linear algebraic operator, say $\tilde{L}$. The relation between the vector of the forces f_e and vector of the displacements d_e for the nodal points of an element becomes

$$f_e = [\int \tilde{L}^T D \tilde{L} d(vol.)] \, d_e = K_e d_e \tag{14}$$

where K_e is called the stiffness matrix of the element. Once the stiffness matrices for all the elements have been calculated, an overall structural stiffness matrix K is composed by a superposition of the stiffness matrices for all the elements, and the total equilibrium equation for the whole body is written as:

$$\tilde{f} = K\tilde{d} \tag{15}$$

where $\tilde{f}$ is a vector of applied nodal forces in the whole body, and $\tilde{d}$ is a vector of nodal displacements. If boundary conditions are known, the displacement at any nodal point can be calculated.

A FEM computer program, called FEMMA, has been developed for two dimensional and three dimensional elastic and non-linear elastic analysis [Szostak-Chrzanowski 1988]. The program can utilize the following input data: initial stress, initial strain, body forces, forces and displacement boundary conditions, elasticity parameters, and strengh parameters. The output includes displacements, strain and stress in each element, principal stresses, and a list of the elements whose stresses exceed a given failure criteria. The program has been written in FORTRAN 77 on an IBM 3090 mainframe, IBM PC AT, and Apple Macintosh Plus with external hard drive.

3. ENHANCEMENT OF DEFORMATION MODELLING BY AN INTEGRATED ANALYSIS

Although geometrical analysis and physical interpretation are to serve different purposes, the distinction between them should not be taken as absolute. As mentioned earlier, the first step in geometrical analysis is to identify possible deformation models. The prediction models established using either the statistical method or the deterministic method will help in providing a preliminary deformation pattern for the geometrical analysis. The discrepancies between the results of geometrical analysis and the predicted values or rejection of the predicted model may lead to the discovery of an anomaly, resulting in the further refinement of modelling, as shown in an example in section 4 below.

In order to use the deterministic method the causative factors should be well defined. However, this may not always be the case. The geometrical analysis provides the deformation trend which assists in finding the possible causes of the deformation (qualitative physical interpretation). Having identified the possible causes, the deterministic analysis is applied to calculate the deformation detail and confirm the suspected cause, as shown in another example in section 5 below.

In the comparison of the two methods of physical interpretation, the statistical method possesses some undeniable merits. But, it requires a comparatively large amount of data on both causative and response quantities in order to obtain a reliable model. This suggests that the method cannot be used in the initial period of operation of a structure because of insufficient data. On the other hand, the deterministic modelling, which is more versatile, may have large errors due to imperfect knowledge of the material properties and incorrect modelling of the behaviour of the material(elastic instead of plastic or neglect of creep, etc.). Therefore, a combination of the deterministic and statistical methods is advocated for an optimal modelling of deformations.

Let $x_i(t)$ ($i = 1, 2, ..., m$) be the ith causative factor. Then its effect $d_i(t)$, the displacement, at a nodal point, is calculated using the FEM. The total displacement $d(t)$ is the summation of all the effects:

$$d(t) = d_1(t) + d_2(t) + ... + d_m(t) \qquad (16)$$

Since there are several uncertainties in the deterministic modelling, $d(t)$ may significantly depar⌐ from the real value. One can improve the model by introducing some additional terms taki⌐ care of those deformations which can not be calculated by the deterministic method and ῠ calibrating effects for each causative factor or for some of them. Thus, equation (16) can be enhanced as

$$d'(t) = k_1 d_1(t) + k_2 d_2(t) + ... + k_m d_m(t) + Me \qquad (17)$$

where k_i ($i = 1, 2, ..., m$) are the calibration constants; M is a matrix, like the deformation matrix B in equation (1), whose elements are some selected base functions and are functions of position and time; and e is the vector of unknown coefficients. Again, take as an example physical interpretation of the deformation of a concrete dam. The well defined causative factors are hydrostatic pressure and thermal expansion of the concrete. Their effects can be determined by FEM. The horizontal upstream-downstream displacement of a point $d(t)$ is

$$d(t) = d_p(t) + d_T(t) \qquad (18)$$

where $d_p(t)$ is the hydrostatic component and $d_T(t)$ the thermal component. Since the thermal component is proportional to the thermal expansion coefficient α, and the hydrostatic pressure component is inversely proportional to Young's modulus E of concrete, The calibration constants are (α/α_0) and (E_0/E), respectively, where α_0 and E_0 are the corresponding values used in the FEM computation. The possible irreversible deformation cannot be calculated with the deterministic method and additional terms, e.g., $\theta_1 t + \theta_2 t^r \ln(t)$ (see equation (9)), could be introduced. Thus the refined model becomes

$$d'(t) = (\alpha/\alpha_0)d_p(t) + (E_0/E)d_T(t) + [\theta_1 t + \theta_2 t^r \ln(t)], \quad 0 < r < 1 \qquad (19)$$

Let l be the vector of the deformation measurements as defined in section 2. Equation (5) (repeated as equation (20b)) expresses the relation between the observations and the deformation model for the geometrical analysis. The displacements calculated using the deterministic method could also contribute to the determination of the geometrical status of a body. Let the calculated displacements for all the nodal points and all the time epochs, denoted by $\widetilde{d}$, be treated as "observations". Then they are related to the deformation model as:

$$\widetilde{d} + \delta = G_d c + k'_1 \widetilde{d}_1 + k'_2 \widetilde{d}_2 + ... + k'_m \widetilde{d}_m - \widetilde{M}e \qquad (20a)$$

$$l + v = Ay + Gc \qquad (20b)$$

where $\widetilde{d}_i$ is the component of $\widetilde{d}$ produced by the ith causative factor, δ is the vector of residuals, $k'_i = (1 - k_i)$, $i = 1, 2, ..., m$, G_d is the matrix transforming the deformation parameters c to the displacement vector $\widetilde{d}$, and $\widetilde{M}$ is constructed by superimposing matrix M in equation (17)

evaluated at all the nodal points and at all the time epochs. If the accuracies of l and $\tilde{d}$ are compatible, one can solve simultaneously for c, k_i (i = 1, 2, ..., m), and e, and perform statistical tests. In this way the geometrical analysis, the physical interpretation of the deterministic method, and that of the statistical method are combined into one model. In practice, the accuracies of the displacements calculated from the deterministic method are usually much lower than those from direct observation. In this case, equation (20) is reduced to

$$(\tilde{d} - G_d\hat{c}) + \delta = k'_1 \, \tilde{d}_1 + k'_2 \, \tilde{d}_2 + ... + k'_m \, \tilde{d}_m - \tilde{M}e \qquad (21)$$

where $\hat{c}$ is the estimators from the geometrical analysis. Equation (21) is the model for physical interpretation by combining the deterministic method with the statistical method.

In order to better understand the methods of deformation analysis, a flowchart summarizing the different analysis methods and their interaction is presented in Figure 1.

4. INTEGRATED ANALYSIS OF GROUND SUBSIDENCE IN A MINING AREA

The UNB Generalized Method has been applied in an intergrated analysis of survey data collected in rugged mountainous terrain of western Canada near Sparwood, British Columbia, over an undergound coal mining operation [Fisekci and Chrzanowski 1981]. The purpose of the surveys was to monitor ground movements caused by extraction of a 200 m by 700 m panel of a 12 m thick and steeply inclined coal seam (Figure 2). Three types of observations were used in the integrated analysis of the ground subsidence above the panel: changes in coordinates of 15 points determined by terrestrial geodetic methods, changes in the coordinates of 29 points calculated from aerial photogrammetric surveys, and changes in ground tilts at three stations obtained from remotely controlled bi-axial tiltmeters. Several possible deformation models were fitted to the observations and statistically tested. The best fitted model of this rather complicated case of the ground subsidence is shown in Figure 3.

Besides the described deformations, the extraction of the coal panel produced surface cavings above the upper edge of the panel near the outcrop and long cracks near the mountain ridge, which could not be readily explained. The suspected fault, shown in Figure 2, which was approximately mapped at the level of the mining workings could be a possible explanation. However, the geometry of the fault, its dip angle, and depth had not been identified. Therefore, a comparison of the above geometrical model of the subsidence with the deterministic model for those uncommon topographical and mining conditions has been of great interest in order to confirm the existence of the fault which could be very essential in predicting the ground behaviour in the case of a further expansion of the coal extraction.

Due to very limited information on the geology and tectonics of the mining area, the deterministic modelling of the subsidence was difficult. An iterative non-linear elastic finite element analysis has been performed using the program FEMMA and a method, known as the S-C Method, of ground subsidence prediction developed by Szostak-Chrzanowski [1988]. The modelling has been complicated by the fact that besides the unknown fault parameters, the in situ Young's modulus, E, of the rock was also not known. According to the very limited information available to the authors, the strata above the extracted panel consisted of a medium strong sandstone and shale formation with an average unit weight of $27N/m^3$. Thus, the deterministic modelling had to be calibrated first for the average Young's modulus without the suspected fault.

Two iterative analyses have been made: one without introducing the suspected fault into the FEM model; and the second, with the fault zone represented by a string of elements between the developed crack on the suface and the mapped discontinuity in the rock masses in the underground workings. The analyses have shown that the best agreement between the geometrical and deterministic modelling is obtained for E = 2 GPa, which is quite reasonable for weak sandstone and shale. The FEM results with the fault have shown incomparably better
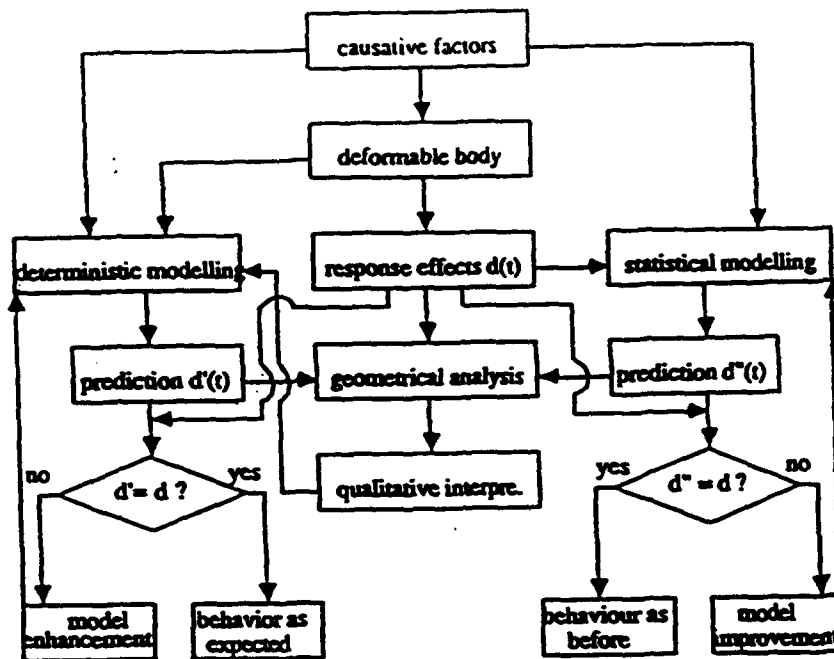
Figure 1: Flowchart of deformation analysis methods and their interaction
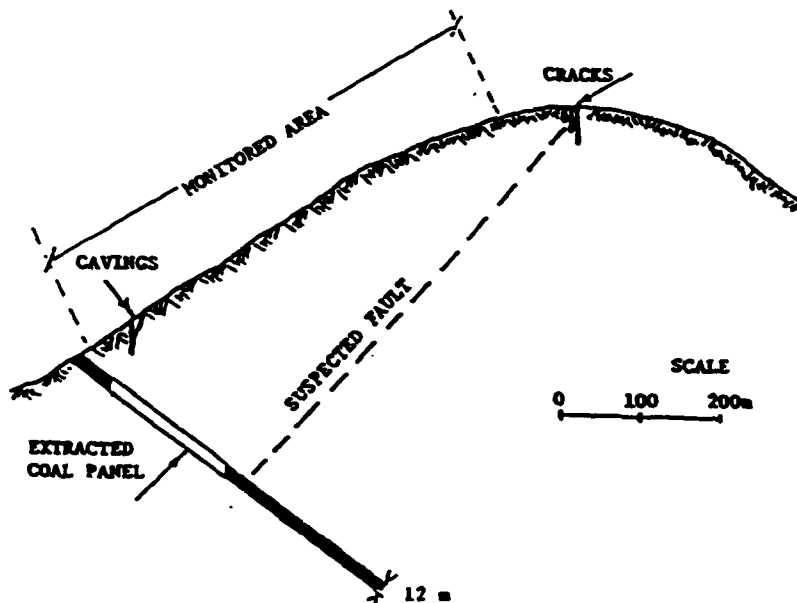

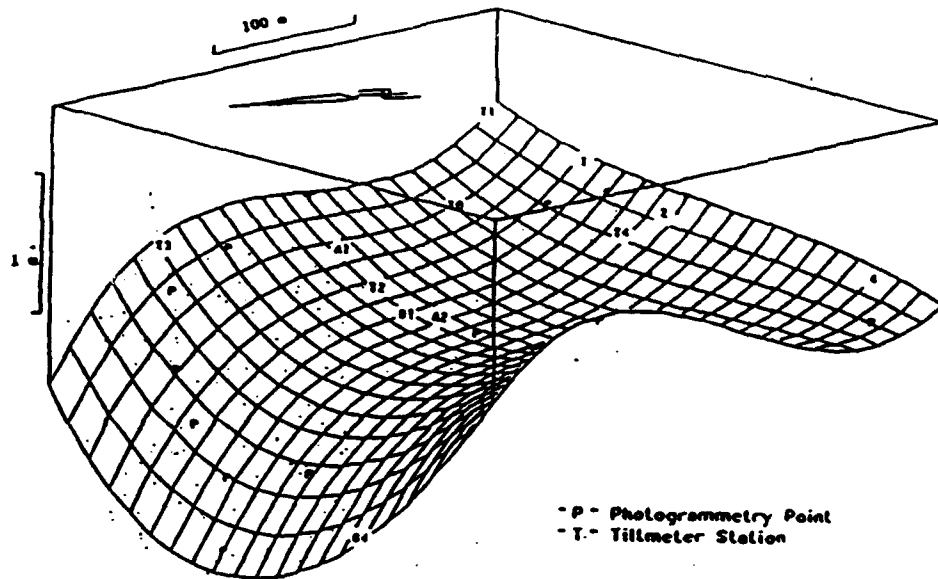Figure 2: Vertical cross section of the Sparwood coal field

Figure 3: Ground subsidence model obtained from geodetic, photogrammetric and tiltmeter measurements
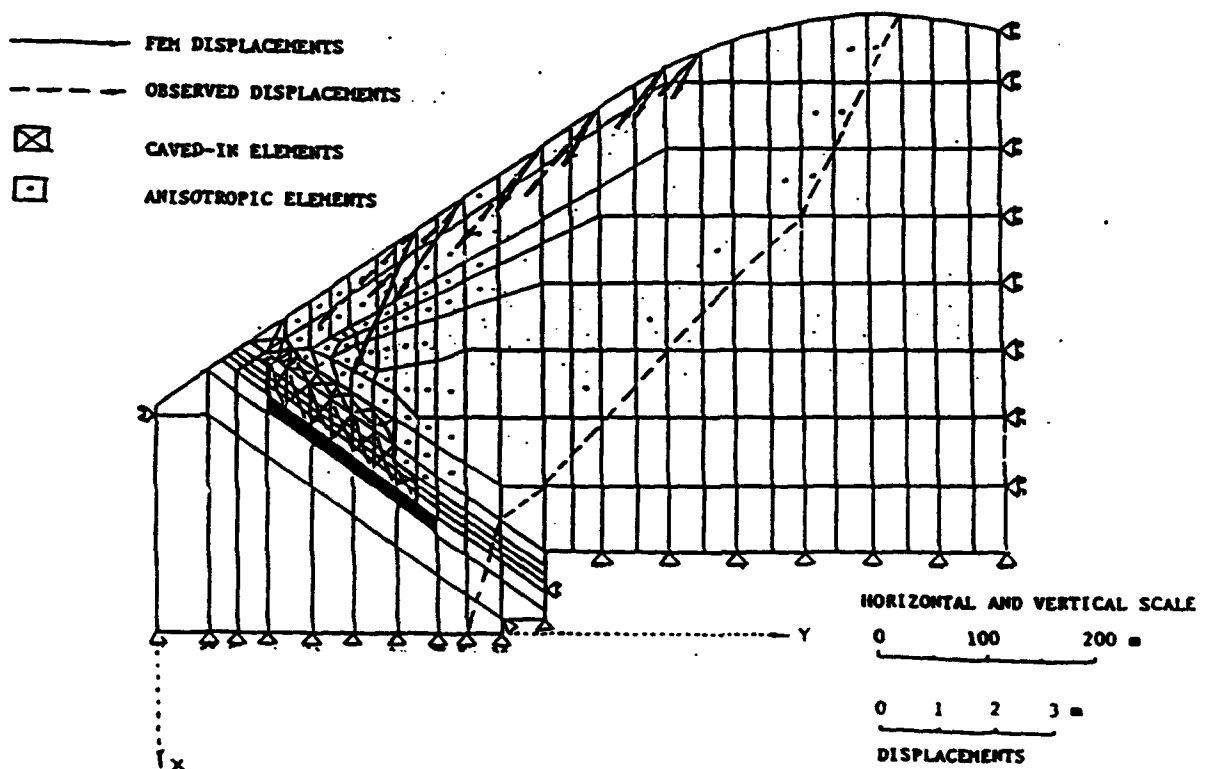

Figure 4: FEM displacements versus the observed values in the Sparwood coal field

agreement with the observed values than without the fault. Figure 4 gives the final FEM model with the fault and the comparison of the observed and calculated displacements.

Combination of the geometrical and deterministic analyses in this example has revealed the existence of the discontinuity in the area. In the refined geometrical analysis, the parameters of rigid block movement would be included in the deformation model. In addition, the deformation survey scheme would need to be expanded beyond the ridge of the mountain in order to better monitor the deformation of the area.

5. INTEGRATED ANALYSIS OF THE DEFORMATIONS AT A HYDRO-ELECTRIC POWER GENERATING STATION

In 1988, the UNB Generalized Method was applied in an integrated analysis of the deformations of the structures at a hydro-electric power generating station in eastern Canada. The generating station under investigation is comprised of a rockfill dam, sluiceway, a 42 m high concrete gravity dam(intake) connected to the powerhouse structure by six penstocks. In the mid-1970s, opening of vertical construction joints was noticed immediately downstream from the turbine/generator block in the powerhouse (Figure 5). The joint is currently open about 30 mm, and shows a steady rate of expansion of about 3mm/year at the upper generator floor, decreasing to about 1mm/year 12 m below. At the same time, leakage through horizontal construction joints in the spillway and the intake has become evident. In 1985, a longitudinal expansion of the intake and the resulting obstruction to the movement of the adjacent spillway gate were observed.

Numerous theories were put forward to explain the abnormal structural deformations and behaviour of the concrete, particularly in the powerhouse and the intake. At first, the theories included regional and local rock movements, transfer of water load through the penstocks to the powerhouse, effects of alkali-aggregate reactivity in concrete, residual stress and squeeze and/or rebound of the foundation. Recently, alkali-aggregate reaction has been determined to be the prime cause of the deformations.

To better understand the mechanism and causes of deformation, an extensive monitoring scheme was developed which includes precision geodetic surveys and measurements with geotechnical instruments such as multi-rod borehole extensometers, invar tape extensometers, suspended and inverted pendula, and various joint meters and tell-tales across the joint openings and structural cracks. The frequency of the repeated measurements varies from weekly observations with some of the geotechnical instrumentation to annual measurements of the main geodetic monitoring network. Figure 5 shows the location of typical instrumentation used in a cross section of the powerhouse/intake structure. Since 1986, the University of New Brunswick (UNB) has added observations with the satellite Global Positioning System (GPS) to the monitoring scheme to monitor regional stability.

Trend analysis of all observations has been performed indicating that the deformations were linear in time with fairly constant rates after having compensated for seasonal periodic variations. Therefore, the average rates of observation change could be taken for the spatial trend analysis. Figure 6 gives an example of rates of deformations (mm/year) obtained from a sample of measurements in one upstream-downstream cross-section of the powerhouse.

The absolute displacements of geodetic points, when compared with the pendula, tell-tale, jointmeter and tape and borehole extensometer measurements at the generator and turbine floors, indicate that the powerhouse is expanding not only downstream but also toward the intake. The tape extensometer measurements indicate that the tailrace part of the structure expands at a much larger rate than the upstream part of the powerhouse. Examination of the borehole and tape extensometer measurements at lower levels also indicates expansion of the foundation concrete both vertically and horizontally, with increased rates in the downstream portion of the whole structure. They also show that the foundation bedrock is perhaps stable, unless it moves as one solid block. The levelling and borehole extensometer data indicate an overal uplift and tilt of the powerhouse. The plumbline and jointmeter measurements indicate a possible relative rotation of the blocks separated by the open joint.

LEGEND

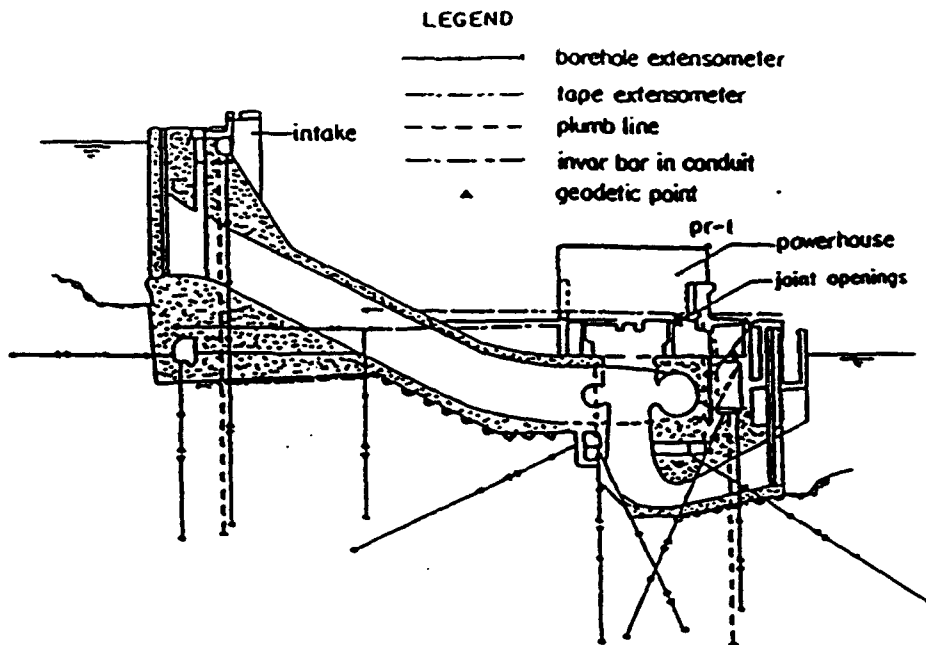|	
———————	borehole extensometer
—·——··—	tape extensometer
— — — —	plumb line
—··——··—	invar bar in conduit
▲	geodetic point

Figure 5: Instrumentation in a cross section of the powerhouse/intake structure
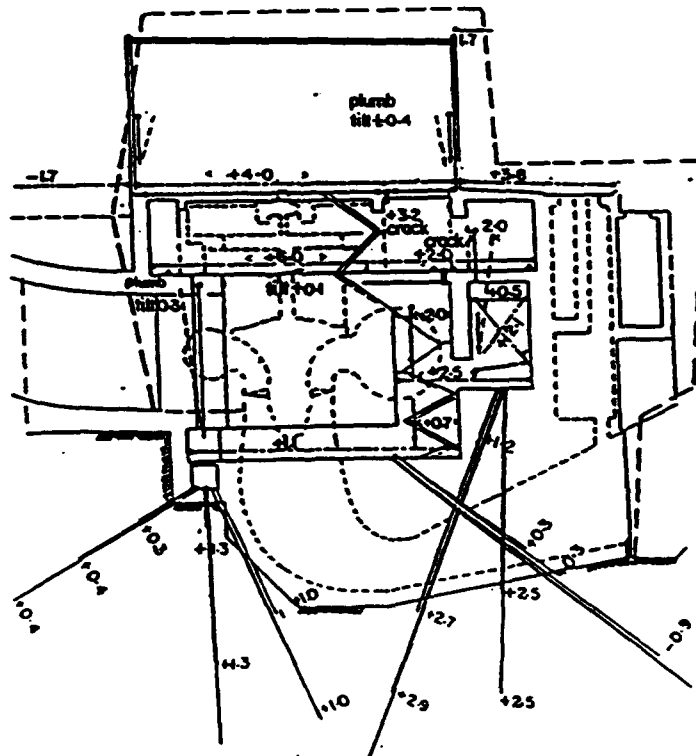
Figure 6: Average deformation and displacement rates [mm/yr] in an upstream-downstream cross section of the powerhouse

As discussed earlier, the goal of the analysis is to find displacement functions which fit into all the observed rates of deformation, in the statistically best way. Once the function has been determined, all basic deformation parameters such as strain components, rotations, and rigid body movements can be calculated at any desired point of the deformable body. The reliability of the displacement function depends on the number of redundant observations and quality of the observations which are included in the deformation modelling.

When analysing the powerhouse, it had to be taken into account that the structure is not homogeneous. There is a zone of the massive concrete foundation, a number of galleries and other empty spaces, turbines, the upper practically empty shell with the roof structure, and, finally the developed joint openings. Each zone may be expected to have different responses to the forces producing the deformation. The foundation bedrock, which may also demonstrate a deformation, must be also considered in the deformation modelling. Being restricted to only about 30 observables in each cross-section of the powerhouse, the division into various deformation zones had to be simplified considerably to provide a reasonable redundancy for the accuracy evaluation. About 20 different functions (full or partial polynomials) have been attempted in fitting the observations data. Figure 7 gives a graphical display of the displacement field, which has been accepted as the statistically best deformation model in a cross-section of the powerhouse. The displacement field and the strain field derived from it indicate a volumetric expansion of the whole structure. The volumetric expansion causes opening of the dilatation gaps and creation of cracks at the contact zones between the expanding and stable materials. One has to emphasize that the derived displacement field gives only a picture of idealized average behaviour of the strucure which, in reality, is very non-homogeneous. Many more observations would be needed to obtain more detail on the behaviour of various zones of the powerhouse. This, however, is not really needed for understanding the general mechanism of the deformation. Since the foundation bedrock seems to be stable, as revealed by the borehole extensometer measurements, the only explanation for the volumetric expansion is an action of body forces such as swelling of the concrete. There is no indication of any downstream rotation of the downstream portion of the powerhouse which could be suggested by the crack opening being larger at the upper levels. It could be produced by the non-homogeneity in the strengh of the material at different levels, mentioned previously.

To verify the findings of the geometrical analysis on the volumetric growth of the structure, finite element modelling of the deformations has been carried out using FEMMA. In the analysis, the assumed growth of the concrete was introduced as an initial strain rate, with the basic rate for solid concrete blocks in the powerhouse being 0.2 mm/m/year (deduced from the geometrical analysis). Because of a non-homogeneous distribution of the steel reinforcement in the structures, a non-uniform expansion of the structure could be expected. Therefore, the input strains in the FEM analysis have been differentiated from one zone to another by reducing the basic rate proportionally to the percentage content of the reinforcement steel in concrete. Figure 8 shows the FEM mesh for the powerhouse and gives a comparison between the displacement field derived from the FEM and from the geometrical analysis. An excellent agreement has been achieved. Thus, in spite of the complexity involved with the various sources and quality of data, the various locations for observation, and behaviour of the structure, the integrated analysis of the deformations of the powerhouse has been achieved and has fully confirmed the preliminary assumption on the growth of concrete being the main source of the deformation.

6. CONCLUDING REMARKS

Deformation analysis is an interdisciplinary subject requiring the knowledge of different fields and a close cooperation among the related specialists. The combination of geometrical analysis and physical interpretaion will contribute greatly to a better understanding of the deformation phenomena. Research activity in this area is just beginning and more work should be done. This paper provides the basic concept on the integrated approach to the deformation analysis. Two practical examples have demonstrated that deformation modelling has been enhanced by combining the geometrical and deterministic analyses.
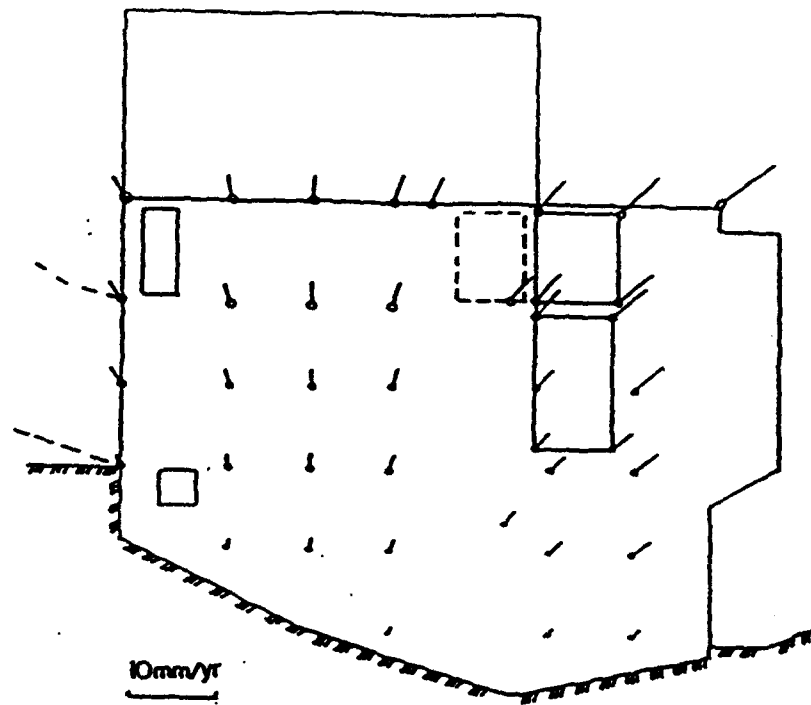
Figure 7: Modelled displacement field (annual rates of displacements)
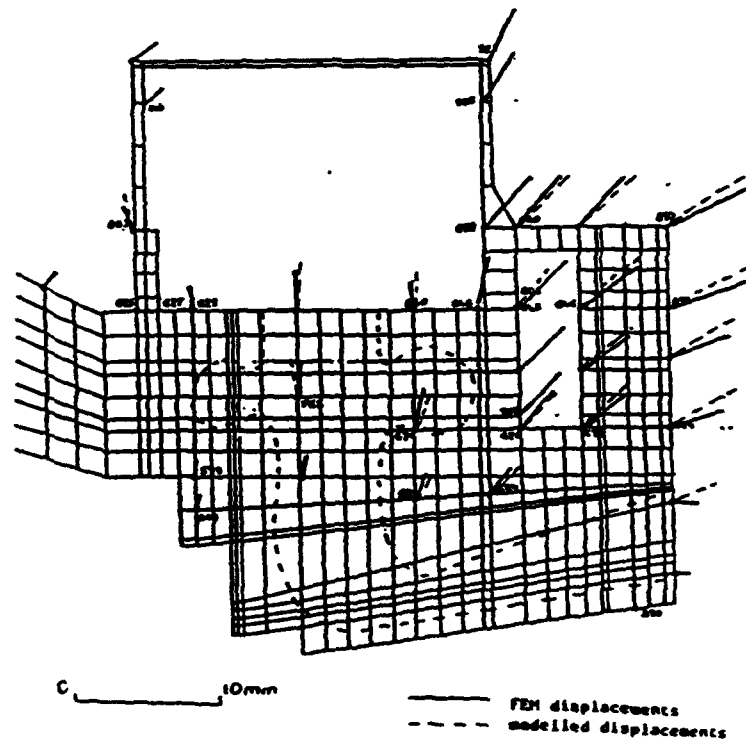
Figure 8: FEM displacements versus the geometrical analysis of observations

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INTEGRATED ANALYSIS OF DEFORMATION SURVEYS AT MACTAQUAC

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Water Power
& DAM CONSTRUCTION
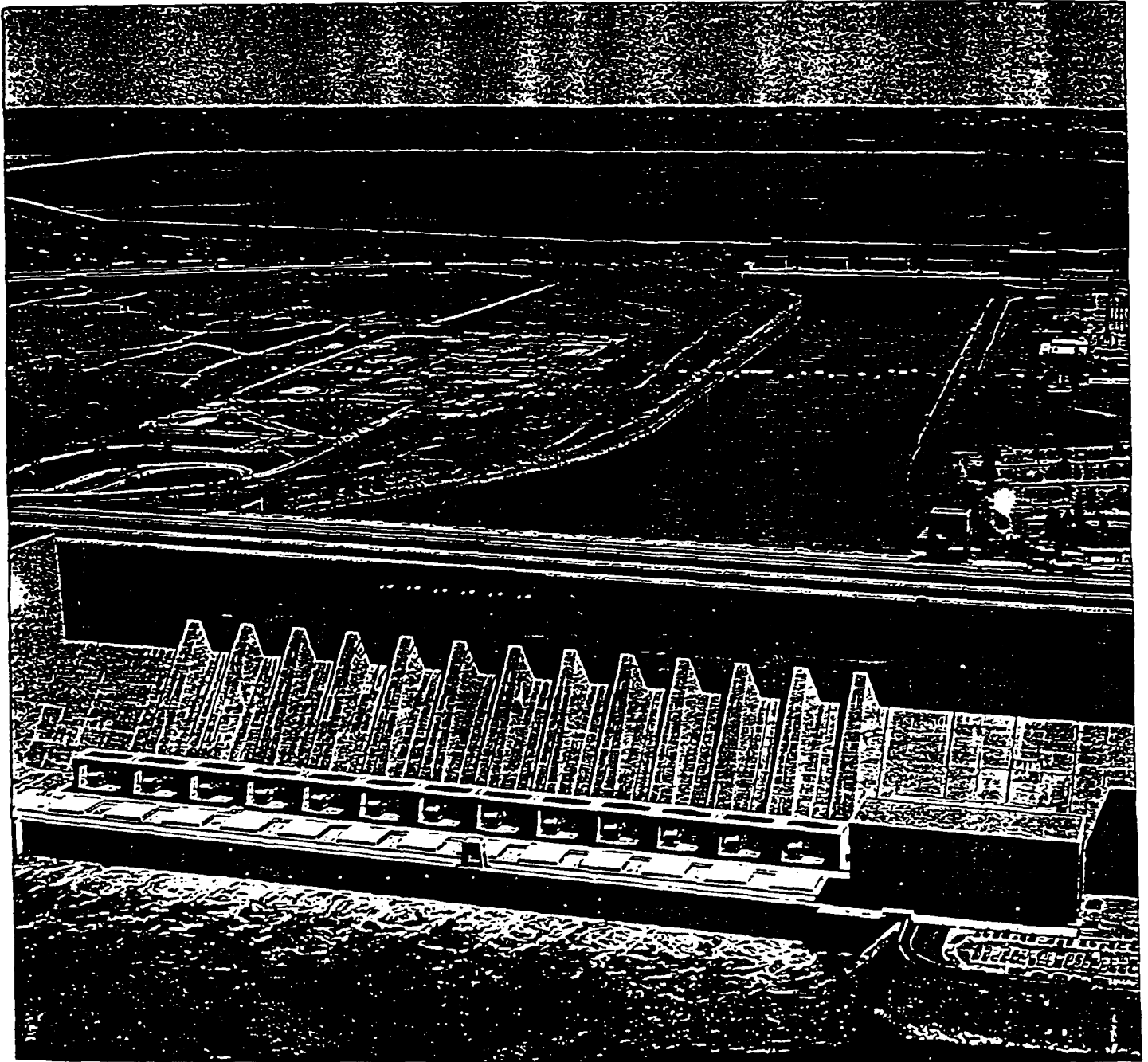
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Hydro and the environment

Integrated analysis of deformation surveys at Mactaquac

By A. Chrzanowski, Chen Yong-qi, J.M. Secord, A. Szostak-Chrzanowski, D.G. Hayward, G.A. Thompson and Z. Wroblewicz, Professor*, Honorary Research Associate* Instructor*, Research Assistant*, Director**, Project Manager** and Superviser of Surveys**

Unusual remedial measures have recently been taken at the Mactaquac dam and associated structures in Canada, to relieve and control the long-term effects of alkali-aggregate reaction in the concrete; the measures taken are described in a separate article in our supplement this month on p4. This article focuses on the generalized method of deformation analysis which was developed at the University of New Brunswick (UNB) to study the problems which were occurring. A complex monitoring system had been implemented at the scheme, and the sources and quality of data obtained varied considerably. The integrated analysis which was developed provided a unique tool to describe mathematically the displacement field from the various types of observed data.

The Mactaquac generating station (Fig. 1) was constructed on the Saint John river in the Province of New Brunswick, eastern Canada, between 1964 and 1968. It comprises a rockfill dam, a sluiceway, a 42 m-high concrete gravity dam (intake and spillway), and a powerhouse with six generating units connected to the intake structure by six penstocks. In the mid-1970s, opening of vertical construction joints was noticed immediately downstream from the turbine/generator blocks in the powerhouse (Fig. 2). The joint is currently open about 30 mm, and shows a steady rate of expansion of about 3 mm/year at the upper generator floor, decreasing to about 1 mm/year 12 m below. At the same time, leakage through horizontal construction joints in the spillway and intake became evident. In 1985, a longitudinal expansion of the intake and resulting obstruction to the movement of the adjacent spillway gate were observed. More details on the noted deformations and damage are given in a paper by Hayward and co-authors[1].

Numerous theories were put forward to explain the abnormal structural deformations and behaviour of the concrete, particularly in the powerhouse and the intake. At first, the theories included regional or local rock movements, transfer of water load through the penstocks to the powerhouse, effects of alkali-aggregate reactivity in concrete, residual stress and squeeze and/or rebound of the foundation. New Brunswick Electric Power Commission commissioned a major consulting firm, as well as a Board of Review comprising international experts, to assist in analysis of the problems and implementation of remedial measures. Alkali-aggregate reaction was determined to be the prime cause of the deformations.

To understand better the mechanism and causes of deformation, an extensive monitoring scheme was developed which included precision geodetic surveys and measurements with geotechnical instruments such as multirod borehole extensometers, invar tape extensometers, suspended and inverted plumblines, and various joint meters and tell-tales across the joint openings and structural cracks. The frequency of the repeated measurements varies from weekly observations with some of the geotechnical instrumentation to annual measurements of the main geodetic monitoring network (Fig. 3). Fig. 2 shows the location of typical instrumentation used in a cross-section of the powerhouse/intake structure. Several thousands of observations have been collected over the last few years. A detailed description of the monitoring instrumentation and measuring techniques is given by Wroblewicz and co-authors[2]. Since 1986, the University of New Brunswick (UNB) has added observations with the satellite Global Positioning System (GPS) to the monitoring scheme, to monitor regional stability[3].
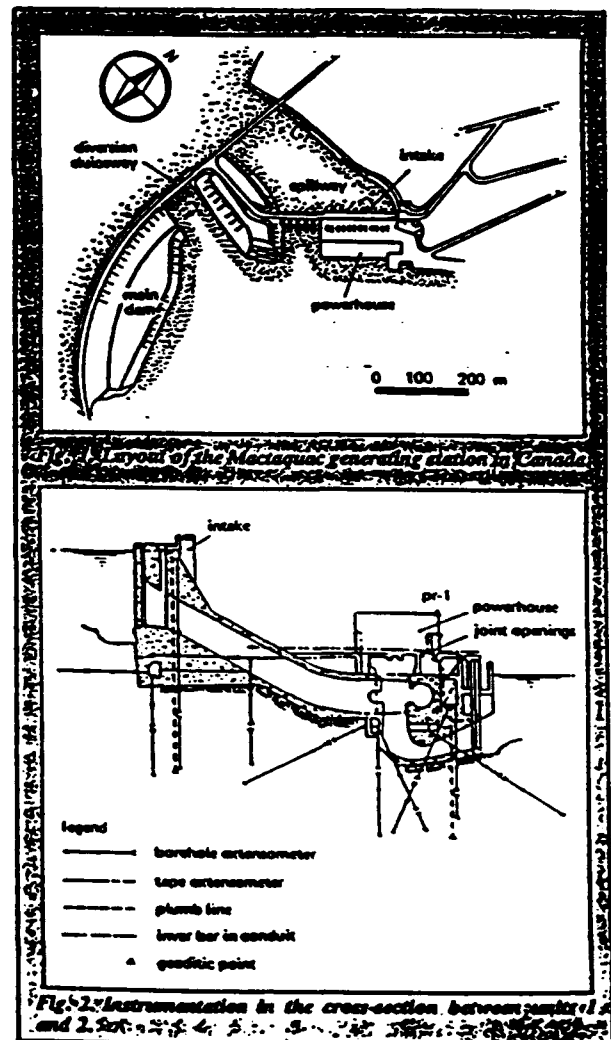
Fig. 1: Layout of the Mactaquac generating station in Canada

Fig. 2: Instrumentation in the cross-section between units 1 and 2.

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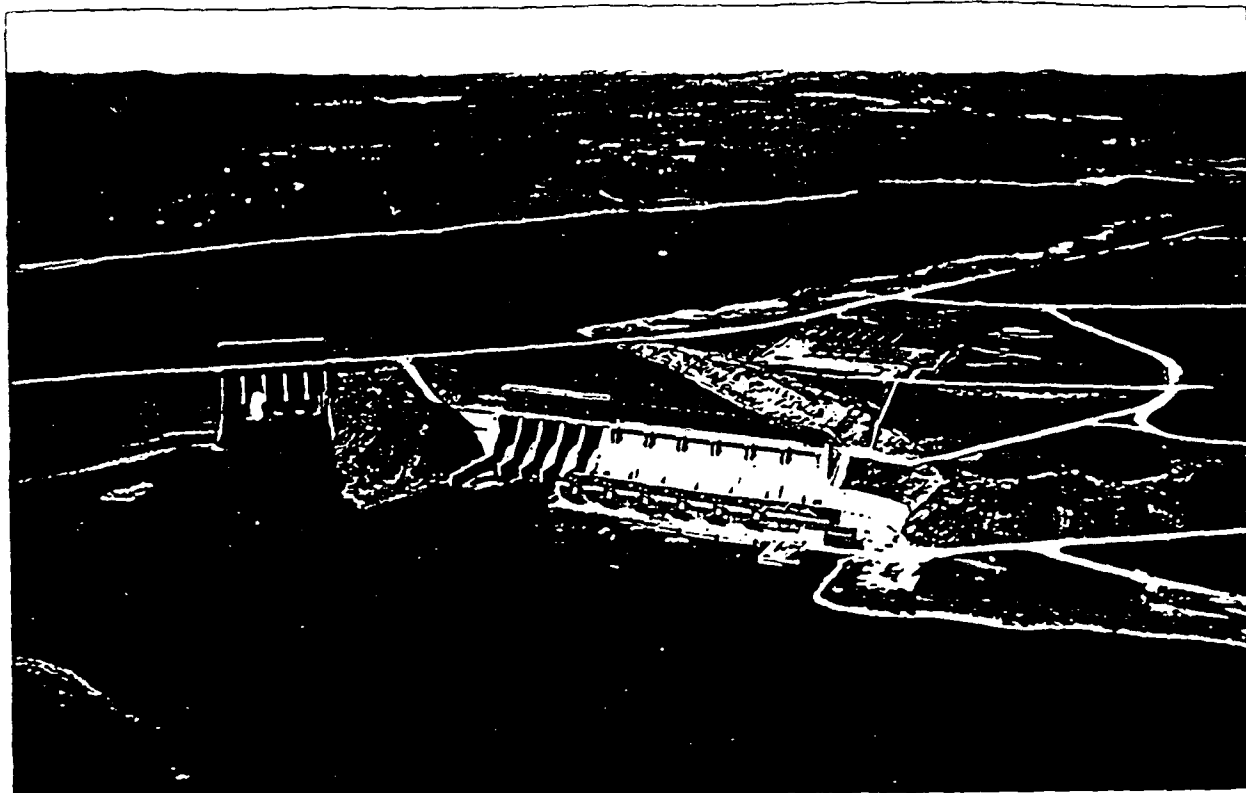
▲
The 630 MW Mactaquac generating station, showing the 300 m-long intake and spillway structure.

Generally, in all deformation studies, any type of observation, whether geodetic or geotechnical, even if scattered in space and time, adds valuable information and helps provide a better understanding of the deformation mechanism and subsequent physical interpretation. Each type of observation supplies valuable data on particular deformation parameters. However, in isolation, none of the individudal types of measurement gives a full picture of the behaviour of the structure, the foundation, or the surroundings. A geotechnical instrument supplies only very localized information on only one component of deformation, possibly without correlation to other instrument locations. In addition, the observational accuracies of geotechnical instruments do not necessarily correspond to those claimed by the manufacturers. On the other hand, geodetic surveys, which are readily evaluated, supply overall information on absolute displacements but are comparatively slow, their accuracy is not always sufficient, and they do not provide continuous information on the behaviour of the monitored structure.

Therefore, the geodetic and geotechnical methods complement each other and, to take full advantage of all the observations, they should be integrated in a simultaneous analysis of the deformation. Until a few years ago, such an integrated analysis was not possible because of the lack of a proper methodology. Between 1981 and 1985, a generalized method for the geometrical analysis of deformation surveys was developed at UNB[1,2,3] which allows for the integration of any type of deformation observations, even if scattered in space and time and having configuration defects.

During the summer of 1987, the UNB generalized method was applied to the analysis of deformations of the powerhouse and intake structures at the Mactaquac generating station.

Deformation analysis

As accepted by 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 relationships 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 involve only discrete points, then the displacement function must be approximated through some selected model which fits observation data in the best possible way.

The displacement function can be written in matrix form as:

$$d(x, y, z, t) = B(x, y, z, t)c \qquad \ldots (1)$$

where $B(x, y, z, t)$ is the deformation matrix with its elements being some selected base functions, and c is the vector of unknown coefficients. A vector Δl of changes in any type of observations, for instance, changes in tilts, in distances or in coordinates derived from geodetic surveys, can be expressed in terms of the displacement function as:

$$\Delta l = AB_{yc} \qquad \ldots (2)$$

where A is the transformation matrix relating the observations to the displacements of points at which the observations are made, and B_y is constructed from the above matrix $B(x, y, z, t)$ and related to the points included in the observables.

If redundant observations are made, the elements of the vector c are determined through a least squares approximation and their statistical significance can be calculated.

One tries to find the simplest possible displacement function, which would fit to the observations in the statistically best way.

The search for the best deformation model (displacement function) is based either on prior knowledge of the expected deformations, or on the qualitative analysis of the deformation trend, deduced from all the observations taken together. The latter procedure had to be applied in the case of Mactaquac, since the cause of deformations had not been clearly identified.

The deformation analysis is carried out in five steps:

● evaluation of observations;
● deformation trend analysis and selection of a few possible deformation models which seem to match the trend and which make physical sense;
● least squares fitting of the model or models into the observation data and statistical testing of the models;
● selection of the best model which has as few coefficients as possible with as high a significance as possible (preferably all the coefficients should be significant at probabilities greater than 95 per cent) and which gives as small a quadratic form of the residuals as possible;
● graphical presentation of the displacement field and the derived strain field.

The above steps have been applied to the analysis of deformations of the powerhouse and intake structures. The present observation scheme in these structures is concentrated in a few cross-sections. Consequently, the trend

analysis and the subsequent final deformation analysis has been limited to a two-dimensional analysis of only the cross-sections.

To undertake an integrated analysis, the spatial correlation of the observations was maintained by describing the locations of all the points or stations of measurement in a common three-dimensional Cartesian coordinate system.

Evaluation of observations
General remarks

All the above-mentioned types of observations have been evaluated for their usefulness in the deformation analysis. The task was not easy, because the results of measurements, in addition to the inherent observation and instrument calibration errors, were contaminated by seasonal (thermal) cyclic expansions of the measured objects, and changeable thermal expansion of the mechanical components of the geotechnical instrumentation (particularly the tape and borehole extensometers). Since the cyclical nature of the temperature influence has a period of about one year, at least two years of observations are needed to take account of this and other deformation trends. Thus, any deformation parameters derived from the observations taken over a time span shorter than two years must be treated with caution. Since most of the instruments at the intake/spillway structures had only been installed recently, the integrated analysis concentrated mainly on the powerhouse, where the monitoring scheme had been established in the period 1983-1984.

Geodetic surveys

A trilateration network (Fig. 3) of 16 reference stations and several object points on the downstream portion of the powerhouse and on the top deck of the intake has been measured annually since 1983, using a Kern Mekometer ME3000 precision electronic distance meter. In addition, high precision levelling surveys have been repeated bi-monthly inside and around the structures connected to deep benchmarks a few kilometres away from the generating station.

To determine absolute displacements of the object stations from various combinations of pairs of survey campaigns, the observation data have been processed following the steps of the UNB generalized method in which the identification of stable reference stations and displacement trend of object points is obtained through an iterative weighted transformation.

The calculated horizontal displacements indicated a systematic downstream movement of the tailrace deck of the powerhouse at the average rate of 3 mm/year. This information played an important role in the overall trend analysis of the deformations.

The levelling surveys were used mainly in the determination of tilts (changes in the height differences) between neighbouring benchmarks within the structures rather than in the determination of the absolute vertical movements of the structures. The latter proved to be unreliable because of the length of the connecting surveys.

Measurements with geotechnical instrumentation

To distinguish between the seasonal (thermal) expansions of the structures and the actual deformation trend, all the geotechnical measurements have been analysed through a
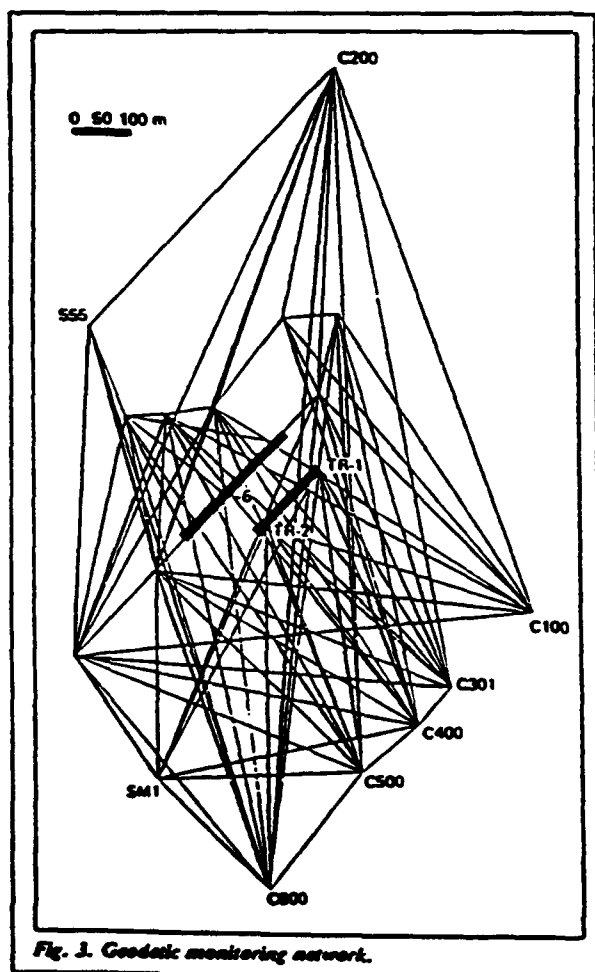

Fig. 3. Geodetic monitoring network.

least squares fit of the cyclic function:

$$y = a_1\cos(\omega) + a_2\sin(\omega) + a_3t + a_4 \quad \ldots(3)$$

to the obersvation data, where $\omega = 2\pi/1$ year, and a_3 is the rate of change of the observation (extension, tilt, inclination, and so on). The values of a_3 and their standard deviations have been the most important parameters in the deformation analysis. In most cases, particularly in borehole extensometer and plumbline observations, a very smooth plot of the observation data and good fitting of the cyclic curve has been obtained with standard deviations smaller than 0.2 mm/year. An example is shown in Fig. 4.

Some problems were encountered with the invar tape extensometer measurements. Most of the tape measurements in the powerhouse began in 1984, giving a long enough time interval to perform a good analysis of the thermal effects. However, during this period, the tape was broken on several occasions, and the tension device was either repaired or exchanged. Therefore, the results lacked continuity and included many gaps or slips, which required a lengthy process of filtering the available data through least squares estimation of the values of the slips. The slips have been added to the cyclic function (3) and solved as nuisance unknown parameters. The uncertainty (standard deviation) of the corrected changes in the distances has been established as being about 0.02 mm/m/year.

Trend analysis of the deformations

The evaluation of observations has indicated that the deformations were linear in time, with their rates fairly constant. Therefore, the average rates of observation change could be taken for the spatial trend analysis. Fig. 5 gives an example of rates of deformations (mm/year) obtained from a sample of measurements in one cross-section of the powerhouse.

The absolute displacement of the geodetic points, when compared with the plumbline, tell-tale, jointmeter and tape extensometer measurements at the generator turbine floors, indicates that the powerhouse is expanding not only downstream but also towards the intake. The tape extensometer measurements indicate that the tailrace part of the structure expands at a much larger rate than the upstream part of the powerhouse. Examination of the borehole and tape extensometer measurements at lower levels also indicates expansion of the foundation concrete both vertically and horizontally, with the increased rates in the downstream portion of the whole structure. They also show that the foundation bedrock is perhaps stable, unless it moves as one solid block. The levelling and borehole extensometer data indicate an overall uplift and tilt of the powerhouse. The plumblines and jointmeter measurements indicate a possible relative rotation of the blocks separated by the opened joint.

The deformation trend of the intake and spillway structures could not be analysed fully because most of the instruments have been installed only recently and the observation data have not covered at least a whole cycle of thermal expansion. Therefore, only a very approximate deformation trend analysis for the intake could be performed, which indicates that there seems to be an interaction between the deformation of the intake and the powerhouse.

Following the above summarized deduction procedure and looking at various possible forces acting on the power-
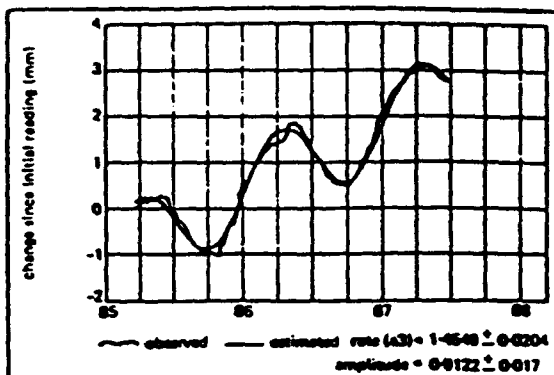
Fig. 4. Example of the least squares fitting of a cyclic function to borehole extensometer measurements; determination of the extension rate is shown in mm/year.
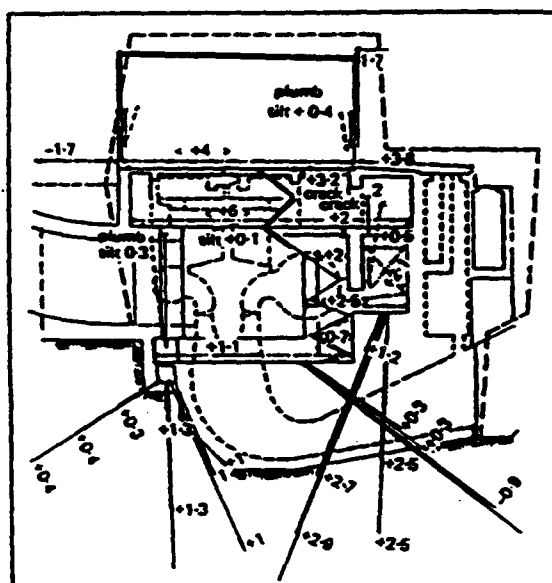
Fig. 5. Average deformation and displacement rates in mm/year, and deformation trend between units 1 and 2.

house (not excluding possible tectonic forces and possible interaction between the intake and the powerhouse), the analysis has arrived at the overall geometrical deformation trend as shown by the dashed lines in Fig. 5. Analysis in other cross-sections has confirmed the trend.

Final deformation modelling

As discussed earlier, the goal of the analysis is to find displacement functions which fit statistically into all the observed rates of deformation. Once the function has been determined, all basic deformation parameters such as strain components, rotations, and rigid body movements can be calculated at any desired point of the deformable body. The reliability of the displacement function depends on the number of redundant observations and quality of the observations which are included in the deformation modelling.

Regarding the powerhouse, it had to be taken into account that the structure is not homogeneous. There is a

Fig. 6. Modelled displacement field (annual rate of displacements).

zone of the massive concrete foundation, a number of galleries and other empty spaces, turbines, the upper practically empty shell with the roof structure, and, finally the developed joint openings. Each zone may be expected to have different responses to the forces producing the deformation. The foundation bedrock, which may also demonstrate a deformation, must be also considered in the deformation modelling.

Being restricted to only about 30 observables in each cross-section of the powerhouse, the division into various deformation zones had to be simplified considerably to provide a reasonable redundancy for the accuracy evaluation.

About 20 different functions (full or partial polynomials) have been attempted in fitting the observation data. Fig. 6 gives a graphical display of the displacement field, which has been accepted as the statistically best deformation model in the cross-section of the powerhouse.

The displacement field and the strain field derived from it indicate a volumetric expansion of the whole structure. The volumetric expansion causes opening of the dilatation gaps and the creation of cracks at the contact zones between the expanding and stable materials. One has to emphasize that the derived displacement field gives only a picture of idealized average behaviour of the structure which, in reality, is very non-homogeneous. Many more observations would be needed to obtain more detail on the behaviour of various zones of the powerhouse. This, however, is not really needed for understanding the general mechanism of the deformation. Since the foundation bedrock seems to be stable, as revealed by the borehole extensometer measurements, the only explanation for the volumetric expansion is an action of body forces such as swelling of the concrete, unless the flow of the water through the turbines and outlets and the interaction with the deforming intake structure cause a complicated fracturing, rotation and uplifting of the whole powerhouse. There is no indication of any downward rotation of the downstream portion of the powerhouse which could be suggested by the crack opening being larger at the upper levels. It could be produced by the non-homogeneity, mentioned previously, in the strength of material at different levels.

To verify the findings of the geometrical analysis on the

volumetric growth of the structures, finite element modelling of the deformations has been carried out using FEMMA software developed by Szostak-Chrzanowski[8] at the University of New Brunswick. In the analysis, the assumed growth of concrete was introduced as initial strain rates, with the basic rate (deduced from the geometrical analysis) for solid concrete blocks in the powerhouse being equal to 0.2 mm/m/year. Because of a non-homogeneous distribution of the steel reinforcement in the structures, a non-uniform expansion of the structures could have been expected. Therefore, the input strains in the FEM analysis have been differentiated from one zone to another by reducing the basic rate proportionally to the percentage content of the reinforcement steel in concrete. An excellent agreement has been obtained between the displacement fields derived from the FEM and from the geometrical analyses.

Conclusions

In spite of the complexity involved with the various sources and quality of data, the various locations for observation, and behaviour of the structure, an integrated analysis of the deformations of the powerhouse has been achieved.

The integrated analysis has been possible by using the UNB generalized method, which provides a unique tool to describe mathematically the displacement field from various types of observables. The displacement model allows for the derivation of the strain field which is essential for subsequent physical interpretation of the deformation.

□

Acknowledgement

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COMBINATION OF GEOMETRICAL ANALYSIS WITH PHYSICAL INTERPRETATION FOR THE ENHANCEMENT OF DEFORMATION MODELLING

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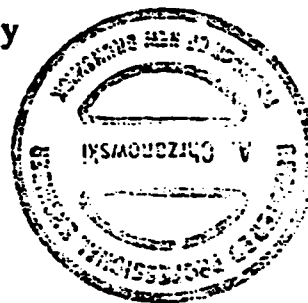
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GEOMETRICAL ANALYSIS OF DEFORMATION SURVEYS

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Abstract

The geometrical analysis of deformation surveys deals with the determination of the geometrical status of a deformable body — the change of its shape and dimensions. Since the deformations are usually very small and at the margin of measuring errors, very careful analysis and statistical testing of the results are required. The deformation of a body is fully described in three-dimensional space if 9 deformation parameters (6 strain components and 3 differential rotation components) can be determined at each point. These deformation parameters can be calculated from the well-known strain-displacement relationships if a displacement function representing the deformation body is known.

A methodology for finding the "best" fitting displacement function has been developed by the authors, and is known as the *UNB Generalized Method*. The Method consists of three basic processes: preliminary identification of deformation models through a trend analysis, estimation of the deformation parameters through a least-squares fitting of selected displacement functions to repeated deformation observations, and the final selection of the "best" model based on the diagnostic checking of the model and statistical testing of individual deformation parameters. The Method is applicable to any type of geometrical analysis, both in space and in time, including the detection of an unstable area and the determination of strain components and relative rigid body motions within a deformed object. It allows utilization of any type of surveying data and geotechnical measurements with configuration defects in the observation scheme. Computer program DEFNAN helps to apply the generalized method in practice. Examples of its applications are presented.

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1. Introduction

Expanding exploitation of mineral resources under populated areas, rapid progress in the development of large and sensitive engineering constructions, and growing interest in the study of earth crustal movements have all put new demands on the accuracy, survey methodology, and analysis of deformation measurements.

The instruments and methods of conventional geodetic and photogrammetric surveys, though still useful in collecting global deformation data, cannot satisfy all the requirements of contemporary deformation monitoring. Typical requirements are accuracies in the order of 10^{-6} and 10^{-7}, continuous monitoring with automatic recording, and telemetric data acquisition. Special instrumentation for the detection of deformations, for example, precision tiltmeters, inverted pendula, strainmeters, extensometers, mechanical and laser alignment equipment, hydrostatic levels and interferometers, is being used in structural, geotechnical, tectonic, and rock mechanics monitoring.

In order to provide a strong basis of data for any deformation analysis, the surveyor must employ new technologies and must be able to integrate all types of measurements into a comprehensive "network" of observables. This compels the surveyor to have a good understanding of the purpose and the methods of analysis of deformation surveys.

The analysis of deformations deals usually with very small deformation quantities which are at the margin of measuring errors. Therefore, a very careful accuracy analysis and statistical testing of the results are required in order to make proper decisions on the acceptance of the deformation models.

Thus the survey methods, their design and the analysis of the deformation surveys become very complex. Till now, surveyors have been little, or not at all, involved in the deformation interpretation which usually has been done by other specialists. Emphasis must be placed on the danger inherent when the surveyor, whose realm is measurement processes and the associated errors and statistical considerations, is not able to direct the interpretation of the data. Often severe misinterpretation regarding a phenomena can occur if due regard for the "quality" of the data is not given. Certainly no specialist should be the sole analyst, and it is the interaction of the survey engineer with the data user that should be encouraged. Thus a strong interaction between the survey engineer and other specialists who are in charge of the geotechnical, construction, or geophysical project is necessary during the entire life of a project.

In the past few years, more attention has been paid to the analysis of deformation surveys than ever before. In 1978, Commission 6 of the Fédération Internationale des Géomètres (FIG) created an ad hoc committee on the analysis of deformation

measurements under the chairmanship of Dr. Chrzanowski. The main task of the committee has been to compare different approaches to deformation analysis using the same measuring data with an ultimate goal to prepare a proposal for guidelines and specifications for all aspects of deformation analysis, including studies in the following items:

(1) optimization and design of monitoring networks with geodetic and non-geodetic observables;

(2) assessment of the observation data, detection of outliers, and systematic errors;

(3) geometrical analysis of deformations;

(4) physical interpretation of deformations, e.g., establishment of load-deformation relationships.

During the period 1978-1982, membership in the committee was limited to only five research centres in order to avoid difficulties and delays in the exchange of information and organization of the working meetings. The five groups, called by the names of their location (with the names of the original chief investigators in parentheses) were: Delft (J. Kok), Fredericton (A. Chrzanowski), Hannover (W. Niemeier and H. Pelzer), Karlsruhe (B. Heck and J. Van Mierlo), and Munich (W. Welsch). After the third FIG symposium on deformation surveys, which was held in Budapest in 1982, 14 more groups joined the committee. A full list of the member groups was given in Chrzanowski and Secord [1983]. At the 1986 XVIII FIG Congress in Toronto, a general theory of deformation analysis was presented, and the approaches developed by the groups of the committee were compared in the general theory [Chrzanowski and Chen, 1986].

In these notes, the authors will provide a contemporary methodology for the analysis of deformation measurements. Due to the space limitations, no detailed derivation of the formulae will be provided, but references may be consulted.

2. General Background on the Analysis of Deformation Surveys

2.1 General Classification of Deformation Analysis Methods

If acted upon by external forces (loads), any real material deforms, i.e., changes its dimensions and shape. Under the action of loads, internal stresses (force per unit area) are produced. If the stresses exceed certain critical values, the material fails (breaks). Thus the following two aspects of deformation should be distinguished in the analysis of deformation surveys:

(1) geometrical, if we are interested only in the geometrical status of the deformable body, the change of its shape and dimensions;

(2)   physical, if we want to determine the physical status of the deformable body, the state of internal stresses, and, generally, the load-deformation relationship.

In the first case, information on the acting forces and stresses and on physical properties of the body are of no interest to the interpreter or are not available. As a final result of the geometrical analysis of deformation surveys, usually only relative displacements of discrete points are given with their variance-covariance matrix. The geometrical analysis is of particular importance when the deformable structure is supposed to satisfy certain geometrical conditions, such as verticality or the alignment of some of its components. In that case, the results of the deformation surveys are directly utilized in an adjustment of the geometrical status.

In a more refined geometrical analysis, when an overall picture of the geometrical status is required, the displacement field (or fields) for the entire body is approximated through a least-squares fitting of a selected displacement function (deformation model) into the observed displacements, as discussed in Chrzanowski et al. [1983]. The displacement field may be readily transformed into a strain field through the well-known strain-displacement relationship.

In the physical analysis of deformations, the load-deformation relationship may be modelled by using either an *empirical (statistical) method,* through a correlation of observed deformations with the observed loads; or a *deterministic method,* which utilizes information on the loads, properties of the material, and physical laws governing the stress-strain relationship.

In this presentation, only the geometrical analysis of deformation surveys will be discussed. The physical interpretation of deformations will be briefly discussed in another presentation by Chen and Chrzanowski.

2.2 Classification of Geodetic Monitoring Networks

Generally, in deformation measurements by geodetic methods, whether they are performed for monitoring engineering structures or ground subsidence in mining areas or tectonic movements, two basic types of geodetic networks are distinguished [Chrzanowski et al., 1981]:

(1)   absolute networks in which some of the points are, or are assumed to be, outside the deformable body (object) thus serving as reference points (reference network) for the determination of absolute displacements of the object points (Figure 2.1);

(2)   relative networks in which all the surveyed points are assumed to be located on the deformable body (Figure 2.2).

In the first case, the main problem of deformation analysis is to confirm the stability
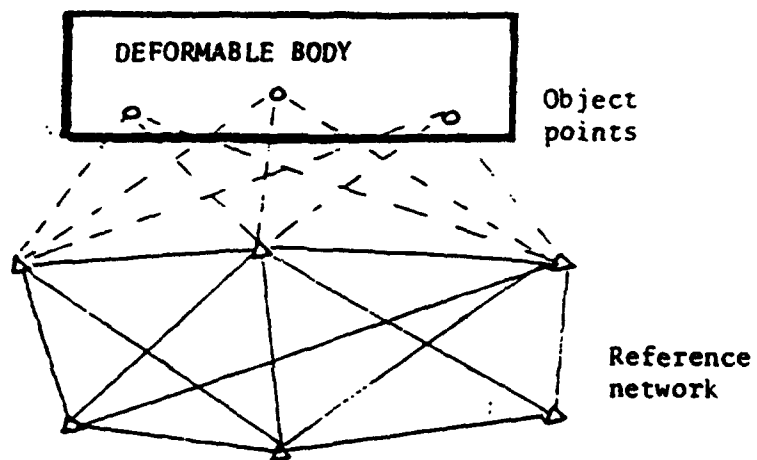
Figure 2.1   Absolute monitoring network
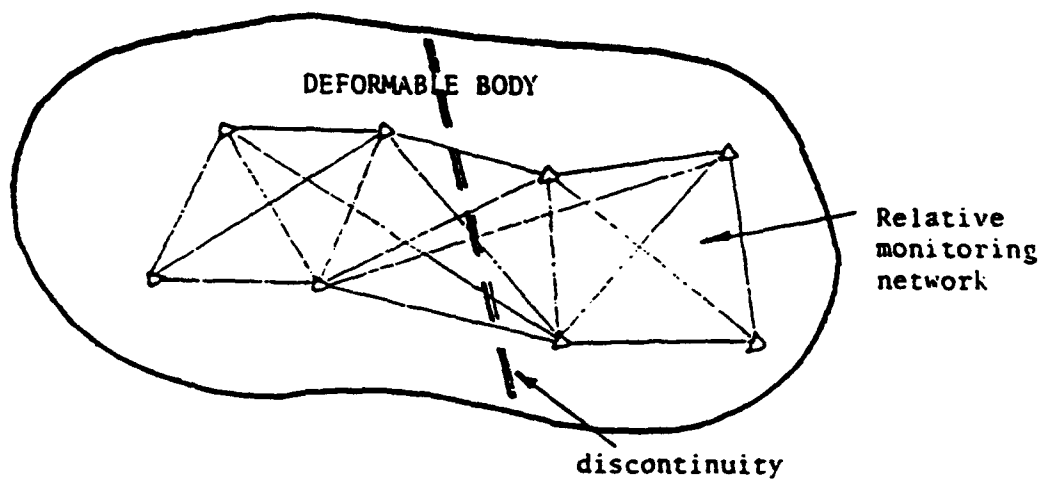
Figure 2.2 Relative monitoring network

of the reference points and to identify the possible single point displacements caused, for instance, by local surface forces and wrong monumentation of the survey markers. Once the stable reference points are identified, the determination of the geometrical state of the deformable body is rather simple.

In relative networks, deformation analysis is more complicated because, in addition to the possible single point displacements like in the reference network, all the points undergo relative movements caused by strains in the material of the body and by relative rigid translations and rotations of parts of the body if discontinuities in the material (tectonic faults, for instance) are present. The main problem in this case is to identify the deformation model. From repeated geodetic observations, it is necessary to distinguish between the deformations caused by the extension and shearing strains, by the relative rigid body displacements and by the single point displacements.

3. Deformation Modelling

3.1 Deformation Parameters

The deformation of a body is fully described in three-dimensional space if 9 deformation parameters, 6 strain components and 3 differential rotation components, can be determined at each point. 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 if a displacement function representing the deformation of the body is known. Denote the displacement function by

$$d(x, y, z; t - t_0) = \begin{bmatrix} u(x, y, z; t - t_0) \\ v(x, y, z; t - t_0) \\ w(x, y, z; t - t_0) \end{bmatrix} \tag{3.1}$$

with u, v, w as the components respectively of the displacement in the x, y, z directions, which are functions of both position and time. Then the normal strains designating elongation or compression in the directions x, y, z are calculated from:

$$\varepsilon_x = \partial u / \partial x \quad , \qquad \varepsilon_y = \partial v / \partial y \quad , \qquad \varepsilon_z = \partial w / \partial z \quad , \tag{3.2}$$

and the shear strains characterizing the distortion of the angles between initially corresponding lines are obtained as

$$\varepsilon_{xy} = (\partial u / \partial y + \partial v / \partial x) / 2$$
$$\varepsilon_{xz} = (\partial u / \partial z + \partial w / \partial x) / 2 \tag{3.3}$$
$$\varepsilon_{yz} = (\partial v / \partial z + \partial w / \partial y) / 2 \quad .$$

The differential rotations around the x, y, z axes are expressed as

$$\omega_x = (\partial v/\partial z - \partial w/\partial y)/2$$

$$\omega_y = (\partial u/\partial z - \partial w/\partial x)/2 \qquad\qquad (3.4)$$

$$\omega_z = (\partial u/\partial y - \partial v/\partial x)/2 \ ,$$

respectively. In general, the above derived quantities are time dependent and their derivatives with respect to time provide the strain rate.

Certain functions of these strain parameters, for instance, maximum strain, dilatation, pure shear, simple shear, and total shear, may also be of interest and their definitions can be found in, e.g., Sokolnikoff [1956] or Frank [1966]. Thus the main task of deformation analysis is to obtain a displacement function, which characterizes the deformation in space and in time.

3.2 Deformation Models

Since, in practice, deformation surveys are made only at discrete points, the deformation of a body must be approximated through some selected model which fits into the observation in the best possible way. The displacement function (eqn. (3.1)) can be expressed in matrix form as:

$$d = B c \ , \qquad\qquad (3.1')$$

where B is called the *deformation matrix with its elements being functions of the position* of the observation points and of time, and c is the vector of unknown coefficients to be estimated. For illustration, examples of typical deformation models in two-dimensional space are given below.

(1) Single point displacement or a rigid body displacement of a group of points, say, block B (Figure 3.1a) with respect to block A. The deformation model is expressed as:

$$u_A = 0 \ , \quad v_A = 0 \ ; \quad u_B = a_0 \ \text{and} \ v_B = b_0 \ , \qquad\qquad (3.5)$$

where the subscripts represent all the points in the indicated blocks.

(2) Homogeneous strain in the whole body and differential rotation (Figure 3.1b), the deformation model is linear as

$$u = \varepsilon_x x + \varepsilon_{xy} y - \omega y$$

$$v = \varepsilon_{xy} x + \varepsilon_y y + \omega x \qquad\qquad (3.6)$$

where the physical meaning of the coefficients is defined in eqns (3.2) to (3.4) with ω_z in eqn. (3.4) being replaced by ω.

(3) A deformable body with one discontinuity (Figure 3.1c), say, between blocks A and B, and with different linear deformations in each block plus a rigid body displacement of B with respect to A. Then the deformation model is written as

$$u_A = \varepsilon_{xA}x + \varepsilon_{xyA}y - \omega_A y$$
$$v_A = \varepsilon_{xyA}x + \varepsilon_{yA}y + \omega_A x \qquad (3.7a)$$

and

$$u_B = a_0 + \varepsilon_{xB}(x - x_0) + \varepsilon_{xyB}(y - y_0) - \omega_B(y - y_0)$$
$$v_B = b_0 + \varepsilon_{xyB}(x - x_0) + \varepsilon_{yB}(y - y_0) + \omega_B(x - x_0) , \qquad (3.7b)$$

where x_0, y_0 are the coordinates of any point in block B.

The components Δu_i and Δv_i of a total relative dislocation at any point i located on the discontinuity line between blocks A and B can be calculated as:

$$\Delta u_i = u_B(x_i, y_i) - u_A(x_i, y_i) \qquad (3.8)$$

and

$$\Delta v_i = v_B(x_i, y_i) - v_A(x_i, y_i) . \qquad (3.9)$$
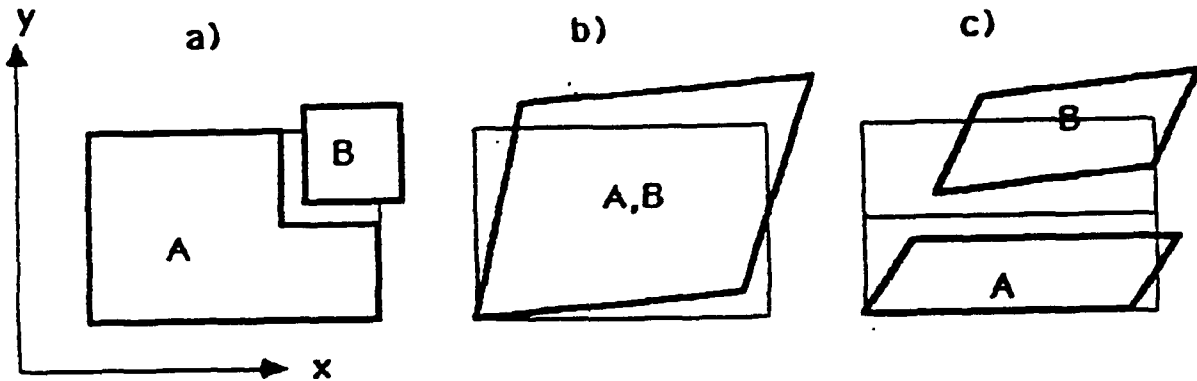
Figure. 3.1 Typical deformation models.

Usually, the actual deformation model is a combination of the above simple models or, if more complicated, it is expressed by non-linear displacement functions which require fitting of higher-order polynomials or other suitable functions.

If time dependent deformation parameters are sought, then the above deformation models will contain time variables. For instance, in the first model above (eqn. (3.5)), if the velocity (rate) and acceleration of the dislocation of block B with respect to block A are to be found, the deformation model would be

$$u_A = 0 , \quad v_A = 0 , \quad u_B = \dot{a}_0 t + \ddot{a}_0 t^2 \text{ and } v_B = \dot{b}_0 t + \ddot{b}_0 t^2 \qquad (3.10)$$

and, in the model of the homogeneous strain, if a linear time dependence is assumed, the model becomes:

$$u(x, y, t) = \dot{\varepsilon}_x xt + \dot{\varepsilon}_{xy} yt - \dot{\omega} yt \qquad (3.11)$$
$$v(x, y, t) = \dot{\varepsilon}_{xy} xt + \dot{\varepsilon}_y yt + \dot{\omega} xt , \qquad (3.12)$$

where the dot above the parameters indicates their rate (velocity) and the double dot their acceleration.

3.3 The Functional Relationship Between the Deformation Model and the Observed Quantities.

Any observation, geodetic or photogrammetric, or geotechnical measurement made in deformation surveys will contribute to the determination of deformation parameters and should be fully utilized in the analysis. The functional relationships between different observable types and the deformation model, defined in eqns (3.1) and (3.1'), are given below using a local coordinate system.

(1)   Observation of **coordinates** of point i, for instance, the coordinates derived from photogrammetric measurements or obtained using space techniques:

$$\begin{bmatrix} x_i(t) \\ y_i(t) \\ z_i(t) \end{bmatrix} = \begin{bmatrix} x_i(t_0) \\ y_i(t_0) \\ z_i(t_0) \end{bmatrix} + \begin{bmatrix} u_i \\ v_i \\ w_i \end{bmatrix} , \qquad (3.13)$$

or

$$r_i(t) = r_i(t_0) + d_i = r_i(t_0) + Bc , \qquad (3.13')$$

where r_i is the position vector of point i, and the others are defined in eqn. (2.1).

(2)   Observation of **coordinate differences** between points i and j, e.g., height difference (levelling) observation, pendulum (displacement) measurement, and alignment survey:

$$\begin{bmatrix} x_j(t) - x_i(t) \\ y_j(t) - y_i(t) \\ z_j(t) - z_i(t) \end{bmatrix} = \begin{bmatrix} x_j(t_0) - x_i(t_0) \\ y_j(t_0) - y_i(t_0) \\ z_j(t_0) - z_i(t_0) \end{bmatrix} + \begin{bmatrix} u_j - u_i \\ v_j - v_i \\ w_j - w_i \end{bmatrix} \qquad (3.14)$$

or

$$r_j(t) - r_i(t) = r_j(t_0) - r_i(t_0) +$$
$$+ \{B(x_j, y_j, z_j; t-t_0) - B(x_i, y_i, z_i; t-t_0)\} c . \qquad (3.14')$$

If the components of the displacement obtained from a pendulum observation do not coincide with the coordinate axes, a transformation to the common coordinate system has to be performed. Similarly, a coordinate transformation may be required in alignment surveys which provide a transverse displacement of a point with respect to a straight line defined by two base points.

(3) Observation of **azimuth** from point i to point j

$$\alpha_{ij}(t) = \alpha_{ij}(t_0) + [(-\cos\alpha_{ij}) / (S_{ij}\cos\beta_{ij}) , (\sin\alpha_{ij}) / (S_{ij}\cos\beta_{ij})] \begin{bmatrix} u_j - u_i \\ v_j - v_i \end{bmatrix} \quad (3.15)$$

where β_{ij} and S_{ij} are the vertical angle and spatial distance from point i to point j, respectively. The observation of a horizontal angle is expressed as the difference of two azimuths.

(4) Observation of the **distance** between points i and j:

$$S_{ij}(t) = S_{ij}(t_0) + (\cos\beta_{ij} \sin\alpha_{ij} , \quad \cos\beta_{ij} \cos\alpha_{ij} , \quad \sin\beta_{ij}) \begin{bmatrix} u_j - u_i \\ v_j - v_i \\ w_j - w_i \end{bmatrix} . \quad (3.16)$$

(5) Observation of **strain** along the azimuth α and vertical angle β at point i:

$$\epsilon(t) = \epsilon(t_0) + p^T E p , \quad (3.17)$$

where

$$p^T = (\cos\beta \sin\alpha, \cos\beta \cos\alpha, \sin\beta)$$

$$E = \begin{bmatrix} \partial u/\partial x & \partial u/\partial y & \partial u/\partial z \\ \partial v/\partial x & \partial v/\partial y & \partial v/\partial z \\ \partial w/\partial x & \partial w/\partial y & \partial w/\partial z \end{bmatrix}$$

(6) Observation of a **vertical angle** at point i to point j:

$$\beta_{ij}(t) = \beta_{ij}(t_0) + \left(\frac{\sin\beta_{ij} \sin\alpha_{ij}}{S_{ij}}, - \frac{\sin\beta_{ij} \cos\alpha_{ij}}{S_{ij}}, \frac{\cos\beta_{ij}}{S_{ij}} \right) \begin{bmatrix} u_j - u_i \\ v_j - v_i \\ w_j - w_i \end{bmatrix} \quad (3.18)$$

(7) Observation of a **horizontal tiltmeter**:

$$\tau(t) = \tau(t_0) + (\partial w/\partial x) \sin\alpha + (\partial w/\partial y) \cos\alpha \quad (3.19)$$

where α is the orientation of the tiltmeter.

In the above formulae, the quantities u, v, w and their derivatives are replaced by the deformation model which is explicitly expressed in eqn. (3.1'). Thus all the observations are functions of the unknown coefficients c.

4. Remarks on the Adjustment of Monitoring Networks

As discussed in section 3.3, the deformation parameters can be directly estimated from the observations. However, if the observation scheme includes a complete geodetic network (without configuration defect), it is recommended that the whole procedure of deformation analysis be separated into two parts:

(1) adjustment of the network for each campaign;

(2) fitting of a deformation model into displacements (quasi-observables) calculated from paired differences in the adjusted coordinates.

The adjustment process provides an opportunity for detecting outliers and systematic errors in the observations, as well as for the evaluation of the quality of the observations. Appendices I and II give a brief review on the detection of outliers and on the assessment of the observations, respectively, using methods developed at UNB [Chen 1983; Chen and Chrzanowski, 1985; Chen et al., 1986].

If subjected to the proper transformation (see section 5), the displacements calculated from the adjusted coordinates give a picture of the deformation pattern and help in the identification (trend analysis) of the deformation model.

For the sake of completeness in the discussion of deformation analysis, some remarks on the adjustment of monitoring networks are given below.

Deformation monitoring networks are mostly free networks, suffering from datum defects. Consider the n-vector of observations l with dispersion measured by $\sigma_0^2 Q$ in a monitoring network such that

$$l + v = A_y y + A_x x , \tag{4.1}$$

where v is the n-vector of residuals; x is the vector of coordinates of surveyed points; y is the vector of nuisance parameters, e.g., the orientation unknown for each round of directions; and A_y, A_x are corresponding configuration matrices. The least-squares criterion leads to the normal equations:

$$\begin{bmatrix} A_y^T Q^{-1} A_y & A_y^T Q^{-1} A_x \\ A_x^T Q^{-1} A_y & A_x^T Q^{-1} A_x \end{bmatrix} \begin{bmatrix} y \\ x \end{bmatrix} = \begin{bmatrix} A_y^T Q^{-1} l \\ A_x^T Q^{-1} l \end{bmatrix} . \tag{4.2}$$

Eliminating vector y, one gets

$$A_x^T [Q^{-1} - Q^{-1} A_y (A_y^T Q^{-1} A_y)^{-1} A_y^T Q^{-1}] A_x \, x =$$
$$= A_y^T [Q^{-1} - Q^{-1} A_y (A_y^T Q^{-1} A_y)^{-1} A_y^T Q^{-1}] l , \tag{4.3}$$

or, more compactly, as

$$N x = w . \qquad (4.3')$$

Due to datum defects in the monitoring network, the coefficient matrix, N, of the normal equations is singular. Therefore, one must define datum equations to solve for x. Let $D^T x = 0$ be the datum equations in which the rank of matrix D is equal to the number of datum defects in the network. Then, the solution of eqn (4.3') becomes

$$\hat{x} = N_D^- w \quad , \quad Q_x = N_D^- \qquad (4.4)$$

with

$$N_D^- = (N + DD^T)^{-1} - H(H^T DD^T H)^{-1} H^T . \qquad (4.5)$$

The matrix H generates the null space of matrix N, i.e., NH=0. For example, for a -triangulation network, matrix H reads as:

$$H^T = \begin{bmatrix} 1 & 0 & 1 & 0 & \ldots & 1 & 0 \\ 0 & 1 & 0 & 1 & \ldots & 0 & 1 \\ -y_1^o & x_1^o & -y_2^o & x_2^o & \ldots & -y_m^o & x_m^o \\ x_1^o & y_1^o & x_2^o & y_2^o & \ldots & x_m^o & y_m^o \end{bmatrix} \qquad (4.6)$$

where x_i^o, y_i^o are the coordinate components of point i with respect to the centroid of the network. For a trilateration network, the last row of H^T in eqn. (4.6) disappears.

In the general case of a three-dimensional network consisting of m points, one expression of matrix H has, for the maximal case of seven datum defects, the structure:

$$H = \begin{bmatrix} 1 & 0 & 0 & 0 & z_1^o & -y_1^o & x_1^o \\ 0 & 1 & 0 & -z_1^o & 0 & x_1^o & y_1^o \\ 0 & 0 & 1 & y_1^o & -x_1^o & 0 & z_1^o \\ . & . & . & . & . & . & . \\ . & . & . & . & . & . & . \\ . & . & . & . & . & . & . \\ 1 & 0 & 0 & 0 & z_m^o & -y_m^o & x_m^o \\ 0 & 1 & 0 & -z_m^o & 0 & x_m^o & y_m^o \\ 0 & 0 & 1 & y_m^o & -x_m^o & 0 & z_m^o \end{bmatrix}_{(3m) \times (7)} \qquad (4.7)$$

where x_i^o, y_i^o, and z_i^o are the approximate coordinates of the points in the directions x, y, and z, respectively, with respect to the centroid of the network. The first three columns of matrix H correspond to the translation of the network in the directions x, y, z; the second three columns take care of the rotation of the network at the centroid about the x, y, z axes, respectively; the last column accounts for the change in scale.

The solution of eqn. (4.3') with respect to the datum $D^T x = 0$ can also be realized through a similarity transformation from any solutions (say x_u) as

$$x = S x_u \quad , \quad Q_x = S Q_{xu} S^T \tag{4.8}$$

with

$$S = (I - H(D^T H)^{-1} D^T) = I - H(H^T W H)^{-1} H^T W , \tag{4.9}$$

where $W = D(D^T D)^{-1} D^T$. The matrix W in eqn. (4.9) can be interpreted as a weight matrix in the definition of the datum. If all the points in the network are of the same importance in defining the datum, then $W = I$ and eqn. (4.8) becomes the inner constraints solution.

If only some points are used to define the datum, then the other points are given zero weight. For more details, refer to Chen [1983].

From the adjustment, the a posteriori variance factor $\hat{\sigma}_0^2$ is calculated from

$$\hat{\sigma}_0^2 = v^T Q^{-1} \hat{v} / df , \tag{4.10}$$

with the degrees of freedom $df = (n - u_y - u_x + d)$, where v is the vector of estimated residuals, u_x, u_y are the numbers of unknown parameters x and y, respectively, and d is the number of datum defects. When there are k a posteriori variance factors σ_i^2 $(i=1,...,k)$ with the degrees of freedom df_i from the adjustment of k epochs of observations, the pooled a posteriori variance factor can be calculated from

$$\hat{\sigma}_0^2 = (\sum_i^k df_i \, \hat{\sigma}_i^2) / (\sum df_i) , \tag{4.11}$$

if the Bartlett test [Bjerhammar, 1973] allows the non-rejection of the null hypothesis H_0 :
$\sigma_1^2 = \sigma_2^2 = ... = \sigma_k^2$.

5. Identification of Deformation Models

Generally, the analysis of deformation surveys consists of three basic processes:
(1)   preliminary identification of the deformation model;
(2)   estimation of the deformation parameters;
(3)   diagnostic checking of the deformation models and the final selection of the "best" model.

Identification procedures are applied to a set of data to indicate the kind of deformation that warrants further investigation. After a tentative formulation of the deformation trend, estimates of deformation parameters or the coefficients of the models are obtained using the least-squares technique. After the parameters have been estimated, diagnostic checks are performed to determine the adequacy of the fitted model or to indicate

potential improvements. Those three processes necessarily overlap and should be performed as an iterative three-step procedure.

When the deformation observations are scattered in time, a simultaneous handling of all the observations in the deformation analysis may be necessary (see section 6). The identification of the deformation model in such cases may be difficult unless the model can be assumed from a priori knowledge of the deformation mechanism. In practice, the observations are usually grouped in distinct epochs of time. Then performing the analysis on pairs of epochs is preferred to the direct simultaneous analysis of all the epochs. The analysis of pairs of epochs has the following advantages:

(1)   Single point movement in a reference network does not usually follow certain time functions and, therefore, the main interest lies in the localization of unstable points between two epochs of time.

(2)   An analyst of deformation measurements is often curious about what happened to the deformable body between the most recent surveying campaign and the previous one.

(3)   Through the analysis of successive pairs of epochs of observations, the deformation trend in the time domain will be recognized.

An important step in the analysis of pairs of epochs of observations is to identify the deformation pattern in the space domain. Moreover, if the deformation is postulated to be of a linear nature in time, then all the observations made at different epochs of time can be reduced to the observed rate of change of the observation.

Here, a method developed at the U.S. Geological Survey [Prescott et al., 1981] should be mentioned. In their method, all the observations of each line in a trilateration network are plotted against time, and then a linear time function (without precluding the possibility of nonlinearity) is fitted to each of the plots. The slope of each fitted straight line is an estimate of the average rate at which the line was changing during the time period covered by the observations. The standard deviation in the rate is also calculated. Then the problem reduces to the estimation of the deformation rate. Therefore, analysis of multi-epoch observations becomes the estimation of the deformation rate. In this case, the main task is again to identify the deformation pattern in space.

As already mentioned, the selection of deformation models may be based on a priori information or on trend analysis from the displacement pattern. If a monitoring network suffers from datum defects, which is usually the case, a method of iterative weighted transformation [Chen, 1983] can be used to yield the "best" picture of the displacement field, as discussed below.

When comparing two campaigns, the vector of displacements and its cofactor matrix

are calculated as

$$d = \hat{x}_2 - \hat{x}_1 \quad , \quad Q_d = Q_{\hat{x}1} + Q_{\hat{x}2} \qquad \text{with } \hat{\sigma}_0^2 \text{ from eqn. (4.11).} \qquad (5.1)$$

Because unstable points are not identified, the displacements calculated from eqn. (5.1) may be biased by a pre-selected datum or by a different datum definition in the adjustment of two campaign observations. A typical example for the latter case is the monitoring scheme in which a triangulation network was used in the first campaign and a trilateration or triangulateration network in the second campaign.

To overcome this problem, a method of iterative weighted transformation has been developed. Let d_1 and Q_{d1} be calculated from eqn. (5.1). The transformation of d_1 into another datum is computed from eqns. (4.8) and (4.9) as

$$d_{k+1} = (I - H(H^TWH)^{-1} H^TW) d_k = S_k d_k . \qquad (5.2)$$

At the outset, the weight matrix W is taken as the identity, then in the $(k+1)$th transformation, the weight matrix is defined as

$$W = \text{diag}\{1/|d_i(k)|\} , \qquad (5.3)$$

where $d_i(k)$ is the ith component of the vector d_k after the kth iteration. The iterative procedure continues until the differences between the successive transformed displacements (i.e., $d_{k+1} - d_k$) approach zero. During this procedure, some $d_i(k)$ may approach zero causing numerical instabilities because $W_i = 1/|d_i(k)|$ becomes very large. Thus, a lower bound is set. When $|d_i(k)|$ is smaller than the lower bound, its weight is set to zero. If in the following iterations the $d_i(k+1)$ becomes significantly large again, the weights can be changed accordingly. The method provides a datum which is robust to unstable reference points giving an unbiased depiction of displacements. In the last iteration, say $(k+1)^{th}$, the cofactor matrix should also be calculated as:

$$Q_{dk+1} = S_k Q_{d1} S_k^T . \qquad (5.4)$$

Comparing the displacements of each point against its confidence ellipse, one can identify the reference points which are most probably unstable. The following are two examples used to illustrate the method.

The first example is a simulated relative geodetic network across a fault line (Figure. 5.1), where only directions were measured in the first campaign and both directions and distances were measured in the second. A 200 mm relative movement in the y direction of block B with respect to Block A was introduced. Without considering measuring errors, the displacement fields using the proposed method and the method of the inner constraints solution are portrayed in Figures. 5.2 and 5.3, respectively, coupled with the real displacement pattern in dashed lines. As one can see, the displacement field obtained from the method of iterative weighted transformation is much closer to the real situation, compared with the constraint solution.
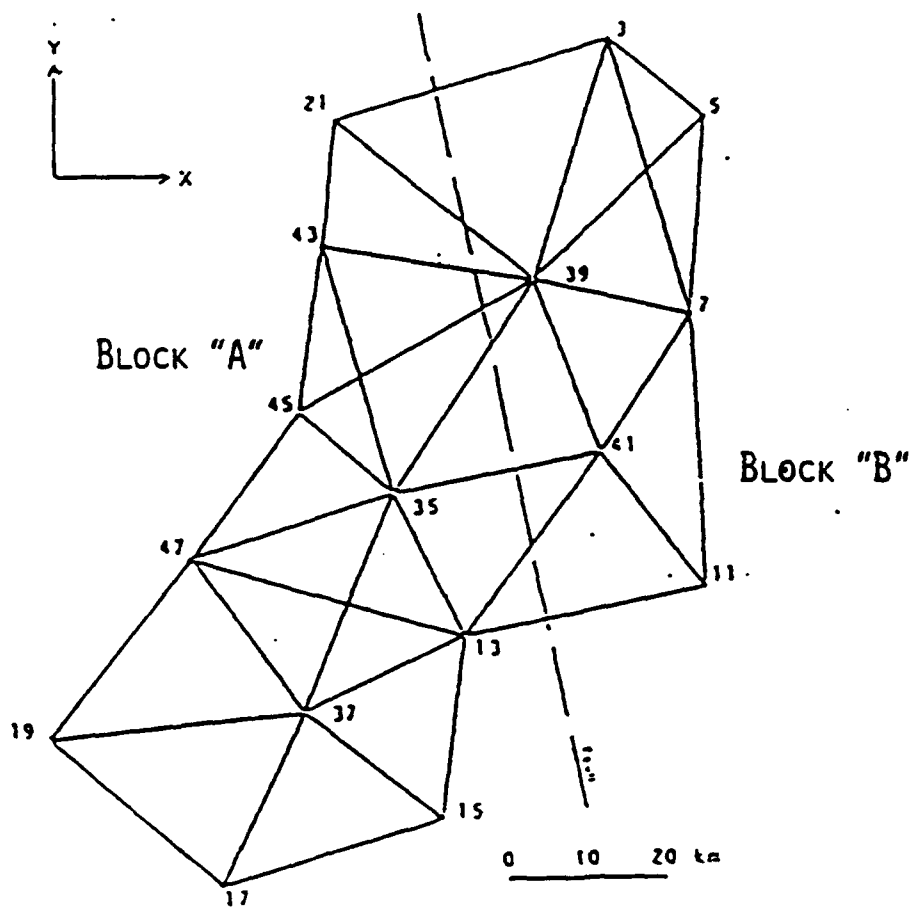
Figure 5.1 Typical campaign of the simulated relative geodetic network.
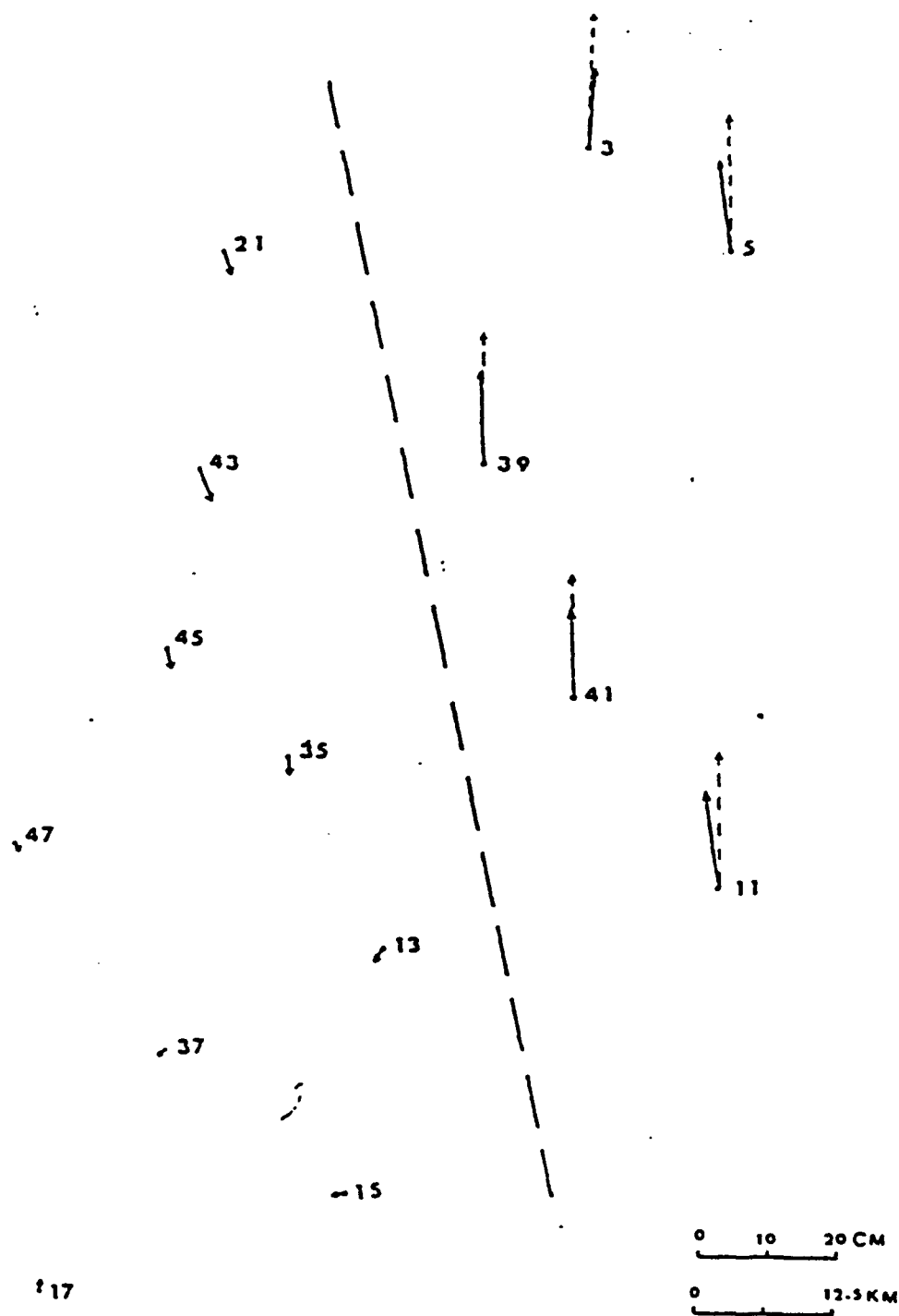
Figure 5.2 Simulated relative geodetic network—Displacement field after the iterative weighted transformation (solid) versus the actual displacements (broken).
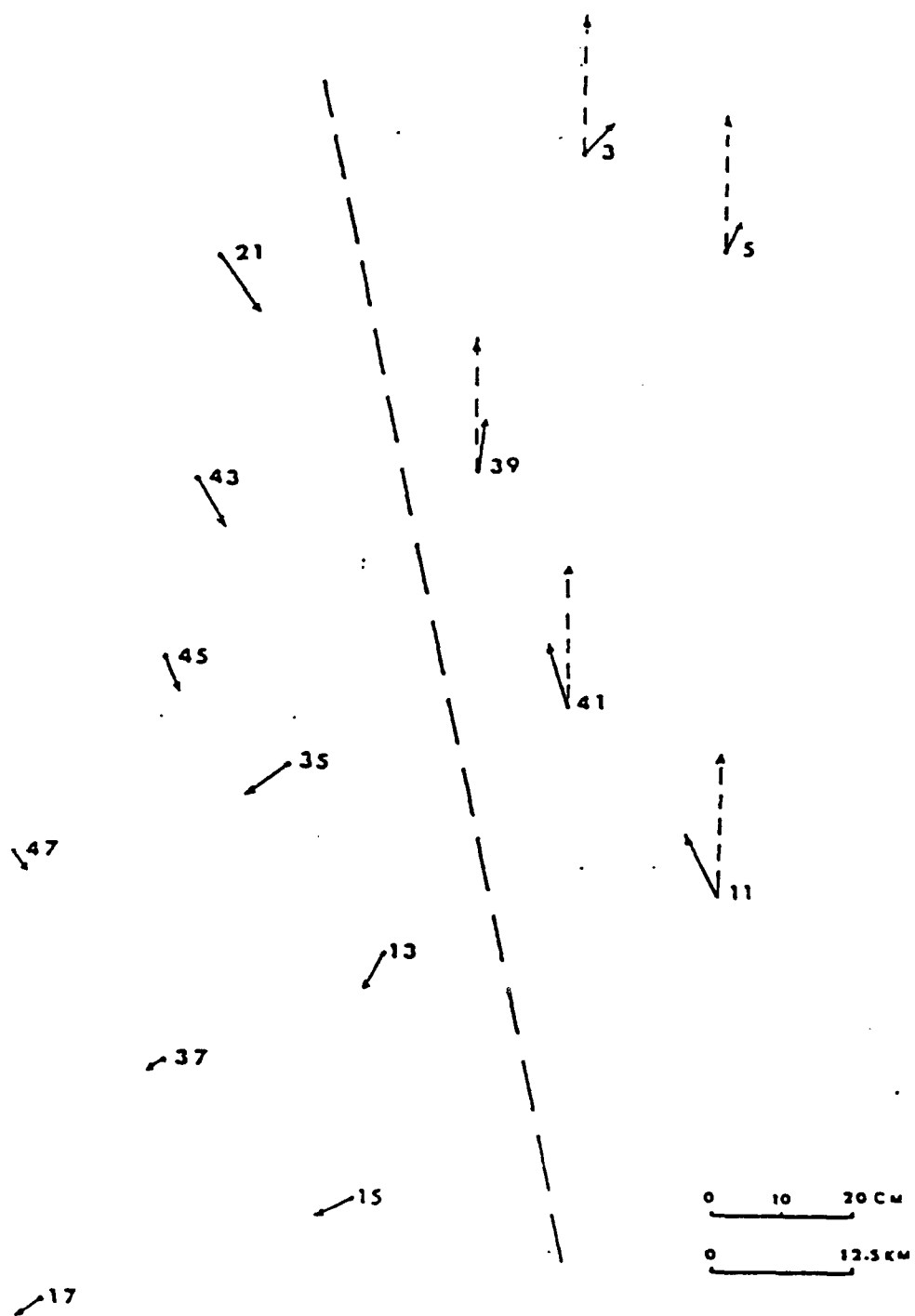
Figure 5.3 Simulated relative geodetic network—Displacement field under inner
constraints (solid) versus the actual displacements (broken)

In the second example, the method is applied to a dam monitoring network consisting of a reference network of six stations from which a number of targetted points on the dam were positioned. The same network in actual conditions is shown in the example of section 10 (Figure 10.1) A simulated movement in steps of 1 mm was introduced to station 5 in the y direction between the successive survey campaigns (total of 11 campaigns). The displacement field coupled with the error ellipses at 95% confidence level were calculated using the method of iterative weighted transformation and the method of the inner constraints solution. Table 1 summmparizes the results of the identification of the points suspected as being unstable because their displacements extended beyond the confidence region at 95%.

TABLE 1.

Points outside the 95% confidence region after introducing simulated displacements to station 5.

Suspected Unstable point*	Accumulated simulated displacement of station 5 in mm									
	1	2	3	4	5	6	7	8	9	10
1							0	0	0	0
2										0
3					0	0	0	0	0	0
4				0	0	0	0	0	0	0
5	XO	XO	XO	XO	XO	XO	XO	XO	XO	XO
6									0	0

* X: using the method of iterative weighted transformation.

  0: using the method of the inner constraints solution.

It is clear from Table 1 that the method of iterative weighted transformation identified the unstable point correctly, while the method of inner constraints solution declared more suspected unstable points.

The method of the weighted transformation is flexible. If some points are more likely to move, a weight of zero is assigned to each of these points during the iterative process. For example, if the points on one side of a tectonic fault may likely move with

respect to the points on the other side, then only the points on the one side are used to define a datum.

6. Estimation of Deformation Models

Let y_i (i=1,2,..., k) be the vector of observations in epoch i, including quasi-observations (e.g., the coordinates of points from an adjustment of geodetic network of photogrammetric surveys), geotechnical measurements (using strainmeters, extensometers, tiltmeters, etc.), and individual geodetic observations, and P_i be the weight matrix of y_i. The weight matrix for the coordinates of points estimated from eqn. (3.3') is taken as N, and for the other observed quantities it is taken in the conventional way as the inverse of the cofactor matrix. Because of datum defects and possible configuration defects in a monitoring network, the weight matrix P_i is, in general, considered as being singular [Chrzanowski et al., 1983]. Determination of the coefficients of a deformation model, $d(x, y, z; t-t_1) = B(x, y, z; t-t_1)c$, is based on the following functional relations:

$$\begin{bmatrix} y_1 \\ y_2 \\ . \\ . \\ . \\ y_k \end{bmatrix} + \begin{bmatrix} \delta_1 \\ \delta_2 \\ . \\ . \\ . \\ \delta_k \end{bmatrix} = \begin{bmatrix} I & 0 \\ I & \widetilde{B}_2 \\ . & . \\ . & . \\ . & . \\ I & \widetilde{B}_k \end{bmatrix} \begin{bmatrix} \xi \\ c \end{bmatrix} \qquad (6.1)$$

with weight matrix $P = \text{diag}\{P_1, P_2, ..., P_k\}$, ξ is the expected value of y_1, and δ_i is a vector of residuals after fitting the deformation model to the y_i; matrix $\widetilde{B}_i$ is a function of the position of points and time. If y_i is the vector of coordinates, then $\widetilde{B}_i = B_i$. If y_i is the vector of observations rather than of the coordinates of points, $\widetilde{B}_i = AB_i$, where matrix A is the transformation matrix (or configuration matrix) relating the observations to the coordinates. In order to keep the same population of vector y_i in each epoch, dummy observations with zero weight are put in the place of observations missing in campaign i in the vector y_i. Applying the principle of least squares to model (6.1), the normal equations read:

$$
\begin{bmatrix} \sum\limits_{1}^{k} P_i & \sum\limits_{2}^{k} P_i \tilde{B}_i \\ \\ \sum\limits_{2}^{k} \tilde{B}_i^T P_i & \sum\limits_{2}^{k} \tilde{B}_i^T P_i \tilde{B}_i \end{bmatrix} \begin{bmatrix} \xi \\ \\ c \end{bmatrix} = \begin{bmatrix} \sum\limits_{1}^{k} P_i y_i \\ \\ \sum\limits_{2}^{k} \tilde{B}_i^T P_i y_i \end{bmatrix}
\tag{6.2}
$$

The coefficient matrix of the normal eqns. (6.2) may be singular with rank defects

$$
rd \left\{ \sum_{1}^{k} P_i \right\} = d
\tag{6.3}
$$

which is equal to the number of remaining datum and configuration defects not determined in at least one epoch.

Eliminating ξ from eqn. (6.2) allows the vector c and its accuracy to be calculated from:

$$
\hat{c} = N_c^{-1} [\sum_{2}^{k} \tilde{B}_i^T P_i y_i - \sum_{2}^{k} \tilde{B}_i^T P_i (\sum_{1}^{k} P_i)^- \sum_{1}^{k} P_i y_i]
\tag{6.4}
$$

and

$$
\hat{C}_c = \sigma_o^2 N_c^{-1} = \sigma_o^2 [\sum_{2}^{k} \tilde{B}_i^T P_i \tilde{B}_i - \sum_{2}^{k} \tilde{B}_i^T P_i (\sum_{1}^{k} P_i)^- \sum_{2}^{k} P_i \tilde{B}_i]^{-1}
\tag{6.5}
$$

To serve as the a priori variance factor σ_o^2, the pooled variance (a posteriori to the campaign adjustments) factor obtained from eqn. (4.11) is used.

As was already mentioned, an indispensible step in the deformation analysis is the analysis of pairs of epochs of observations. In this special case, for each pair of epochs. eqn. (6.4) reduces to:

$$
\hat{c} = (\tilde{B}^T P_{\Delta y} \tilde{B})^{-1} \tilde{B}^T P_{\Delta y} \Delta y ,
\tag{6.6}
$$

in which

$$
\Delta y = y_2 - y_1
$$
$$
P_{\Delta y} = P_2 - P_2 (P_1 + P_2)^- P_2 .
$$

If y_i stands for observations l_i, solution (6.6) becomes:

$$
\hat{c} = [B^T A^T P_{\Delta l} A B]^{-1} B^T A^T P_{\Delta l} (l_2 - l_1)
\tag{6.7}
$$

with

$$P_{\Delta l} = (Q_1 + Q_2)^{-1} .$$

This is called the "observation approach." If y_i stands for the estimated coordinates x_i from eqn. (4.3'), then

$$\hat{c} = (B^T P_d B)^{-1} B^T P_d \; d , \qquad (6.8)$$

with $d = (x_2 - x_1)$ and $P_d = N_1 (N_1 + N_2)^- N_2$. Then it is named the "displacement approach."

7. Assessment of the Deformation Models

7.1 General Remarks on Statistical Testing

Analysis of deformation surveys involves several tests of hypotheses. Consider the observation equations:

$$l = Ax + v , \qquad (7.1)$$

where l is a vector of n observations with normal distribution and dispersion $\sigma_0^2 Q$, x is a vector of u unknown parameters, A is the design matrix (or configuration matrix), v is the vector of residuals, σ_0^2 is the a priori variance factor, and Q is a cofactor matrix so that the observations have a weight matrix $P = Q^{-1}$. If the null hypothesis $H_0 : Hx = w$ is to be tested against an alternative $H_a : Hx \neq w$, then the test statistic can be obtained by imposing the constraints $Hx = w$ on the parameters x. Under the null hypothesis, model (7.1) becomes:

$$\begin{cases} l = Ax + v \quad \text{with } \sigma_0^2 Q \\ \\ Hx = w \end{cases} \qquad (7.2)$$

The acceptance of the null hypothesis at a certain significant level α is ensured by the inequality:

$$T = [((R_1 - R_0) / R_0)(df/(df_1 - df))] \leq F(\alpha; df_1 - df, df) , \qquad (7.3)$$

or, equivalently,

$$T' = [(R_1 - R_0)/R_1)(df_1/(df_1 - df)] \leq$$
$$\leq (df_1 \, F(\alpha; df_1 - df, df))/(df + (df_1 - df) \, F(\alpha; df_1 - df, df)) , \qquad (7.4)$$

where R_0 and R_1 are the quadratic forms of the residuals from the adjustment of model (7.1) and model (7.2), respectively. The corresponding degrees of freedom are:

$$df = n - \text{rank}\{A\} ,$$

and

$$df_1 = n - \text{rank}\{A^T : H^T\} + \text{rank}\{H^T\} .$$

In the practice of hypothesis testing, $(R_1 - R_0)$ and R_0 in the statistics (7.3) and (7.4) can be calculated, depending on the problem at hand, in three different ways [Chen, 1983]:

(i)   from separate adjustments of model (7.1) and model (7.2),

(ii)   from the adjustment of model (7.1),

$$R_1 - R_0 = (Hx - w)^T (H(A^T Q^{-1} A)^{-1} H^T)^{-1} (Hx - w) \qquad (7.5)$$

(iii)   from the adjustment of model (7.2)

$$R_1 - R_0 = v^T Q^{-1} A_2 (A_2^T Q^{-1} Q_v Q^{-1} A_2)^{-} A_2^T Q^{-1} v , \qquad (7.6)$$

where v and Q_v are the vector of residuals and its cofactor matrix, respectively, and matrix A_2 generates the space whose union with the solution space of model (7.2) equals the solution space of model (7.1).

7.2 Assessment and Final Selection of the Deformation Model

The global appropriateness of a deformation model can be tested using a quadratic function of the residuals δ_i in eqn. (6.1) as

$$\Delta R = \sum_1^k \delta_i^T P_i \delta_i , \qquad (7.7)$$

where the notation has been defined in section 6. The quantity ΔR follows a chi-squared distribution with degrees of freedom being

$$df_c = \sum_1^k r\{P_i\} - u + d \qquad (7.8)$$

where u is the dimension of the vector of unknowns $(\xi^T : c^T)$ of eqn. (6.1), $r\{P_i\}$ is the rank of matrix P_i, and d has been defined in eqn. (6.3). If the following inequalities hold:

$$\Delta R \le \sigma_0^2 \chi^2(df_c; \alpha) , \qquad (7.9)$$

when the a priori variance factor is known, or

$$\Delta R \le \hat{\sigma}_0^2 df_c \ F(df_c, df; \alpha) , \qquad (7.10)$$

when the pooled variance factor is used and $df = f_1 + ... + f_k$, as defined in eqn. (4.11), then the deformation model is globally acceptable at the $(1-\alpha)$ confidence level. When $c=0$, the test statistic (7.9) or (7.10) can be regarded as an extension of the global congruency test, which originated from Pelzer [1971].

The significance of the individual parameter $\hat{c}_i$ or a group of u_i parameters, $\hat{c}_i$ which is a subset of $\hat{c}$, is revealed by testing the null hypothesis $H_0 : c_i = 0$ or $c_i = 0$ versus the alternative hypothesis $H_a : c_i \ne 0$ or $c_i \ne 0$. Their significances are indicated by

$$\hat{c}_i^2 / (\sigma_0^2 q_{ii}) \ge F(1, df; \alpha) \qquad (7.11)$$

and

$$\hat{c}_i^T Q_{ci}^{-1} \hat{c}_i / (\sigma_o^2 u_i) \geq F(u_i, df; \alpha) \tag{7.12}$$

where q_{ii} is the ith diagonal element of $Q_{\hat{c}}$ and Q_{ci} is a submatrix of Q_c. If the global test fails, localization in time domain or in space domain should be performed. Displaying the residuals will help in improving the model.

Since more than one of several possible models could fit the data reasonably well, the authors have set the following criteria for selection of the "best model":

(1)   the model passes the global statistical test and all parameters are significant beyond some level of α as 0.10 or 0.05,

(2)   if more than one model satisfies the above criteria, then the model with the fewest parameters is selected.

(3)   if no model satisfies the criteria of (1), then physically-based rationale and minimum error of fit are used.

8.   Summary of the Generalized Method of Geometrical Analysis

The presented approach to the geometrical analysis of deformation surveys has been named by the authors the *UNB Generalized Method.*

As shown in the previous sections, the method is applicable to any type of geometrical analysis, both in space and in time, including the detection of an unstable area and the determination of strain components and relative rigid body motion within a deformed object. It allows utilization of different types of surveying data and geotechnical measurements. In practical application, the approach consists of three basic processes: identification of deformation models; estimation of the deformation parameters; diagnostic checking of the models and the final selection of the "best" model.

The analysis procedures using the approach can be summarized in the following steps:

(1)   Assessment of the observations using the minimum norm quadratic unbiased estimation (MINQE) principle (Appendix I) to obtain the variances of observations and possible correlations of the observations within one epoch or between epochs, if the a priori values are not available.

(2)   Separate adjustment of each epoch of geodetic or photogrammetric observations, if such are available, for detection of outliers (Appendix II) and systematic errors. If correlations of the observations between epochs are not negligible, then simultaneous adjustment of multiple epochs of observations is required.

Step 1 and 2 overlap because the existence of outliers and systematic errors will influence

the estimated variances and covariances and adopted variances and covariances of the observations will affect outlier detection.

(3)   Comparison of pairs of epochs; selection of deformation models based on a priori considerations and trend analysis from the displacement pattern if such is available from the observations. If a monitoring network suffers from datum defects, the method of *iterative weighted transformation* is used to yield the "best" picture of the displacement pattern.

(4)   Estimation of the coefficients of deformation models and their covariance using all available information.

(5)   Global test on the deformation model; testing groups of the coefficients or an individual one for significance.

The above three steps should be considered as an iterative three-step procedure, so they necessarily overlap.

(6)   Simultaneous estimation of the coefficients of the deformation model in space and in time if the analysis of pairs of epochs of observations suggests that it is worth doing.

This simultaneous estimation must be performed if the observations are scattered in time. The iterative three-step procedure is still valid. The possible deformation models can be selected either based on a priori considerations or by plotting the observations versus time for trend analysis.

(7)   Comparison of the models and choice of the "best" model. Since more than one of several possible models could fit the data reasonably well, the "best" model is selected according to the criteria:

(a)   the model passes the global statistical test at an acceptable probability;

(b)   if more than one model passes the global test, then the model with the fewest significant coefficients is selected.

(c)   if the two above criteria cannot be satisfied, then rationale based on physical ground and minimal error of fit is used.

(8)   Calculation of the desired deformation characteristics and their accuracies from the parameters of the "best" model.

(9)   Graphical display of the deformation model.

A detailed description of the above steps with practical examples can be found in Secord [1984]. Some applications will be given in case studies presented at this workshop. The reader is also referred to Chen [1983], Chrzanowski et al. [1983; 1985], and Chrzanowski and Secord [1983; 1985].

9. Computer Program Cluster "UNB DEFNAN"

The UNB generalized method has been implemented through software developed in FORTRAN 77 on an IBM 3090 mainframe and on an IBM PC/AT. The cluster of programs has the same behaviour in either system with the only variations being in file management and in graphical display. Hence, the following description of the modules is applicable to either system.

The cluster is most easily described with reference to Figure 9.1 which shows the arrangement of modules.

Any campaign of measurement, l_i, however populated, is utilized. If the configuration of measured relationships is complete, then an adjustment is performed resulting in least-squares estimates, $\hat{x}_i$, under explicit minimal constraints in 1, 2, or 3 dimensions. The intention behind "UNB DEFNAN" has been to remain flexible enough to accept the estimated coordinates and their variance-covariance matrix from any style of adjustment program which is then received by module CORDIF.

Following from at least two campaign adjustments are the analysis of trend and the modelling using coordinate differences, dx_s, in program module CORDIF within which there are several submodules. The first, CORDIFD, initiates the comparison of a pair of campaigns by creating a set of displacements versus minimal explicit constraints. It is this upon which the datum independent weighted transformation and modelling are based. Also, CORDIFD allows the segregation of stations common to the two campaigns or of only those stations of interest, e.g., only the reference network stations. The displacements can be readily depicted against their respective ellipses at any desired α level through graphics packages on either system. The weighted transformation is performed by submodule CORDIFW producing a datum independent indication of trend which may be visualized as displacement vectors with ellipses at any specified α level. Any collection or arrangement of stations can be considered in the modelling which is done in a very flexible submodule CORDIFM. Any model can be accommodated, provided that the functional relationship has been coded. Full statistical testing of the model and its constituents and any desired characteristics may be derived and their significance levels determined. One example is linear homogeneous strain for which the basic parameters are ϵ_x, ϵ_y, ϵ_{xy}, ω with a possible a_0 and b_0. From this, the maximal and minimal strains, ϵ_{max} and ϵ_{min} and their orientation plus the vector of relative rigid body movement, d with azimuth A_d, would be derived accompanied by their standard deviations and $(1-\alpha)$ levels.

Circumvention of a campaign adjustment, especially when not allowed by the lack of a substantial configuration, requires considering the observations themselves. This may be

$l_i . P_{li} \longrightarrow$ [1-, 2-, or 3- dimensional adjustment]

$$\hat{x}_i . C_{xi}$$

CORDIF $\longrightarrow$ CORDIFD $\quad dx_s - x_k - x_j . C_{dxs}$

CORDIFW $\quad dx_v . C_{dxv}$

CORDIFM $\quad dx + v - Bc$
$\quad\quad\quad\quad\quad\quad\quad \hat{c} . C_c$

OBDIF $\longrightarrow$ OBDIFD $\quad dl - l_k - l_j : \quad dl + v - Adx$
$\quad\quad\quad\quad\quad\quad\quad\quad d\hat{x} . C_{dx}$

OBDIFW $\quad dx_v . C_{dxv}$

OBDIFM $\quad dl + v - ABc$
$\quad\quad\quad\quad\quad\quad\quad \hat{c} . C_c$

SIMSOL

Figure 9.1 Program cluster "UNB DEFNAN."

readily done through module OBDIF which treats observation differences, dl, in much the same way as the dx were treated in CORDIF with the additional flexibility of being capable of dealing with observables other than the customary geodetic angular and linear measurements. Displacements, dx, can be estimated from the dl in submodule OBDIFD with results similar to those of CORDIFD, even with configuration and multiple datum defects associated with totally isolated but repeated observations. The weighted transformation, dx_w, can be obtained through submodule OBDIFW for an indication of trend. Similarly as in CORDIFM, modelling may be done using the dl in submodule OBDIFM.

If the trend as indicated through the campaign comparisons would indicate the feasibility of a model considering all or many campaigns simultaneously, then this may be accomplished through module SIMSOL. This simultaneous solution can accommodate as many campaigns as desired with as few as one observation in a campaign. Rates, acceleration, and higher-order parameters may be estimated with full statistical assessment and the analysis of the observations. :

Altogether, the program cluster "UNB DEFNAN" provides a flexible and versatile deformation analysis package which can also be utilized in the preanalysis and design of deformation monitoring schemes.

10. An Example of a Reference Geodetic Network

A pure triangulation network of 6 concrete pillar reference stations and 10 uniquely intersected dam crest points (Figure 10.1) was observed twice with 47 directions first and 53 directions in the second campaign. Least-squares estimations of the coordinates, $\hat{x}_1$, $\hat{x}_2$, were made under explicit minimal constraints involving stations 5 and 6 (considered as "fixed" and errorless) using UNB program GEOPAN (GEOdetic Plane adjustment and ANalysis). No observation in either campaign was detected as being an outlier under the τ max criterion at 0.95. The pooled variance factor, $\sigma_0^2 = 0.95278$, had df = 31 degrees of freedom. With each campaign having the same stations and datum, the observed displacement components d were obtained through the simple differencing of coordinates, through module CORDIFD, as

$$d = \hat{x}_2 - \hat{x}_1 , \tag{10.1}$$

with cofactor matrix

$$Q_d = Q_1 + Q_2 . \tag{10.2}$$

Within the d and Q_d are zero elements corresponding to the coordinates of the constraining stations 5 and 6.
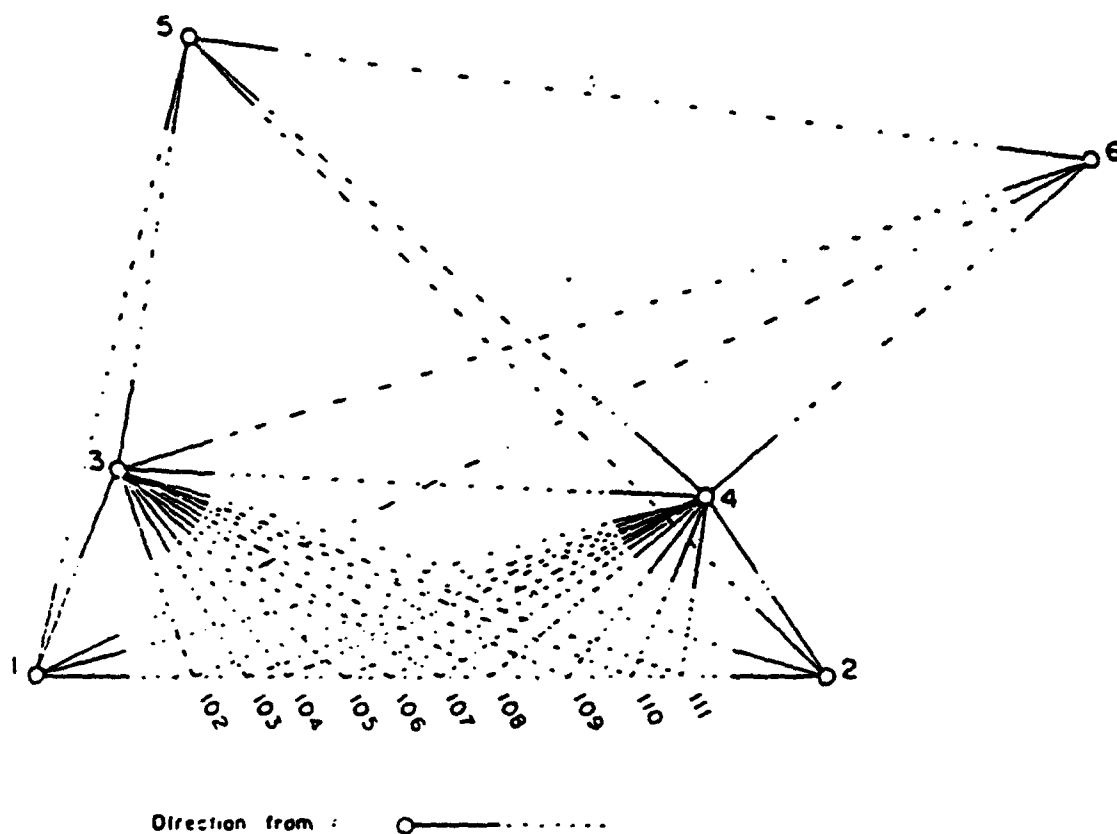
Figure 10.1 Reference geodetic network.
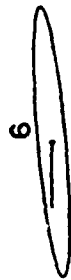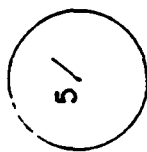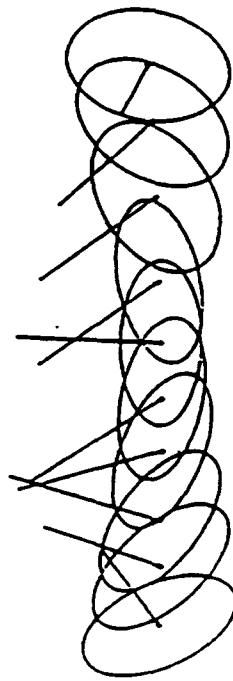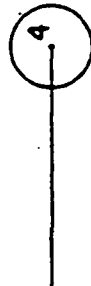
Figure 10.2 Displacements and confidence regions at α = 0.05 after weighted transformation of station displacements.
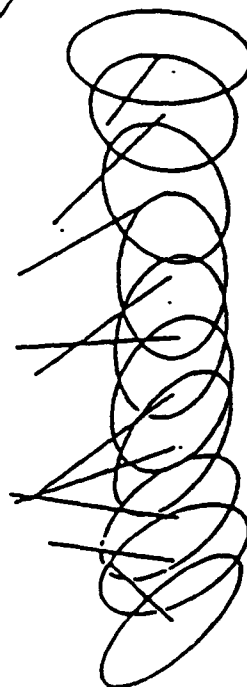
Figure 10.3 Displacements and confidence regions at α = 0.5 derived from model of single point movement.

After the converged iteration of the weighted transformation of d from eqn. (10.1), the unique, datum independent displacement pattern produced by module CORDIFW using eqn. (5.2) is shown in Figure 10.2. Obviously, reference station 4 has moved significantly while the other reference stations remain stable at 0.95.

Having an a_i and b_i for each object point and also for station 4, and modelling the block of points 1, 2, 3, 5, and 6 as stable, the deformation model consisted of 22 parameters (i.e., 11 pairs of displacement components) which were estimated using module CORDIFM and eqn. (6.6). A plot of these displacements and their associated confidence regions at 0.95 is given in Figure 10.3. With 6 degrees of freedom in the modelling, the global test on the adequacy of the model was not rejected at 0.95 since:

$$T^2 = (\hat{\sigma}_{oc}^2) / (\sigma_o^2) = 1.7628 < F(6, 31; 0.05) = 2.41 \ .$$

Thus, with the a priori knowledge of the intention of the network serving as a reference, this model was adopted as appropriate.

11. Acknowledgements

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APPENDIX I
DETECTION OF OUTLIERS

There are two concepts of outlying observations in statistics. One is the "mean shift model," where an outlier has the distribution of $N(\mu + \lambda, \sigma^2)$ instead of $N(\mu, \sigma^2)$, and the other is the "variance-inflation model" in which an outlying observation is distributed as $N(\mu, a^2\sigma^2)$, $a^2 > 1$, i.e., its variance is larger than expected. Different strategies for reweighting observations in a least-squares adjustment for detection of outliers are based on the latter concept. In these notes, only the "mean shift model" will be discussed.

Consider the Gauss-Markoff model $(l, Ax, \sigma^2 Q)$. Let an n vector of observations l be partitioned into two groups: l_1 and l_2 with l_1 being of dimension n_1 and free of outliers, and l_2 being of dimension n_2 and containing suspected outliers, denoted by δ. The mean shift model reads:

$$\begin{bmatrix} l_1 \\ l_2 \end{bmatrix} + \begin{bmatrix} v_1 \\ v_2 \end{bmatrix} = \begin{bmatrix} A_1 & 0 \\ A_2 & I \end{bmatrix} \begin{bmatrix} x \\ \delta \end{bmatrix}, \tag{I.1}$$

where v_i, A_i (i=1,2) are corresponding vectors of residuals and configuration matrices, respectively. More compactly, eqn. (I.1) is written as:

$$l + v = Ax + E\delta. \tag{I.1'}$$

If the observations are uncorrelated, i.e., matrix Q is diagonal, model (1) is equivalent to the observation equations after outlying observations l_2 are removed. The statistical tests on outliers are to confirm, at a certain confidence level $(1-\alpha)$, the null hypothesis $H_0 : \delta = 0$ versus an alternative one $H_a : \delta \neq 0$. In the practice of outlier detection, an adjustment is performed with the original Gauss-Markoff model $(l, Ax, \sigma_0^2 Q)$. The least-squares estimation of the residuals and their cofactor matrix is

$$\hat{v} = [A(A^T Q^{-1} A)^{-1} A^T Q^{-1} - I] \, l = M \, l, \tag{I.2}$$

and

$$Q_{\hat{v}} = Q - A(A^T Q^{-1} A)^{-1} A^T, \tag{I.3}$$

respectively. The quadratic form of the residuals

$$R_1 = \hat{v}^T Q^{-1} \hat{v} \tag{I.4}$$

follows a $\sigma_0^2 \chi^2$ distribution with degrees of freedom df = n-rank(A). Introducing vector δ in model (I.1') will result in a reduction in the quadratic form of the residuals

$$\Delta R = R_1 - R_0 - \hat{v}^T Q^{-1} A_2 (A_2^T Q^{-1} Q_{\hat{v}} Q^{-1} A_2)^{-1} A_2^T Q^{-1} \hat{v}, \tag{I.5}$$

which will be non-centrally $\sigma_0^2 \chi^2$ distributed with degrees of freedom n_2 if the null hypothesis is to be rejected. The quantity ΔR is statistically independent of $(R_1 - \Delta R)$. If there are no other suspected outliers in the observations, then $(R_1 - \Delta R)$ will follow a central $\sigma_0^2 \chi^2$ distribution with degrees of freedom being $(df - n_2)$. Confirmation of the suspected outliers is made:

(i) $\quad T_1 = \Delta R / n_2 \sigma_0^2 \geq F(\alpha; n_2, \infty)$, $\tag{I.6}$

if the a priori variance factor σ_0^2 is available;

(ii) $\quad T_2 = \Delta R / n_2 \tilde{\sigma}_0^2 \geq F(\alpha; n_2, df - n_2)$, $\tag{I.7}$

if the a posteriori variance factor σ_0^2 is used and estimated from

$$\tilde{\sigma}_0^2 = (R - \Delta R) / (df - n_2); \tag{I.8}$$

(iii) $T_3 = \Delta R / n_2 \hat{\sigma}_0^2 \geq (df\, F(\alpha; n_2, df - n_2)) / ((df - n_2) + F(\alpha; n_2, df - n_2))$, $\tag{I.9}$

if the a posteriori variance factor is computed from $\hat{\sigma}_0^2 = R / df$.

It is important to point out that the conclusions about outliers using the tests (I.7) and (I.9) are identical, because expression (I.9) can be derived directly from expression (I.7).

As a special case, if only one outlier is suspected, say, the ith observation, matrix A_2 in eqn. (I.5) is replaced by vector e_i, which is an n-vector with a unit value in the ith position and zeros elsewhere. Then eqn. (I.5) is reduced to

$$\Delta R_i = (e_i^T Q^{-1} \hat{v})^2 / e_i^T Q^{-1} Q_{\hat{v}} Q^{-1} e_i. \tag{I.10}$$

In addition, if the observations are statistically independent, i.e., matrix Q is diagonal, then the expression ΔR_i is further simplified as

$$\Delta R_i = \hat{v}_i^2 / q_{v_i}, \tag{I.11}$$

where q_{v_i} is the ith diagonal element of Q_v, and v_i is the ith component of v. In the case of one suspected outlier, the statistical tests (I.6), (I.7), and (I.9) become the w-test [Kok, 1984; Baarda, 1968]:

$$w_i = \sqrt{\Delta R_i / \sigma_0^2} \geq \sqrt{F(\alpha; 1, \infty)} = n(\alpha/2) , \tag{I.12}$$

the t-test [Heck, 1981]

$$t_i = \sqrt{\Delta R_i / (\tilde{\sigma}_0^2)_i} \geq \sqrt{F(\alpha; 1, df-1)} = t(\alpha/2; df-1) , \tag{I.13}$$

and the τ-test [Pope, 1976]

$$\tau_i = \sqrt{\Delta R_i / \hat{\sigma}_0^2} \geq \sqrt{(df\, F(\alpha; 1, df-1)) / ((df-1) + F(\alpha; 1, df-1))} = \tau(\alpha/2; df) , \tag{I.14}$$

respectively. $(\tilde{\sigma}_0^2)_i$ in eqn. (I.13) is calculated from

$$(\tilde{\sigma}_0^2)_i = (\hat{v} Q^{-1} \hat{v} - \Delta R_i) / (df-1) . \tag{I.15}$$

The τ-distribution is not very popular in statistics and no tabulated critical values are available, but they can easily be calculated by comparing the expressions (I.13) and (I.14)

as
$$\tau(\alpha/2; df) = \sqrt{df} \cdot t(\alpha/2); df-1) / \sqrt{(df-1) + t^2(\alpha/2; df-1)} . \tag{I.16}$$

Since the statistical tests (I.7) and I.9) are equivalent, so are the τ-test and the t-test.

The difficulty in the detection of outliers lies in the localization of outlying observations, especially when multiple outliers are present. An efficient strategy has been developed, and the interested readers are referred to Chen et al. [1986].

APPENDIX II
ASSESSMENT OF OBSERVATIONS

In the above discussions, the variance-covariance matrix of the observations or the weight relationship among the observations is assumed to be known. This, however, may not be the case in many practices, especially in heterogeneous networks. Assessment of the observations may have to be performed. The technique of MINQE (minimum norm quadratic estimation) provides a tool to estimate variance-covariance components.

Consider the linear Gauss-Markoff model (l, Ax, C). Assume that the variance-covariance matrix, C, can be decomposed as

$$C = \sum_1^k \theta_i T_i , \qquad (II.1)$$

where T_i are known matrices, θ_i are variance-covariance components to be estimated. Applying the MINQE principle, the $\theta = (\theta_1, \theta_2, ..., \theta_k)^T$ can be estimated from

$$\theta = S^{-1} q, \qquad (II.2)$$

where the (i,j)th element of matrix S is

$$s_{ij} = tr\{R \ T_i \ R \ T_j\} , \qquad (II.3a)$$

and the ith component of vector q is

$$q_i = l^T R \ T_i R \ l , \qquad (II.3b)$$

and

$$R = C^{-1}[I - A(A^T C^{-1} A)^- A^T C^{-1}] . \qquad (II.3c)$$

Since C is unknown, an iterative computation procedure has to be conducted. Let $\theta_i^{(o)}$, $\forall i$ be the a priori value of θ_i, then C in the above formulae is replaced by

$$C(\theta_i) = \sum_1^k \theta_i^{(o)} T_i .$$

From eqn. (II.2), θ is estimated and used as a priori values in the second iteration. If the process continues and the solution for θ converges, the final estimation of θ is independent of the selection of a priori values $\theta_i^{(o)}$, $\forall i$. This technique is called the iterated MINQE. For a detailed theoretical discussion and application, readers are referred to Chen [1983] and Chen and Chrzanowski [1985].

**STANDARDS AND SPECIFICATION FOR DEFORMATION SURVEYS
PREPARED FOR ALBERTA ENIRONMENT, CANADA**

DEFORMATION SURVEYS

STANDARDS AND SPECIFICATIONS FOR DEFORMATION SURVEYS

Prepared for

Alberta Environment

Edmonton, Alberta

by

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March, 1988

Preface

The research work upon which this document is based was made possible by a research grant from the Research Management Division, Alberta Environment to Dr. W.F. Teskey, P.Eng., Department of Surveying Engineering, The University of Calgary.

Users of this document will be the Survey Branch and Dam Safety Branch of Alberta Environment. It is also anticipated that this document will be used by private survey firms contracted by Alberta Environment to carry out specific deformation survey projects. Use of this document will ensure a uniform high quality of results for deformation survey projects regardless of whether Alberta Environment or a private survey firm performs the work.

The standards and specifications presented in this document cover deformation surveys carried out by conventional survey methods, photogrammetric survey methods and GPS (Global Positioning System) survey methods. Since it is possible that two or even three of these methods might be used together on a particular project, the standards are based upon the magnitude of a detectable single point movement and thus are independent of the method applied. For the same reason, the specifications sections are subdivided primarily according to the procedures required to complete a deformation survey project rather than according to the survey method.

The specifications for deformation surveys carried out by conventional survey methods are based upon the author's experience in a number of deformation survey research projects. The specifications for deformation surveys carried out by photogrammetric survey methods are adapted from the following references:

Brown, D. C. (1984). "Tools of the Trade", from 'Close-Range Photogrammetry and Surveying: State of the Art', Proceedings of a workshop held as part of the American Society of Photogrammetry and the American Congress on Surveying and Mapping 1984 Fall Convention, September 9-14, San Antonio, Texas, pages 83-252.

Chapman, M. A. (1986). "Photogrammetric Deformation Measurements", from 'Deformation Measurements Workshop', Proceedings of the Second Workshop on Modern Methodology in Precise Engineering and Deformation Surveys (PEDS II), October 31 and November 1, Massachusetts Institute of Technology, Cambridge, Massachusetts, pages 66-95.

The specifications for deformation surveys carried out by GPS survey methods are adapted

from the following references:

Hothem, L. D. (1985). "Proposed Revisions to Geodetic Survey Standards and Preliminary Specifications for Geodetic Surveys Using Relative Positioning GPS Techniques", National Geodetic Survey, Rockville, Maryland, 20 pages.

Rapatz, P. J. V., Craymer, M., Kleusberg, A., Langley, R. B., Quek, S. H., Tranquilla, J. and Wells, D. (1987). "Procedures and Specifications for Urban GPS Surveys", Technical Report Number 131, Department of Surveying Engineering, University of New Brunswick, Fredericton, New Brunswick, 36 pages.

Wells, D., Beck, N., Delikaraoglou, D., Kleusberg, A., Krakiwsky, E. J., Lachapelle, G., Langley, R. B., Nakiboglu, M., Schwarz, K.-P., Tranquilla, J. and Vanicek, P. (1986). "Guide to GPS Positioning", Canadian GPS Associates, Fredericton, New Brunswick, approximately 300 pages.

In describing localization of deformations in the "Analysis of Data" section reference is made to:

Teskey, W. F. (1988). "Integrated Analysis of Geodetic, Geotechnical, and Physical Model Data to Describe the Actual Deformation Behavior of Earthfill Dams Under Static Loading", Dr.-Ing. Thesis, Institute for Application of Geodesy to Civil Engineering, Stuttgart University, Stuttgart; published as a technical report by the Department of Surveying Engineering, The University of Calgary, Calgary, approximately 150 pages.

The standards and specifications are currently restricted to the determination of deformations by the use of multiply-observed deformation survey networks. Other methods of determining deformations are not discussed in this document. These methods include:

(a) the direct measurement of deformation parameters (e.g. tilt, strain, stress and so on);

(b) the real time processing of continuously recorded deformation data;

(c) the structural finite element method; and,

(d) the integrated analysis of deformation measurements (the structural finite element method rigorously combined with actual deformation measurements).

The transmission of data from remote sites through the use of telemetry is also not discussed in this document.

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1. Definition of Standards and Specifications

1.1 Standards (definition): the minimum levels of quality required to meet specific objectives. The quality standards for deformation surveys are the magnitudes of detectable single point movements with respect to a stable computational base (datum) in a deformation survey network.

1.2 Specifications (definition): the deformation survey network design, monumentation, targetting, instrumentation, equipment adjustment, instrument calibration and adjustment, observational procedures, preprocessing of data, and analysis of data necessary to meet the required deformation survey standard, as well as the presentation of results to document a deformation survey project.

2. The Standards

The first and second order standards for deformation surveys, based on the magnitude of detectable single point movements, are given on the following page.

DEFORMATION SURVEYS

Table 1

Magnitudes of Detectable Single Point Movements, 1st and 2nd Order Deformation Surveys

Order \ Extent	Linear Extent of Deformation Network [2]					
	10 m	100 m	1 km	5 km	10 km	100 km
1st order [1]	0.54 mm	0.9 mm	4.5mm	20.5 mm	40 mm	400 mm
2nd order [1]	2.1 mm	3 mm	12 mm	52 mm.	100 mm	1000 mm

Notes

1. Standards: 1st order: 0.5mm+4ppm (4mm/km) < 5km
                                             4ppm   > 5km

      2nd order: 2mm+10ppm (10mm/km) < 5km
                                 10ppm   > 5km

2. Linear extent of deformation network is the maximum spatial distance between the two most widely separated points in the network.

3. Limit for photogrammetric deformation surveys: 50m (1st order), 100m (2nd order); limit for conventional deformation surveys: 10km; GPS satellite deformation surveys can be of global extent, if necessary.

3. The Specifications

In the specifications, reference is made to (x,y,z) coordinates in a local system. The z-coordinate is to be interpreted as the height component which will be:

(a) the local orthometric height if conventional survey observations only, or conventional survey observations plus photogrammetric survey observations, are used;

(b) the local Euclidean height if photogrammetric survey observations only are used; and,

(c) the local ellipsoidal height if GPS survey observations only, or GPS survey observations plus conventional survey observations and/or photogrammetric survey observations, are used.

3.1 First Order Specifications

3.1.1 Deformation Network Design

Deformation network design is used to determine network configuration, observations and instrumentation required to attain the first order or second order standard, assuming that all other specifications applying to the actual deformation survey are followed.

It is possible to carry out deformation network design in a completely rigorous manner just as ordinary network design can be carried out by using the preanalysis option which is available in most network adjustment computer program systems. (Deformation network design can be carried out by a "sensitivity analysis" in which prechosen movements are tested to determine if they are detectable.) No existing computer program systems, however, have the capability to perform deformation network design. In the absence of such computer program systems, the following empirical method, based on results from about ten deformation survey projects in which the same or similar network observations were made in each epoch of time, shall be used:

$$DSPM = \sqrt{2} * PPP$$

with DSPM the Detectable Single Point Movement; and,

PPP the Precision of the Point Position.

The PPP is determined as follows:

definition of point position	precision of point position
1D (z)	S_z
2D (x,y)	$\sqrt{(S_x^2 + S_y^2)}$
3D (x,y,z)	$\sqrt{(S_x^2 + S_y^2 + S_z^2)}$

with S_x, S_y and S_z the estimated precisions, at the 95% confidence level, of the x, y and z coordinates respectively. A network is generally less sensitive to a single point movement of a given magnitude than it is to the same magnitude of movement for a group of points, thus the magnitude of a detectable single point movement has been used to define the criteria for first and second order deformation surveys.

It should be noted that ordinary network design as well as deformation network design is not nearly so critical for networks observed by photogrammetric methods or GPS satellite methods, as it is for networks observed by conventional survey methods. The reason for this is that in networks observed by photogrammetric or GPS satellite methods there are usually a large number of redundant observations covering the entire network. The result is that network precision, reliability and detectability of movements are reasonably homogeneous throughout the network. By contrast, in networks observed by conventional survey methods there can be "weak" areas with respect to precision, reliability and detectability of movements. However, because deformation network design, even by the empirical method just described, will produce valid results for design purposes for a deformation network observed by any method or any combination of methods, the empirical method shall be applied in all cases.

3.1.2   Monumentation

3.1.2.1 Monument (definition): any structure or device which serves to precisely define the position of a point in a deformation survey network. In order to accurately represent possible movements a monument must be stable with respect to the immediately

surrounding area.

3.1.2.2 Monumented points are classified as follows:

(a) reference points, which are points that are located off the deforming structure, and are generally "occupied"; and,

(b) object points, which are points that are located on the deforming structure, and are often unoccupied.

In the following sections, the reference and object points which are described are to be used for both levelling observations and horizontal observations in order to eliminate the need for special types of monumentation suitable for only one of these types of observations. For this reason, deep bench marks, which are suitable as reference points for levelling observations only, are not included.

The reference and object points described, if properly installed, can be expected to have a long term stability of less than 0.5 mm both horizontally and vertically.

3.1.2.3 Reference points in earth shall be founded at a depth equal to at least twice the depth of frost penetration in the area. (In Alberta a 3 metre depth will be satisfactory in most cases.) These reference points may be either a steel pipe pile (see Figure 1) or a cast-in-place reinforced concrete pile (see Figure 2) installed as described below.

(a) A steel pipe pile (nominal diameter 20 cm, wall thickness not less than that for standard weight pipe) is installed by driving it to "refusal". If refusal occurs at a depth of less than twice the depth of frost penetration in the area, the pile shall be removed and another installation attempted at a different location; or, alternately, a reinforced concrete pile may be installed instead. Steel pipe piles placed in oversized predrilled holes and backfilled shall not be used as reference points.

(b) A cast-in-place reinforced concrete pile (nominal diameter 20 cm) is installed by first drilling a hole to the required depth, i.e. at least twice the depth of frost penetration in the area. A "cage" of longitudinal steel reinforcing bars (ratio of cross-sectional area of steel to concrete not less than 0.02) and nominal circular steel reinforcing bars is then placed in the hole, followed by placement and compaction of concrete having a 28-day compressive strength of not less than 15 megaPascals. Precast reinforced concrete piles driven into predrilled holes or placed in oversized predrilled holes and backfilled

Figure 1
Steel Pipe Pile
Reference Point

DEFORMATION SURVEYS

stainless steel bolt
$\frac{5}{8}$ inch diameter 11NC

stainless steel plate
nominal thickness 2cm

1.5m±

steel pipe
nominal diameter 20cm;
wall thickness not less
than that for
standard weight pipe
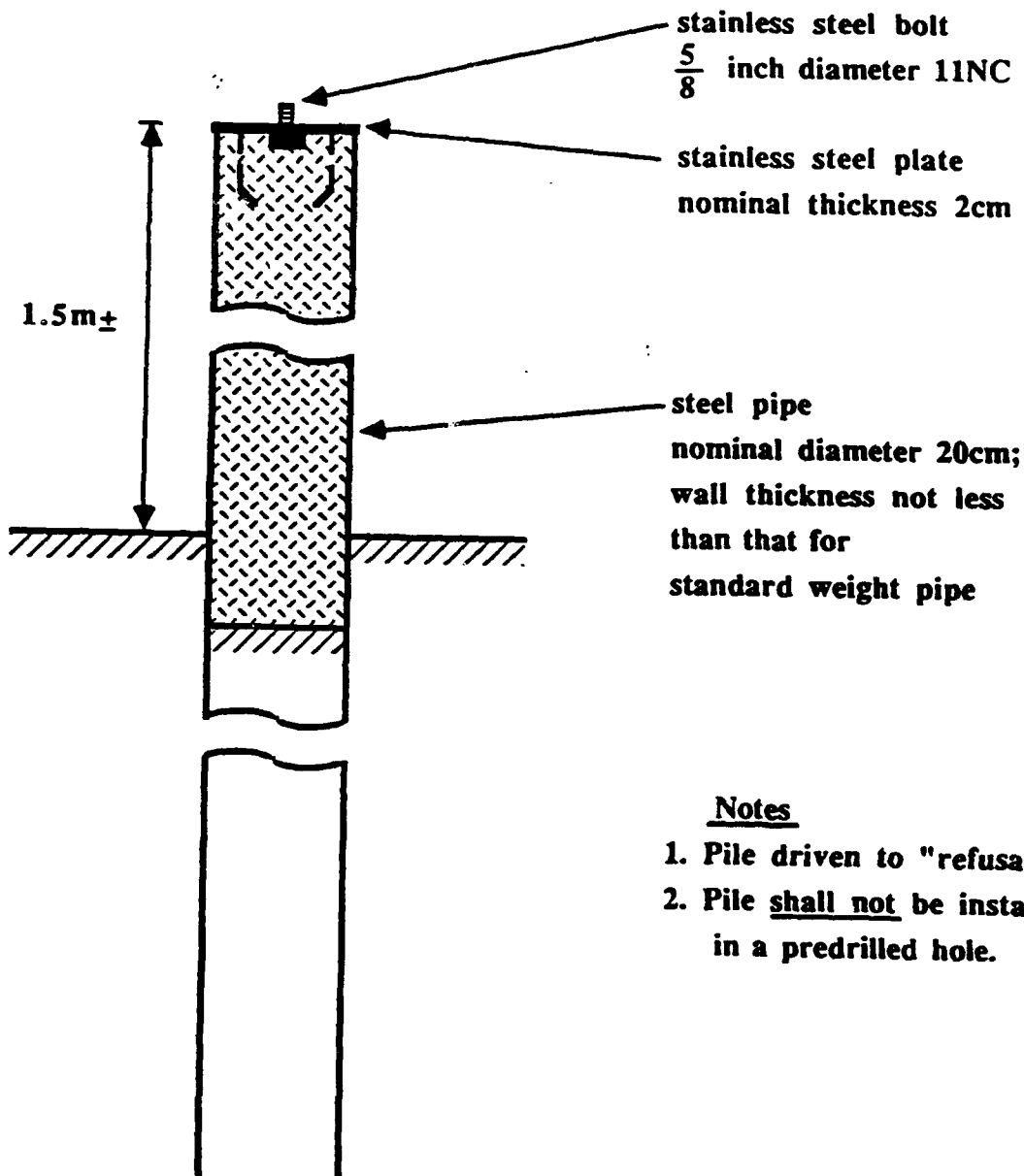
Notes

1. Pile driven to "refusal".
2. Pile **shall not** be installed
   in a predrilled hole.

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Figure 2
Reinforced Concrete Pile
Reference Point

DEFORMATION SURVEYS

stainless steel bolt
$\frac{5}{8}$ inch diameter 11NC

stainless steel plate
nominal thickness 2cm

reinforced concrete pile
nominal diameter 20cm

1.5m±
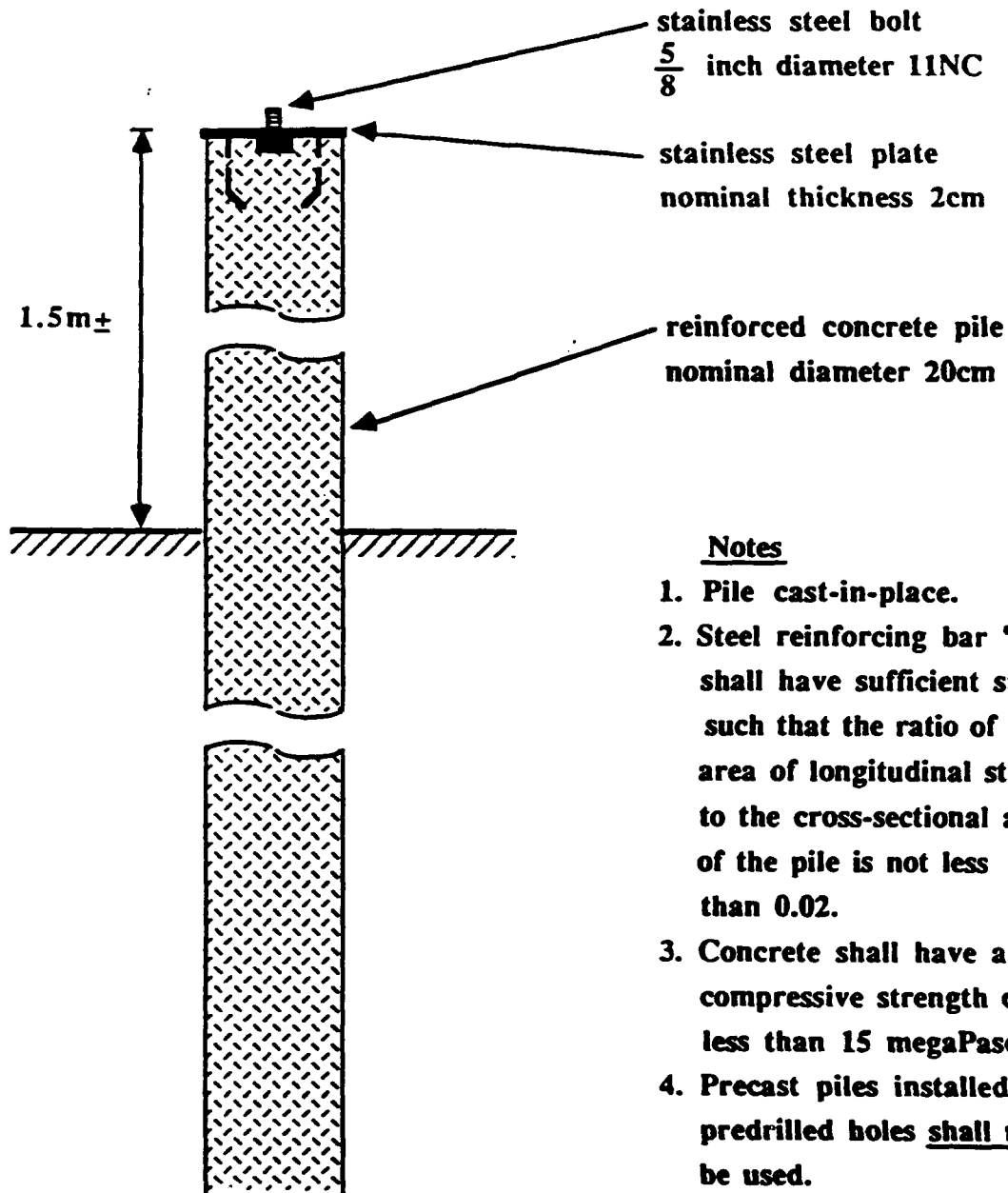
Notes
1. Pile cast-in-place.
2. Steel reinforcing bar "cage" shall have sufficient steel such that the ratio of the area of longitudinal steel to the cross-sectional area of the pile is not less than 0.02.
3. Concrete shall have a 28-day compressive strength of not less than 15 megaPascals.
4. Precast piles installed in predrilled holes shall not be used.

shall not be used as reference points.

It is preferable, especially if typical lengths of lines of observation are less than 1 kilometre, that a reference point extend above ground level to a convenient height (about 1.5 m) so that equipment can be force centered. For either a steel pipe pile reference point extending above ground level or a cast-in-place reinforced concrete pile reference point extending above ground level, a stainless steel plate not less than 2 cm thick is cast into the top of the pile using a minimum of four steel reinforcing bar anchors welded to the underside of the plate. In the centre of the plate a short 5/8 inch diameter 11NC stainless steel bolt, on which the survey equipment is force centered, is drilled through and welded to the plate.

For reference points extending above ground level, that portion of each steel or concrete pile above ground level shall be insulated with a cylindrical sleeve of insulation material, having an R value of not less than 10, for the entire period of time during which survey observations are made. The insulation material can be permanently installed or a number of portable insulating sleeves of the same size can be used. The purpose of the insulation material is to eliminate temperature induced pile movements caused by solar radiation.

For either a steel pipe pile reference point terminating at or slightly below ground level or a cast-in-place reinforced concrete pile reference point terminating at or slightly below ground level, a stainless steel tablet and stub is secured to the hardened concrete as described in section 3.1.2.4. The stainless steel tablet shall have a convex surface (for levelling observations) and an etched cross (for horizontal observations) at the highest point of the convex surface.

For reference points terminating at or slightly below ground level, consideration should be given to protecting the reference point with a cylindrical rim and cover. When not in use, the cover is buried with a few centimeters of earth in order to eliminate vandalism. When the reference point is to be used, it can easily be located with a metal detector.

If it is possible to plan a deformation survey project well in advance of the time first epoch observations are required, reference points shall be installed in a given year and not used until the following year. If such advance planning is not possible, reference points shall not be used for at least one month after installation in order to minimize the effects of

pile rebound (applicable to both steel and concrete piles) and pile shrinkage (applicable to only concrete piles).

3.1.2.4 Reference points in rock or concrete shall consist of a stainless steel tablet (as described in section 3.1.2.3) with a steel reinforcing bar stub of suitable length welded to the underside. A carbide bit shall be used to drill a hole, approximately 50% larger than the stub, in sound rock or concrete, and the stub with attached tablet shall be secured to the rock or concrete using epoxy adhesive to completely fill the void between the stub and the rock or concrete.

3.1.2.5 Object points in earth shall consist of a nominal 3 metre length of square steel HSS (hollow structural section) with a nominal side length of 5 cm and a wall thickness not less than that for standard weight square steel HSS. (See Figure 3.) The base of the HSS is sharpened by cutting it at a 45° angle. Welded approximately 15 cm from the base of the pipe is one length of 10 mm thick 20 cm diameter circular helix with a pitch of 7 cm. Welded to the top of the pipe is a steel plate not less than 5 mm thick. In the centre of the plate a short 5/8 inch diameter 11NC steel bolt, on which survey equipment is force centered, is drilled through and welded to the plate. The HSS object point is drilled into the ground with the use of a power unit. Consideration should be given to protecting the threads of the bolt with a cap during the time survey equipment is not attached to the bolt.

3.1.2.6 Object points in rock or concrete may be either a steel bolt, on which survey equipment is force centered; or a steel insert, into which survey equipment is force centered. The installation of these two types of object points is described below.

(a) A steel bolt to be used as an object point shall be drilled through and welded to a 5 cm diameter, 1 cm thick steel plate. A steel reinforcing bar stub of suitable length shall be welded to the head of the bolt. A carbide bit shall be used to drill a hole, approximately 50% larger than the stub, in sound rock or concrete, and the stub with attached bolt and plate shall be secured to the rock or concrete using epoxy adhesive to completely fill the void between the stub and the rock or concrete. Consideration should be given to protecting the threads of the bolt with a protective cap during the time survey equipment is not attached to the bolt.

Figure 3
HSS with Helix Base
Object Point

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20cm circular helix
1cm thick

steel bolt

$\frac{5}{8}$ inch diameter 11NC

steel plate welded to HSS
nominal thickness 5mm

3m±

square steel HSS
(hollow structural section)
nominal side length 5cm;
wall thickness not less
than that for
standard weight HSS

7cm

15cm±
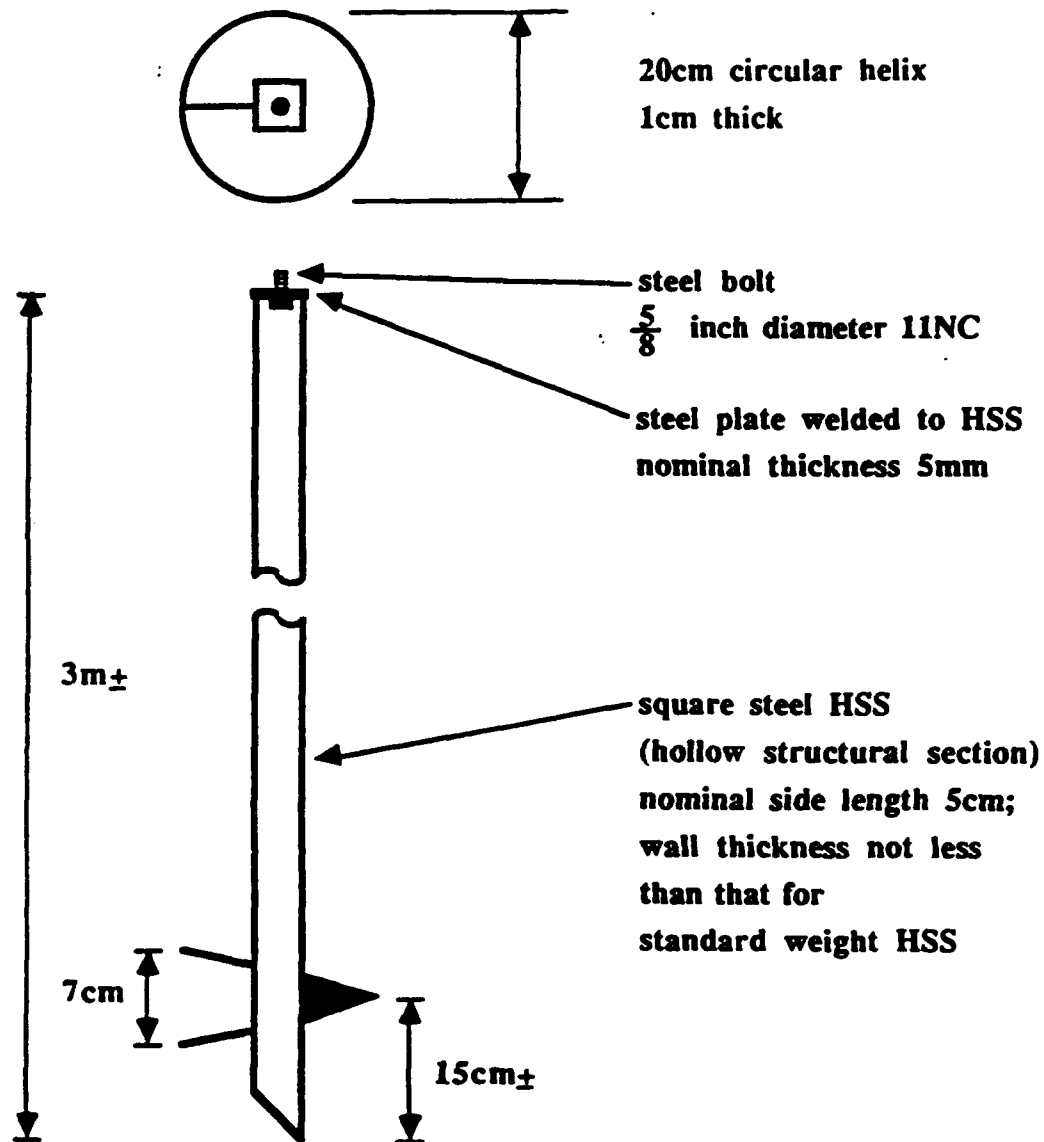
(b) Steel inserts designed for installation in rock or concrete are off-the-shelf items. Several proprietory designs are available but the basic features of steel inserts and the method of installation are similar regardless of the particular design. To install a steel insert in rock or concrete, a hole, just slightly larger than the insert, is first drilled into the rock or concrete using a carbide bit. The hole is cleaned, and the insert is inserted into the hole. To permanently secure the insert in the hole, a lead plug is driven into the base of the insert. Because the bottom portion of the insert is slotted in a longitudinal direction, this action forces the insert against the wall of the hole, permanently securing it. Steel inserts installed with a horizontal orientation shall have a nominal diameter of not less than 1/2" (12.7 mm) in order to bear the downward force of a levelling rod supported at the end of a stainless steel "plug" (see section 3.1.3.6). Steel inserts installed with a vertical orientation may be of any convenient size larger than 1/4" (6.35 mm) nominal diameter. Consideration should be given to protecting the threads of the insert with a bolt during the time survey equipment is not attached to the insert.

3.1.2.7 Object points on steel, masonry or other man-made materials shall be designed such that they are permanently affixed. For object points on steel members a short steel bolt welded to the steel member might be suitable. For masonry or other man-made materials, a steel bolt, plate and rebar stub, or a steel insert (see section 3.1.2.6) might be suitable.

3.1.2.8 An identifier (numeric or alphanumeric) shall be stamped in steel at a suitable location on all reference and object points. A permanent record shall be kept of the identifier, description, location and condition of each reference and monitoring point.

3.1.3 Targetting

3.1.3.1 Target (definition): any device with a well-defined aiming point which is placed vertically over, or attached to, a monument for purposes of making measurements to the point when the deformation survey network is observed. Usually the device is installed only for the period of time required to make the survey measurements. In some cases the

monument itself may be a suitable target.

3.1.3.2 Targets for angular measurements (horizontal, vertical or both) shall be:

(a) standard force centered target sets designed for 1" theodolites;

(b) standard force centered target set-prism combinations used with a particular total station (theodolite and electrooptical distance measurement instrument);

(c) the centres of force centered prisms to which electrooptical distance measurements are made; or,

(d) the monuments themselves.

Target set-prism combinations not matched to a particular total station system shall not be used. In addition, target set-prism combinations for total station systems which are non-coaxial (the same optics are not used for angular and distance observations) shall be tilting target set-prism combinations so that the target set and prism can be tilted together to allow perpendicular alignment to the line of observation (see section 3.1.7.5).

3.1.3.3 Targets for taped distance measurements shall be the monuments themselves.

3.1.3.4 Targets for subtense bar distance measurements shall be the targets permanently affixed to the ends of standard subtence bars.

3.1.3.5 Targets for distances measured with EDM (electronic distance measurement) systems shall be the prisms included with the system. Prisms not matched to a particular EDM system shall not be used.

3.1.3.6 Targets for spirit levelled height difference measurements shall be the monuments themselves or, in the case of steel inserts installed with a horizontal rather than a vertical orientation, the target shall be a stainless steel "plug" about 15 cm long (or longer if obstructions might be encountered) which is threaded into the steel insert at the time the measurement is made. The stainless steel plug shall consist of a round bar threaded at one end. At the opposite end shall be a sphere of slightly larger diameter than the rod. The plug shall be machined in a lathe from one piece of stainless steel bar. If more than one "plug" is to be used on a given project, all "plugs" shall be machined to exactly the same size. A hole

shall be drilled through the bar near the sphere so that a small screwdriver can be used to tighten the "plug" into the insert.

3.1.3.7 Targets for photogrammetric deformation surveys shall preferably consist of a white dot on a black background. The diameter of the white dot shall be selected to yield an average diameter of images of about 60 microns. The diameter of the black background shall be approximately five times the diameter of the white dot.

Retrotargets (targets made from reflective sheeting such as "Scotchlite" manufactured by the 3M Company) may also be used in conjunction with a camera equipped with a ring strobe to illuminate the targets. (Where such a system is used, the targets will appear as white dots on the photographic media. The same system can be used in full sunlight or complete darkness.)

3.1.3.8 Targets for GPS (Global Positioning System) deformation surveys shall be the monuments themselves.

3.1.4 Instrumentation

3.1.4.1 Optical theodolites shall have a telescope magnification of 30X or greater, a plate level with a sensitivity of 20"/2mm graduation or better, an AVCC (automatic vertical circle compensator), and a coincident micrometer reading direct to 1" or less. Electronic theodolites shall have a telescope magnification of 30X or greater, a plate level with a sensitivity of 20"/2mm graduation or better together with an AVCC, or a tilt sensor or tilt sensors to automatically compensate for instrument dislevellment together with a circular level or plate level to roughly level the instrument, and a circle reading system accurate to 3" or less. (An electronic theodolite with a circle reading system accurate to 3" or less is roughly equivalent, in terms of precision, to an optical theodolite with a coincident micrometer reading direct to 1" or less. Hereafter, such an electronic theodolite will be referred to as a nominal 1" or better electronic theodolite.)

3.1.4.2 Steel or invar tapes may be used to measure distances up to 10 metres.

3.1.4.3 Tensioned steel tape, invar tape or invar wire measuring units, which attach directly to the steel bolt or steel insert of object survey points, if available, shall be used instead of steel or invar tapes. (The maximum range of these devices is about 30 metres.)

3.1.4.4 Subtense bars may be used to measure distances up to 30 metres.

3.1.4.5 All distances in a conventional survey network which are to be measured and are longer than 30 metres shall be measured with infrared or laser EDM systems. Distances in the range of 10 metres to 30 metres may be measured with infrared or laser EDM systems. No distance less than 10 metres shall be measured with laser or infrared EDM systems. Microwave EDM systems shall not be used. Barometers shall read direct to 2 mm mercury or less. Thermometers shall read direct to 1°C or less. It shall not be necessary to measure any distances with multiple wavelength instruments such as the Terrameter.

3.1.4.6 Spirit levelled height differences shall be measured with the standard precise levelling equipment: automatic level with telescope magnification of 40X or greater, compensator having a sensitivity equal to or less than 10"/2mm level vial graduation, parallel plate micrometer reading direct to 0.1 mm; invar, double scale rod or rods with a permanently attached circular level and 2 rod supports and with graduations equal to the range of the parallel plate micrometer; portable steel turning points. The compensator in the automatic level shall be a free suspension compensator rather than a mechanical compensator in order to minimize the effect of electromagnetic fields (see section 3.1.7.6).

3.1.4.7 All survey equipment mounted on tripods shall be equipped with an optical plummet located either in the instrument itself or in a detachable tribrach.

3.1.4.8 Metal tripods, as well as towers constructed of any material, shall not be used.

3.1.4.9 Only metric cameras shall be used for photogrammetric deformation surveys. (A metric camera is a camera which has fiducial marks and whose interior orientation parameters (calibrated focal length, location of the calibrated principal point, calibrated lens distortion) have been determined and can be used as approximations in bundle adjustment

with self-calibration. (See section 3.1.8.1.) A metric camera also has some device through which the photographic media may be flattened at the time of exposure.) Usually only one metric camera, moved from exposure station to exposure station, is used in a photogrammetric deformation survey project but the use of more than one metric camera is permissible. Hereafter, however, in all sections relating to photogrammetric deformation surveys, it will be assumed that only one metric camera will be used in a particular epoch of deformation survey measurements.

3.1.4.10   The device used to measure image coordinates (monocomparator, stereocomparator or analytical stereocompiler) for a particular epoch of deformation survey measurements shall read directly to 1 micron or better.

3.1.4.11   GPS receivers (each of which consist of an antenna, a signal receiver, a processor and a recording unit) shall be capable of measuring the phase of at least one of the two GPS carrier frequencies. The receivers shall be capable of tracking at least four satellites simultaneously. The minimum data sampling rate shall be one sample per minute. The receivers shall be capable of displaying an indication of proper operation and data quality. At least one receiver shall be capable of accepting input from an external frequency standard. Different receiver types shall not be used together in a given epoch of deformation survey measurements.

All antennae shall have the position of the best fit centre of phase variation clearly marked on the antennae housing. Separate marks shall be indicated for each operating frequency. The positions of the best fit centre of phase variation marks shall be certified by the manufacturer as being correct to 1mm. (Note that for most antennae this will require that the antennae be oriented correctly in azimuth; see section 3.1.7.9.) The certification shall be repeated and the marks altered, if necessary, whenever an antenna is repaired or modified. Different antenna types shall not be used together in a given epoch of deformation survey measurements.

GPS data is generally recorded on digital cassette tapes or floppy disks. (If any other type of recording media is to be used, it shall be tested before proceeding to the field.) A new cassette tape or disk shall be used for each session. In cases in which the recycling of recording media is necessary, care shall be taken to ensure that all the recorded data has

been extracted from the tapes, checked and archived.

Data recorders shall be capable of recording, for each data sample, the phase of the GPS carrier frequency, the receiver clock time and the signal strength.

3.1.5 Equipment Adjustment

3.1.5.1 Very few adjustments are possible in 1" or better optical theodolites or nominal 1" or better electronic theodolites. In addition, all systematic errors due to theodolite maladjustment or defects in theodolite construction, with the exception of optical plummet maladjustment, are eliminated or rendered insignificant by proper instrument setup and observational procedures. To reduce optical plummet maladjustment to an insignificant level, optical plummets shall be checked and adjusted, if necessary, each time an epoch of deformation survey network observations is to be made. If the optical plummet is located in the instrument alidade, the optical plummet shall be checked by first mounting the instrument on a tripod and accurately levelling it (see section 3.1.7), then rotating the instrument through 180° about its vertical axis. A deviation of the position of the optical axis of the plummet, as shown by two marks made on a piece of paper fixed to the ground or floor, indicates maladjustment. If maladjusted, the adjustment shall be corrected to the midpoint position between the two marks by adjusting screws on either the objective or reticule of the optical plummet. If the optical plummet is located in the tribrach, it shall be checked by clamping the theodolite horizontally on a table such that the tribrach is free to rotate. The tribrach is then rotated through 180°, and any deviation of the position of the optical axis of the plummet, as shown by two marks made on a piece of paper fixed to a wall at least 1.5 metres away, indicates maladjustment. If maladjusted, the adjustment shall be corrected as described previously. For both optical plummets located in the instrument alidade and those located in the tribrach, the optical plummets may also be checked by using auto-collimation with a dish of mercury. In this method the instrument is first mounted on a tripod and accurately levelled over a dish of mercury. Noncoincidence of the image of the centre of the plummet objective with the intersection of the cross hairs on the plummet reticule indicates maladjustment. If maladjusted, the adjustment shall be corrected as described previously, so that coincidence is obtained.

3.1.5.2 No adjustments shall be made in steel or invar tapes; instead, they shall be standardized and the observed distances corrected accordingly.

3.1.5.3 No adjustments shall be made in steel tape, invar tape or invar wire measuring units; instead, instrumental systematic errors shall be determined by calibration and standardization procedures and the observed distances shall be corrected for the systematic errors.

3.1.5.4 The distance between the targets at each end of a subtense bar shall not be adjusted; instead, this distance shall be determined very accurately by a standardization procedure.

The longitudinal axis of a subtence bar is set perpendicular to the line between the observing theodolite and the centre of the subtence bar by an optical sight. This optical sight is located at or near the centre of the subtence bar; its optical axis is meant to be perpendicular to the longitudinal axis of the subtence bar. This condition shall be checked and, if necessary, the optical axis of the sight shall be adjusted each time an epoch of deformation survey network observations is to be made. The check shall be made against a 90° angle set out with a 1" theodolite. The 90° angle shall be marked by three points and the two legs of the angle shall be at least 30 metres long.

3.1.5.5 No adjustments shall be made in barometers, thermometers and EDMI's (electronic distance measuring instruments); instead, instrumental systematic errors shall be determined by calibration and standardization procedures and the observed pressures, temperatures and distances, respectively, shall be corrected for these systematic errors.

In a total station instrument (an EDMI combined with a theodolite) the optical axes of the EDMI and the theodolite shall be aligned parallel to one another so that separate pointings are not required for angular and distance measurements. This shall be accomplished by accurately pointing the total station instrument to its matched target set-prism combination using the theodolite, and adjusting the mechanical device connecting the theodolite and EDMI so that the maximum return signal is received by the EDMI.

3.1.5.6 All systematic errors associated with spirit levelled height difference

measurements, with the exception of maladjustment of the circular levels used to set the levelling rods vertical, can be rendered insignificant by proper instrument setup and observational procedures. To reduce the maladjustment of the levelling rods' circular levels to an insignificant amount, the circular levels shall be checked and adjusted, if necessary, each time an epoch of deformation network observations is to be made. The check shall be made in two orthogonal directions, one facing the scale of the levelling rod and the other at right angles to the first direction, using the string attached to a plumb bob to define the plumbline. If a circular level is maladjusted, the adjustment shall be corrected by adjusting the screws fixing the circular level to the levelling rod such that the rod is vertical in two orthogonal directions when the circular level is centered.

3.1.5.7 The only adjustment which shall be made in tribrachs is the adjustment of the optical plummet, if the tribrach is so equipped. This adjustment is described in section 3.1.5.1.

3.1.5.8 The metric camera used for a particular epoch of deformation survey measurements shall not be adjusted.

3.1.5.9 The device used to measure image coordinates (monocomparator, stereocomparator or analytical stereocompiler) for a particular epoch of deformation survey measurements shall not be adjusted.

3.1.5.10 The GPS receivers used for a particular epoch of deformation survey measurements shall not be adjusted.

3.1.6 Instrument Calibration and Standardization; GPS Survey Validation

3.1.6.1 Calibration (definition): the determination of instrumental constants.

Standardization (definition): the determination of the scale of an instrument.

Validation (definition): the confirmation, by comparison of three-dimensional survey network coordinate differences determined by conventional survey methods and GPS

survey methods, that GPS receivers, procedures and software are capable of meeting first order deformation survey standards.

3.1.6.2 No calibration or standardization is required in 1" or better optical theodolites or nominal 1" or better electronic theodolites.

3.1.6.3 Steel or invar tapes shall be standardized at the standardization temperature (usually 20°C) by comparing their lengths to the length of a standard tape that has been standardized by the National Research Council, Ottawa, or by some other agency which, like the National Research Council, has an interferometer to make very accurate distance measurements over short distances (0.1 ppm (parts per million) accuracy over distances up to 100 metres), and will issue a certificate of standardization. The standard tape shall be kept in a secure location at normal room temperature and used only for the purpose of standardizing the length of other tapes or wires. This standardization shall be carried out each time an epoch of deformation survey network observations is to be made. A permanent record shall be kept of the actual length of each steel or invar tape at each point in time at which it was standardized.

3.1.6.4 Steel tape, invar tape or invar wire measuring units shall be calibrated and standardized at the standardization temperature (usually 20°C) each time an epoch of deformation survey network observations are to be made. The standardization procedure shall be the same as that described in section 3.1.6.3, i.e. the steel tape, invar tape or invar wire in the measuring unit shall be standardized by comparing its length to the length of the standard tape. The calibration procedure, which can be conveniently carried out in a "calibration frame" designed by the manifacturer of the steel tape, invar tape or invar wire measuring unit, determines the additive constant of the instrument. This procedure shall be carried out at the same time as the standardization procedure. A permanent record shall be kept of the additive constant and standardization error of each steel tape, invar tape or invar wire measuring unit at each point in time at which it was calibrated and standardized.

3.1.6.5 The distance between the targets at each end of a subtense bar shall be standardized by the National Research Council, Ottawa, or by some other agency which,

like the National Research Council, has an interferometer to make very accurate distance measurements over short distances, and will issue a certificate of standardization. This standardization procedure shall be carried out once every two years. A permanent record shall be kept of the actual distance between subtence bar targets at each point in time at which it was standardized.

3.1.6.6   Barometers and thermometers shall be calibrated each time an epoch of deformation survey network observations are to be made using an EDMI. (Standardization is not necessary because of the relatively narrow range of pressures and temperatures which will be encountered in a given epoch of deformation survey network observations using an EDMI.) The calibration shall consist of the determination of an additive constant for each barometer and thermometer. The additive constant for a given barometer shall be the difference (to the nearest mm) between the barometer reading and the reading of an adjacent mercury barometer. The additive constant for a given thermometer shall be the difference (to the nearest 0.5°C) between the thermometer reading and the reading of an adjacent scientific thermometer reading direct, and certified correct, to 0.1°C. The pressure and temperature at which barometers and thermometers, respectively, are calibrated shall be approximately the same as the pressure and temperature that one would expect when the epoch of deformation survey network observations are to be made using an EDMI. A permanent record shall be kept of the additive constant and the temperature or pressure at which it was determined, for each barometer and thermometer at each point in time at which it was calibrated.

EDMI's shall be calibrated and standardized each time an epoch of deformation survey network observations is to be made. The calibration of an EDMI shall include the determination of the additive constant and the cyclic error. The additive constant shall be determined from distance measurements in all combinations on a baseline of 5 to 8 stations. The maximum distance shall be the maximum range of the EDMI to one prism under fair atmospheric conditions. The spacing of the baseline stations shall be multiples of the unit length (usually 10 m) of the EDMI in order to minimize the effects of cyclic errors. The additive constant shall be computed by a least squares adjustment in which:

(a) the proper geometric and meteorological corrections are made to the distance observations;

(b) the distance observations are weighted according to an initial estimate of the standard deviation of each distance as given in section 3.1.9; and,

(c) the additive constant and the baseline distances between successive points are the unknowns.

Scale shall not be determined in a baseline calibration unless:

(a) the baseline has been calibrated with an EDMI accurate to 1ppm (1 part per million or 1mm per km) or better, for example, a Mekometer; and,

(b) it can be shown by a deformation analysis (see section 3.1.9) that a particular baseline has been stable over a period of a least 2 years.

The cyclic error of an EDMI shall be determined over one unit length (usually 10 m) of the instrument and a cyclic error diagram shall be plotted showing the cyclic error correction versus the unit length. In determining the cyclic error, the unit length shall be approximately 100 m from the instrument.

EDMI 's shall be standardized by measuring the actual frequency output of the instrument over the range of ambient temperatures in which the EDMI will be operating in the field. A comparison of the actually frequency output versus the nominal frequency output enables one to compute the standardization error of the instrument.

For each EDMI a permanent record shall be kept of.

(a) the additive constant;

(b) the cyclic error (over a unit length); and,

(c) the standardization error (over a range of ambient operating temperatures)

at each point in time at which the particular EDMI was calibrated and standardized.

3.1.6.7 The collimation error of automatic levels shall be determined by a Princeton Test each time an epoch of deformation survey network observations is to be made. A permanent record shall be kept of the collimation error of each automatic level at each point in time at which it was calibrated. In addition to a Princeton Test being performed for each epoch of deformation survey network observations, a two-peg test shall be performed daily. An automatic level which requires frequent correction of the collimation error shall not be used.

An average "rod constant" (difference between left scale reading and right scale reading) shall be determined for each precise levelling rod. The rod constant shall be

determined indoors using an automatic level located approximately 5 m from the rod. A total of 30 left scale and right scale readings shall be taken: 10 between 0 m and 1 m, 10 between 1 m and 2 m, and 10 between 2 m and 3 m. All differences shall be averaged to determine the rod constant to 0.01 mm. The determination of the rod constant shall be carried out once every year. A permanent record shall be kept of the rod constant of each precise levelling rod at each point in time at which it was determined.

Precise levelling rods shall be standardized by the National Research Council, Ottawa, or some other agency which, like the National Research Council, has an interferometer to make very accurate distance measurements over short distances, and will issue a certificate of standardization. This standardization procedure shall be carried out once every two years. A permanent record shall be kept of the standardization error of each precise levelling rod at each point in time at which it was standardized.

3.1.6.8 The metric camera used for a particular epoch of deformation survey measurements shall only be calibrated by a qualified technician. The calibration shall be performed annually. The camera shall also be calibrated analytically using bundle adjustment with self-calibration. (See section 3.1.8.1.)

3.1.6.9 The device used to measure image coordinates (stereocomparator, monocomparator or analytical stereocompiler) for a particular epoch of deformation survey measurements shall only be calibrated by a qualified technician. The calibration shall be performed annually.

3.1.6.10 Prior to any GPS surveys being carried out, a GPS survey validation (see section 3.1.6.1) shall be performed on at least four survey points which include the minimum and maximum point spacings expected in the network. The locations of the stations shall be such that baselines can be measured which are approximately at right angles to each other. (It is recommended that the survey network used for the GPS survey validation be the reference survey network (see section 3.1.2.2) established for the deformation survey project.) The validation shall be considered successful if all discrepancies between coordinate differences determined by the GPS survey and the conventional survey are less than 2ppm of the spatial distance between the points in

question. (At the present time (March 1988) this level of accuracy on the short lines typically encountered in deformation surveys (100m to 5km) is difficult to attain, especially for the height component (larger random error due to satellite geometry, possible systematic error due to change in local equipotential surface-local ellipsoid separation, uncertainty of the location of the centre of phase variation, difficulty of measuring height of antenna); however, it is expected that in a few years with hardware and software improvements and a full constellation of 18 satellites, this accuracy will be attainable.)

3.1.7 Observational Procedures

3.1.7.1 To make angular measurements a theodolite shall first be accurately plumbed over the occupied point, either by attaching the theodolite directly to the point with a tribrach (in the case of a reference point above ground level), or by using a tripod and tribrach with optical plummet (in the case of a reference point at ground level or an object point). Optical theodolites and electronic theodolites not equipped with a tilt sensor or tilt sensors shall be accurately levelled using the AVCC. To ensure that the theodolite has not become dislevelled during the time horizontal angular measurements are made, the AVCC shall be used to measure dislevellment on completion of the measurements. If the dislevellment is greater than 10" the measurements shall be repeated. In electronic theodolites equipped with a tilt sensor or tilt sensors, the hardware in the electronic theodolite or the software controlling the electronic theodolite shall be capable of automatically compensating for the effect of instrument dislevellment on all horizontal angular measurements, after the instrument has first been levelled with its circular or plate level. (Levelling with a circular or plate level will ensure that the dislevellment being compensated for is less than 01'.) To ensure that the electronic theodolite has not become grossly dislevelled during the time the horizontal angular measurements are made, the instrument shall be interrogated to determine dislevellment on completion of the measurements. If the dislevellment is greater than 02' the measurements shall be repeated.

Parallax shall be eliminated by first focussing the reticule, then the objective lens of the theodolite.

Most 1" optical theodolites are equipped with lighting equipment so that the horizontal and vertical circles can be read without natural light. In theodolites so equipped,

the lighting equipment (and fully charged batteries) shall always be used in order to provide uniform lighting conditions.

Very hot or very cold objects and grazing lines, which under adverse conditions can produce large errors in angular measurements due to lateral refraction, shall be avoided whenever possible. Where it is not possible to avoid very hot or very cold objects and grazing lines, the observations which might be affected shall be flagged for future reference.

Neutral atmospheric conditions (cloudy day, slight breeze to thoroughly mix the lower atmosphere and thus make it uniform) are best for all types of conventional survey measurements, including angular measurements with theodolites. If possible, angular measurements shall be planned for these days. In addition, when zenith angles are to be observed, they shall be observed, if possible, between 10AM and 3PM when the rate of change of vertical refraction is usually smallest. Making angular measurements on very hot days shall be avoided, but if this is not possible, the theodolite shall be protected from direct solar radiation with an umbrella.

When a theodolite is being used to measure zenith angles for the purpose of determining height differences, it shall not, if possible, be operated near external electromagnetic fields produced by high voltage power lines, large transformers, large generators and similar equipment. If this cannot be avoided, all distances measured from the affected point shall be flagged for future reference.

The reiteration method specified for control surveys (see, for example, "Alberta Bureau of Surveying and Mapping Survey Control Branch Specifications and Instructions") shall be used in observing directions. The commonly used repetition method shall not be used in observing deformation survey networks. Directions and zenith angles shall be observed in four rounds. In all theodolites, with the exception of certain electronic theodolites which are designed to automatically compensate for horizontal collimation error based on FL (face left) and FR (face right) pointings and readings on one target, FL and FR pointings and readings shall be made on all targets. All horizontal circle and vertical circle readings shall be recorded manually (to 0.1") or electronically so that proper data reduction can be carried out prior to network adjustment and deformation analysis.

Theodolites or gyrotheodolites shall not be used to make observations of azimuth because such observations cannot be made with sufficient accuracy (unless very special

equipment is used and very special and time consuming procedures are adopted) such that they will have an significant beneficial effect on deformation analysis.

3.1.7.2 When measuring distances with steel or invar wire tapes, the uncorrected taped distance (to the mm), the temperature of the tape, the tension applied to the tape, the slope of the tape, the unsupported length or lengths of the tape and the standardization error of the tape shall be recorded.

Plumb bobs shall not be used to plumb a particular mark on a tape over a point; instead, whenever possible, direct point-to-point distances shall be taped. If taping of direct point-to-point distances is not possible, a tripod and theodolite (with an optical plummet and AVCC) shall be plumbed and levelled over one point. The telescope shall be pointed directly at the second point and the distance between the centre of the trunnion axis and the second point shall be taped. In situations in which there is a low obstacle between the two points, two theodolites will be required, and the taped distance shall be between the centres of the trunnion axes of the two theodolites.

Each distance measured with a steel or invar tape shall be measured independently two separate times, by repeating the setup required to make the distance measurement.

3.1.7.3 When measuring distances with tensioned steel tape, invar tape or invar wire measuring units, the uncorrected distance (to 0.01 mm), the temperature of the tape or wire, the tension applied to the tape or wire, the slope of the tape, the unsupported length or lengths of the tape or wire and the standardization error of the tape or wire shall be recorded.

Each distance measured with a tensioned steel tape, invar tape or invar wire measuring unit shall be measured independently two separate times, by repeating the setup required to make the distance measurement.

3.1.7.4 In measuring distances with a subtense bar, the subtense bar and theodolite (1" or better optical theodolite or nominal 1" or better electronic theodolite) shall first be plumbed and levelled over the points defining each end of the line of observation, using the procedures described in section 3.1.7.1. The subtense bar shall be set perpendicular to the line of observation using the optical sight. The angle subtended by the subtense bar at the

theodolite station shall then be measured in four rounds by the reiteration method (see section 3.1.7.1). In order to be able to reduce the measured distance to a point-to-point distance, the HI (height of instrument) and HT (height of target) shall be measured to 0.5 cm with a tape and recorded.

Each distance measured with a subtense bar shall be measured independently two separate times, by repeating the setup required to make the distance measurement.

3.1.7.5 To measure distances with an EDMI, the EDMI shall first be accurately plumbed over the occupied point, either by attaching the EDMI directly to the point with a tribrach (in the case of a reference point above ground level), or by using a tripod and tribrach with optical plummet (in the case of a reference point at ground level or an object point). If the EDMI is a stand-alone unit, the tribrach into which the EDMI fits shall be levelled with the plate level or AVCC of any optical theodolite (a 1" theodolite is not necessary). If the EDMI is combined with an optical or electronic theodolite in a total station system, the same levelling procedure shall be used for EDMI and theodolite. (See section 3.1.7.1.)

Extreme variations in terrain (e.g. line of observation across a valley or line of observation across a large body of water), which under adverse conditions can produce large errors because of unknown atmospheric pressure and temperature along the largest portion of the line of observation, shall be avoided whenever possible. Where it is not possible to avoid extreme variations in terrain, the observations which might be affected shall be flagged for future reference. Neutral atmospheric conditions (cloudy day, slight breeze to thoroughly mix the lower atmosphere and thus make it more uniform) are best for all types of conventional survey measurements, including distance measurements with EDMI 's. If possible, distance measurements with EDMI 's shall be made on these days.

Whenever possible an EDMI shall not be operated near external electromagnetic fields produced by high voltage power lines, large transformers, large generators and similar equipment. If this cannot be avoided, all distances measured from the affected point shall be flagged for future reference.

Before distance measurements are made with an EDMI, the instrument shall be allowed to warm up for the period of time specified by the manufacturer. The EDMI shall also only be operated within the range of ambient temperatures specified by the manufacturer (usually -20°C to +40°C), and with batteries which are fully charged at the

beginning of each day during which the EDMI is used.

Pressure and temperature shall always be measured at the instrument station; and, if the target station is a reference network point (rather than an object point), pressure and temperature shall also be measured at the target station. Pressure shall be measured at any convenient location near the station. Temperature shall be measured in a location shaded from the sun, exposed to the wind, at least 1.5 m above the ground and at least 1.5 m from the observer and the instrument. The most suitable method of meeting all these conditions is to measure the temperature with the probe of an electronic thermister attached to the top of a separate tripod which is shaded from the sun by an umbrella. The temperature display unit, attached to the probe by a small cord, can be read at any convenient location. The correction for atmospheric conditions shall not be applied at the time a distance is measured but rather when the network data is preprocessed (see section 3.1.8), i.e. if the EDMI allows for the atmospheric correction to be preset, it shall be preset to "zero".

Prior to a distance being measured with an EDMI, the prism (or target set-prism combination) at the target station shall be aligned perpendicular (to within 10^0) to the line of observation from instrument station. When measuring the distance between instrument and target stations, the EDMI shall be pointed at the centre of the prism so that the return signal strength is maximized. (In properly adjusted total station instruments, the EDMI is pointed at the centre of the prism when the theodolite is accurately pointed at the target of the target set-prism combination.)

Each distance shall be measured and recorded four separate times by repointing the EDMI between each reading. In order to be able to reduce the set of four uncorrected measured distances from the EDMI to a point-to-point distance or a horizontal distance, and in certain cases to determine height differences (see section 3.1.7.6), the following information shall be recorded:

(a) the pressure (to 2 mm) and the temperature (to 1^0C);

(b) the HE (height of EDMI), HP (height of prism), HTH (height of theodolite) and HT (height of target) to 0.5 cm if only distances are required; or the HE and HP to 0.5 cm and the HTH and HT to 0.5 mm if both distances and height differences are required (see section 3.1.7.6); and,

(c) the zenith angles observed in four rounds.

3.1.7.6 The observational procedures for the determination, by means of precise spirit levelling, of height differences in geodetic networks are well established; see, for example, "Specifications and Recommendations for Control Surveys and Survey Markers" by Surveys and Mapping Branch, Energy, Mines and Resources Canada. In this section precise spirit levelling procedures are modified so that they are suitable for deformation survey networks.

The observational procedures for the determination, by means of precise spirit levelling, of height differences in deformation survey networks are as follows:

(a) Only one double scale invar rod shall be used (so that only one "plug", if required, has to be used; see section 3.1.3.6), unless two rods are being used to reduce the time required to observe the network. When one rod is used, the number of setups between any two points, for which a height difference is required, shall be minimized. When two rods are used, the number of setups will be minimized as previously described, under the restriction that there shall be an even number of setups.

(b) Levelling lines shall only be run in one direction. (Considering the standards for deformation surveys given in section 2 (see Table 1), this is generally adequate. In hilly terrain, however, it may be specified that levelling lines shall be run in both directions to minimize the refraction error. In these cases, the height differences shall be computed as the means of forward and backward runs.)

(c) A cloth tape shall be used to balance total BS (backsight) and FS (foresight) distances between any two deformation survey network points. If these distances cannot be balanced they shall be recorded so that the height difference can be adjusted for the collimation error as given by the results of the latest Princeton Test (see section 3.1.6.7).

(d) The levelling rod shall be moved from its BS position to its FS position as quickly as possible, and the two rod readings in each position shall be made as quickly as possible.

(e) The maximum length of a line of sight shall be 50 m as long as the rod readings satisfy the "data screening" criteria (see section 3.1.8).

(f) The line of sight shall not be less than 0.5 m above the ground.

Neutral atmospheric conditions (cloudy day, slight breeze to thoroughly mix the lower atmosphere and thus make it uniform) are best for all types of conventional survey

measurements, including height difference measurements using precise levelling equipment. If possible, precise levelling shall be planned for these days. Carrying out precise levelling on very hot days shall be avoided but if this is not possible, the automatic level shall be protected from direct solar radiation with an umbrella.

Whenever possible an automatic level shall not be operated near electromagnetic fields produced by high voltage power lines, large transformers, large generators and similar equipment. If this cannot be avoided, all rod readings taken with the automatic level when it is set up in the affected area shall be flagged for future reference.

All BS and FS readings shall be recorded manually or electronically (to 0.01 mm) so that proper data reduction can be carried out prior to network adjustment and deformation analysis.

Height differences can also be determined by one of the methods of trigonometric levelling; this is referred to briefly in section 3.1.7.5. In all of these methods, however, there are important accuracy limitations that must be dealt with. These limitations are a result of:

(a) the accuracy with which HTH (height of theodolite) and HT (height of target) are determined; and,

(b) continuous changes in vertical refraction.

The limitation associated with item (a) can be minimized by using a standard target (which allows HT to be determined to about 0.5 mm) and using a level and a metre rule reading direct to 1 mm to determine HTH (which allows a determination also to about 0.5 mm). A properly adjusted rod level (see section 3.1.5.6 for the adjustment procedure) shall be used with the metre rule to ensure that the metre rule is vertical when HTH and HT are determined. The limitation associated with item (b) can only be minimized by some method of continuously estimating the coefficient of refraction during the time zenith angles are observed.

The coefficient of refraction can be estimated by observing the zenith angles at one or both ends of a line of known length joining two points whose height difference is also known. This can be accomplished in a deformation survey network by first determining the height differences between reference network points by a precise spirit levelling network adjustment involving only reference network points. Lines of observation between pairs of reference network points ("reference lines") are then chosen such that

they pass near the centroid of the object points so that representative coefficients of refraction can be obtained. By combining four rounds of FL and FR measurements of zenith angle at one or both ends of a given reference line with observations of zenith angles to object points, good estimates of coefficients of refraction can be obtained. All heights of objects points with respect to the chosen vertical datum can then be determined by a minimum of two horizontal angles and one zenith angle (corrected for refraction, as described above, and earth curvature).

The neutral atmospheric conditions described earlier are best for height difference measurements using trigonometric levelling; thus, if possible, trigonometric levelling should be planned for days having these conditions. More importantly, because the rate of change of vertical refraction is usually smallest between 10AM and 3PM, observations of zenith angles from which height differences are to be determined shall be made only between these hours.

If only one theodolite is used on the reference line, difference in height to an object point from one observation of zenith angle can be determined to about 3 mm on a 200 m line. If two theodolites are used in a near simultaneous mode on the reference line, difference in height to an object point from one observation of zenith angle can be determined to about 1.5 mm on a 200 m line. (In the absence of systematic errors in the determination of the coefficients of refraction, lower standard deviations of height difference will be achieved after network adjustment.) The estimated standard deviation of the height difference determinations are proportional to the square of the slope distance; therefore, these standard deviations rapidly deteriorate with longer distances.

3.1.7.7   The metric camera used for a particular epoch of deformation survey measurements may be mounted on or in any suitable camera platform (e.g. tripod, rolling height-adjustable camera stand, crane bucket, helicopter, fixed wing aircraft). It is extremely important that camera motion at the time of exposure be minimized. If the camera platform is to be a helicopter or a fixed wing aircraft, it is strongly recommended that a camera with an image motion compensator be used. In a typical application, 5 to 20 exposure stations may be required around the deforming object to get sufficient precision for the object point coordinates; see section 3.1.1. To ensure that all object point targets are sharply defined on photographic media (glass plate or metric camera film), experienced

personnel shall be used to load the photographic media, take the exposures, unload the photographic media and develop the photographic media.

3.1.7.8   The device used to measure image coordinates (monocomparator, stereocomparator or analytical stereocompiler) for a particular epoch of deformation survey measurements shall be operated by an experienced photogrammetric operator. It is preferable that the recording of image coordinates be as automated as possible in order to avoid introducing gross errors into the bundle adjustment with self-calibration. For the same reason, systems which are capable of automatically driving to and centering on the image target (e.g. a retrotarget) are preferred over systems in which the oparator must manually center on the image target.

3.1.7.9 In using GPS receivers to determine three-dimensional coordinate differences, all procedures specified by the receiver manufacturer shall be followed.

Quartz crystal receiver oscillators shall be warmed up for 24 hours prior to an observing session. Atomic oscillators (rubidium vapour cell, cesium beam tube or hydrogen maser) shall be warmed up for one hour prior to an observing session. An uninterruptable power source shall be used with the receiver oscillator. For codeless receivers which must be externally synchronized to UTC (universal time coordinated) synchronization shall take place before each day of observing.

At each survey point at which GPS survey observations are to be made, the antenna shall have at least a 5m separation from any object that projects above the horizontal plane passing through the antenna. Each antenna shall bear a directional mark so that the antenna can be oriented correctly in azimuth.

Whenever possible the antenna shall be set up such that:

(a) no solid object with an elevation greater than 20° above the antenna will block the incoming GPS signals; and,

(b) the antenna is not near metallic structures, flat surfaces or electromagnetic fields produced by high voltage power lines, large transformers, large generators and similar equipment.

Each survey point at which GPS survey observations are to be made shall be occupied at least one-half hour before observing is to commence. During this time the

equipment shall be set up and tested.

Each antenna assembly shall be mounted on a conventional survey tripod using a tribrach equipped with an optical plummet. Each optical plummet shall be checked and adjusted, if necessary, each time an epoch of GPS survey observations are to be made. (The adjustment procedure is described in section 3.1.5.1.) For each antenna the height of the best fitting centre of phase variation of the antenna above the survey point (HA) shall be determined using a level and a metre rule reading direct to 1mm. (This will allow HA to be determined to 0.5mm.) Separate measurements shall be made, at the beginning and end of each session, for the HA for each operating frequency. If a given receiver is to remain at the same station for more than one observing session, the receiver antenna shall be repositioned and HA remeasured at the beginning and end of each observing session.

For each observing session, not less than four satellites shall be observed simultaneously using at least three receivers. Satellite observations shall be made in all four quadrants. Satellites shall only be observed when the HDOP (horizontal dilution of precision) and VDOP (vertical dilution of precision) values will reach a minimum value of 6 or less sometime during the observing session. Satellites shall not be observed at elevations less than 20° above the horizon. Each observing session shall last a minimum of two hours.

In a deformation survey network in which GPS survey observations are to be made, a GPS receiver shall be permanently stationed (i.e. for the duration of the epoch of measurements) at one of the reference network points (the "master station") and shall gather data in every observing session. The same master station shall be used each time an epoch of deformation survey network observations are to be made using GPS receivers. At all other survey points at which GPS survey observations are to be made, 100% of the points shall be included in at least two observing sessions and 75% of the survey points shall be included in at least three observing sessions. For the deformation survey network as a whole, not less than three baselines shall be connected to each point, and not less than 10% of the baselines shall be repeated baselines, with an approximately equal number of north-south and east-west repeated baselines.

At each survey point at which GPS survey observations are being made, cloud cover, temperature, pressure and relative humidity shall be recorded every 30 minutes during the observing session. Barometers and thermometers shall be calibrated each time

an epoch of deformation survey observations are to be made using GPS receivers. The calibration procedure is described in section 3.1.6.6.

GPS survey observations shall not be made during thunder storms or during periods of intense sunspot activity.

A handwritten field log shall be prepared for each set of observations taken in each observation session. Each field log shall include the following information:

(a) project name;

(b) operator name;

(c) date (day, month, year and day of year);

(d) manufacturer, model and serial number of receiver, antenna and data recorder;

(e) station name;

(f) session number;

(g) start time and end time (UTC);

(h) satellites observed;

(i) height of antenna;

(j) cloud cover, temperature, pressure and relative humidity (every 30 minutes during the observing session);

(k) site sketch noting all nearby obstructions, metallic surfaces, flat surfaces and electromagnetic fields; and,

(l) receiver problems.

3.1.8 Preprocessing of Data

3.1.8.1 Preprocessing of conventional survey data shall include:

(a) the screening of data by application of a rejection test at the time the observations are made in order to reject observations which are probable outliers; and,

(b) the application of atmospheric, instrumental, standardization and geometric corrections so that observations can be introduced directly into a survey network adjustment computer program system.

(No gravity corrections of any type shall be required in deformation survey networks observed by conventional survey methods because such networks are of limited extent (see section 2) and gravity anomalies or changes in deflection of the vertical in such areas are

almost always very small; and, whatever small systematic errors are produced due to neglecting the gravity corrections are common to all epochs (the variation of gravity with time is very very small and similar observations are made in each epoch), thus they will cancel out when coordinate differences are computed in the deformation analysis.)

Preprocessing of conventional survey data can be done manually or by means of small field or office computer systems. When the latter are used, it shall not be necessary to carry out the preprocessing exactly as specified in this section as long as it can be shown that some form of data screening has been applied, and that all atmospheric, instrumental, standardization and geometric corrections have been properly applied. The use of data collectors for collecting conventional survey data, especially those which are capable of preprocessing the data, is strongly recommended.

Preprocessing of photogrammetric survey data shall include:

(a) the screening of measured image coordinates in order to reject observations which are probable outliers; and,

(b) the determination of three-dimensional object coordinates and the associated variance-covariance matrix in a local (x, y, z) coordinate system through the use of a computer program system for bundle adjustment with self-calibration.

In the bundle adjustment with self-calibration the following parameters shall be treated as weighted unknowns: focal length, (x, y) position of the principal point, and coefficients for radial lens distortion, asymmetric lens distortion and photographic media unflatness. An estimated standard deviation of 2 microns shall be used for all image coordinates. Because the distances between the exposures stations and the object are so short in photogrammetric deformation surveys resulting in negligible errors due to atmospheric refraction, no atmospheric refraction corrections shall be applied. The earth curvature correction shall only be applied to the height component if the photogrammetric survey is to be combined with a conventional and/or GPS survey.

Preprocessing of GPS survey data shall include:

(a) the determination of three-dimensional coordinate differences and the associated variance-covariance matrix in a local (x,y,z) coordinate system for each baseline observed; and,

(b) the screening of three-dimensional coordinate differences for each baseline observed in order to reject those coordinate differences which are probable outliers.

The corrections which are analogous to the atmospheric, instrumental, standardization and geometric corrections for conventional survey observations are carried out by the GPS processing software.

3.1.8.2 The following rejection tests shall be applied to angular observations made with a theodolite:

(a) if a reduced direction differs from the mean reduced direction by more than 2" it shall be rejected, and the entire set of directions shall be reobserved;

(b) if a reduced zenith angle, which is not being used to compute a height difference, differs from the mean reduced direction by more than 4" it shall be rejected, and the entire set of zenith angles shall be reobserved; and,

(c) if a reduced and corrected zenith angle, which is being used to compute a height difference, differs from the mean reduced and corrected zenith angle by more than 2" it shall be rejected, and the entire set of zenith angles shall be reobserved.

(In this document a round is defined as one FL and one FR pointing on each target; a set is defined as a specified number of rounds. More than one set at a station usually implies completely independent observations on a different day with a different observer.)

Except in the case of zenith angles which are observed for the purpose of determining height differences and thus must be corrected for earth curvature and refraction, no atmospheric corrections, instrumental corrections (other than those made internally in electronic theodolites), standardization corrections and geometric corrections are required for angular observations made with a theodolite. (The skew-normal correction, the normal section to geodesic correction and the arc-to-cord correction which are applied to directions measured in geodetic networks of continental extent need never be applied in a deformation survey network.)

3.1.8.3 The following rejection test shall be applied to each distance measured with a steel or invar tape: if the difference between the two independently measured distances is greater than 2 mm, both distances shall be rejected and the distance reobserved. When this rejection test has been satisfied, a single uncorrected distance shall be computed as the mean of the two independently measured distances.

The following corrections shall be applied to a mean uncorrected distance measured

with a steel or invar tape:

(a) the correction due to the difference between the temperature of the tape and the standardization temperature of the tape (not applicable for invar tapes);

(b) the correction due to the difference between the tension applied to the tape and the standardization tension of the tape;

(c) the correction due to the unsupported length or lengths of the tape;

(d) the correction due to the slope of the tape (not applicable if point-to-point distances required); and,

(e) the correction due to the standardization error of the tape.

3.1.8.4 The following rejection test shall be applied to each distance measured with a steel tape, invar tape or invar wire measuring unit: if the difference between the two independently measured distances is greater than 0.02 mm, both distances shall be rejected and the distance reobserved. When this rejection test has been satisfied, a single uncorrected distance shall be computed as the mean of the two independently measured distances. The corrections applied to a mean uncorrected distance shall be the same as those given in section 3.1.8.3.

3.1.8.5 The rejection tests which shall be applied to distances measured with a subtense bar are the same as those given in section 3.1.8.2. When this rejection test has been satisfied for the two independently measured sets of directions, the distances shall be computed (using the actual length of the subtence bar between the targets rather than the nominal length), and corrected for HTH (height of theodolite) and HT (height of target) if a point-to-point distance is required. A single distance shall then be computed as the mean of the two independently measured distances.

3.1.8.6 The following rejection test shall be applied to each distance measured with an EDMI: if the maximum difference among the four independently measured distances is greater than 5 mm, all distances shall be rejected and the distance reobserved. When this rejection test has been satisfied, a single uncorrected distance shall be computed as the mean of the four independently measured distances.

The following corrections shall be applied to a mean uncorrected distance measured

with an EDMI:

(a) The correction due to the difference between atmospheric conditions at the time the distance was measured and standard atmospheric conditions. The Barrell and Sears formula, neglecting the partial water vapour pressure term which is insignificant for infrared or laser EDMI's specified for deformation surveys (see section 3.1.4.5), shall be applied, using the presssure and temperature measured at the instrument station (if these measurements were only made at the instrument end of the line) or the means of the pressures and temperatures measured at the instrument and target station.

(b) The correction due to the additive constant of the EDMI and prism combination.

(c) The correction due to the standardization error of the EDMI.

(d) The geometric corrections which include the following:

(i) The correction due to unequal HE (height of EDMI), HP (height of prism), HTH (height of theodolite) and HT (height of target). (This correction, which is required whether the distance is reduced to a horizontal distance or a point-to-point distance by the use of HE, HP, HTH and HT, plus zenith angle corrected for earth curvature and estimated coefficient of refraction, is especially important for short distances and must always be made unless (HE-HTH)=(HP-HT).)

(ii) The arc distance to slope distance correction. (This correction can be ignored in almost all cases since it is only 0.3 mm for a 10 km line in average atmospheric conditions.)

(iii) The slope distance to horizontal distance correction. (This correction, which can be applied by using height differences determined by spirit levelling plus HE and HP, or by using zenith angles corrected for earth curvature and coefficient of refraction, shall always be applied when horizontal distances are required.)

(iv) The horizontal distance at station elevation to horizontal distance at reference elevation correction. (This correction, which is equal to approximately 16 mm/km for a height difference of 100 m, shall always be applied.)

3.1.8.7 The following rejection tests shall be applied at each setup of the automatic level:

(a) the difference between the readings on the left scale and right scale of the rod shall be within 0.2 mm of the "rod constant" (see section 3.1.6.7); and,

(b) the difference between the height difference computed from the BS (backsight) and FS

(foresight) readings on the left scale of the rod and the height difference computed from the BS and FS readings on the right scale of the rod shall not be greater than 0.2 mm.

When these rejection tests have been satisfied, a single height difference shall be computed as the mean of the height difference computed from the left scale readings and the height difference computed from the right scale readings. This single height difference shall then be corrected, if necessary, for vertical collimation error. (The correction is only necessary if BS and FS distances have not been balanced.)

A further rejection test shall be applied to height difference observations made with precise levelling equipment. To apply this rejection test the levelling network shall be divided into a number of loops in such a way that all height difference observations between network points are included at least once. The loops may be formed in any manner; the only restriction is that long narrow loops shall be avoided. A height difference misclosure in a loop shall be rejected if it is greater than 0.2*√(no. of setups). If rejected loops are encountered, further loops shall be formed and examined in order to determine height differences between points that are common to loops which have been rejected. This procedure is often successful in locating height difference errors. (A common cause is sinking or rebound of the automatic level tripod between BS and FS readings; thus the importance of a stable setup and quick readings.) If such errors are located, the legs in which the errors occurred shall be reobserved and the rejection test shall be repeated. If the errors cannot be located by this procedure, they are at the marginally detectable level, and can only be detected by "data snooping" (see section 3.1.9).

For height difference determinations by one of the methods of trigonometric levelling, rejection tests shall be applied to:

(a) the zenith angles measured with a theodolite;

(b) the distances measured with an EDMI; and,

(c) the determination of HTH using a level, a metre rule reading direct to 1 mm and a rod level.

The rejection test applied in (a) is described in section 3.1.8.2. The rejection test applied in (b) is described in section 3.1.8.6. The following rejection test shall be applied for (c): if the difference between the two independent determinations of HTH is greater than 0.5 mm, both determinations shall be rejected and HTH shall be determined two more times. When

this rejection test has been satisfied, a single HTH shall be computed as the mean of the two independent determinations.

3.1.8.8 Some form of data screening shall be applied to image coordinates measured with a monocomparator, stereocomparator or analytical stereocompiler. This data screening, which preferably is incorporated in data processing software, may be a simple as a comparison of double measured image coordinates. If this method is applied the maximum discrepancy shall be 2 microns. When such a discrepancy is encountered both measurements shall be rejected and the image coordinates shall be remeasured.

A further level of data screening, called "data snooping" (see section 3.1.4) shall be applied after the bundle adjustment with self-calibration has been completed. Data snooping values shall be computed for all image coordinate observations. The observations corresponding to all such values which are greater than 4 shall be examined carefully as possible blunders. All observations identified as blunders shall be deleted from the adjustment and the adjustment shall be rerun.

3.1.8.9 Preprocessing of GPS survey data, as noted in section 3.1.8.1, includes the determination of three-dimensional coordinate differences for each baseline observed and the screening of these coordinate differences.

The determination of three-dimensional coordinate differences involves:

(a) data recording of the carrier beat phase observations;

(b) elimination of gross errors in the carrier beat phase observations;

(c) detection and removal of the carrier beat phase ambiguities; and,

(d) the least squares adjustment of the carrier beat phase observations to determine the three dimensional coordinate differences.

Data recording is dependent upon the receiver and recording/processing system used. The elimination of gross errors, which is also dependent upon the receiver and the recording/processing system used, may be done either manually or automatically.

The detection and removal of cycle slips may be performed either by:

(a) holding points fixed in the adjustment and manually inspecting residuals;

(b) examining the data for breaks and then modelling the data with a piecewise continuous polynomial for each satellite; or,

(c) forming receiver-satellite-time triple difference observations to determine point coordinates and then searching triple difference residuals to identify large discrepancies in double differences.

The least squares adjustment of the phase observations to determine the three-dimensional coordinate differences may be carried out by the receiver manufacturer's software or by third party software. The software shall provide observation residuals to ensure that no systematic errors, such as undetected or wrongly corrected cycle slips, exist.

Screening of GPS survey data shall be carried out by examining coordinate difference discrepancies in repeated baselines and loop closures.

For repeated baselines:

(a) the coordinate difference discrepancy in any component, with respect to the average coordinate difference for that component, shall not exceed 5mm;

(b) the maximum discrepancy in coordinate difference in any component shall not exceed 10mm;

(c) the resultant of the coordinate difference discrepancy, with respect to the average resultant of the coordinate difference discrepancy, shall not exceed 5mm;

(d) the maximum discrepancy in the resultant of coordinate differences shall not exceed 10mm;

(e) the baseline length discrepancy, with respect to the average baseline length, shall not exceed 5mm; and,

(f) the maximum discrepancy is baseline length shall not exceed 5mm.

For loop closures:

(a) at least three independent observing sessions shall be represented in each loop;

(b) no more than four baselines shall be combined to form a loop;

(c) the average misclosure in any component shall not exceed 5mm;

(d) the maximum misclosure in any component shall not exceed 20mm;

(e) the average resultant misclosure shall not exceed 5mm;

(f) the maximum resultant misclosure shall not exceed 20mm;

(g) the rms (root mean square) value of each of the three misclosure components shall not exceed 5mm; and,

(h) all of the baselines shall meet the criteria, noted in (a) to (g) inclusive above, for

inclusion in a loop.

In this section it has been assumed that GPS survey observations produced by the GPS processing software are in the form of three-dimensional coordinate differences. Some GPS processing software packages are capable of providing network solutions (i.e. three-dimensional coordinates x, y, z in a local coordinate system), in addition to or instead of, baseline solutions (i.e. three-dimensional coordinate differences dx, dy, dz). It shall be acceptable to use such network solutions as long as some form of data screening (e.g. "data snooping"; see section 3.1.9) has been built in to the software, and residuals (or, if possible, standardized residuals upon which data snooping is based) are computed for all of the baseline solutions so that poor GPS survey data can be deleted or down weighted.

3.1.9 Analysis of Data

3.1.9.1 The analysis of data in deformation surveys shall include:

(a) the adjustment of observations made in each epoch in order to determine coordinates of points and associated variance-covariance matrices plus other information required for the localization of deformations; and,

(b) the localization of deformations between chosen epochs in order to determine statistically significant point movements.

Adjustment and localization may be carried out by separating the two-dimensional (horizontal) analyses from the one-dimensional (vertical) analyses; or, three-dimensional analyses (horizontal and vertical) may be used.

3.1.9.2 The following estimated standard deviations shall be used in adjustments of conventional survey observations:

(a) for directions: 2 arc seconds;

(b) for zenith angles (all results in arc seconds):

$$\sqrt{ (2)^2 + ((0.3 * 206265 * S) / 2R)^2 }$$

when simultaneous or near simultaneous zenith angles are observed and corrected for earth curvature and computed coefficient of refraction;

$$\sqrt{ (2)^2 + ((0.6 * 206265 * S) / 2R)^2 }$$

when 1-way zenith angles are observed and corrected for earth curvature and

computed coefficient of refraction; and,

$$\sqrt{(2)^2 + ((1.0 * 206265 * S)/2R)^2}$$

when 1-way zenith angles are observed and corrected for earth curvature and estimated coefficient of refraction (value of 0.13 often used);

with S the approximate horizontal distance (km); and,

R the approximate radius of the earth (6370 km);

(c) for distances measured with a steel or invar tape: 2 mm;

(d) for distances measured with a steel tape, invar tape or invar wire measuring unit: 0.02 mm;

(e) for distances measured with a subtence bar (result in mm):

$$\sqrt{(2 * (0.5)^2 + (5S^2 * 10^{-3})^2)}$$

with S the approximate horizontal distance (m); and,

(f) for distances measured with an EDMI (result in mm):

$$\sqrt{((5)^2 + (2S_s * 10^{-3})^2)}$$

with S_s the approximate spatial distance (m).

For adjustments involving the combination of conventional and/or GPS survey observations with photogrammetric survey observations, the photogrammetric survey observations shall be considered to be the three-dimensional coordinates and associated variance-covariance matrix in a local (x, y, z) coordinate system which were determine through bundle adjustment with self-calibration. The photogrammetric survey observations shall be given as previously specified so that the combination of all possible types of survey data (conventional, photogrammetric, GPS) can easily be accomplished in a survey network adjustment computer program system.

If only photogrammetric survey observations have been made in a particular deformation survey project, free network constraints (also referred to as "inner constraints") shall be used to define the datum. If photogrammetric survey observations are to be combined with conventional survey observations, the datum shall be defined by the constraints to be used in the conventional survey adjustment (see section 3.1.9.5). If photogrammetric survey observations are to be combined with GPS survey observations (with or without conventional survey observations), the GPS survey observations shall be used to define the datum (see below).

For adjustments involving GPS survey observations, the observations shall be

either:

(a) the three-dimensional coordinate differences and the associated variance-covariance matrix in a local (x, y, z) coordinate system for each baseline observed; or,

(b) the three-dimensional coordinates and the associated variance-covariance matrix in a local (x, y, z) coordinate system for the entire network.

GPS survey observations shall be given as specified in (a) or (b) above so that the combination of all possible types of survey data (conventional, photogrammetric, GPS) can easily be accomplished in a survey network adjustment computer program system.

In any deformation survey network in which GPS survey observations are to be combined with conventional and/or photogrammetric survey observations, the GPS survey observations (x, y, z or dx, dy, dz in a local system) shall be used to define the datum (i.e. location, orientation, scale). This choice of datum is the most convenient, and will also ensure that there will be no network distortions which would almost certainly occur if conventionally determined network coordinates were used to define the datum. The only exception, at present, to the previous statement is that the z (height) component is more accurately determined by conventional precise levelling; and, in addition, the GPS height component and the conventionally determined height component differ by the change in local equipotential surface-local ellipsoid separation. (This change may amount to 1cm/km in mountainous terrain.) Because of the factors, if GPS survey heights are to be used with heights determined by conventional precise levelling, all GPS survey heights shall be corrected for change in local equipotential surface-local ellipsoid separation.This correction shall be determined by fitting a plane to three well-spaced points at which both GPS survey heights and heights from conventional precise levelling have been determined.

3.1.9.3 The network adjustment computer program system shall compute the following quantities:

(a) the point coordinates and their associated variance-covariance matrix;

(b) the point error ellipses;

(c) the quadratic form of the residuals;

(d) the total redundancy of the network; and,

(e) the estimated global variance factor ((c)/(d)).

If the network adjustment involves only conventional survey observations, the following

quantities shall also be computed:

(f) the redundancy number of each observation;

(g) the "data snooping" value for each observation;

(h) the quadratic form of residuals for each observation type (i.e. directions, zenith angles, etc.);

(i) the sum of redundancy numbers for each observation type; and,

(j) the estimated variance factor for each observation type ((h)/(i)).

3.1.9.4 The adjustment of observations and localization of deformations shall be performed in two stages. The first stage shall be the adjustment of observations and localization of deformations in the reference network. The second stage shall be the adjustment of observations and localization of deformations in the entire network.

3.1.9.5 In the adjustment of observations in the reference network, all the reference network points and all the observations directly connecting reference network points shall be included in the adjustment. Ordinary minimum constraints, or free network constraints (also often referred to as "inner constraints"), shall be used. Ordinary minimum constraints for networks in which only conventional survey observations have been made are:

type of network	ordinary minimum constraint(s)
1D (z)	z of 1 point fixed
2D (x,y) with distances	x and y of 1 point fixed;
	azimuth to 2nd point "fixed"
	(standard deviation of azimuth = 0.1")
2D (x,y) without distances	x and y of 2 points fixed
3D (x,y,z) with distances	x, y and z of 1 point fixed;
	azimuth and zenith angle to 2nd point "fixed";
	zenith angle to 3rd point "fixed"
	(standard deviations of azimuth and zenith angles = 0.1")
3D (x,y,z) without distances	x,y and z of 2 points fixed;
	x,y or z of 3rd point fixed

For two-dimensional and three-dimensional networks these ordinary minimum constraints

shall be applied to points on opposite sides of the network. If "data snooping" values (standardized residuals) are computed, the observations corresponding to all such values which are greater than 4 shall be examined carefully as possible blunders. Special attention shall be paid to observations flagged for future reference at all time the observations were made. (See section 3.1.7.) All observations identified as blunders shall be deleted from the adjustment, and the adjustment shall be rerun. One should expect the estimated global variance factor (section 3.1.9.3 (d)) and the estimated variance factors for each observation type (section 3.1.9.3 (i)) to be close to 1. If estimated variance factors for each observation type are computed and if any are less than 0.5 or greater than 2.0, all of input variances shall be rescaled by the ratios (estimated variance factor of observation type) / (estimated global variance factor) and the adjustment rerun. All epochs of observations shall be adjusted in the same manner with regard to constraints applied, "data snooping" and rescaling of input variances.

3.1.9.6 For the localization of deformations in the reference network the same minimum constraints shall have been applied in the two epochs being analysed. The localization of deformations shall be performed by a global test and a number of local tests (one for each unconstrained point) of the quadratic forms of the coordinate differences between epochs. (The localization of deformations is described in Teskey (1988) which is cited in the Preface.) If ordinary minimum constraints are used instead of free network constraints, the localization of defu₁ mation results from several combinations of ordinary minimum constraints shall be examined to ensure the stability of reference points between epochs. Any reference points which are shown not to be stable between epochs shall be excluded from the reference network.

3.1.9.7 In the adjustment of observations in the entire network, the constraints shall be either:

(a) all reference network points, shown to be stable by the procedure described in section 3.1.9.6, fixed; or,

(b) all reference network points, shown to be stable by the procedure described in section 3.1.9.6, forming the free network computational base.

(Application of constraints (a) above will result in the network being slightly

overconstrained; this, however, will have only a small effect on the deformation analysis since the stability of these points have been confirmed by the procedure described in section 3.1.9.6.) Examination of "data snooping" values and rescaling of input variances for each epoch of observations shall be the same as that described in section 3.1.9.5.

3.1.9.8 For the localization of deformations in the entire network the same constraints shall have been applied in the two epochs being analyzed. The global test and local tests for deformation shall be the same as those referred to in section 3.1.9.6. The final results are apparent movements of all object points (and any excluded reference points), and the results of the localization test for each of these points which indicate whether or not the point movements are statistically significant. (If desired, the same localization test can be applied to movements of groups of points, but it is recommended that this only be done if one has apriori information that a movement of a group of points might have occurred.)

3.1.10   Presentation of Results

3.1.10.1 The results of each deformation survey project shall be accompanied by a project report. Any deviations from the specifications given in this document (e.g. input standard deviations of observations) shall be described in the project report.

3.1.10.2 It shall not be necessary to include the survey observations except when GPS survey observations have been used. All GPS survey data obtained shall be submitted, on the original media, with the project report, along with the complete set of handwritten field logs (see section 3.1.7.9).

3.1.10.3 The project report shall include a tabular summary of each network adjustment. Each such summary shall include:
(a) the constraints applied;
(b) the point names;
(c) the adjusted point coordinates to the nearest 0.1 mm;
(d) the standard deviations of point coordinates to the nearest 0.1 mm;
(e) the dimensions of error bars (at the one standard deviation level) to the nearest 0.1 mm

for one-dimensional network point coordinates (same as (d) for one-dimensional networks), the dimensions of axes of error ellipses (at the one standard deviation level) to the nearest 0.1 mm plus orientation angle to the nearest 0.1 degree for two-dimensional network point coordinates and the dimensions of axes of error ellipsoids (at the one standard deviation level) to the nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for three-dimensional network point coordinates;

(f) the quadratic form of the residuals;

(g) the total redundancy of the network; and,

(h) the estimated global variance factor ((f)/(g)).

If network adjustments involve only conventional survey observations, the following quantities shall also be included in the tabular summaries of network adjustments:

(i) the redundancy number of each observation;

(j) the "data snooping" value for each observation;

(k) the quadratic form of residuals for each observation type (i.e. directions, zenith angles, etc.);

(l) the sum of redundancy numbers for each observation type; and,

(m) the estimated variance factor for each observation type ((k)/(l)).

In network adjustments of conventional survey observations in which systematic errors (e.g. EDMI additive constant) have been parametrized, the solution for the parameters and their full variance-covariance matrix shall also be given.

3.1.10.4 The project report shall include a tabular summary of each localization of deformations. Each such summary shall include:

(a) the constraints applied in the two network adjustments;

(b) the point names;

(c) the apparent displacements to the nearest 0.1 mm and associated directions (with those statistically significant flagged) to the nearest 0.1 degree; and,

(d) the dimensions of error bars (at the one standard deviation level) to the nearest 0.1 mm for one-dimensional network point displacements, the dimensions of axes of error ellipses (at the one standard deviation level) to the nearest 0.1 mm plus orientation angle to the nearest 0.1 degree for two-dimensional network point displacements and the dimensions of axes of error ellipsoids (at the one standard deviation level) to the

nearest 0.1 mm plus out-of-plane angles to the nearest 0.1 degree for three-dimensional network point displacements.

3.1.10.5 The project report shall include figures with plan or cross-sectional views showing the outline of the structure, the location of deformation network points and their names. All reference points shall be denoted by one symbol; all object points shall be denoted by a different symbol. Movements shall be plotted as vectors with their associated error bars or error ellipses. Statistically significant movements shall be flagged. Only displacements between two chosen epochs shall be plotted on a given figure. Displacement contours shall not be plotted.

The project report shall also include figures showing one-dimensional cumulative displacements of certain points in certain critical directions versus time. The error bar associated with each cumulative displacement shall be plotted with the displacement. Data from all deformation analyses performed on the project shall be included. Statistically significant cumulative displacements shall be flagged. (Critical one-dimensional cumulative displacements might be, for example, movements in the downstream and vertical directions of a small number of points on the crest of a dam, or movements in the downhill and vertical directions of a small number of representative points in a slide.)

3.2 Second Order Specifications

The second order specifications contained in the subsequent sections (sections 3.2.1 to 3.2.10 inclusive) are the same as the first order specifications except for relatively minor changes relating mainly to monumentation, conventional deformation surveys and analysis of data. This approach is followed because for all types of deformation surveys, and for photogrammetric deformation surveys and GPS deformation surveys in particular, if less than a very good network design and less than very good observational conditions are encountered, the first order standard will not be attained. On this basis the first order standard should be considered the best attainable using the current state-of-the-art procedures, and the second order standard should be considered the minimum acceptable.

3.2.1 Deformation Network Design

Deformation network design will be the same as that described in section 3.1.1.

3.2.2 Monumentation

Monumentation shall be the same as that described in section 3.1.2 with the following exceptions:

(a) The use of insulation material on the above-ground portion of cast-in-place reinforced concrete piles or steel pipe piles driven to refusal (see section 3.1.2.3) shall not be necessary.

(b) It shall be permissible to use the HSS with helix base (see section 3.1.2.5) for reference points in earth instead of cast-in-place reinforced concrete piles or steel pipe piles driven to refusal.

(c) Either one of the following may be used as object points in earth:

    (i) the HSS with helix base; or,

    (ii) nominal 18 mm diameter smooth steel bars.

The bars shall be driven to "refusal" with a large hammer. The bars shall be installed in 1 m long sections using a steel cap to protect the 5/8 inch diameter 11NC threads as the sections are driven into the ground. The first section of bar for each installation shall have a sharp point machined at one end. The sections shall be threaded together and the threads shall be locked using a quick setting thread sealant or a quick setting epoxy adhesive. Consideration should be given to protecting the threads of the last section of bar with a cap during the time survey equipment is not attached.

3.2.3 Targetting

Targetting shall be the same as that described in section 3.1.3.

3.2.4 Instrumentation

Instrumentation shall be the same as that described in section 3.1.4.

3.2.5 Equipment Adjustment

Equipment adjustment shall be the same as that described in section 3.1.5.

3.2.6 Instrument Calibration and Standardization

Instrument calibration and standardization shall be the same as that described in section 3.1.6 with the following exceptions:

(a) It shall not be necessary to standardize steel or invar tapes, or the distance between the targets at each end of a subtense bar.

(b) EDMI scale error may be determined along with additive constant in a baseline calibration, i.e. it shall not be necessary to determine EDMI scale error by measuring the actual frequency output of the EDMI. It shall also not be necessary to determine the EDMI cyclic error.

(c) A "rod constant" shall be determined at the beginning of a deformation survey project from five left scale and right scale readings taken between 1 m and 2 m. The same "rod constant" for a given rod shall be applied throughout the project. It shall not be necessary to keep a permanent record of the "rod constant". It shall also not be necessary to standardize each precise levelling rod.

3.2.7 Observational Procedures

Observational procedures shall be the same as those described in section 3.1.7 with the following exceptions:

(a) It shall not be necessary to use lighting equipment in theodolites or subtense bars.

(b) It shall not be necessary to use umbrellas to shade instruments from direct solar radiation.

(c) It shall not be necessary to observe zenith angles, from which height differences are to be determined, only between 10AM and 3PM.

(d) It shall not be necessary to measure pressure and temperature at both ends of a line being measured with an EDMI, i.e. in all cases it shall only be necessary to measure pressure and temperature at the instrument station.

(e) It shall not be necessary to use two theodolites in a near simultaneous mode to determine height differences from zenith angle observations.

3.2.8 Preprocessing of Data

Preprocessing of data shall be the same as that described in section 3.1.8 with the following exceptions:

(a) The following rejection test shall be applied to all angular observations made with a theodolite (directions and zenith angles): if a reduced angular observation differs from the mean reduced angular observation by more than 4", the angular observation shall be rejected and the entire set of angular observations reobserved.

(b) The following rejection test shall be applied to each distance measured with a steel or invar tape: if the difference between the two independently measured distances is greater than 5 mm, both distances shall be rejected and the distance reobserved.

(c) The following rejection test shall be applied to each distance measured with a steel tape, invar tape or invar wire measuring unit: if the difference between the two independently measured distances is greater than 0.05 mm, both distances shall be rejected and the distance reobserved.

(d) The following rejection test shall be applied to each distance measured with an EDMI: if the maximum difference among the four independently measured distances is greater than 8 mm, all the distances shall be rejected and the distance reobserved.

(e) The following rejection tests shall be applied at each setup of the automatic level:
   (i) the difference between the readings on the left scale and the right scale of the rod shall be within 0.3 mm of the "rod constant" (see section 3.2.6); and,
   (ii) the difference between the height difference computed from the BS (backsight) and FS (foresight) readings on the left scale of the rod and the height difference computed from the BS and FS readings on the right scale of the rod shall not be greater than 0.3 mm.

It shall not be necessary to apply any further rejection tests to spirit levelled height difference measurements.

3.2.9 Analysis of Data

Analysis of data shall be the same as that described in section 3.1.9 with the following exceptions:

(a) The following estimated standard deviations shall be used in the adjustments of conventional survey observations:

   (i) for directions: 3 arc seconds;

   (ii) for zenith angles (all results in arc seconds):

$$\sqrt{ (3)^2 + ((1.0 * 206265 * S) / 2R)^2 }$$

   when 1-way zenith angles are observed and corrected for earth curvature and <u>computed</u> coefficient of refraction; and,

$$\sqrt{ (3)^2 + ((1.5 * 206265 * S) / 2R)^2 }$$

   when 1-way zenith angles are observed and corrected for earth curvature and <u>estimated</u> coefficient of refraction (value of 0.13 often used);

   with S the approximate horizontal distance (km); and,

   R the approximate radius of the earth (6370 km);

   (iii) for distances measured with a steel or invar tape: 5 mm; and,

   (iv) for distances measured with a steel tape, invar tape or invar wire measuring unit: 0.05 mm.

(b) It shall not be necessary that the network adjustment computer program system compute items (f) to (j) inclusive of section 3.1.9.3.

(c) The localization of deformations in the reference network and the entire network may be performed by the localization method referred to in section 3.1.9.6; or, by the error ellipse method, in which any displacement extending outside the boundary of an error ellipse corresponding to the point displacement at a prechosen confidence level (usually 95%) is deemed to be statistically significant.

3.2.10 Presentation of Results

Presentation of results shall be the same as that described in section 3.1.10 with the following exception: it shall not be necessary that the project report include items (i) to (m) inclusive of section 3.1.10.3.

FRENCH LEGISLATION ON INSPECTION AND SURVEILLANCE OF DAMS

French Legislation on Inspection and Surveillance of Dams
(from ICOLD, [1989, pp. 91-94])

1. General

French law requires inspection and surveillance of dams that might affect public safety.

The instructions are set out in an "interministry circular" dated 14 August 1970 bringing up to date earlier circulars of 1927 and 1928.

The circular applies to all dams more than 20 m high (above natural ground level) and to all others whose failure would have serious consequences to human life.

The surveillance of a dam is the responsibility of its owner.

A special Government Department ensures that this surveillance is properly carried out by the owner.

2. Principle Instructions in the Circular

2.1 Owner Records

The owner must create or keep up-to-date records containing all documents relating to the structure, including construction drawings, results of geological and geotechnical investigations, hydrological data, description of maintenance and repair works, drawings of the instruments, results of the measurements, documents relating to reservoir operation etc.

2.2 First Filling

Proposals for procedures during first filling must be approved by the *Comité Technique Permanent des Barrages* (Permanent Technical Committee on Dams), an interministerial committee created in 1966. The proposals must be submitted at the same time as the design of the project.

The proposals have to include:

    (1)   description and layout of instruments,
    (2)   filling programme including pauses for measurements,
    (3)   reading frequency, and

(4)   steps to be taken in case of problems (emergency drawdown, who is to be informed)

The owner must report fully to the Government Department within 6 months after filling, and the report must contain a complete analysis of the behaviour of the dam during the first filling.

It is amplified as necessary in an operator's annual report

2.3 Operator's Register

This basically consists of:

(1)   periodic inspections of the structure, surrounding area and discharge equipment, and
(2)   instrument readings.

The frequency of inspections depends on the size of the dam and the occurrence of problems.

The operator's annual report must be sent to the Government Department, giving the information in the operator's register and the results of measurements. Every two years, this report must contain a full analysis of the dam's behaviour.

2.4 Visits by the Government Department

The Government Department makes at least one visual inspection of each dam annually with the operator.

Within 5 years after first filling, the Government Department makes a full inspection of each dam. This includes inspection of parts normally submerged. Thereafter, this inspection is repeated every 10 years.

In principle, such inspections require the reservoir to be emptied. However, if this would cause serious problems, miniature submarines or underwater TV can be used.

2.5 Special Rules for Old Dams

Old dams were not built on modern design criteria. Many have little or no instrumentation, even though safety factors are lower than would be expected today.

The circular of 14.08.70 provides of a special review of these dams. Each Government Department has to list the dams thought to need such review and give an order of priority. The owner is then invited to commission a report by engineers or consultants, containing inter alia recommendation on the instruments to be installed. The report is submitted to the Permanent Technical Committee on Dams.

A SPECIFICATION FOR DEFORMATION SURVEYS OF THE OHAKURI DAM
(NEW ZEALAND)

A SPECIFICATION FOR DEFORMATION SURVEYS OF THE OHAKURI DAM

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ELECTRICORP PRODUCTION
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1. INTRODUCTION

This document is a specification for undertaking three dimensional deformation monitoring surveys on Electricorp Productions (a division of the Electricity Corporation of NZ Ltd.) Arapuni Power Station which includes the various Arapuni structures and the surrounding country. Arapuni Power Station is on the Waikato River between Waipapa (upstream) and Karapiro Dams. This specification details the various survey components, appropriate instruments and field techniques, sets accuracy standards, describes suitable computational, adjustment and analysis methods and specifies the formats for reporting on such surveys. Appendix 13 shows a resume of technical details of the Arapuni Power Station.

2. DESCRIPTION OF SURVEY COMPONENTS

The complete deformation survey is made up of a number of components which are described below. Appendix 2 to 6 (App.2-6) shows the location of the survey pillars, Bench Marks (BM's) and structure points that make up the monitoring network. In this specification survey station names are shown in Italics. The first surveys were undertaken at Arapuni in 1931, 1949 and 1960. In 1975 the network was improved and extended with the construction of survey pillars replacing ground blocks. The emphasis of the early surveys at Arapuni were on horizontal monitoring with vertical monitoring being mainly from 1975. During the station reimbursement from 1989 to 1990 the network was increased to monitor all the major structures during and after the station commissioning.

2.1 CONTROL LEVELLING

The vertical monitoring of the network is based on precise levelling from a number of BM's surrounding the various Arapuni structures. Appendix 2 shows the overall layout of these BM's with App.3-6 showing the main structures in more detail. The levels are based on Moturiki datum. The full levelling network covers an area of approx. 1.5 square km from the dam in the south to the downstream Waikato river bridge in the north, from 200m west of the spillway in the west to the centre of Arapuni village in the east. The levelling is made up of many interlocking circuits which traverse through all of the main structures with numerous BM's and pillars levelled in the general ridge area between the headrace and the Power House (PH). Some of the levelling lines are done one way only but are included in circuits. *Arapuni Fundamental BM* (in village) is the area origin with pillar *G* to the west occasionally used as origin for some recent surveys. BM *AB22* to the north east of the village was part of the circuit in 1985 but will be omitted in future. Precise reciprocal vertical angles are used to transfer the levels across the tailrace from pillars *B1* to *B4* just upstream of the PH.

2.2 STRUCTURE LEVELLING

The precise levelling of the structure points involves levelling five pins on the dam crest, eighteen pins on the PH floor, six pins on the PH roof, eight pins on the spillway structure, eight pins on the headrace intake structure, seven pins on the tailrace weir (more if flows are very low). Appendix 7 shows a schedule of what BM's and structure points are levelled in the different survey types. The spillway and headrace intake levelling is included in the control levelling routes.

2.3 HORIZONTAL CONTROL

The horizontal control for the network is based on twenty seven survey pillars, and one beaconed trig station (*Trig A No.2*). The seven pillars *D, E, F, G, J, M, N* and *Trig A No.2* form an outer network with numerous interlocking lines. However tree growth within the area since the last full survey of this network of 1985 has meant that many of the lines would require prohibitive clearing and so a minimum requirement of lines between these pillars is shown in App.2. Within this network are other networks of pillars (see section 2.4) that monitor various structures. These are all interlinked with the outer network. Some of these pillars are shown in App.3. By triangula o n techniques the survey pillars are coordinated in a Type A survey from probably *G* and *Trig A No.2* held fixed at previous values on the NZ Map Grid.

2.4 HORIZONTAL COORDINATION OF STRUCTURE POINTS

The following points on structures are coordinated in a Type A survey (see 3.). The five points on the upstream crest of the dam, three points in a line on three of the five downstream piers of the spillway structure, seven points on the headrace intake structure, four points on the PH roof (these are not the levelled points) and up to nine (possibly only three out of the water) on the tailrace weir. From 1975 the regular annual monitoring at Arapuni has been on a survey called the "Ridge Survey" which consists of two near parallel lines (approx. 130m apart running east/west) which run through either end of the PH (the four PH roof points are in this survey) and have their terminal pillars on the right bank above the PH and some 200m west of the spillway. These two lines of marks are called *A1* (east) to *A6* being the northern line and *B1-B6*. All except the four PH roof points (*A&B, 2&3*) are pillars with *A&B (4,5&5A)* being on the ridge between the headrace and the PH. The results of the ridge survey have in the past only been presented as distances and changes between points on each line and offsets and changes from the line between the terminal pillars. The recent horizontal monitoring of the above mentioned structures (except the dam) is either from some of these ridge survey pillars or from additional pillars installed since 1989. The horizontally monitored structure points are coordinated by either directions and in some cases distances from at least one pillar. In the Type A spillway survey the points C2-C4 are set up on and distance measurements made to C2-C5. Appendix 8 shows a schedule of pillar and structure point directions

and distances for the different survey types. The results from the spillway survey are also expressed as distance, offsets and changes between other points on the line and offsets between the terminal pillars. All of the structure points described above are precise levelled except the four PH roof points. The pillars E and A1 have moved in the past and these are referenced to nearby points (*Block E* is adjacent to pillar *E*) with *Pin 57* on the swing bridge anchor being close to A1. The four terminal pillars of the ridge survey have a series of four buried iron tubes close by and on line to two of the other terminal pillars. They were set up around 1975 to check on possible movements of the pillars. It is not anticipated that these points will be surveyed again and so there is no further reference to them in this specification.

2.5. OPTICAL PLUMBING OF SHAFTS

Two of the vertical shafts in the ridge area have points in place to check for tilting of the country by optical plumbing methods from the bottom of the shafts The shaft positions are shown on App.2-3. Appendix 9 shows photographs of the optical plummet and the gridded target on the shaft ceiling. Appendix 10 shows details of nearby reference marks to the plumbing points set up in case the plummet mounting points or targets were damaged.

3. SURVEY TYPES

The extent and frequency of the deformation surveys at Arapuni are defined by three survey types. Type A surveys are undertaken every six years and comprise all components. Type B surveys are undertaken annually and are generally restricted to the structures. Type C surveys are intended to be undertaken in an emergency situation (eg. following a significant earthquake) and are designed to do a minimum of field work to detect any significant deformations. Appendix 1 shows a table of all survey components, a history of when they were and proposed to be next surveyed. Following is a detailed description of the extent of each survey type. It should be pointed out that Type C survey components are included in a Type A and B and Type B survey components are included in a Type A survey.

3.1 TYPE A SURVEYS

Type A surveys are undertaken in approximately August every six years (next survey in 1997) and include all the levelling as shown in App.2-7, the full horizontal network as shown in App.2-3 and the structure point coordination described above and shown in App.3-6 and as per the schedule in App.8. The optical plumbing in the two shafts is also undertaken. These Type A surveys will check on deformations over a wider area than the annual Type B surveys and will check on the validity of the assumptions of control mark stability made in the previous five annual surveys.

3.2 TYPE B SURVEYS

Type B surveys are undertaken in approximately August annually. They are designed to provide an annual statement on the integrity of the Arapuni structures from a minimum of field work. Generally the same structure points are levelled and coordinated as in a Type A survey except they are fixed from fewer observations from BM's and pillars that are assumed to be stable (albeit with some checks) at previous values. The levelling is undertaken in six separate unconnected areas viz: the dam, spillway, headrace intake, ODS ridge area, PH floor and tailrace weir. Appendix 7 shows a schedule of what marks are levelled in a Type B survey and the plans in App. 3-6 show the various routes. The PH roof levelling is not done. All of the levelling in these areas commence at adjacent BM's, pillars or pins that are connected during the Type A surveys. In a Type B horizontal survey eight of the twenty seven pillars are occupied. These are pillars *A1, A6, B1, B6, Steps, ODS, C5* and *B*. Both direction and distance observations shall be made between these pillars and to others (where applicable) to provide checks on their stability. From these pillars direction and distance observations are made to all the structure points described in 2.4 above except for two of the four PH roof points. The number of tailrace weir points coordinated will depend on the tailrace flows at the time. Appendix 8 shows the schedule of observations for a Type B survey. Generally the structure points are coordinated by one polar ray (direction and distance) whereas in a Type A survey these points are coordinated by several rays of directions and sometimes distance. the optical plumbing in the two shafts is also done. In the event of a Type B survey revealing significant changes, discussions with the deformation engineer and the client (Electricorp) must be held to decide what extra survey work is required and the funding of that work.

3.3 TYPE C SURVEYS

Type C surveys are intended to be undertaken as a result of an event that may have caused significant damage to the structure. The survey is defined as being the same as a Type B but would concentrate on areas of concern. The levelling shall use precise equipment but the procedure may be relaxed depending on the nature of the emergency eg. levelling one way only in more instances. As many checks between the observing pillars (and others) as possible shall be made in the available time to check on their relative stability. The procedure for a Type C survey needs to be flexible depending on the nature of the emergency and the consultant surveyor would need to respond quickly and have some interim results to the client within twelve hours of notification. It is probable that a Type A or B survey would follow some time after the emergency.

4. INSTRUMENTATION

The deformation surveys are of a precise nature and the instruments employed must be of a suitable precision and be correctly adjusted and

calibrated to achieve the necessary accuracy. The precise level must be an automatic type capable of reading to 0.1mm or better and used in conjunction with two precise invar staves. A Zeiss Ni007 is a typical instrument. The theodolite used for horizontal and vertical angles must be capable of reading to better than 1" of arc. A Wild T3 or T2000S are typical instruments. The EDM used for distances should be capable of measuring distances of up to 1km (Type A surveys) with reading to 1mm or better and standard errors of +/-3mm +/- 3ppm (parts per million) or better. A Wild DI20 and Sokkisha SET2 are typical instruments. The theodolite (s) used for the precise vertical angles (see 5.2) can be a 1" instrument. A typical instrument is a Wild T2 or a Sokkisha SET2. The optical plummet used for checking the shafts should be an automatic type capable of 1:200000 accuracy. A Wild ZL or ZNL are suitable instruments. The suitability of new instruments and techniques for deformation surveys needs to be continually assessed for possible accuracy improvements and efficiency gains. However if instruments outside the above specification are intended to be used they must be fully specified and approved by the client.

5. FIELD METHODS

Prior to commencement of field work the consultant surveyor shall discuss his intended on site activities with the Arapuni operating staff. Some clearing of pillar sight lines will be required especially prior to the Type A surveys. This may be a major exercise and some prior investigation and feasibility on what lines shall be cleared shall be undertaken. An indication of what may be required is shown on the plan in App.2 and on the horizontal observation schedule in App.8. Approval to enter private land for the outlying pillars and trigs shall be sought before entry. Appendix 14 includes a full set of location diagrams for finding the BM's and pillars. The field work must be observed in suitable conditions with avoidance of significant shimmer being very important for angle measurement. Weather conditions, (the dam, spillway, headrace and tailrace water levels), dates, personnel, instrument types and serial numbers must be recorded. The following field procedures will achieve the required accuracies specified in 6. below. Any significant departure from these procedures must be fully detailed and approved by the client before being used.

5.1 LEVELLING

Before and after the precise levelling a collimation check on the level shall be undertaken. Either of the two staves can be used to monitor points but the constant difference between them must be determined and applied if necessary. The stave bubbles must be checked. Except on lines under 5m length the bottom 0.5m of the stave shall not be observed. Sight lengths shall not exceed 35m with set out distances being taped or carefully paced. The difference between the back and fore sight lengths in any set up must not differ by more than 1.0m and the accumulated total of back and foresight distances between BM's must not differ by more than 2.0m. An observing

procedure whereby a centre wire reading on each of the two scales is read with the observing sequence always starting and finishing on the back sight is a minimum requirement. All levelling must be returned, (or double levelled) preferably not immediately. The exceptions to this are shown in App. 2-4,6. In a Type A survey these are from pillars A to J, pillar C5 to BM B24 (near the dam) and from H592 to H593. The circuits including these lines must however close to 3mm times the square root circuit distance in km. In a Type B survey the circuit on the spillway (App.6) may be levelled one way (closure to 0.6mm) along with the PH floor levelling (App.4) circuits (closure 0.6mm). The levelling across the tailrace weir between 90WL2-90WR2 and 90WL4-90WR4 is done by reciprocal precise levelling as is the line across the upstream end of the headrace between Nail 89A and pillar C1. Parts or all of a Type C survey may be levelled one way depending on the nature of the emergency. Suitable change points for the control levelling are 150*8mm square section nails driven into the seal. On grass or soft ground substantial firmly driven pegs with domed tacks are necessary. Heavy three footed change plates can be used on concrete surfaces. A field calculator/computer system of recording, computing and printing the height differences and checks is preferred.

5.2 HORIZONTAL AND VERTICAL ANGLES

Horizontal angle measurement is undertaken by observing sets of directions to a number of consecutive stations. Directions to pillars and structure points can be observed together if desired. All observed points have constrained centring so no tripods are necessary. The exception to this is the Type A survey check to Block E and Pin 57 (for pillar E and A1 movement) which are ground marks and need tripods. The trig beacon at A No.2 must be checked for a Type A survey. Particular care must be taken with the instrument levelling on steep lines. An umbrella shall be used to shield the instruments in strong sunlight. For observations to pillars and structure points at least 4 sets of directions shall be observed in a Type A survey. For lines of around 100m or shorter 3 sets is sufficient. For a Type B survey 3 sets of directions is sufficient. If directions are observed on more than one occasion from a pillar at least two common points must be observed on each occasion. The pillars that are not precise levelled need to have their heights checked in each Type A survey by trigonometric levelling (heights are used for EDM slope correction and detecting gross deformations). Vertical angles need to be reciprocal but not simultaneous. Three sets of verticals are a minimum. The levelling across the tailrace to A1, B1 etc. shall be checked by a method of precise simultaneous reciprocal vertical angles in a Type A survey. This is done between eccentric set ups at B1 and B4 on this approx. 120m measured line. Vertical angle observations to the centre of the theodolites telescope are normally done. A method of observing a precise stave to determine the theodolite height is recommended.

5.3 DISTANCE MEASURING

Lines between pillars in a Type A survey shall be measured both ways although the same way on separate occasions is acceptable. In Type B and C surveys lines can be measured one way. The number of measurements taken at a station depends on how the EDM operates but a minimum of S separate measures of the distance is required. Sufficient meteorological data (temperature and pressures) needs to be taken to ensure that the atmospheric correction can be computed to 1ppm or better. Meteorological equipment needs to be periodically checked against a standard. For lines over 10 degrees slope, tilting reflector assemblies are needed. The new pillars (after 1989) C1, C5, ODS, *Headrace, Pylon, Steps, Wall* and *Bush* all have flat stainless steel plates with a protruding 5/S"UNC stud imbedded in the pillar top. For these, EDM heights are measured from the flat plate. There is a known relationship between the top of stud, the flat plate and the base BM pin (where applicable) for these pillars. All other pillars have a 1"BSP threaded stud with a 3/S" centre hole which require a pillar plate or pillar pin to mount a tribrach. These items are shown in App.12 which shows special survey items. Usually at Arapuni a set of ten numbered pillar plates (for the A and B pillar lines) are used. EDM heights are measured from the top of these plates and there is a known relationship between the top of the screwed in plate, the top of the imbedded 1" pin and the base BM pin where applicable. The structure monitoring points that are measured to (except the PH roof and dam points) have a 5/S"UNC protruding stud with a Ramset nail some S0mm adjacent for the purpose of EDM height measurements. There is a known height relationship between these points. The EDM must have a recent calibration on a multi pillar base line to determine the constant and scale error of the instrument preferably in similar weather conditions. It is recommended that the WORKS Dey St. EDM test base in Hamilton is used for this purpose. The summation of measurement on the A and B lines and on the spillway survey (C2-C5) also serve as a check on the EDM constant.

5.4. OPTICAL PLUMBING OF SHAFTS

The optical plumbing in the two shafts is undertaken in a Type A and B survey. The most successful method for reading the gridded target in the shaft ceiling has been for an assistant to read the grid reference (to 1mm) after having moved a circular target (a washer is suitable) into position on the illuminated target with directions from the observer below using radio communication. At least four readings are made (in groups of two) in plummet positions 1S0 degrees apart.

5.5 OTHER NOTES FOR FIELD WORK

The PH roof points and dam points need a special adaptor to mount a tribrach. These are called a "Tribrach Stud Short or Long Adaptor" shown in App.12. The levelling on the tailrace weir right bank requires 2m staves or similar as access to the right bank through the tunnel is very restricted. The levelling on the PH roof (Type A survey) has not been done under full

generating conditions before. Excessive vibration may mean that this levelling is not possible. Keys and approval from the operating staff may be necessary for access to the tailrace weir, the PH roof and floor, the ODS, the headrace intake and the spillway. The screen cleaner on the headrace intake may need to be shifted to see to 90H3-5 from pillars ODS and *Headrace*. On the tailrace weir point 90WL3 a special item called a "Stud Extender" (see App.12) is necessary to clear part of the adjacent gate raising mechanism. A gaurd rail has to be removed to attach a tribrach to 90WL2. For the optical plumbing in the shafts detachable water shields are located at each site. These are positioned above the instrument allowing observations through a small hole. Excessive water falling in Shaft C may prevent any observations here. Prior to the survey discussions with the Arapuni operating staff should be held to ascertain the prevailing water conditions in Shaft C.

6. REDUCTION AND ACCURACY OF DATA

6.1 LEVELLING

The field recording/computing system will provide all the procedural checks and height difference summaries in the field. The difference between the two derived height differences in any instrument set-up must not exceed 0.35mm or the set up must be repeated. The instrument collimation error must be kept to within 1mm/25m. The forward and return runs shall be set up in a spreadsheet for comparison and final level computation. The comparison between the mean forward and return difference in height between adjacent pins on a structure must not exceed 0.4mm. The difference between the forward and return runs between adjacent BM's and the misclose in the large Type A circuits must be less than 3mm times the square root of the km distance between BM's or circuit length. The misclose around the PH and spillway circuits must be 0.6mm or less.

6.2 HORIZONTAL AND VERTICAL ANGLES

A field system of recording, computing and printing the individual sets of directions in terms of the selected origin is desirable. The Standard Error (SE) for any individual direction on lines over 100m from the 3-4 sets shall not exceed 2.5" with the mean SE of all such directions at one station not exceeding 2.0". The mean of all SE's in the survey shall not exceed 1.5". For vertical angles an SE of 3" for the mean angle is required. The computation of the *A&B 1&6* quadrilateral horizontal angle misclose should be done and must be less than 3". The precise vertical angle connection between B1 and B4 shall close with the precise levelling to 3mm times the square root distance in km (circuit length) or better.

6.3 DISTANCES

A field system of recording, detecting gross errors and printing the measured data is desirable. The field data then needs to be corrected for the

atmospheric conditions, slope, elevation above sea level, instrumental constant and scale error to produce spheroidal distances. Slope corrections are to be computed from adopted or derived station heights and not from one way vertical angles. In a Type A survey the forward and return distances are best input into a spreadsheet where the differences and means can be computed. The mean difference between the two measurements for lines between pillars must be less than 3mm with no individual lines over 6mm. The final mean corrected distance before adjustment needs to be accurate to 3mm or better in a Type A survey.

6.4. OPTICAL PLUMBING

The optical plumbing simply requires the mean of the readings at each site to be computed.

7. COMPUTATION AND ADJUSTMENT OF DATA

The following are acceptable techniques for computing and adjusting the data to achieve the required results. Other techniques may be permissible but must be fully documented and approved by the client before their use.

7.1 LEVELLING

The computation of the control BM levels involves selecting a BM as a network origin. This decision will be based on the agreement with a nearby check BM, and what changes are apparent on the other network BM's compared with previous surveys. The most likely origin in a Type A survey are *Arapuni Fundamental* or Pillar G being on the extremities of the network. Orthometric corrections are applied in the computations if their effect is greater than 0.05mm. In a Type A survey the structure point levelling shall be in terms of the one selected origin for that survey. In a Type B survey the origin mark used adjacent to the structure levelled (as shown in the plans in App.3-6) shall be held fixed at its previous Type A survey level unless evidence shows it is unreliable. The levelling in a Type A survey comprises many circuits. The hierarchy of circuit adjustment shall be that the smallest circuits are adjusted first with no further adjustment in that circuit if it is linked to a larger circuit. The levelling to the dam area should be computed using the return line (*H593-B*) with the two one way lines serving only as a check. The circuit comprising the precise VA's and the return levelling from B1 to H592 should be adjusted based on distances between marks between B1 and H592.

7.2 HORIZONTAL CONTROL

The mean observed directions and reduced distances of the network (excluding observations to structure deformation points) contain many redundancies. This data must be adjusted by a suitable least squares computer program that will allow at least a two dimensional adjustment to produce final coordinates of the points. The software needs to handle the

input of directions and distances either in spheroidal format or distances corrected only for instrumental and atmospheric factors. In the latter case EDM and station heights are supplied. The software must be capable of weighting observations separately and must produce observation residuals, coordinate SE's and error ellipse details at the 95% confidence levels. It is likely that several different adjustments would be undertaken to suitably weight or reject inferior observations, to produce minimum residuals and to try different fixed points in the adjustment. One adjustment should be undertaken with only one fixed point and an orientation (a free net) to assess the quality of the survey data. The final selection of fixed points in the network adjustment may be done empirically based on the experience and knowledge of the network and all the evidence from the current and previous survey data. In a Type A survey it would be anticipated that initially G would be used as the origin of coordinates with Trig A No.2 (and probably pillar F) for orientation in the adjustment. It is anticipated that for a Type B survey pillar B6 would be used as the origin of coordinates with orientation checks to G, F, A6 and Trig A No.2 (if visible) all held fixed at the previous Type A values with new coordinates being computed for five other occupied pillars C5, ODS, A1, B1 and Steps. A Type C survey may not involve any least squares adjustment but may only involve a limited check on the stability of the occupied pillars. The adjustment software used must be capable of applying an overall scale correction and desirably will be able to perform similarity transformations of one survey over another to obtain a best mean fit to assist in the selection of fixed (stable) points.

7.3 STRUCTURE POINT COORDINATION

Once the coordinates of the pillars used for the structure point observations have been decided on another least squares adjustment is undertaken with all the mean observed directions to the structure points input. Again this may involve several adjustments to suitably weight or reject inferior observations. In a Type B survey pillar B is held fixed at its previous Type A values along with A and F. In the computation of some of the structure points in a Type B there are no redundancies. In a Type C survey the coordinates of the structure points could be computed manually. The results of the survey on points A&B (2-5A) are also presented as distances between points and offsets from the terminal pillar line (and as changes since the previous and base survey). The adjusted distances and directions shall be used to compute this data.

S. ANALYSIS AND PRESENTATION OF DATA

The final levels and coordinates from the survey need to be added to the spreadsheet of all previous surveys and the changes shown to 0.1mm for levels and 1mm for coordinates and offsets since the previous and the base survey (for that mark). If the previous survey for that mark is not the most recent Arapuni survey then it is to be indicated as such. See App.11 for a sample spreadsheet of coordinates and changes. Changes have to be assessed

as real or within the allowable survey error. Plots of points from different surveys with their standard error ellipses may be necessary to assess this. Scrutiny of previous survey reports will also be necessary. Three dimensional changes (if significant) since the previous and base survey for the monitoring points shall be shown on plan(s) which show sufficient local detail. The correlation between the shaft plumbing and ridge levelling results shall be discussed. Any identifiable trends from the current and previous surveys need to be identified and discussed. If considered appropriate, graphs of movements against time shall be produced. If the survey date is significantly different from August any seasonal effect needs to be considered. The effect of significantly different water levels during the survey shall also be considered.

9. REPORT FORMATS

One of the key elements of the report format is that the latest report will show the final levels and coordinates for all surveys to date. The survey shall be reported on in two formats. A comprehensive report shall cover all aspects of the survey with a full list of appendices. This report is intended to be a full documentation of the work and would normally be retained by the consultant surveyor. However copies shall be provided to Electricorp if requested. At least two copies of this report shall be produced. A brief report will include the same conclusions, summary and recommendations but may omit some detail leading up to these. This report will be for Electricorp and at least four copies shall be produced. Following is a more detailed description of the contents of these reports.

9.1 COMPREHENSIVE REPORT

a)   The report shall begin with a Title Page followed by a table of Contents then by a list of Appendices.

b)   This will be followed by an Abstract which will include a brief introduction, what survey was undertaken and a brief summary of the findings of the survey.

c)   The Introduction shall include a brief history of Arapuni surveys, the purpose and scope (ie. survey type) and timing of the work and what organisation undertook the survey.

d)   The Field Work section will detail dates, personnel, instruments, weather, what methods were employed and any problems encountered.

e)   Reduction of Data will detail what computations were applied to the raw data before adjustment.

f)   Adjustment of Data will detail how the data was adjusted, what assumptions were made including a discussion on origin selection and

may include a discussion on aspects of the least squares adjustments including the observation residuals and coordinate standard errors. This section may be broken up into vertical and horizontal survey sections.

g) Analysis of Data shall include a discussion on changes since previous surveys and whether these are real or within the survey tolerance. The effect on the results of any abnormal conditions or circumstances need to be discussed. There shall be conclusions from the results for each different component part of the survey.

h) Recommendations shall confirm or otherwise the next planned survey date and suggest any changes or improvements to the survey.

i) The Summary will include essentially what is covered in the Introduction and briefly discuss the findings and conclusions of the survey and any major recommendations.

The report shall have a full set of appendices as follows:

1) A table showing the history of all survey components to date and the proposed future surveys similar to App.1 of this specification.

2) Plans of the area and networks similar to App.2-6 of this specification sufficient to show the monitoring marks in all areas and three dimensional changes (where applicable).

3) A spreadsheet of the control and structure levelling and changes since the previous and base survey. This shall show the origin(s) for each survey.

4) Any plots or graphs that may be necessary to depict or clarify the levelling results.

5) A spreadsheet of the horizontal control and structure point coordinates and changes since the previous and base survey.This shall show the origin(s) for each survey

6) A spreadsheet of the ridge survey (A and B line) offsets and changes since the previous and base survey.

7) As in 6) above but for the ridge survey distances.

8) Two spreadsheets as in 6) and 7) above but for the spillway marks (C2-C4).

9) Any plots or graphs that may be necessary to depict or clarify the results.

10) A spreadsheet and position plot of the readings from the optical plumbing.

And any other appendices that may be required from time to time to enhance the presentation of data. The following appendices shall only be included in the comprehensive report:

11) A list of key survey personnel along with, instruments, serial numbers and details of EDM calibrations for all surveys since 1978.

12) A list of survey reports since 1975.

13) A schedule of levelling on BM's, pillars and structure points similar to App.7 of this specification.

14) A schedule of observed directions and distances from pillars and to structure points similar to App.S of this specification.

15) A spreadsheet of levelling computations.

16) A plan showing the levelling type and routes for the survey.

17) A summary of EDM (measuring point) levels used for pillars and structure points (that were measured to) and the source of those levels. The constant level differences between the following shall be catalogued: the central pillar pin, plate and base pin for the new pillars, the base pin and the top of the screw on plate for all other pillars and the difference between the EDM Ramset height nail and the structure monitoring points (where applicable).

18) A summary of horizontal angle miscloses on the A,B quadrilateral for all surveys.

19) A summary of the pillar E monitoring data for all surveys.

20) A copy of the input data and the key outputs from the horizontal control least squares adjustment.

21) A copy of the input data and the key outputs from the horizontal structure point least squares adjustment.

22) A copy of the survey mark location diagrams.
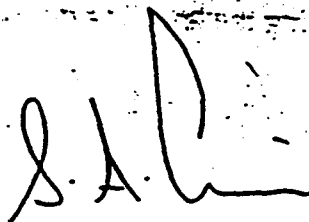
And any other appendices that may be required from time to time to improve the documentation of data. For Type B surveys all of the above appendices shall be included except 19).

9.2 BRIEF REPORT

The brief report shall include a-c) and f-i) in 9.1 above in the text of the report and appendices 1-10) in 9.1 above. A Type C survey would in the first instance be reported on verbally followed by a written brief summary of the changes. As a Type A or B survey would probably follow, the Type C data would be included in the Type A or B report.

10. STORAGE OF DATA

The raw field data and computations from surveys shall be held in safe keeping by the consultant surveyor and be available to Electricorp on request. These records shall be retained until otherwise notified by Electricorp. The computer generated contents of the report shall be electronically filed in two entirely different locations by the consultant surveyor. One of these locations may be with Electricorp. The text of the report shall be filed in the Word Processor format used as well as in ASCII (TXT) format. The relevant appendices shall be filed in the spreadsheet format used as well as in Lotus spreadsheet format. All files must be accessible by MS-DOS computers. Electricorp shall be supplied with copies of the data on disc on request.

S.A CURRIE
Survey Manager
WORKS Geothermal
November 1991

MEASURING INSTALLATIONS FOR DAM MONITORING (SWISS NATIONAL COMMITTEE ON LARGE DAMS)

Measuring Installations for Dam Monitoring
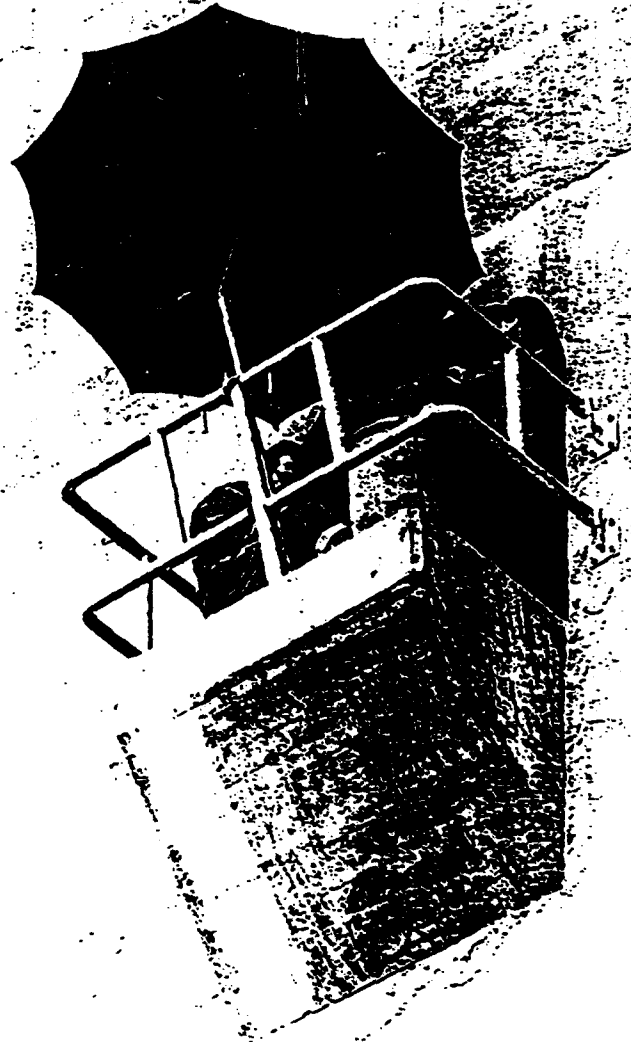
Concepts · Reliability · Redundancy

Preface

Among several hundreds of dams Switzerland has 161 large dams, most of them constructed in the second half of the 20th century. From the beginning of construction of dams in Switzerland, the engineers were at any time conscious of their responsibility. Consequently, importance was attached to dam safety and surveillance of dam behaviour. The first time a dam was provided with more extensive monitoring equipment was during the construction of the 55 m high Montsalvens arch dam in 1920, when instruments for geodetic survey, clinometer and temperature measurements were installed. In 1932 the first plumb-line measurements were performed at the 114 m high Spitallamm arch dam.

In course of time a wide range of measuring instruments was developed in order to meet all requirements of purpose, precision, reliability and longevity. Today specialized manufacturers offer a widespread collection of equipment for dam monitoring. Close cooperation between consulting engineers, dam owners and manufacturers helps to continue the development of suitable instruments to assure dam safety.

The Subcommittee on Dam Monitoring of the Swiss National Committee on Large Dams is to be thanked for the preparation of the present paper which marks the purpose and goals of the installation of dam monitoring equipment, and we hope it will be of value to everybody engaged in dam engineering.

Our particular gratitude goes to "wasser, energie, luft – eau, énergie, air", the official periodical of our national Committee, which made it possible to print this special issue edited in honour of the participants at the 16th ICOLD-Congress in San Francisco, June 1988.

The Swiss National Committee on Large Dams

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Editor

Measuring Installations for Dam Monitoring

Concepts, Reliability, Redundancy

Part 1: Concepts

1. Introduction

Any dam is subjected to external loads that cause deformation and permeability of the structure and its foundations. Deformation and seepage are clearly a function of such loads. Any sign of abnormal dam behaviour could possibly threaten dam safety. Loads and the dam's response to them should therefore be carefully monitored for any sign of abnormality as early as possible, and action promptly taken before that abnormality becomes a threat to safety. Monitoring consists of both measurements and visual inspections, neither being sufficient on their own. Every dam should be equipped with appropriate instrumentation according to the set goals and according to dam type and size as well as to particular site conditions.

As experience in dam engineering grows constantly and measurement techniques become more sophisticated, dam instrumentation should be regularly checked for suitability. It may need to be supplemented or even refurbished altogether.

When designing a dam monitoring system or evaluating an existing one, the following should be borne in mind:

- a dam and its foundations form a whole that is embedded in the surrounding terrain, which may also have an impact;
- an abnormality in dam behaviour may occur either quite rapidly or only gradually;
- in the event of some abnormality, its cause should be identifiable through analysis of the measurement data.

Thus, a monitoring system should be capable of monitoring both short- and longterm behaviour. It should also be capable of distinguishing between behaviour of the dam and that of its foundations and surrounding terrain. To assess behaviour in the short term, analysing relatively few data is usually sufficient, provided that the data are selected in such a manner that they clearly show whether behaviour is normal or not. These main parameters must therefore be checked quite frequently. This will be helped by instruments that are simple to operate, combined with measurement methods producing results that can be easily interpreted. For the collection and analysis of data on long-term behaviour, including analysis of some detected abnormality, more differentiated measurement methods should be used. Since monitoring in such cases will be less frequent, measurement procedures and interpretation of results may require more sophisticated instrumentation and analysis methods or even the intervention of specialists.

Monitoring data should be available at all times with the required reliability. Any malfunction or interruption in the monitoring may jeopardize dam surveillance as a whole, and raise doubts about safety. Even if defective installations and instruments are replaced immediately, there is some danger of the fixed datum being altered. Continuity of the data series will be lost and long-term analysis made more difficult. Therefore, preference should be given to measuring installations and instruments which are robust and have a long service life. A reduced accuracy is less serious than an interruption in instrument operation. Maintenance and accuracy checks at regular intervals are therefore essential to ensure reliability and longevity.

The measuring range should be extensive enough, so that excessive load cases and/or abnormal behaviour may be recorded, if possible, without restraint. It is precisely such cases that might threaten dam safety and need appropriate countermeasures.

Accuracy of the monitoring data should not be beyond the scope of existing facilities for analysis and interpretation. It is important to remember that knowledge about the theoretical principles affecting dam behaviour is to a certain extent still approximative, and that such principles may be influenced by phenomena (e.g. the impact of seasonal climatic variations on the behaviour of the foundation) that are not quantifiable, for the time being at any rate.

Even if sturdy and easy to operate instruments have been installed, defects and failures cannot be ruled out. Thus, any monitoring system should be sufficiently redundant.

By redundancy we mean keeping parallel but separate sets of instruments and, in addition, facilities for evaluating data by double-checking, using alternative measurement methods (e.g. plumbline-traverses, alignment-triangulation, and settlement gauge-levelling). Of the two, double-checking is preferable, because it invariably provides other helpful data. Its usefulness is, therefore, twofold.

2. External loads

Dam deformation is caused only in part by the reservoir level. Other factors come into play, such as temperatures in concrete dams, and the self-weight of the fill in embankment dams. Earth pressure may also be a decisive factor. In either type of dam, seepage rates depend essentially on the reservoir level and to a lesser degree on precipitations or melting snow. A monitoring system should, therefore, include regular measurements of the reservoir level and of representative temperature and precipitation data.

Generally, *reservoir levels* are today measured with pressure balances. Double-checking is essential. This can be done for instance by installing a manometer on either an existing or new pipe connected to the reservoir. The measuring range should extend at least as far as the dam crest, because it is important to know extreme values of the water level, for an immediate judgement of flood risk as well as the subsequent assessment of peak inflows.

Temperature measurement is required to determine the impact of temperature variations on concrete dam deformation on the one hand, and on the other, to determine whether precipitations consist of rain or snow, or whether the snow-melting period has begun (which the ambient air temperature at the dam site will indicate). Where daily temperature readings cannot be guaranteed, it is recommended to install a temperature recorder or, at least, a thermometer indicating maximum and minimum values. Here, a double check is not strictly necessary, since other temperature measurement methods can always be used in case of failure. Assessing the impact of temperature on concrete dam deformation will be helped by installing a sufficient number of thermometers at various locations within the dam. Use may then be made either of temperature gauges embedded in the concrete, or thermometers inserted in drillholes. Redundancy would consist of a greater number of gauges than strictly necessary.

Precipitation gauging, essential near any embankment dam, is recommended practice for concrete dams as well. Daily readings are adequate. Although the gauge need not

be located right at the dam site, it should not be too far removed from it. If daily readings cannot be guaranteed, a rainfall recorder may be used. A redundancy is not essential in this instance, since data can always be obtained from other gauging stations further away.

Earth-pressure-gauging, to determine the overall stress on critical structural elements, may be useful in particular cases of embankment dams or debris check dams. Interpretation of results is, however, problematical.

3. Deformation

Dam deformation patterns vary according to type of dam, foundation conditions and external loads. Due to the differences in construction materials the behaviour of concrete dams is completely different from that of embankment dams. In concrete dams, deformation is mainly elastic, depending on reservoir water pressure and temperature variations. Permanent deformation may, however, be caused by the subsoil adapting to the new loads, concrete aging or foundation rock fatigue. In such cases, deformation is without danger for so long as it does not exceed some critical value. The case of earth dams is altogether different. Deformation here is to a large extent permanent. Under the impact of the self-weight of the embankment and the hydrostatic pressure of the reservoir water, the fill material (and the foundation if consisting of soil) continues to settle – albeit at a decreasing rate – for decades after construction. In addition, permanent horizontal deformation of the embankment are due to reservoir water pressure and are mainly perpendicular to the embankment centerline. Actual elastic deformation is slight, and not typical of earth dam behaviour.

In view of the difference between concrete and embankment dam behaviour, a monitoring programme cannot be organized in the same way for both types. In concrete dams, monitoring is essentially a matter of observing behavioural trends in both elastic and plastic deformation. The work consists of comparing effective (i.e. measured) deformation to the predicted normal behaviour, assessed through analysis or some other method. In embankment dams on the other hand, permanent deformation trends should be closely monitored for any sign of abnormality. Deformation values vary considerably according to type of dam: they are expressed in millimeters and centimeters in concrete dams, and centimeters or decimeters in the case of embankment dams.

Deformation of a dam and its foundations may be determined by measuring the *spatial displacement of selected control points* from reference points, themselves controlled in position. If the reference points are located inside the dam, only relative deformation values will be recorded. Absolute displacement values are obtained if the reference points are located outside the dam (in the foundation or surrounding terrain) and beyond the region that may be affected by the dam or the reservoir. Although relative data are adequate for routine monitoring, assessing permanent deformation requires absolute data, so that a monitoring system confined to the interior of a dam would be inadequate in this case. Ideally, in concrete dams, the reference points should be located in the rock foundation at a depth unaffected by the reservoir. In that event, absolute data could be obtained by frequent use of simple measurement devices. Fixed reference points located in the vicinity of the dam but outside the range of its impact are, moreover, essential to identify behavioural trends in the surroundings of the dam. Thus, monitoring arrangements in the dam plane should be

supplemented by and connected to a vast triangulation network and levellings. Thus, monitoring dam deformation according to set goals requires a spatial, i.e. *three-dimensional measuring installation.*

The monitoring of dam or foundation deformation will be helped if *displacements are measured at points along both horizontal and vertical lines* (measurement along lines) extending as far as possible into the foundation and including it. Redundancy – essential in this case – is then achieved by measuring the displacements at the points intersecting the orthogonal lines of this network, using various methods.

If a dam includes inspection galleries and shafts, deformation values along vertical lines can be obtained by using plumb-lines (both hanging and inverted) and along horizontal lines by traverses, both methods being standard practice. Where there are neither galleries nor shafts (as in embankment, thin arch and small gravity dams), the same result can be achieved by an orthogonal network of survey targets on the downstream face. These targets are sighted by angle measurement (combined with optical distance measurements if required) from reference points outside the dam. The geodetic measurement of deformation does have one disadvantage, however. Because it is costly and can be performed by specialists only, it cannot be performed frequently. In routine but more frequent monitoring – of short-term behaviour – the work may be confined to observing trends at selected points (usually, along the crest but occasionally along vertical lines) by simple angle measurement or by an alignment supplementing the measuring installation. The settlement of an embankment dam can easily be monitored by levelling of the crest. Here, redundancy is not essential since levelling can easily be repeated. However, it is essential to extend levelling to some distance beyond the abutments. Alternative measuring methods to assess deformation of an embankment should include settlement gauges, hose levelling devices or extensometers.

Measuring lines may be extended into the foundation by inverted plumb-lines and extensometers (preferably multi-rod extensometers aimed in 2, preferably 3 directions to detect spatial displacements). In some cases use may also be made of a sliding micrometer, preferably incorporating an inclinometer, to determine not only strains but also inclinations. Where there are exploratory and drainage galleries, traverses may be extended into the abutments. Redundancy may be dispensed with if plenty of foundation and abutment-gauging instruments are available.

4. Seepage

Every reservoir entails seepage through the dam structure and its foundation, even if there is a grout curtain. In concrete dams, seepage is usually slight and confined to permeable areas of the concrete, as well as along joints and at the contact between rock and concrete. But any unusual rise in the seepage rate is a danger warning. Seepage flows cause uplift pressure, which should be carefully monitored in concrete dams in view of its considerable impact on stability. In embankment dams, seepage flow through the embankment is similar to that in the foundation, since construction materials (including those used for the impervious core, if any) are more or less pervious. Seepage through and underneath the embankment causes pore-water and uplift pressure, which has a crucial impact on stability. Seepage should therefore be carefully monitored, because any abnormal rate may indicate a development within the embankment or the foundation that may be a serious threat to safety.

The *total seepage rate*, in either type of dam, will indicate whether seepage as a whole may be considered normal. Gauging may be either volumetric (using a calibrated container and a stopwatch) or by using a gauging weir or flume. Since both methods are simple and reliable, redundancy is not necessary. As far as possible, however, *partial seepage rates* (i.e. those occurring in selected, isolated zones) should be monitored. If an abnormality is detected, the critical zone will be all the more easy to identify, as will the search for the cause of the seepage.

In embankment dams constructed from soluble or easily erodable materials – or founded on such materials – it is recommended to monitor *turbidity*, at least at regular intervals if not constantly. This should be followed by periodic *chemical analysis* of the seepage water. In this way useful data can be obtained for assessing the stability of the embankment and foundation materials, and of the grout curtain in particular.

The pattern of seepage and *pore-water pressures* – especially in the foundation and the impervious core – has a significant impact on the normal behaviour of embankment dams. Since pore-water pressures should not exceed design values, they must be carefully monitored, possibly by pressure cells. The greater the number of measurement profiles and number of cells per profile, the more useful the data obtained will be. This method, redundant in itself, is highly advisable in view of the considerable failure rate of pressure cells.

Although recent experience shows that the installation of pressure cells, even in existing embankment-type dams, is possible, refurbishing is not always feasible. In such cases seepage will have to be monitored by gauging the *phreatic line* in selected points. This may be done by using standpipe piezometers, in which the piezometric level is checked (e.g. by a light or acoustic gauge). Since gauging the phreatic line provides a redundancy with regard to pore-water pressure measurements, and its evolution is an important behavioural indicator, standpipe piezometers should be standard equipment in any embankment dam and should be installed in several cross sections.

Seepage underneath a concrete dam causes *uplift pressures* that act against the – stabilizing – effect of the dam's self-weight. Although such pressures are usually controlled by a grout curtain and also, in some cases, by drainage holes, the effectiveness of both should be checked, and uplift pressures carefully monitored, except in cases where the dam would continue to be safe even under the most extreme uplift pressures. Since foundation conditions are heterogeneous (fracturing), uplift pressures should be measured in as many cross sections as possible and at several points between the upstream and downstream face, to monitor the decrease in uplift pressures. Although measurements taken at the rock-concrete contact are usually adequate, it may be advisable, in exceptional cases, to gauge the pressure at various depths. To measure uplift pressure at the rock-concrete interface, piezometers connected to a manometer have proved to be highly reliable, accurate and robust. Since seepage rates are frequently low despite high pressure levels, true pressure readings may not be obtained for quite some time (several days or months). To avoid incorrect gauging, the piezometers should be kept under constant pressure. Readings can be distorted or interrupted, either by clogging of the piezometer intake or the connecting pipe, or by some defect in the pressure transducer. Thus, a drop in the pressure reading should never be viewed with too much optimism. Pressure gauging deep inside the foundation may be performed using pressure cells and standpipe piezometers connected to a manometer. However, since failure in uplift pressure-gauging instruments cannot be ruled out, redundancy is essential. This can be achieved by installing a larger number of measuring devices than is strictly necessary, and/or by a double set of instruments at each observation point.

In cases where the foundation is being drained, *drainage water discharge* should be monitored. Any fall in the flow rate may indicate clogging in the drainage system. Gauging may be either volumetric or by using a gauging weir, both methods being reliable enough not to require redundancy.

The discharge of any *spring* located downstream of the dam should be carefully monitored. Any variation in the discharge may indicate some abnormality in the seepage. But here again, volumetric and/or gauging weir methods are sufficiently reliable not to require redundancy.

Part 2: Measuring installations and methods

Explanations to the tables

Column 1: Purpose

This column indicates the measurement data required, grouped by loads and reactions (indicators of concrete and embankment dam behaviour).

Column 2: Measuring installation, measuring devices, measuring methods

The most suitable and commonly used instruments/ methods to obtain the required data are listed in this column.

Column 3: Requirements

The requirements to be fulfilled by the instruments/methods are defined as follows:

R — high *reliability* is required for the data which are indispensable for the proper monitoring of a dam and which must be available at all times.

L — for important data it is necessary to use – together with sufficient redundancies – *long-lived* measuring equipment, whenever its refurbishing, the replacement of parts or the establishment of relations to previous measurements are exceptionally time-consuming or impossible.

M — *measuring ranges* must be wide enough to cover exceptional loads or unexpected behaviour.

P — the required *precision* must cover all errors of the complete measuring installation and procedure (inaccuracy of the instruments and their centering as well as effects of temperature, embedding materials, frictions, wear, zero-point deviations, non-linearities etc.).

Redundancy means both the (independent) duplication of a measuring device or the possibility to check or reconstruct a measurement by means of another measuring installation.

Column 4: Remarks

This column includes important indications and details or characteristics of measured data and equipment.

A) Loads

Purpose	Measuring installation — Measuring devices — Measuring methods	Requirements — R = Reliability — L = Longevity — M = Measuring range — P = Precision	Remarks
Water level			
Hydraulic loads	Pressure balance, Float, Staff gauge, Manometer, Light gauge, Acoustic gauge	R: very high — L: nil — M: above crest level — P: ±10 cm — Redundancy: absolutely necessary	Important measurement. Range must also cover the flood levels
Temperatures			
Air and Water External thermal load Influence on snow melt	Thermograph, Continuous recording of temperature variation, Thermometer, Min., max. and instantaneous temperature	R: nil — L: nil — M: −30°C to +40°C — P: ±1°C — Redundancy: desirable; necessary for thermograph	These instruments can easily be replaced
Concrete Internal thermal loads (as they directly influence concrete deformation)	Electrical thermometer, Thermometer placed in boreholes	R: very high — L: very high — M: −10°C to +60°C — P: ±1°C — Redundancy: necessary; provide enough instruments	A measuring range up to +60°C is only necessary during the construction period; when installed later a range up to +30°C is sufficient
Rainfall			
Rainfall in the dam area influence on water percolation	Rain gauge	R: moderate — L: nil — Redundancy: not necessary	Such measurements are not absolutely necessary in the immediate vicinity of the dam
Earth pressure			
Essential structural parts subject to embankment loads	Earth pressure cell	R: moderate — L: high — M: total overburden (0 to 3 N/mm²) — P: ±5% of M — Redundancy: not necessary	The deformation modulus of the instrument must be adjusted to the type of embankment material. Interpretation and results are problematic

8) Indicators of concrete dam behaviour

Purpose	Measuring installation Measuring devices Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Deformation of dam and foundation			
Displacements along vertical lines extending into the rock for comparison with measuring results observed during previous periods, as well as with the assumptions and results of the structural analyses	Plumbline, inverted plumbline; measuring device in two directions, with optical sighting of the plumbline wire; the wire serves as vertical reference axis	R and L: very high M: max. calculated deflection + 50% P: ± 0.2 mm resp. ± 1% of M Redundancy: absolutely necessary by means of: – spare measuring device – calibrating station for the measuring device – combination with triangulation, traverse, alignments and extensometers	– Well-tried and precise device – Short measuring time – Teletransmission possible; measuring device must not influence the plumbline position
	Angular measurements and electro-optical distance measurements from stations located downstream of the dam	Requirements to be determined from case to case Redundancy: absolutely necessary: possible displacements of the measuring stations must be checked periodically by triangulation or inverted plumbline	– Well-tried but exacting measuring methods to be used only where the installation of plumblines is not possible – Measurements require favourable weather conditions – Precision depends upon distance and refraction
Displacements along horizontal lines extending into abutments and valley sides	Wire alignment measuring device in one direction with optical sighting of the wire, which marks a vertical reference plane	R and L: very high M: max. calculated deflection + 50% P: ± 0.2 mm resp. ± 1% of M Redundancy: absolutely necessary by means of: – spare measuring device – calibrating station for measuring device – combination with triangulation, plumblines and extensometers	– Equivalent to plumblines; precision independent from length of wire – Applicable only to straight structures – Max. length limited by quality and weight of the wire
	Levelling	R and L: moderate P: ± 1 mm Redundancy: necessary according to circumstances in combination with triangulation; groups of reference points must be provided on both valley sides	– Well-tried and simple method when modern instruments are used
	Optical alignment	Requirements to be fixed from case to case Redundancy: absolutely necessary in combination with triangulation and plumblines	– Well-tried and simple method otherwise, same remarks as for angular measurements
	Measurements of angles and distances	See same item under "Displacements along vertical lines"	See same item under "Displacements along vertical lines"
	Traverse	Requirements to be fixed from case to case Redundancy: absolutely necessary in combination with triangulation and plumblines	– Very exacting measurement, attachment to triangulation and plumblines is absolutely necessary

B) Indicators of concrete dam behaviour (continuation)

Purpose	Measuring installation / Measuring devices / Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Deformation of dam and foundation (continuation)			
Variations in length and deflections along boreholes global measurements on long stretches or differential measurements along a chain of short stretches	Rod or wire extensometers with one or more rods (wires)	R, L and M: to be fixed from case to case P: ±0.5 mm Redundancy: not always necessary; can be achieved by – installing extensometers in several comparable locations – subdividing the full length in several parts – combination with inverted plumbline or levelling	– Placing of anchors and grouting of the protective sleeves are critical operations – Teletransmission possible
:	Sliding micrometer differential length variations Sliding micrometer with inclinometer differential deflections, partly combined with sliding micrometer Deflectometer global or partial respectively differential deflections (chain of deflectometers)	Requirements to be fixed from case to case Redundancy: not necessary	– Precision highly dependent on the instrument guiding system; certain devices give very accurate and reliable results – Placing and grouting of the guiding sleeves is a critical operation – Recommended for the localization of discontinuities (cracks and/or joints) and to observe their movements – Measurement and interpretation time consuming
Special displacements of individual points of the dam and its surroundings	Triangulation/trilateration combined from case to case with: – traverses and levellings – electro-optical distance measurements – optical plumbing, plumblines – alignments, extensometers	R and L: very high P: (three times the expected mean error) ≤ ±3 mm for measuring stations and important reference points ≤ ±5 mm for other points Redundancy: absolutely necessary by means of: – superabundant measuring points and elements – combination with other measuring installations	– The geodetic survey network must cover a large area to enable the long term observation of the structure as well as of its surrounding area and checking possible displacements of reference points for other measurements (redundancy) – Exacting measurement which can be carried out only at long intervals; requires provision of reduced measurements for rapid appraisals – All data and indications on measuring and evaluation methods to be filed safely
Movements of cracks and joints at accessible places – expansions – sometimes shear movements	Micrometer Deformeter Dilatometer Deflectometer	R and L: according to purpose M: 10 mm P: ±0.05 mm Redundancy: according to purpose	– Measurements in gallery walls or recesses are often not representative for the behaviour of the whole mass – Adequate check-marks can often replace a measuring device – Teletransmission possible
Local rotations in the vertical plan (inclinations)	Clinometer – with a level and a micrometer – with direct display (electronic)	R and L: high M and P: according to purpose Redundancy: this measurement is recommended only if combined with other measuring installations such as plumblines for instance	– Near to cavities results are often influenced by stress concentration and transfer effects – Results may be improved by short chains of measuring stretches
Local specific deformations to check stresses	Electric deformeters embedded in concrete combined with temperature measurements	R and L: high M: – deformations 2 mm/m – temperature –10°C to +50°C P: – stress ±0.2 N/mm² – temperature ±0.2°C Redundancy: necessary by means of – superabundant instruments – other types of instrument for comparison	– Frequent instrument failures – Behaviour often influenced by local material conditions at the instrument site – Analysis of the records problematic

Purpose	Measuring installation Measuring devices Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Seepage through dam and foundation			
Quantity of seepage and drained water (by zones and in total)	Volumetric measurements with calibrated container and stopwatch resp. by volume displacement (for example by means of a calibrated rod in boreholes inclined downwards)	R and L: moderate M: max. expected discharge + 100% P: ±0.05 l/s resp. ±5 to 10% of M Redundancy: not necessary	– Method limited to moderate discharges up to 10 l/s, the container's filling time must be at least 10 s
	Weir, measuring flume sometimes with recorder sonar gauge	R and L: high M: max. expected discharge + 100% P: ±5% of M Redundancy: not necessary	– Deposits must be removed periodically – Not recommended for discharges 0.05 l/s – At the collecting point of the total dam seepage a recorder and an alarm signal should be provided for
	Measurement of flow in pipes e.g. in pipes of drainage water pumps – venturimeter (measurement of the pressure differential) – sonar or magneto-inductive measurement (measurement of the velocity of flow)	R and L: high M: max. expected discharge + 100% P: ±5% of M Redundancy: necessary by means of other measuring devices of additional gauges	– Simple means for a periodical check of the indications must be provided for (manometers, weirs, free flow measuring flume)
Pressure of the water circulating in the foundation (uplift and water pressure in rock joints)	Open borehole/standpipe (piezometer) Gauging of the water level by a cable line with light or acoustic signal	R: nil L: high M: total length of borehole P: ±0.2 m resp. ±1% of M Redundancy: necessary; installation of piezometers in groups	– Borehole cased watertightly down to the pressure measuring area; protection of head of borehole against penetration of surface waters, mud, stones, etc. – Ensure permanent aeration
	Closed borehole Pressure indication by high precision manometer	R and L: high M: total height between manometer and dam crest P: ±0.5 m resp. ±1% of M Redundancy: necessary; installation of piezometers in groups	– Well-tried method – Pipes and connections to manometers must be watertight – Do not relieve pressure artificially, to allow the observation of max. pressures even if they need a long time to build up – Periodical venting of pipes required – Periodical check of manometers absolutely necessary
	Pneumatic, hydraulic or electrical pressure cells installed in individual boreholes or at several levels in the same borehole	R and L: high M: total height between cell and dam crest P: ±0.5 m resp. ±1% of M Redundancy: necessary; provision of a great number of cells or disposition in groups	– Central reading of pressure cells spread over a large area possible – Hydraulic measures possible only if the measuring station lies below the minimum pressure level – Careful selection of the filter type in order to avoid its early clogging – Placing of cells exacting especially if several of them must be installed in the same borehole

Purpose	Measuring installation Measuring devices Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Determination of dam and foundation			
Displacements along vertical lines for comparison with measuring results of previous periods	Angular measurements and electro-optical distance measurements from stations located downstream of the dam	Requirements to be fixed from case to case Redundancy: absolutely necessary; possible displacements of the measuring stations must be checked periodically by means of triangulation	– Well-tried but exacting measuring methods – Measurements require favourable weather conditions – Precision depends upon distance and retraction
Displacements along horizontal lines extending into abutments and valley sides	Wire alignment measuring device in one direction with optical sighting of the wire, which marks a vertical reference plane	R and L: very high M: max. calculated deflection + 50% P: ±1 mm resp. ±1% of M Redundancy: absolutely necessary by means of: – spare measuring device – calibrating station for measuring device – combination with triangulation and extensometers	– Precision independent from length of wire – Applicable only to straight structures – Max. length is limited by quality and weight of the wire
	Levelling	R and L: moderate P: ±1 mm Redundancy: necessary according to circumstances in combination with triangulation; groups of reference points must be provided on both valley sides	– Well-tried and simple method when modern instruments are used
	Optical alignment	Requirements to be fixed from case to case Redundancy: absolutely necessary in combination with triangulation	– Well-tried and simple method. Otherwise, same remarks as for angular measurements
	Measurements of angles and distances	See same item under "Displacements along vertical lines"	See same item under "Displacements along vertical lines"
	Traverse	Requirements to be fixed from case to case Redundancy: absolutely necessary in combination with triangulation	– Very exacting measurement; attachment to the triangulation is absolutely necessary
Settlements due to dead weight and hydraulic loads	Vertical settlement gauge	R and L: high M: 50–100 m P: ±5 cm (construction phase) ±1 cm (operation, after reinstallation) Redundancy: necessary with levelling	– Pipe elements <6 m – Verticality during placing to be checked carefully – Difficulties with inclined systems – Electrical gauges – Combination with pipe-inclinometer possible
	Hose levelling device	R and L: high M: a few meters P: ±1 cm Redundancy: necessary with a settlement gauge and levelling	– Communication tubes with direct reading on the glass standpipe, three tubes per measuring point – Very accurate, somewhat clumsy, sensitive to frost – Deaeration of the measuring fluid necessary

C) Indicators of embankment dam behaviour (continuation)

Purpose	Measuring installation Measuring devices Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Deformation of dam and foundation (continuation)			
Settlements and displacements along lines global measurements on long stretches or differential measurements along chains of short stretches	Rod or wire extensometer with one or more rods (wires)	R, L, M to be fixed from case to case P: ± 1 mm Redundancy: not always necessary, can be achieved by − installing instruments in several comparable locations − subdividing the full length in several parts − combination with alignment or levelling	− Placing of anchors and grouting of the protective sleeves are exacting operations − Teletransmission possible
:	Pipe-inclinometer, deflectometer global or partial, respectively differential deflections (chain of deflectometers)	Requirements to be fixed from case to case Redundancy: not necessary	− Electrical plumbline probe in standpipe with guide grooves − Precision highly dependent upon guiding system − Placing and grouting of the guiding sleeves are exacting operations − Recommended for the localization of discontinuities (cracks and/or joints) and to observe their movements − Measurement and interpretation time consuming
Special displacement of individual points of the dam and its surroundings	Triangulation/trilateration combined from case to case with: − traverses and levellings − electro-optical distance measurements − optical plumbing − alignment, extensometers − vertical settlement gauges	R and L: very high P: (three times the expected mean error) ≤ ± 5 mm for measuring stations and important measuring points ≤ ±10 mm for other points Redundancy: absolutely necessary by means of: − superabundant measuring points and elements − combination with other measuring installations	− The geodetic survey network must cover a large area to enable the long term observation of the structure as well as of its surrounding area and checking possible displacements of reference points for other measurements (redundancy) − Exacting measurement which can be carried out only at long intervals; requires provision of reduced measurements for rapid appraisals − All data and indications on measuring and evaluation methods to be fixed safely
Movements of cracks and joints at accessible places − expansions − sometimes shear movements	Micrometer Deformeter Dilatometer Deflectometer	R and L: according to purpose M: 10 mm P: ±0.05 mm Redundancy: according to purpose	− Measurements in gallery walls or recesses are often not representative for the behaviour of the whole mass − Adequate check-marks can often replace a measuring device − Teletransmission possible

Purpose	Measuring installation / Measuring devices / Measuring methods	Requirements R = Reliability L = Longevity M = Measuring range P = Precision	Remarks
Seepage through dam and foundation			
Quantity of seepage and drained water (by zones and in total)	Volumetric measurements with calibrated container and stopwatch resp. by volume displacement (for example by means of a calibrated rod in boreholes inclined downwards)	R and L: moderate M: max. expected discharge + 100% P: ±0.05 l/s resp. ±5 to 10% of M Redundancy: not necessary	– Method limited to moderate discharges up to 10 l/s, the container's filling time must be at least 10 s
	Weir, measuring flume sometimes with recorder Sonar gauge	R and L: high M: max. expected discharge + 100% P: ±5% of M Redundancy: not necessary	– Deposits must be removed periodically – Not recommended for discharges <0.05 l/s – At the collecting point of the total dam seepage a recorder and alarm signal should be provided for
	Measurements of flow in pipes e.g. in pipes of drainage water pumps – venturimeter (measurement of the pressure differential) – sonar or magneto-inductive measurement (measurement of the velocity of flow)	R and L: high M: max. expected discharge + 100% P: ±5% of M Redundancy: necessary by means of other measuring devices or additional gauges	– Simple means for a periodical check of the indications must be provided for (manometers, weirs, free flow measuring flume)
Pressure of the water circulating in the dam (core and shells) and in the foundation (uplift and pore-water pressure)	Open borehole/standpipe (piezometer) Gauging of the water level by a cable line with light or acoustic signal	R: nil L: high M: total length of borehole P: ±0.2 m resp. ±1% of M Redundancy: necessary; installation of piezometers in groups	– Borehole cased watertightly down to the pressure measuring area; protection of head of borehole against penetration of surface waters, mud, stones, etc. – Ensure permanent aeration – Operating availability checked by flushing operations
	Closed borehole Pressure indication by high precision manometer	R and L: high M: the total hight between manometer and dam crest P: ±0.5 m resp. ±1% of M Redundancy: necessary; installation of piezometers in groups	– Well-tried method – Pipes and connections to the manometers must be watertight – Do not relieve pressure artificially, to allow the observation of max. pressures even if they need a long time to build up – Periodical venting of pipes required – Periodical check of manometers absolutely necessary
	Pneumatic, hydraulic or electrical pressure cells installed individually in the embankment or in boreholes or at several levels in the same borehole	R and L: high M: total height between cell and dam crest P: ±0.5 m resp. ±1% of M Redundancy: necessary; provision of a great number of cells or disposition in groups	– Central reading of pressure cells spread over a large area possible – Hydraulic measures possible only if the measuring station lies below the minimum pressure level – Careful selection of the filter type in order to avoid its early clogging – No splicing in cables and pipes – Cables and pipes are threatened by differential settlements
Detection of physical or chemical alterations (erosion, dissolution)	Turbidimeter	R and L: high M: 0–500 ppm P: ±1 ppm Redundancy: necessary, analysis of water samples in laboratory	– Indicates amount of dissolved or suspended materials – A protected location is important – Calibration by means of laboratory analysis of seepage water

8. Législation

8.1 Règlement d'exécution

*de l'article 3bis de la loi concernant la police des eaux
(Règlement concernant les barrages)*

(Du 9 juillet 1957, état au 1 janvier 1979)

Le Conseil fédéral suisse,

vu l'article 3bis de la loi fédérale du 22 juin 1877 concernant
la police des eaux dans les régions élevées;
vu les articles 2, 4 et 89 de la loi fédérale du 23 mars 1962 sur
la protection civile;
vu les articles 2 et 4 à 10 de l'ordonnance du 27 novembre
1978 sur la protection civile, de même que l'ordonnance du
21 octobre 1970 concernant le Service territorial,

arrête:

I. Généralités

Article premier
Sont soumis au présent règlement les barrages dont la hau-
teur de retenue au-dessus du niveau d'étiage du cours
d'eau ou du niveau du thalweg est de 10 m au moins ou, si
cette hauteur est de 5 m au moins, ceux dont la retenue est
supérieure à 50000 m³.
Le présent règlement est aussi applicable par analogie aux
barrages qui, bien que n'atteignant pas les dimensions pré-
vues à l'article premier, constituent un grave danger pour
les régions situées en aval.
Il en est de même, lorsque la sécurité publique est en jeu,
pour les barrages en rivière créant une accumulation prin-
cipalement au moyen de vannes mobiles.

Art. 2
Des exceptions seront autorisées s'il est présumable que
l'exécution de certaines dispositions du présent règlement
n'est pas en rapport avec l'importance de l'ouvrage et avec
les dangers qu'il représente pour les régions situées en
aval.

Art. 3
Le Département fédéral des transports, des communica-
tions et de l'énergie exerce, du point de vue de la police des
eaux, la haute surveillance sur les ouvrages de retenue
soumis au présent règlement.
Il décide, de concert avec le Département militaire fédéral et
les autres départements intéressés, si l'on se trouve en pré-
sence des cas spéciaux visés par les articles 1er et 2 et
autorise, le cas échéant, des exceptions. A cette fin, les pro-
jets généraux des ouvrages de retenue doivent lui être sou-
mis par l'intermédiaire de l'autorité cantonale compétente.
Le Département fédéral des transports, des communica-
tions et de l'énergie décide, de concert avec le Département
militaire fédéral et l'Office fédéral de la protection civile et
après avoir entendu les cantons intéressés, quels sont les
lacs d'accumulation pour lesquels un système d'alarme-
eau est nécessaire. Il fixe, dans chaque cas, de concert avec
les départements intéressés et après avoir entendu les can-
tons et les propriétaires des ouvrages, l'étendue des zones
rapprochées qui doivent être équipées d'appareils d'alar-
me-eau. Le Conseil fédéral peut exceptionnellement pres-
crire la transformation du système d'alarme-eau en un sys-
tème constamment prêt à fonctionner avec télécommande
automatique.

8. Legislation

8.1 Executive Decree

*Concerning Article 3bis of the Federal Law Regarding
the Supervision of Hydraulic Structures (Dam
Regulation)*

(July 9, 1957, state January 1, 1979)

The Federal Council

based on Article 3bis of the Federal Law of June 22, 1877
regarding the Supervision of Hydraulic Structures in High
Mountain Areas
based on Articles 2, 4 and 89 of the Federal Law of March 23,
1962 regarding Civil Protection
based on Articles 2 and 4 – 10 of the regulation of November
27, 1978 regarding Civil Protection and
based on the decree of October 21, 1970 concerning Terri-
torial Services

decides:

I. General

Article 1
Subject to this decree are all dams
– with a minimum imponding head of 10 m above the low
  water level of existing waters or above the existing terrain,
  or
– with a minimum imponding head of 5 m and a reservoir
  capacity of more than 50000 m³.
Dams below the above dimensions may be subject to this
regulation, if they considerably endanger the population in
the area downstream of the dam.
Weirs providing damming mainly by gates may be governed
by this regulation, if required by the interests of public
safety.

Article 2
Exemptions are granted if the fulfilment of certain articles of
this decree is unreasonable, compared to the importance of
the structure and the danger to the downstream area.

Article 3
The supervision of facilities for storing water subject to this
decree is incumbent on the Federal Department for Trans-
portation and Energy.
The Department decides in agreement with the Federal Mili-
tary Department and other interested Departments on
exemptions regarding Article 1 and 2 and authorises excep-
tions if necessary. To this end, the preliminary designs of
dams must be submitted to the Department through the
medium of the competent State Authority.
The Federal Department for Transportation and Energy de-
cides in agreement with the Federal Military Department and
the Federal Office for Civil Protection and after hearing the
interested States on the necessity of water alarm systems
for damming facilities. It decides in agreement with the dif-
ferent Departments and after hearing the interested States
and the owners of the dams on the extent of the close alarm
zones to be provided with water alarm equipment. In excep-
tional cases the Federal Council may specify permanent
water alarm systems with automatic remote release.

Art. 4
L'autorité exerçant la haute surveillance peut, d'entente avec le canton intéressé et après avoir pris contact avec le maitre de l'œuvre, nommer des experts chargés de traiter les questions relevant de la haute surveillance. Les frais y relatifs sont à la charge du maitre de l'œuvre.

Art. 5
Pour élaborer les projets des différentes parties de l'installation, le maitre de l'œuvre doit prendre assez tôt contact avec les autorités de surveillance de la Confédération et du canton. L'autorité cantonale compétente soumet ces projets à l'autorité de haute surveillance, pour approbation, dans un délai convenable avant le début des travaux. Ceux-ci ne peuvent être entrepris qu'une fois les projets approuvés.

Doivent être joints aux projets:

a Les résultats des études géologiques et techniques du sol de fondation et du bassin d'accumulation;

b. Les calculs statiques ou de stabilité;

c. Les résultats des examens préalables, exécutés avec la participation d'un laboratoire d'essai des matériaux ou d'un laboratoire de mécanique des terres officiellement reconnus, concernant les matériaux prévus pour la construction de l'ouvrage;

d. Toutes les indications nécessaires, de caractère technique et hydraulique, concernant les installations prévues pour l'évacuation des crues et pour la vidange, ainsi que la durée requise pour l'abaissement préventif du bassin d'accumulation.

Les modifications des projets qui s'écartent considérablement des plans approuvés, ainsi que les transformations et les agrandissements apportés par la suite aux ouvrages doivent être communiqués à l'autorité de haute surveillance et faire l'objet d'une approbation spéciale.

Art. 6
Avant d'approuver les plans et les modifications apportées aux projets, l'autorité de haute surveillance les soumet, pour avis, aux départements intéressés.

Art. 7
Durant la période de construction et après l'achèvement de l'ouvrage, l'autorité de haute surveillance sera renseignée sur:

a. Tous les résultats des essais de béton et de mécanique des terres. Ces essais seront exécutés suivant un programme établi en collaboration avec l'autorité de haute surveillance et en application de l'article 4 de l'ordonnance du 21 août 1962 concernant le calcul, l'exécution et l'entretien des constructions placées sous la surveillance de la Confédération (Ordonnance sur les normes de construction). Les résultats des essais seront consignés dans des rapports partiels, puis, une fois les travaux achevés, rassemblés dans un rapport final;

b Les résultats des injections nécessaires pour la consolidation et l'étanchement du sous-sol;

c. Les résultats des mesures effectuées conformément à l'article 12. Ces résultats devront être rassemblés dans des rapports annuels; l'article 25 est réservé;

d. Les principaux plans d'exécution.

Art. 8
Le propriétaire de l'ouvrage est tenu d'accorder en tout temps le libre accès à ses installations aux fonctionnaires et autres personnes chargés par les offices fédéraux de l'exécution de la présente ordonnance; il leur fournira les renseignements et les documents dont ils ont besoin pour remplir leur tâche.

Article 4
The Supervising Authority may in agreement with the interested States and after contact with the owner appoint experts for the treatment of problems in connection with the supervision. All relevant costs have to be paid by the owner.

Article 5
The final design of the different parts of the plant is to be prepared in contact with the Supervising Authorities of the Confederation and the State. It must be submitted through the State Authorities to the Federal Supervising Authority at reasonable notice before the start of construction. The construction may not start before approval of the final design.
The final design must include:

a) the results of the geological and technical investigations of the subsoil at the dam-site and in the reservoir area;

b) static and stability analyses;

c) the results of the investigations of construction materials foreseen for the construction of the dam. These pre-investigations have to be carried out with the assistance of recognized material testing or soil mechanics laboratories;

d) all relevant technical and hydraulic information on spilling facilities and for emptying of the reservoir. The time required for the precautionary drawdown of the reservoir has to be specified.

Essential changes to the design and later alterations and extensions of the plant have to be submitted to the Supervising Authority for approval.

Article 6
The Supervising Authority notifies the involved Federal Departments before the approval of the project or its changes.

Article 7
During and after construction the Supervising Authority has to be notified about:

a) all results of concrete control and all soil mechanical tests. These tests have to follow a procedure established in co-operation with the Supervision Authority and according to article 4 of the Executive Decree of August 21, 1962 concerning the Design, Execution and Maintenance of Constructions Placed under the Supervision of the Confederation (Executive Decree on Construction Standards). The results of these tests are to be submitted as intermediate reports and gathered in a final report when construction has been completed;

b) the results of all grouting required to consolidate and impermeabilize the subsoil;

c) the results of all measurements according to Article 12 in yearly reports (Article 25 is reserved);

d) the main as-built drawings.

Article 8
The owner is obliged to grant access to the plant at all times to all civil servants commissioned with the execution of this decree. He is furthermore obliged to give all necessary information and make available all documents. The same rule applies to delegates and experts of the Supervising Authorities.

Art. 9

Le contrôle exercé par les organes officiels et leurs mandataires ne dégagera en aucune façon le propriétaire de l'ouvrage de sa responsabilité civile et de ses obligations en cas de dommages.

II. Ouvrages

Art. 10

Les barrages doivent être construits de manière à répondre aux exigences particulières qui leur sont demandées, compte tenu de la nature du sous-sol, du type de l'ouvrage et du mode d'exécution prévu.

Art. 11

Il y a lieu de prévoir, à l'intérieur des barrages, les aménagements nécessaires aux contrôles et aux révisions éventuelles.

Art. 12

Des installations adaptées à l'importance de l'ouvrage seront aménagées pour mesurer les déformations subies par les fondations et le corps du barrage, les pertes d'eau par infiltration, les températures à l'extérieur et à l'intérieur de l'ouvrage, la pression de l'eau sur la surface de fondation du barrage, ainsi que les pressions de l'eau interstitielle et, éventuellement, les lignes de saturation dans les digues. Ces mesures devront, autant que possible, être entreprises déjà pendant la construction de l'ouvrage.

Art. 13

Une installation de dimensions suffisantes sera aménagée pour évacuer les crues lorsque le bassin est plein. On justifiera la crue admise pour calculer l'ouvrage.

Art. 14

Une vidange de fond devra être installée pour permettre la vidange du bassin d'accumulation et la régularisation du niveau du lac. Si les circonstances l'exigent, deux vidanges seront aménagées à des niveaux différents.
En règle générale, les vidanges seront équipées de deux vannes, l'une faisant fonction de dispositif de sécurité, l'autre de dispositif d'exploitation et de réglage.

Art. 15

Les ouvrages de décharge seront complétés au besoin par des installations permettant d'évacuer l'eau sans dommages.

Art. 16

Si la rupture de conduites raccordées à la prise d'eau peut entraîner d'importants dommages, il y a lieu de prévoir un dispositif de fermeture automatique pouvant être actionné manuellement et à distance.

Art. 17

Des installations seront aménagées pour enregistrer les niveaux de l'eau et les débits écoulés du bassin. L'ampleur de ces installations dépendra de l'importance du barrage.

Art. 18

Le barrage et la centrale seront reliés par une installation offrant toute garantie pour la transmission de renseignements et d'ordres.

Article 9

By the supervision of the Federal Authorities or its delegates the owner of the plant is in no way released from his liabilities and responsibilities in case of damages.

II. Constructional Requirements

Article 10

The construction of dams has to conform to the requirements of the individual case, to the subsoil conditions, to the type of dam selected and to the planned way of construction.

Article 11

In the interior of the dams all constructional facilities shall be provided in view of their monitoring and possible revision.

Article 12

According to the importance of the structure, the necessary facilities have to be provided for the measurement of:
a) the displacements of the dam, its foundation and abutments,
b) the seepage water,
c) the temperature in and outside of the dam,
d) the uplift pressure in the foundation contact zone of concrete dams,
e) the pore water pressure and, if necessary
f) the seepage line in earth dams.

The observation of these data has to start possibly during construction.

Article 13

To discharge floods at full reservoir, an appropriately designed spillway has to be provided. The flood data used for the design have to be submitted.

Article 14

For the emptying and regulation of the reservoir, a bottom outlet has to be provided. If necessary, two outlets at different elevations have to be envisaged.
As a rule the outlets must have two closing devices of which one serves as safety device and the other as operating or regulating device.

Article 15

If necessary, facilities for the safe discharge of water releases are to be provided downstream of spillways and outlets.

Article 16

If the rupture of a conduit, beyond the water intake, might cause considerable damage, an automatic closing device has to be provided, which can be released manually and by remote control.

Article 17

Devices for the recording of the reservoir level and the water releases from it have to be provided. Size and extent of these devices have to be in accordance with the importance of the dam.

Article 18

A reliable communication link between dam and operating center has to be established for the transmission of messages and signals.

III. Mesures spéciales en vue de garantir la sécurité publique

Art. 19
Pour les mesures spéciales de sécurité à ordonner, il sera tenu compte aussi bien des dommages pouvant résulter de la destruction de l'ouvrage que des dépenses supplémentaires nécessaires, pour assurer une plus grande sécurité.

Art. 20
Les différentes parties de l'ouvrage seront projetées et construites de façon à satisfaire non seulement aux exigences techniques normales d'exploitation, mais encore à assurer, eu égard aux régions sises en aval, une sécurité aussi grande que possible contre la destruction de l'ouvrage par des faits de guerre.

Pour les barrages de retenue, il s'agira d'élucider dans chaque cas si cette sécurité peut être obtenue par des mesures constructives, par une vidange rapide du bassin d'accumulation ou par ces deux moyens combinés.

Art. 21
Les ouvrages de vidange des bassins d'accumulation devront avoir une forme et des dimensions assurant rapidement, en cas de besoin, la vidange ou l'abaissement partiel de la retenue, compte tenu de la capacité d'écoulement du cours d'eau.

Pour éviter les destructions dues à des faits de guerre, le Département militaire fédéral fera les préparatifs nécessaires à l'abaissement préventif du niveau des bassins d'accumulation. Le niveau pourra être abaissé soit par l'utilisation de l'eau accumulée de divers ouvrages – l'exploitation d'autres aménagements étant, au besoin, momentanément suspendue – soit, de façon accélérée, par le fonctionnement des dispositifs de vidange, soit enfin par la combinaison des deux moyens.

La compétence d'ordonner l'abaissement des bassins d'accumulation pour des raisons de guerre est réglée par l'article 3bis, 5e alinéa, de la loi fédérale du 22 juin 1877 concernant la police des eaux dans les régions élevées.

Art. 22
Les parties de l'ouvrage dont la destruction mettrait particulièrement en danger les régions situées en aval (p.ex. les dispositifs d'ouverture des vannes de fond ou de fermeture des conduites forcées à leur partie supérieure) devront si possible être aménagées sous terre et à l'abri des bombardements.

Art. 22bis
Le Département militaire fédéral pourra ordonner des mesures complémentaires de protection, telles que la pose de câbles aériens et de filets protecteurs ou le camouflage. Si des formations de défense antiaérienne ou de surveillance sont chargées en permanence de la défense de certains barrages, les propriétaires de ces ouvrages seront tenus de leur fournir le logement nécessaire.

Art. 23
Dans les cas fixés à l'article 3, 3e alinéa, un système d'alarme-eau sera installé de façon qu'il soit utilisable non seulement en temps de service actif de l'armée, en cas de danger dû à des faits de guerre, mais aussi temporairement en temps de paix, lorsque la population habitant en aval de lacs d'accumulation est manifestement mise en péril.

La région inondée en cas de destruction totale du barrage est divisée en une zone rapprochée et une zone éloignée. La zone rapprochée comprend la région inondée dans un délai de deux heures au plus, tandis que la zone éloignée

III. Special Provisions for Public Safety

Article 19
Special safety provisions have to consider the expected damage caused by the destruction of a structure as well as the necessary extra expenditure for increased safety.

Article 20
The different elements of a plant have to be designed and built in such a way as to satisfy the normal operational requirements. Furthermore, they have to grant maximum safety to the downstream area also in case of destruction due to acts of war or the like.

In the case of dams, it has to be examined individually whether this safety can be achieved by constructional provisions, by a rapid emptying of the reservoir or a combination of both methods.

Article 21
The discharging facilities have to be designed for a rapid emptying or drawdown of the reservoir taking into account the capacity of the downstream river-bed.

To prevent destructions in case of war, the Federal Military Department makes the necessary preparations for a precautionary drawdown of reservoirs. The drawdown may be achieved by turbination of the stored water in certain plants and the simultaneous shut-down of others by the use of the discharge facilities or finally by a combination of both possibilities.

The responsibilities for giving such orders in case of war are ruled by Article 3bis, clause 5 of the Federal Law of June 22, 1877 concerning the Supervision of Hydraulic Structures in High Mountain Areas.

Article 22
Elements of the plant, the destruction of which particularly endanger the downstream area (e.g. control devices on bottom outlets, valves at the upper end of penstocks) are to be located, if ever possible, underground and bomb-proof.

Article 22bis
The Federal Military Department may specify additional safety measures such as rope barriers, nets, camouflage. If dams are specially protected by permanent anti-aircraft or guard troops, the owner has to provide the necessary and appropriate accommodation.

Article 23
In all cases mentioned in Article 3, clause 3, a water alarm system is to be installed, which can be used not only in case of impending or actual war, but temporarily also in peacetimes, if a danger to the downstream area of a dam is recognizable.

The whole area flooded in case of the total collapse of a dam is divided into two zones. The close zone consists of the area flooded within two hours at the most, the distant zone of the rest of the flooded area. In the close zone, the alarm is raised by the Military Warning Service through special alarm facilities. In the distant zone, alarm is given through appropriate means of the Army and transmitted to the population by the means of the Civil Protection Organization.

The owner has to provide for the Military Warning Service the necessary facilities such as blast-protected alarm centers and observation posts as well as furnished accommodation situated nearby. In the close zone the owner has to install automatic alarm devices working independently from the electric supply net and complying with the directions of the Federal Military Department and the Federal Post, Telephone and Telegraph Service.

s'étend sur le reste du territoire touché par l'inondation. Dans la zone rapprochée, l'alarme est donnée à l'aide d'appareils d'alarme-eau spéciaux par le service d'alerte de l'armée mobilisé à cet effet. Dans la zone éloignée, l'alarme est déclenchée à l'aide des dispositifs appropriés de l'armée et transmise à la population par les moyens de la protection civile.

Le propriétaire de l'ouvrage fera construire les installations nécessaires telles que centrales d'alarme-eau et postes d'observation capables de résister à la pression d'une explosion ainsi que le logement du détachement d'alarme-eau, installé à proximité. Dans la zone rapprochée, il devra installer des appareils d'alarme automatiques et indépendants du réseau électrique, conformes aux règlements du Département militaire fédéral et de l'Entreprise des postes, téléphones et télégraphes.

Les centrales d'alarme-eau sont reliées aux installations de transmission de renseignements et d'ordres, établies en vertu de l'article 18.

Les aménagements et les installations d'alarme-eau existants qui ne répondent pas aux exigences posées par le 3e alinéa, seront adaptés.

Dans les zones d'inondation communes à plusieurs lacs d'accumulation, les installations d'alarme-eau seront établies de façon à être utilisables pour chacun de ces lacs. Les propriétaires des ouvrages en cause se répartiront équitablement les frais de construction et d'entretien correspondant à ces zones. Si aucun accord ne peut être conclu, le différend sera tranché par l'autorité de haute surveillance.

Les offices fédéraux et cantonaux de la protection civile se prépareront à accomplir également en temps de paix toutes les tâches dont ils seront chargés en période de service actif, en tenant compte des problèmes spéciaux posés par les dangers que présentent les lacs d'accumulation. Il s'agit en particulier de la diffusion de l'alarme dans les zones éloignées et des instructions à donner à la population quant à la manière de se comporter.

Le propriétaire de l'ouvrage doit entretenir ou faire entretenir les installations d'alarme-eau selon les règles de l'art. Elles seront mises à disposition de la troupe, en bon état de marche, pour les cours d'instruction périodiques et en cas de danger. Le contrôle annuel de ces installations ainsi que les essais d'alarme seront ordonnés par le Département militaire fédéral, après entente avec le propriétaire de l'ouvrage.

Les frais occasionnés par la mise à disposition ainsi que l'utilisation des lignes téléphoniques des PTT sont à la charge du propriétaire de l'ouvrage, conformément aux conditions d'abonnement en vigueur. Il n'est pas perçu de droits régaliens. Le service d'alerte de l'armée, hommes, locaux et matériel, sera mis en service gratuitement en cas d'utilisation temporaire en temps de paix. Les dépenses correspondant aux tâches assignées à la protection civile selon le 7e alinéa sont à la charge de celle-ci.

Art. 23bis

Les propriétaires et possesseurs de biens-fonds et de bâtiments sont tenus d'autoriser la pose et l'utilisation d'installations et d'équipements pour l'alarme-eau dans la zone rapprochée et de permettre aux organes de contrôle d'y accéder. Ces obligations s'appliquent aussi aux fermiers, locataires et autres occupants des immeubles.

Si des dégâts sont causés par les travaux d'aménagement ou par les mesures de contrôle ou si l'utilisation des locaux nécessaires est entravée de façon excessive, les lésés ont droit à un dédommagement équitable.

Le propriétaire de l'ouvrage possède le droit d'expropria-

Water alarm centers have to be connected with the communication link stipulated in Article 18.

Existing alarm facilities not complying with the requirements according to clause 3 hereof have to be modified. In areas belonging to flood zones of different dams, the joint use of the alarm devices has to be provided. The cost for the installation and the maintenance are adequately distributed among the various owners. In the case of disagreement, the Supervising Authority takes the decision.

The offices for Civil Protection of the Confederation and the States take all precautions to fulfill their wartime duties already in peacetime. They prepare the alarm in the distant zones and instruct the population on the appropriate behaviour.

Water alarm facilities are to be professionally maintained and kept in working condition by the owner. They may be used by the Military Warning Service for training purposes. The yearly control of the water alarm system and its tests are arranged by the Federal Military Department in agreement with the owners.

The cost for the preparation and use of telephone lines has to be carried by the owner. No state monopoly tax is levied. The Military Warning Service, personel, accommodation and material are available in peacetime and temporarily free of charge. The cost of clause 7 hereof is covered by the Civil Protection Organization.

Article 23bis

Owners, tenants and residents of buildings and real estate in the close zone are obliged to enable the installation and the use of water alarm devices and to grant access to these devices to all control personnel.

Damage caused by the installation and the controls and unimputable impairment of the use of such premises constitute a claim for indemnification.

The owner has the power of expropriation as far as necessary for the installation and maintenance of water alarm devices. The practise of this power is governed by the regulations of the Federal Law of June 20, 1930 regarding Expropriation.

The same principles apply for the installation and use of telephones or other devices for the electric or radio-electric transmission of signals and messages, serving the alarm system or the transmission of water alarms in the close zone.

tion, dans la mesure où celui-ci est nécessaire pour la construction et l'entretien des ouvrages et des installations. L'exercice de ce droit se règle sur les dispositions de la loi fédérale du 20 juin 1930 sur l'expropriation.

Les mêmes règles sont applicables pour la pose et l'utilisation de matériel téléphonique ou autres équipements de transmission électrique ou radioélectrique de signes ou de sons, utilisés dans les installations d'alarme-eau ou les liaisons établies pour donner l'alarme dans la zone rapprochée.

IV. Mise en exploitation

Art. 24

On ne procéuera à la mise en eau du bassin qu'au moment où l'état d'avancement des travaux le permettra sans mettre en danger des intérêts publics. Les dispositifs de fermeture et de sécurité seront préalablement contrôlés quant à leur bon fonctionnement et devront être maintenus en état de marche.

Le programme de la première mise en eau sera remis en temps utile aux autorités de surveillance.

Art. 25

Avant le début de la première mise en eau, le programme des mesures à faire conformément à l'article 12 ainsi que la fréquence des rapports y relatifs devront être fixés avec l'accord de l'autorité de haute surveillance.

Art. 26

Après l'achèvement des travaux, les autorités de surveillance vérifieront si le barrage a été construit conformément aux plans approuvés par les autorités et selon leurs instructions. Ce contrôle sera consigné dans un procès-verbal qui mentionnera l'état du barrage, les résultats essentiels des observations faites conformément à l'article 25, ainsi que les constatations faites au sujet du fonctionnement des diverses parties de l'ouvrage.

V. Exploitation et entretien

Art. 27

Le propriétaire de l'ouvrage d'accumulation en organisera l'exploitation de façon à en assurer l'utilisation et la surveillance d'une manière satisfaisante.

L'état de marche des vannes des vidanges et des évacuateurs de crues doit être contrôlé au moyer. d'essais de fonctionnement exécutés à intervalles convenables, en général une fois par an. L'autorité de haute surveillance en sera préalablement informée.

Art. 28

Toutes les observations et mensurations nécessaires pour vérifier le comportement du barrage seront exécutées régulièrement puis interprétées sans délai.

Les ouvrages de retenue devront être contrôlés chaque année par des ingénieurs civils expérimentés. Les propriétaires d'ouvrage qui n'en ont pas dans leur personnel feront contrôler leurs installations par des ingénieurs venant du dehors.

Le propriétaire de l'ouvrage devra en outre faire contrôler celui-ci périodiquement par des spécialistes en construction de barrages reconnus (ingénieurs, géologues). Ces contrôles porteront aussi bien sur le barrage proprement dit que sur les environs du bassin de retenue. A moins que des raisons particulières n'imposent des contrôles plus fréquents, ceux-ci auront lieu au moins tous les cinq ans. Les résultats de ces contrôles seront consignés dans des

IV. Initial Operation

Article 24

The filling of the reservoir may only start when the state of the construction work permits it without danger to public interests. The closing and safety devices have to be tested beforehand with respect to their operational ability and have to be kept operative at all times.

The intended program for the first filling of the reservoir has to be submitted to the Supervising Authorities in due time.

Article 25

Before the first filling of the reservoir, the program and the reports for all measurements according to Article 12 have to be determined in agreement with the Supervising Authorities.

Article 26

After completion, the dam is checked whether it is built in accordance with the approved design and with the instructions of the Supervising Authorities.

The result of this inspection is recorded. The structural condition of the dam, the essential results of the observations performed according to Article 25 and the operating ability have to be checked.

V. Operation and Maintenance

Article 27

The owner has to organize his operations in such a way that a reliable control of the dam is guaranteed.

The outlets and the moving parts of the spillway have to be tested for operational readiness by performance tests within appropriate periods, as a rule once a year. The Supervising Authority has to be informed in advance of such tests.

Article 28

All observations and measurements necessary for the monitoring of the behaviour of the dam have to be regulary taken and immediately evaluated.

Dams have to be checked once a year by experienced civil engineers. Owners who have no such personnel have to charge engineers from outside their organization with these inspections.

Furthermore, the owner is obliged to have periodical checks of the dam made by recognized experts in dam construction (engineers, geologists). These inspections have to cover the dam itself as well as the surroundings of the reservoir. They have to be undertaken at least every five years, provided that no special reasons call for shorter periods.

The results of all inspections have to be recorded in reports. These reports have to be submitted to the Supervising Authority.

If monitoring or inspections call for remedial actions, they have to be taken immediately and the Supervising Authority notified thereof.

rapports et portés à la connaissance de l'autorité de haute surveillance.

Les mesures qui se révéleront nécessaires à la suite des observations et des contrôles effectués seront appliquées immédiatement; l'autorité de haute surveillance en sera informée.

Art. 29

En cas d'événements extraordinaires, tels que comportement anormal du barrage, séismes, glissements de terrain, éboulements, avalanches et autres, qui menaceraient la sécurité du barrage ou feraient craindre des crues exceptionnelles, la direction de l'usine prendra sans délai les mesures propres à écarter le péril qui menace l'installation de retenue. Le cas échéant, elle abaissera préventivement le niveau du lac d'accumulation, compte tenu de la capacité d'écoulement du cours d'eau. Elle se fera conseiller par des spécialistes. Les organes de surveillance du canton et de la Confédération seront renseignés par la voie la plus rapide.

Les autorités cantonales mobilisent les organismes de protection civile pour l'exécution des mesures nécessaires selon l'article 23, 7e alinéa.

Sur demande des autorités cantonales, la Subdivision du service territorial mobilisera son service d'alerte et fera procéder à la mise en service des installations d'alarme-eau, y compris le réseau de transmission.

La direction de l'usine alertera par la voie la plus rapide le poste d'alarme désigné par le canton. Ce dernier alertera à son tour les autorités locales, les chefs locaux et les commandants des sapeurs-pompiers de guerre indépendants, les entreprises industrielles et de transport, les usines électriques, les établissements militaires et les services et commandements militaires qui se trouvent dans les régions menacées.

L'alarme-eau sera déclenchée par le Service d'alerte de l'armée, sous la responsabilité du propriétaire du lac d'accumulation, qui en donnera l'ordre.

Si les circonstances le permettent, ces mesures seront ordonnées avec l'accord de l'autorité fédérale de haute surveillance. S'il y a lieu, le Conseil fédéral pourra imposer au canton et à la direction de l'usine l'exécution des mesures nécessaires.

Art. 30

Le propriétaire de l'ouvrage est tenu de constituer un dossier concernant le barrage et de le tenir constamment à jour (journal du barrage).

Le dossier, dont les pièces seront classées avec ordre, devra notamment contenir:

a. Les documents concernant le projet, parmi lesquels les plans définitifs principaux et les données justificatives sur la stabilité et les sollicitations du barrage;

b. Les résultats des essais des matériaux utilisés dans la construction et des autres recherches opérées;

c. Des données sur l'exécution des travaux;

d. Les procès-verbaux de réception des ouvrages selon article 26;

e. Les résultats des recherches faites ultérieurement, ainsi que toutes les données sur l'entretien et la surveillance avec les résultats de toutes les mensurations faites;

f. Des données concernant les travaux complémentaires ou les transformations apportées;

g. Les rapports sur les contrôles effectués par des spécialistes;

h. Le schéma de l'organisation d'alarme-eau indiquant le genre et l'emplacement des liaisons et des appareils d'alarme spéciaux qui ont été installés;

Article 29

In case of extraordinary events such as unusual behaviour of the dam, earthquakes, landslides, rockfalls, avalanches, etc. which could affect the safety of the dam or induce an extraordinary flood, the plant management has to take immediately all appropriate actions to avert an impending threat to the dam. Possibly, the precautionary drawdown of the reservoir within the capacity of the downstream riverbed has to be started. The management appoints qualified professional engineers as consultants. The Supervising Authorities of the State and the Confederation have to be informed as quickly as possible.

The State Authorities call up the Civil Protection Organization to carry out the necessary actions according to Article 23, clause 7.

On request of the State, the Army mobilizes her Military Warning Services, who activate the water alarm system and its pertinent communication net.

The plant management warns the Alarm Organization designated by the State as quickly as possible. This Alarm Organization alerts the Local Councils of the endangered areas, the local commanders and commanding officers of Independent Military Fire Brigades, industrial and transport companies, power stations, as well as military works, services and headquarters.

The water alarm is released by the Military Warning Service by arrangement with and at the responsibility of the owner.

Circumstances permitting, these precautions are taken in agreement with the Federal Supervising Authorities. If necessary, the Federal Council may request the State and the plant management to act accordingly.

Article 30

The owner takes care that all documents concerning the dam are filed and permanently up-dated (Records of the Dam).

The records have to include, clearly filed, in particular:

a) the design with the essential as-built drawings and the necessary analyses of stability and stresses of the dam;

b) the results of the tests on construction materials and of other investigations;

c) information on the execution of the construction work;

d) the records of the final inspection of construction according to Article 26;

e) the results of the subsequent investigations and all information on maintenance and monitoring with all results;

f) information on additional constructions and reconstructions;

g) all reports on inspections performed by experts;

h) scheme of the water alarm organization, showing type of communication, installed alarm devices and their exact position;

i) a list of responsible Federal and State Authorities to be informed or alerted by the owner in case of an extraordinary event.

The documents have to be presented to the Supervising Authorities of the Confederation and the State on demand and at any time.

i. La liste des organes compétents sur le plan fédéral et
   cantonal qui doivent être avertis ou alarmés par le pro-
   priétaire de l'ouvrage en cas d'événement extraordi-
   naire.

Ces pièces doivent être présentées aux autorités de surveil-
lance de la Confédération et du canton à chaque réquisi-
tion.

*VI. Dispositions transitoires, dispositions concernant
la protection juridique et dispositions finales*

Art. 31
L'autorité de haute surveillance décide dans chaque cas, de
concert avec les départements intéressés et après avoir en-
tendu le propriétaire de l'ouvrage, dans quelle mesure les
dispositions du présent règlement sont applicables aux ou-
vrages existants.

Art. 32
Conformément aux dispositions générales sur la justice ad-
ministrative fédérale, le recours est ouvert jusqu'au Conseil
fédéral contre les décisions prises par les départements
compétents.

Art. 33
Le présent règlement entre immédiatement en vigueur.

*IV. Transitory Provisions, Provisions for Juridical
Protection, Final Provisions*

Article 31
The Supervising Authority determines in agreement with the
interested Departments and after hearing the owner as to
what extent the articles of this decree are valid for existing
plants.

Article 32
Against decisions of the competent Departments, that are
based on this decree, a complaint can be lodged with the
Federal Council according to the General Provisions of the
Federal Jurisdiction.

Article 33
This decree becomes effective at once.

8.2 Révision du règlement concernant les barrages

Une révision du règlement concernant les barrages est pré-
sentement en cours. Elle a principalement pour objet de ré-
duire à quelques heures le temps de mise en fonction du
système alarme-eau en temps de paix, qui est aujourd'hui
de 48 heures au maximum. Son entrée en vigueur est prévue
pour 1985. Seuls les articles 23 et 29 ont été modifiés et ils
auront la teneur suivante:

Article 23 (projet)

1 Dans les cas fixés à l'article 3, 3e alinéa, un système
d'alarme-eau sera installé de façon qu'il soit utilisable à tout
moment lorsque la population habitant en aval de lacs d'ac-
cumulation est manifestement mise en péril.
2 La région inondée en cas de destruction totale du bar-
rage est divisée en une zone rapprochée et une zone éloi-
gnée. La première comprend la zone inondée en un délai de
deux heures au plus, tandis que la seconde s'étend au reste
du territoire touché.
On peut renoncer à définir une zone rapprochée si la région
inondée est petite et si le système d'alarme prévu pour la
zone éloignée est jugé suffisant.
3 Dans la zone rapprochée, l'alarme est donnée à l'aide des
sirènes alarme-eau; dans la zone éloignée, elle a lieu par le
déclenchement de l'alarme générale et la diffusion, à la ra-
dio, d'instructions sur le comportement à suivre. L'ordon-
nance sur la protection civile fixe les signaux d'alarme et, en
cas de nécessité, l'emploi de signaux de remplacement.
Lorsque le régiment d'alerte[1] entre en service, les prescrip-
tions de l'ordonnance concernant l'organisation territoriale
et le service territorial sont en outre applicables.
Quand une panne technique affecte les moyens de trans-
mission ou le système d'alarme, on a recours à des moyens
de remplacement. En particulier, s'il s'agit d'une panne des

[1] autrefois: service d'alerte de l'armée

8.2 Revision of the Dam Regulation

A revision of the dam regulation is now in process. Its aim is
to modify the water alarm system in order to reduce the wa-
ter alarm delay in peacetime from 48 hours (maximal time at
present) to a few hours. The revision will presumably come
into force in 1985. The articles 23 and 29 will be changed
and read as follows:

Article 23 (draft)

1 In all cases mentioned in Article 3, clause 3, a water alarm
system is to be installed, which can be used at any time
when a danger to the downstream area of a dam is
recognizable.
2 The whole area flooded in case of the total collapse of a
dam is divided into two zones. The close zone consists of
the area flooded within two hours at the most, the distant
zone of the rest of the flooded area. The definition of a close
zone is not necessary, if the flooded area is small and a dis-
tant zone alarm system is sufficient.
3 In the close zone, the alarm is raised through water alarm
sirens. In the distant zone, alarm is given through release of
the general alarm and announcement of behaviour instruc-
tions by radio. The alarm signals to be used are specified in
the regulations regarding Civil Protection. If necessary,
alternative alarm signals may be used. If the Warning Regi-
ment[1] is on duty, the regulations of the Decree Concerning
Territorial Services are also valid. For the case of technical
breakdowns in the communication and alarm system
alternative systems have to be prepared by taking advan-
tage of the available means. In particular, if the water alarm
sirens fail in the close zone, alarm shall be raised as in the
distant zone.
4 The following degrees of preparedness are valid for the
water alarm system:

[1] formerly: Warning Service

sirènes de l'alarme-eau, l'alarme est donnée de la même manière que dans la zone éloignée.

4 Le système d'alarme-eau comprend les degrés de préparation suivants:

a. Degré de préparation 1: liaison interrompue entre la Centrale d'alarme-eau et les sirènes de cette alarme, sirènes alarme-eau verrouillées, aucun personnel d'alarme en service;

b. Degré de préparation 2: liaison entre la Centrale d'alarme-eau et les sirènes de cette alarme prête à fonctionner, sirènes alarme-eau verrouillées, aucun personnel d'alarme en service;

c. Degré de préparation 3: système d'alarme-eau prêt à fonctionner, aucun personnel d'alarme en service;

d. Degré de préparation 4: système d'alarme-eau prêt à fonctionner, personnel d'alarme en service.

Le déverrouillage des sirènes alarme-eau par le propriétaire de l'ouvrage s'effectue sur l'ordre des organes compétents cités à l'alinéa 5. Le propriétaire demande le raccordement aux PTT si nécessaire.

En règle générale, le degré de préparation 2 est valable pour le cas stratégique normal (temps de paix), et le degré de préparation 4 dans tous les autres cas stratégiques.

5 Sont compétents pour changer de degré de préparation:

a. Les autorités cantonales et l'Office fédéral de l'économie des eaux si le danger résulte d'un événement naturel ou est d'orgine technique; dans le cas stratégique normal (temps de paix), tous deux peuvent demander à la Division du service territorial l'engagement de formations du régiment d'alerte;

b. Le commandement de l'armée si le danger est dû à des faits de guerre.

Les organes ci-dessus s'informent mutuellement, sans retard, des ordres visant à modifier des degrés de préparation.

6 Sont à la charge des cantons:

a. la planification, la préparation et l'exécution des mesures destinées à assurer la réception des ordres d'alarme et la diffusion de l'alarme générale dans les zones menacées;

b. la planification et la préparation visant l'évacuation de la population, y compris l'information préalable de celle-ci;

c. la formation du personnel requis pour exécuter les tâches décrites sous a et b;

d. la préparation de l'information destinée à la population et aux entreprises de transports, quant à la situation et au comportement à suivre.

Les cantons doivent disposer d'une permanence et d'un service d'alarme prêts en permanence.

7 Le propriétaire doit construire à ses frais les installations nécessaires tels que

a. les centrales alarme-eau et les postes d'observations capables de résister à l'onde de choc d'une explosion;

b. les logements du détachement d'alarme-eau, installés à proximité;

c. dans la zone rapprochée, des sirènes d'alarme automatiques et indépendantes du réseau électrique;

d. la liaison téléphonique et la liaison par radio depuis le barrage, pour pouvoir atteindre les autorités cantonales; en temps de paix, la liaison radio ne doit être garantie que durant le degré de préparation 4.

Les appareils d'alarme doivent être conformes aux règlements du Département militaire fédéral et des PTT. Les centrales d'alarme-eau sont reliées aux installations de transmission de renseignements et d'ordres établies en application de l'article 18.

a) degree of preparedness 1: link between water alarm centre and water alarm sirens disconnected; water alarm sirens locked, no warning staff on duty;

b) degree of preparedness 2: link between water alarm centre and water alarm sirens in working condition, water alarm sirens locked, no warning staff on duty;

c) degree of preparedness 3: water alarm system in working condition, no warning staff on duty;

d) degree of preparedness 4: water alarm system in working condition, warning staff on duty.

The owner unlocks the water alarm sirens by order of those authorized in clause 5. If necessary, he asks the Federal Post, Telephone and Telegraph Service to connect the communication links. Ordinarily, degree of preparedness 2 is valid in the strategic base case (peacetime), in all other strategic cases, degree of preparedness 4 is valid.

5 Authorized to change the degree of preparedness are:

a) the State Authorities and the Federal Office for Water Economy in case of technical or natural threat; in the strategic base case (peacetime), both of them may ask the Territorial Service to mobilize the formations of the Warning Regiment;

b) the Army High Command in case of threat by acts of war.

The above-mentioned authorities inform each other about ordered changes in the degree of preparedness.

6 The States provide for:

a) planning, preparation and implementation of the measures guaranteeing the reception of the alarm orders and giving the general alarm in the endangered area;

b) planning and preparing the evacuation including the previous information of the population;

c) training of the staff needed for the implementation of the measures mentioned in a and b;

d) preparation of the information for the population and the transport companies concerned about the threat and their appropriate behaviour.

The States have to maintain an alarm organization always ready for action.

7 The owner has to provide at his own costs the necessary facilities, such as:

a) blast-resistant water alarm centres and observation posts;

b) accommodations for a detachment of the Warning Regiment situated nearby;

c) in the close zone, automatic alarm sirens working independently from the electric supply net;

d) a telephone and radio link between the dam and the State Authorities. The radio link has to be guaranteed in peacetime for degree of preparedness 4 only.

The alarm devices have to comply with the specifications of the Federal Military Department and the Federal Post, Telephone and Telegraph Service. Water alarm centres have to be connected to the communication link stipulated in Article 18.

8 In areas belonging to flood zones of different dams, the joint use of the alarm devices has to be provided. The costs for the installation and the maintenance are adequately distributed among the various owners. In the case of disagreement, the Supervising Authority takes the decision.

9 Water alarm facilities are to be professionally maintained and kept in working condition by the owner. They may be used by the Warning Regiment for training purposes. The yearly control of the water alarm system and its tests are arranged by the Federal Military Department in agreement with the owners.

8 Lorsqu'une zone d'inondation est commune à plusieurs lacs d'accumulation, les installations d'alarme-eau sont établies de façon à être utilisables pour chacun d'eux. Les propriétaires des ouvrages en cause se répartissent équitablement les frais de construction et d'entretien. L'autorité de haute surveillance tranche les différends.

9 Le propriétaire de l'ouvrage doit faire entretenir les installations d'alarme-eau selon les règles de l'art. Elles seront mises à la disposition de la troupe, sans frais et en état de marche, pour les cours d'instruction périodiques et les mobilisations. Le contrôle annuel des ces installations ainsi que les essais d'alarme seront ordonnés par le Département militaire fédéral, après entente avec le propriétaire de l'ouvrage.

10 Les frais occasionnés par la mise à disposition ainsi que l'utilisation des lignes téléphoniques des PTT sont à la charge du propriétaire de l'ouvrage, conformément aux conditions d'abonnement en vigueur. Il n'est pas perçu de droit régalien. Les mobilisations du régiment d'alerte n'entraînent aucun frais. Les cantons et les communes prennent à leur charge les tâches qui leur sont assignées.

Article 29 (projet)

1 En cas d'événement extraordinaire, tels qu'un comportement anormal du barrage, un séisme, un glissement de terrain, un éboulement, une avalanche et autre menace pour la sécurité du barrage, ou qui ferait craindre une crue exceptionnelle, le propriétaire de l'ouvrage doit prendre sans délai les mesures propres à écarter le péril pour la retenue. Le cas échéant, il abaissera préventivement le niveau du lac d'accumulation, compte tenu, lorsque cela est possible, de la capacité d'écoulement du cours d'eau. Il se fera conseiller par des spécialistes. Les organes de surveillance du canton et de la Confédération seront renseignés par la voie la plus rapide.

2 Quand les circonstances le permettent, les mesures sont prises d'un commun accord avec l'autorité de haute surveillance. Au besoin, cette dernière peut demander au canton et au propriétaire de l'ouvrage de prendre les mesures nécessaires. Si l'événement n'est plus maîtrisable avec certitude, le système d'alarme-eau est mis en état de fonctionner.

3 Lorsque le degré de préparation 4 de l'alarme-eau est décrété, l'Office fédéral de l'économie des eaux ordonne la mise en état d'alerte de la Centrale nationale d'alarme. Les autorités cantonales convoquent les organismes locaux d'alarme et renseignent tant la population des zones d'inondation que les entreprises de transports (notamment les CFF et les PTT) sur les dangers et le comportement à avoir en cas de déclenchement de l'alarme. Ils déterminent la région à alarmer et en informent la Centrale nationale dans la mesure où il ne faut pas alarmer toute la zone éloignée. Les communes veillent à ce que l'alarme générale puisse être diffusée en tout temps dans les zones menacées. Dans le cas stratégique normal (temps de paix), les établissements, services et commandements militaires seront informés par l'état-major du groupement de l'état-major général, dans tous les autres cas par le commandement de l'armée.

4 Sont compétents et responsables pour le déclenchement de l'alarme:

a. dans la zone rapprochée, le propriétaire en cas de danger résultant d'un événement naturel ou d'origine technique, le régiment d'alerte si le danger est dû à des fait de guerre;

10 The cost for the preparation and use of telephone lines has to be carried by the owner according to the normal conditions of use. No state monopoly tax is levied. Engagements of the Warning Regiment are at no cost for the owner. Services by States and Municipalities are at their own costs.

Article 29 (draft)

1 In case of extraordinary events such as unusual behaviour of the dam, earthquakes, landslides, rockfalls, avalanches, etc. which could affect the safety of the dam or induce an extraordinary flood, the plant management has to take immediately all appropriate actions to avert an impending threat to the dam. Possibly, the precautionary drawdown of the reservoir has to be started, if feasible within the capacity of the downstream river-bed. The management appoints qualified professional engineers as consultants. The Supervising Authorities of the State and the Confederation have to be informed as quickly as possible.

2 Circumstances permitting, these precautions are taken in agreement with the Supervising Authority. If necessary, the Supervising Authority may request the State and the plant management to act accordingly. If the event can no longer be controlled safely, the water alarm system has to be activated.

3 If the degree of preparedness 4 has been ordered for the water alarm system, the Federal Office for Water Economy arranges for the engagement of the National Alarm Centre. The State Authorities call up the local alarm organizations and instruct the population in the flood zones as well as the transport companies, particularly the Swiss Federal Railway and the Federal Post, Telephone and Telegraph Service, about the threat and the behaviour in case of releasing of the water alarm. The State Authorities determine the area to be alarmed and inform the National Alarm Centre, if only a part of the distant zone is to be alarmed. The Municipalities take care that the general alarm may be released at any time in the endangered area. In the strategic base case (peacetime), the military works, services and headquarters concerned are to be informed by the Group for General Staff Services, in all other strategical cases by the Army High Command.

4 Authorized and responsible for the release of the water alarm are:

a) in the close zone the owner in case of technical or natural threat and the Warning Regiment in case of acts of war;

b) in the distant zone the National Alarm Centre and the Warning Regiment if the latter or part of it is on duty.

5 In the close zone, the water alarm has to be released, if

a) in case of technical or natural threat a flood wave can probably not be prevented any longer;

b) in case a destruction was produced by acts of war leading to an important water release. In case of damage without important water release, the owner has to proceed according to clause 1.

6 Immediately after the release of the water alarm in the close zone, the owner or the Warning Regiment has to inform the National Alarm Centre as well as the State Authorities. Immediately on receipt of such an information, the National Alarm Centre releases the general alarm in the distant zone by radio, if the State Authorities concerned have not previously decided something else. The National Alarm Centre also diffuses by radio the prepared behaviour instructions. If the Warning Regiment is on duty, its information on radio-telephony, channel 3, is also considered as

b. dans la zone éloignée, la Centrale nationale d'alarme et le régiment d'alerte, si celui-ci est totalement ou partiellement engagé.

5 L'alarme-eau doit être déclenchée dans la zone rapprochée si.

a. lors de dangers résultant d'un événement naturel ou d'origine technique, une vague d'inondation ne peut plus, selon toute probabilité, être évitée;

b. en cas de faits de guerre, il y a destruction entraînant un écoulement important. Lors de dégâts sans écoulement important, le propriétaire de l'ouvrage agira selon les directives prévues au chiffre 1.

6 Après le déclenchement de l'alarme-eau, le propriétaire de l'ouvrage ou le détachement du régiment d'alerte doit, sans délai, informer tant la Centrale nationale d'alarme que les autorités cantonales. Dès la réception d'un tel message, la Centrale nationale d'alarme ordonne tout de suite par radio le déclenchement de l'alarme générale dans toute la zone éloignée, dans la mesure où les autorités des cantons concernés n'ont encore rien décidé d'autre; elle diffuse également par radio les instructions déjà préparées au sujet du comportement à suivre. Si une formation du régiment d'alerte est engagée, il lui appartient aussi de diffuser le message d'alerte par le canal 3 de la télédiffusion. Ce message a également valeur d'ordre pour le déclenchement de l'alarme générale; sa teneur doit concorder avec la mission d'alarme de la Centrale nationale.

7 Tout organisme qui constate une panne, tant dans le système de transmission que dans le système d'alarme, est tenu d'en informer sans délai les autorités cantonales. Celles-ci mettent en place les moyens de remplacement et en informent immédiatement tous les organes concernés. En cas d'urgence, elles peuvent utiliser à cet effet les moyens de la Centrale nationale d'alarme.

order to release the general alarm. Its contents must correspond to the alarm order from the National Alarm Centre.

7 Anyone who detects a breakdown in the communication link or alarm system is obliged to inform the State Authorities. The latter determine an alternative alarm system and inform at once all concerned. In case of urgency, it may be done by means of the National Alarm Centre.