Bingley, R. M. (1990) Electronic theodolite intersection systems. PhD thesis, University of Nottingham.

University of Nottingham
Department of Mining Engineering
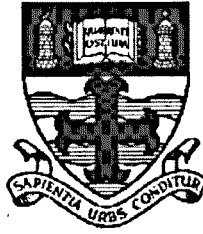
ELECTRONIC THEODOLITE INTERSECTION SYSTEMS

by

Richard Mark Bingley, BEng

Thesis submitted to the University of Nottingham for the degree of
Doctor of Philosophy

May 1990

ABSTRACT

Abstract

The development of electronic surveying instruments, such as electronic theodolites, and concurrent advances in computer technology, has revolutionised engineering surveying; one of the more recent examples being the introduction of Electronic Theodolite Intersection Systems (ETISs).

An ETIS consists of two or more electronic theodolites and a computer, with peripheral hardware and suitable software. The theoretical principles on which they are based have been known for a long time, but intersection has seldom been used as a method of measurement. The main reasons for its re-evaluation were the introduction of one-second electronic theodolites and the ability to interface these on-line to a computer.

The last decade has seen the development of several commercially available systems and probably even more in-house developed systems. Such systems are capable of performing real-time, non-contact, three-dimensional coordinate determination to a high accuracy, enabling their use in a wide variety of applications.

This thesis details all aspects of ETISs. Initially, the theoretical principles on which the systems are based are developed. The components of a system are then detailed and a review of current commercially available systems and their applications is given. The thesis then concentrates on the development of an ETIS by the author and details its' application in both industrial measurement and

deformation monitoring. Finally, the thesis concludes with a discussion on the factors affecting the accuracies attainable with ETISs.

CONTENTS

Contents

Chapter Three:
Components of an Electronic Theodolite Intersection System

Chapter Six:
Field Evaluation of UN ETIS for Industrial
Measurement

Chapter Eight:
Factors Affecting Accuracies Attainable with
Electronic Theodolite Intersection Systems

LIST OF FIGURES

List of Figures

Chapter Six

Appendix G

LIST OF TABLES

List of Tables

Chapter Seven

LIST OF PLATES

List of Plates

Chapter Three

Chapter Four

Chapter Five

Chapter Six

Chapter Seven

CHAPTER ONE

INTRODUCTION

Introduction

Over the last two decades, engineering surveying has been revolutionised by the development of electronic surveying instruments, such as electronic theodolites, and concurrent advances in computer technology. These developments led to the introduction of automated survey systems and, more recently, software-controlled measurement techniques, such as Electronic Theodolite Intersection Systems (ETISs).

The first ETISs were developed, for industrial measurement, by surveying equipment manufacturers in the early 1980s and since then there have been several commercially available systems developed. Such systems are capable of performing real-time, non-contact, three-dimensional coordinate determination to a high accuracy, enabling their use in a wide variety of applications.

In October 1986 the Department of Mining Engineering, at the University of Nottingham, was awarded a Science and Engineering Research Council grant (Grant Ref No: GR/D/86164) to obtain all the equipment and instrumentation necessary to develop an ETIS. The aims of the research performed by the author, and reported in this thesis, were to develop, and perform field trials with, the system. It is intended that this thesis should provide a reference on all aspects of ETISs, and should be especially useful for those wishing to develop, apply or operate such systems.

All ETISs are based on the same principles but differ in their specific components and the complexity of the analysis they perform. Their basic function is to determine three-dimensional point

coordinates to a high accuracy, typically 1 part in 200 000, ie 50 μm over a distance of 10 m. To fulfil this basic function, their main components are two, or more, electronic theodolites and an on-line computer, with peripheral hardware and suitable software.

Coordinate determination is based on the principle of three-dimensional spatial intersection, where an unknown point is intersected from at least two control points. To effect measurement, electronic theodolites are located above the control points and the coordinates of the unknown point are calculated from; the coordinates of the control points and the horizontal and vertical angles measured by the theodolites. In addition to three-dimensional point coordinates, the probable error and an ellipsoid of error about the point are usually calculated, to provide an indication of the accuracy of coordinate determination.

As well as the main components outlined above there are several peripheral components essential to the operation of an ETIS. These include theodolite stands, suitable targets for marking the points to be measured, and a calibrated scale bar. The main components of a system can be considered as fixed and should, therefore, be suitable for use in all of the proposed measurement tasks. The peripheral components of a system can be considered as variable. They are often the cheapest components and can, therefore, be selected to suit a specific measurement task.

In using a system, operators should be familiar with the principles of ETISs and their main components, and should be fully aware of the peripheral components available, and the considerations

which should be taken into account in their selection for an individual measurement survey. With this in mind, Chapter Two details the principles of ETISs and Chapter Three details the individual components of an ETIS, and outlines their role.

Before giving details of the system developed by the author, Chapter Four outlines the developments in commercially available systems which have taken place in the 1980s; from the early manual systems to the latest automated systems, and reviews the current systems and their applications.

Chapters Five, Six and Seven give details of the development of an ETIS, by the author, at the University of Nottingham. This system has been named "UN ETIS" (University of Nottingham Electronic Theodolite Intersection System).

In order that UN ETIS be flexible for use in a variety of measurement tasks the system was split into two separate halves; field components and office components, and the software was developed in-house. The development of the system began in October 1987 with the aims of:

(i) Selecting and purchasing the individual field and office components of the system.

(ii) Developing software on a field computer to perform the necessary field calculations and record the measured data for later analysis.

(iii) Developing software on an office computer to process, store and analyse the measured data.

Chapter Five outlines the selection and specification of the individual field and office components, and details the in-house development of the field and office software by the author. Once the main development of the system was complete the aims of the research were to:

(i) Test the field and office software under a range of operating parameters using theoretical values.

(ii) Become familiar with the system and develop the necessary observational skills before commencing field trials.

(iii) Investigate the use of the system in industrial measurement.

(iv) Perform field trials to monitor deformation due to mining.

An important activity in industrial processes is that of performing industrial measurement. For coping with measurement tasks that occur in industry, there is an ever increasing need for measuring equipment that is suitable for universal use, and offers high accuracy. Particularly in the case of large-scale objects, such as occur in aerospace and automotive engineering, and applications involving industrial robots, this need is paramount. Chapter Six details an example of industrial measurement using UN ETIS, for the calibration of production robots.

The increased possibility of mining affecting important structures or sensitive industrial processes, and the need to monitor the deformation of underground openings, necessitates short-range relative deformation monitoring to a high accuracy. The accuracy required is

usually sub-millimetre and, in the case of sensitive industrial processes, can be of the order of tens of microns, which is not attainable by conventional monitoring techniques. Chapter Seven concentrates on the short-range relative monitoring of the deformation of a sensitive industrial process due to mining. With reference to a specific example, the glass grinding and polishing process at Pilkington Glass Ltd in Doncaster, details are given of the field evaluation of UN ETIS and comparison is made between UN ETIS and a refined precise levelling technique.

Finally, the thesis concludes with a discussion on the factors affecting the ultimate accuracies attainable with ETISs. Chapter Eight considers these factors and the steps which can be taken to reduce, or eliminate, their influence.

The overall conclusions are summarised in Chapter Nine.

A considerable section of this thesis describes the development of field and office software for UN ETIS. Computer code has not been included in the text, since an in-depth knowledge of computing and of the language used would be required if this were to be an aid to understanding the text. Instead, the algorithms and principles on which they are based are discussed, since they can be more readily followed and adapted.

Published works relating directly to the subject of this thesis are referenced by the author's name and the year of publication, for example [Bingley 1987]. The complete list of references in alphabetical order is listed commencing on page 350. Where more than

one paper is referenced by the same author, they appear in chronological order. A bibliography of further works, not directly referenced in the text, is listed commencing on page 359.

Throughout this thesis a number of references are made to "the Department". This has been used as an abbreviation of "the Department of Mining Engineering, University of Nottingham".

CHAPTER TWO

PRINCIPLES OF ELECTRONIC THEODOLITE
INTERSECTION SYSTEMS

Principles of Electronic Theodolite Intersection Systems

2.1 Introduction

An ETIS enables the determination of three-dimensional point coordinates to a high accuracy. Coordinate determination is based on the principle of three-dimensional spatial intersection, where an unknown point is intersected from at least two control points. The coordinates of the unknown point are calculated from the coordinates of the control points and the observed horizontal and vertical angles subtended by the unknown point at each control point. In addition to three-dimensional point coordinates, the probable error and an ellipsoid of error about the point can be calculated to provide an indication of the accuracy of coordinate determination.

A measurement survey can be performed in a local coordinate system set up specifically for ETIS measurement, in a coordinate system defined by the object being measured or in the National Grid coordinate system. However, the first stage in any measurement survey is to determine the coordinates of the control points, in the chosen coordinate system, by a suitable setting up procedure. The setting up procedure used depends on the coordinate system and the nature of the measurement survey, and in some cases it may include a transformation from an initial local coordinate system to an object coordinate system.

This Chapter outlines the principles of coordinate measurement by an ETIS and the various methods of setting up a

9

coordinate system. The equations required at all stages of a measurement survey are detailed.

2.2 Coordinate Systems

A cartesian coordinate system (Figure 2.1) is defined by three planes. The three planes have a point O in common and intersect in pairs in three lines X'OX, Y'OY, Z'OZ. O is termed the origin, the three lines the coordinate axes and the three planes the coordinate planes.

A point P in a coordinate system may be defined by drawing three lines parallel to the coordinate axes; PL parallel to X'OX, PM parallel to Y'OY and PN parallel to Z'OZ. The plane MPN cuts OX at L', LPN cuts OY at M' and LPM cuts OZ at N', thus, defining a parallelepiped the faces of which are parallel to the coordinate planes, and edges parallel to the coordinate axes [Sommerville 1934]. Every point P has a unique set of coordinates (x, y, z) determined by the three lengths measured along the coordinate axes, x = OL', y = OM' and z = ON'.

For the purpose of ETIS measurement three coordinate systems are considered:

(i)   Local coordinate system.
(ii) Object coordinate system.
(iii) National Grid coordinate system.

10
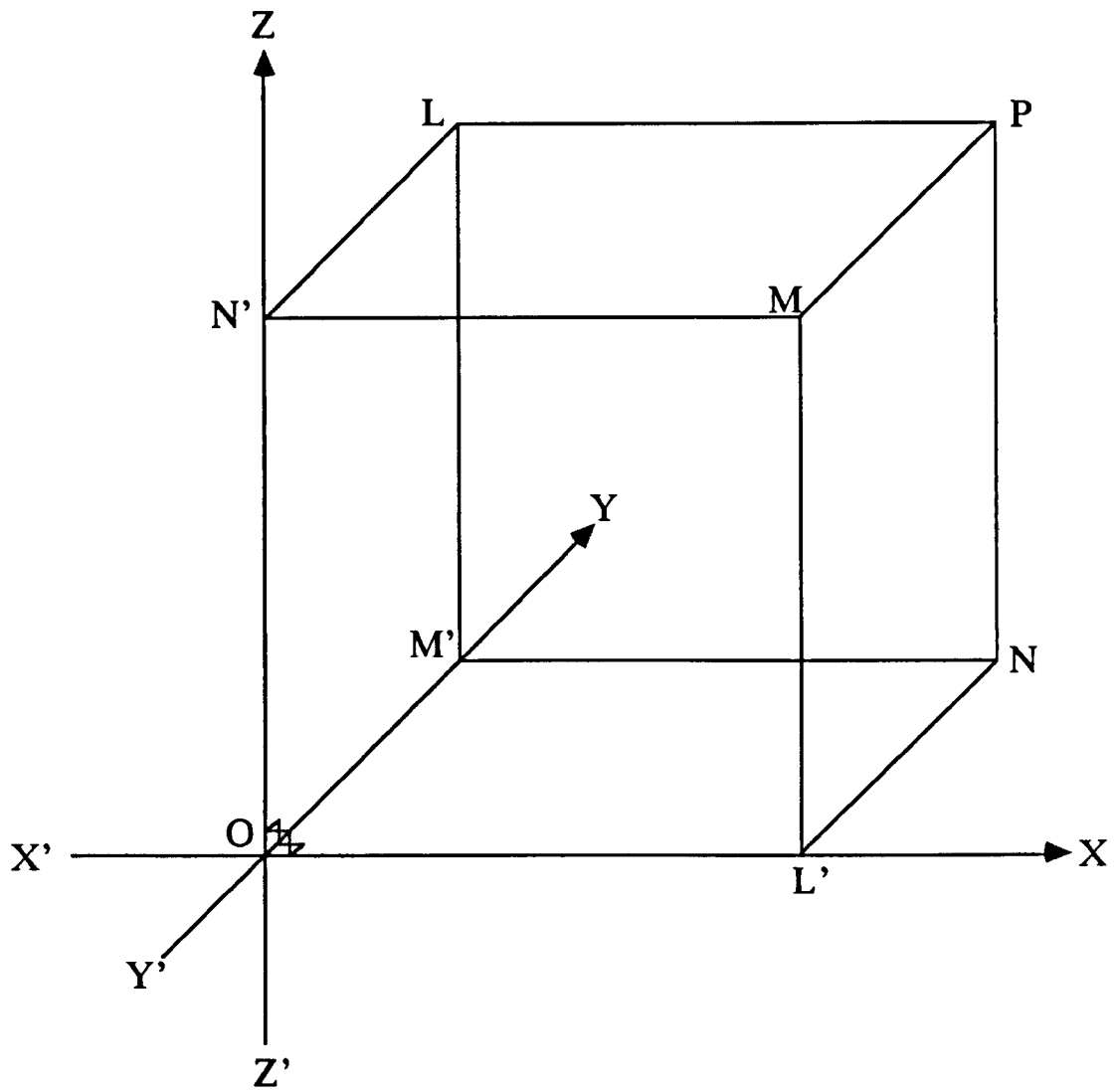
Figure 2.1 Cartesian Coordinate System.

11

2.2.1 Local Coordinate System

A local coordinate system can be defined by a baseline consisting of two control points which remain fixed throughout a measurement survey. The orientation of the baseline may be controlled by the orientation of the coordinate system or may be freely selectable. Figure 2.2 shows three possible baseline configurations for defining a local coordinate system. In each case, the baseline is formed from control point 1 (C_1) and control point 2 (C_2). The three configurations are:

(i) In the general case the baseline is not parallel to any axis of the local coordinate system and the coordinates of the two control points are (x_1, y_1, z_1) and (x_2, y_2, z_2) respectively.

(ii) If the orientation of the baseline is freely selectable it is convenient to consider the baseline as being parallel to the x-axis of the local coordinate system. In this case the y coordinates of the two control points are equal.

(iii) The simplest definition may be obtained by considering the first control point as the origin and the second control point as being on the x-axis of the local coordinate system. In this case the coordinates of control point 1 are (0, 0, 0) and the coordinates of control point 2 are (x_2, 0, z_2).

A local coordinate system is used for:

- industrial measurement, where object dimensions derived from local coordinates are sufficient.
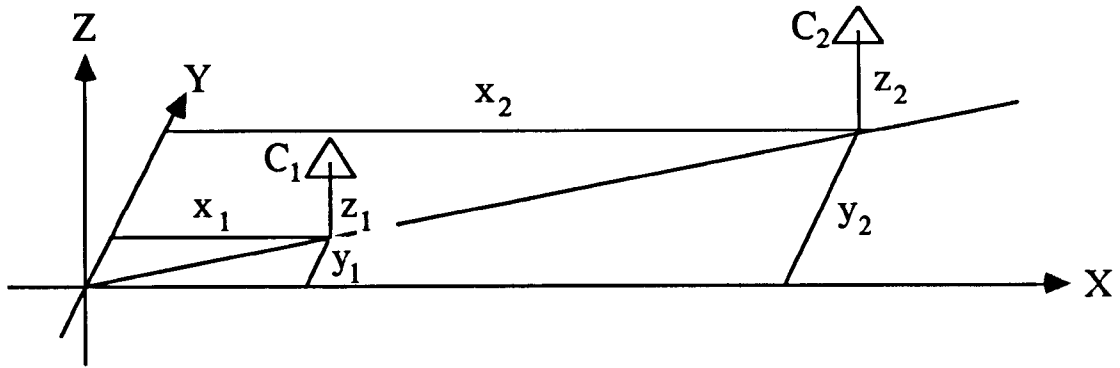
(i)   General case.
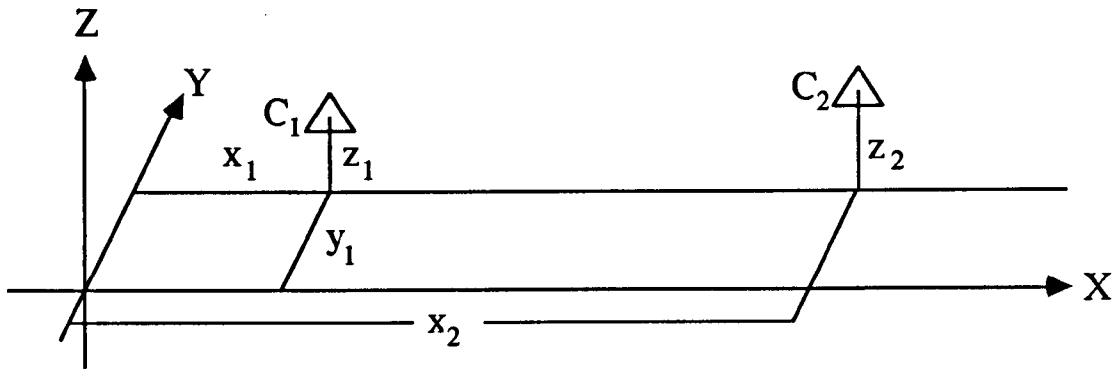
(ii)   Baseline parallel to the x-axis (ie $y_1 = y_2$).
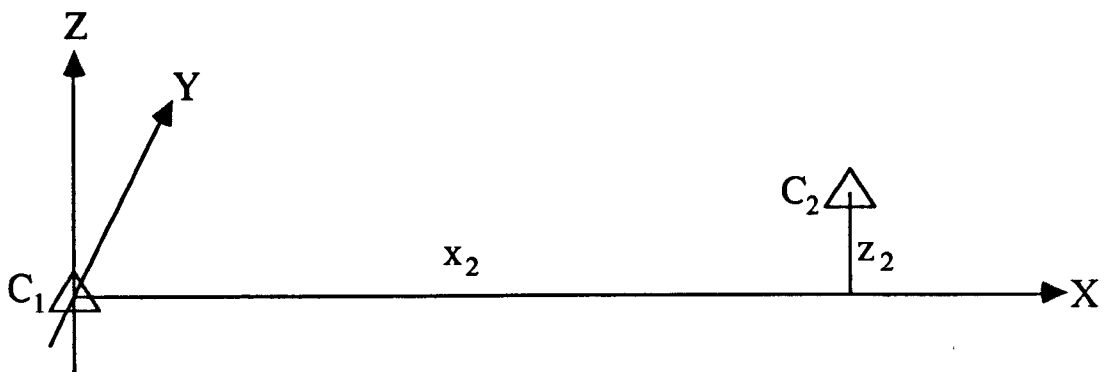
(iii) Baseline on the x-axis, control point 1 as origin (ie $y_1 = y_2 = 0$).

Figure 2.2 Local Coordinate System Configurations.

13

- relative deformation monitoring, where periodic measurement of relative dimensions is required.

- input to a transformation in to another coordinate system.

2.2.2 Object Coordinate System

An object coordinate system is defined by the object being measured. An object coordinate system is used for:

- industrial measurement, where the object coordinate system is defined by dimensions on a technical drawing (Figure 2.3(i)).

- periodic industrial measurement, where the object coordinate system is defined by an initial survey local coordinate system in to which subsequent survey local coordinate systems are transformed.

- measuring a large object, where it is necessary to perform several surveys, in separate local coordinate systems, and transform all in to a standardised object coordinate system (Figure 2.3(ii)).

- absolute deformation monitoring, where periodic measurement of absolute coordinates is required.

2.2.3 National Grid Coordinate System

When survey measurements are performed between points some distance apart, the irregular nature of the earth's topographic surface prevents it being used as a reference system for defining points near to it. This surface is almost an oblate spheroid and is represented by the geoid, defined as the equipotential surface arising from both the
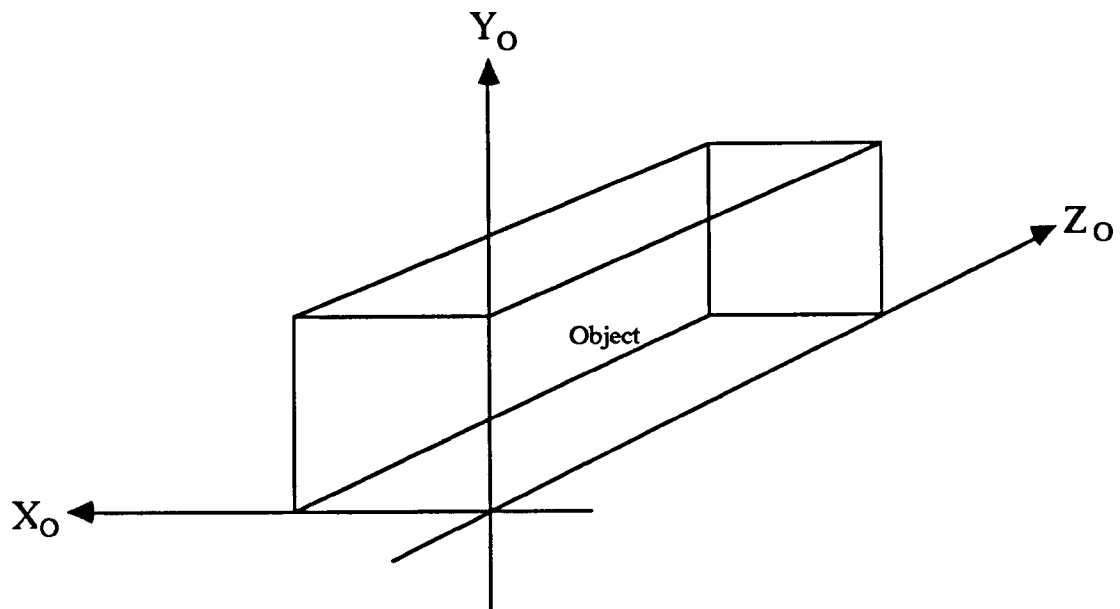
(i) Object coordinate system defined by dimensions on a technical drawing.
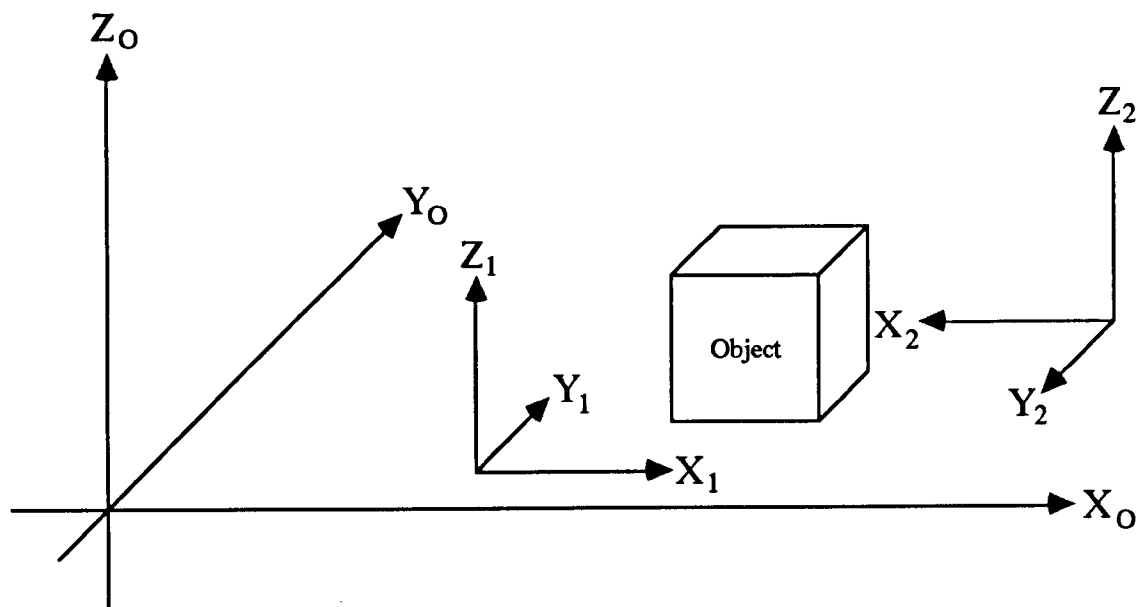
(ii) Measuring a large object, where it is necessary to perform two surveys in separate local coordinate systems and transform both in to a standardised object coordinate system.

Figure 2.3 Object Coordinate System Configurations.

15

earth's attraction and rotation which coincides with the mean level of the sea. In the UK the National Grid is the National Reference coordinate system, which is based on the Airy reference spheroid and the Transverse Mercator projection [Cooper 1974].

In the National Grid coordinate system, certain corrections have to be made to measured angles and distances before they are used in the computation of coordinates. Measured angles have to be corrected for angular distortion and slope distances have to be reduced to National Grid distances. However, the correction for angular distortion, the (t - T) correction, is approximately 1"/km and can, therefore, be ignored with sight lengths of less than 1 km.

Long-range ETIS measurement, including sight lengths greater than 100 m, can be performed in the National Grid coordinate system for:

- non-contact measurement of coordinates and relative distances in the National Grid.
- absolute deformation monitoring of ground and structures which have to be correlated to the National Grid. For example, monitoring subsidence deformations resulting from underground mineral extraction.

There is, however, a problem with using the National Grid coordinate system for ETIS measurement, as the accuracy of the measurement survey is dependent on the accuracy of the determined control point coordinates. If control point coordinates are determined from ordnance survey pillars, by conventional triangulation or

16

resection, then their accuracy will depend on their position relative to the pillars and the accuracy of the pillars themselves. In practice the accuracy of ETIS measurement can be maintained over long range by setting up an approximate National Grid coordinate system.

An approximate National Grid coordinate system can be set up by assuming one control point is fixed and measuring the slope distance between control points to a high accuracy. The x, y coordinates of the fixed control point and the orientation of any baselines with respect to true North can be approximated from an Ordnance Survey map. The z coordinate of the fixed control point can be obtained from a local bench mark or from contour data. The x, y coordinates are used to determine the Local Scale Factor and Local Mean Radius of the earth and only need to be approximated to an accuracy of ±100 m. The z coordinate is more critical, it affects the reduction of slope distances to National Grid distances and has to be approximated to an accuracy of ±1 m.

2.3 Setting Up a Coordinate System

The first stage in a measurement survey is to set up a coordinate system. Setting up involves calculating the coordinates of a minimum of two control points in a specific coordinate system. The method used depends on the measurement task and the coordinate system required. There are five methods detailed in this section, namely, setting up by:

(i) Known control point coordinates.

(ii) Accurate distance measurement between control points.

(iii) The use of a calibrated scale bar.

(iv) Resection from points of known coordinates.

(v) The bundle adjustment.

In all cases, except the bundle adjustment, a coordinate system is set up by establishing a series of baselines. Each baseline consists of two control points and it is necessary to define a horizontal reference angle for each control point on the baseline.

2.3.1 Horizontal Reference Angle Baseline

In calculating the coordinates of an unknown point, the horizontal and vertical angles measured from at least two control points are used. The vertical angles are measured with respect to the zenith and the horizontal angles with respect to a horizontal reference angle.

Considering a local coordinate system with two control points, only one baseline is established and it is convenient to define the horizontal reference angles as zero for both control points.

If more than two control points are used, a number of baselines are established and it is necessary to define the horizontal reference angle for each control point on a specific baseline. This is best illustrated by an example; considering the three control points A, B and C in Figure 2.4. In this case two baselines are established, AB and BC. The initial baseline AB is established with horizontal reference angles of zero for both control points. The second baseline BC is
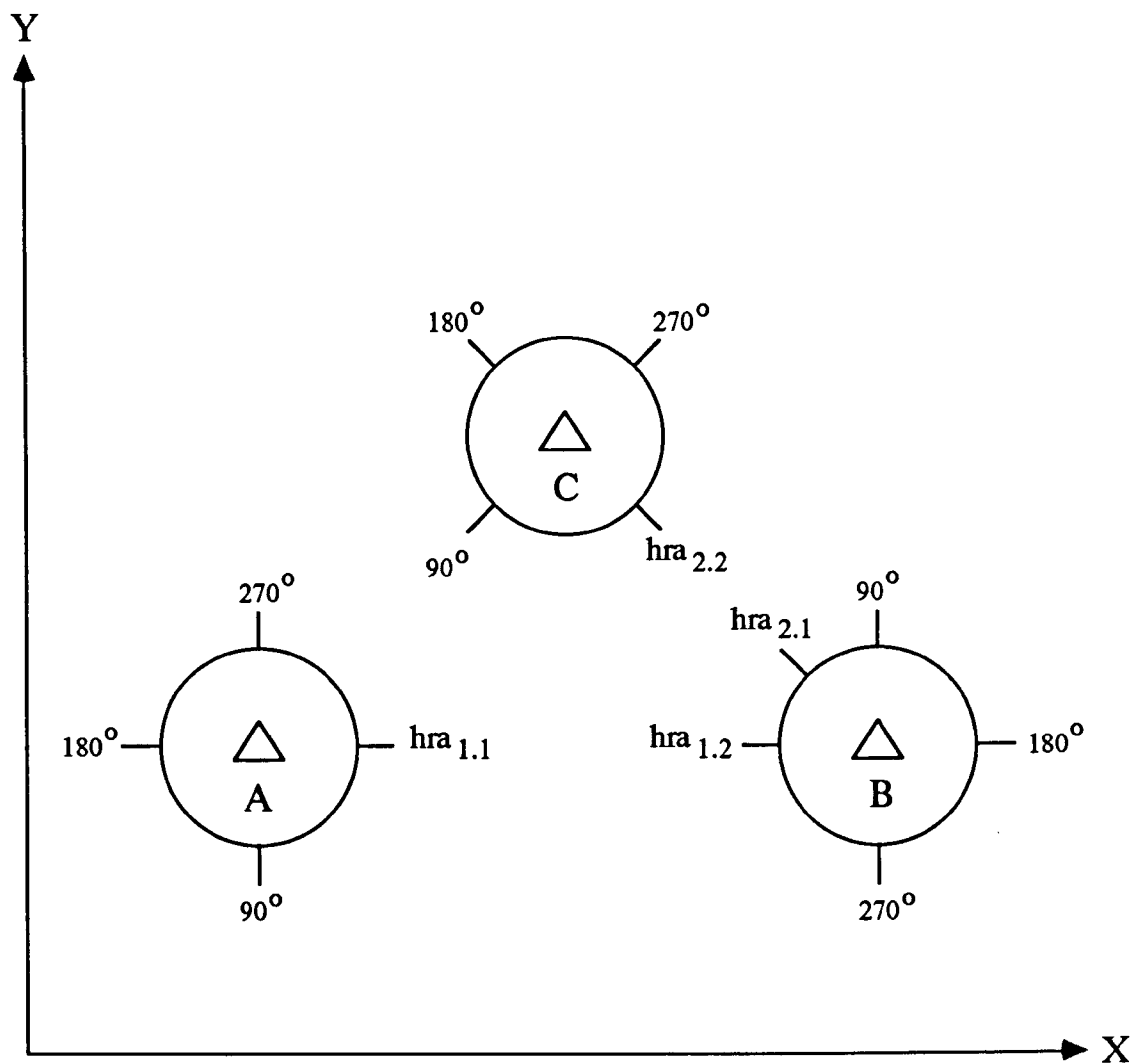
$$\text{hra}_{1.1} = \text{horizontal reference angle of baseline 1, control point 1.}$$

$$\text{hra}_{1.2} = \text{horizontal reference angle of baseline 1, control point 2.}$$

$$\text{hra}_{2.1} = \text{horizontal reference angle of baseline 2, control point 1.}$$

$$\text{hra}_{2.2} = \text{horizontal reference angle of baseline 2, control point 2.}$$

Figure 2.4 Horizontal Reference Angle Baseline.

established with the horizontal reference angle at C defined as zero and the horizontal reference angle at B defined by the horizontal circle reading with respect to the previous baseline AB, in this case 045 degrees.

For any number of additional control points, the respective subsequent baselines are established in the same way as the second baseline BC.

2.3.2 Known Control Point Coordinates

The simplest setting up option available is to use known control points. The coordinates of the control points can be known or determined in an initial survey. Subsequent measurement surveys can be performed quicker and easier as the setting up of a new coordinate system is not required.

This option is especially useful if periodic surveys are being performed to the same object. The disadvantage of using this option is that flexibility in measurement is reduced. It is also advisable to perform periodic re-setting of the coordinate system to check the stability of the known control points.

2.3.3 Accurate Distance Measurement Between Control Points

A coordinate system can be set up by measuring the slope distance between control points to a high accuracy, by laser interferometry or Electronic Distance Measurement (EDM).

For each baseline one control point (x_1, y_1, z_1) is considered fixed, the coordinates of the second control point (x_2, y_2, z_2) are unknown and the azimuth of the baseline is controlled by the orientation of the baseline in the coordinate system. The coordinates of the unknown control point are obtained, from Figure 2.5, as follows

$$x_2 = x_1 + sd \cdot \cos v \cdot \sin az$$
$$y_2 = y_1 + sd \cdot \cos v \cdot \cos az$$
$$z_2 = z_1 + sd \cdot \sin v + (h_1 - h_2)$$

where

sd = slope distance.

v = reciprocal vertical angle.

az = azimuth of baseline.

h_1 = instrument height at control point 1.

h_2 = instrument height at control point 2.

Considering a local coordinate system with two control points it is convenient to assume the baseline is parallel to the x-axis of the coordinate system. The solution then reduces to

$$x_2 = x_1 + sd \cdot \cos v$$
$$y_2 = y_1$$
$$z_2 = z_1 + sd \cdot \sin v + (h_1 - h_2)$$

If more than two control points are used the setting up procedure is repeated for the series of baselines. For any number of baselines, the baseline azimuth is determined with respect to the

21
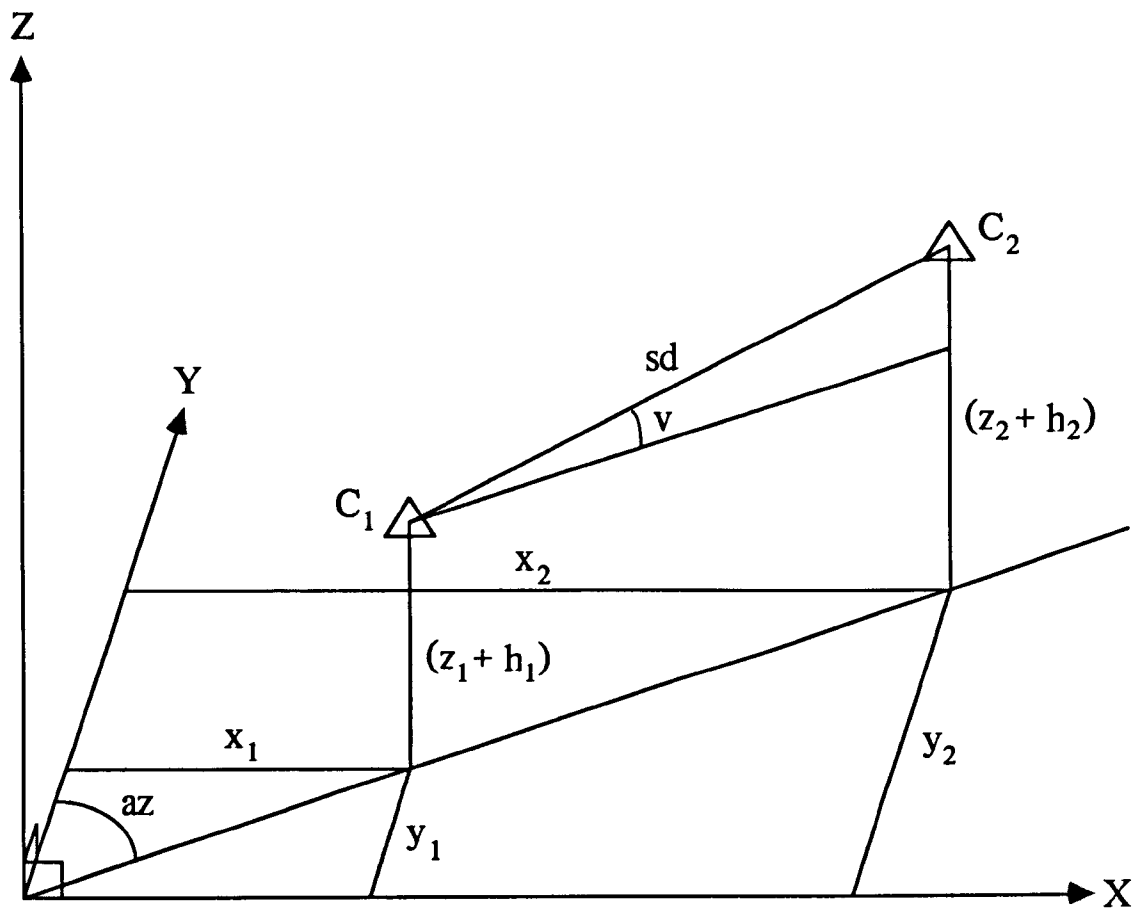
Figure 2.5 Setting up by Accurate Distance Measurement Between Control Points.

previous baseline and in a local coordinate system it is convenient to assume that the initial baseline is parallel to the x-axis of the coordinate system.

2.3.4 The Use of a Calibrated Scale Bar

The most common method of setting up a local coordinate system is by the use of a calibrated scale bar, the exact length of which is known to a high accuracy. Setting up is performed in two stages:

       (i)   Setting up of an approximate local coordinate system.

       (ii) Scaling up of the approximate local coordinate system to an accurate local coordinate system.

For each baseline one control point (x_1, y_1, z_1) is considered fixed, the coordinates of the second control point (x_2, y_2, z_2) are unknown and the azimuth of the baseline is controlled by the orientation of the baseline in the coordinate system. The approximate coordinates of the second control point (x_2', y_2', z_2') are determined as in section 2.3.3, but the distance between the two control points is estimated, as opposed to being accurately measured.

The scaling up of the approximate local coordinate system to an accurate local coordinate system is performed by simultaneous sightings to the two end points on a calibrated scale bar (Figure 2.6). The scaling up procedure can be repeated several times with the scale bar in a different position in the measurement space.

23
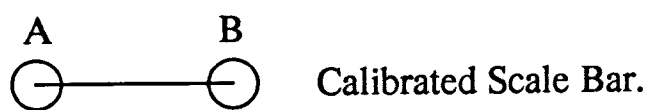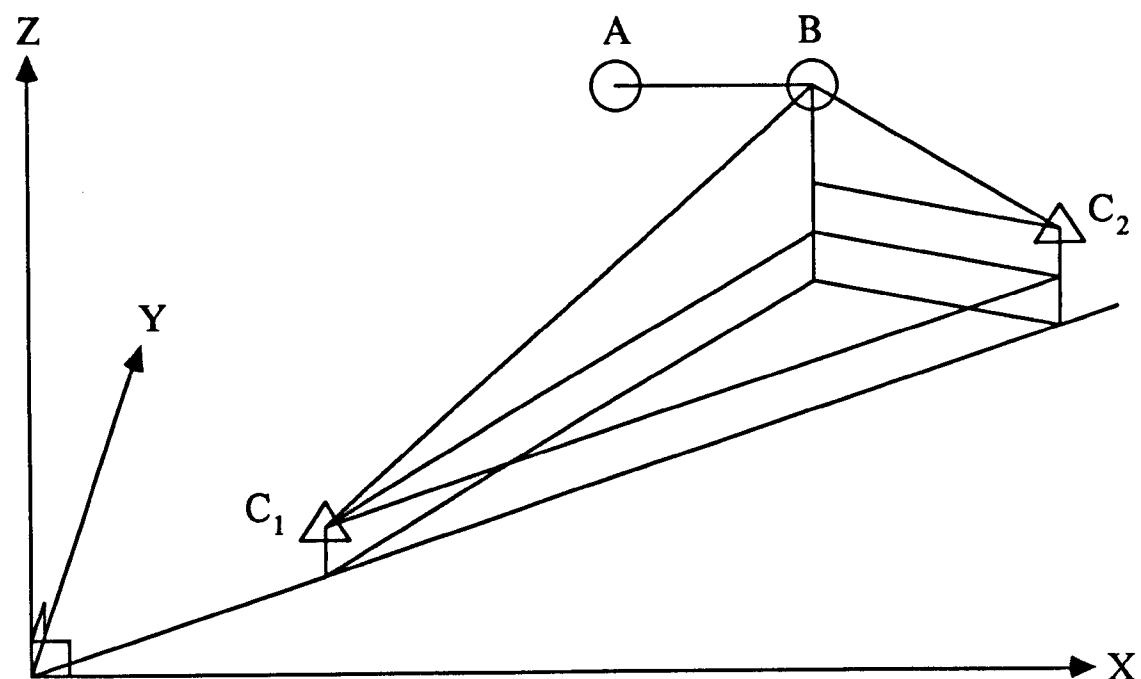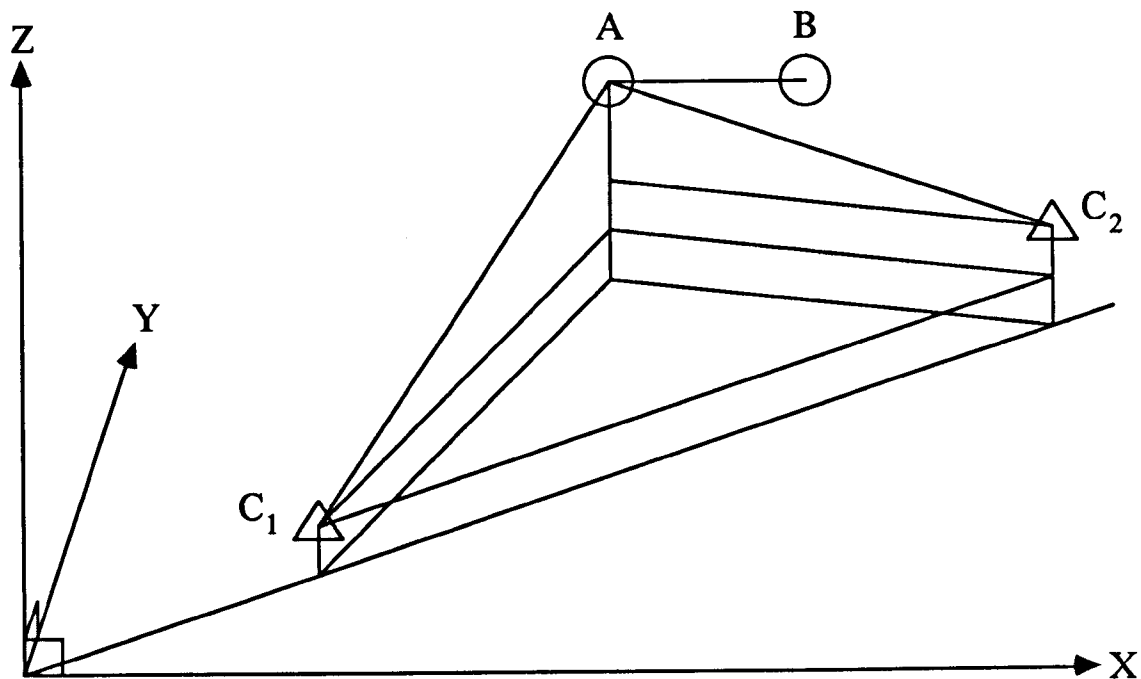
Figure 2.6 Setting up by the use of a Calibrated Scale Bar.

24

The scaled up coordinates of the second control point (x_2, y_2, z_2) are determined by calculating a scale factor (sf). The scale factor is calculated from the scale bar length and the coordinates of the end points on the scale bar (x_A, y_A, z_A) and (x_B, y_B, z_B) as follows

$$sf = \frac{\text{scale bar length}}{\sqrt{(x_A - x_B)^2 + (y_A - y_B)^2 + (z_A - z_B)^2}}$$

$$x_2 = x_1 + sf \cdot (x_2' - x_1)$$
$$y_2 = y_1 + sf \cdot (y_2' - y_1)$$
$$z_2 = z_1 + sf \cdot (z_2' - z_1 - (h_1 - h_2)) + (h_1 - h_2)$$

2.3.5 Resection from Points of Known Coordinates

Resection from any number of control points to at least three fixed points, of known coordinates, enables the calculation of the coordinates of the control points. The coordinates of a control point are calculated in two stages, calculation of the:

(i) x, y coordinates by a three point resection.

(ii) z coordinate from observed vertical angles.

A three point resection requires the known x, y coordinates of three fixed points A, B and C and two measured angles α and β. The solution involves calculating the coordinates of an auxiliary point D in the first instance, and using these values as a means of orienting the observations at the control point. The geometry of a three point resection is illustrated in Figure 2.7 and the sequence of calculations is as follows:
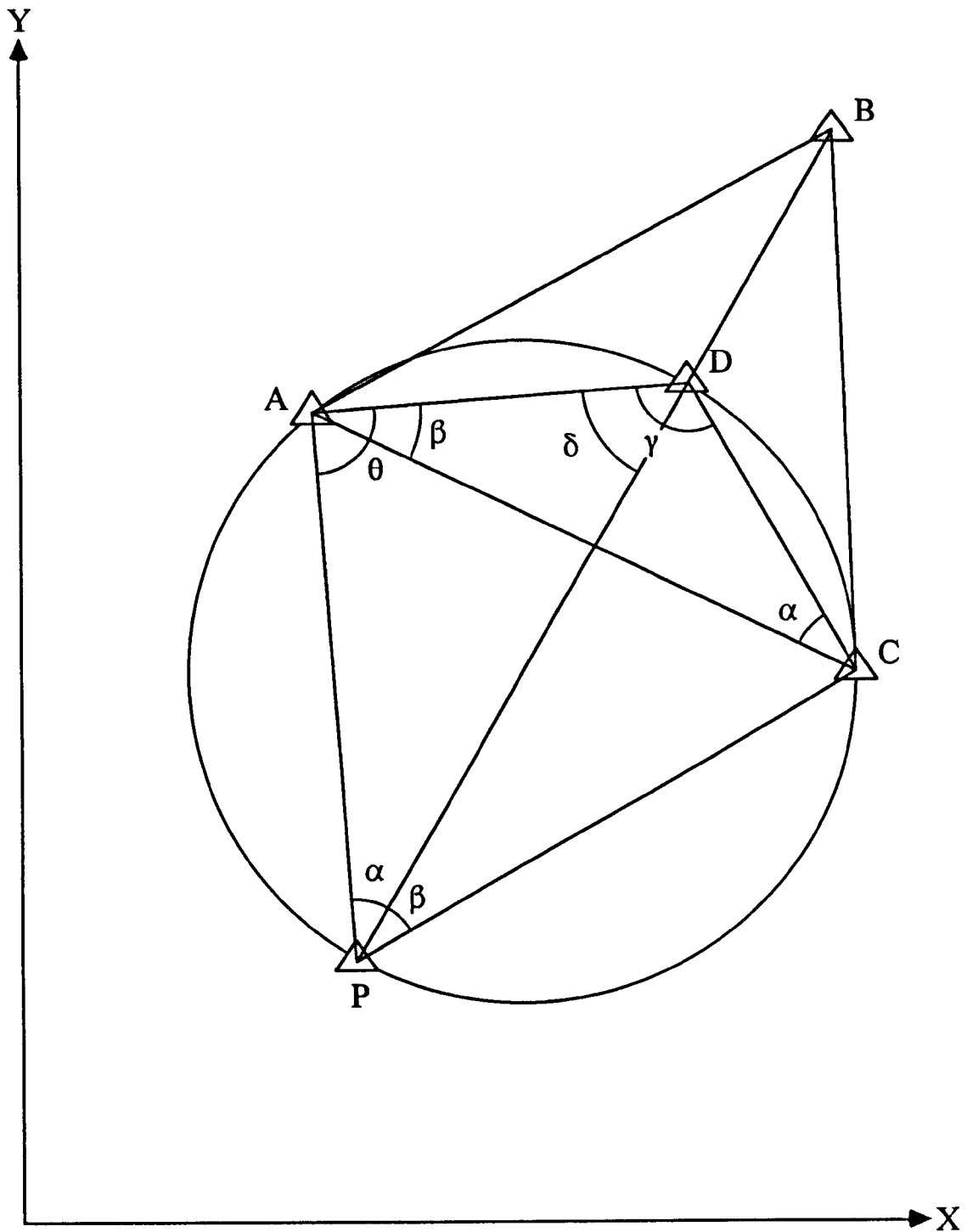
Figure 2.7 Three Point Resection.

26

(i) Calculate the angle γ.

$\gamma = 180 - (\alpha + \beta)$

(ii) Calculate the azimuth α_{AC} and distance d_{AC}.

$\alpha_{AC} = \tan^{-1}\dfrac{(x_A - x_C)}{(y_A - y_C)}$

$d_{AC} = \sqrt{(x_A - x_C)^2 + (y_A - y_C)^2}$

(iii) Calculate the azimuth α_{AD} and distance d_{AD}.

$\alpha_{AD} = \alpha_{AC} - \beta$

$d_{AD} = d_{AC} \cdot \dfrac{\sin \alpha}{\sin \gamma}$

(iv) Calculate the x, y coordinates of point D.

$x_D = x_A + d_{AD} \cdot \sin \alpha_{AD}$

$y_D = y_A + d_{AD} \cdot \cos \alpha_{AD}$

(v) Calculate the azimuth α_{DB}.

$\alpha_{DB} = \tan^{-1}\dfrac{(x_D - x_B)}{(y_D - y_B)}$

(vi) Calculate the angles δ and θ.

$\delta = \alpha_{DA} - \alpha_{BD}$

$\theta = 180 - (\alpha + \delta)$

(vii) Calculate the azimuth α_{DP} and distance d_{DP}.

$\alpha_{DP} = \alpha_{BD}$

$d_{DP} = d_{AD} \cdot \dfrac{\sin \theta}{\sin \alpha}$

(viii) Calculate the x, y coordinates of control point P.

$x_P = x_D + d_{DP} \cdot \sin \alpha_{DP}$

$y_P = y_D + d_{DP} \cdot \cos \alpha_{DP}$

Calculation of the z coordinate of a control point is performed to each of the fixed points, for example control point 1, resection point A, as follows:

(i) Calculate the plan distance from known x, y coordinates.

$$pd_A = \sqrt{(x_A - x_1)^2 + (y_A - y_1)^2}$$

(ii) Correct the vertical angle for curvature and refraction.

(iii) Calculate the z coordinate.

$$z_1 = z_A - pd_A . \tan v_1 + (h_1 - h_A)$$

where

pd_A = plan distance.

v_1 = corrected vertical angle.

h_1 = instrument height at control point 1.

h_A = target height at resection point A.

The final z coordinate of the control point is calculated as the mean of the z coordinates derived from each of the fixed resection points.

2.3.6 The Bundle Adjustment

The concept of the bundle adjustment has long been understood by land surveyors and photogrammetrists, and has been used in industrial measurement involving metric cameras for a number of years. The bundle adjustment is a simultaneous, three-dimensional, adjustment of all the observations in a measurement network. In the case of an ETIS, the observations are the horizontal and vertical angles from each of the control points to each of the bundle points sighted by that control point. The adjustment is based upon minimising the

pointing errors that occur when the observed lines of sight do not exactly coincide at the target [Grist 1986a].

The mathematical model behind the bundle adjustment is a set of equations called the collinearity equations. These equations state that a unit vector formed by the observed horizontal and vertical angles is collinear with a ray from the nodal point of the theodolite through the target. If all observations were made without error, there would be no need for adjustment. The bundle adjustment computes the positions of the control points and the coordinates of a number of common, but not necessarily defined, bundle points that minimise these observational errors using the method of least squares. This technique guarantees the best fit of the measuring system to the coordinate system of the object to be measured.

The bundle adjustment determines the position and orientation of each control point relative to the coordinate system. Figure 2.8 shows the unknown parameters for a two control point set up, each control point is defined by six parameters; three coordinates (x, y, z) and three rotations (ω, ϕ, κ).

For a two control point set up there are twelve unknown control point parameters, however, if setting up in a local coordinate system all of the parameters for one control point can be assumed equal to zero, by defining the control point as the origin of the local coordinate system. This assumption reduces the number of unknowns to six. Each measurement to a bundle point introduces two equations for each control point and three further unknowns, the coordinates of the bundle point. Therefore, each bundle point adds one more equation
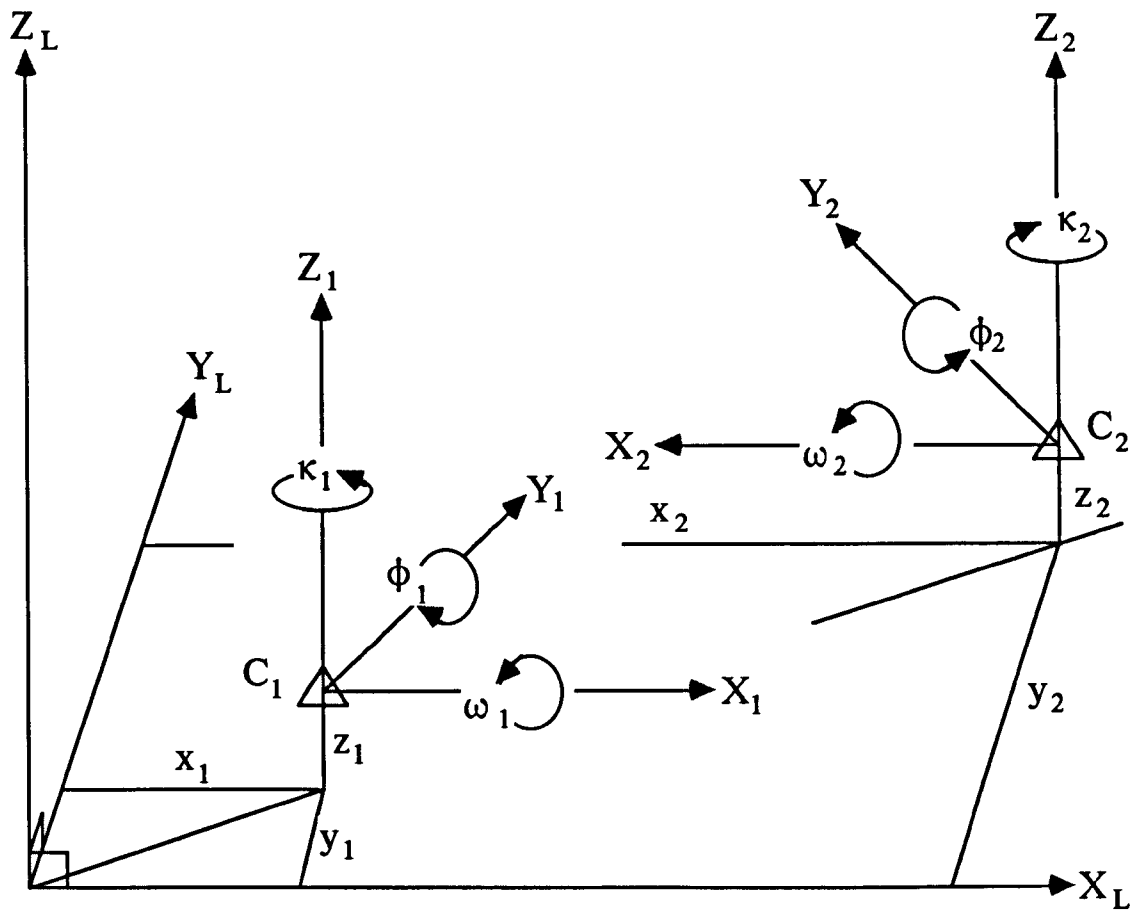
X_L, Y_L, Z_L	Local coordinate system axes.
X_1, Y_1, Z_1	Control point 1 coordinate system axes.
X_2, Y_2, Z_2	Control point 2 coordinate system axes.
x_1, y_1, z_1	Local coordinates of control point 1.
x_2, y_2, z_2	Local coordinates of control point 2.
ω_1, ϕ_1, κ_1	Rotation angles at control point 1.
ω_2, ϕ_2, κ_2	Rotation angles at control point 2.

Figure 2.8 Bundle Adjustment Control Point Parameters.

than the number of unknowns that it introduces. If five bundle points are observed, enough information is obtained to solve for the orientation of the control points, provided that two of the bundle points are points on a calibrated scale bar. This scale constraint allows the bundle adjustment to solve for the remaining unknown in the system.

The bundle adjustment also permits the direct setting up in an object coordinate system. Instead of assuming that one control point is the origin, the object coordinate system can be defined by sighting to bundle points of known coordinates, thus allowing the direct computation of the coordinates of the control points. In fact, any combination of assumptions about the control points and bundle points can be used to define the object coordinate system in whatever way best suits the measurement task.

If the number of bundle points is increased beyond the minimum required, the redundancy of the solution is increased and subsequently the accuracy of coordinate system set-up is increased.

Since the adjustment is based on sighting common bundle points, there is no requirement of intervisibility between any of the control points and no restriction on how close the control points are located to each other.

2.4 Coordinate Measurement by an ETIS

Once the setting up of a coordinate system is complete, and the control point coordinates are known, measurement can be performed to points in the coordinate system. The determination of

point coordinates is based on the principle of three-dimensional spatial intersection, in which an unknown target point is intersected from at least two control points of known coordinates.

The full calculation of coordinates by an ETIS includes:

(i) Calculation of three-dimensional point coordinates by spatial intersection.

(ii) A least squares adjustment of coordinates.

(iii) Calculation of the probable error.

(iv) Calculation of an ellipsoid of error.

For ease of calculation a single baseline consisting of two control points; control point 1 (C_1) and control point 2 (C_2), is considered at all stages of calculation.

2.4.1 Three-Dimensional Spatial Intersection

The coordinates of a target point are calculated from the coordinates of the control points and the observed horizontal and vertical angles subtended by the target point at each control point, as shown in Figure 2.9.

The first stage in the calculation is to convert the zenith angle and horizontal angle, observed at each control point, to a vertical angle and azimuth. For two control points, depending on the face of the theodolites the calculation is
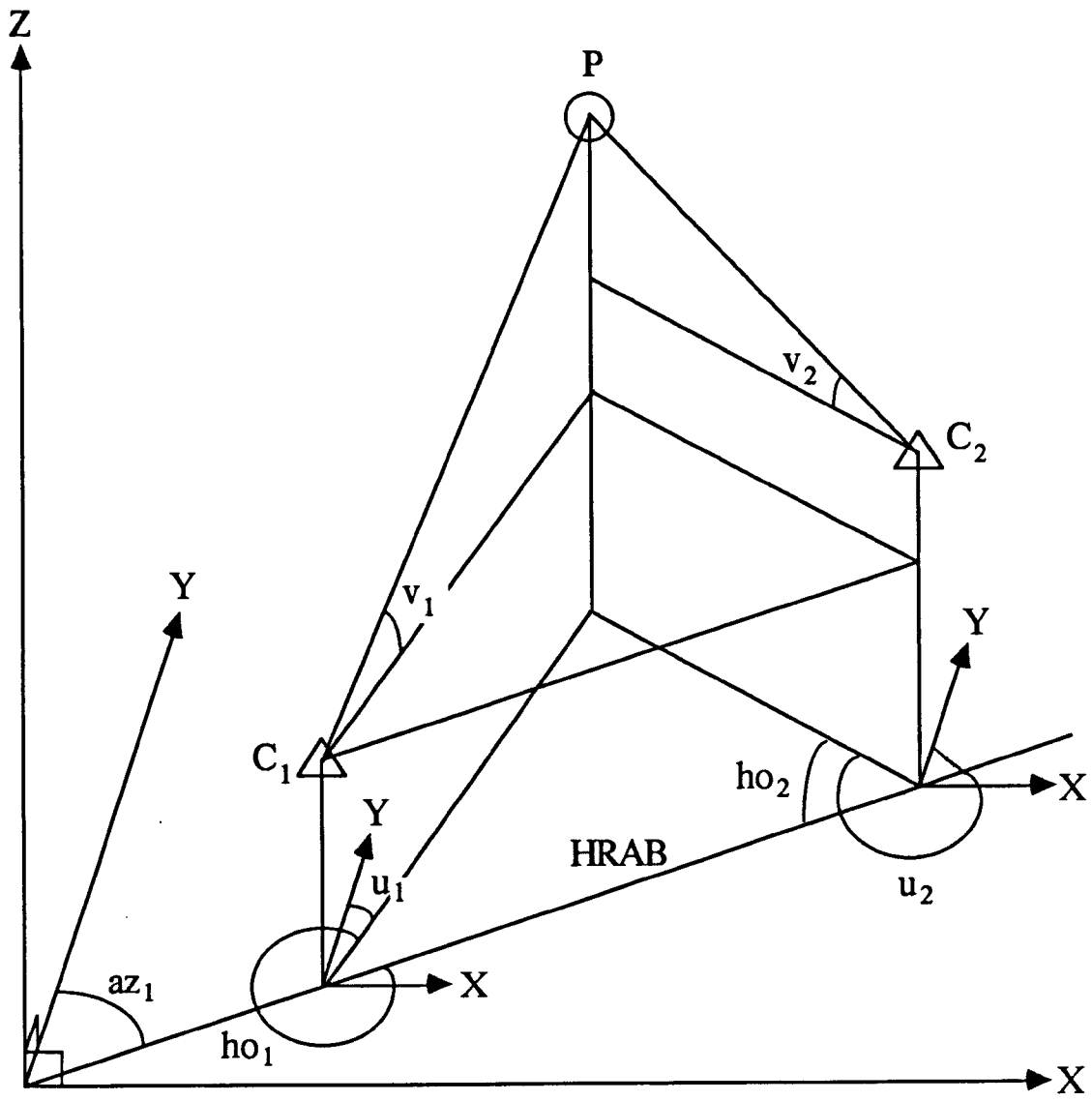
HRAB = Horizontal Reference Angle Baseline.

Figure 2.9 Three-Dimensional Spatial Intersection.

Control Point 1	Control Point 2

Theodolite Face Left

$$v_1 = 90° - ze_1 \qquad v_2 = 90° - ze_2$$

$$ho_1 = hc_1 - hra_1 \qquad ho_2 = hc_2 - hra_2$$

$$u_1 = ho_1 + az_1 \qquad u_2 = ho_2 + az_2$$

Theodolite Face Right

$$v_1 = ze_1 - 270° \qquad v_2 = ze_2 - 270°$$

$$ho_1 = hc_1 - hra_1 - 180° \qquad ho_2 = hc_2 - hra_2 - 180°$$

$$u_1 = ho_1 + az_1 \qquad u_2 = ho_2 + az_2$$

Equation 2.1

where

v = measured vertical angle.

ze = observed zenith angle.

u = measured azimuth (control point to target point).

ho = observed horizontal angle.

hc = theodolite horizontal circle reading.

hra = horizontal reference angle.

az_1 = baseline azimuth (control point 1 to control point 2).

az_2 = baseline azimuth (control point 2 to control point 1).

$\qquad = 180° + az_1$

The second stage in the calculation is to convert the measured vertical angles and azimuths to direction cosines (l, m, n). The direction cosines of the lines from control point to target point form the direction cosine equations

	Control Point 1		Control Point 2

$$l_1 = \cos v_1 \cdot \sin u_1 \qquad\qquad l_2 = \cos v_2 \cdot \sin u_2$$

$$m_1 = \cos v_1 \cdot \cos u_1 \qquad\qquad m_2 = \cos v_2 \cdot \cos u_2$$

$$n_1 = \sin v_1 \qquad\qquad\qquad n_2 = \sin v_2$$

Equation 2.2

The coordinates of the target point (x_P, y_P, z_P) can be calculated from each control point. The calculation of coordinates requires the coordinates of the control points, the direction cosines and the slope distances as follows

$$x_P = x_1 + l_1 \cdot t_1 \qquad\qquad x_P = x_2 + l_2 \cdot t_2$$

$$y_P = y_1 + m_1 \cdot t_1 \qquad\qquad y_P = y_2 + m_2 \cdot t_2$$

$$z_P = z_1 + n_1 \cdot t_1 \qquad\qquad z_P = z_2 + n_2 \cdot t_2$$

Equation 2.3

where

t_1 = slope distance (control point 1 to target point).

t_2 = slope distance (control point 2 to target point).

However, before the coordinates of the target point can be calculated the two slope distances have to be calculated, for this calculation there can be two cases [Allan 1987];

(i) the two lines intersect, or

(ii) the two lines do not intersect.

Considering the first case, if the two lines intersect, the length of the normal to both lines is zero and the two slope distances can be calculated by solving the coordinate equations (equation 2.3) as follows

$$t_1 = \frac{l_2 \cdot (y_1 - y_2) + m_2 \cdot (x_2 - x_1)}{(l_1 \cdot m_2 - l_2 \cdot m_1)}$$

$$t_2 = \frac{l_1 \cdot (y_1 - y_2) + m_1 \cdot (x_2 - x_1)}{(l_1 \cdot m_2 - l_2 \cdot m_1)}$$

However, in practice the two lines will only intersect by chance, so we must consider the case where the lines do not intersect. This results in two target points; P_1 from control point 1 (C_1) and P_2 from control point 2 (C_2). The geometry of non-intersecting lines is shown in Figure 2.10, where the following geometrical relationships can be defined:

(i)   The vector C_1C_2 mapped on to C_1P_1 is p

$p = (l_1, m_1, n_1) \cdot (x_2 - x_1, y_2 - y_1, z_2 - z_1)$ Equation 2.4

(ii) The vector C_1C_2 mapped on to P_2C_2 is q

$q = (l_2, m_2, n_2) \cdot (x_2 - x_1, y_2 - y_1, z_2 - z_1)$ Equation 2.5

(iii) The three-dimensional intersection angle (c) between the two lines is given by

$\cos c = (l_1, m_1, n_1) \cdot (l_2, m_2, n_2)$ Equation 2.6

(iv) It is also known that

$1 = (l_1, m_1, n_1) \cdot (l_1, m_1, n_1)$

36
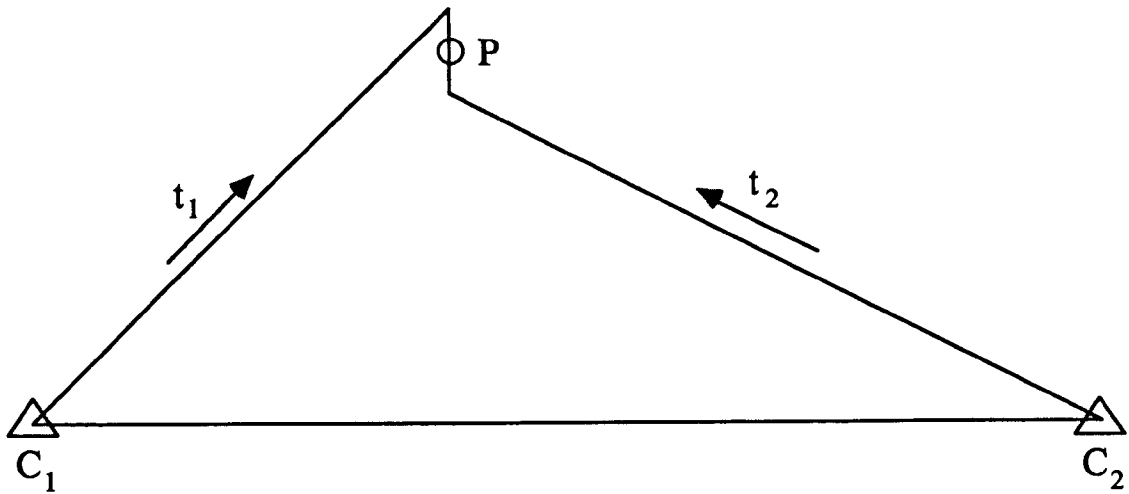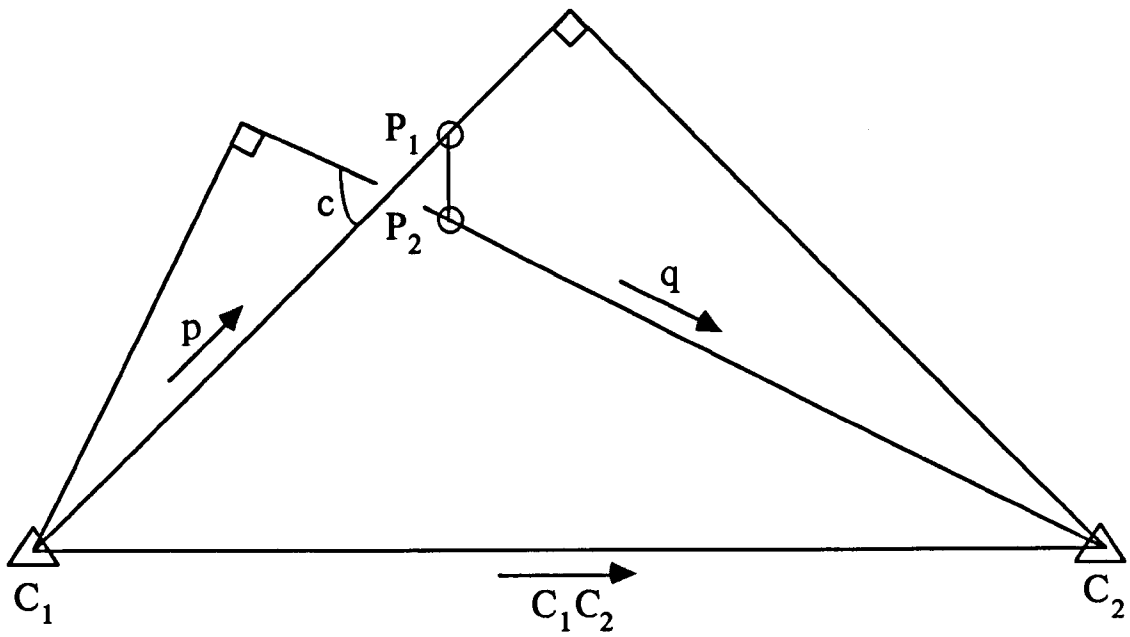
Figure 2.10 Geometry of Non-Intersecting Lines.

37

$$1 = (l_2, m_2, n_2) \cdot (l_2, m_2, n_2) \qquad \text{Equation 2.7}$$

From the coordinate equations (equation 2.3) for target points P_1 and P_2 the common normal P_1P_2 is given by

$$P_1P_2 = (x_2 - x_1, y_2 - y_1, z_2 - z_1) +$$
$$t_2 \cdot (l_2, m_2, n_2) - t_1 \cdot (l_1, m_1, n_1) \qquad \text{Equation 2.8}$$

P_1P_2 is perpendicular to both C_1P_1 and C_2P_2 therefore

$$P_1P_2 \cdot (l_1, m_1, n_1) = 0$$
$$P_1P_2 \cdot (l_2, m_2, n_2) = 0 \qquad \text{Equation 2.9}$$

Combining equations 2.8 and 2.9 and considering the geometrical relationships given in equations 2.4 to 2.7, gives

$$p + t_2 \cdot \cos c - t_1 = 0 \qquad \text{Equation 2.10}$$
$$q + t_2 - t_1 \cdot \cos c = 0 \qquad \text{Equation 2.11}$$

Equations 2.10 and 2.11 can be solved simultaneously for the slope distances t_1 and t_2 to give

$$t_1 = \frac{(p - q \cdot \cos c)}{\sin^2 c} \qquad \text{Equation 2.12}$$
$$t_2 = \frac{(p \cdot \cos c - q)}{\sin^2 c} \qquad \text{Equation 2.13}$$

Equation 2.13 can be simplified further by considering equation 2.12 to give

$$t_2 = t_1 . \cos c - q \qquad \text{Equation 2.14}$$

From the direction cosine equations (equation 2.2), the coordinate equations (equation 2.3) and the slope distance equations (equations 2.12 and 2.14) the coordinates of P_1 and P_2 can be determined. The mean of these two sets of coordinates are then taken as the raw coordinates of the target point P

$$x_P = \frac{x_1 + l_1 . t_1 + x_2 + l_2 . t_2}{2}$$

$$y_P = \frac{y_1 + m_1 . t_1 + y_2 + m_2 . t_2}{2}$$

$$z_P = \frac{z_1 + n_1 . t_1 + z_2 + n_2 . t_2}{2} + \frac{h_1 + h_2 - 2 . h_P}{2}$$

$$\text{Equation 2.15}$$

where

h_1 = instrument height at control point 1.

h_2 = instrument height at control point 2.

h_P = target height at target point.

Consideration should be given to the curvature of the earth and refraction. At a sight distance of 10 m the curvature of the earth can be calculated to be approximately 10 μm. In calculating the coordinates of a target point the measured vertical angles can be corrected as follows

$$v = v' + \frac{c}{2} - r \qquad \text{Equation 2.16}$$

where

v   = corrected vertical angle.

v'   = observed vertical angle.

c   = correction for curvature.

$$= \frac{\text{spheroid distance}}{Rm}$$

Rm = local mean radius of the earth of the line of sight.

r   = correction for refraction.

= 0.074 . c

Calculation of raw coordinates in any coordinate system can be summarised by the following steps, where

u   = mean azimuth (control point - target point).

v'   = observed vertical angle (control point - target point).

v   = corrected vertical angle (control point - target point).

h_1 = instrument height at control point 1.

h_2 = instrument height at control point 2.

h_p = target height at target point.

t   = coordinate system slope distance.

sd   = equivalent measured (EDM) slope distance.

pd   = coordinate system plan distance.

ngd = National Grid distance.

sph   = spheroidal distance.

lsf   = local scale factor of the line of sight.

Rm = local mean radius of the earth of the line of sight.

40

Lm = mean height above mean sea level of the line of sight.

vd = coordinate system vertical distance.

(i)   Calculate initial azimuths and vertical angles (u_1, v_1', u_2, v_2') from equation 2.1.

(ii)   Calculate direction cosines from equation 2.2 and coordinate system slope distances (t_1 and t_2) from equations 2.12 and 2.14.

(iii)   In a local coordinate system, assume values for local scale factor, and local mean radius of the earth, of the lines of sight

$$lsf_1 = 1$$
$$Rm_1 = 6\ 370\ 000\ m$$
$$lsf_2 = 1$$
$$Rm_2 = 6\ 370\ 000\ m$$

In the National Grid coordinate system, calculate values for local scale factor, and local mean radius of the earth, of the lines of sight from the National Grid coordinates of the control points and approximate coordinates of the target point [Hodges et al 1990].

(iv)   In a local coordinate system, the coordinate system slope distances are reduced to coordinate system plan distances, which are considered equal to spheroid distances

$$pd_1 = t_1 \cdot \cos v_1$$

41

$$sph_1 = pd_1$$

$$pd_2 = t_2 \cdot \cos v_2$$

$$sph_2 = pd_2$$

In the National Grid coordinate system, the coordinate system slope distances are reduced to National Grid distances and spheroid distances

$$ngd_1 = t_1 \cdot \cos v_1$$

$$sph_1 = \frac{ngd_1}{lsf_1}$$

$$ngd_2 = t_2 \cdot \cos v_2$$

$$sph_2 = \frac{ngd_2}{lsf_2}$$

(v) Correct the observed vertical angles for curvature and refraction by equation 2.16.

(vi) Repeat steps (ii), (iii) and (iv) using the corrected vertical angles (v_1 and v_2).

(vii) In a local coordinate system, the equivalent measured (EDM) slope distances are equal to the coordinate system slope distances

$$sd_1 = t_1$$

$$sd_2 = t_2$$

In the National Grid coordinate system, the equivalent measured (EDM) slope distances are calculated from the spheroid distances

42

$$vd_1 = sph_1 . \tan v_1 + (h_1 - h_P)$$

$$Lm_1 = \frac{2 . z_P + vd_1}{2}$$

$$pd_1 = \frac{(Rm_1 + Lm_1) . sph_1}{Rm}$$

$$sd_1 = \frac{pd_1}{\cos v_1}$$

$$vd_2 = sph_2 . \tan v_2 + (h_2 - h_P)$$

$$Lm_2 = \frac{2 . z_P + vd_2}{2}$$

$$pd_2 = \frac{(Rm_2 + Lm_2) . sph_2}{Rm_2}$$

$$sd_2 = \frac{pd_2}{\cos v_2}$$

(viii) In a local coordinate system, the raw coordinates are calculated as follows

$$x_P = \frac{(x_1 + pd_1 . \sin u_1) + (x_2 + pd_2 . \sin u_2)}{2}$$

$$y_P = \frac{(y_1 + pd_1 . \cos u_1) + (y_1 + pd_2 . \cos u_2)}{2}$$

$$z_P = \frac{(z_1 + sd_1 . \sin v_1) + (z_2 + sd_2 . \sin v_2)}{2} + \frac{h_1 + h_2 - 2 . h_P}{2}$$

On inspection it can be seen that this is the same as equation 2.15.

In the National Grid coordinate system, the raw coordinates are calculated as follows

$$x_P = \frac{(x_1 + ngd_1 . \sin u_1) + (x_2 + ngd_2 . \sin u_2)}{2}$$

$$y_P = \frac{(y_1 + ngd_1 \cdot \cos u_1) + (y_1 + ngd_2 \cdot \cos u_2)}{2}$$

$$z_P = \frac{(z_1 + sd_1 \cdot \sin v_1) + (z_2 + sd_2 \cdot \sin v_2)}{2} + \frac{h_1 + h_2 - 2 \cdot h_P}{2}$$

(ix) An indication of the accuracy of target point intersection can be obtained by calculating the mis-intersections (Dx, Dy, Dz) of the two lines of sight.

In a local coordinate system, the mis-intersections are calculated as follows

$$Dx = \frac{(x_1 + pd_1 \cdot \sin u_1) - (x_2 + pd_2 \cdot \sin u_2)}{2}$$

$$Dy = \frac{(y_1 + pd_1 \cdot \cos u_1) - (y_1 + pd_2 \cdot \cos u_2)}{2}$$

$$Dz = \frac{(z_1 + sd_1 \cdot \sin v_1) - (z_2 + sd_2 \cdot \sin v_2)}{2} + \frac{h_1 - h_2}{2}$$

In the National Grid coordinate system, the mis-intersections are calculated as follows

$$Dx = \frac{(x_1 + ngd_1 \cdot \sin u_1) - (x_2 + ngd_2 \cdot \sin u_2)}{2}$$

$$Dy = \frac{(y_1 + ngd_1 \cdot \cos u_1) - (y_1 + ngd_2 \cdot \cos u_2)}{2}$$

$$Dz = \frac{(z_1 + sd_1 \cdot \sin v_1) - (z_2 + sd_2 \cdot \sin v_2)}{2} + \frac{h_1 - h_2}{2}$$

2.4.2 Least Squares Adjustment of Coordinates

The definition of least squares is that the most probable value is obtained when the sum of the squares of the residual errors is a

44

minimum. The least squares adjustment adjusts the raw coordinates obtained by three-dimensional intersection in section 2.4.1 to least squares adjusted coordinates. In addition to adjusting the coordinates, an ellipsoid of error around the intersected point is derived, as detailed in section 2.4.4.

Considering the condition equations 2.2 and 2.3, and linearizing by Taylor's Theorem we obtain an equation in the form

$$d = A\,c \qquad\qquad \text{Equation 2.17}$$

where

d = vector of the differences between the observed and computed observations, the computed observations being derived from the control point and target point coordinates.

$$= \begin{bmatrix} t_1 \cdot \Delta v_1 \\ t_1 \cdot \cos v_1 \cdot \Delta u_1 \\ t_2 \cdot \Delta v_2 \\ t_2 \cdot \cos v_2 \cdot \Delta u_2 \end{bmatrix}$$

A = matrix of partial derivatives of the condition equations.

45

$$
= \begin{bmatrix} -\sin v_1 . \sin u_1 & -\sin v_1 . \cos u_1 & \cos v_1 \\ \cos u_1 & -\sin u_1 & 0 \\ -\sin v_2 . \sin u_2 & -\sin v_2 . \cos u_2 & \cos v_2 \\ \cos u_2 & -\sin u_2 & 0 \end{bmatrix}
$$

c = vector of corrections to the approximate coordinate parameters.

$$
= \begin{bmatrix} \Delta x_P \\ \Delta y_P \\ \Delta z_P \end{bmatrix}
$$

Linearization by Taylor's Theorem is detailed in Appendix A and the derivation of matrices d, A and c is detailed in Appendix B. The least squares method used to obtain a best estimate of the coordinate corrections is obtained on introducing residual errors to equation 2.17

$$ d = A c + r \qquad\qquad \text{Equation 2.18} $$

where

r = vector of residual errors.

The least squares solution of equation 2.18 for the vector c is obtained by

$$ c = N^{-1} F $$

where

$$N = A^T W A$$

$$F = A^T W d$$

W = weight matrix of the observations formed from the reciprocals of the variances of the measured angles (v_1, u_1, v_2, u_2). As the angles are independent then the covariances of the angles are equal to zero and only the variances are used to form the weight matrix.

$$= \begin{bmatrix} 1/\sigma_{V1}^2 & 0 & 0 & 0 \\ 0 & 1/\sigma_{U1}^2 & 0 & 0 \\ 0 & 0 & 1/\sigma_{V2}^2 & 0 \\ 0 & 0 & 0 & 1/\sigma_{U2}^2 \end{bmatrix}$$

The corrections obtained are added to the approximate coordinates and these updated coordinates are used to obtain new values of $\Delta v_1, \Delta u_1, \Delta v_2, \Delta u_2$ for the vector d, which in turn is used to compute new corrections. This procedure is repeated until the last vector of corrections is insignificantly small. The least squares adjustment is an iterative process which can be summarised as follows:

(i)   Compute the A matrix from the observations.

(ii)   Compute the weight matrix W from the variances of the observations.

(iii) Compute the N and N^{-1} matrices.

(iv)   Compute the calculated observations from control point and target point coordinates and obtain new values of $\Delta v_1, \Delta u_1, \Delta v_2, \Delta u_2$ for the vector d.

(v)   Compute the F matrix.

(vi) Compute the corrections vector c and correct the previous iteration coordinates.

(vii) Repeat steps (iv), (v) and (vi) until the correction values are below a certain limit.

2.4.3 Probable Error

Probable coordinate errors are calculated as the theoretical minimum errors obtainable with a certain geometric set up and are based on instrumental accuracy alone. Instrumental accuracy is given as

δu = theoretical angle error of a horizontal angle.

δv = theoretical angle error of a vertical angle.

Considering a two control point set up, the probable coordinate errors are obtained by considering the coordinate errors arising from four different angle error combinations, namely

P	δu_1	δv_1	δu_2	δv_2
1	+	+	+	+
2	+	+	-	+
3	+	-	+	+
4	+	-	-	+

The probable coordinate errors are obtained as

$$Px = \frac{|2 . \delta x_{P1}| + |2 . \delta x_{P2}| + |2 . \delta x_{P3}| + |2 . \delta x_{P4}|}{8}$$

$$Py = \frac{|2 . \delta y_{P1}| + |2 . \delta y_{P2}| + |2 . \delta y_{P3}| + |2 . \delta y_{P4}|}{8}$$

$$Pz = \frac{|2 \cdot \delta z_{P1}| + |2 \cdot \delta z_{P2}| + |2 \cdot \delta z_{P3}| + |2 \cdot \delta z_{P4}|}{8}$$

<div align="right">Equation 2.19</div>

Equation 2.19 is derived in Appendix C. For each of the four angle error combinations, considering equation 2.3 in the perfect case of intersection

$$2 \cdot x_P = x_1 + l_1 \cdot t_1 + x_2 + l_2 \cdot t_2$$
$$2 \cdot y_P = y_1 + m_1 \cdot t_1 + y_2 + m_2 \cdot t_2$$
$$2 \cdot z_P = z_1 + n_1 \cdot t_1 + z_2 + n_2 \cdot t_2$$

The errors in the coordinates are determined by introducing errors in the direction cosines and calculated slope distances, for example δx_P

$$2 \cdot \delta x_P = \frac{\partial x_P}{\partial t_1} \cdot \delta t_1 + \frac{\partial x_P}{\partial l_1} \cdot \delta l_1 + \frac{\partial x_P}{\partial t_2} \cdot \delta t_2 + \frac{\partial x_P}{\partial l_2} \cdot \delta l_2$$

$$2 \cdot \delta x_P = l_1 \cdot \delta t_1 + t_1 \cdot \delta l_1 + l_2 \cdot \delta t_2 + t_2 \cdot \delta l_2$$

The errors in all coordinate axes are given by

$$2 \cdot \delta x_P = l_1 \cdot \delta t_1 + t_1 \cdot \delta l_1 + l_2 \cdot \delta t_2 + t_2 \cdot \delta l_2$$
$$2 \cdot \delta y_P = m_1 \cdot \delta t_1 + t_1 \cdot \delta m_1 + m_2 \cdot \delta t_2 + t_2 \cdot \delta m_2$$
$$2 \cdot \delta z_P = n_1 \cdot \delta t_1 + t_1 \cdot \delta n_1 + n_2 \cdot \delta t_2 + t_2 \cdot \delta n_2$$

<div align="right">Equation 2.20</div>

The errors in the direction cosines are determined from equation 2.2, for example δl_1

<div align="center">49</div>

$$\delta l_1 = \frac{\partial l_1}{\partial v_1} \cdot \delta v_1 + \frac{\partial l_1}{\partial u_1} \cdot \delta u_1$$

$$\delta l_1 = - \sin v_1 \cdot \sin u_1 \cdot \delta v_1 + \cos v_1 \cdot \cos u_1 \cdot \delta u_1$$

The errors in all of the direction cosines are given by

$$\delta l_1 = - \sin v_1 \cdot \sin u_1 \cdot \delta v_1 + \cos v_1 \cdot \cos u_1 \cdot \delta u_1$$

$$\delta m_1 = - \sin v_1 \cdot \cos u_1 \cdot \delta v_1 - \cos v_1 \cdot \sin u_1 \cdot \delta u_1$$

$$\delta n_1 = \cos v_1 \cdot \delta v_1$$

$$\delta l_2 = - \sin v_2 \cdot \sin u_2 \cdot \delta v_2 + \cos v_2 \cdot \cos u_2 \cdot \delta u_2$$

$$\delta m_2 = - \sin v_2 \cdot \cos u_2 \cdot \delta v_2 - \cos v_2 \cdot \sin u_2 \cdot \delta u_2$$

$$\delta n_2 = \cos v_2 \cdot \delta v_2$$

Equation 2.21

The errors in the calculated slope distances are determined from equations 2.12 and 2.13. For both distances the following errors are present

$$\delta p = (x_2 - x_1) \cdot \delta l_1 + (y_2 - y_1) \cdot \delta m_1 + (z_2 - z_1) \cdot \delta n_1$$

$$\delta q = (x_2 - x_1) \cdot \delta l_2 + (y_2 - y_1) \cdot \delta m_2 + (z_2 - z_1) \cdot \delta n_2$$

$$\delta c = \cos^{-1}(\cos (c + \delta c)) - \cos^{-1}(\cos c)$$

Equation 2.22

where

$$\cos c = l_1 \cdot l_2 + m_1 \cdot m_2 + n_1 \cdot n_2$$

$$\cos (c + \delta c) = (l_1 + \delta l_1) \cdot (l_2 + \delta l_2) +$$
$$(m_1 + \delta m_1) \cdot (m_2 + \delta m_2) +$$
$$(n_1 + \delta n_1) \cdot (n_2 + \delta n_2)$$

Considering distance t_1

$$\delta t_1 = \frac{\partial t_1}{\partial p} \cdot \delta p + \frac{\partial t_1}{\partial q} \cdot \delta q + \frac{\partial t_1}{\partial c} \cdot \delta c$$

$$= \frac{\delta p}{\sin^2 c} - \frac{\cos c \cdot \delta q}{\sin^2 c} + \frac{((q + q \cdot \cos^2 c - 2 \cdot p \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

Equation 2.23

Considering distance t_2

$$\delta t_2 = \frac{\partial t_2}{\partial p} \cdot \delta p + \frac{\partial t_2}{\partial q} \cdot \delta q + \frac{\partial t_2}{\partial c} \cdot \delta c$$

$$= \frac{\cos c \cdot \delta p}{\sin^2 c} - \frac{\delta q}{\sin^2 c} - \frac{((p + p \cdot \cos^2 c - 2 \cdot q \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

Equation 2.24

The derivation of equations 2.23 and 2.24 is given in Appendix D. The calculation of probable errors can be summarised as:

(i)   Fix values for δv_1, δu_1, δv_2 and δu_2 for the ith angle error combination.

(ii)   Calculate the errors in the direction cosines δl_1, δm_1, δn_1, δl_2, δm_2 and δn_2 from equation 2.21.

(iii) Calculate the errors δp, δq and δc from equation 2.22.

(iv) Calculate the errors in the calculated slope distances δt_1 and δt_2 from equations 2.23 and 2.24.

51

(v)   Calculate the errors in the coordinates δx_{Pi}, δy_{Pi} and δz_{Pi} from equation 2.20 for the ith angle error combination.

(vi)   Repeat steps (i) to (v) for each of the four angle error combinations.

(vii) Calculate the probable errors Px, Py and Pz from equation 2.19.

2.4.4 Ellipsoid of Error

From the matrices obtained in the least squares adjustment an ellipsoid of error, formed from maximum variances about the target point, can be determined. The residuals matrix is obtained at the end of the least squares adjustment by

$$r = A c - d$$

where

r = vector of residual errors.

A = matrix of the coefficients of the unknown coordinate parameters.

c = overall correction matrix to the approximate coordinate parameters.

(final least squares adjusted coordinates - initial raw coordinates)

d = final vector of the differences between the observed and computed observations.

(final calculated angles - observed angles)

The reference variance is obtained as

$$\sigma_0{}^2 = \frac{r^T \, W \, r}{n - n_o}$$

where

$n - n_0$ = redundancy of measurement.

n_0 = parameters required for solution.

= 3, ie u_1 and u_2 and (v_1 or v_2).

n = total number of observations.

= $2 \cdot N_T \cdot N_O$

where

N_T = number of theodolites used to sight a target point.

N_O = number of observations / target point.

The variance - covariance matrix is then found by

$$Q = \sigma_0{}^2 \cdot N^{-1}$$

where

$$Q = \begin{bmatrix} \sigma_{XX} & \sigma_{XY} & \sigma_{XZ} \\ \sigma_{XY} & \sigma_{YY} & \sigma_{YZ} \\ \sigma_{XZ} & \sigma_{YZ} & \sigma_{ZZ} \end{bmatrix}$$

The variance - covariance matrix is transformed into a diagonal matrix containing the three ellipsoid values Ex, Ey and Ez

$$E = MQM^T$$

$$= \begin{bmatrix} Ex^2 & 0 & 0 \\ 0 & Ey^2 & 0 \\ 0 & 0 & Ez^2 \end{bmatrix}$$

where

M = rotation matrix comprising of rotation angles ω, ϕ, κ.

The composition of the rotation matrix M is detailed in Appendix E. The rotation angles ω, ϕ, κ are calculated independently from values in the variance - covariance matrix [Methley 1986], as follows

$$\tan 2\omega = \frac{2 \cdot \sigma_{YZ}}{\sigma_{YY} - \sigma_{ZZ}}$$

$$\tan 2\phi = \frac{2 \cdot \sigma_{XZ}}{\sigma_{ZZ} - \sigma_{XX}}$$

$$\tan 2\kappa = \frac{2 \cdot \sigma_{XY}}{\sigma_{XX} - \sigma_{YY}}$$

2.5 Transformation of Coordinate Systems

In some cases a transformation from a local coordinate system to an object coordinate system may be performed. A three-dimensional transformation is defined by seven transformation parameters (Figure 2.11), namely

54
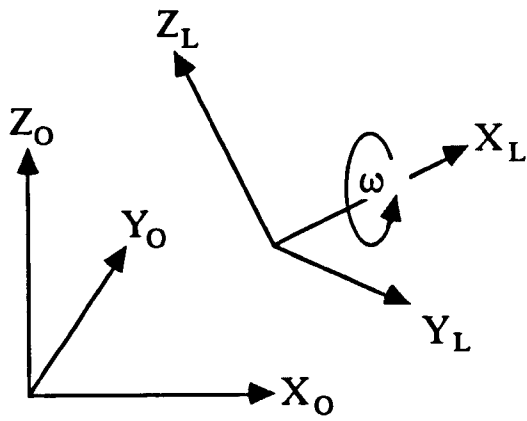
(i)   Initial coordinate systems.   (ii)   After rotation about x-axis.

O   Object coordinate system.   O   Object coordinate system.

L   Local coordinate system.   ω   Local coordinate system
                                                after ω rotation.
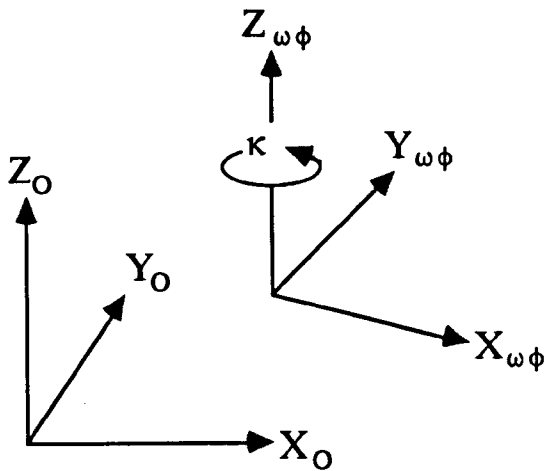
(iii)   After rotation about y-axis.   (iv)   After rotation about z-axis.
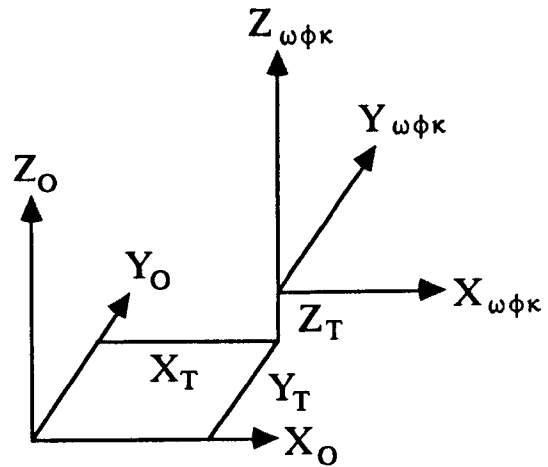
O   Object coordinate system.   O   Object coordinate system.

ω φ   Local coordinate system   ωφκ   Local coordinate system
        after ω and φ rotations.   after ω, φ and κ rotations.

Figure 2.11 Three-Dimensional Transformation Parameters.

λ = scale change.

ω = rotation angle about the x-axis.

ϕ = rotation angle about the y-axis.

κ = rotation angle about the z-axis.

X_T = translation of the origin in the x-axis.

Y_T = translation of the origin in the y-axis.

Z_T = translation of the origin in the z-axis.

The rotation angles are rotations of the coordinate system about the coordinate axes, as opposed to rotations of the coordinate axes themselves, and form a rotation matrix M. The composition of the rotation matrix M is detailed in Appendix E.

The transformation of local coordinates to object coordinates is performed by

$$X = \lambda \cdot (M_{11} \cdot x + M_{12} \cdot y + M_{13} \cdot z) - X_T$$
$$Y = \lambda \cdot (M_{21} \cdot x + M_{22} \cdot y + M_{23} \cdot z) - Y_T$$
$$Z = \lambda \cdot (M_{31} \cdot x + M_{32} \cdot y + M_{33} \cdot z) - Z_T$$

where

x, y, z = local coordinates.

X, Y, Z = object coordinates.

To determine the seven transformation parameters, n points whose coordinates are known in both the local and object coordinate systems are required, the minimum value of n being three. The determination of the parameters is performed in three stages:

(i) Initial estimation of λ, κ, X_T and Y_T.

(ii) Initial estimation of ω, ϕ and Z_T.

(iii) Least squares solution to obtain the final seven parameters.

The three stages are now detailed.

(i) Initial estimation of λ, κ, X_T and Y_T.

Initial estimates of the four parameters λ, κ, X_T and Y_T are given by a two-dimensional linear conformal transformation in the XY plane [Methley 1986]

$$X = x \cdot a - y \cdot b + X_T$$
$$Y = x \cdot b + y \cdot a + Y_T$$

where

$$a = \lambda \cdot \cos \kappa$$
$$b = \lambda \cdot \sin \kappa$$
$$\kappa = \tan^{-1} \frac{a}{b}$$
$$\lambda = \sqrt{a^2 + b^2}$$

The determination of the four parameters requires at least two points with known coordinates in both coordinate systems, as far apart as possible for reliability. For ease of calculation in computer terms the parameters a, b, X_T and Y_T are determined by a least squares solution for the n points involved in the transformation

$$d = A\,p$$

where

d = vector of known object coordinates.

$$= \begin{bmatrix} X_1 \\ Y_1 \\ X_n \\ Y_n \end{bmatrix}$$

A = matrix of the coefficients of the unknown parameters.

$$= \begin{bmatrix} x_1 & -y_1 & 1 & 0 \\ y_1 & x_1 & 0 & 1 \\ x_n & -y_n & 1 & 0 \\ y_n & x_n & 0 & 1 \end{bmatrix}$$

p = vector of initial estimates of the unknown parameters.

$$= \begin{bmatrix} a \\ b \\ X_T \\ Y_T \end{bmatrix}$$

The initial estimates of the parameters are obtained as

$$p = N^{-1}\,F$$

where

$$N = A^T W A$$

$$F = A^T W d$$

W = weight matrix formed from the error ellipsoid values of the transformation point local coordinates.

$$= \begin{bmatrix} 1/Ex_1^2 & 0 & 0 & 0 \\ 0 & 1/Ey_1^2 & 0 & 0 \\ 0 & 0 & 1/Ex_n^2 & 0 \\ 0 & 0 & 0 & 1/Ey_n^2 \end{bmatrix}$$

(ii) Initial estimation of ω, ϕ and Z_T.

Initial estimates of the remaining three parameters ω, ϕ and Z_T are obtained from the equation [Methley 1986]

$$Z = z + \omega . y - \phi . x + Z_T$$

where x, y, z are the original values multiplied by the initial scale change determined in (i) and ω and ϕ are assumed to be small. The parameters are again determined by a least squares solution for the n points involved in the transformation

$$d = A p$$

where

d = vector of the differences between object and local z coordinates.

59

$$= \begin{bmatrix} Z_1 - z_1 \\ Z_n - z_n \end{bmatrix}$$

A = matrix of the coefficients of the unknown parameters.

$$= \begin{bmatrix} y_1 & -x_1 & 1 \\ y_n & -x_n & 1 \end{bmatrix}$$

p = vector of initial estimates of the unknown parameters.

$$= \begin{bmatrix} \omega \\ \phi \\ Z_T \end{bmatrix}$$

The initial estimates of the unknown parameters are obtained as

$$p = N^{-1} F$$

where

$$N = A^T W A$$

$$F = A^T W d$$

W = weight matrix formed from the error ellipsoid values of the transformation point local coordinates.

$$= \begin{bmatrix} 1/Ez_1^2 & 0 \\ 0 & 1/Ez_n^2 \end{bmatrix}$$

The initial estimates of ω and ϕ are derived as rotations of the coordinate axes themselves, therefore, their signs have to be

reversed in order to achieve the equivalent rotation of the coordinate system about the axes.

(iii) Least squares solution to obtain the final seven parameters.

Once initial estimates of the seven parameters have been obtained, a least squares solution is used to determine the final values of the seven parameters [Moffit & Mikhail 1980].

In order to compute the seven parameters at least seven equations are required. Any point of known local and object coordinates yields three condition equations

$$f_1 = X - \lambda . (M_{11} . x + M_{12} . y + M_{13} . z) - X_T = 0$$
$$f_2 = Y - \lambda . (M_{21} . x + M_{22} . y + M_{23} . z) - Y_T = 0$$
$$f_3 = Z - \lambda . (M_{31} . x + M_{32} . y + M_{33} . z) - Z_T = 0$$

Equation 2.25

the linearized form of these three equations for the minimum amount of control can be expressed as

$$d = B p$$

Equation 2.26

where

d = vector of the differences between the coordinates as calculated by transformation and the object coordinates.

61

$$= \begin{bmatrix} \lambda \cdot (M_{11} \cdot x + M_{12} \cdot y + M_{13} \cdot z) + X_T - X \\ \lambda \cdot (M_{21} \cdot x + M_{22} \cdot y + M_{23} \cdot z) + Y_T - Y \\ \lambda \cdot (M_{31} \cdot x + M_{32} \cdot y + M_{33} \cdot z) + Z_T - Z \end{bmatrix}$$

B = matrix of the partial derivatives of the condition equations.

$$= \begin{bmatrix} \partial f_1/\partial\lambda & \partial f_1/\partial\omega & \partial f_1/\partial\phi & \partial f_1/\partial\kappa & \partial f_1/\partial X_T & \partial f_1/\partial Y_T & \partial f_1/\partial Z_T \\ \partial f_2/\partial\lambda & \partial f_2/\partial\omega & \partial f_2/\partial\phi & \partial f_2/\partial\kappa & \partial f_2/\partial X_T & \partial f_2/\partial Y_T & \partial f_2/\partial Z_T \\ \partial f_3/\partial\lambda & \partial f_3/\partial\omega & \partial f_3/\partial\phi & \partial f_3/\partial\kappa & \partial f_3/\partial X_T & \partial f_3/\partial Y_T & \partial f_3/\partial Z_T \end{bmatrix}$$

p = vector of corrections to the approximate parameters.

$$= [\ \Delta\lambda \quad \Delta\omega \quad \Delta\phi \quad \Delta k \quad \Delta X_T \quad \Delta Y_T \quad \Delta Z_T \ ]$$

Linearization by Taylor's Theorem is detailed in Appendix A and the derivation of the partial derivatives of matrix B is detailed in Appendix F. The least squares method used to obtain a best estimate of the coordinate corrections is obtained on introducing residual errors to equation 2.26

$$d = A\,r + B\,p \qquad\qquad \text{Equation 2.27}$$

where

r = vector of residual errors.

A = matrix of the coefficients of the residuals.

$$= \begin{bmatrix} \partial F_1/\partial x & \partial F_1/\partial y & \partial F_1/\partial z \\ \partial F_2/\partial x & \partial F_2/\partial y & \partial F_2/\partial z \\ \partial F_3/\partial x & \partial F_3/\partial y & \partial F_3/\partial z \end{bmatrix}$$

$$= -\lambda M$$

To obtain the least squares solution and solve the correction matrix p

$$p = N^{-1} F$$

where

$$N = B^T (A Q A^T)^{-1} B$$
$$F = B^T (A Q A^T)^{-1} d$$

Q = weight matrix of the observed local coordinates, compiled from the error ellipsoid values for the point.

$$= \begin{bmatrix} 1/Ex^2 & 0 & 0 \\ 0 & 1/Ey^2 & 0 \\ 0 & 0 & 1/Ez^2 \end{bmatrix}$$

The corrections obtained are added to the initial approximations of the seven parameters and these updated parameters are used to obtain new values for the matrices A, B and d, which in turn are used to compute a new p matrix. This procedure is repeated until the last vector of corrections is insignificantly small.

For ease of computation, in computer terms, the matrices N and F may be formed by a summation process [Mikhail 1976]. Data for each point is handled separately effecting great savings in memory storage. The matrices N and F are calculated point by point under the assumption of block diagonality of the A and Q matrices, common to most transformations, ie

$$N = \Sigma \ [B^T (A \ Q \ A^T)^{-1} \ B]_i \qquad i = 1 .. n \text{ points}$$

$$F = \Sigma \ [B^T (A \ Q \ A^T)^{-1} \ d]_i \qquad i = 1 .. n \text{ points}$$

To determine the seven transformation parameters at least three points of known coordinates in both coordinate systems are required, however, in practice more than the minimum of three points would be used to increase the redundancy of the solution and subsequently increase the accuracy of transformation. It is also possible to define fixed values for any of the transformation parameters, for example, to define the z-axis as fixed in both coordinate systems, the rotation angles ω and ϕ would be set equal to zero in the solution.

2.6 Conclusions

A measurement survey can be performed in a local coordinate system, object coordinate system or the National Grid. The first stage in a measurement survey is to determine the coordinates of the control points in the defined coordinate system by a suitable setting up procedure. There are various methods available for setting up a coordinate system which permit extremely flexible use of an ETIS. The method used depends on the nature of the measurement survey and the coordinate system required.

Once a coordinate system has been set up an ETIS enables the determination of three-dimensional point coordinates to a high accuracy. Initial raw coordinates are calculated by spatial intersection and a simple indication of the accuracy of coordinate determination is obtained by considering the mis-intersection values. A least squares adjustment is applied to calculate the final coordinates and an actual ellipsoid of error about the point. Comparison of the actual and probable errors enables a better assessment to be made of the accuracy of coordinate determination.

The minimum requirement of an ETIS is that it is capable of performing all of the calculations detailed in this Chapter, in the field.

CHAPTER THREE

COMPONENTS OF AN ELECTRONIC THEODOLITE
INTERSECTION SYSTEM

Components of an Electronic Theodolite Intersection System

3.1 Introduction

The main components of an ETIS are, at least two, electronic theodolites and an on-line computer, with peripheral hardware and suitable software. In addition to the main components there are several peripheral components essential to the operation of an ETIS. These include theodolite stands, suitable targets for marking the points to be measured and a calibrated scale bar.

The main components of a system can be considered as fixed and should, therefore, be suitable for use in a variety of measurement surveys. The electronic theodolites are the most expensive components of a system and careful consideration should be given to both the number and type required. In selecting the type of electronic theodolites, consideration should be given to their claimed accuracy and ease of use, and in using electronic theodolites, operators should be fully aware of the care and attention required to achieve these claimed accuracies. The other main components of a system are the computer hardware and software. Although these do not directly affect the accuracy of the system, they should be capable of utilising the measured data and providing output in a suitable format.

The peripheral components of a system can be considered as variable. They are often the cheapest components and can, therefore, be selected to suit a specific measurement task. In using a system, operators should be fully aware of the peripheral components available

and the considerations which should be taken into account in their selection for an individual measurement survey.

In practice, in order to obtain the best results with an ETIS, operators should be familiar with each component, therefore, this Chapter details the individual components of an ETIS and outlines their role.

3.2 Electronic Theodolites

A theodolite is a surveying instrument capable of measuring horizontal and vertical angles to a high accuracy. With a conventional optical mechanical theodolite an angle is measured optically by observing a precise graduation on a glass circle within the instrument. The angle is obtained in degrees, minutes and seconds of arc, in general the degrees and minutes are read directly from the circle and the seconds are obtained on adjustment of a micrometer.

Since the introduction of microprocessors in the 1970s, their processing capabilities have increased many fold, whilst, at the same time their size has decreased considerably. These rapid advances resulted in the development of the current electronic systems of angle measurement - electronic theodolites. An example of an early electronic theodolite is the Kern E1, which was introduced at the XV FIG Congress in Stockholm in 1977.

Electronic theodolites are similar in appearance and basic operation to optical mechanical theodolites. Their accuracy has increased to the point where "one second" electronic theodolites, which

have an accuracy of 0.3" to 0.6" of arc, are common place, for example the Kern E2 and Wild T2000. In addition, electronic theodolites have many advantages over optical mechanical theodolites, namely:

(i) Measured angles are output in a digital format. This enables their direct output on an LCD display, and automatic transmission to data recorders and computers. In addition, the LCD display removes the need for micrometer adjustment and enables output in a variety of units, such as sexagesimal degrees, decimal degrees, gon or mil.

(ii) Vertical axis tilt, caused by mislevelment of a theodolite, is eliminated by a liquid compensator. This calculates the errors in the horizontal and vertical angles due to vertical axis tilt and automatically corrects the output angles.

(iii) Instrument-related errors and deviations from ideal conditions can be determined and automatically included in computation procedures. These errors are present in optical mechanical theodolites but they must be determined by the observer using suitable measuring and calculation procedures.

However, the electronic components incorporated in electronic theodolites can only replace the mechanical system to a limited extent and precision engineering still plays an important part when a high degree of accuracy is required. The prerequisite for an electronic theodolite is, therefore, that the mechanical and electronic elements crucial to the accuracy of the instrument, namely; mechanical stability, compensation system, circle accuracy, interpolation procedure and telescope, are very well matched to each other [Munch 1984].

3.2.1 Components of an Electronic Theodolite

An electronic theodolite consists of three main components, containing the functional elements of the instrument, namely:

(i) Fixed base unit.

(ii) Rotatable alidade with standards.

(iii) Telescope.

The fixed base unit forms the lower theodolite unit. It is used to centre the theodolite on the theodolite stand and contains some form of levelling system, consisting of either two or three levelling screws. With the conventional three levelling screw system, eg Wild T2000, the instrument is levelled by the simultaneous adjustment of two levelling screws, followed by the adjustment of the third levelling screw. With the two levelling screw system, eg Kern E2, the instrument is tilted by the levelling screws about a centre ball and socket joint. The two levelling screw system has the following advantages:

(i) The tilting axis remains constant, regardless of the lift of the levelling screws.

(ii) The levelling procedure is faster and more convenient.

(iii) The weight of the instrument is absorbed by the ball rather than the levelling screws.

The alidade forms the upper theodolite unit. It contains the reading and adjustment facilities and supports the axis for the telescope. Horizontal angles are measured in a plane defined by the rotation of the alidade about the centre of the lower theodolite unit. The alidade level

is used to "visually" level the instrument over the control point, at which stage, the instrument can be "digitally" levelled. As in optical mechanical theodolites, the theodolite standards support the telescope, ensuring that its horizontal axis bearings are concentric with and perpendicular to the vertical axis, and house the vertical circle. In the case of electronic theodolites, the standards also house the compensator, the electronics system for processing the measurement signals, function switches, permanent adjustment switches and the LCD displays.

The telescope of the electronic theodolite permits the accurate centring of the reticule on a target. There are two types of telescope, namely; conventional and panfocal. The panfocal telescope is designed to allow easier pointing at any distance, as the magnification and field of view vary with the focusing distance. When focusing to distant targets the magnification is at a maximum, whereas over shorter distances, the field of view widens and the magnification is reduced. Furthermore, there are two types of reticule, namely; conventional cross hairs and concentric circles, and the type used depends on personal preference.

3.2.2 Principles of Circle Measurement

Circle measurement in electronic theodolites is based on the use of a photolithographically coded circle and a series of photoelectric detector elements, or photodiodes. Circle readings are obtained as a combination of a coarse measurement procedure and a fine measurement, or interpolation, procedure, on which the accuracy of the instrument depends.

There are two systems of circle measurement used in electronic theodolites; incremental / static and absolute / dynamic. The principles of these two systems are now detailed, with reference to two particular electronic theodolites.

3.2.2.1 Incremental / Static Circle Measurement

Incremental / static circle measurement is used in the Kern E2 electronic theodolite. The circle measuring system consists of a glass circle which is scanned at a fixed scanning position. As with an optical mechanical theodolite, rotation of the alidade or telescope causes the respective horizontal or vertical circle to rotate.

Each circle is graduated into 20 000 radial marks, with the space between each mark being equal to the width of the mark. The so called grating constant is equal to one mark and one space, and has an angular value of 0.02 gon. However, circle readings are not determined from the location of an individual graduation mark, but from a section comprising of 200 marks.

Both coarse and fine measurement are performed using a moire fringe pattern. Interference patterns of this type are formed when closely ruled parallel lines are superimposed at a slight angle to each other. In the case of the Kern E2, the pattern is formed by focusing a slightly magnified section of one side of the circle on to an identical section from the diametrically opposite circle position. From a section comprising of 200 marks a moire pattern with a full period (light-dark variation) is formed. The brightness distribution over this moire period is approximately sinusoidal and can be scanned by four

photodiodes, equally spaced over the moire period, enabling both coarse and fine circle measurement (Figure 3.1).

Coarse circle measurement is performed by an incremental process whilst the alidade or telescope are being rotated. As a circle is rotated it causes a reverse movement of the diametrically opposite circle images. A rotation of half of the grating constant (ie one graduation mark width or 0.01 gon) causes a full moire period to pass the scanning position. Coarse circle measurement is based on counting the number of periods as they pass the scanning position, and incrementing the circle readings accordingly.

Fine circle measurement is performed statically ie with a stationary moire pattern. The objective of fine measurement is to establish the location of the scanning position within a single moire period. Each of the photodiodes generates a signal which is proportional to the brightness at that point of the moire period and the fine measurement, or interpolation, value is determined from the amplitudes of these signals.

With the Kern E2 a coarse measuring accuracy of ±0.01 gon (2') and a fine measuring accuracy of ±0.1 mgon (0.3") are obtained. The high accuracies attainable with the instrument are due to each reading incorporating 200 graduations, reducing the effect of any inconsistency in a single graduation, and scanning at diametrically opposite circle positions, eliminating the effect of any eccentricity in the circle. The fine measurement interpolation is repeated at intervals of 0.3 seconds and the circle readings are updated at the same rate. A full description of the Kern E2 circle measuring system can be found in

min   max   min

Brightness
Distribution

Diodes   1   2   3   4

Moire
Pattern

Diodes   1   2   3   4

Diode phase angle   ρ   ρ + 180

Resultant voltage
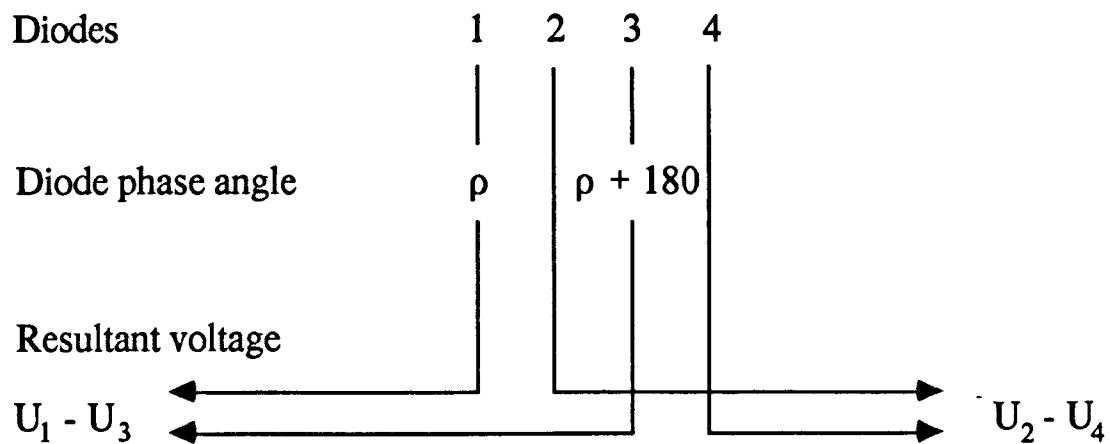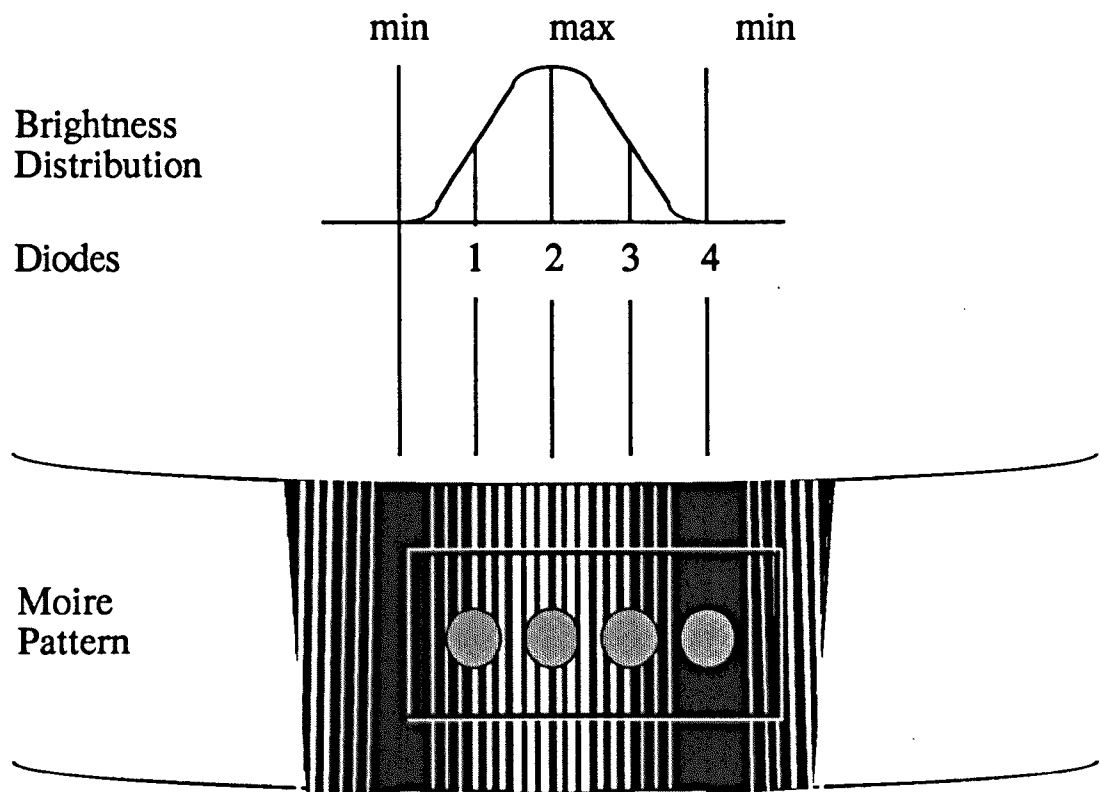
$U_1 - U_3$ $U_2 - U_4$

Fine measurement phase angle $(\rho) = \tan^{-1} \dfrac{U_1 - U_3}{U_2 - U_4}$

Fine measurement value $= \dfrac{\rho}{360} \times 0.01$ gon

Figure 3.1 Principles of Incremental / Static Circle Measurement
[Munch 1984].

[Munch 1984] and accuracy testing of the instrument is detailed in [Maurer 1985].

3.2.2.2 Absolute / Dynamic Circle Measurement

Absolute / dynamic circle measurement is used in the Wild T2000 electronic theodolite. The circle measuring system consists of a rotating glass circle which is scanned by two pairs of photodiodes, which are positioned diametrically opposite each other. Each pair of photodiodes consists of a fixed photodiode, which corresponds to the "zero mark" on the circle, and a moveable photodiode, which corresponds to the direction of the telescope. Rotation of the alidade or telescope causes a rotation of the moveable photodiodes about the circle and, unlike the incremental / static measuring system, does not cause a rotation of the circle itself.

Circle measurement is based on the rotating glass circle, which is rotated by a motor drive system within the instrument and makes a complete revolution for every circle measurement. The circle is graduated into 1024 equally spaced intervals, comprising of a reflecting line and a space. As the circle is rotated, the graduations cause a square wave signal to be generated at each photodiode, enabling coarse and fine circle measurement (Figure 3.2).

Coarse circle measurement involves determining the number of whole graduations between a fixed photodiode and its' respective moveable photodiode. For this there are reference marks on the circles. As the circle is rotated, as soon as one photodiode recognises a reference mark, a counter begins to count the number of
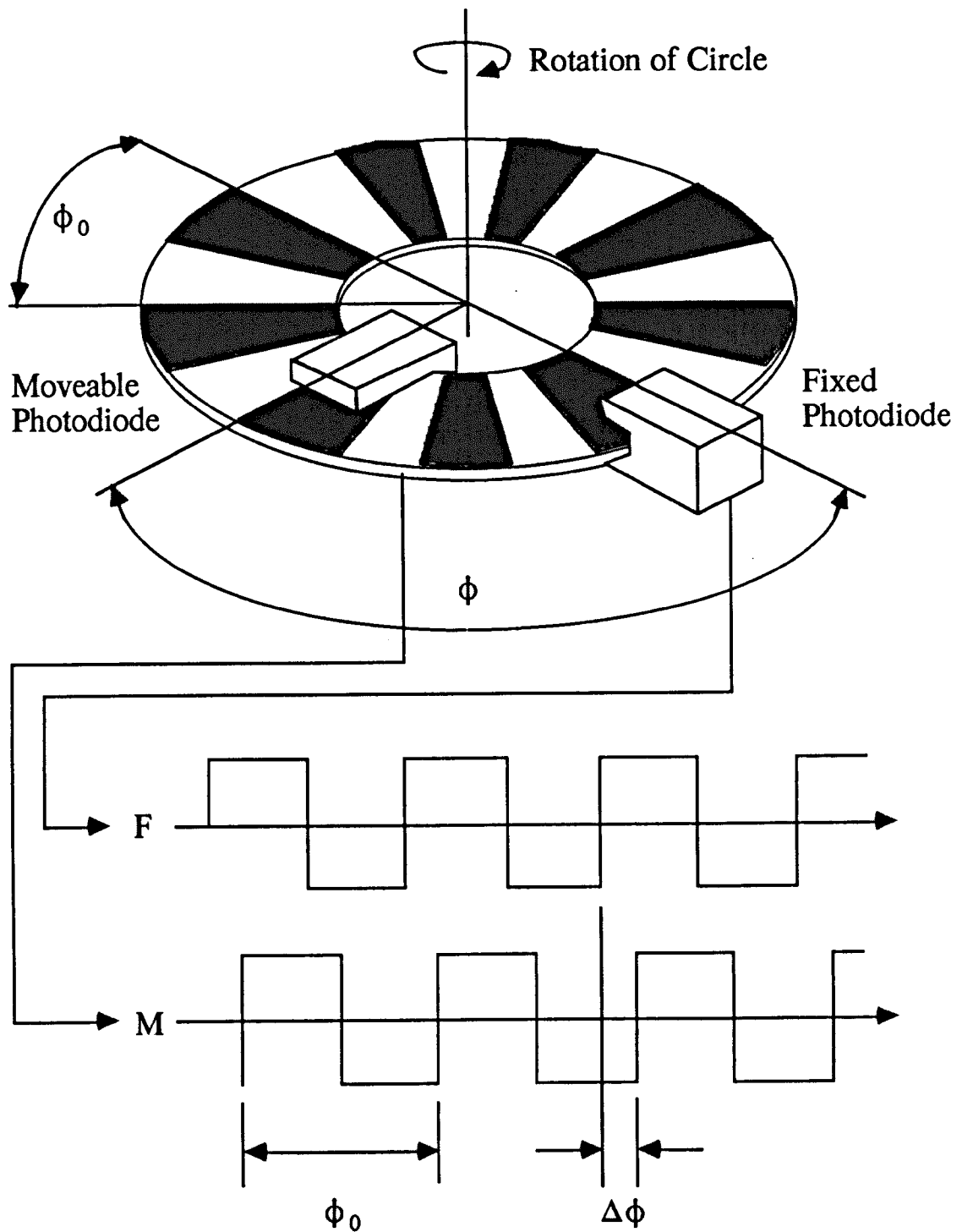
ϕ_0 = single graduation, comprising of a reflective line and a space.

$\Delta\phi$ = phase difference between the signals. = fine measurement value.

ϕ = circle reading. = $n \cdot \phi_0 + \Delta\phi$

Figure 3.2 Principles of Absolute / Dynamic Circle Measurement
[Katowski & Salzmann 1983].

whole graduations until the other photodiode recognises the same reference mark.

Fine circle measurement involves determining the phase difference between the signals from a pair of photodiodes. Over 500 measurements (from alternate pairs of photodiodes, on opposite sides of the circle) of the fine measurement, or interpolation, value are evaluated during a single rotation of the circle.

With the Wild T2000, the time taken for the entire process of obtaining a single circle reading is 0.6 seconds, half of which is used to accelerate the motor to its design speed. A coarse measuring accuracy of ±0.1 gon (20') and a fine measuring accuracy of ±0.15 mgon (0.5") are obtained. The high accuracies attainable with the instrument are due to scanning of the full circle for every reading, eliminating circle graduation errors, and scanning at diametrically opposite circle positions, eliminating the effect of any eccentricity in the circle. A full description of the circle measuring system and of the laboratory tests to assess the accuracy of the Wild T2000 can be found in [Katowski & Salzmann 1983].

3.2.3 Liquid Compensator

Vertical axis tilt, caused by mislevelment of a theodolite, can falsify the horizontal and vertical angle measurements. In electronic theodolites, a liquid compensator is used to calculate the errors in horizontal and vertical angles due to vertical axis tilt and automatically correct the output angles. This feature is especially

valuable when working over large ranges of vertical angles, a situation which can occur in ETIS measurement.

The liquid compensator is separate from the optical path of the circle scanner and the compensation values can thus be separated from the readings at any time. This is extremely useful as the values can be used to "digitally" level the instrument or for the determination of individual instrumental corrections, such as compensator zero point error.

Liquid compensators work on the principle that a liquid always presents a horizontal surface, no matter how far it is tilted. The Kern E2 Liquid Compensator is illustrated in Figure 3.3. The spot of light from an illumination diode is reflected from the surface of the liquid on to a large-area photodiode, which acts as a two-dimensional position detector. Using the photo-electric currents generated when the light spot strikes the diode, the shifted position relative to a reference point, compensator zero point, can be determined in coordinate form. The two coordinates are a measure of the inclination of the vertical axis of the theodolite, in the telescope axis direction and in the tilting axis direction respectively.

The compensator inclination components are applied to the circle readings as follows

$$z = z' + i(F)$$
$$h = h' + i(K) \cdot \cot z$$

where
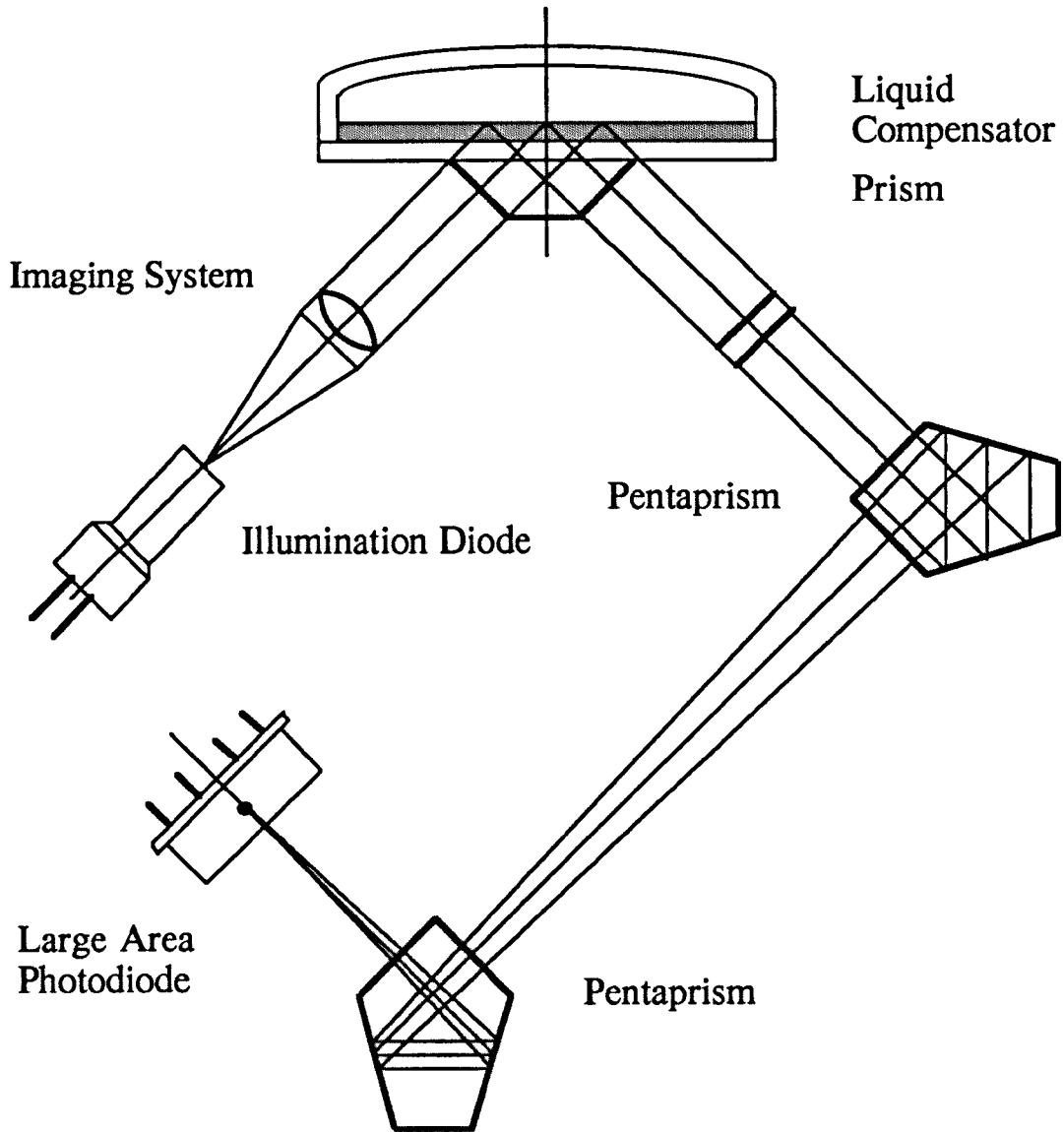
Figure 3.3 Kern E2 Liquid Compensator [Munch 1984].

z   = corrected vertical circle reading.

z'   = uncorrected vertical circle reading.

i(F) = component of the vertical axis inclination in the telescope axis direction.

h   = corrected horizontal circle reading.

h'   = uncorrected horizontal circle reading.

i(K) = component of the vertical axis inclination in the tilting axis direction.

The typical resolution of a liquid compensator, considering the Kern E2, is better than ±0.1 mgon (0.3") over a range of ±0.05 gon (2.5'). However, the compensator should not be used as a means of replacing accurate "visual" levelling of the instrument. It is always advisable to "visually" level the instrument by use of the alidade level, to ensure that the level of the instrument remains in the compensator range.

3.2.4 Circle Orientation

In ETIS measurement the coordinates of an unknown point are calculated from the horizontal and vertical angles measured from control points. An electronic theodolite positioned over a control point is used to measure the horizontal and vertical angles. The vertical angles are measured with respect to the zenith and the horizontal angles with respect to a horizontal reference angle.

The vertical circle of an electronic theodolite is zero by definition when the telescope axis coincides with the vertical axis and the lens system is pointing in the zenith direction. The deviation of the

vertical axis from the direction of gravity is independently accounted for by the liquid compensator.

The horizontal circle of an electronic theodolite can be oriented in three ways:

(i) A "zero button" on the instrument sets the circle reading to zero at the current alidade position.

(ii) The circle can be rotated independent of the instrument by means of a circle drive, as performed with optical mechanical theodolites.

(iii) Transmission of a circle value from an external data recorder or computer.

3.2.5 Collineation of Electronic Theodolites

In section 2.3.1 the horizontal reference angle baseline was defined. In practice, electronic theodolites are positioned above each control point. To establish a horizontal reference angle baseline the theodolites have to be sighted on each other and their horizontal reference angles read or set by the computer. The mutual sighting of two electronic theodolites to a high accuracy over short distances, less than 15 m, is performed by a process known as collineation.

The collineation of two theodolites is performed as follows:

(i) Mutually sight the theodolite telescopes to gain approximate collimation.

(ii) Collimate the theodolites with each focused near infinity.

(iii) Collimate the theodolites with each focused on a point inbetween them.

(iv) Repeat steps (ii) and (iii) until collimation is maintained in both cases. The theodolites are then collinear.

With the theodolites focused as outlined in steps (ii) and (iii), their cross hairs should be visible from each other. Collimation is performed as follows:

(i) Note the circle readings in both theodolites.

(ii) Finely adjust the horizontal and vertical circles of theodolite 1 until the cross hairs of both theodolites are in coincidence. Note the angular adjustment (seconds of arc) that was required in both circles of theodolite 1.

(iii) Adjust theodolite 1 by one half of the total angular adjustment required and theodolite 2 by the other half. The cross hairs of both theodolites should be in coincidence again.

It is important that when adjusting the focusing of the theodolites, the displayed circle readings should remain constant. These values need to be noted before touching the telescope, as they may change slightly as focusing is performed.

When the theodolites are collinear the horizontal reference angles can be read from the theodolites or set by the computer. The reciprocal vertical angles can also be read if setting up by accurate

distance measurement between control points or by the use of a calibrated scale bar.

3.2.6 Checking, Determination and Adjustment of Instrumental Errors

It is essential that electronic theodolites are not treated as black boxes which only need switching on, a case which can occur if they are being used by non-surveyors. Electronic theodolites are precise instruments which require checking and adjustment if the claimed high accuracies are to be obtained.

With all theodolites, mechanical adjustments can be made to reduce instrumental errors. Certain errors are present which can be determined by the observer using suitable measuring and calculation procedures. With optical mechanical theodolites these errors can only be corrected by mechanical adjustment, whereas with electronic theodolites these errors can be determined and automatically included in computation procedures.

Collimation error is the error in horizontal circle readings taken on opposite faces of a theodolite. It can be determined by sighting to a fixed point and measuring the horizontal circle reading on both faces. This error is removed by adjusting the reticule sideways by means of adjustment screws on the telescope itself.

The alidade level can be adjusted mechanically, although the use of the compensator for digital levelling means any error in the alidade level is cancelled. However, it is still important to adjust the

alidade level regularly as it serves as a method of "visually" levelling the instrument prior to the final "digital" levelling.

Index error is the error in vertical circle readings taken on opposite faces of a theodolite. It can be determined by sighting to a fixed point and measuring the vertical circle reading on both faces. With an optical mechanical theodolite the instrument would have to be mechanically adjusted until the opposite circle readings are equal, whereas with an electronic theodolite, the index error can be determined by the instrument and included in vertical angle measurements. When the index error has been determined it can be applied as a temporary or permanent adjustment. A permanent adjustment is usually performed by setting a permanent index error value on a series of switches inside the instrument. A temporary adjustment is applied from memory whilst the instrument remains switched on, the adjustment being lost when the instrument is switched off.

3.2.7 Automated Electronic Theodolites

Automated electronic theodolites have recently been introduced by the Kern and Wild companies, as the Kern E2-SE and the Wild TM3000V respectively. In all ETIS measurement the electronic theodolites are required to sight accurately to a series of targets. To perform this automatically the automated electronic theodolite incorporates a CCD array camera, laser and motor drive system for focusing and turning angles.

With a non-automated electronic theodolite, such as the Kern E2, the coarse and fine movement of the instrument and the centring of the reticule on the target is performed manually by the operator. This manual operation can be extremely time consuming, and the correct centring of the reticule is subjective and, therefore, can't be guaranteed.

The first step towards automating the electronic theodolite was taken by the use of servo motors to drive the theodolite axes. These made it possible to automatically drive the theodolite axes to known positions. The next step was to automate the subjective element of centring the reticule accurately on a target. This automatic centring is made possible by incorporating a CCD array camera, with wide-angle optics, into the telescope of the theodolite. Image-processing software is used to determine the preliminary, approximate, position of each target whose image reaches the camera's image plane. The software then determines the adjustment required in each of the theodolite axes, by comparing the approximate target position with the telescope axis, and drives the theodolite to point accurately at the target.

A processor is included in the automated electronic theodolite to control and coordinate the measuring process and its sequence. In general, once a target is within the field of view of the CCD array camera, then the accurate centring of the reticule on the target is automatically controlled by the processor. The target can be placed in the field of view of the automated electronic theodolite by two methods; remote operation or computer control. In remote operation, a theodolite is guided by an operator, usually by the use of a joystick, as the camera image is viewed on a monitor. With computer control, if

the approximate coordinates of target points are known then the computer can drive the theodolites to place the target in the field of view.

To enable target recognition by the automated electronic theodolite, either, special targets are fixed to an object or a laser light spot target is used. If a laser light spot target is used, then one theodolite has to incorporate a laser and motor driven focusing. For more details on the automation of electronic theodolites the reader is referred to [Katowski 1987].

3.3 Computer Hardware and Software

The function of the computer in an ETIS is to process, store and analyse the measured data, using various software routines. The computer hardware should permit interfacing to electronic theodolites, and should be capable of providing both screen and hard copy output of data, in an understandable format.

The minimum requirement of the software is that it is capable of performing all of the calculations detailed in Chapter One, in the field. In addition, specific software may be required to enable manipulation and analysis of the measured data, either in the field or in the office at a later stage. For example, in industrial measurement, the software should be capable of manipulating geometrical figures, such as lines, circles, planes and spheres, formed from measured points on an object [Grist 1986b]. Furthermore, the software may be required to output the measured data in a format suitable for input to non-ETIS software or a CAD system.

3.3.1 Computer Hardware Configuration

In general, there are two possible cases of computer hardware configuration; mains powered and battery powered. Mains powered computer hardware configurations consist of:

(i) A microcomputer, including screen, keyboard and internal memory.

(ii) External memory devices, ie hard disk drive and / or floppy disk drive.

(iii) Hard copy output devices, ie printer or plotter.

The computer hardware is powered from the mains, via a power distribution unit, which also drives the electronic theodolites. The computer hardware and power distribution unit are housed in a purpose-built trolley during measurement. In addition, the trolley is designed to store the electronic theodolites and cables, to increase the portability of the system.

In order to overcome the constraint of requiring mains power, battery powered computer hardware configurations can be used. These consist of:

(i) A field or lap-top microcomputer, with integral screen, keyboard and internal memory.

(ii) An external memory device, ie floppy disk drive.

(iii) Hard copy output devices, ie printer or plotter.

Each component is powered by internal batteries and each electronic theodolite is powered individually by re-chargeable batteries.

The greater capacity of a mains powered configuration has the following advantages over a battery powered configuration;

(i) unlimited measurement time,

(ii) faster processing of data,

(iii) greater data storage capacity,

(iv) on-line help facilities, and

(v) higher resolution output of graphical data.

Whereas, the battery powered configuration has the advantage of being a truely portable system which can be used practically anywhere.

3.3.2 Interfacing of Electronic Theodolites to a Computer

A basic requirement of the computer in an ETIS is that it permits interfacing to electronic theodolites. The RS 232 interface connection is a data communications standard established by the Electronic Industries Association (EIA) for setting up electrical and mechanical interface requirements. This standard interface is now included in most microcomputers and all electronic theodolites, permitting the transmission of data in either direction.

In the interfacing, the electronic theodolite is considered as the "slave" and the computer as the "master". The electronic theodolite reacts to controls from the computer and only transmits data on request

from the computer. The data which can be requested by the computer includes measured values, such as horizontal and vertical circle readings, and status parameters, such as internal switch settings, index error and compensator inclination components. In addition, the computer can transmit data to the electronic theodolites including horizontal circle readings and display prompts, such as the calculated values in setting out.

Since several devices, such as electronic theodolites and electronic distance meters, can be connected to a computer simultaneously, they must be labelled to distinguish and address them individually. Device types are distinguished by a character determined by the software in the device. For example, in the Kern ASB (ASCII Single Bus), an electronic theodolite is labelled with a T and an electronic distance meter is labelled with a D. If more than one device of the same type is connected they are labelled further with a device number from 1 to 9, which is set for the device by an internal switch. For example, in the Kern ASB, theodolite 1 is fully labelled as T1 thus enabling the computer to specifically address this device.

3.3.3 Computer Software Requirements

Software is required to process, store and analyse the measured data. In general, ETIS software can be divided into eight sections, namely:

    (i)   Administrative data input.

    (ii)   Setting up a coordinate system.

    (iii) Coordinate measurement.

(iv)   Transformation of coordinate systems.

(v)   Special functions.

(vi)   File handling.

(vii) Data transfer.

(viii) Help functions.

Administrative data input allows specific job parameters such as the date, time, location, nature of the measurement survey and operator names to be stored for future reference. As stated previously, the minimum requirement of software for an ETIS is that it is capable of performing all of the calculations detailed in Chapter One, in the field. These calculations are contained in sections (ii), (iii) and (iv).

Special functions include; the formation and manipulation of geometrical figures, such as lines, circles, planes and spheres, formed from measured points on an object; the setting out of points of known coordinates and the comparison of coordinates in different data files.

The file handling function permits the creation of new files, editing of points in files and the ability to list the measured data of any point in any file. The data transfer option permits the transfer of measured data from the ETIS computer to another computer, for example, from a battery powered field computer to an office computer or from an ETIS computer to a CAD system.

Help functions are designed to provide on-line instruction on the use of a system. Ideally, they should be available at any stage of a measurement survey. If available on request, they can provide

invaluable help to operators who are learning to use a system and are not obstructive to operators who are familiar with a system.

Software for an ETIS can be purchased, developed in-house or a combination of both. Purchased software is readily available and can be tailored to the users requirements prior to purchase. However, it can form an expensive part of the system and it can be difficult to decide on the exact software required at the time of purchase. This can lead to the system containing non-applicable software or, more importantly, lacking important software required at a later date. If the software is developed in-house, although time and money are required to develop the software, it can be easily updated to suit specific user requirements. A good compromise is obtained by purchasing the ETIS software and developing in-house software to analyse the measured data as required.

3.4 Peripheral Components

In addition to the electronic theodolites, computer hardware and software there are several peripheral components which are required to complete an ETIS. Although, these items are often the cheapest components of a system, in some ways they are the most important and careful consideration should be given to their selection for a specific measurement survey.

The peripheral components which need to be considered are:

(i) Theodolite stands.

91

(ii) Targets.

(iii) Calibrated scale bar.

3.4.1 Theodolite Stands

In ETIS measurement, it is generally the case that the theodolites need not be precisely located over or under specific station marks. The theodolites are mounted on some form of stand which should support the theodolite at a comfortable height for the observer and, most importantly, ensure the stability of the theodolite throughout a measurement survey. There are three types of stand, pictured from right to left in Plate 3.1, which may be used as part of an ETIS, namely:

(i)   Conventional survey tripod.

(ii) Industrial measurement stand.

(iii) Permanent monument.

A conventional survey tripod consists of three adjustable legs made of wood or a light alloy. This is convenient to use as it allows the theodolite to be set up at any height and is easily transported. However, it is not particularly stable over long periods of time, although the stability may be improved by the use of a floor support or the location of the legs in drill holes.

An industrial measurement stand generally consists of a heavy central tube supported at its' base by three feet. This is stable to the point where it may be considered as an adjustable permanent monument, as the theodolite height can be adjusted to suit the observer. The stand can be transported over short distances by wheels, which are

Plate 3.1

Theodolite Stands.
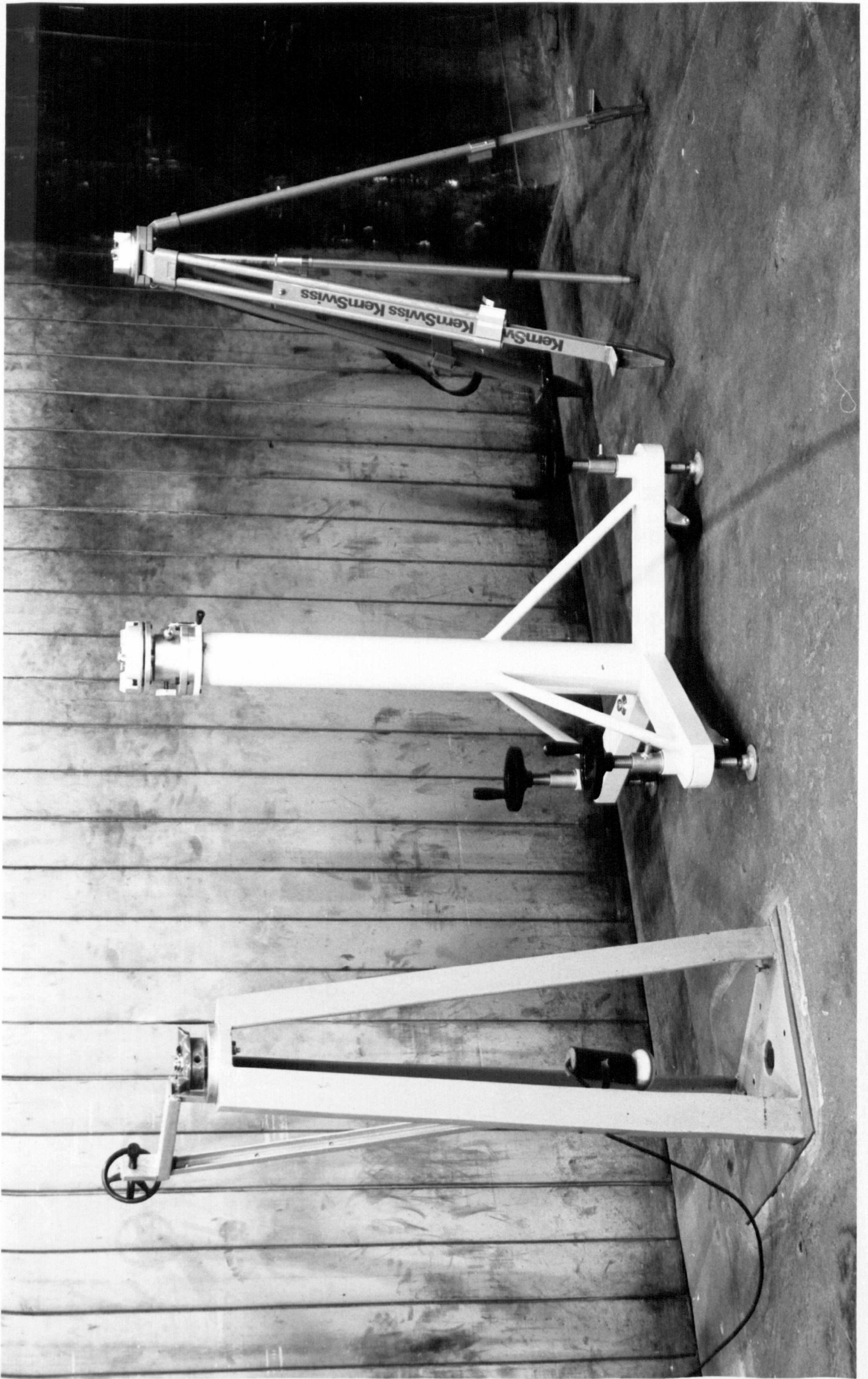
then raised by three screw jacks on the feet, however, the weight of the stand make it difficult to transport over any significant distance, for example, by car. An attempt to combine the stability of an industrial measurement stand and the portability of a survey tripod is the "flat-pack" stand, which was developed at the National Engineering Laboratory and is detailed in [Woodward 1987].

The most stable form of theodolite stand is a permanent monument. This may take the form of a concrete or steel pillar incorporating a conventional tripod head and is usually constructed for a specific measurement task. The permanent monument is useful if numerous measurement surveys are to be performed from the same control points or if maximum stability is required for a particular application.

3.4.2 Targets

The pointing and centring of the theodolite reticule on to the target is fundamental to achieving the optimum precision from an ETIS [Grist 1986b]. Ideally the target should present an image, the sighting of which gives rise to no discrepancy in the centring of the theodolite reticule. However, in practice this is never achievable and a compromise has to be reached where the target presents a sufficient image at varying distances and from varying directions.

There are many possible targets which can be used with an ETIS, however, these can be split in to three main types, namely:

(i)   Plane targets.

(ii) Three-dimensional targets.

(iii) Laser light spot target.

Plane targets are usually self-adhesive and take the form of concentric circles (Plate 3.2), crosses or a combination of both. The target image can be viewed from varying distances but will only remain in a perfect form when viewed from a single direction. When viewed from other directions the target image becomes distorted, ie the concentric circles are viewed as ellipses, and centring of the theodolite reticule is difficult.

Three-dimensional targets are precisely manufactured objects, such as spheres or cones, made of plastic or metal. They are designed to present a suitable target image from all directions. Spherical targets are available in various sizes (Plate 3.2) and are designed to present a circular target image, whereas, cone targets are designed so as the point at the top of the cone forms the target image. Three-dimensional targets are usually mounted on a base which has a screw thread at one end. The screw thread permits the target to be located, either, directly on the object or in a specially manufactured mount, depending on the specific measurement task. However, it must be noted that when measuring actual dimensions on an object, the height of a three-dimensional target assembly has to be determined and taken into account in the calculations.

With both plane and three-dimensional targets, in addition to the type and size of target required for a measurement task, the correct illumination of the target to suit all observers is a critical consideration. Correct illumination can be achieved by the use of self-

Plate 3.2

Calibrated Scale Bar and Targets.

illuminable three-dimensional targets or a number of external light sources.

There are some applications where it is impractical to position targets on the object itself, because it is inaccessible or in a hazardous environment. If the object has no points of easily identifiable detail, then measurements become difficult if not impossible. Under these circumstances a laser eyepiece can be used to form a laser light spot target on the object. In this case a laser beam is projected through the optical system of one theodolite along the optical axis and focussing of the telescope has the effect of focussing the laser beam, thereby producing a light spot target for other theodolites to observe [Grist 1986b].

For automated electronic theodolites the target has to be automatically recognised by the theodolite. The targets used are, either, a specially manufactured plane target or a laser light spot target. An example of a specially manufactured plane target is that used in conjunction with the Kern E2-SE. This takes the form of a white circle which, tests have shown, produces the best accuracy [Zhou 1986].

3.4.3 Calibrated Scale Bar

In short-range ETIS measurement, involving baselines of less than 15 m in length, the setting up of a coordinate system usually involves the use of a calibrated scale bar. This can be made of invar or carbon fibre, but carbon fibre is generally used due to its' light nature and negligible coefficient of expansion. The scale bar should incorporate two precisely defined targets and be calibrated by

determining the distance between the two targets to a high accuracy, typically 1 μm, by laser interferometry.

Ideally, the length of the calibrated scale bar should be similar to the size of the object being measured. However, due to problems in stability and transportation a 1 m calibrated scale bar, as shown in Plate 3.2, is used in most cases.

3.5 Conclusions

In purchasing, or developing, an ETIS the initial consideration should be the main components; electronic theodolites, computer hardware and software. These can be considered as fixed and should, therefore, be suitable for use in a variety of measurement surveys.

With each individual measurement survey, careful pre-planning should be performed, to select the ideal peripheral components; theodolite stands, targets and calibrated scale bar, in order to obtain the best results with an ETIS.

In general, if an ETIS is to be purchased there are several commercially available systems. These systems are based around the manufacturers own electronic theodolites and software. However, the manufacturers software is generally capable of running on a variety of computer hardware and of controlling up to eight electronic theodolites. If developing an ETIS, any electronic theodolites and computer hardware can be used, and in-house software developed to suit specific user requirements. Chapter Four details the various commercially

available systems and Chapter Five details the development of an ETIS by the author, in the Department.

CHAPTER FOUR

COMMERCIALLY AVAILABLE ELECTRONIC
THEODOLITE INTERSECTION SYSTEMS

Commercially Available Electronic Theodolite Intersection Systems

4.1 Introduction

ETISs were originally developed by surveying equipment manufacturers in the early 1980s, for industrial measurement. This new generation of software-controlled theodolite-based measuring systems offers enormous progress in high-precision three-dimensional measurement in a wide variety of applications. The systems are portable and permit measurement in situations where, previously, it was difficult or not possible at all. In addition they can complete a complex programme of precise measurements in a fraction of the time required just a few years ago.

Throughout this decade there have been several commercially available systems developed, all based on the same principles but differing in their components, operation and complexity. From the early manual systems to the latest automated systems this Chapter aims to outline the developments which have taken place in the 1980s and to compare and contrast the current commercially available systems.

4.2 Development Chronology

At present ETISs are being applied to perform a wide variety of measurement tasks, although they were first developed to enable the use of coordinates for industrial measurement. The first system was proposed in the USA by Hewlett Packard when, in 1980,

they introduced their 3820A Coordinate Determination System. Following this another American company, Keuffel & Esser introduced AIMS (Analytical Industrial Measuring System) in 1981. The Hewlett Packard Coordinate Determination System is no longer manufactured and the Keuffell & Esser AIMS is not available in the UK [Price 1989].

The two surveying equipment manufacturers who have dominated the market in the UK have been Wild and Kern, and the developments by both of these companies have been similar. At about the same time that the Keuffell & Esser AIMS became available, Wild produced their Remote Measuring System (RMS2000) and shortly afterwards in 1983, Kern produced their first Electronic Coordinate Determination System (ECDS1).

The Wild RMS2000 and Kern ECDS1 were both mains powered systems, however, both companies also saw the need for battery powered systems. Wild developed an off-line version of the RMS2000, the Wild Mini RMS2000, and Kern were involved in the development of the Optimal DCS-200 (Dimensional Control System), which became available in 1986. In both cases the battery powered system was limited in the analysis it could perform in the field, but provision was made to transfer the measured data to the mains powered software for further analysis.

Since their introduction, manual systems have been improved through further developments by the Kern and Wild companies. The Kern ECDS1 has been superseded by the Kern ECDS2, which became available in 1986, and the Optimal DCS-200 is now a self-contained battery powered system, marketed by Optimal Software

Ltd alone. The Wild RMS2000 and Mini RMS2000 have been superseded by the Wild TMS (Theodolite Measuring System), which became available in 1988 and can be run on, either, a mains powered personal computer or a battery powered lap-top computer.

Similarly, both companies have also developed automated systems; the Kern SPACE (System for Positioning and Automated Coordinate Evaluation), which became available in 1987, and the Wild ATMS (Automated Theodolite Measuring System), which became available in 1988.

4.3 Manual Systems

The first ETISs developed were manual systems, so called as their operation is performed manually. With a manual system, personnel are required to operate the main components of the system, namely, the electronic theodolites and the computer. A single operator is required for each electronic theodolite and for the computer, although the computer operator may also operate an electronic theodolite.

The early manual systems, the Wild RMS2000 and the Kern ECDS1, were superseded by the current manual systems for a number of reasons:

(i) They were restricted in the number of theodolites they could use - current systems permit the use of up to eight theodolites.

103

(ii) They only operated on specific computer hardware - current systems permit the software to run on a wide selection of hardware.

(iii) The original software did not permit the transfer of data to non-ETIS software or CAD systems.

There are five manual systems detailed in this Chapter, namely the; Wild RMS2000, Kern ECDS1, Optimal DCS-200, Kern ECDS2 and Wild TMS. Applications of manual systems are then given in section 4.3.6.

4.3.1 Wild RMS2000

The Wild RMS2000 was the first ETIS available in the UK. There are two versions of the RMS2000, on-line and off-line. The on-line version is mains powered and permits the use of up to four theodolites. Each theodolite is connected by a cable, via an electronic multiplexer, to the computer. The cables provide for the bi-directional data transfer between computer and theodolite, and also supply power to the theodolites. The off-line version is battery powered and permits the use of only two theodolites. The theodolites are linked to a Wild GRE3 data terminal, which records the theodolite circle readings and computes the target point coordinates. This recorded data is then transferred to the RMS2000 computer for further processing and analysis, thus, the entire RMS2000 software is available to the off-line system.

Both versions, the on-line and off-line, use Wild T2000 or T2000S electronic theodolites, the specifications of which are included in Table 4.1.

	Wild T2000	Wild T2000S
Telescope Specification		
Type	anallactic	panfocal
Magnificaton	32x	43x
Shortest Focusing Distance (m)	1.7	0.6
Measurement Specification		
Circle Measurement System	A / D	A / D
Circle Display Units:		
400 gon	yes	yes
360 sexagesimal	yes	yes
360 decimal	yes	yes
6400 mil	yes	yes
Smallest Unit Displayed:		
mgon	0.01	0.01
”	0.1	0.1
degrees	0.00001	0.00001
mil	0.0001	0.0001
Angle Measuring Accuracy:		
mgon	0.15	0.15
”	0.5	0.5
Compensator Specification		
Type	twin-axis	twin-axis
Working Range:		
gon	0.2	0.2
’	10	10
Accuracy:		
mgon	0.03	0.03
”	0.1	0.1

Table 4.1 Specifications of Wild T2000 and Wild T2000S Electronic Theodolites.

105

The RMS2000 software was developed to run on a Wang 2200 series computer which has 64 Kbytes of RAM, a 4 Mbyte hard disk drive and a 1 Mbyte floppy disk drive. In addition, the system permits the use of any RS 232 compatible printer.

The RMS2000 was developed jointly by two companies, Wild and Leitz. Leitz originally developed the MESCAL software for their three-dimensional coordinate measuring equipment and the continuous development of this software, over a number of years, resulted in software for the RMS2000. The RMS2000 software comprises of routines for the various setting up procedures, measurement, connection of geometrical elements and the transformation of a local coordinate system to an object coordinate system.

The software permits setting up by all of the methods detailed in section 2.3, excluding the bundle adjustment. However, colineation is aided by the presence of targets on the theodolites. The Wild T2000S has an internal target and the Wild T2000 has an external target mounted on top of the telescope.

When an object is being measured, any geometrical elements may be determined directly, for example, straight lines, circular arcs, planes, spherical and cylindrical surfaces. From these measured elements, new elements can be formed, for example, spatial distances, perpendiculars, parallels, angles and lines and points of intersection. Similarly, a series of points obtained by measurement can be assessed for straightness, flatness or roundness. Thus, the system is capable of determining the dimensions of an object, as shown on

technical drawings, and performing checks, on such things as surface quality, angularity and straightness.

Full details of the Wild RMS2000 can be found in [Katowski 1985]. As the RMS2000 was the first ETIS released in the UK many systems were initially sold. However, in 1989 the Wild and Kern companies merged and, as far as the UK is concerned, are marketing the Kern ECDS2 in preference to the Wild RMS2000. In addition, in the USA, the RMS2000 has now been superseded by the Wild TMS.

4.3.2 Kern ECDS1

The Kern ECDS1 is mains powered and permits the use of only two theodolites. Each theodolite is connected by a cable, via a power distribution unit, to the computer. The cables provide for the bi-directional data transfer between computer and theodolite, and also supply power to the theodolites. The system uses either, Kern E1 or Kern E2 electronic theodolites, the specifications of which are included in Table 4.2.

The system software was developed to operate specifically on a DEC micro PDP-11 computer, which has 256 Kbytes of RAM, a 10 Mbyte hard disk drive and twin floppy disk drives, each with a capacity of 400 Kbytes. The system permits hard copy output by means of a DEC LA 50 matrix printer, or an optional plotter.

The software can be split into four sections; Project Definition, System Orientation, Data Capture and Special Functions.

	Kern E1	Kern E2
Telescope Specification		
Type	anallactic	anallactic
Magnificaton	32x	32x
Shortest Focusing Distance (m)	1.5	1.5
Measurement Specification		
Circle Measurement System	I / S	I / S
Circle Display Units:		
400 gon	yes	yes
360 sexagesimal	yes	yes
360 decimal	optional	optional
6400 mil	optional	optional
Smallest Unit Displayed:		
mgon	1	0.1
"	2	1
degrees	*	*
mil	*	*
Angle Measuring Accuracy:		
mgon	0.6	0.15
"	2	0.5
Compensator Specification		
Type	twin-axis	twin-axis
Working Range:		
gon	0.05	0.05
'	2.5	2.5
Accuracy:		
mgon	0.03	0.03
"	0.1	0.1

Table 4.2 Specifications of Kern E1 and Kern E2 Electronic
Theodolites.

The two theodolites are connected on-line to the computer, enabling the continuous transmission of the theodolite circle values to the computer. From these values, the main "Data Capture" function computes the coordinates of the target point and displays them on the screen. The screen display shows the current theodolite circle readings, the three-dimensional coordinates of the target point and the mis-intersection in the z coordinate. Measurement to a target point is performed by entering a point name and recording the displayed data at the press of a button. This data can then be printed out at any stage, along with any comments about the target point.

Setting up of a coordinate system is performed by the "System Orientation" function, which enables setting up by the use of a calibrated scale bar, and the transformation of a local coordinate system to an object coordinate system.

With the "Special Functions", the operator has access to a series of subroutines for the investigation of the geometrical relationship of measured points or theoretical points, derived from the measured points. The functions available are;

    (i)   spatial distances between points,

    (ii)   alignment checks and flatness of planes,

    (iii) relative position of two straight lines,

    (iv) relative position of two planes,

    (v)   circle and centre computation,

    (vi) position of points on spherical and parabolic surfaces,

and

Plate 4.1

Kern ECDS1.
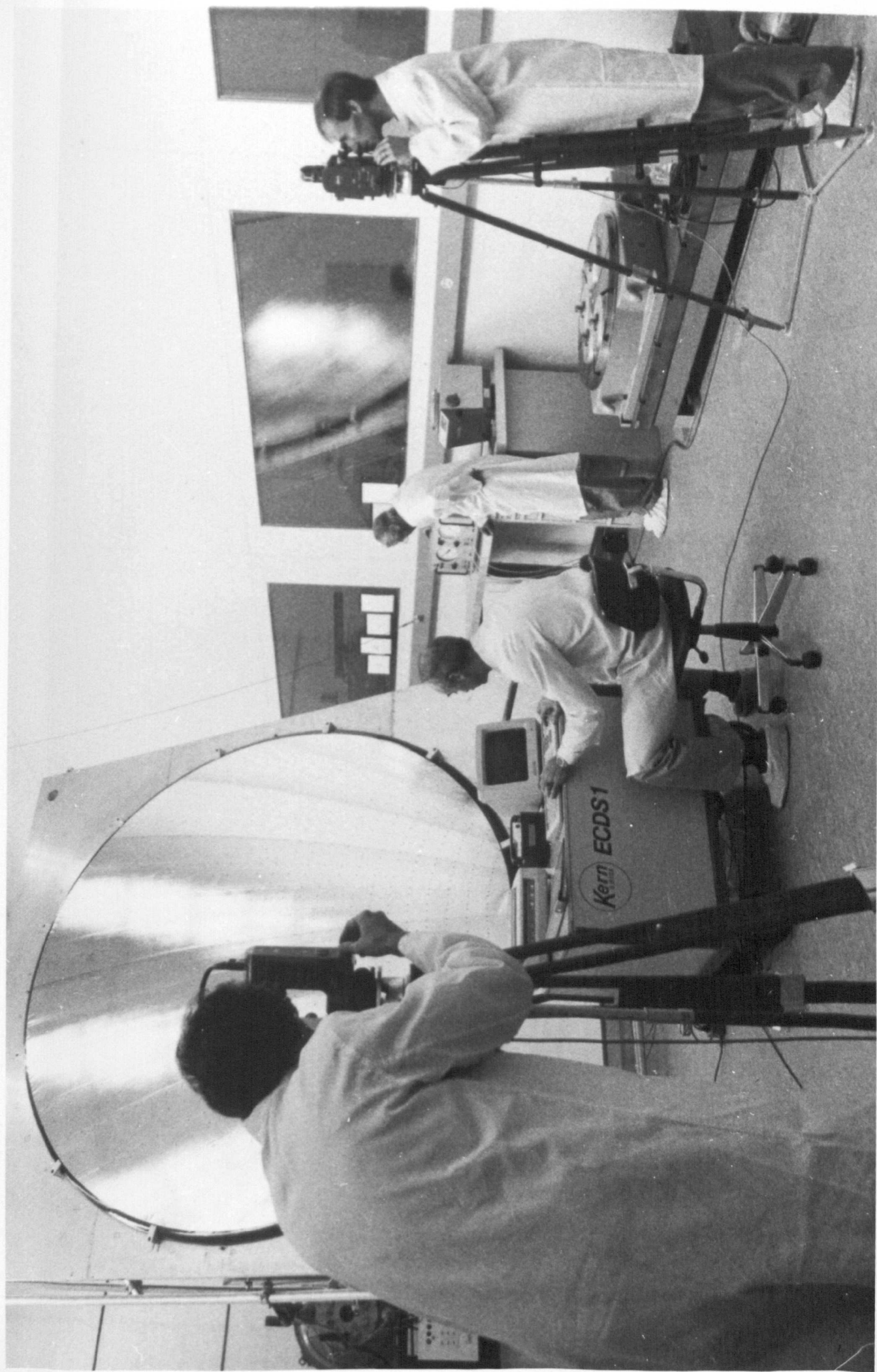
(vii) comparison of the coordinates of points in different data files.

These special functions enable the system to output immediate statements concerning the status of the object, on the screen or printer. A further special function is the laying-out function, which permits the user to define the coordinates of a point. The system calculates the individual theodolite circle readings required to intersect the point, enabling the point to be set out to a high accuracy.

Plate 4.1, on the previous page, shows the Kern ECDS1 performing measurements to determine the shape of a soft-top antenna reflector. Full details of the Kern ECDS1 can be found in [Lardelli 1984]. For the reasons stated previously, in section 4.3, the ECDS1 was superseded by the ECDS2 in 1986.

4.3.3 Optimal DCS-200

The Optimal DCS-200 consists of two or three electronic theodolites, a battery powered computer with software, and battery powered peripherals. As the system is not dependent on mains power, it can be considered truely portable.

The DCS-200 permits the use of any electronic theodolites and also allows manual entry from conventional optical mechanical theodolites. The electronic theodolites are usually connected to the computer by cables, enabling on-line, continuous transmission of the theodolite circle values to the computer. However, a further

enhancement to the system is the ability to link the theodolites using a telemetry system.

The system is based around the Husky Hunter, a relatively low cost, ruggedised, hand-held computer with an internal memory of 352 or 496 Kbyte, and which can be linked to a wide selection of peripherals. The software can be split into four sections, namely; Measurement, Coordinate Manipulation, Coordinate Analysis and Coordinate Transfer.

The "Measurement" function, includes the setting up of a coordinate system and coordinate measurement. Setting up can be performed by all of the methods detailed in section 2.3, the bundle adjustment having been recently included in the software, as reported in [Leitch et al 1988]. Coordinate measurement can be performed by intersection, and also by polar location using an Electronic Total Station.

All of the observed target point coordinates are determined in a local coordinate system, from which the "Coordinate Manipulation" function can transform them to an object coordinate system. Transformation of target point coordinates can be performed in real-time, as they are measured, or at a later stage.

With the "Coordinate Analysis" function, the DCS-200 software has a number of analysis modules, all of which may be operated in real-time [Thompson 1987]. The modules available are:

(i)   Mean Points - This module enhances the precision of the system by allowing the operator to measure a target point a number of times and then compute the mean, with interactive editing of the individual observations allowed.

(ii) File Comparison - This module enables the user to compare points from two different files. The discrepancies between the compared points are displayed and checked against a pre-set tolerance.

(iii) Fitting to Geometrical Figures - A number of modules exist to determine how closely a series of target points match a selected geometric shape. These include line functions, plane functions, circle and sphere functions, with the software permitting the use of up to fifty points to define each shape.

(iv) Distances - The distances between any two target points in each axis direction, in any axis plane and in three-dimensions, may be computed.

(v) Setting Out - Points of known coordinates can be set out to a high accuracy as the software calculates the individual theodolite circle readings required to intersect the point.

The "Coordinate Transfer" function has been developed to enable the DCS-200 to be linked to peripheral hardware, CAD systems or other computers. The DCS-200 has a well developed file handling section which enables up to 5 000 points to be held on the computer at any time. The points are held in files of up to 500 points, which can be transferred to a battery powered floppy disk drive for external storage. Similarly, a printer can be linked to the computer to permit a printout of the file data, or of the screen display. This can be performed in the field, on a battery powered printer, or later in the office, on a mains powered printer. Furthermore, the "Coordinate Transfer" function

113

permits two way transfer between CAD systems or other computers, such as a mains powered computer containing Kern ECDS2 software. This enables design dimensions to be loaded in to the DCS-200, prior to a measurement survey, and downloading of measured data for further analysis and permanent storage, after a measurement survey.

4.3.4 Kern ECDS2

The Kern ECDS2 permits the use of up to eight electronic theodolites, and a wide selection of computer hardware. The system is mains powered and each theodolite is connected by a cable, via a power distribution unit, to the computer. The cables provide for the bi-directional data transfer between computer and theodolite, and also supply power to the theodolites. The system uses Kern E2 electronic theodolites, the specifications of which are included in Table 4.2.

The software for the ECDS2 is available for the MS-DOS operating system and was written for the IBM PC/XT/AT series. This means that any IBM compatible system, such as the Hewlett Packard Vectra, can be used, where the computer hardware configuration includes [Freeman 1987]:

(i) A computer with 512 Kbyte of RAM, a maths co-processor and a colour screen.

(ii) A hard disk, with 10 Mbyte capacity, and at least one floppy disk drive.

(iii) Any RS 232 compatible printer or plotter.

114

The ECDS2 software can be split into six sections, namely; Project Definition, System Orientation, Data Capture, Special Functions, Help Functions and Data Transfer. As the ECDS2 software was upgraded from the ECDS1 software it performs all of the functions previously detailed in section 4.3.2.

In addition, the software is capable of determining target point coordinates from up to eight electronic theodolites and contains the improved "System Orientation" function to enable setting up by the bundle adjustment. The bundle adjustment enables setting up of a coordinate system, with more than two theodolites, to be performed to a higher accuracy and take no longer than setting up with two theodolites. The increased accuracy is gained by removing the need to colineate the theodolites, which also allows the theodolites to be located in very close proximity to each other, if required.

Further improvements in the ECDS2 software are the "Help Functions" and "Data Transfer" function. The "Help Functions" are available at any time to provide abbreviated instructions on the screen. The "Data Transfer" function permits the transfer of measured data to non-Kern software, for example, Metromec's Metrosoft, Autodesk AG's Autocad and Lotus Development Corporation's Lotus 1-2-3 spreadsheet, or CAD systems.

Each of the functions performed by the ECDS2 are actually separate programs such as the bundle adjustment, data capture and the individual special functions. This modularity simplifies the programming task and makes it possible for users to add their own

115

functions. Full details of the Kern ECDS2 can be found in [Lardelli 1988].

4.3.5 Wild TMS

The Wild TMS permits the use of up to eight electronic theodolites, and a wide selection of computer hardware. Each theodolite is connected by a cable, via a GIF11 interface unit, to the computer. The cables provide for the bi-directional data transfer between computer and theodolite, and also supply power to the theodolites.

An important feature of the TMS is the great flexibility and large choice of components available for specific requirements. The TMS software runs on any AT type personal computer under the MS DOS environment. Hence, the system is available as a mains powered system, running on a wide range of personal computers, or as a battery powered system, running on a lap-top computer. In addition, any Wild electronic theodolites can be used, including the T1600, T2000, T2000S, T2002 and T3000. The specifications of these Wild electronic theodolites are given in Tables 4.1 and 4.3.

The TMS software consists of several modules with different functions. The four main functions are; Control of the Theodolites, Geometrical Measurement Solutions, Analysis of Measurement Results and Data Handling. The software is menu-based, and operation is controlled from a main menu of ten options:

1 Guided 2 Station Orientation.

	Wild T1600	Wild T2002	Wild T3000
Telescope Specification			
Type	anallactic	anallactic	panfocal
Magnificaton	30x	32x	43x
Shortest Focusing Distance (m)	1.7	1.7	0.6
Measurement Specification			
Circle Measurement System	I / S	A / D	A / D
Circle Display Units:			
400 gon	yes	yes	yes
360 sexagesimal	yes	yes	yes
360 decimal	yes	yes	yes
6400 mil	yes	yes	yes
Smallest Unit Displayed:			
mgon	1	0.1	0.01
"	1	0.1	0.1
degrees	*	0.00001	0.00001
mil	0.01	0.0001	0.0001
Angle Measuring Accuracy:			
mgon	0.5	0.15	0.15
"	1.8	0.5	0.5
Compensator Specification			
Type:	single-axis	twin-axis	twin-axis
Working Range:			
gon	0.1	0.055	0.055
'	6	3	3
Accuracy:			
mgon	0.3	0.03	0.03
"	1	0.1	0.1

Table 4.3 Specifications of Wild T1600, Wild T2002 and Wild T3000 Electronic Theodolites.

2 Measurement / Orientation.

3 Build / Inspect.

4 Coordinate Systems.

5 Shape Fit.

6 Analysis.

7 Input / Output / Edit.

8 Special Applications.

9 Job Setup.

0 Change Job / Job Identifier / Comments.

The "Control of the Theodolites" function controls the transfer of data between the electronic theodolites and the computer, and enables automatic initialisation of the theodolites by the computer.

The "Geometrical Measurement Solutions" function includes the setting up of a coordinate system, coordinate measurement and the transformation of coordinate systems. The setting up of a coordinate system can be performed by all of the methods detailed in section 2.3, including the bundle adjustment, which was not available with the Wild RMS2000. In practice, it can be performed from option 1 (Guided 2 Station Orientation) or option 2 (Orientation / Measurement). Option 1 guides the operator in setting up a coordinate system with two control points, or in orienting two control points of a multiple control point set up, which is particularly useful for operators unfamiliar with the system. Option 2 permits the setting up of a coordinate system, consisting of more than two control points, and also controls coordinate measurement. In addition to the conventional measurement of target points, the software has several features such as average readings, correcting for target thickness, bolt hole measurement

and hidden point measurement. Setting out or inspection of points on an object can be performed from option 3 (Build / Inspect). The coordinates of the point to be set out are converted into circle readings, and the differences between the calculated and present circle readings are displayed in each theodolite, to guide the operator to the defined point. Finally, option 4 (Coordinate Systems) permits the transformation of coordinate systems.

In the "Analysis of Measurement Results" function the software uses single points or groups of points, obtained from coordinate measurement, to form geometrical figures which are then available for analysis. The formation of geometrical figures is controlled by option 5 (Shape Fit) which enables lines, planes, circles, cylinders, paraboloids and spheres to be formed, using least squares best fit techniques. All points and shapes can then be used in option 6 (Analysis) which includes; comparison of point coordinates, distance evaluations between all points and shapes, the definition of parallels to existing lines or planes and the determination of intersections between geometrical figures.

The "Data Handling" function provides routines for data input and output, via the RS 232 interface, and file management. Data input and output is controlled from option 7 (Input / Output / Edit) which also permits the operator to change, insert or delete data from files. Option 9 (Job Setup) enables the definition of job parameters, such as measurement units, electronic theodolite identification, operator identification, scale bar definition and hidden point rod definition. Option 10 (Change Job / Job Identifier / Comments) enables a new job

to be defined, which defines the files in which measurement survey data is stored.

The other option on the main menu is option 8 (Special Applications) which enables operators to integrate their own software modules to the system. Full details of the Wild TMS can be found in [Katowski & Huep 1988] and [Wild 1989a].

4.3.6 Applications of Manual Systems

Manual systems have numerous applications in any situation where real-time coordinate determination to an high accuracy is required, for example:

(i) In industrial measurement applications when an object is difficult to measure by a conventional method, such as a coordinate measuring machine (CMM). For example, a large object with dimensions too large to be measured by a CMM, or an object which must be measured in-situ.

(ii) In the case of a structure constructed of several fabricated parts. If the parts are fabricated separately, perhaps at several different locations, it is possible to check the dimensional relationship of the parts prior to assembly, while alterations can easily be effected, thereby eliminating the risk of costly delays on assembly. Similarly, if it is necessary to replace part of a structure, it is possible to measure the defective part in-situ before it is removed, and fabricate the new part accordingly. This not only permits trouble-free assembly of the new part, but also means that the defective part can remain in place and, perhaps, operational until the new part has been fabricated.

(iii) On any complex machinery, where parts are moveable or removeable, it is possible to align parts in a pre-determined direction.

(iv) In the calibration of industrial robots.

(v) In certain areas non-contact measurements are essential. This may be for a number of reasons, because; contact would deform the surface being measured, thereby creating erroneous observations, the object to be measured may be in a hazardous environment or the object may be inaccessible.

(vi) To enable the setting out of an object at a pre-determined point within a three-dimensional coordinate framework to a high accuracy.

(vii) Any situation where it is required to monitor the movement or deformation of a structure over a period of time. Such as, cooling towers, dams and areas subject to landslides or subsidence.

4.4 Automated Systems

Since 1987 automated ETISs have been available, so called as their operation is performed automatically. These automated systems are based on manual systems, but they use automated electronic theodolites, the principles of which are detailed in section 3.2.7. In addition, an automated system also has to include a video frame store in the computer [Zhou & Roberts 1987] and image-processing software. The output from an automated electronic theodolite is fed to the video frame store and image-processing software is then used to automatically centre the theodolite on either a pre-determined or a targeted point. The automated electronic theodolites can be driven to an accuracy of

less than one second of arc and the image-processing software is capable of centring the reticule on the target to a similar accuracy.

A typical measurement survey with an automated ETIS includes; manual setting up of a coordinate system, although the image-processing software is available for fine centring of the theodolites, automatic coordinate measurement to a number of points, pre-programmed analysis of the measured data and final results output. Thus, in theory, once the setting up procedure is complete, automated systems permit automatic unattended measurement to be performed.

There are two automated systems currently available in the UK, namely the; Kern SPACE and Wild ATMS. Details of these two systems and their applications are given in the following sections.

4.4.1 Kern SPACE

The Kern SPACE was the first automated ETIS to be developed. It combines Kern E2-SE automated electronic theodolites, ECDS2 software and vision image-processing software. The specifications of the Kern E2-SE are given in Table 4.4. As with the ECDS2, the SPACE software runs on any IBM compatible computer and the software is stored as separate programs. The vision software is also a separate program which is loaded by the SPACE system when the computer is switched on. This allows access to the vision software at all times, without affecting the size or complexity of the calling program. It is also useful to users who may wish to make use of the vision software in their own programs, or replace it with different software.

	Kern E2-SE	Wild TM3000V
Telescope Specification		
Type	panfocal	panfocal
Magnificaton	43x	43x
Shortest Focusing Distance (m)	0.6	0.6
Measurement Specification		
Circle Measurement System	I / S	A / D
Circle Display Units:		
400 gon	yes	yes
360 sexagesimal	yes	no
360 decimal	optional	yes
6400 mil	optional	no
Smallest Unit Displayed:		
mgon	0.1	0.01
"	1	*
degrees	*	0.00001
mil	*	*
Angle Measuring Accuracy:		
mgon	0.15	0.15
"	0.5	0.5
Compensator Specification		
Type	twin-axis	twin-axis
Working Range:		
gon	0.05	0.055
'	2.5	3
Accuracy:		
mgon	0.03	0.03
"	0.1	0.1

Table 4.4 Specifications of Kern E2-SE and Wild TM3000V Automated Electronic Theodolites.

SPACE has two modes of measurement, namely: automatic and profiling. In automatic mode, the targets are fixed as specially manufactured white circles, which are positioned at known approximate locations permitting very precise, fast measurement to be performed. The approximate locations of the targets are stored in a file on the computer, from where the software reads them and automatically measures each point. Profiling is performed by measuring a grid of points within a defined area. The operator has to define the boundaries of the area to be measured and the point density required. In this case, the laser of one theodolite is used to provide a target for other theodolites, whose CCD cameras are activated by the laser spot on the object. If a sufficiently dense grid is used, the distance from a theodolite to the object surface will not vary significantly from point to point, and this can then be used as an estimate of the distance to the next point. If the boundaries are well defined and the object surface is suitable then the laser light spot target will be found. However, if at any time the image-processing software can't find the laser light spot target it will search along the path of the laser and, if this fails, the operator is notified and manual intervention is necessary.

In both modes, the theodolites are moved simultaneously and then centred individually. When used in automatic mode, movement of the theodolites requires 5 seconds and the centring of each theodolite requires 1.25 seconds. Therefore, considering a two theodolite set up, a single measurement to a target point is performed in 7.5 seconds and, therefore, one hundred target points can be measured in approximately 12.5 minutes. In the profiling mode, operation is faster as the automated theodolite will accept a pointing tolerance, which speeds up the driving process. It is important to note that the accuracy

will not suffer due to this tolerance as the exact circle readings will still be read from the theodolites, and the image-processing software will correct for mis-pointing. With a pointing tolerance of 10 seconds of arc the time required to move the theodolites is reduced to 3 seconds, also, the process is speeded up further as the theodolite which incorporates the laser doesn't require centring by the image-processing software. Thus, considering a two theodolite set up, a single measurement to a target point is performed in 4.25 seconds, ie 3 seconds for movement and 1.25 seconds to centre one theodolite. At this rate, measurement of a surface requiring profiling of 1 000 points would require approximately 1 hour 10 minutes to complete.

Full details of the Kern SPACE are given in [Gottwald & Berner 1987] and [Roberts & Moffitt 1987].

4.4.2 Wild ATMS

The Wild ATMS is the automated version of the TMS. It combines Wild TM3000V automated electronic theodolites, TMS software and image-processing software to create an automated ETIS. The specifications of the Wild TM3000V are given in Table 4.4. As with the TMS, the ATMS software runs on a wide range of personal and lap-top computers.

At present, the image processing software permits the use of three types of target; laser light spot, retro-reflective and LED. In the case of the laser light spot, the target is projected from a TM3000L automated electronic theodolite. This is equipped with a speckle eliminator, which causes the projected laser dot to describe a circle, on

which 26 measurement points are made for each target point. In the case of the retro-reflective and LED targets, these are measured using a method of intensity level measurement and 50 measurement points are made per target. In each case the image-processing software determines the centroid of the target from the numerous measurement points and the centroid is the result of an ellipse fit calculated for each measurement point.

ATMS has two modes of measurement, namely; automatic and line following. In automatic mode, the targets are positioned at known approximate locations permitting very precise, fast measurement to be performed. The approximate locations of the targets are stored in a file on the computer, from where the software reads them and automatically measures each point. In line following mode, the system measures an unlimited number of targets whose approximate coordinates are not known. This routine uses retro-reflective targets which are located at equidistant intervals on a tape. For the measurement the tape has to be attached to the object and the first two points have to be aimed manually. From this initial procedure, the system can then measure the succesive points on the tape.

Furthermore, there are two interesting points to note about the operation of ATMS. Firstly, if more than two automated electronic theodolites are used then the software decides, based on intersection angles and zenith angles, which instruments should measure a specific target point. Secondly, an essential point in deformation surveys, if a target point is not detected from its' approximate coordinates, then a search process is initiated and the theodolite tracks a spiral path from its' original setting until it detects the target point.

Full details of the ATMS are given in [Wild 1989b] and [Katowski 1989].

4.4.3 Applications of Automated Systems

Automated systems can perform all of the applications of manual systems, detailed in section 4.3.6, but with several advantages:

(i)   They are faster than manual systems, which can reduce the cost of a measurement survey by some five or six times.

(ii) They are more accurate and more reliable than manual systems, as the subjective element of measurement is eliminated.

(iii) They require less personnel as, once set up, measurement is automatic and an operator is not required for each theodolite.

(iv) They allow unattended operation in hazardous areas.

In addition, automated systems have several applications which would be difficult, or impractical, to perform by a manual system, such as:

(i)   Automatic calibration of robots on automated production lines, where measurement is required continuously or at short time intervals.

(ii) Profiling of shaped surfaces, such as antennae.

(iii) Automatic monitoring in hazardous environments, such as areas subject to slope failure or landslides, thus, enabling failure to be predicted and precautionary measures to be taken.

4.5 Conclusions

This Chapter has outlined the developments in commercially available ETISs which have taken place in the 1980s and detailed the current manual and automated systems available from a number of surveying equipment manufacturers.

In purchasing a commercially available system there are several considerations to be taken into account, namely; the type of system (manual or automated), the system power source (mains or battery), the suitability of the software and the type and number of theodolites. In general, the first two choices, the type of system and system power source, are made depending on the nature of the proposed measurement tasks. The specific system purchased then depends on the latter two choices, namely, the suitability of the software and the type and number of theodolites.

The main purpose of these commercially available systems is the fast and accurate determination of coordinates for industrial measurement and engineering applications. In the aerospace and automotive industries, telecommunications and robotics, there is a constant call for ever higher quality and greater rationalisation. Increasingly, therefore, these industries are facing measuring tasks for which conventional methods are no longer adequate, and it is in these industries that commercially available systems have found their major applications to date.

CHAPTER FIVE

THE DEVELOPMENT OF AN ELECTRONIC THEODOLITE INTERSECTION SYSTEM AT THE UNIVERSITY OF NOTTINGHAM (UN ETIS)

The Development of an Electronic Theodolite Intersection System at the University of Nottingham (UN ETIS)

5.1 Introduction

In October 1986 the Department was awarded a Science and Engineering Research Council grant (Grant Ref No: GR/D/86164) to obtain all the equipment and instrumentation necessary to develop an ETIS. This system has been named UN ETIS.

In order that UN ETIS be flexible for use in a variety of measurement tasks two initial decisions were taken. Firstly, the system would be split into two separate halves, namely; field components and office components, and secondly, the software would be developed in-house. The development of the system began in October 1987 with the aims of:

(i) Selecting and purchasing the individual field and office components of the system.

(ii) Developing software on a field computer to perform the necessary field calculations and record the measured data for later analysis.

(iii) Developing software on an office computer to process, store and analyse the measured data.

This Chapter outlines the selection and specification of the individual field and office components, and details the in-house development of the field and office software.

5.2 Field Components of UN ETIS

The field components of UN ETIS are two Kern E 2 electronic theodolites and a Husky Hunter 2 computer, with peripheral hardware and in-house developed field software (Figure 5.1).

The field components are battery powered. The Husky Hunter 2 computer transmits data to, and from, the two Kern E2s. The simultaneous connection of both theodolites to the computer is facilitated by two cables, and an interface box. The interface box is connected to the computers RS 232 interface, and can be removed at any time to permit a floppy disk drive, or printer, to be connected directly to the computer.

The minimum requirement of the software for an ETIS is that it is capable of performing all of the calculations detailed in Chapter One, in the field. The field software for UN ETIS has been developed by the author to meet this specification, and to permit the transfer of measurement survey data to an office computer.

5.2.1 Kern E2 Electronic Theodolites

The first stage in developing UN ETIS was the selection of the electronic theodolites. Recent research, within the Department, had been concerned with the evaluation and application of Electronic Total Stations (ETSs). As part of this research, in 1981, the Department acquired on loan a Kern E1 (299896) for evaluation. In 1982, the Department purchased a Kern E1 (302189), which was replaced by a Kern E2 (327974) in 1984. Extensive evaluation of these three
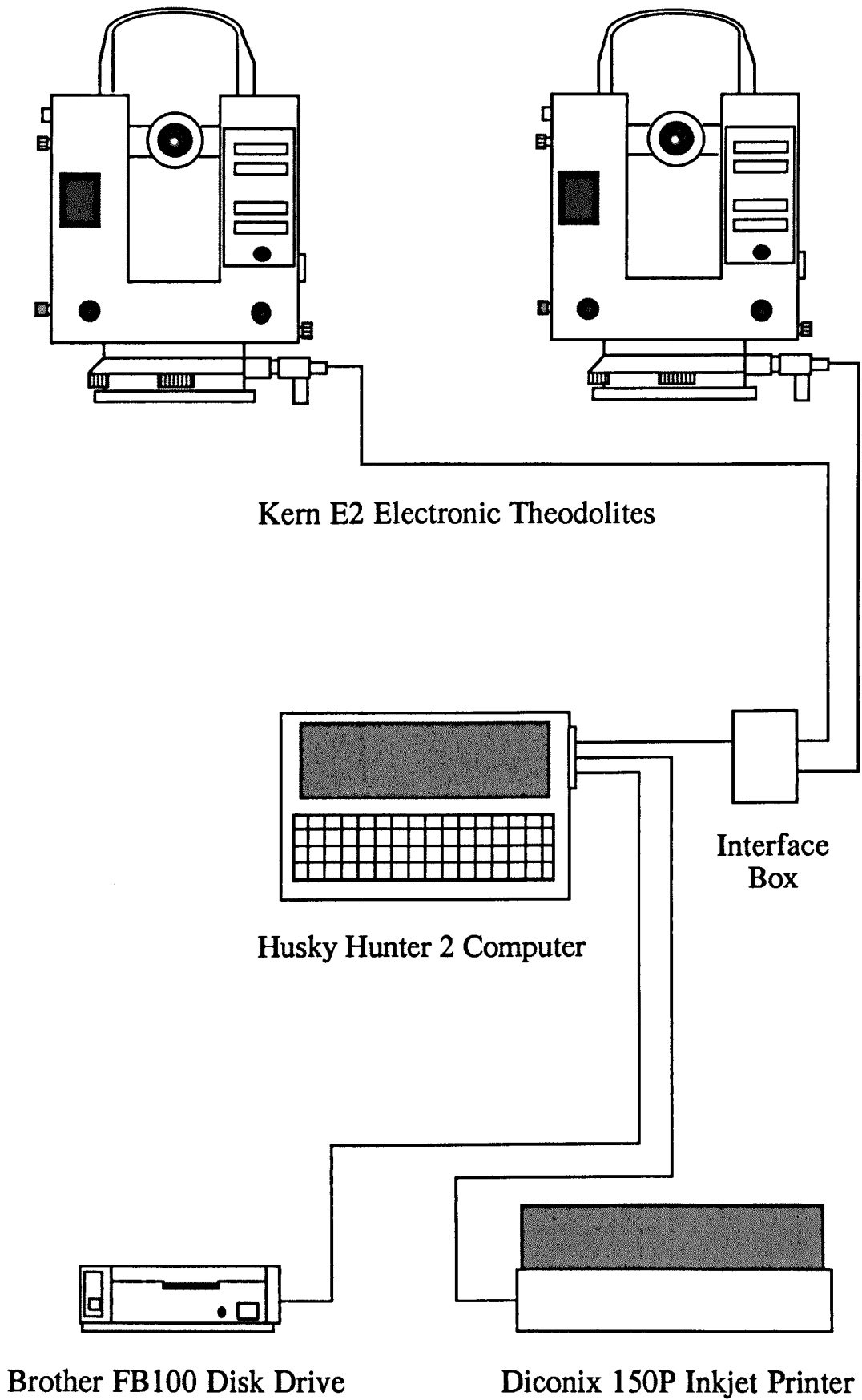
Figure 5.1 Schematic Diagram of the Field Components of UN ETIS.

132

electronic theodolites was performed, and details are given in [Ketteman 1985] and [Cooper 1988].

In 1986, there were two "one-second" electronic theodolites available which were considered suitable for UN ETIS, the; Kern E2 and Wild T2000. As part of a BEng thesis the author performed an evaluation of these two electronic theodolites in 1986 / 87, details of which are given in [Bingley 1987]. After consideration was given to; the past evaluations of ETSs performed by the Department, the experience gained with Kern electronic theodolites within the Department and the technical support provided by Kern in the UK, a decision was made, in 1987, to purchase two Kern E2 electronic theodolites (352393 and 352396).

The principles of electronic theodolites are covered in Chapter Three. This section outlines the specific operation of the Kern E2, and data communication in the Kern ASB system, details of which are given in [Kern 1984]. Also included in this section are the results of an evaluation of the two Kern E2s (352393 and 352396), performed on the Llangollen Test Network in April 1988.

5.2.1.1 Kern E2 Operation

The components of the Kern E2 with respect to Plates 5.1 and 5.2 are:

      a - Finder collimator.

      b - Focusing ring.

      c - Diopter ring.

d - Data recorder holder.

e - Function switch.

f - Data transfer cable.

g - Cable connector.

h - Power / data transfer cable socket.

i - Power cable.

j - Coarse-fine circle drive.

k - Support plate.

l - Levelling screws.

m - Horizontal circle clamp.

n - Compensator switch.

o - Horizontal circle "zero" button.

p - Internal switches for permanent adjustment of index error.

q - Internal switches for setting measuring units, compensator and ASB device number.

r - Four-fold display.

s - Reticule illumination control.

t - Two-fold display.

u - Horizontal circle coarse-fine adjustment control.

v - Optical plummet.

w - Vertical circle coarse-fine adjustment control.

x - Vertical circle clamp.

y - Data recorder holder.

z - Data transfer cable socket.

The compensator switch has the following settings:

Plate 5.1

Kern E2 Electronic Theodolite, Face Left.

Plate 5.2

Kern E2 Electronic Theodolite, Face Right.
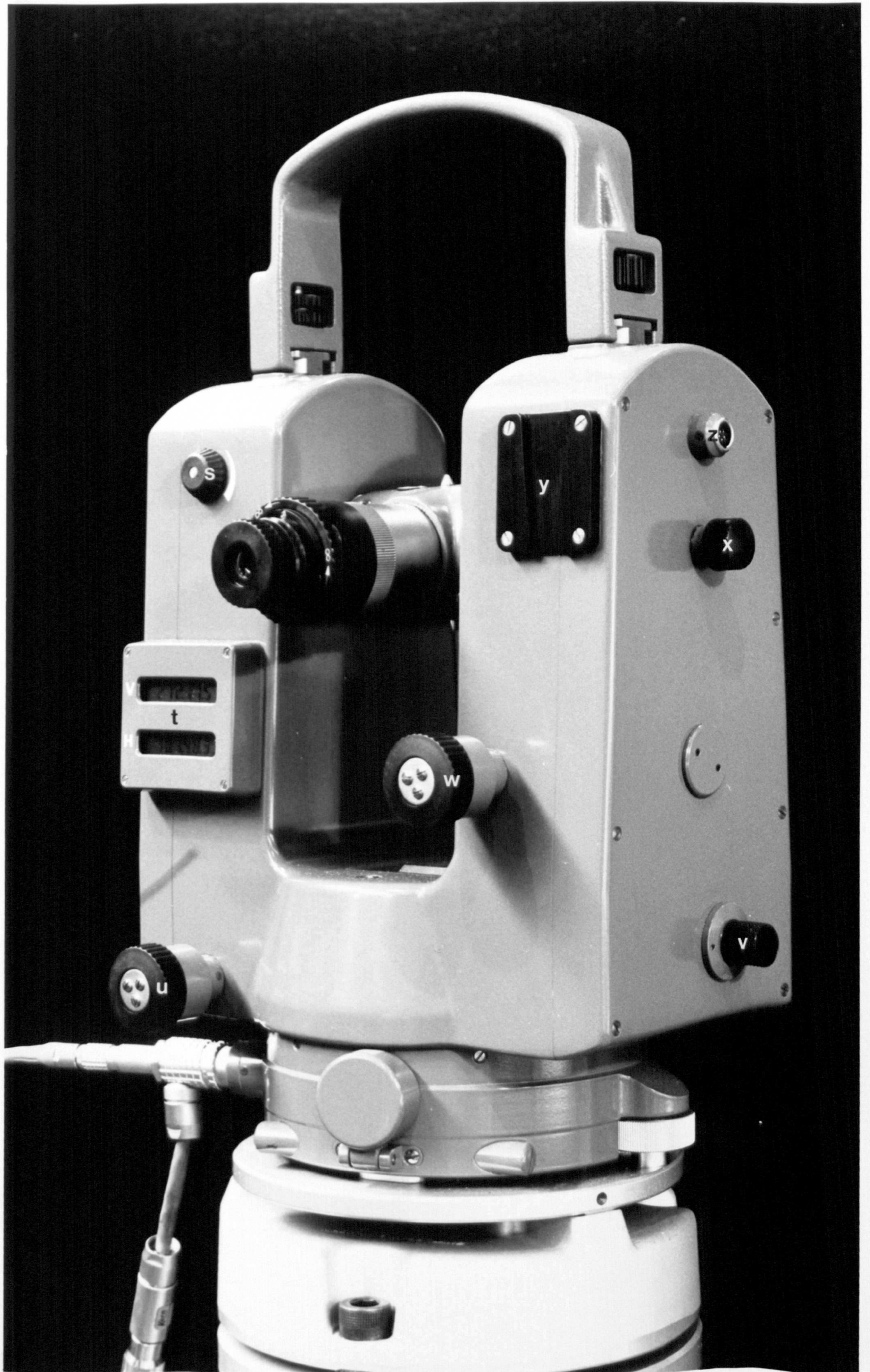
OFF   The compensator inclination components are displayed separate from the unadjusted circle readings.

ON   The displayed circle readings are adjusted by the compensator inclination components.

The function switch has the following settings:

OFF   The instrument is switched off.

ON   The instrument is switched on.

LAMP   The instrument is switched on and the displays and reticule are illuminated. The additional power consumption is negligible, and the reticule illumination can be adjusted to suit the operator and conditions.

INDEX   The instrument is switched on and illuminated. This setting permits determination of the index error and effects a temporary adjustment.

The first step in the operation of any surveying instrument is the setting up and levelling. Once the Kern E2 is located on a theodolite stand it has to be levelled, by a two levelling screw system. For precise work it is generally the case that the instrument is "visually" levelled, using the plate bubble, then "digitally" levelled, by means of the compensator. "Visual" levelling of the instrument by the plate bubble is illustrated in Figure 5.2. "Digital" levelling of the instrument is performed by switching the compensator switch to OFF. The inclination components are then shown in the top two windows of the four-fold display. To "digitally" level the instrument to a high
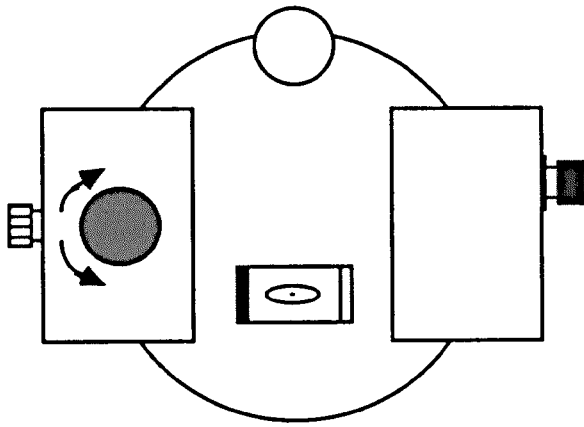
(i) Set the vertical axis clamp above one levelling screw and centre the alidade bubble using the first levelling screw.
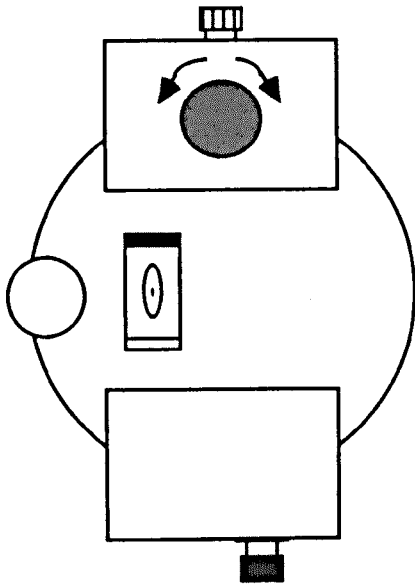
(ii) Rotate the alidade through 90 degrees and centre the alidade bubble using the second levelling screw.
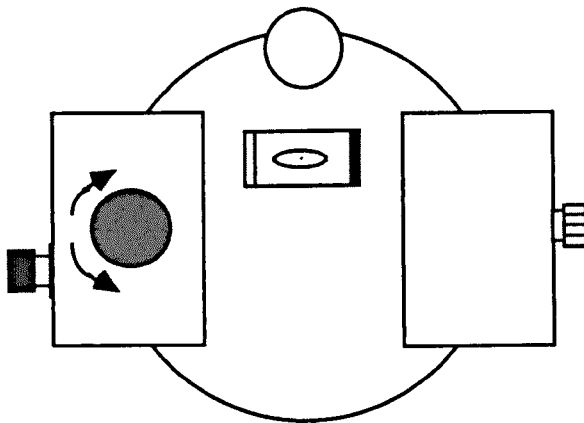
(iii) Rotate the alidade through a further 90 degrees and centre the alidade bubble using the first levelling screw.

Figure 5.2 Levelling the Kern E2 Electronic Theodolite.

138

accuracy, these components should be zeroed by adjustment of the levelling screws without moving the instrument.

Each of the Kern E2s is powered by an individual battery, which has a short cable permanently attached. This is joined to the instrument by a cable connector, which also permits the attachment of a data cable.

Once the instrument is levelled, then operation begins with an initialisation procedure. When the instrument is first switched on, consecutive numbers and segment patterns appear in all six displays, so as to check the displays and the individual segments. The characters appear in the display, at intervals of about one second, in the following sequence; 1, 2, 3, 4, 5, 6, 7, 8, 9, L, H, P, A, -, 0. This pattern of characters is continuously repeated until the vertical circle mark is read in, by turning the telescope through the horizon. At this point, the vertical circle reading is displayed in the two windows marked V, the top window of the four-fold display has the prompt H000, and a zero horizontal circle reading is displayed in the two windows marked H. To complete the initialisation procedure, the alidade has to be rotated to the diametrically opposite horizontal circle position, at which point the H180 prompt is displayed in the top window of the four-fold display. Then, after 5 to 10 seconds, the H180 prompt disappears and the full horizontal circle reading is displayed in the two windows marked H.

The vertical and horizontal circle readings are displayed to an accuracy of 0.1 mgon (1") and are updated every 0.33 seconds. To sight a target, the vertical and horizontal circle clamps are taken off and the telescope is pointed approximately to the target. Once in the field of

view of the telescope, the clamps are put on. Final accurate centring of the reticule on the target is then performed using the circle coarse-fine adjustment controls.

5.2.1.2 Data Communication in the Kern ASB System

The ASB is an interface-bus system developed by Kern. The ASB specifications are designed to enable Kern electronic instruments to be connected to computer hardware, via an RS 232 interface. It is based on half-duplex data transmission over a single line, ie data can be transmitted in either direction, and permits several devices to be connected at any one time.

As stated in Chapter Three, individual devices are labelled by a device type and ASB device number. In the case of an ETIS, a number of electronic theodolites can be connected to the computer. These are labelled by setting device numbers on switches inside the individual theodolites. So, for example, in a two theodolite set up, theodolite 1 is labelled as T1 and theodolite 2 as T2.

When considering an ETIS, the most important data requested are the horizontal and vertical circle readings. In the Kern ASB system these are given code numbers of 11 and 12, and have the format

11+HHH.HHHH
12+ZZZ.ZZZZ

Control characters are required to ensure a regulated data exchange when several devices are connected to a computer, and to separate data being transmitted. They are ASCII characters which are transmitted by the computer. The following characters are used

Character	Value	Effect
<STX>	2	Start transmission
<ETX>	3	End transmission
<CR>	13	Carriage return
<LF>	10	Line Feed

An example of a request, from the computer to the theodolite, is given below

<STX>T1112<CR><LF>

where

<STX>	= Start transmission.
T	= Type of device.
1	= ASB number.
11	= Code number
2	= Number of consecutive values requested.
<CR><LF>	= Termination of request.

The result of this request is that the theodolite (device type T), with ASB number 1, transmits horizontal and vertical circle readings (code numbers 11 and 12) to the computer, ie the theodolite replies with the circle readings, for example

11+180.0013 12+100.4025<CR><LF>

Data transfer is ended by the transmission of an <ETX> character from the computer. Similarly, the transmission of a horizontal circle reading, from the computer to the theodolite, is performed by a code number 72, as follows

<STX>T172+000.0000<CR><LF><ETX>

The result of this transmission is that the horizontal circle of the theodolite with ASB number 1 is set to 000.0000, by the computer.

5.2.1.3 Evaluation Results on Llangollen Test Network

The Llangollen Test Network designed to test various survey instruments and methods, was established more than fifteen years ago [Hodges & Scoble 1974]. Prior to 1988, the network consisted of seven main stations, as shown in Figure 5.3, encompassing a wide variety of topography. Inter-station lengths range from 743 to 2999 m, and differences in station levels range from 10 to 294 m.

The stations consist of BKS markers embedded in concrete bases which are augered, where possible, down to bedrock. The precise position of a station is indicated by finely etched crosslines on a circular brass plate affixed to a recess in the marker head. The stations are all below ground level, and are covered by steel caps when not in use. When the network was set up it was considered necessary to design special targets for angular measurements. The target, illustrated in
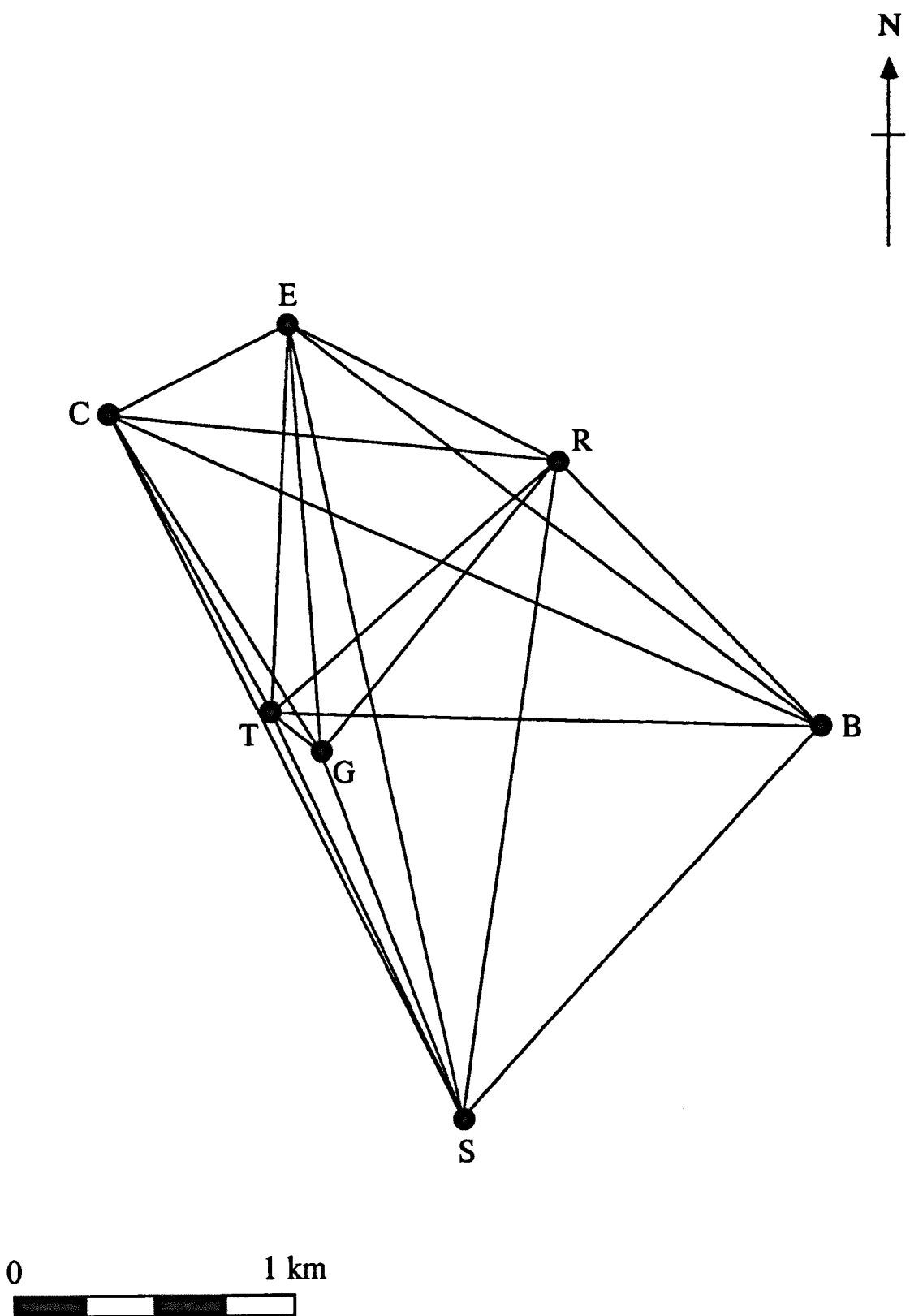
Figure 5.3 Llangollen Test Network.

Figure 5.4, consists of a heavy tripod which supports a ranging pole, through a special adaptor. To ensure accurate centring over the station, the ranging pole contacts the brass plate through a chimney which fits over the station head. The ranging pole, which is used to measure horizontal angles, is set vertical, to a high accuracy, by means of a spot bubble. Vertical angles are observed to a cross-piece attached to the ranging pole, exactly 1.5 m above the station, which can be rotated to face the observer. The width of the gap between the two arms of the cross-piece has been calculated to equal the thickness of a telescope cross hair, when observed over 1500 m, the average sight length on the network.

In April 1988, the two Kern E2 electronic theodolites, purchased as part of UN ETIS, were evaluated on the network. Prior to 1988, the vertical and horizontal angles of the network had been measured on at least ten separate occasions, since 1975. The horizontal angles had actually been measured on eleven occasions using a Kern DKM3 optical mechanical theodolite, and on five occasions with various electronic theodolites. The vertical angles had been measured on ten separate occasions with a variety of optical mechanical and electronic theodolites. It is generally considered that the standard errors for the horizontal and vertical angles are less than 1 second of arc. The accepted mean values (prior to 1988), and the deviations from these mean values, by the angles measured in 1988, are given in Tables 5.1 and 5.2. The measured angles are the mean of three complete double faced rounds, with horizontal and vertical angles being recorded simultaneously.
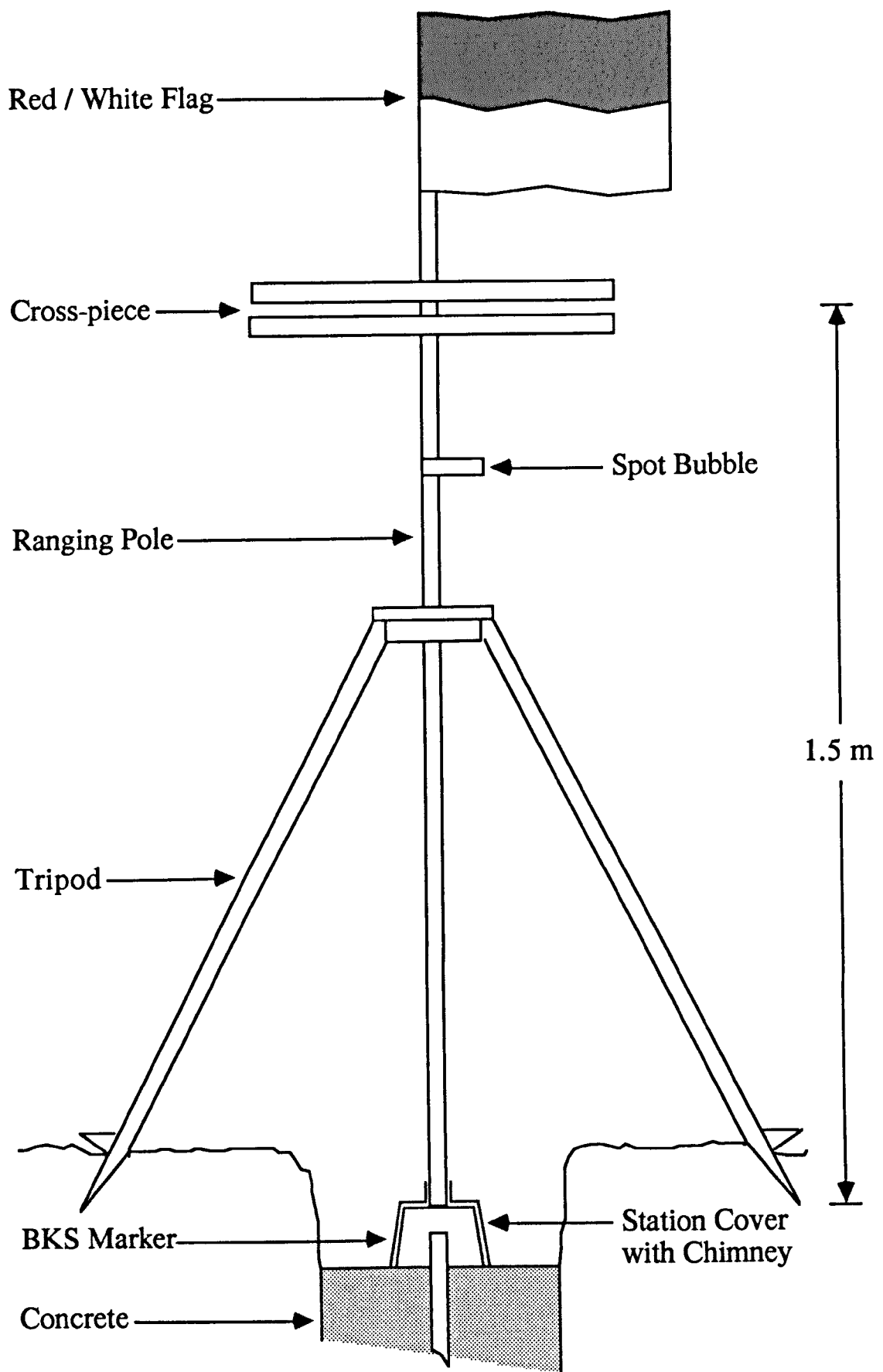
Figure 5.4 Llangollen Test Network Target and Station Design.

145

Horizontal Angle	Accepted Mean (Prior to 1988)	Kern E2 (352 393) Deviation (")	Kern E2 (352 396) Deviation (")
C T E	030 42 08.0	-0.3	0.3
E T R	045 12 26.2	0.5	-0.2
R T B	043 39 14.7	-0.7	-1.7
B T S	061 40 10.6	-1.6	-2.3
S T C	178 46 00.2	2.8	4.1
S B T	049 45 11.1	-0.4	2.6
T B C	022 20 06.3	-0.3	1.0
C B E	013 50 58.4	1.9	-1.7
E B R	008 28 55.3	0.0	1.2
R B S	265 34 49.2	-0.5	-3.4
B R S	050 59 53.6	-0.3	0.1
S R T	040 40 51.8	2.5	0.2
T R C	047 11 21.6	3.1	4.4
C R E	021 55 55.3	0.7	-2.0
E R B	199 11 58.7	0.3	0.0
E C R	033 45 00.7	0.6	-0.4
R C B	018 48 00.6	-1.6	-0.6
B C T	038 06 04.5	-1.5	-0.8
T C S	000 42 15.5	-0.2	-1.8
S C E	268 38 39.1	1.2	2.2
R E B	010 43 04.7	-2.4	-0.4
B E S	039 15 13.7	-1.4	0.6
S E T	015 42 00.6	4.1	3.4
T E C	058 38 44.4	2.6	2.6
C E R	235 40 57.0	-4.0	-4.3
C S T	000 31 43.9	2.8	0.4
T S E	013 46 07.1	-0.1	0.9
E S R	020 13 35.4	-2.4	-3.1
R S B	034 34 54.6	-1.3	-0.6
B S C	290 53 39.5	-2.2	-1.5

Table 5.1 Horizontal Angle Measurements on the Llangollen Test Network, 1988.

Vertical Angle	Accepted Mean (Prior to 1988)	Kern E2 (352 393) Deviation (")	Kern E2 (352 396) Deviation (")
TE+	10 11 55.9	0.6	0.9
TR+	06 48 10.5	1.2	1.8
TB+	00 16 42.4	1.9	0.6
TS+	10 06 03.4	0.9	-0.7
TC+	09 19 19.4	-0.1	0.6
BT-	00 17 43.1	-2.8	-1.4
BC+	03 53 25.9	1.8	2.1
BE+	05 49 30.6	2.9	3.0
BR+	06 31 00.8	2.0	1.5
BS+	08 29 00.1	1.1	2.2
RS+	02 55 34.6	4.1	4.1
RT-	06 48 46.1	-1.4	-1.8
RC+	01 12 08.4	0.9	1.6
RE+	04 46 46.8	0.4	2.0
RB-	06 31 39.5	-0.8	0.8
CR-	01 12 57.5	-1.5	-0.2
CB-	03 54 46.9	-1.6	0.4
CT-	09 19 51.8	-1.8	-0.5
CS+	01 47 05.1	1.9	0.9
CE+	04 26 49.6	1.7	0.7
EB-	05 50 40.7	-2.0	-0.7
ES+	00 37 04.0	2.3	2.0
ET-	10 12 32.0	-4.0	-1.3
EC-	04 27 11.6	-1.9	-0.6
ER-	04 47 18.9	-0.4	-0.7
ST-	10 06 45.8	-1.5	-1.1
SE-	00 38 19.9	-0.9	-0.9
SR-	02 56 37.0	-2.7	-1.7
SB-	08 29 51.7	-1.7	0.3
SC-	01 48 21.9	-2.6	-0.9

Table 5.2 Vertical Angle Measurements on the Llangollen Test Network, 1988.

147

From Tables 5.1 and 5.2, for the two Kern E2s evaluated, the mean horizontal angle deviations are ±1.5" and ±1.6", and the mean vertical angle deviations are ±1.7" and ±1.3". Investigations by [Schofield 1980], suggest that the expected deviation for a one second theodolite, on the network, is ±2.23". Thus, in a practical environment the two Kern E2s proved to be performing well within the expected accuracy.

5.2.2 Computer Hardware Configuration

The next stage in developing UN ETIS was the selection of the necessary computer hardware for the field. As stated in Chapter Three, there are two possible cases of computer hardware configuration; mains powered and battery powered. In order for UN ETIS to be flexible for use in a wide variety of measurement tasks, it was decided to develop the system around a battery powered hardware configuration.

The first item to be selected was the computer, and a decision had to be made between a field or lap-top computer. Lap-top computers were immediately ruled out, because of their size, weight and susceptibility to damage from unstable environmental conditions, such as extreme cold, dust and rain. From the field computers available in 1988, the Husky Hunter 2 was selected due to its' successful application, both in conventional surveying [Fort 1987] and as part of the Optimal DCS-200 system.

In selecting the peripheral computer hardware, a floppy disk drive and printer were required that could be powered by either

internal, re-chargeable, batteries or mains power. The selection was based on the hardware being used in the Optimal DCS-200 system; a Brother FB100 floppy disk drive and Diconix 150P Inkjet printer being selected. The individual items of computer hardware for the field are now detailed.

5.2.2.1 Husky Hunter 2 Computer

The original Husky Hunter was released in 1983, and was followed by the Husky Hunter 2, which was released in 1987. The Husky Hunter 2 is a self-contained field computer, which has desk-top computer power, but in a compact and portable form, see Plate 5.3. It is believed to be one of the smallest general purpose computers in the world, as well as probably being the toughest field computer on the market. The main factors which were considered in the selection of the Husky Hunter 2 were:

(i) Portability. It weighs 1.15 Kg and has dimensions of 216 mm x 156 mm x 32 mm.

(ii) Environmental Robustness. It is ideal for outdoor use as it is resistant to moisture, dust, vibration, shock and electromagnetic interference.

(iii) Programming Capabilities. It has a CP/M operating system, extended Basic interpreter, full qwerty keyboard and a flexible text editor.

(iv) Integral Screen. It has an 8 lines x 40 columns backlit LCD display, which acts as a window to a virtual screen of 24 lines x 80 columns.

Plate 5.3

Husky Hunter 2 Computer.

(v) Memory. The operating system is stored in ROM, leaving the full 496 Kbytes of RAM for user programs and data storage. In addition, user programs and data are stored indefinitely, as the memory is battery supported

(vi) Communications. It has an RS 232 serial port and redefinable communications parameters (the communications parameters for UN ETIS are given in Table 5.3).

The Husky Hunter 2 is powered by four "AA" size cells. These can be either disposable Alkaline-Manganese, which have an operational life of approximately 45 hours, or re-chargeable Nickel-Cadmium cells, which have an operational life of approximately 14 hours. However, it should be noted that a typical applications program will not require continuous power and a further feature of the Husky Hunter 2 is its' ability to conserve battery life by automatically switching off after a period of inactivity, typically five minutes. There are also two safety precautions provided by the Husky Hunter 2. Firstly, the operator is provided with a warning of impending battery failure, and, secondly, programs and data are not corrupted on battery failure, as the Husky Hunter 2 has an additional re-chargeable battery permanently built in.

On switching on, the operator is placed in the Husky Hunter 2 File Manager, DEMOS (Disk EMulation Operating System). This controls the execution of user programs, and enables access to system files and commands by the function keys. Initially, there are eight function keys which can be selected:

1 DIR   - display a directory list of file names.

151

Device Type	Kern E2	Brother FB100 Disk Drive	Diconix 150P Inkjet Printer	Downloading to HP 9816	Downloading to HP 319
Transmission Parameters					
Baud Rate	1200	4800	1200	1200	1200
Protocol	none	none	Xon / Xoff	none	none
Parity	even	8bit	even	even	even
CTS	y	n	n	n	n
DTR	y	y	n	y	y
LF	n	n	y	n	n
Echo	n	n	n	n	n
T / O	10	10	50	10	10
Null	0	0	0	0	0
Receiving Parameters					
Baud Rate	1200	4800	1200	1200	1200
Protocol	none	none	Xon / Xoff	ACK / NAK	none
Parity	even	8bit	even	even	even
RTS	true	true	off	off	off
DSR	n	y	n	n	n
DCD	y	n	n	y	y
T / O	10	10	50	10	10
Serig	off	off	off	off	off

Table 5.3 Husky Hunter 2 Communications Parameters for UN ETIS.

2 STAT - display the status of a file ie the size of the file, the read / write access to the file and the remaining file space available.

3 BAS - access the Basic interpreter.

4 COMS - initialise the communications parameters.

5 CLCK - initialise the internal clock.

6 TERM - enable terminal emulation.

7 EDIT - activate the text editor.

8 KEYS - alternate the function keys.

On selecting KEYS, there are a further eight function keys presented. The most important of these are:

6 REN - rename a file.

7 SEND - send data via the RS 232 serial port.

Files are named in accordance with the general format of CP/M. They consist of up to eight alphanumeric characters separated by a period from an extension of three alphanumeric characters, which informs DEMOS of the type of file. There are five extensions used by DEMOS, namely:

.HBA - Hunter Basic file containing program source code.

.COM - CP/M compatible object code file.

.DTA - Hunter Basic data file.

.TXT - ASCII text file.

.SYS - A "pseudo file", generally in ROM.

Files are stored, as on magnetic disk, in records of 128 bytes, grouped by 16s into 2 Kbytes. This means that the smallest file size is 2 Kbytes, but, on the other hand, very large files can also be stored. Since the Basic interpreter resides in ROM, there are 54 Kbytes of RAM available for a Basic source program, with associated variables and arrays. However, it is advisable not to exceed a maximum size of 36 Kbytes, so as to enable the file to be loaded into the Basic interpreter for editing. On entering the Basic interpreter the operator is presented with a further eight function keys, namely:

1 FILE - display a directory list of file names.

2 RUN   - execute a program.

3 EDIT - edit a program.

4 LIST - list a program on the screen.

5 SAVE - write a program into the file space.

6 LOAD - load a program into the Basic work space.

7 KILL - delete a file in RAM storage.

8 SYST - return to DEMOS file manager.

Functions 2 to 7 are performed by specifying a file name after selecting the function key. In addition, functions 2 to 5 can also be performed on the file currently loaded into the Basic work space, by not specifying a file name. For a full description of the Husky Hunter 2, including details of the file manager, the Basic interpreter and its' communications abilities, the reader is referred to [Husky 1984].

5.2.2.2 Brother FB100 Disk Drive

The Brother FB100 disk drive is a 3.5 inch storage device used for transferring files to, and from, the Husky Hunter 2 computer. It is lightweight (0.85 Kg including batteries), portable (with dimensions of 166 mm x 133 mm x 55 mm) and can operate in a wide range of environmental conditions (temperature range of 5 - 40 °C and humidity range of 20 - 80 %).

The disk drive is connected to the Husky Hunter 2 via the RS 232 interface, and can be accessed from DEMOS or Hunter Basic software. At the present time, the disk drive is accessed from DEMOS by the Husky Disk Operating System (HDOS), details of which are given in [Husky 1987]. This is compiled Basic software, which is executed from DEMOS by typing *HDOS.COM*. The operator is then presented with a menu of the following options:

1 Disk directory	- list the files on the disk.
2 File load	- copy a file from the disk to the computer.
3 File dump	- copy a file from the computer to the disk.
4 Erase disk file	- erase a file from the disk.
5 Rename disk file	- rename a disk file.
6 Disk format	- format a disk.
7 Exit to DEMOS	- return to the file manager.

The purchasing of this software, with the disk drive, removed the need to develop in-house software to access the disk drive

directly from a Hunter Basic program. HDOS performs all of the functions required by UN ETIS, and permits the operator to permanently store data files on the disk drive, at any stage of a measurement survey. In addition, once a measurement survey is complete, a final copy of the data files can be stored on disk, allowing them to be removed from the Husky Hunter 2, thus increasing the available memory for future measurement survey data. Also, at any time in the future, a measurement survey can be loaded from the disk drive, to the Husky Hunter 2, if required.

5.2.2.3 Diconix 150P Inkjet Printer

The Diconix 150P Inkjet printer is a thermal inkjet printer, with a print width of 177 mm, and variable print quality. It is lightweight (1.7 Kg including batteries), portable (with dimensions of 270 mm x 163 mm x 50 mm) and can operate in a wide range of environmental conditions (temperature range of 10 - 40 ^{o}C and humidity range of 10 - 90 %). A full specification of the printer is given in [Diconix 1987].

The printer is connected to the Husky Hunter 2 via the RS 232 interface, and can be accessed from DEMOS, or Hunter Basic software. At the present time, the only hard copy output which can be obtained in the field is a printout of a measurement survey data file. This is performed from DEMOS, by editing the *.DTA* file (Husky Basic data file) and saving it as a *.TXT* file (ASCII text file). This ASCII text file can then be transferred, via the RS 232 interface, to the printer using the SEND command. In-house Hunter Basic software could be developed to provide formatted hard copy output in the field,

but this is not considered necessary. Instead, formatted hard copy output is obtained in the office, using in-house developed software, which is detailed in section 5.3.2.3.

5.2.3 Field Software

As stated previously, the field software for UN ETIS has been developed in-house, to meet the specifications detailed in Chapter One. In addition, the field software permits the transfer of measurement survey data from the Husky Hunter 2 field computer to an office computer for processing, archival storage, and analysis.

The field software for UN ETIS has been written in Hunter Basic. The main program is called *ETISMAIN.HBA*, and from this a further four program modules are called, namely:

TRNSFORM.HBA - transformation program module.

MANIP_PT.HBA - point manipulation program module.

CRD_COMP.HBA - coordinate comparison program module.

RES>DOWN.HBA - program module to convert an Hunter Basic data file to an ASCII text file for downloading to the office computer.

Some modularisation had to be performed, to maintain the size of the main program within the limit of 36 Kbytes set for a single Hunter Basic program. This modularisation is invisible to the operator, who executes the software from DEMOS by entering *ETISMAIN.HBA*,

from which point on, execution continues as if in a single program. The only disadvantage occurs when quitting a module and returning to the main program. This can take approximately twenty seconds, as opposed to the instantaneous response which would be effected if in a single program.

In addition to the main program, and the four program modules, there is a Husky Basic data file *(INITFILE.DTA)*, which is essential to the correct operation of the software. This enables the operator to quit and re-execute the main program, without the need to re-define certain parameters. This is essential in two cases, as it enables both the execution of program modules from the main program, and the operation of the disk and printer from DEMOS at any stage of a measurement survey. In both cases, on re-executing the main program, the software sets the values for the initial parameters, and the control point set up parameters, to those stored in *INITFILE.DTA*. The initial parameters include; measurement survey name and number, the coordinate system, the mode of electronic theodolite input and the accuracy to which numeric data is displayed. The control point set up parameters include; control point names, instrument heights and control point coordinates.

Measurement survey data is stored in Hunter Basic data files enabling read / write access by the software. Data from an individual survey is stored in three types of data file:

(i) Coordinate file, containing the final accepted coordinate data for each point ie the point name and the least squares adjusted coordinates of the point.

158

(ii) Results file, containing data for each point from each stage of the coordinate calculation ie the point name, the names of the two control points from which the point was measured, the target height at the point, the instrument heights at the two control points, the angles measured from the control points to the point and the deviations of these angles, the raw coordinates of the point and the mis-intersections in the raw coordinates, the least squares adjusted coordinates of the point, and the ellipsoid of error and probable error at the point.

(iii) Transformation file, which contains the seven transformation parameters derived during a measurement survey.

Data files for a specific measurement survey are identified by a file name, which consists of four alphanumeric characters, a survey number, which consists of two numeric characters, and a file type character (_C, _R or _T). If a series of surveys are being performed, then it is logical that they should have the same file name; each individual survey is then referred to by a survey number. A data file name is therefore formed from:

<file name> <survey number> <file type character> .DTA

For example, consider a measurement survey which, for simplicity, has a file name of *EXAM* and is survey number one *(01)* in a series of surveys. In this case the software will create the following Hunter Basic data files:

(i)   coordinate file - *EXAM01_C.DTA.*

(ii) results file - *EXAM01_R.DTA.*

(iii) transformation file - *EXAM01_T.DTA.*

159

Furthermore, if a second survey *(02)* is performed in this series, the software will create the following Hunter Basic data files:

(i) coordinate file - *EXAM02_C.DTA.*

(ii) results file - *EXAM02_R.DTA.*

(iii) transformation file - *EXAM02_T.DTA.*

The UN ETIS field software is menu based, the structure of the software is illustrated in Figure 5.5. This reduces the required keyboard input and doesn't restrict operation through a set series of commands, ie the operator is free to select any menu option at any stage of a measurement survey. On execution of the software the operator is presented with a main menu from which further menus are selected. The main menu has the following options:

F - FILE OPTIONS

I - INIT PARAMETERS

M - COORD MEASUREMENT

S - SET UP CRD SYSTEM

The initial parameters menu, obtained by selecting option I on the main menu, permits the operator to define the initial parameters stored in *INITFILE.DTA*. These parameters can be re-defined at any stage of a measurement survey, but they are usually defined at the start of a new measurement survey. On selecting main menu option I, the operator is presented with a menu of five options:

A - DISP ACC

INITIAL PARAMETERS

A - DISP ACC - 6
C - CRD SYSTEM - :lo
F - FILE NAME - exam
I - INPUT - AUT
S - SURVEY NO - 01

Option (Q - MAIN MENU)

(i)   Calculation of mean angles & deviations.
(ii) Calculation of raw coordinates &
      mis-intersections.
(iii) Least squares adjustment, calculation of the
      ellipsoid of error and probable error.
(iv) Transformation of coordinates (if required).

SYSTEM SET UP OPTIONS MENU

C - COORDINATES
D - DIST / (AZ)
R - RESECTION
S - SCALE BAR
T - TRANSFORM

Option (Q - MAIN MENU)

MAIN OPTIONS MENU

F - FILE OPTIONS
I - INIT PARAMETERS
M - COORD MEASUREMENT
S - SET UP CRD SYSTEM

Option (Q - QUITS)

MEASUREMENT OPTIONS MENU

C - CALCULATION
E - EDIT / VIEW
I - INPUT - AUT
O - OBSERVATION

Option (Q - MAIN MENU)

STORE POINT DATA TO
HUNTER BASIC DATA FILES

FILE OPTIONS MENU

C - COORD COMPARISON
L - LIST RESULT DATA
P - POINT MANIPULATN
R - RESULTS>DOWNLOAD

Option (Q - MAIN MENU)

COORDINATE MEASUREMENT

N - ENTER NEW STATION SET UP
P - POINT FROM SAME STATIONS

Option (Q - MAIN MENU)

Figure 5.5 UN ETIS Field Software Structure.

161

This option permits the operator to select the accuracy (3, 4, 5 or 6 decimal places) to which numeric data is displayed. This accuracy also sets the iteration limits for the least squares adjustment of coordinates.

C - CRD SYSTEM

The software permits measurement in three coordinate systems, namely; local, object and National Grid. The coordinate system chosen causes point names to be appended with a three character flag (:lo, :ob or :ng), to define in which coordinate system the point was measured. For example, a point measured in a local coordinate system which, for simplicity, the operator calls *ptl*, will be stored in the data files with a full point name of *ptl:lo*.

F - FILE NAME

This option enables the operator to define a name, consisting of four alphanumeric characters, for the series of measurement surveys being performed.

I - INPUT

This option permits the operator to define the mode (automatic or manual) of input from the electronic theodolites to the Husky Hunter 2 computer.

S - SURVEY NUMBER

This option enables the operator to define a survey number, consisting of two numeric characters, for the individual measurement survey being performed.

Once the initial parameters have been set, the operator quits from the initial parameters menu and returns to the main menu. The

next stage, in a new measurement survey, is the setting up of a coordinate system. The software permits the setting up of a coordinate system by all of the methods detailed in Chapter Two, excluding the bundle adjustment. Setting up is performed from the system set up options menu, obtained by selecting main menu option S, which contains four setting up options, and a transformation option:

C - COORDINATES

This option enables the operator to enter the known coordinates of any number of points. In addition, point coordinates can also be retrieved from memory, for inspection.

D - DIST / (AZ)

This option enables setting up of a coordinate system by accurate distance measurement between control points.

The software permits the entry of a baseline azimuth, if required, and as part of the setting up procedure, the software prompts the operator to perform the collineation procedure.

R - RESECTION

This option enables setting up of a coordinate system by resection from three fixed points of known coordinates.

S - SCALE BAR

This option enables setting up of a coordinate system by the use of a calibrated scale bar.

At the start of this setting up procedure, the software prompts the operator to perform the collineation

163

procedure. The software permits the scaling up to be performed from a combination of up to four, independent, scale bar positions.

T - TRANSFORM

This option does not permit the direct setting up of a coordinate system, and thus is not selected at the start of a new measurement survey. If a transformation of coordinate systems is required, at some stage of a measurement survey, then the operator returns to the system set up options menu and selects this option.

The software permits the transformation of coordinate systems in three cases; local to National Grid, local to object (full three-dimensional) and local to object (with a fixed zenith). In each case, it requires the local and known, object or National Grid, coordinates of four transformation points to be present in the measurement survey data files. From the coordinates of these four transformation points, it calculates the transformation parameters, which are then stored in a transformation file, and used to transform the coordinates of all subsequent points measured in the present survey.

Once a coordinate system has been set up and the coordinates of the control points have been determined, then the operator quits from the system set up options menu, and returns to the main menu. Measurement to points of unknown coordinates is performed from the coordinate measurement menu, obtained by selecting main menu option M, which has two options:

N - NEW STATION SET UP

In general, this option is selected after a coordinate system has been set up, and before coordinate measurement to the first unknown point. Firstly, the operator has to enter the new control point names, and their instrument heights. The software obtains the control point coordinates from the coordinate file, and these control point set up parameters are stored in *INITFILE.DTA*. Then, the operator enters the target point name and target height. At this stage the software prompts the operator to perform the collineation procedure, prior to coordinate measurement.

The only exceptions to this are, if a coordinate system has been set up by accurate distance measurement between control points, or by the use of a calibrated scale bar. In these cases, collineation has already been performed as part of the setting up procedure and coordinate measurement should be performed by selecting option P.

P - POINT FROM SAME STATIONS

This option is selected after the first point has been measured, or if collineation is not required, and is used to initiate coordinate measurement to points measured from the same control point set up.

It is important to note that the control point set up parameters are stored in *INITFILE.DTA*, and remain fixed until they are changed by, either, option N, or the setting up of another coordinate system. Thus, as

a number of points are usually measured from the same control point set up, coordinate measurement is initiated by selecting option P, and entering both the target point name and target height.

Once the target point has been named, the operator controls coordinate measurement to that point from the measurement options menu, which has four options:

C - CALCULATION

This option initiates the calculation procedure, once the required number of observations have been performed to the unknown point.

E - EDIT / VIEW

This option permits the operator to view the blocks of observations, which have been performed to the unknown point. The operator can also delete a block, if required.

I - INPUT

This option permits the operator to define the mode (automatic or manual) of input from the electronic theodolites to the Husky Hunter 2 computer.

O - OBSERVATION

If the input mode is automatic, then this option initiates the transfer of circle readings from the electronic theodolites to the Husky Hunter 2 computer. The circle readings are displayed on the screen once transfer is complete.

If the input mode is manual, then the software prompts the operator to enter the circle readings from the electronic theodolites via the keyboard of the Husky Hunter 2 computer.

A maximum of six observation blocks can be included in the calculation procedure. Once the operator is satisfied with the observations then calculation is initiated, by selecting option C. There are three stages to the calculation. At each stage the operator is provided with some indication of the accuracy of coordinate determination, and has two options; A - to accept that stage of the calculations and initiate the next stage or, E - to return to the measurement options menu to re-edit the observations, and / or perform further observations. The three stages of the calculation are:

(i) Calculation of mean angles and deviations. If six blocks of observations have been taken, then this stage of the calculation takes approximately 20 seconds. At this stage it is easy to detect a mis-reading, as the deviation of the angle which has been mis-read will be much greater than a few seconds of arc.

(ii) Calculation of raw coordinates and mis-intersections. This stage of the calculation takes approximately 35 seconds. At this stage it is easy to detect a sighting error which may not have been detected at the previous stage, eg if the two theodolites are sighting to different target points. In addition, the mis-intersection in the z coordinate provides an early indication of the magnitude of the ellipsoid of error. These mis-intersection values are useful, as they may convince the operator of the need to re-edit the observations, prior to the time consuming part of the calculation, the least squares adjustment.

167

(iii) Least squares adjustment, calculation of the ellipsoid of error and probable error. This stage of the calculation takes approximately 30 seconds, plus an additional 5 seconds for each iteration required. At this final stage in the calculation, the accuracy of coordinate determination is obtained by comparing the actual ellipsoid of error with the probable error. The final decision to either accept, or re-edit the calculated data for a point, is made by the operator, depending on some accepted order of magnitude for the ratio of these two errors.

If measurements are being performed in a local coordinate system or the National Grid, then once the least squares adjusted coordinates have been accepted, the measured data is stored in the coordinate and results data files. If measurements are being performed in an object coordinate system, the local coordinate data is stored in the coordinate and results data files and a transformation is then performed, before the object coordinate data is stored in the coordinate and results data files. When storage of the measured data is complete, the operator is returned to the coordinate measurement menu, from where another point can be measured, if required.

The other menu available to the operator is the file options menu, which is obtained on selecting option F in the main menu. The operator is then presented with a further menu, of four options:

C - COORD COMPARISON

This option permits the comparison of coordinates. This includes; a file point comparison in which the three-dimensional coordinates of a point are

168

compared in successive measurement surveys, and a two point comparison in which the three-dimensional coordinates of two independent points are compared.

L - LIST RESULT DATA

This option lists the results data of a point, in the present measurement survey.

P - POINT MANIPULATN

This option enables the manipulation of point data within the Hunter Basic data files. This manipulation includes; copying a point from the present survey to another selected survey, deleting a point from the present survey, making a new point in the present survey, by combining up to six other points, and renaming a point in the present survey.

R - RESULTS>DOWNLOAD

This option enables the downloading of measurement survey data from the Husky Hunter 2 field computer to an office computer.

This is usually selected at the end of a measurement survey. The present results file is then converted to an ASCII text file and the communications parameters required for downloading are set by the software.

5.3 Office Components of UN ETIS

The office components of UN ETIS consist of a mains powered computer, with peripheral hardware, and in-house developed office software. The specifications for these components require that they are capable of; accepting downloaded measurement survey data

from the Husky Hunter 2 field computer, performing archival data storage, on hard disk and floppy disk, processing and analysing the measurement survey data, and providing output in a suitable format.

At present, the software is operational on two separate computer hardware configurations, the specifications of which are given. This section then details the office software developed by the author, with particular attention being paid to the analysis function, and the software developed for deformation monitoring applications.

5.3.1 Computer Hardware Configuration

In the early stages of the development of UN ETIS, in 1987, the Department already had an office computer hardware configuration capable of fulfilling the requirements of the system. The Hewlett Packard (HP) 9816 computer, and peripheral hardware, was purchased in 1983 for research, and development of the Hasp automated survey system. A great deal of research within the Department, from 1983 to 1988, has centred around the application of this system to mine surveying. However, recent research in the field of computer aided visibility studies prompted the Department to purchase the HP 319 computer, and peripheral hardware, in 1989. The availability of both computer hardware configurations, therefore, prompted the development of the office software on both computers.

A schematic diagram of the HP 9816 office components of UN ETIS is shown in Figure 5.6. The HP 9816 computer was supplied with 1.25 Mbytes of internal memory, of which 0.25 Mbytes is occupied by the operating system. External memory is provided by a 15 Mbyte

Husky Hunter 2 Computer

**Winchester 15 Mbyte
Hard Disk Drive**

Epson MX-100 Printer

HP 9816 Computer

Cotron Colour Monitor

HP 7580A Plotter
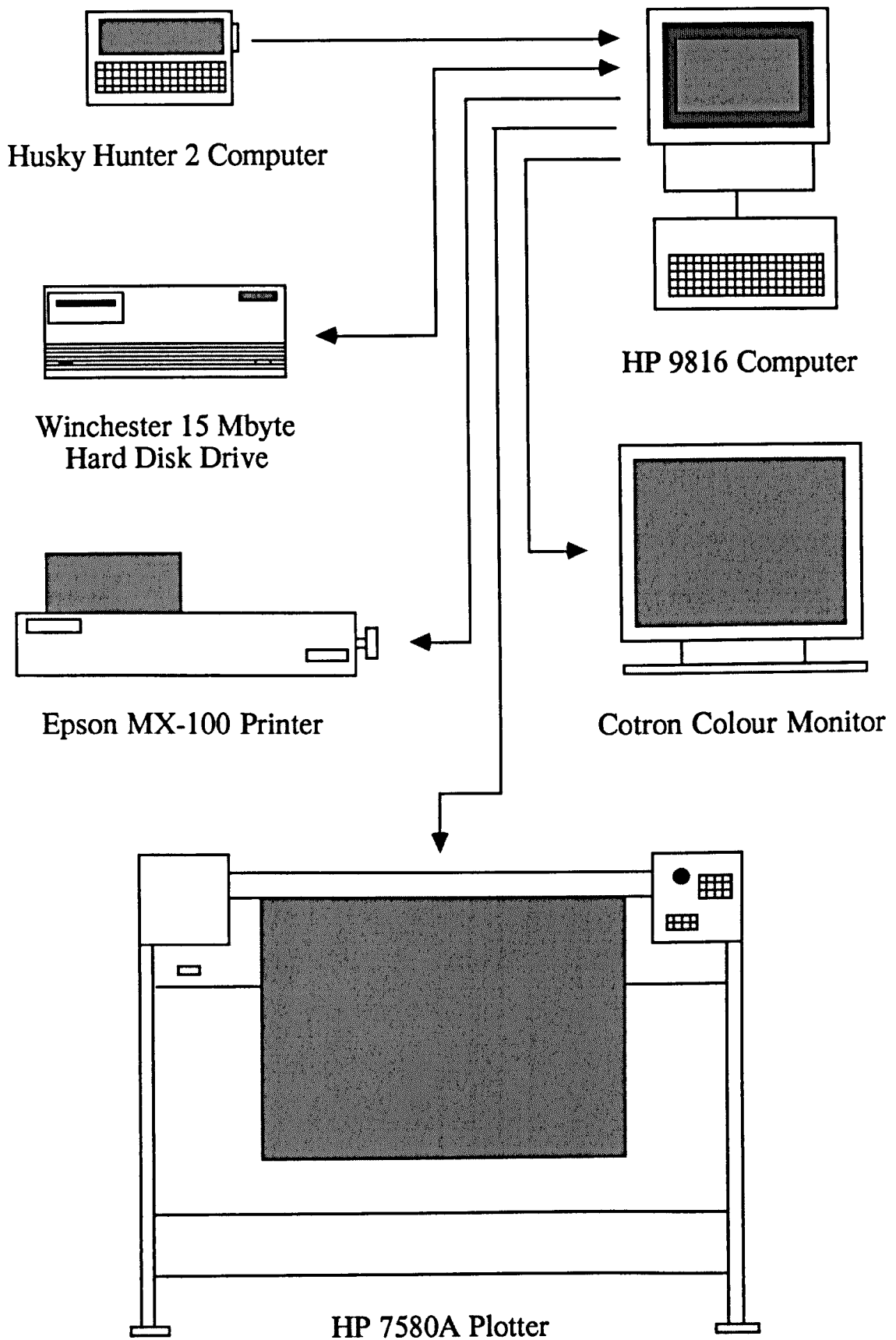
Figure 5.6 Schematic Diagram of the HP 9816 Office Components of
UN ETIS.

Winchester hard disk drive, with an internal 3.5 inch floppy disk drive. Output of data is in two forms; textual and graphical. Textual data output is generated on an integral screen and hard copy output can be obtained, during program execution, on an Epson MX-100 line printer. Graphical output is generated on an eight colour Cotron 14 inch RGB graphics screen, with a resolution of 512 x 390 pixels. Hard copy graphical output can be produced, via the Hasp data processing package, on a HP 7850A, A1 sized, grit wheel plotter, which has an eight pen carousel and a plotting speed of 60 cm/sec, with a positional accuracy in excess of 0.025 mm.

A schematic diagram of the HP 319 office components of UN ETIS is shown in Figure 5.7. The HP 319 computer was supplied with 4 Mbytes of internal memory. External memory is provided by two Winchester hard disk drives, of 81 and 304 Mbytes respectively. There is also a dual 3.5 inch floppy disk drive, and a 9 track tape drive, which handles tapes capable of storing up to 65 Mbytes. The system runs under the Unix operating system, which provides a multi-tasking environment, through the use of a series of text and graphics windows, displayed on a 1024 x 768 pixel resolution raster display. Both textual and graphical information can be output on a HP Paintjet colour inkjet printer, capable of displaying 330 colours at a resolution of 180 dots per inch. Hard copy textual data output can be obtained during program execution, and hard copy graphical output is obtained as raster images up to A4 size.

With both of the computer hardware configurations, the computers have in-built RS 232 and HPIB (Hewlett Packard Interface Bus) interfaces. The HPIB interface is used to connect the peripheral

Husky Hunter 2 Computer

HP 319 Computer

Winchester 81 & 304
Mbyte Hard Disk Drives

HP 9121 Twin
Floppy Disk Drive

1024 x 768
Colour Monitor

HP 9144 Cartridge
Tape Drive

HP 3630A Paintjet
Colour Inkjet Printer
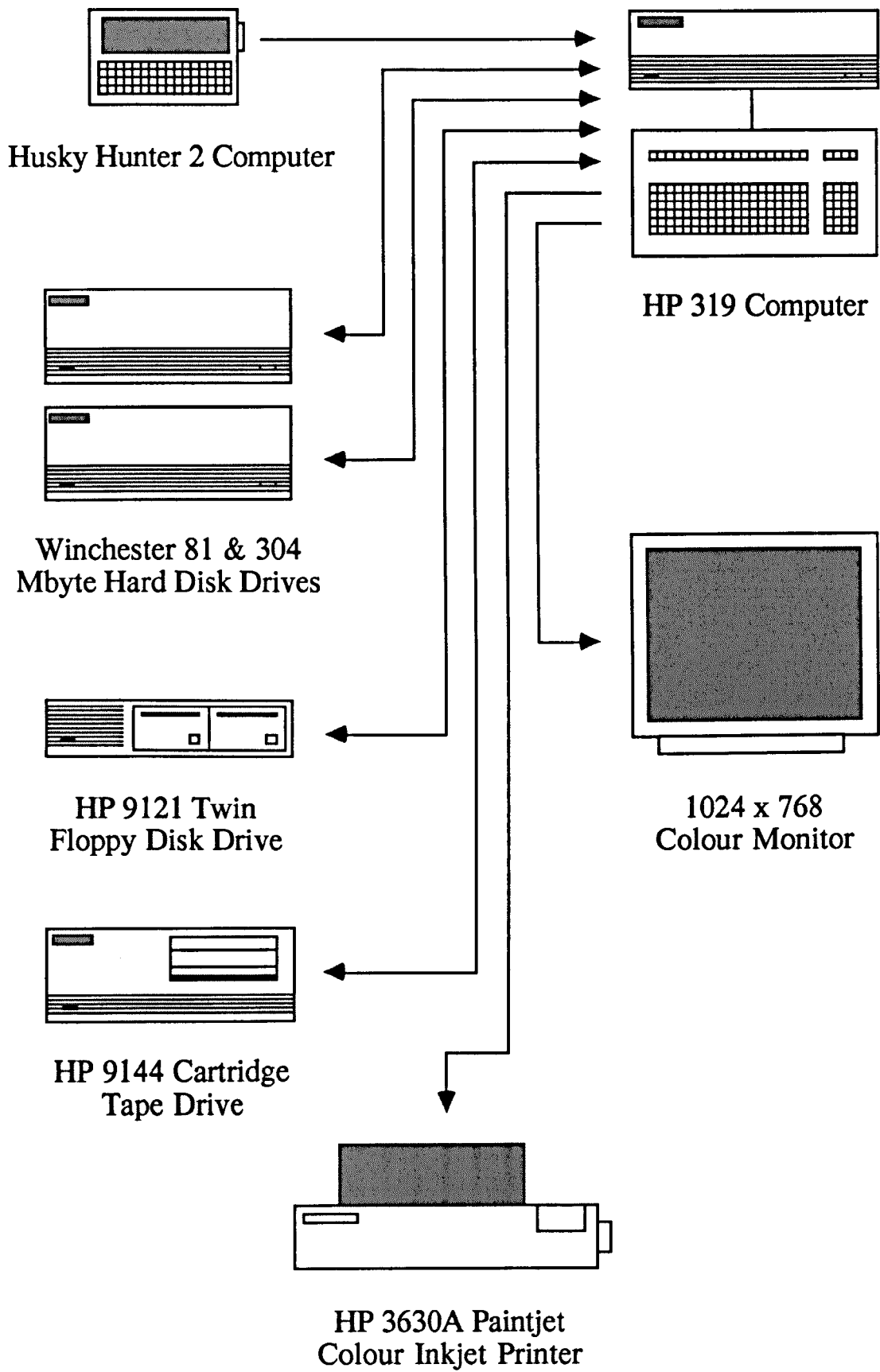
Figure 5.7 Schematic Diagram of the HP 319 Office Components of
UN ETIS.

173

hardware, and the RS 232 interface is used to connect the Husky Hunter 2 field computer for downloading.

5.3.2 Office Software

The office software was developed in-house in the Pascal programming language. In developing the software the main differences occurred due to the HP 9816 being a Pascal Workstation, and the HP 319 running under the Unix operating system. However, the development of the software ensured the functionality remained the same in both hardware configurations. In both cases, the office software performs three functions:

(i) Read - enables downloading of measurement survey data from the Husky Hunter 2 field computer to the office computer.

(ii) Format - formats the downloaded data into a Pascal data file, to enable manipulation on the office computer.

(iii) Analysis - analyses the formatted data and provides suitable textual and graphical output of the analysis.

The structure of the office software is illustrated in Figure 5.8. On the HP 9816, the three functions are performed by three separate Pascal programs, namely; *ETIS_READ, ETIS_FRMT* and *ETIS_ANAL,* whereas on the HP 319, the read function is performed by a Unix shell program *etis_read,* and the format and analysis functions are performed by two Pascal programs; *etis_frmt* and *etis_anal.*

FORMATTED PASCAL DATA FILE

FORMAT FUNCTION

DOWNLOADED ASCII TEXT FILE

READ FUNCTION

HUSKY HUNTER ASCII TEXT FILE

INITIALIZE SURVEY DATA

Prefix name of surveys ..... DEMO

Enter new prefix name [Y/N] ?

Maximum number of surveys 10

Enter new maximum number [Y/N] ?

Precision of display data 3

Enter new display precision [Y/N] ?

GEOMETRICAL FUNCTIONS

D - DISTANCE FUNCTION

L - LINE FUNCTIONS

P - PLANE FUNCTIONS

V - VIEW SURVEY POINTS

Q - MAIN MENU

ANALYSIS FUNCTION

VIEW SURVEY POINTS

F - SHOW FACE : FALSE

N - VIEW SURVEY NO : 01

P - PLANE OF VIEW : XY

S - SELECT POINTS TO VIEW

V - VIEW SELECTED POINTS

Q - PREVIOUS MENU

FILE DATA INTERROGATION

A - ANALYSIS OF RESULTS

C - COORDINATE DATA

E - ERROR DATA

F - FOUR POINT DATA

P - POINT COMPARISON

R - RESULTS DATA

Q - MAIN MENU

DEFORMATION ANALYSIS

A - AXIS MOVEMENT AT A POINT

C/I - CHANGE / INITIALIZE FACE PARAMS

L - DEFORMATION LINE

P - PLANE MOVEMENT AT A POINT

R - RELATIVE DEFORMATION

V - VIEW SURVEY POINTS

Q - MAIN MENU

Figure 5.8 UN ETIS Office Software Structure.

175

5.3.2.1 Read Function

As part of the field software on the Husky Hunter 2 computer, at the end of a measurement survey, the Husky Basic data file of results is converted to an ASCII text file, for downloading to the office computer. The ASCII text file is downloaded as a series of data strings. The maximum length of a single data string that is permitted by the office computers is 255 characters. For this reason, the results for each observed point are contained in two strings; an observed data string and a calculated data string. The observed data string contains the point name, the names of the two control points from which the point was measured, the target height at the point, the instrument heights at the two control points, the angles measured from the control points to the point, and the deviations of these angles. The calculated data string contains the raw coordinates of the point and the mis-intersections in the raw coordinates, the least squares adjusted coordinates of the point, and the ellipsoid of error and probable error at the point.

The read function places the downloaded text data in a text file, of the same structure, on the office computer for permanent data storage and entry into the format function. On downloading, the operator has to enter a survey name and survey number for the downloaded text file, and it is logical that these should be the same name and number as the downloaded Husky Basic data file.

5.3.2.2 Format Function

This function takes the downloaded text file, obtained by the read function, and formats the text into a Pascal data file, which contains the same data as the Husky Basic results file on the Husky Hunter 2 computer.

On executing the program, the operator is initially requested to enter the downloaded text file name and survey number. The data is formatted by reading in a single data string, from the text file, and sequentially extracting the individual data from the string. This is performed by removing the first value in the data string, setting this value to a new parameter, and reforming the data string, without the extracted value. For example, consider the following observed data string

"point:lo","control1:lo","control2:lo", 0, 0, 0, 0.78539816, 0, 5.49778714, 0, 1.5, 0.6, 0.3, 1.5

The first stage in formatting the data is to record the length of the data string and the position of the first separating comma. In general, the data before the comma is set to a new parameter, ie point name = *point:lo*, and the string is reduced to starting after the position of the first comma, ie

"control1:lo","control2:lo", 0, 0, 0, 0.78539816, 0, 5.49778714, 0, 1.5, 0.6, 0.3, 1.5

This procedure is then repeated up to the last piece of data, in this case the fourth angular deviation value, of 1.5 seconds of arc, which is just read in as it stands. The whole process is repeated for every downloaded data string, and formatting ends on the detection of a point called *"EOF"* at the end of the text file.

5.3.2.3 Analysis Function

This function analyses the measurement survey data in the formatted Pascal data file. The analysis programs are menu driven, the structure of the menus is illustrated in Figure 5.8, and the mode of output is controlled by the operator. The use of menus reduces the amount of keyboard entry required. To perform most procedures the operator only has to select the menu options, enter point names and a survey number, and then select the mode of output.

An important aspect of the analysis software is its' ability to analyse a series of points, consisting of any combination of points, as defined by the operator. A series of points can range from a single point, to all of the points in a data file and, to prevent time consuming entry of individual point names, a series can be defined by the entry of a single point name containing wild card characters. There are four wild card characters, namely; dash (-), star (*), plus (+) and question mark (?), and their effects are best illustrated by an example data set of point names:

p1.1:lo, p1.1:ob, p1.2:lo, p1.2:ob
p2.1:lo, p2.1:ob, p2.2:lo, p2.2:ob

178

If no wild card characters are present then the software accepts the entered point name as the name of a single point, eg *p1.2:ob* will select the single point *p1.2:ob.* A dash (-) is used to select a sequential series of points in a data file, starting with the point name prior to the dash, ending with point name after the dash and including all points in between, eg *p1.2:lo-p2.1:lo* will select the series of points *p1.2:lo, p1.2:ob* and *p2.1:lo.* A star (*) is used at the end of a point name to select all points beginning with the characters prior to the star, eg *p1** will select the series of points *p1.1:lo, p1.1:ob, p1.2:lo* and *p1.2:ob.* To select all points in a file a single star is entered as the point name. A plus (+) is used to select a sequential series of points in a file, starting with the point name prior to the plus, and selecting each point at a specified increment through the file, eg *p1.1:lo+2* will select the series of points *p1.1:lo, p1.2:lo, p2.1:lo* and *p2.2:lo.* A combination of question marks (?), up to a maximum of three, can be used to represent any number of characters in a point name, eg *p???:lo* will select the series of points *p1.1:lo, p1.2:lo, p2.1:lo* and *p2.2:lo.*

On executing the program the operator is presented with a main menu of options:

D - Deformation Analysis

F - File Data Interrogation

G - Geometrical Functions

I - Initialize Survey Data

K - Keep Graphics: (TRUE / FALSE)

V - View Survey Points

Q - Quit

On selecting a main menu option, with the exception of option K, the operator is presented with a further sub-menu of options. Each of the main menu options is now detailed.

(i) Deformation Analysis

On selecting main menu option D, the operator is presented with the deformation analysis menu, which has the following options:

A   - Axis Movement at a Point
C/I - Change or Initialize Face Params
L   - Deformation Line
P   - Plane Movement at a Point
R   - Relative Deformation
V   - View Survey Points
Q   - Main Menu

The output from the options is graphical. If deformation is due to mining, the coal face parameters (x, y coordinates of the centre of the face) can be input and stored in a face file, which can be updated after each survey is complete. Initial input to the face file is by the option I and, once a face file is present, the operator can then update and change the face parameters by option C. If a face file is present, then this permits the graphical output of the spatial relationship between survey points and the face. However, if deformation is not due to mining, then this is replaced with a survey point key listing the names of the points represented on the graphical output.

The axis movement at a point option (A), and plane movement at a point option (P), perform analysis in two cases; the analysis of a single point in successive surveys, or the analysis of a series of points in an individual survey. With both options, the operator has to enter the name of the point to observe and a survey number. In the case of a single point, the survey number is used as the initial survey of the succession of surveys, whereas, in the case of a series of points, the survey number defines the individual survey.

Option A, in the deformation analysis menu, plots the movement of a chosen survey point, in any axis, against the survey number or point number. On selecting option A the operator is presented with the following menu of options:

A   - All Axes Movement

F/K - Face / Point View or Survey Point Key

X   - X Axis Movement

Y   - Y Axis Movement

Z   - Z Axis Movement

Q   - Deformation Menu

In all cases the movement is shown as a bar chart of axis coordinate movement against survey / point number. The single axis options output one bar chart, and the all axes option outputs three bar charts and either a face / point view, or a survey point key (Figure 5.9).

Option P, in the deformation analysis menu, plots the movement of a chosen survey point in a plane formed by two axes. On
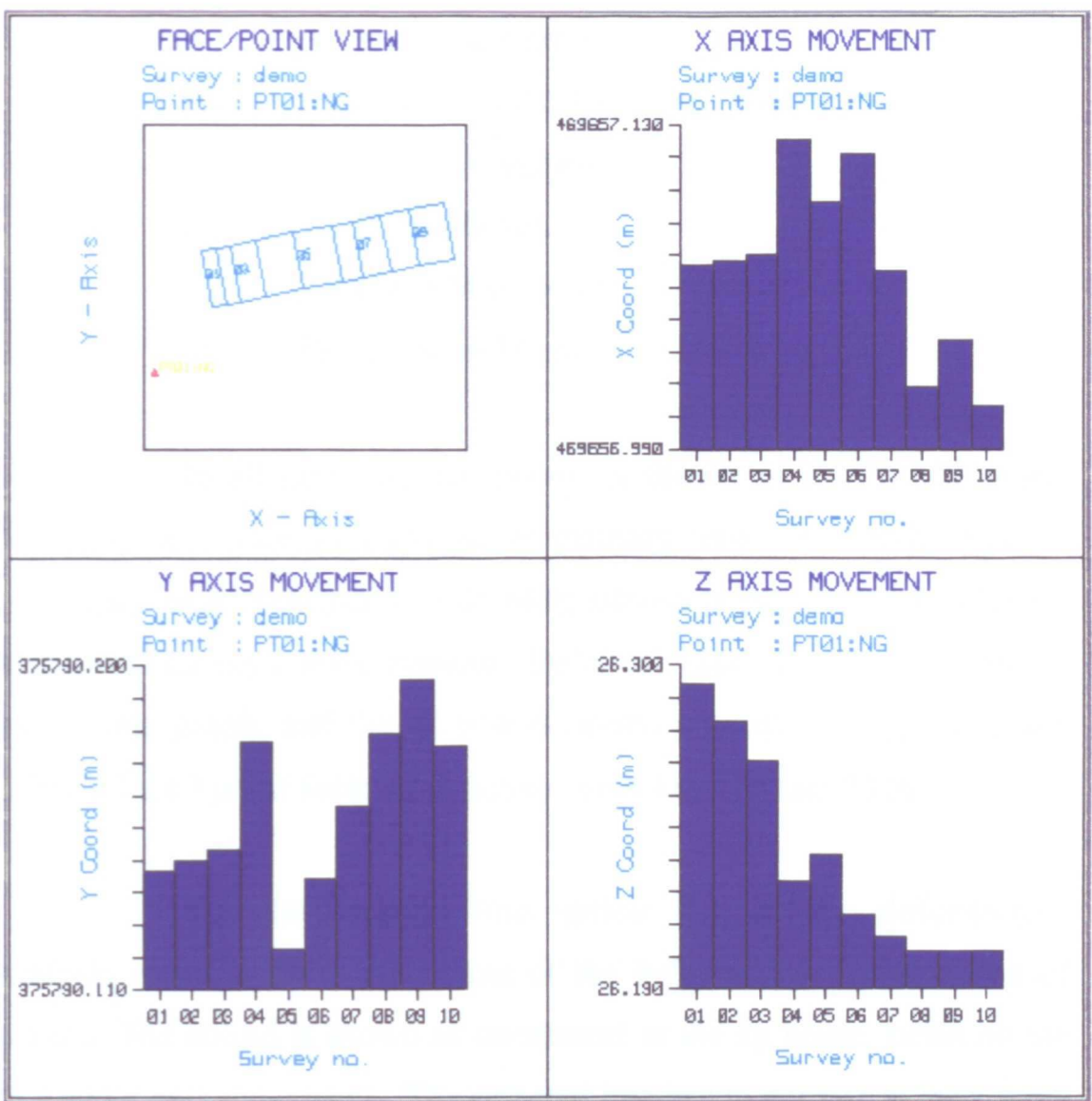
Figure 5.9 UN ETIS Example Software Output - All Axes Movement.

selecting option P, the operator is presented with the following menu of options:

A   - All Planes Movement

F/K - Face / Point View or Survey Point Key

1   - XY Plane Movement

2   - YZ Plane Movement

3   - XZ Plane Movement

Q   - Deformation Menu

In all cases the movement is shown as a series of linked points on a square grid graph, of primary axis coordinate, against secondary axis coordinate, with each survey point plotted having its' associated survey / point number displayed. The single plane options output one graph, and the all planes option outputs three graphs, and either a face / point view, or a survey point key (Figure 5.10).

The deformation line option (L), in the deformation analysis menu, enables the output of the deformation due to a line of points. The output is shown as movement in the xy plane, strain on the line and z axis movement. The selected line has to contain at least three points, eg a subsidence line across a face advance. On selecting option L, the operator is presented with the following menu of options:

A - All Line Movement

F - Face / Line View

I - Initialize Line

L - Linework (ON / OFF)

P - XY Plane Movement
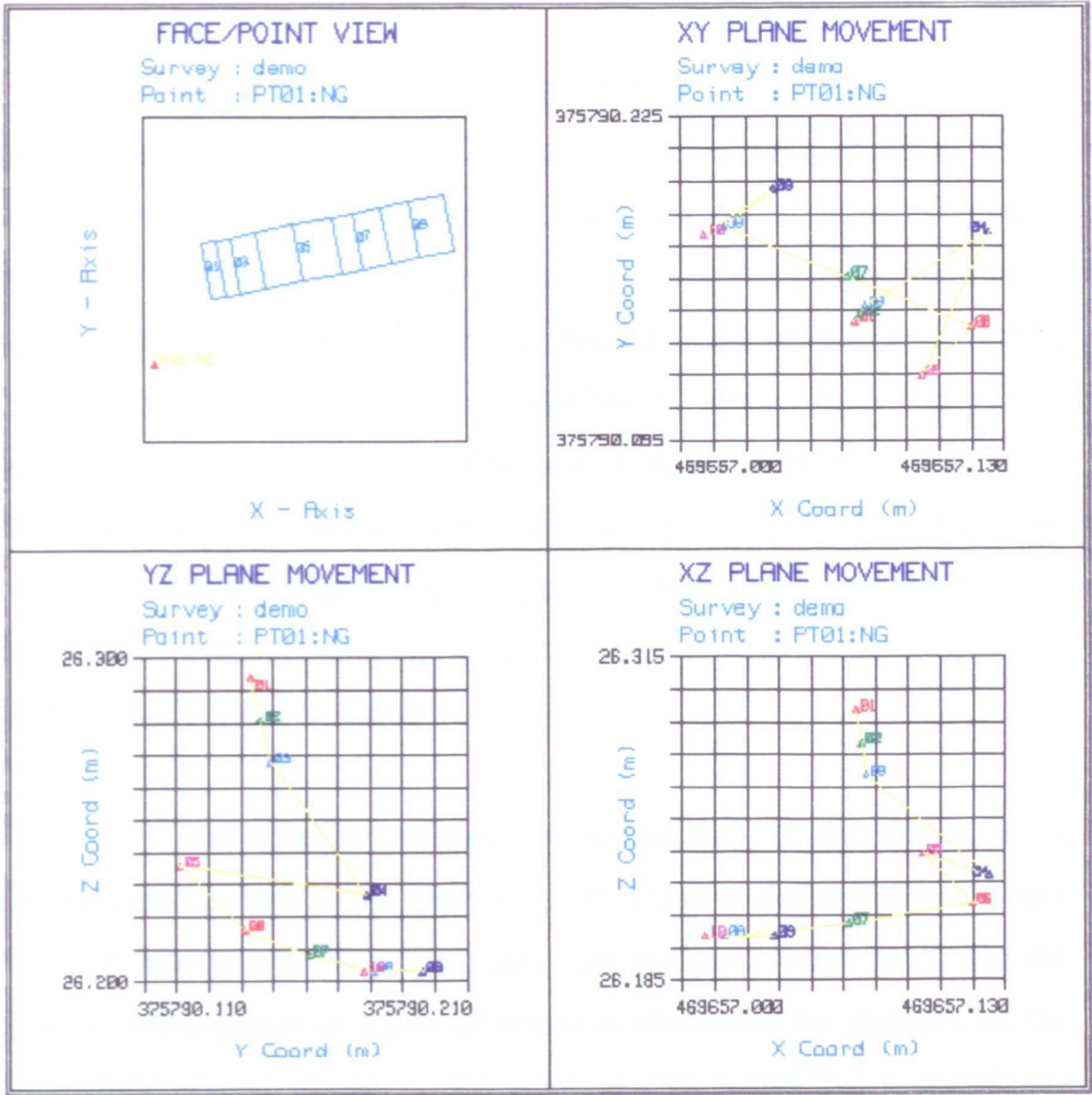
Figure 5.10 UN ETIS Example Software Output - All Planes Movement.

Y - Defm Line Strain

Z - Z Axis Movement

Q - Deformation Menu

First, the operator has to initialize a line from option I, by defining the points on the line. Strain is shown on a graph of strain against distance / point name, and z axis movement is shown on a graph of z coordinate against distance / point name. In both cases the surveys are plotted with a unique colour / symbol combination, and a key listing the survey number and colour / symbol is shown with the graph. Movement in the xy plane is shown as a series of linked points, on a square grid graph of x coordinate against y coordinate, with each point plotted having a unique colour / symbol, and a key listing point name and colour / symbol being shown with the graph. An example output of All Line Movement is shown in Figure 5.11.

The relative deformation option (R), in the deformation analysis menu, enables the analysis of the relative deformation due to a series of four points. The four points are analysed in six pairs, and the relative deformation of a pair of points is illustrated by changes in; the three-dimensional distance, the relative strain and the z coordinate difference, between the points. On selecting option R the operator first has to select four points, and is then presented with the following menu of options:

A - All Rltv Movement

D - Dist Rltv Movement

F - Face / Point View

S - Strain Relative
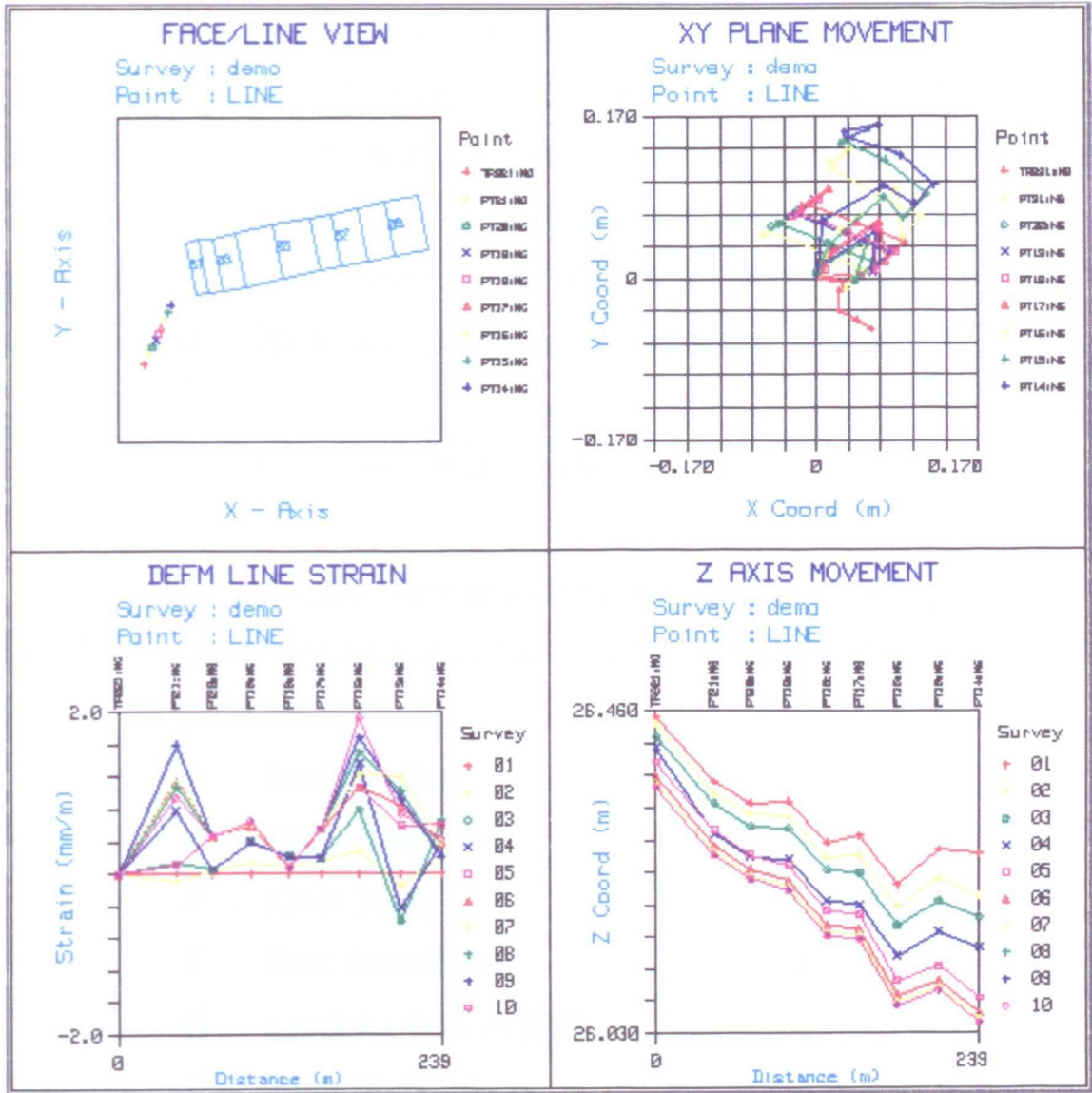
Figure 5.11 UN ETIS Example Software Output - All Line Movement.

Z - Z Rltv Movement

Q - Deformation Menu

In all cases, the four selected points are analysed in six pairs, and the movement is represented by a bar chart. Each bar chart is split into six sections, with each section showing relative movement against survey number, for a pair of selected points. The single axis options output one bar chart, and the all rltv movement option outputs three bar charts, and a face / point view if applicable (Figure 5.12).

(ii) File Data Interrogation

On selecting main menu option F, the operator is presented with the file data interrogation menu, which has the following options:

A - Analysis Data

C - Coordinate Data

E - Error Data

F - Four Point Data

G - Graphics (ON / OFF)

P - Point Comparison

R - Results Data

Q - Main Menu

All of the options permit output to the screen or printer (hard copy). The output from options C, E and P is tabular, and the output of R is textual, with additional graphics output controlled by the boolean set by option G. If option G is set to ON, then the textual output is accompanied by a graphical display of the relative points
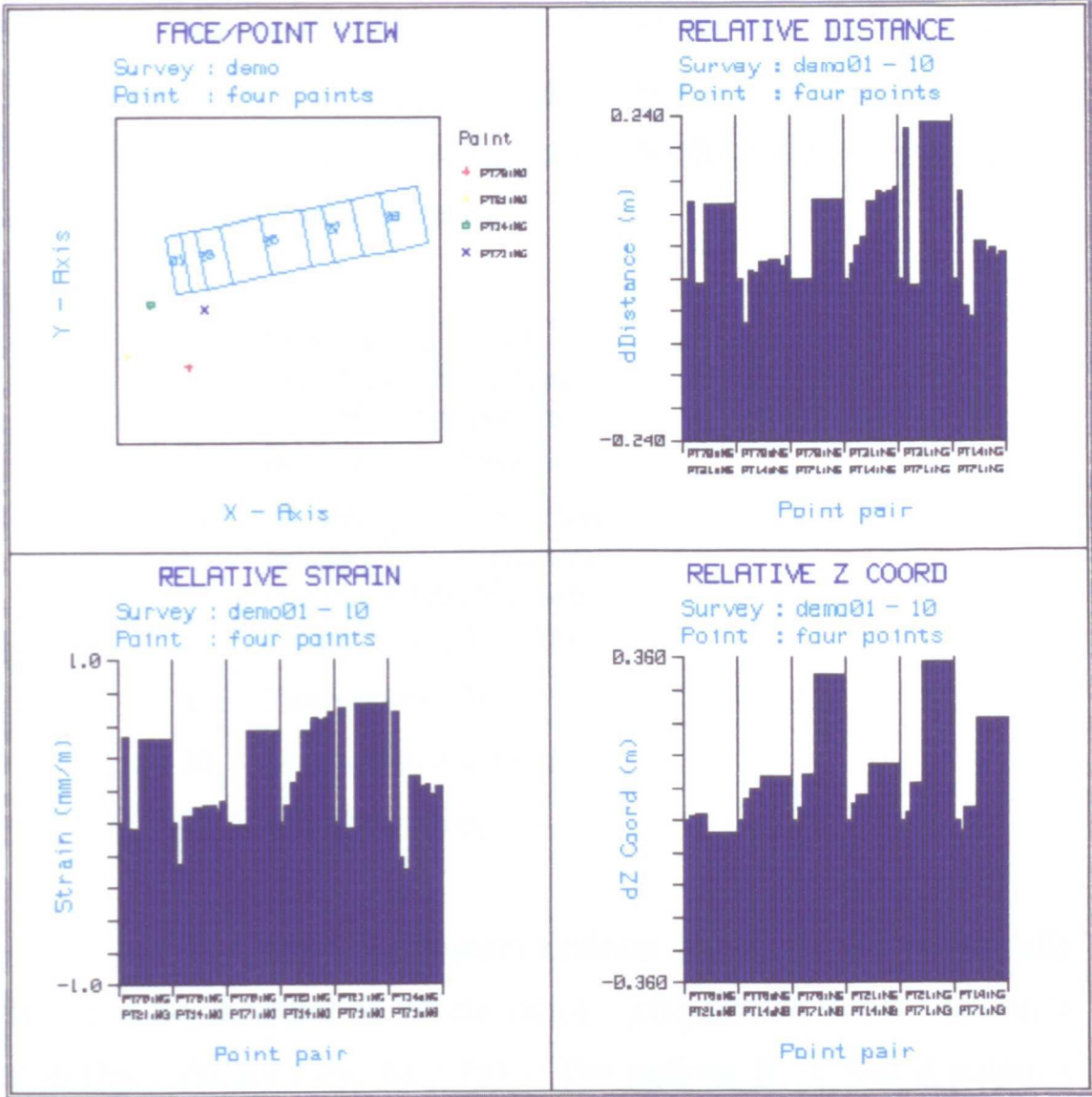
Figure 5.12 UN ETIS Example Software Output - All Rltv Movement.

involved in the intersection, and the three-dimensional intersection angle.

The analysis data option (A), in the file data interrogation menu, enables the operator to analyse the data of a set of chosen points, in a chosen survey. On selecting option A, the operator is presented with the following menu of options:

```
E  -  Error Distribution
X  -  Ex / Px Distribution
Y  -  Ey / Py Distribution
Z  -  Ez / Pz Distribution

G  -  Geometry Distribution
C  -  Int Angle Distribution
1  -  Length 1 Distribution
2  -  Length 2 Distribution

I  -  Intersection Data

M  -  Mean Analysis Data

Q  -  File Data Menu
```

The error distribution options (E, X, Y, Z) graphically represent the distribution of the ratios - ellipsoid of error / probable error (Ex / Px, Ey / Py, Ez / Pz) . The options X, Y and Z output a single bar chart of ratio against point number, and the option E outputs three bar charts of ratio against point number, and a survey point key (Figure 5.13). The geometry distribution options (G, C, 1, 2) graphically represent the distribution of the geometrical factors affecting the intersection, namely; intersection angle and sight lengths. The options C, 1 and 2 output a single bar chart of geometrical factor against point number, and the option G outputs three bar charts of geometrical factor against point number, and a survey point key (Figure

189
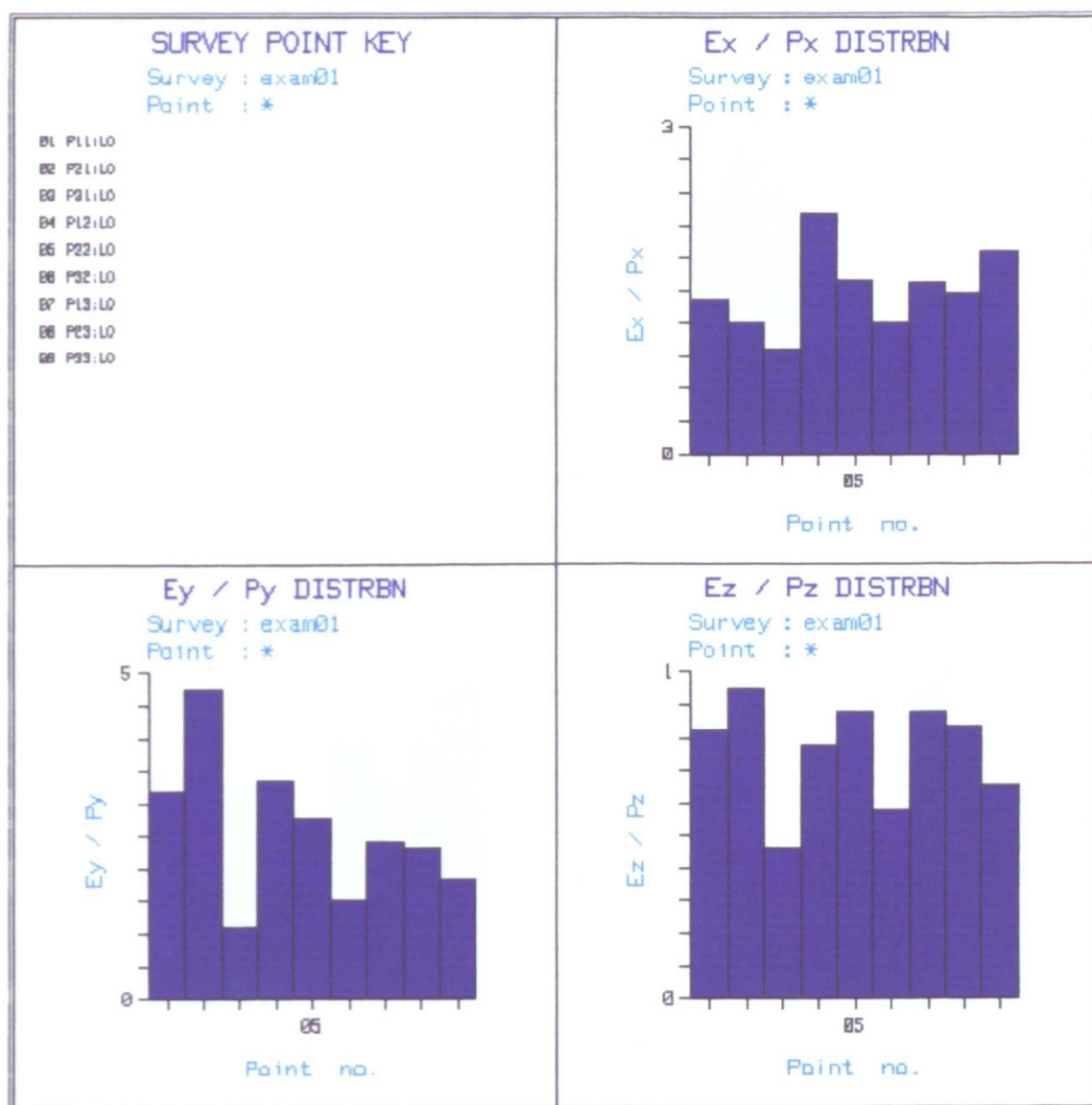
Figure 5.13 UN ETIS Example Software Output - Error Distribution.
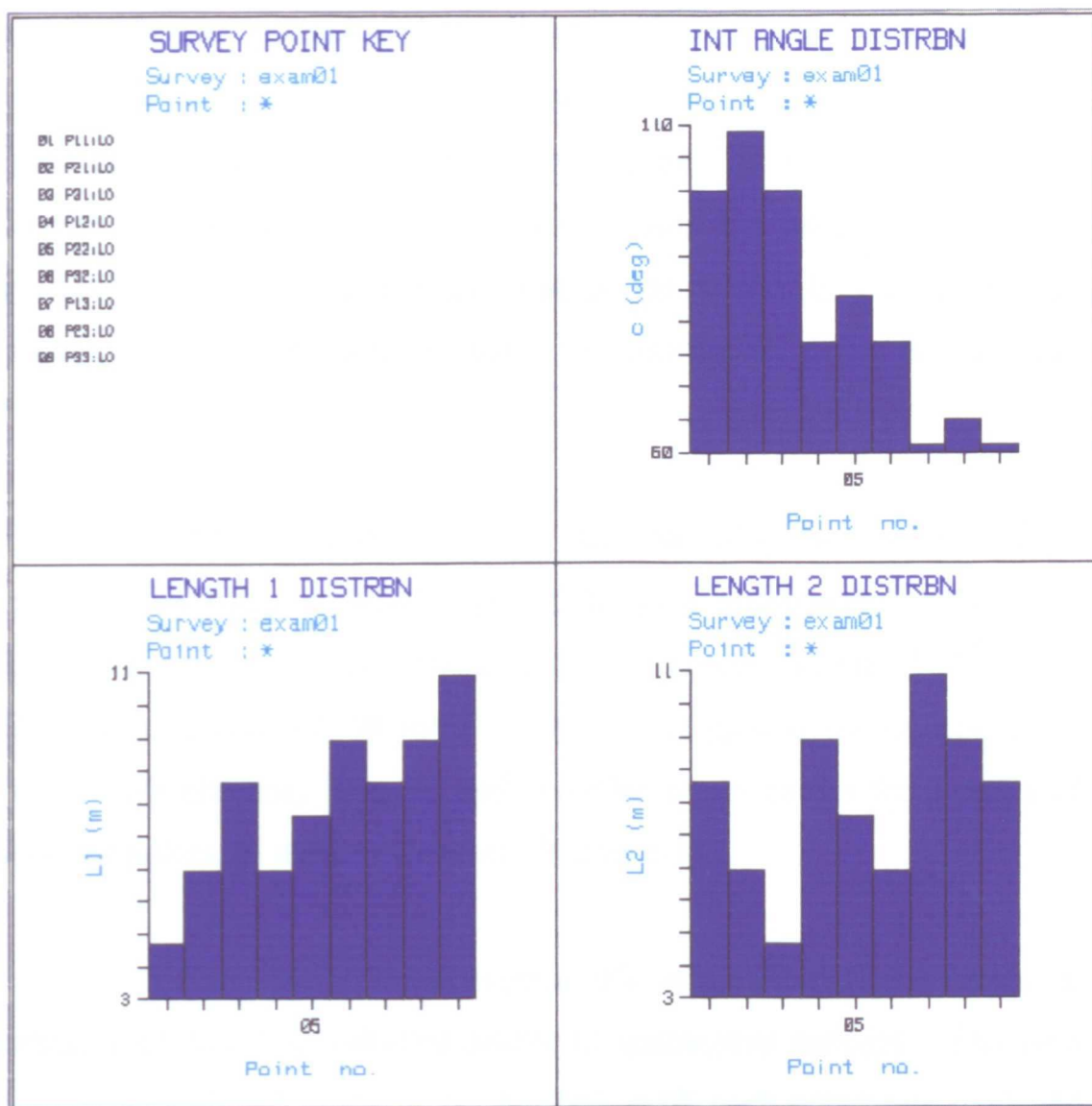
Figure 5.14 UN ETIS Example Software Output - Geometry
Distribution.

5.14). The intersection data option (I) outputs the geometrical factors mentioned above in a tabular form (Figure 5.15). The mean analysis option (M) outputs the mean data for a series of points, in a chosen survey. This output includes the; total number of points selected, mean angular deviations, mean mis-intersection values, mean ellipsoid of error values and mean probable error values for the selected points (Figure 5.16). These four analysis options are used to detect any systematic errors, eg a high angular deviation in one angle, or geometrical defects, which could be eliminated prior to the next measurement survey.

The coordinate data option (C), in the file data interrogation menu, outputs a table of the three-dimensional coordinates for a series of selected points in a chosen survey (Figure 5.17). The error data option (E), in the file data interrogation menu, outputs a table of the ellipsoid of error, and probable error, values for a series of selected points, in a chosen survey (Figure 5.18).

The four point data option (F), in the file data interrogation menu, analyses four selected points in successive surveys. The four points are analysed as six pairs of points, with each pair being analysed in turn. For each pair of points, the output is tabular and shows the three-dimensional distance, and difference in z coordinate, between the points, in successive surveys (Figure 5.19).

The point comparison option (P), in the file data interrogation menu, enables numerical output of coordinate comparison values for a single point in successive surveys, or for a series of points in an individual survey. It lists, in tabular form, the three-dimensional

Point Name	c (deg)	L1 (m)	L2 (m)
P11:LO	100	4.330106	8.291536
P21:LO	109	6.123713	6.123715
P31:LO	100	8.291588	4.330042
P12:LO	77	6.123801	9.354074
P22:LO	84	7.500046	7.499819
P32:LO	77	9.354191	6.123515
P13:LO	61	8.291679	10.897080
P23:LO	65	9.354257	9.353922
P33:LO	61	10.897283	8.291261

Figure 5.15 UN ETIS Example Software Output - Intersection Data

***** DATA FILE EXAM01_R MEAN ANALYSIS DATA *****

Selected Point Name :- *
Total No. of Points :- 9

Mean Angular Deviations (secs)

THEODOLITE 1		THEODOLITE 2		
H A	V A	H A	V A	SURVEY MEAN
0.9	1.0	0.9	0.9	0.9

Mean Mis-Intersections (mm)

Dx	Dy	Dz	SURVEY MEAN
0.0000	0.0336	0.0617	0.0318

Mean Ellipsoid Errors (mm)

Ex	Ey	Ez	SURVEY MEAN
0.0218	0.0265	0.0179	0.0221

Mean Probable Errors (mm)

Px	Py	Pz	SURVEY MEAN
0.0149	0.0115	0.0240	0.0168

Figure 5.16 UN ETIS Example Software Output - Mean Analysis Data

***** DATA FILE EXAM01_R COORDINATE DATA *****

Point Name	X Coord (m)	Y Coord (m)	Z Coord (m)
T1:LO	0.000000	0.000000	0.000000
T2:LO	10.000000	0.000000	0.000000
P11:LO	2.500012	2.500016	2.499936
P21:LO	4.999999	2.499999	2.499976
P31:LO	7.500059	2.499956	2.499955
P12:LO	2.500112	5.000082	2.499912
P22:LO	5.000170	4.999977	2.499846
P32:LO	7.500173	4.999908	2.499843
P13:LO	2.500280	7.500115	2.499764
P23:LO	5.000313	7.500019	2.499741
P33:LO	7.500289	7.499860	2.499709

Figure 5.17 UN ETIS Example Software Output - Coordinate Data

***** DATA FILE EXAM01_R ERROR DATA *****

Point Name	Ex / Px (mm)	Ey / Py (mm)	Ez / Pz (mm)
T1:LO	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000
T2:LO	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000
P11:LO	0.0281	0.0245	0.0213
	0.0199	0.0077	0.0259
P21:LO	0.0262	0.0214	0.0155
	0.0217	0.0045	0.0163
P31:LO	0.0167	0.0121	0.0122
	0.0173	0.0109	0.0264
P12:LO	0.0300	0.0374	0.0190
	0.0136	0.0112	0.0244
P22:LO	0.0226	0.0201	0.0170
	0.0142	0.0073	0.0193
P32:LO	0.0142	0.0219	0.0150
	0.0118	0.0145	0.0259
P13:LO	0.0204	0.0386	0.0230
	0.0130	0.0160	0.0261
P23:LO	0.0183	0.0274	0.0197
	0.0123	0.0118	0.0235
P33:LO	0.0200	0.0352	0.0187
	0.0107	0.0192	0.0285

Figure 5.18 UN ETIS Example Software Output - Error Data

Pt Name 1 / 2	Survey	3D Distance (m)	dZ (m)
P11:LO	01	5.000099	0.000172
P13:LO	02	5.000233	0.000245
P11:LO	01	7.071153	0.000227
P33:LO	02	7.071199	0.000330
P11:LO	01	5.000047	-0.000019
P31:LO	02	5.000051	0.000046
P13:LO	01	5.000009	0.000055
P33:LO	02	4.999944	0.000085
P13:LO	01	7.071024	-0.000191
P31:LO	02	7.071035	-0.000199
P33:LO	01	4.999904	-0.000247
P31:LO	02	4.999913	-0.000284

Figure 5.19 UN ETIS Example Software Output - Four Point Data

Pt Name / Survey	X / dX (m)	Y / dY (m)	Z / dZ (m)
P11:LO	2.500012	2.500016	2.499936
01	0.000000	0.000000	0.000000
P11:LO	2.500162	2.500114	2.499892
02	0.000150	0.000098	-0.000044

Figure 5.20 UN ETIS Example Software Output - Point Comparison

coordinates, and coordinate comparison values, for the selected points. In the case of a single selected point, the comparison values are obtained by comparing the point coordinates in each survey with the point coordinates in the initial survey (Figure 5.20). In the case of more than one selected point, the comparison values are obtained by comparing the point coordinates of each named point with the point coordinates of the first named point.

The results data option (R), in the file data interrogation menu, outputs the full results data from the data file for any combination of chosen points in a chosen survey. The data output, for each selected point includes; the point name, the names of the two control points from which the point was measured, the target height at the point, the instrument heights at the two control points, the angles measured from the control points to the point and the deviations of these angles, the raw coordinates of the point and the mis-intersections in the raw coordinates, the least squares adjusted coordinates of the point, and the ellipsoid of error and probable error at the point (Figure 5.21).

(iii) Geometrical Functions

On selecting main menu option G, the operator is presented with the geometrical functions menu, which has the following options:

D - Distance Function

L - Line Functions

P - Plane Functions

V - View Survey Points

Q - Main Menu

```
***** DATA FILE EXAM01_R RESULTS DATA *****

Point Name      :-  P11:LO      Point Ht  :-  0.00
Theodolite 1    :-  T1:LO       Instr Ht  :-  0.00
Theodolite 2    :-  T2:LO       Instr Ht  :-  0.00

Mean Angles and +/- Deviations
H Angle 1     45 00 02.33      Dev +/-  1.0  secs
V Angle 1     35 15 52.17      Dev +/-  1.0  secs
H Angle 2    288 26 08.00      Dev +/-  0.9  secs
V Angle 2     17 32 51.50      Dev +/-  1.0  secs

Raw Coordinates & Mis-Intersections
X Coord         2.500021  m    Dx    0.0000  mm
Y Coord         2.500023  m    Dx    0.0582  mm
Z Coord         2.499944  m    Dx    0.0582  mm

Least Squares Adjusted Data
X Coord         2.500012  m    Dx    0.0281  mm   Px   0.0199  mm
Y Coord         2.500016  m    Dx    0.0245  mm   Py   0.0077  mm
Z Coord         2.499936  m    Dx    0.0213  mm   Pz   0.0259  mm
```

Figure 5.21 UN ETIS Example Software Output - Results Data

```
***** DATA FILE EXAM DISTANCE FUNCTION DATA *****

Point 1          :-  P11:LO
Survey Number    :-  01

Point 2          :-  P33:LO
Survey Number    :-  02

Coordinate Details  :-

X first point          2.500012  m
Y first point          2.500016  m
Z first point          2.499936  m

X second point         7.500667  m
Y second point         7.499795  m
Z second point         2.499561  m

Spatial Distance Details  :-

Distance in X plane    5.000654  m
Distance in Y plane    4.999779  m
Distance in Z plane   -0.000375  m

3D Spatial Distance    7.071375  m
```

Figure 5.22 UN ETIS Example Software Output - Distance Function

Options D, L and P output data to the screen or printer (hard copy).

The distance function option (D), in the geometrical functions menu, outputs the results of a two point comparison; the comparison of the coordinates of a single point in a chosen survey (a), with a second point in a chosen survey (b). The compared coordinates are output as three-dimensional spatial distances (dX, dY, dZ, dD). The output includes; the three-dimensional coordinates of point 1 in survey a, and of point 2 in survey b, together with the three-dimensional spatial distances (Figure 5.22).

On selecting the Line Functions option (L), in the geometrical functions menu, the operator is presented with the following menu of options:

D - Define a Line
E - Equation of a Line
I - Intersection of Two Lines
P - Distance from a Point to a Line
Q - Geometrical Functions Menu

At present, a straight line can be defined using two points in a chosen survey. At any point during analysis, the equation, and other line information, of a defined straight line can be output to the screen, or printer, by selecting option E. The intersection characteristics of any two previously defined straight lines can be analysed by selecting option I, and the distance from any point in a chosen survey, to a previously defined straight line can be calculated by

selecting option P. Example outputs from the Line Functions option are given in Figure 5.23.

On selecting the Plane Functions option (P), in the geometrical functions menu, the operator is presented with the following menu of options:

D - Define a Plane

E - Equation of a Plane

I - Intersection of Two Planes

L - Intersection of a Line and a Plane

P - Distance from a Point to a Plane

Q - Geometrical Functions Menu

At present, a plane can be defined using three points in a chosen survey. At any point during analysis, the equation, and other plane information, of an existing plane can be output to the screen, or printer, by selecting option E. Other example outputs from the Plane Functions option are given in Figure 5.24. These include; the intersection of any two previously defined planes, resulting in the generation of a new straight line (option I); the intersection of a previously defined plane, with a previously defined straight line (option L);. the distance from any point in a chosen survey to a previously defined plane (option P).

(iv) Initialize Survey Data

Initial data, essential to the correct execution of the analysis function includes; the prefix name of a series of surveys, the maximum

199

```
***** DATA FILE EXAM01_R LINE EQUATION DISPLAY *****

Line Name       :-  LINE1

Line Point 1    :-  P11:LO
Line Point 2    :-  P33:LO

Line Equation   :-


        X - 2.500012    Y - 2.500016    Z - 2.499936
        ------------ = ------------ = ------------
          0.707137        0.707076       -0.000032
```

```
***** DATA FILE EXAM01_R LINE EQUATION DISPLAY *****

Line Name       :-  LINE2

Line Point 1    :-  P31:LO
Line Point 2    :-  P13:LO

Line Equation   :-


        X - 7.500059    Y - 2.499956    Z - 2.499955
        ------------ = ------------ = ------------
         -0.707080        0.707134       -0.000027
```

```
***** DATA FILE EXAM01_R LINE EQUATION DISPLAY *****

Line Name       :-  LINE3

Line Point 1    :-  P31:LO
Line Point 2    :-  P33:LO

Line Equation   :-


        X - 7.500059    Y - 2.499956    Z - 2.499955
        ------------ = ------------ = ------------
          0.000046        1.000000       -0.000049
```

Figure 5.23 UN ETIS Example Software Output - Line Functions

200

```
***** DATA FILE EXAM01_R INTERSECTION : TWO LINES *****

First Line      :-  LINE1
Second Line     :-  LINE2

Intersection    :-  The two chosen lines do not intersect - the
                    shortest distance between the lines is their common
                    perpendicular with end point coordinates given below.

Common Perpendicular Details  :-

X first line        5.000209   m
Y first line        4.999996   m
Z first line        2.499822   m

X second line       5.000209   m
Y second line       4.999996   m
Z second line       2.499860   m

Length              0.000037   m




***** DATA FILE EXAM01_R INTERSECTION : TWO LINES *****

First Line      :-  LINE1
Second Line     :-  LINE3

Intersection    :-  The two chosen lines intersect at the point
                    coordinates given below.

Point of Intersection Details  :-

X intersection      7.500289   m
Y intersection      7.499860   m
Z intersection      2.499709   m

Intersection Angle 44 59 59.43




***** DATA FILE EXAM01_R DISTANCE : POINT / LINE *****

Line        :-  LINE1
Point       :-  P22:LO

Point Details  :-

X point         5.000170   m
Y point         4.999977   m
Z point         2.499846   m

Distance Details  :-

Point to line   0.000027   m
```

Figure 5.23 UN ETIS Example Software Output - Line Functions

(continued)

```
Plane Name        :-  PLANE1

Plane Point 1     :-  P11:LO
Plane Point 2     :-  P31:LO
Plane Point 3     :-  P33:LO

Plane Equation    :-
```

$$- 0.000004 . X + 0.000049 . Y + 1.000000 . Z - 2.500050 = 0$$

```
Plane Name        :-  PLANE2

Plane Point 1     :-  P11:LO
Plane Point 2     :-  P13:LO
Plane Point 3     :-  P33:LO

Plane Equation    :-
```

$$- 0.000011 . X - 0.000034 . Y - 1.000000 . Z - 2.500050 = 0$$

```
Plane Name        :-  PLANE3

Plane Point 1     :-  P11:LO
Plane Point 2     :-  P13:LO
Plane Point 3     :-  P33:LO

Plane Equation    :-
```

$$- 0.000017 . X - 0.000049 . Y - 1.000000 . Z - 2.500057 = 0$$

Figure 5.24 UN ETIS Example Software Output - Plane Functions

```
          ***** DATA FILE EXAM01_R INTERSECTION : TWO PLANES *****

Line Name        :-  LINE4

Line Point 1     :-  PLANE1
Line Point 2     :-  PLANE2

Line Equation    :-


              X - 2.500012     Y - 2.500016     Z - 2.499936
              ------------  =  ------------  =  ------------
                0.707137         0.707076        -0.000032




          ***** DATA FILE EXAM01_R INTERSECTION : LINE / PLANE *****

Line             :-  LINE2
Plane            :-  PLANE2

Intersection     :-  The chosen line and plane do intersect -
                     the intersection point coordinates are given below.

Point of Intersection Details  :-

X intersection       2.500280  m
Y intersection       7.500115  m
Z intersection       2.499764  m




          ***** DATA FILE EXAM01_R DISTANCE : POINT / PLANE *****

Plane            :-  PLANE1
Point            :-  P22:LO

Point Details  :-

X point              5.000170  m
Y point              4.999977  m
Z point              2.499846  m

Distance Details  :-  The point is above the plane.

Point to plane       0.000023  m
```

Figure 5.24 UN ETIS Example Software Output - Plane Functions

(continued)

203

number of surveys performed and the precision to which data is to be displayed. The operator can change this initial data at any point during analysis, by selecting main menu option I. By changing this data during analysis, the operator can analyse data from several different surveys during a single execution of the software. In addition, when the analysis function is executed, the initial data is set to default values from the previous execution. Thus, the operator is presented with the main menu at the start of program execution.

(v) Keep Graphics: (TRUE / FALSE)

Main menu option K permits graphical output to be stored, to enable hard copy output at a later stage. On the HP 9816, the graphical output is stored to the Hasp Database, and can then be edited, and plotted, using Hasp software. On the HP 319, the plotted output is saved as a bit map image, which is then converted to a paintjet file, before plotting.

(vi) View Survey Points

The view survey points option is available as option V in the main menu, the deformation analysis menu and the geometrical functions menu. It enables a graphical output of the relative positions of selected survey points in a two-dimensional plane. On selecting the view survey points option, the operator is presented with the following menu:

F - Show Face (TRUE / FALSE)
N - View Survey No
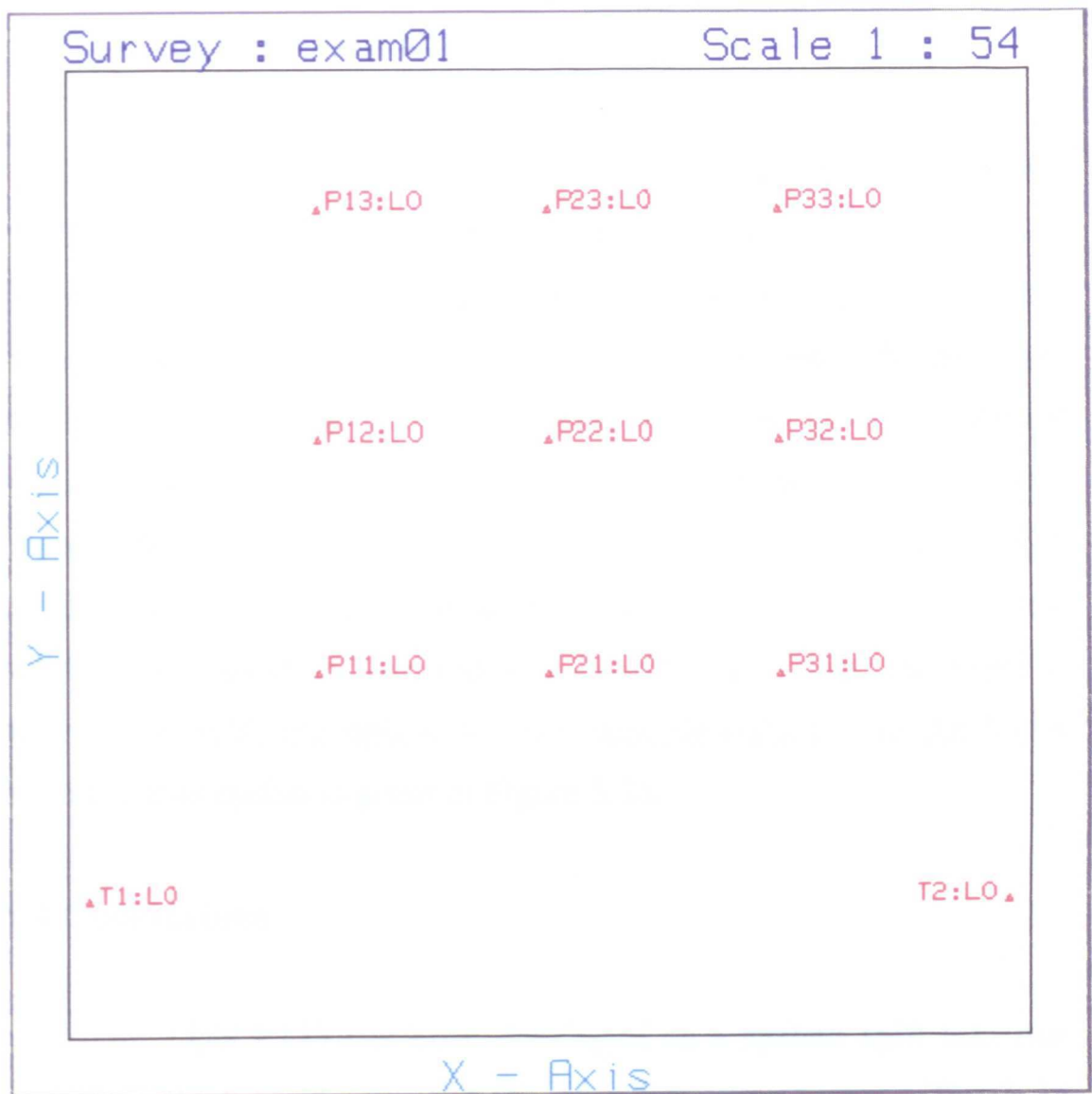
Figure 5.25 UN ETIS Example Software Output - View Survey Points.

P - Plane of View

S - Select Points to View

V - View Selected Points

Q - Previous Menu

Default values are set for the view parameters, ie show face is set to FALSE, view survey number to 1 and plane of view to XY. However, only if a face file is present, and the view is in the XY plane, is the show face option available. On selecting option N, the view survey number is incremented, in a barrel fashion, up to the defined maximum number of surveys. The plane of view option (P), also operates barrel fashion through the three planes; XY, XZ, YZ. Once the operator has defined view survey number and plane of view, the points to be viewed are selected from option S, and graphical output is obtained on selecting option V. An example output from the View Survey Points option is given in Figure 5.25.

5.4 Conclusions

UN ETIS has been developed as a system split into two separate halves; field components and office components. The field components form a battery powered ETIS, capable of performing coordinate determination in the field. The measurement survey data is stored in a suitable format to permit downloading from the field computer to the office computer, for processing, archival storage and analysis.

Unlike the commercially available systems detailed in Chapter Four, the emphasis during development was not on speed of

coordinate determination, or the ability to perform detailed analysis in the field. Instead, the emphasis during development was to create a truely portable system of coordinate determination, for use in the field, whilst maintaining the ability to perform unlimited analysis of the measured data in the office.

The main body of this Chapter has concentrated on the in-house development of the field and office software, by the author. This is now complete, although the inherent flexibility of in-house development enables refinement and further development to be performed in the future, if required.

Once the main development of the system was complete, the aims of the research were to:

(i) Test the field and office software under a range of operating parameters using theoretical values.

(ii) Become familiar with the system, and develop the necessary observational skills, before commencing field trials.

(iii) Investigate the use of the system in industrial measurement, as detailed in Chapter Six.

(iv) Perform field trials to monitor deformation due to mining, as detailed in Chapter Seven.

CHAPTER SIX

FIELD EVALUATION OF UN ETIS FOR INDUSTRIAL MEASUREMENT

Field Evaluation of UN ETIS for Industrial Measurement

6.1 Introduction

An important activity in industrial processes is that of performing industrial measurement. For coping with measurement tasks that occur in industry, there is an ever increasing need for measuring equipment that is suitable for universal use, and offers high accuracy. Particularly in the case of large-scale objects, such as occur in aerospace and automotive engineering, and applications involving industrial robots, this need is paramount.

This Chapter describes some alternative industrial measurement methods and outlines their applications, advantages and disadvantages. The Chapter then details an example of industrial measurement using UN ETIS, for the calibration of production robots, and discusses the results obtained.

6.2 Alternative Instrumentation and Techniques for Industrial Measurement

For most of the applications of ETISs there is one, or more, alternative instruments and / or techniques which could also be used. This section considers five alternatives, namely; mechanical measurement, traditional industrial surveying, coordinate measuring machines, close-range photogrammetry and laser measurement methods.

6.2.1 Mechanical Measurement

Mechanical measurement of small-scale objects can be performed by relatively inexpensive equipment found within engineering workshops, such as dial gauges and vernier calipers. Large-scale, or complex objects are difficult to measure, and may require the construction of a measurement device for a specific measurement task.

There is obviously a limit to the size, and complexity of object which can be measured to high accuracy by mechanical devices. In addition, mechanical measurement can often be inconvenient to use, and can cause undesirable side effects, such as work interruptions and deformation of the object being measured.

6.2.2 Traditional Industrial Surveying

For many years, conventional surveying instruments have been used for numerous measuring and adjustment applications, required by industrial processes.

Levelling instruments, including precise levels and laser levels, can be established horizontally to a high degree of accuracy, thus, permitting the measurement of the height difference between points, or the setting out of points at specified heights. Uses of levelling instruments include the setting of foundations, machinery and machinery elements in horizontal positions.

In the past, the application of the optical mechanical theodolite has been limited to that of an alignment instrument, permitting the alignment of points in straight lines and planes. The applicability of such instruments has been enhanced by numerous accessories, such as autocollimation eyepieces, adapter lenses for short sightings, elbow eyepieces for steep sightings and telescope mounted alignment lasers. However, prior to the introduction of ETISs, the theodolite has seldom been used, as a genuine angle measuring instrument, in industrial measurement.

6.2.3 Coordinate Measuring Machines

While, in theory, objects with dimensions of one metre, or less, can be measured by an ETIS, it is generally considered more appropriate to use a coordinate measuring machine (CMM).

A CMM consists of a rigid machine body, computer controlled probing system and workpiece table, on which the object to be measured is mounted. Measurements are performed by the computer controlled probing system, which consists of a series of probe heads. The movement of each individual probe head is monitored by transducers, which have a typical resolution of 0.5 μm, and three-dimensional coordinate measurements are obtained as the probe heads contact the object surface.

An example of a CMM currently available is the Leitz PMM series. This series consists of six different machines, with object sizes ranging from 654 (600 x 500 x 400 mm) up to 181010 (1800 x 1000 x 1000 mm). These machines permit manual, or automatic,

measurement sequences and can determine three-dimensional coordinates to accuracies of 0.5 μm [Leitz 1986].

There is no doubt that for small-scale objects, such as engine components, tools and gauges, the high accuracy of a CMM cannot be rivalled by an ETIS. However, CMMs have two main disadvantages; firstly, they are stationary measurement systems and, secondly, a specific machine can only measure within a fixed object size range. These two limiting factors mean that the object being measured has to be portable enough to be brought to the machine, which is exactly the opposite case to that of ETISs, which are capable of measuring objects in-situ.

6.2.4 Close-Range Photogrammetry

Photogrammetry can be described as the science of obtaining reliable measurements by means of photographs. The term close-range photogrammetry is usually used to describe the situation where the camera-to-object distance is less than 300 m.

The object being measured is photographed by a number of metric cameras. The resultant photographs are analysed by a stereoplotter, linked directly to a computer, enabling the coordinates of points on the object to be determined to a high accuracy.

In theory, close-range photogrammetry can be used to perform all of the measurement tasks performed by an ETIS. In addition, it provides a permanent record which can be re-examined in the future if more measurements need to be taken. However, its' main

disadvantage is that, unlike an ETIS, it cannot provide results in real-time.

6.2.5 Laser Measurement Methods

The helium-neon (He-Ne) laser has been used extensively in many areas of engineering and survey measurement. As far as industrial measurement is concerned, the most practical application of lasers has been in the dynamic measurement of production robots.

A recently developed, non-contact, technique involves the automatic tracking of a retro-reflective prism, mounted on a robot arm, by two separate laser beams [Parker & Mayer 1987]. The measurement system set up is similar to an ETIS, with the electronic theodolites being replaced by laser measurement units, and the coordinates of tracked points are calculated by spatial intersection. The system is reported to be able to track a trajectory to an accuracy better than ±0.1 mm, in a 1 m cube measurement space, at velocities in excess of 5 m/s.

6.3 Industrial Measurement using UN ETIS

As the main application of ETISs is in industrial measurement, it was logical to evaluate UN ETIS in such an application. The Department of Production Engineering, at the University of Nottingham, is currently performing research on the calibration of production robots, which provided an ideal application.

This research requires some form of high accuracy, non-contact measurement of three-dimensional coordinates. To enable

213

meaningful analysis for calibration purposes, three-dimensional coordinate accuracies of approximately 25 μm are required. Prior to the development of UN ETIS, measurements had been performed using a static laser measurement method. However, this was only capable of obtaining accuracies of approximately 100 μm, which is of a similar order of accuracy to that of the robot. The Department of Production Engineering, therefore, commissioned a series of measurement surveys with UN ETIS.

This section defines the requirements of the calibration of production robots, outlines the principles of the production robot surveys and details the application of UN ETIS in three independent series of measurement surveys.

6.3.1 Calibration of Production Robots

Basically there are two modes of programming for a production robot; on-line and off-line. With on-line programming, the sequence of operations is stepped through directly, with the robot on the object. This method is also called "teaching in". With off-line programming, the user programs the robot motions by the use of object drawings, before the robot starts working on the object. This method is used with ever-increasing frequency, and it necessitates a detailed knowledge of the error characteristics of the robot.

Whereas special attention has to be paid to repeatability accuracy in the on-line mode, positional accuracy plays a decisive role in the off-line mode. This accuracy is limited, as the robot controller is

dependent upon an idealised model, which in reality cannot be achieved due to mechanical design limitations.

In practice, a robot is moved by a combination of six joints to a required position in space. The position of the end effector of the robot arm is defined by six parameters; the spatial position (x, y, z) and the orientation (a, b, c), which defines the pose of the end effector, at the required spatial position. In order for the robot to achieve a required pose, the robot controller must decide how to move each of the six joints in order to arrive at that pose. The most common method used by controllers is the Denavit Hartenburg technique [Foley 1988]. In this, six transformation matrices are produced, which transform each joint to the next joint, ie

$$(x, y, z, a, b, c) = A_1 A_2 A_3 A_4 A_5 A_6 \qquad \text{Equation 6.1}$$

where

A_n = transform matrix to go from joint n to joint (n+1).

= $fn(a_n, \alpha_n, d_n, \theta_n)$

where

n = 1 .. 6.

a_n = transformation constant.

α_n = transformation constant.

d_n = transformation constant.

θ_n = angle taken up by joint n.

By solving equation 6.1 for the six angles (θ_1, θ_2, θ_3, θ_4, θ_5, θ_6), the controller can then issue instructions to the robot to move the six joints to take up the required pose.

Obviously, if any of the constants (a_n, α_n, d_n) assumed by the controller are incorrect, then the robot will produce positional inaccuracy. The aim of the measurement surveys is to extract the actual constants (a_n, α_n, d_n) for a particular robot, and to compare these with the idealised constants, set in the robot controller. The positional accuracy of the robot can then be improved by adjusting the constant values within the robot controller, such that the required pose is achieve.

6.3.2 Principles of the Production Robot Surveys

In order to determine the actual constants (a_n, α_n, d_n) using UN ETIS it is necessary to determine the directions of the six axes of the robot and a point on each of the axes, relative to some base frame. This is performed by measuring points on the end effector as the robot describes a circle in each axis, by the movement of a single joint. Software developed in the Department of Production Engineering performs a least squares best fit on the described circle. From this least squares best fit circle, the direction of the axis can be determined as a perpendicular to the plane of the circle, and a point on the axis can be determined as the centre of the circle.

The circles are described by movement of the six joints in the order joint 6 - joint 5 - joint 4 - joint 3 - joint 2 - joint 1. Once measurements have started on a circle, the robot can only be moved by

216

a single joint until a set of circle measurements are complete. However, to enable the description of a specific circle by a specific joint, the robot can be moved to a suitable starting point, prior to circle measurement, by a combination of the joints which have been previously measured. For example, prior to the description of a circle by joint 4, the robot can be moved to a suitable starting point by a combination of joints 5 and 6.

In all measurement surveys, circle measurements are taken to targets mounted on an L-shaped attachment fixed to the end effector of the robot arm. The L-shaped attachment has two targets mounted on it, one on its' vertical component, the other on its' horizontal component. Five circle points were measured on each circle, eg circle 1

P1.1, P1.2, P1.3, P1.4, P1.5

, and six observations were taken to each circle point, to enable the calculation of point coordinates.

Up to the present time three independent series of measurement surveys have been performed, which are now detailed.

6.3.3 UN ETIS Measurement Survey RBT1

The first series of measurement surveys (RBT1) were performed in June 1988 on a Cincinnati T^3-726 robot. Two measurement surveys were performed.

6.3.3.1 Survey Description

The first measurement survey (RBT101) included the setting up of a local coordinate system, which was performed to two independent scale bar positions, and thirty circle points (six circles). The second measurement survey (RBT102) was performed in the same local coordinate system as measurement survey RBT101, and included sixty circle points (twelve circles).

In both surveys, the theodolites were mounted on wooden tripods, the local coordinate system was set up using a Wild 2 m subtense bar, and adhesive bullseye targets were mounted on the L-shaped extension arm. A theodolite baseline of 4.2 m was used, and the wooden tripods were considered stable and fixed throughout the two measurement surveys.

In measurement survey RBT101, measurements were taken to one of two adhesive bullseye targets, located on the L-shaped attachment. In measurement survey RBT102, it was decided to obtain twice as many target points as the previous survey. Therefore, two additional targets were placed approximately 1 cm from the existing targets, providing a total of four targets, and measurements were taken to one of two adhesive bullseye target pairs.

6.3.3.2 Results and Conclusions

In the first measurement survey (RBT101), the setting up of a local coordinate system, which included four scale bar points, was performed in three hours and the measurement survey, which included

thirty circle points (six circles), was performed in five hours. The second measurement survey (RBT102), which included sixty circle points (twelve circles), was performed in eight hours.

A guide to the general accuracy of the measurement surveys can be obtained from the mean analysis data given in Figures 6.1 and 6.2. The high mean angular deviations were due to two reasons; firstly, temporary index adjustments were not applied prior to the measurement survey, and, secondly, the inexperience of the operators in sighting ellipses formed by the adhesive bullseye targets. The mean ellipsoid of error value for measurement survey RBT101 (23.0 μm) was within the required accuracy (25 μm), whereas for measurement survey RBT102 (51.1 μm) it was not. However, the two surveys combined convinced the Department of Production Engineering that the system was capable of performing three-dimensional coordinate measurement to the required accuracy.

Due to the increased accuracy of measurement, and the production of a three-dimensional ellipsoid of error, as opposed to one mean error, the analysis software originally developed for the laser measurement method was not adequate. It was concluded that further analysis software had to be developed, to suit measurements by UN ETIS. In addition, a more detailed approach to the measurement pattern was to be developed by the Department of Production Engineering, for future surveys.

From a measurement survey point of view, there were obvious improvements that could be made, namely:

```
***** DATA FILE RBT101_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  *
Total No. of Points  :-  38

Mean Angular Deviations (secs)
     THEODOLITE 1              THEODOLITE 2
    H A         V A          H A         V A        SURVEY MEAN
    5.0         15.1         2.1         8.1            7.6


Mean Mis-Intersections (mm)
        Dx             Dy              Dz          SURVEY MEAN
      0.0006         0.0558          0.0181          0.0248


Mean Ellipsoid Errors (mm)
        Ex             Ey              Ez          SURVEY MEAN
      0.0386         0.0209          0.0095          0.0230
Mean Probable Errors (mm)
        Px             Py              Pz          SURVEY MEAN
      0.0041         0.0066          0.0107          0.0071
```

Figure 6.1 UN ETIS Measurement Survey RBT101 - Mean Analysis Data

```
***** DATA FILE RBT102_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  *
Total No. of Points  :-  60

Mean Angular Deviations (secs)
     THEODOLITE 1              THEODOLITE 2
    H A         V A          H A         V A        SURVEY MEAN
    4.5         15.3         1.9         7.9            7.4


Mean Mis-Intersections (mm)
        Dx             Dy              Dz          SURVEY MEAN
      0.0009         0.0625          0.0255          0.0296


Mean Ellipsoid Errors (mm)
        Ex             Ey              Ez          SURVEY MEAN
      0.0801         0.0527          0.0205          0.0511
Mean Probable Errors (mm)
        Px             Py              Pz          SURVEY MEAN
      0.0032         0.0040          0.0085          0.0052
```

Figure 6.2 UN ETIS Measurement Survey RBT102 - Mean Analysis Data

(i) The use of industrial measurement stands, to improve the stability of the theodolites and guarantee their fixed position throughout measurement.

(ii) The use of a carbon fibre scale bar, accurate to a few microns, to improve and guarantee the accuracy of coordinate system set up.

(iii) The use of spherical targets to enable more accurate sighting over a wider range of a circle.

(iv) A greater appreciation, by the operators, of the factors influencing the accuracies attainable with an ETIS, to enable a better coordinate system and measurement procedure to be employed.

6.3.4 UN ETIS Measurement Survey RBT2

The second series of measurement surveys (RBT2) were performed in May 1989, on the same Cincinnati T^3-726 robot as used in the first series of measurement surveys (RBT1). A set-up survey and three measurement surveys were performed.

6.3.4.1 Survey Description

Over the year following the initial series of measurement surveys (RBT1), a detailed analysis by the Department of Production Engineering led to the selection of a more rigorous measurement pattern. From theory, the worst robot positions, ie those liable to show the largest errors, were selected to describe the circles. This totalled nine circles, with some joints being used to describe more than one circle.

The set-up survey (RBT200) defined a local coordinate system, used in each of the subsequent measurement surveys. It was performed to two independent scale bar positions. The first and second measurement surveys (RBT201 and RBT202) included forty-five circle points (nine circles). The third measurement survey (RBT203) included forty-five circle points (nine circles), and fifty-four deformation points.

Plate 6.1 shows the measurement set up. In all surveys, the theodolites were mounted on industrial measurement stands, the local coordinate system was set up using a Kern carbon fibre scale bar (358457), and two 1 mm diameter spherical targets were mounted on the L-shaped extension arm. The x-axis of the local coordinate system was set up at ninety degrees to the x-axis used for the first series of measurement surveys (RBT1), and the theodolite baseline was extended from 4.2 m to 4.9 m, to improve the geometry of measurement. The industrial measurement stands were considered stable and fixed throughout the three measurement surveys.

The analysis theory, and the software developed up to this point, assumed that the robot was rigid in all joints, except the joint being moved. In measurement survey RBT202 dial gauges were placed on the "rigid" joints, to monitor their deformation during measurement. In measurement survey RBT203 the dial gauges were again used, but supplemented by UN ETIS monitoring of the deformation of the "rigid" joints throughout measurement. Three deformation points, 1 mm diameter spherical targets, were monitored as A, B and C on the "rigid" joints. Measurements were taken to these deformation points on the odd circle points, eg circle 1

Plate 6.1

UN ETIS Measurement Survey RBT2.

Pl.1A, Pl.1B, Pl.1C

Pl.3A, Pl.3B, Pl.3C

Pl.5A, Pl.5B, Pl.5C

, and four observations were taken to each deformation point, to enable the calculation of point coordinates.

6.3.4.2 Results and Conclusions

In the set-up survey (RBT200), setting up of a local coordinate system, which included four scale bar points, was performed in three hours. The first and second measurement surveys (RBT201 and RBT202), which included forty-five circle points (nine circles), were both performed in eight hours. The third measurement survey (RBT203), which included forty-five circle points (nine circles) and fifty-four deformation points, was performed in eleven hours.

A guide to the general accuracy of the measurement surveys can be obtained from the mean analysis data given in Figures 6.3 to 6.6 inclusive. The high accuracy of coordinate system set up is outlined by the mean ellipsoid of error (3.4 μm) for the four scale bar points used in the set-up survey RBT200. In the set-up survey, and all three measurement surveys, the mean angular deviations were constant at approximately ±2 seconds of arc.

The mean ellipsoid of error value for measurement survey RBT201 (56.6 μm) was not within the required accuracy (25 μm). In measurement survey RBT201 it was noted that on the geometrically "good" points, the ellipsoid of error was within the required accuracy.

```
***** DATA FILE RBT200_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  S???:LO
Total No. of Points  :-  4

Mean Angular Deviations (secs)
      THEODOLITE 1                  THEODOLITE 2
    H A         V A              H A         V A           SURVEY MEAN
    2.1         4.0              1.3         1.3               2.2

Mean Mis-Intersections (mm)
        Dx              Dy              Dz               SURVEY MEAN
       0.0000          0.0007          0.0060              0.0022

Mean Ellipsoid Errors (mm)
        Ex              Ey              Ez               SURVEY MEAN
       0.0033          0.0052          0.0016              0.0034
Mean Probable Errors (mm)
        Px              Py              Pz               SURVEY MEAN
       0.0048          0.0072          0.0128              0.0083
```

Figure 6.3 UN ETIS Measurement Survey RBT200 - Mean Analysis

Data

```
***** DATA FILE RBT201_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  *
Total No. of Points  :-  46

Mean Angular Deviations (secs)
      THEODOLITE 1                  THEODOLITE 2
    H A         V A              H A         V A           SURVEY MEAN
    1.3         3.0              3.1         1.9               2.4

Mean Mis-Intersections (mm)
        Dx              Dy              Dz               SURVEY MEAN
       0.0001          0.0316          0.1254              0.0523

Mean Ellipsoid Errors (mm)
        Ex              Ey              Ez               SURVEY MEAN
       0.0631          0.0759          0.0308              0.0566
Mean Probable Errors (mm)
        Px              Py              Pz               SURVEY MEAN
       0.0046          0.0049          0.0099              0.0065
```

Figure 6.4 UN ETIS Measurement Survey RBT201 - Mean Analysis

Data

```
                    ***** DATA FILE RBT202_R MEAN ANALYSIS DATA *****

Selected Point Name  :-   *
Total No. of Points  :-   45

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A         V A           H A         V A        SURVEY MEAN
    1.6         2.5           2.5         2.1            2.2

Mean Mis-Intersections (mm)
       Dx           Dy             Dz            SURVEY MEAN
      0.0000       0.0062         0.0487           0.0183

Mean Ellipsoid Errors (mm)
       Ex           Ey             Ez            SURVEY MEAN
      0.0201       0.0237         0.0114           0.0184
Mean Probable Errors (mm)
       Px           Py             Pz            SURVEY MEAN
      0.0044       0.0051         0.0100           0.0065
```

Figure 6.5 UN ETIS Measurement Survey RBT202 - Mean Analysis Data

```
                    ***** DATA FILE RBT203_R MEAN ANALYSIS DATA *****

Selected Point Name  :-   *
Total No. of Points  :-   99

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A         V A           H A         V A        SURVEY MEAN
    1.3         1.9           1.6         1.7            1.6

Mean Mis-Intersections (mm)
       Dx           Dy             Dz            SURVEY MEAN
      0.0000       0.0022         0.0215           0.0079

Mean Ellipsoid Errors (mm)
       Ex           Ey             Ez            SURVEY MEAN
      0.0136       0.0140         0.0058           0.0111
Mean Probable Errors (mm)
       Px           Py             Pz            SURVEY MEAN
      0.0042       0.0051         0.0100           0.0065
```

Figure 6.6 UN ETIS Measurement Survey RBT203 - Mean Analysis Data

However, it appeared that with vertical angles greater than ten degrees, the magnitude of the ellipsoid of error increased rapidly. In measurement survey RBT201, the best robot positions from an analysis point of view were measured, with little regard to the capability of UN ETIS at extreme vertical angles. A combination of "bad" geometry, and the inexperience of the operators in sighting partially defined spherical targets, led to a large ellipsoid of error on some points. It was evident from measurement survey RBT201, that measurements should always be taken as a compromise between the best position for the robot analysis, and a geometrically "good" measurement position for an ETIS.

In measurement survey RBT202, the mean ellipsoid of error value (18.4 μm) was within the required accuracy (25 μm). The dial gauges, used to monitor the stability of the robot, indicated some movement was taking place in joints assumed to be "rigid". This movement appeared to be due to the distribution of the robot's weight, and the fact that the base frame was not fixed to the floor.

In measurement survey RBT203, the mean ellipsoid of error value (11.1 μm) was within the required accuracy (25 μm). The dial gauges were again used, but supplemented by UN ETIS monitoring to three deformation points placed on the "rigid" joints of the robot. The monitoring of these deformation points throughout measurement underlined the fact that movement was taking place about the "rigid" joints, and quantified the deformation. From the results obtained in measurement survey RBT203, it was concluded that all future surveys should include measurement to a "fixed" base frame, and "rigid" joints. By using UN ETIS, as opposed to the dial gauges, the deformation points are in the same coordinate system, and are of similar accuracy, to

the circle points. Therefore, measurements to the "fixed" base frame and "rigid" joints enable deformation to be quantified, and corrections to be applied to the circle measurements, due to the "non-rigidity" of the robot.

6.3.5 UN ETIS Measurement Survey RBT3

After the second series of measurement surveys (RBT2) on the Cincinnati T^3-726 robot, a different robot was chosen for measurement. The third series of measurement surveys (RBT3) were performed in June 1989, on an Asea IRB1000 robot. A set-up survey and one measurement survey were performed.

6.3.5.1 Survey Description

As with the second series of measurement surveys, from theory the worst robot positions, ie those liable to show the largest errors, were selected to describe the circles. This totalled eight circles, with some joints being used to describe more than one circle.

The set-up survey (RBT300) defined a local coordinate system for the subsequent measurement survey. It was performed to four independent scale bar positions. The measurement survey (RBT301) included forty circle points (eight circles), and one hundred and twenty deformation points.

The measurement set up is pictured in Plate 6.2, which shows theodolite 1 and the robot. In both surveys the theodolites were mounted on industrial measurement stands, the local coordinate system

Plate 6.2

UN ETIS Measurement Survey RBT3.

was set up using a Kern carbon fibre scale bar (358457), and two 1 mm diameter spherical targets were mounted on the L-shaped extension arm. The coordinate system set up was restricted by the room housing the robot. Geometrically "good", theoretical, intersection angles were obtained with a theodolite baseline of 5.2 m. However, due to the nature of the robot its' movement space was located lower than the theodolites, which meant care had to be taken during measurement to exclude large negative vertical angles. The industrial measurement stands were considered stable and fixed throughout the measurement survey RBT301.

The Asea IRB1000 is a newer robot and, unlike the Cincinnati T^3-726, is permanently fixed to a concrete floor. These two factors combined led us to believe that the Asea IRB1000 would be more stable throughout measurement. It was decided that six deformation points, 1 mm diameter spherical targets, would be used to monitor the deformation of the robot throughout measurement. Three points were monitored as A, B and C on the "fixed" base frame, and three points were monitored as D, E and F on the "rigid" joints. Measurements were taken to the "fixed" base frame after every circle point, and to the "rigid" joints on the odd circle points eg circle 1

Pl.1A, Pl.1B, Pl.1C, Pl.1D, Pl.1E, Pl.1F

Pl.2A, Pl.2B, Pl.2C

Pl.3A, Pl.3B, Pl.3C, Pl.3D, Pl.3E, Pl.3F

Pl.4A, Pl.4B, Pl.4C

Pl.5A, Pl.5B, Pl.5C, Pl.5D, Pl.5E, Pl.5F

, and six observations were taken to each "fixed" base frame, and "rigid" joint point, to enable the calculation of point coordinates.

6.3.5.2 Results and Conclusions

The set-up survey (RBT300), which included eight scale bar points, was performed in three hours. The measurement survey (RBT301), which included forty circle points (eight circles), and one hundred and twenty deformation points, was performed over three days and took a total of twenty-five hours.

A guide to the general accuracy of the measurement surveys can be obtained from the mean analysis data given in Figures 6.7 and 6.8. The high accuracy of coordinate system set up is outlined by the mean ellipsoid of error (11.8 μm) for the eight scale bar points used in the set-up survey RBT300. In the set-up survey and the measurement survey, the mean angular deviations were constant, and less than ±2 seconds of arc.

The geometrically "good" measurement conditions encountered during measurement survey RBT301 are illustrated in Figure 6.9. From this, it can be seen that the intersection angle remains within a range of 80 - 105 degrees, and the two sight lengths remain within a range of 3.0 - 3.5 m. The mean ellipsoid of error value for the circle points in measurement survey RBT301 (16.2 μm) was within the required accuracy (25 μm). The error distribution for all circle points is shown graphically in Figure 6.10.

231

```
***** DATA FILE RBT300_R MEAN ANALYSIS DATA *****

Selected Point Name   :-  S???:LO
Total No. of Points   :-  8

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A          V A          H A          V A          SURVEY MEAN
    1.0          3.8          1.2          1.1             1.7


Mean Mis-Intersections (mm)
      Dx              Dy              Dz              SURVEY MEAN
    0.0000          0.0039          0.0154             0.0065


Mean Ellipsoid Errors (mm)
      Ex              Ey              Ez              SURVEY MEAN
    0.0124          0.0177          0.0055             0.0118
Mean Probable Errors (mm)
      Px              Py              Pz              SURVEY MEAN
    0.0041          0.0040          0.0097             0.0060
```

Figure 6.7 UN ETIS Measurement Survey RBT300 - Mean Analysis Data

```
***** DATA FILE RBT301_R MEAN ANALYSIS DATA *****

Selected Point Name   :-  P?.?:LO
Total No. of Points   :-  40

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A          V A          H A          V A          SURVEY MEAN
    0.9          2.0          1.7          1.9             1.6

Mean Mis-Intersections (mm)
      Dx              Dy              Dz              SURVEY MEAN
    0.0000          0.0080          0.0267             0.0115


Mean Ellipsoid Errors (mm)
      Ex              Ey              Ez              SURVEY MEAN
    0.0176          0.0213          0.0096             0.0162
Mean Probable Errors (mm)
      Px              Py              Pz              SURVEY MEAN
    0.0038          0.0039          0.0099             0.0059
```

Figure 6.8 UN ETIS Measurement Survey RBT301 - Mean Analysis Data of the Circle Points

232
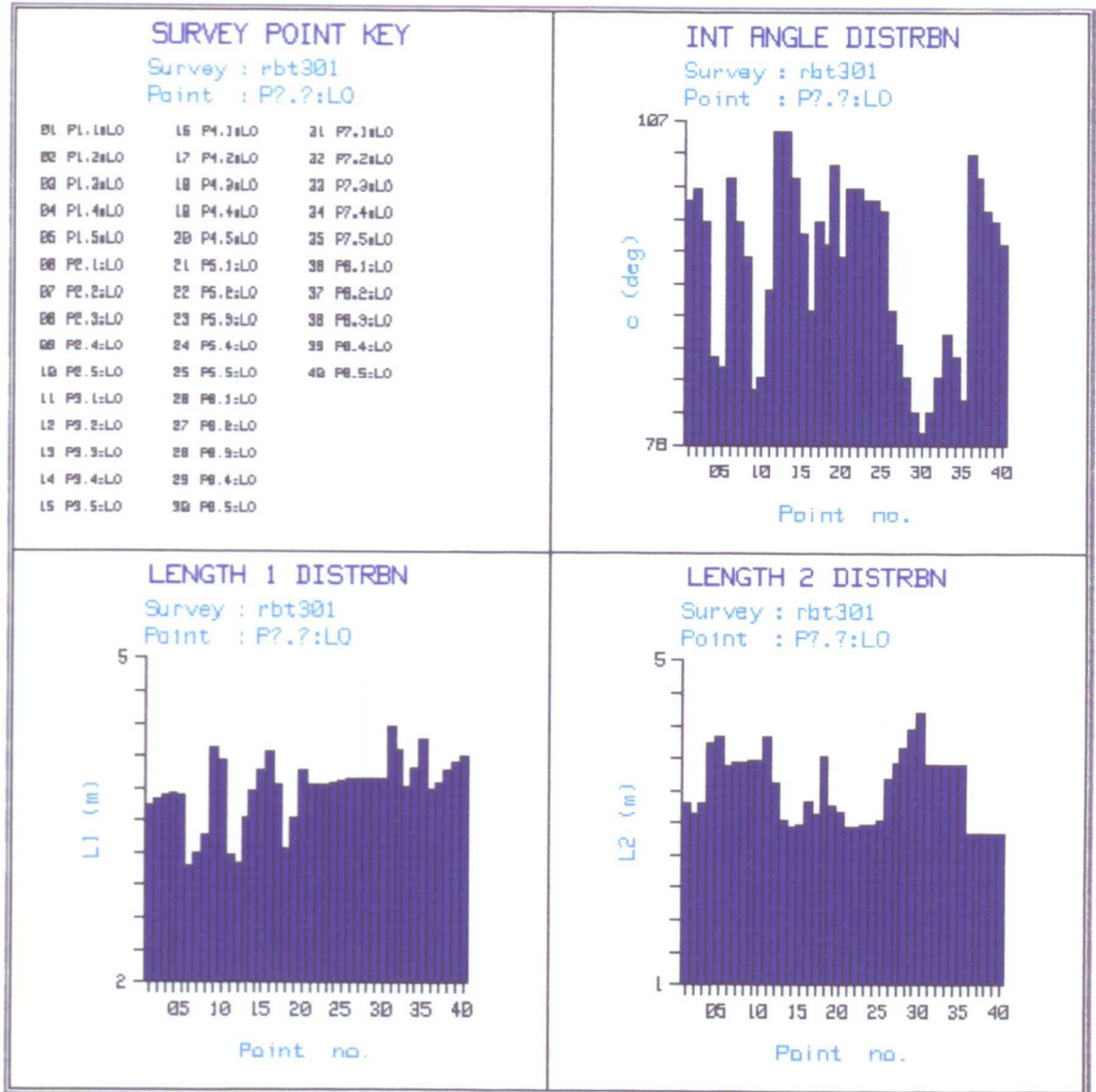
Figure 6.9 UN ETIS Measurement Survey RBT301 - Geometry Distribution of the Circle Points.
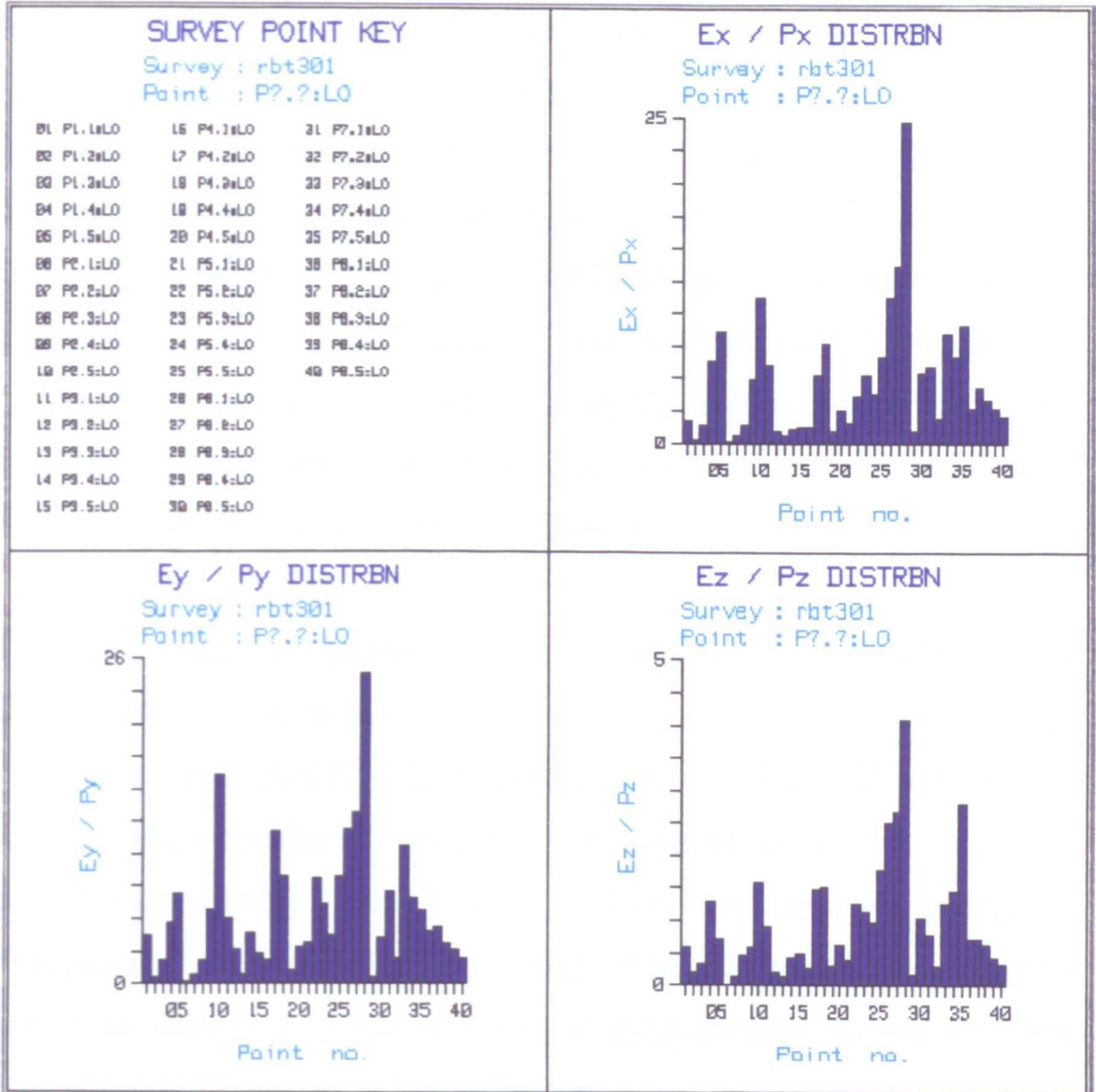
Figure 6.10 UN ETIS Measurement Survey RBT301 - Error
Distribution of the Circle Points.

As stated in section 6.3.4.2, measurements to the "fixed" base frame and "rigid" joints enabled deformation to be quantified, and corrections to be applied to the circle measurements, due to the "non-rigidity" of the robot. The deformation results for the six points (A, B, C, D, E and F) are best illustrated graphically as all axes movement. Deformation point A is shown overleaf (Figure 6.11); and the remaining deformation points are shown in Appendix G (Figures G.1 to G.5). These figures illustrate the magnitude of the movements at assumed "fixed" or "rigid" points, and underline the importance of monitoring deformation during the calibration of a production robot. In addition, these results illustrate the applicability of UN ETIS to deformation monitoring over short distances.

The full analysis of the calibration results, including the determination of the actual constants (a_n, α_n, d_n), is beyond the scope of this thesis. Full details will be given in [Foley 1991]. However, the accurate determination of the actual constants depends on the accuracy of the least squares best fit circle. Therefore, by determining the deviations of the measured circle points from the least squares best fit circle, an indication of the accuracy of measurement survey RBT301 for calibration purposes can be obtained.

The deviations of all the circle points from the least squares best fit circle are given in Table 6.1. From this table, it can be seen that the maximum mean deviation for a single circle, circle 7, is 121.4 μm. The mean of the deviations for all eight circles can be calculated to be 36.6 μm. Disregarding circle 7, the mean of the deviations for the other seven circles is improved to 24.4 μm.
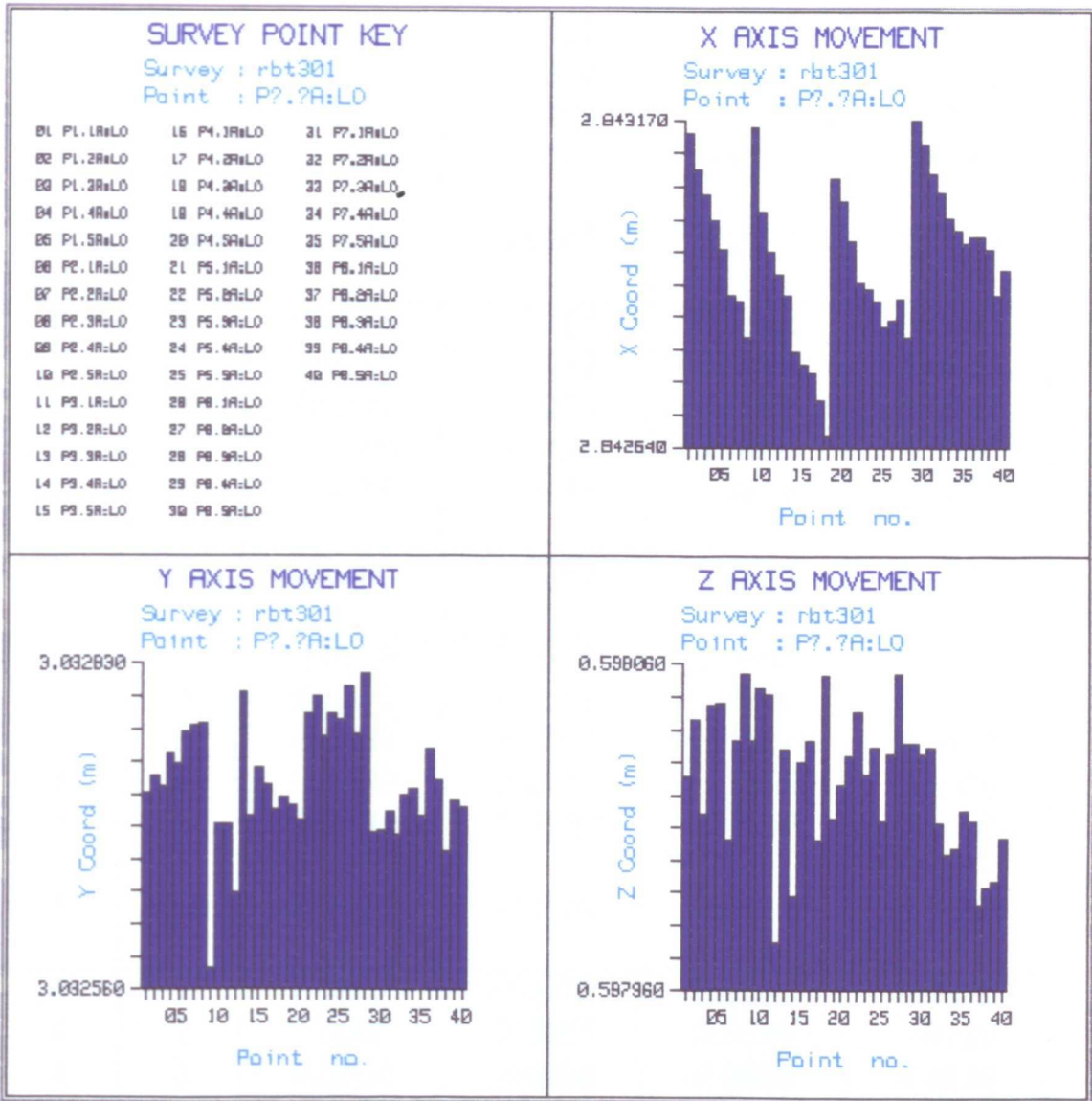
Figure 6.11 UN ETIS Measurement Survey RBT301 - All Axes
Movement of Deformation Point A.

Circle	Point	Deviations (mm)			
		dx	dy	dz	Mean
1	1	0.0021	0.0022	0.0001	0.0015
1	2	0.0021	0.0022	0.0001	0.0015
1	3	0.0163	0.0356	0.0217	0.0245
1	4	0.0053	0.0074	0.0028	0.0052
1	5	0.0106	0.0041	0.0059	0.0069
1	Mean	0.0073	0.0103	0.0061	0.0079
2	1	0.0114	0.0113	0.0004	0.0077
2	2	0.0114	0.0113	0.0004	0.0077
2	3	0.0192	0.0227	0.0303	0.0241
2	4	0.0309	0.0436	0.0994	0.0580
2	5	0.0201	0.0458	0.1670	0.0776
2	Mean	0.0186	0.0269	0.0595	0.0350
3	1	0.0018	0.0014	0.0948	0.0327
3	2	0.0018	0.0014	0.0948	0.0327
3	3	0.0439	0.1162	0.0983	0.0861
3	4	0.1422	0.1572	0.0055	0.1016
3	5	0.0886	0.0520	0.0849	0.0752
3	Mean	0.0557	0.0656	0.0757	0.0657
4	1	0.0007	0.0005	0.0349	0.0120
4	2	0.0007	0.0005	0.0349	0.0120
4	3	0.0020	0.0004	0.0605	0.0210
4	4	0.0116	0.0407	0.0386	0.0303
4	5	0.0113	0.0059	0.0262	0.0145
4	Mean	0.0053	0.0096	0.0390	0.0180

Table 6.1 UN ETIS Measurement Survey RBT301 - Deviations of the Circle Points from the Least Squares Best Fit Circle.

Circle	Point	Deviations (mm)			
		dx	dy	dz	Mean
5	1	0.0192	0.0094	0.0002	0.0096
5	2	0.0081	0.0067	0.0002	0.0050
5	3	0.0250	0.0019	0.0005	0.0091
5	4	0.0523	0.0008	0.0010	0.0180
5	5	0.0000	0.0000	0.0000	0.0000
5	Mean	0.0209	0.0038	0.0004	0.0084
6	1	0.0177	0.0171	0.0000	0.0116
6	2	0.0177	0.0171	0.0000	0.0116
6	3	0.0398	0.0306	0.0505	0.0403
6	4	0.0164	0.0168	0.0714	0.0349
6	5	0.0262	0.0350	0.0794	0.0469
6	Mean	0.0236	0.0233	0.0403	0.0290
7	1	0.1330	0.1376	0.0005	0.0904
7	2	0.1330	0.1376	0.0005	0.0904
7	3	0.2312	0.2496	0.1293	0.2034
7	4	0.1845	0.1807	0.0845	0.1499
7	5	0.0773	0.0208	0.1215	0.0732
7	Mean	0.1518	0.1453	0.0673	0.1214
8	1	0.0029	0.0030	0.0000	0.0020
8	2	0.0029	0.0030	0.0000	0.0020
8	3	0.0143	0.0089	0.0163	0.0132
8	4	0.0109	0.0065	0.0307	0.0160
8	5	0.0016	0.0028	0.0014	0.0019
8	Mean	0.0065	0.0048	0.0097	0.0070

Table 6.1 UN ETIS Measurement Survey RBT301 - Deviations of the Circle Points from the Least Squares Best Fit Circle (continued).

The ellipsoid of error values illustrate the "internal" accuracy of coordinate determination. An indication of the "real" accuracy of the system can only be obtained by comparing the determined coordinates with some standard, which in this case is the least squares best fit circle. From measurement survey RBT301 it can be concluded, therefore, that UN ETIS has not only achieved "internal", but also, "real" accuracies within the required accuracy of 25 μm.

6.4 Conclusions

There can be no doubt that ETISs will play an important role in future industrial measurement, particularly in applications such as the aerospace and automotive industries.

However, the application of ETISs to the calibration of production robots is not so certain. A full appreciation of the requirements of calibration, and the limitations of an ETIS is essential, if the high accuracies required are to be achieved, and if these high accuracies are to be useful in calibration.

In the production robot surveys, performed with UN ETIS, there were several stages in developing what may now be considered a practical solution to the problem. The recommendations of the production robot surveys are that:

(i) The selection of the peripheral components of an ETIS, eg industrial measurement stands, carbon fibre scale bar and spherical targets, should; guarantee the stability and fixed position of the theodolites throughout measurement, guarantee high accuracy of

coordinate system set up, and enable accurate sighting of target points over a wider range of a circle.

(ii) Repeated observations, up to a maximum of six, should be performed to target points, to achieve the highest accuracy in coordinate determination.

(iii) Measurements should always be taken as a compromise between the best position for the robot analysis, and a geometrically "good" ETIS measurement position.

(iv) Deformation measurements should be taken to a "fixed" base frame and "rigid" joints to enable deformation to be quantified, and corrections to be applied to the circle measurements, due to the "non-rigidity" of the robot.

The only disadvantage in using an ETIS for the calibration of production robots is the time involved in measurement. However, although there are instruments and techniques currently available which can perform the measurements faster, the accuracies obtained by an ETIS cannot be rivalled.

CHAPTER SEVEN

FIELD EVALUATION OF UN ETIS FOR THE
MONITORING OF DEFORMATION DUE TO MINING

Field Evaluation of UN ETIS for the Monitoring of Deformation due to Mining

7.1 Introduction

Conventionally, the monitoring of deformation due to mining has involved the measurement of absolute ground movement - subsidence monitoring. Subsidence monitoring involves measuring the movement of ground survey points with respect to stable reference points, situated outside of the zone of influence. Measurements are usually performed over long distances, up to 1 km, and accuracies of the order of millimetres can be achieved. Subsidence monitoring was originally performed to provide the subsidence engineer with data, from which theories on the prediction of ground movement were formulated, and is currently performed to enable comparisons between actual and predicted subsidence values.

The increased possibility of mining affecting important structures or sensitive industrial processes, and the need to monitor the deformation of underground openings, necessitates deformation monitoring to a new, higher order of accuracy. The accuracy required is usually sub-millimetre and, in the case of sensitive industrial processes, can be of the order of tens of microns, which is obviously not attainable by conventional monitoring techniques. However, in most cases there is only need to monitor relative, not absolute, deformation as mining takes place. This relative deformation monitoring can be performed over short distances, up to 100 m, and the high accuracies required can be achieved by:

242

(i) The adaptation of a conventional measurement technique to relative, rather than absolute, monitoring.

(ii) The application of a new measurement technique, ie UN ETIS.

Initially, this Chapter considers conventional surveying instrumentation and techniques. Details are given of their use in long-range absolute deformation monitoring, and their applicability to short-range relative deformation monitoring is considered. The Chapter then concentrates on the relative monitoring of the deformation of a sensitive industrial process due to mining. With reference to a specific example, the glass grinding and polishing process at Pilkington Glass Ltd in Doncaster, details are given of the field evaluation of UN ETIS for short-range relative deformation monitoring and a comparison is made between UN ETIS and an adapted conventional technique.

7.2 Conventional Surveying Instrumentation and Techniques for the Monitoring of Deformation due to Mining

The monitoring of absolute ground movement involves measuring changes in the level of the ground, and the strain on the ground, which is the main cause of structural damage. These two quantities can either be measured independently, or derived from the three-dimensional coordinates of ground survey points.

Changes in the level of the ground can be determined directly, using precise levelling, or by measuring the three-dimensional coordinates of ground survey points, using an electronic total station or the global positioning system. Ground strain can be determined by

monitoring the deformation of a line of ground survey points situated across the zone of influence. It can be calculated independently by measuring changes in the distances between line points, using an electronic distance measurer, or by measuring the three-dimensional coordinates of line points, using an electronic total station or the global positioning system.

This section details four instruments / techniques used for long-range absolute deformation monitoring, namely; precise levelling, electronic distance measurers, electronic total stations, and the global positioning system, and considers their applicability to short-range relative deformation monitoring.

7.2.1 Precise Levelling

Levelling is concerned with the measurement of difference in height between points, or with the determination of the elevation of certain points above a datum. The principle of levelling is to establish the line of collimation of the instrument, from which the level of points can be determined using a precise scale or levelling staff.

Precise levelling does not differ greatly from ordinary levelling, except that the level is usually fitted with a parallel-plate micrometer and the scale, or staff, is made of invar. At present, precise levelling is one of the main methods of subsidence measurement, and most surveys are performed using automatic levels. These contain a compensator which corrects for any slight tilt in the collimation axis, enabling faster and more accurate observations to be performed.

The typical accuracy of a precise level is ±0.3 mm over a 1 km double levelling run. In addition to conventional levelling over long distances, precise levels are used in industrial measurement, as detailed in Chapter Six, which underlines their applicability to short-range relative deformation monitoring (Figure 7.1(i)). However, the obvious disadvantage of precise levelling, when compared to an ETIS, is that it will only measure level, as oppose to three-dimensional coordinates.

7.2.2 Electronic Distance Measurers

Since their introduction, in the early 1970s, electronic distance measurers (EDMs) have developed into an accepted and essential part of a surveyors equipment. Most leading surveying equipment manufacturers now produce a range of EDMs to suit a wide variety of survey tasks. However, most derive distances by measuring either the time delay between pulses of electromagnetic energy; laser-based instruments, or by comparing the transmitted and received signals and deriving the phase difference.

EDMs have found great application in long-range distance measurement, such as absolute deformation monitoring, as they are capable of measuring distances up to a few kilometres, to accuracies of the order of millimetres, at the press of a button. However, their application in short-range relative deformation monitoring is limited by their relatively low accuracy over short distances and the restrictive use of a prism as a target.

A recent development in EDM technology has been the introduction of instruments of sub-millimetre accuracy, eg the Kern
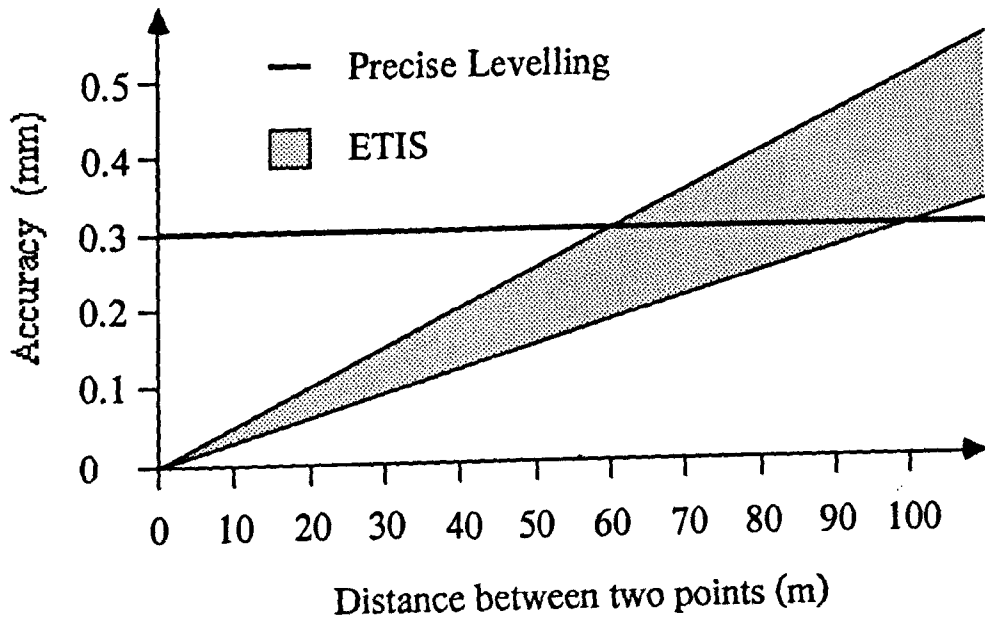
(i) Theoretical Accuracy of the level difference between two points determined by precise levelling and ETIS.
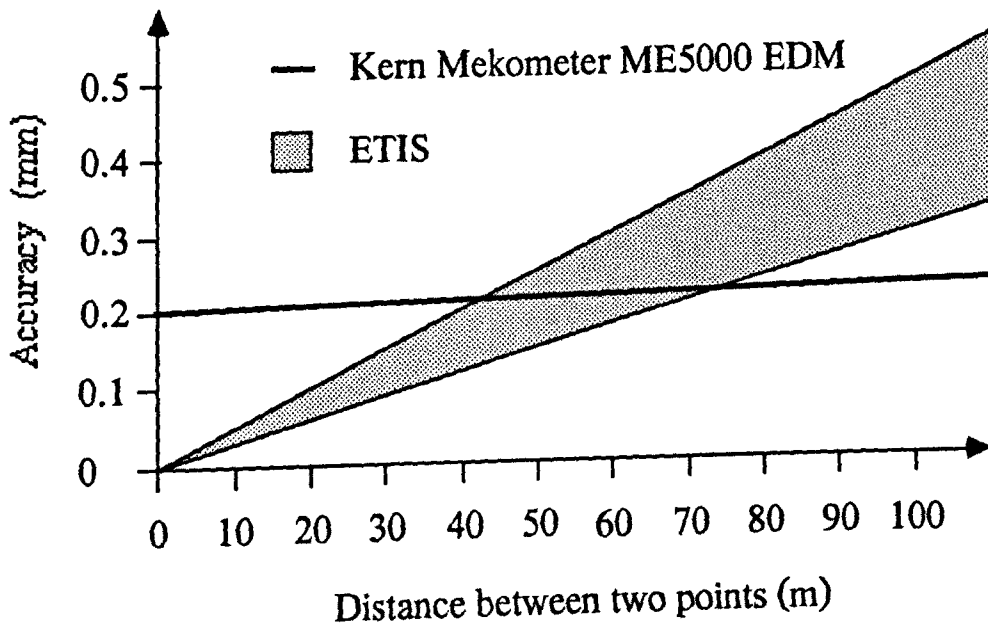
(ii) Theoretical Accuracy of the three-dimensional distance between two points determined by the Kern Mekometer ME5000 EDM and ETIS.

Figure 7.1 Theoretical Accuracies of Precise Levelling, Kern Mekometer ME5000 EDM and ETIS in Short-Range Relative Deformation Monitoring.

246

Mekometer ME 5000. The Mekometer uses a helium-neon laser light source and has an accuracy of $\pm(0.2$ mm + 0.2 ppm), over a range of 20 m to 8 km. In addition, it can be linked to an external computer enabling the measuring range to be extended below 20 m. The high accuracy of the Mekometer makes it applicable in short-range relative deformation monitoring (Figure 7.1(ii)), in situations where range or geometry restrict the use of an ETIS, with its only disadvantage being that it still requires a prism as a target.

Another recent development in EDM technology is the measurement of short distances without a prism. An example of an EDM capable of measurement without a prism is the Wild DISTOMAT DIOR3002, which can measure distances up to 200 m to an accuracy of 5 to 10 mm. This instrument has applications in the accurate determination of cross-sections, such as underground openings, and the monitoring of inaccessible structures from a safe distance. However, the relatively low accuracy of the instrument limits its application in short-range relative deformation monitoring.

7.2.3 Electronic Total Stations

Combining an EDM and an electronic theodolite produces an instrument capable of measuring all of the information required to determine three-dimensional coordinates, from a single instrument set up. The resulting digital electronic output allows the data to be recorded on a data recorder, from where it can later be downloaded to a computer. The combination of an EDM, electronic theodolite and data recorder is known as an electronic total station (ETS). Previous research within the Department underlined the applicability of ETSs to

247

long-range absolute deformation monitoring in mining situations [Hodges & Cooper 1988].

Automated electronic theodolites have recently been introduced by the Kern, Wild and Aga companies. Their application in automated ETISs is detailed in Chapter Four. In addition, these instruments can also be linked with an EDM to form an Automated ETS. In both cases, these automated instruments are capable of automatically and permanently monitoring coordinate changes due to deformation. An example of an automated ETS is the Geodimeter 140 T (autotracking total station) developed by the Aga company, which is capable of automatically following a moving target at speeds of up to 30 km/h, and can be manually controlled with a joystick. The Geodimeter 140 T forms the basis of the Geodimeter 140 SMS (Slope Monitoring System) which was tested at the Aitik copper mine in Sweden. The instrument, which is set up in a permanent stable building and linked to an office computer system by a telemetry interface, follows a preset measuring sequence of a number of points, calculating the coordinates, and monitoring small displacements as early signs of slope failure. If movement is greater than a preset limit then an alarm is set off, allowing security measures to be carried out before the predicted slope failure occurs.

As ETSs incorporate an EDM, their application in short-range relative deformation monitoring is limited by the relatively low accuracy of the EDM over short distances, and the restrictive use of a prism as a target. In addition, although sub-millimetre accuracy EDMs are available these are not theodolite mounted and, hence, can't be used as part of an ETS.

7.2.4 The Global Positioning System

The global positioning system (GPS), introduced in the USA in 1983, is a satellite-based radio navigation system which will provide worldwide three-dimensional coordinates. It is based on simultaneous range measurement to several satellites, which can provide instantaneous navigational accuracy positions The GPS outputs the latitude and longitude of the receivers, and can be linked to a computer used to store the measured data and calculate high precision survey measurements [Ffoulkes-Jones 1987].

By 1993 the full configuration of 21 GPS satellites should be in orbit, so that the required minimum of four satellites will nearly always be in view from any point on Earth. Each of these satellites transmits two code signals using two different frequencies, which contain satellite identification and positional information, as well as providing a means of measurement. There are two measurement modes; the pseudo-range mode and the interferometric phase measurement mode. The pseudo-range mode requires simultaneous information from four satellites to calculate the position of a single point using the phase shift required for each received signal to match a locally generated replica code signal. The interferometric phase measurement mode uses a principle similar to EDM, measuring the phase difference between the received signal and a locally generated signal. The latter mode requires the use of two or more receivers and gives their relative positions, ie bearing, distance and height difference. Over long distances, of the order of several kilometres, relative accuracies of 1 ppm have been achieved, with many authors predicting increased accuracies of 0.01 ppm in the future [Cross 1988].

249

Due to its' high accuracy, simplicity and relatively low cost, it is almost certain that GPS will revolutionise engineering surveying in the near future, as a portable model would effectively remove the need for equipment such as theodolites, levels and EDMs. Regular, high accuracy, observations obtained by GPS may be used in long-range deformation monitoring, in applications such as slope monitoring and subsidence surveys. However, the use of GPS in short-range relative deformation monitoring is limited, due to difficulties in achieving sub-millimetre accuracies over distances below 1 km, and the systems dependence on satellites. For instance, a GPS receiver requires a good all round visibility of the sky, 15 - 20 degrees above the horizon. This is impossible in many environments, and obviously restricts the use of GPS close to tall structures, underground, in tunnels or inside buildings.

7.3 Monitoring of the Deformation of Sensitive Industrial Processes due to Mining

The increased sensitivity to which certain industrial processes are set up, combined with the possibility of mining affecting these processes, requires deformation monitoring to a higher accuracy than has previously been necessary. The accuracy required can be of the order of tens of microns, which is obviously not attainable by conventional monitoring techniques. However, in most cases the industrial process is set up relative to an assumed fixed point on, or close to, the process, and checking of the set up is performed periodically from the assumed fixed point. There is, therefore, in many circumstances, a need to monitor the relative, and not absolute, deformation of the process as mining takes place.

250

In late 1988 the Department began monitoring the deformation of a sensitive industrial process, the glass grinding and polishing process at Pilkington Glass Ltd in Doncaster, due to underground mining operations. Short-range relative deformation monitoring was performed prior to, and during the period of mining activity, and it is intended to be continued after mining has ceased. It was performed by two independent techniques:

(i) Refined precise levelling to specially designed scale targets.

(ii) The application of UN ETIS.

The refined precise levelling technique has been used to monitor the relative deformation along the whole process line. UN ETIS has been used, in two independent measurement surveys, to monitor a small part of the process line.

Initially, this section details the glass grinding and polishing process, and the mining activity which may affect this sensitive industrial process. The section then discusses the two techniques used for short-range relative deformation monitoring, together with the results obtained up to the present time.

7.3.1 Pilkington Glass Grinding and Polishing Process

Pilkington Glass Ltd in the UK produces reinforced glass for export throughout the world. The manufactured glass is ground and polished in the No 9 G & P Workshop, at Doncaster, by a sensitive industrial process.

The process consists of a series of steel tables which run on two parallel mounting slides. A single glass sheet is transported on a steel table, and passes along the process line twice, with opposite sides being ground and polished on each pass. A steel table ascends at the start of the process line and is loaded with a sheet of glass. At the end of the process line the glass is removed from the steel table, which then descends and returns to the start, by travelling underneath the process line. In this way, the steel tables form a moving bed along the length of the process line, and grinding and polishing of the glass is a continuous process.

Grinding is the most critical stage of the process. Manufactured reinforced glass is rough and not particularly transparent. This rough glass has to be ground on both sides, to a uniform thickness. For grinding of the glass to the required tolerance, the level of the tables on the process line is highly critical. If this level is not maintained, the glass is not uniformly ground, and quality will vary across the width of the sheet.

After a sheet of glass has been ground to the required thickness it is polished further along the process line. Polishing is performed using a rouge mixture. The level of the tables in the polishing stage is not as critical as for the grinding stage, however, unpolished sections of glass may arise from mislevelment greater than the accepted tolerance.

The process line is approximately 260 m in length, 3.5 m in width and is supported along its' length by thirty-six pairs of concrete pillars (East & West) set at 7.5 m intervals. Monitoring of the

deformation of the process is performed by monitoring the pairs of supporting pillars, which are located in a tunnel below the process line. Pillar pair 1 is at the start of the process line, where the tables ascend, and pillar pair 36 is at the end of the process line, where the tables descend. However, monitoring can only take place on pillar pairs 9 to 33, as it is beyond these pillars that the tables are raised and lowered.

The process is continuous and is only stopped once a year to allow for re-levelling of the process line, which requires the process line to be dismantled, and takes approximately two weeks. Re-levelling is performed using a straight edge and precise industrial / spirit level, by relatively levelling successive sets of four points, situated above four pillars, along the process line to an accuracy of 50 μm. This method gives no indication of the amount of mislevelment that has occurred over the previous year, however, the process has never had to be re-levelled at any intermediate stage in the past.

7.3.2 Mining Activity

Bentley Colliery, in the South Yorkshire Area of British Coal, is mining coal close to the No 9 G & P Workshop, in the Barnsley Seam. The coal face, shown in Figure 7.2, began production in June 1989. It is phased to advance at a rate of 65 m/month, and end in approximately April 1991. The mining characteristics of the proposed coal face are:

Seam Thickness = 2.3 m
Depth (h) = 590 m (Start)
535 m (End)
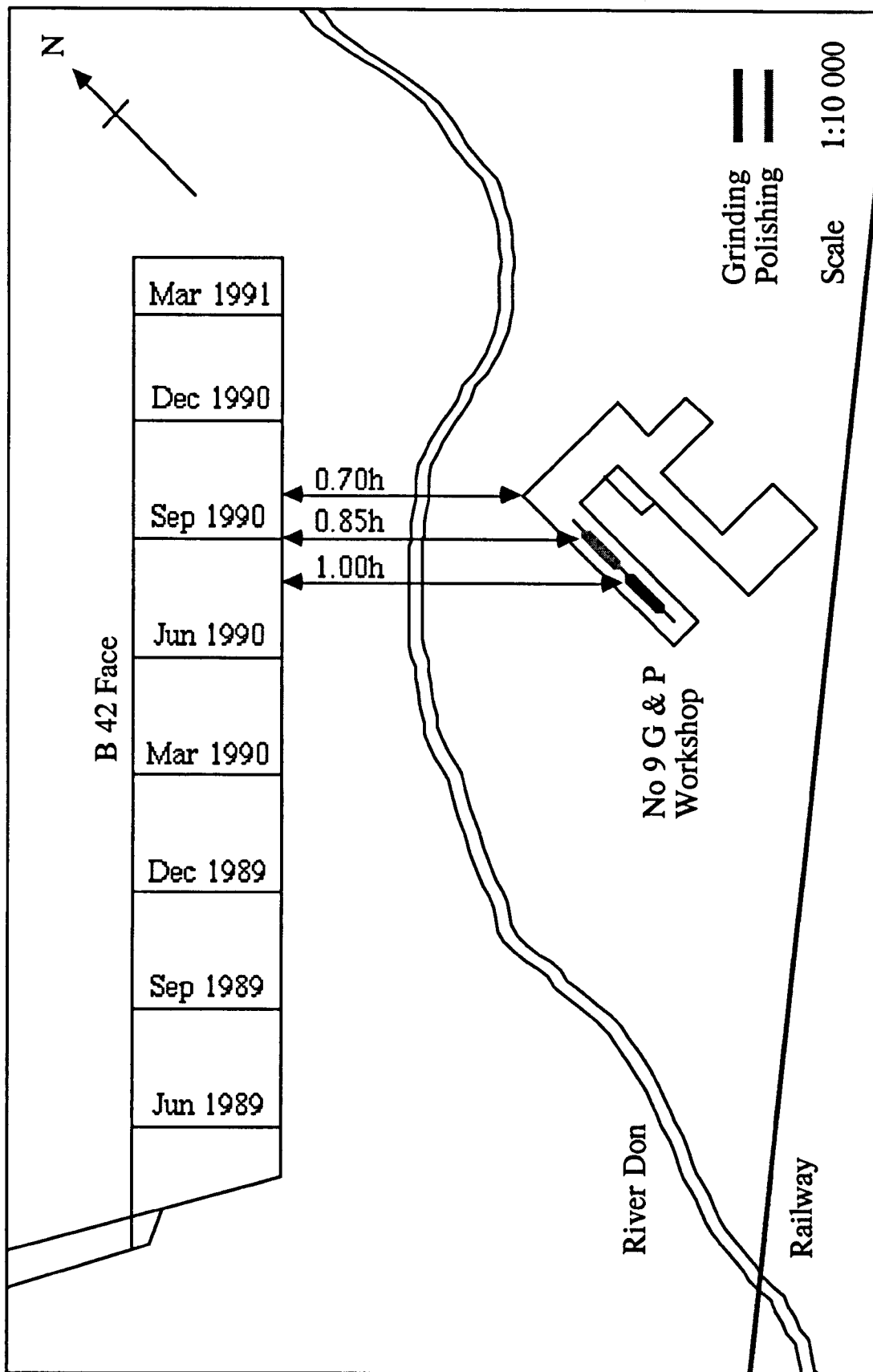
Figure 7.2 Pilkington Survey Mining Activity.

254

Face Width   = 240 m (1150 m advance)

                    230 m (220 m advance)

From Figure 7.2 it can be seen that the coal face lies to the north-west of the works, closest to the polishing section of the process. Initially the planned coal face was at a distance of 390 m, ie 0.70h, from the polishing section, and 0.80h from the grinding section. However, the planned coal face was then rotated so it is now at a distance of 0.85h from the polishing section, and 1.00h from the grinding section.

If current subsidence theory is correct, the process should not be affected by mining activity, however, due to its' sensitive nature and the extreme costs involved if it should be, the process was monitored for deformation.

7.3.3 Pilkington Precise Levelling Survey

To provide a practical solution to monitoring the deformation along the whole process line, to a high accuracy, the Department designed a refined precise levelling technique for monitoring short-range relative deformation. The monitoring program was performed in four phases:

(i) The establishment of a network of precise levelling stations to determine the relative vertical movement of supporting pillars in the No 9 G & P Workshop to an accuracy of ±0.01 cm (±100 µm). The specially designed scale targets were supplied, and wall plugs inserted, by the Survey & General Instrument Co Ltd. (Completed by January 1989).

(ii) Precise levelling of the pillar stations on five separate occasions before underground mining operations commenced. (Completed by April 1989).

(iii) Precise levelling of the pillar stations, at intervals of approximately ten weeks (five times / year), over a two year period as underground mining operations take place. (To be completed by April 1991).

(iv) Further precise levelling of the pillar stations, after underground mining operations have ceased, to be performed as and when considered necessary by the Department and Pilkington Glass Ltd.

7.3.3.1 Survey Description

The precise levelling surveys are performed using a Wild NA2 automatic level, and four specially designed scale targets. Plate 7.1 shows an observation being performed to a scale target mounted in pillar station 9W.

Each precise levelling survey is performed using pillar station 9W as a reference point, assumed to have a level of 10.000 cm. The survey is performed as a series of closed levelling networks, with each network starting from the previous west station, and observations being taken in an anti-clockwise direction.

Considering Figure 7.3, for the first closed network, the instrument is located at pillar pair 10 and the order of observations is 9W, 9E, 11E and 11W. Similarly, for the second closed network, the instrument is located at pillar pair 12 and the order of observations is

Plate 7.1

Pilkington Precise Levelling Survey.

Left panel (i):

Tables Descend

36

N

30

Polishing Section

25

20

Grinding Section

15

→ (ii)

10

Tables Ascend

5

1

Scale 1:1500

(i) Process Line Supporting Pillars.

Right panel (ii):

N

13W ☐   ☐ 13E

☐   ▲ ☐ Wild NA2 2nd Set Up

11W ☐   ☐ 11E

☐   ▲ ☐ Wild NA2 1st Set Up

9W ☐   ☐ 9E

Scale 1:200

(ii) Measurement Set Up.
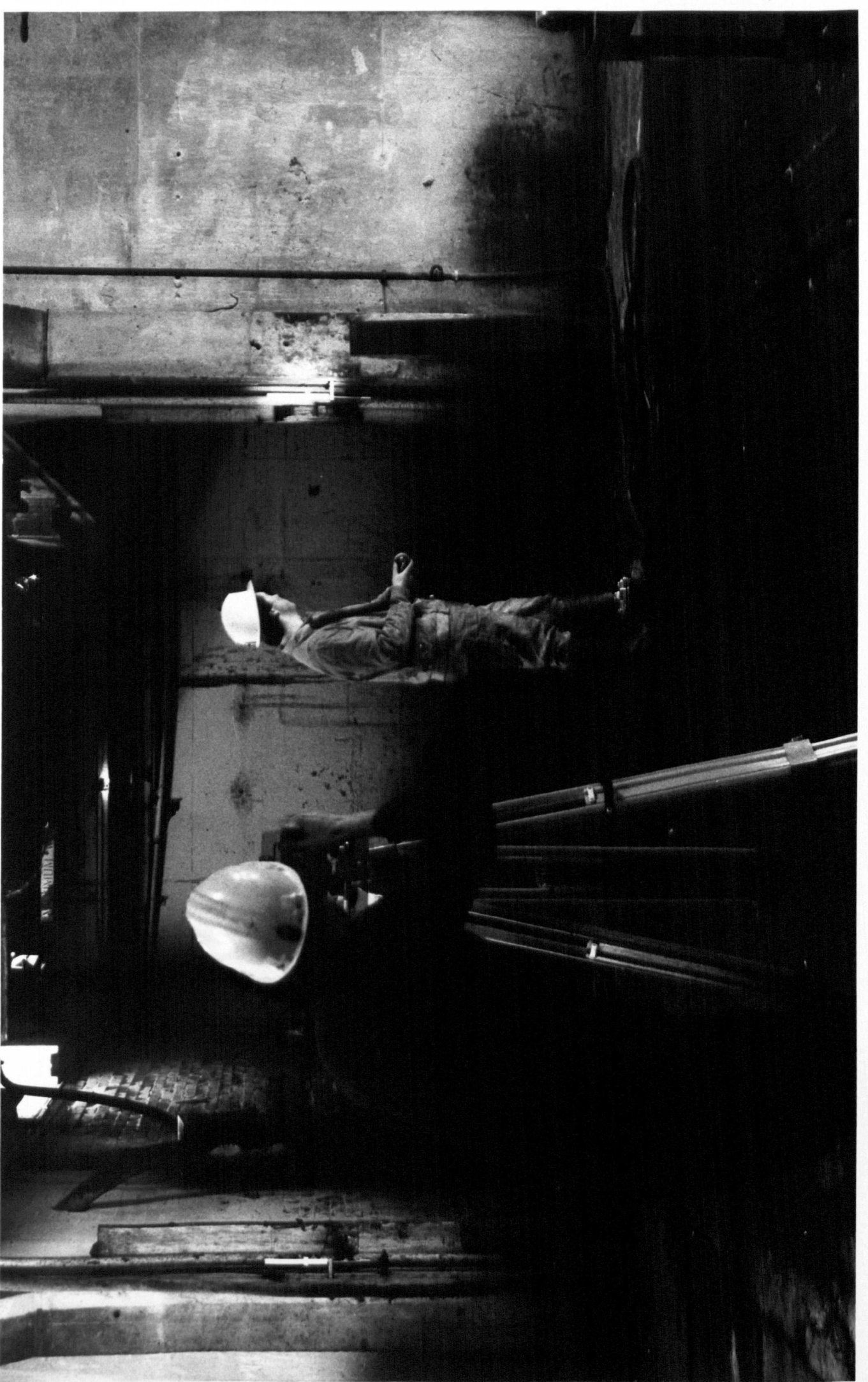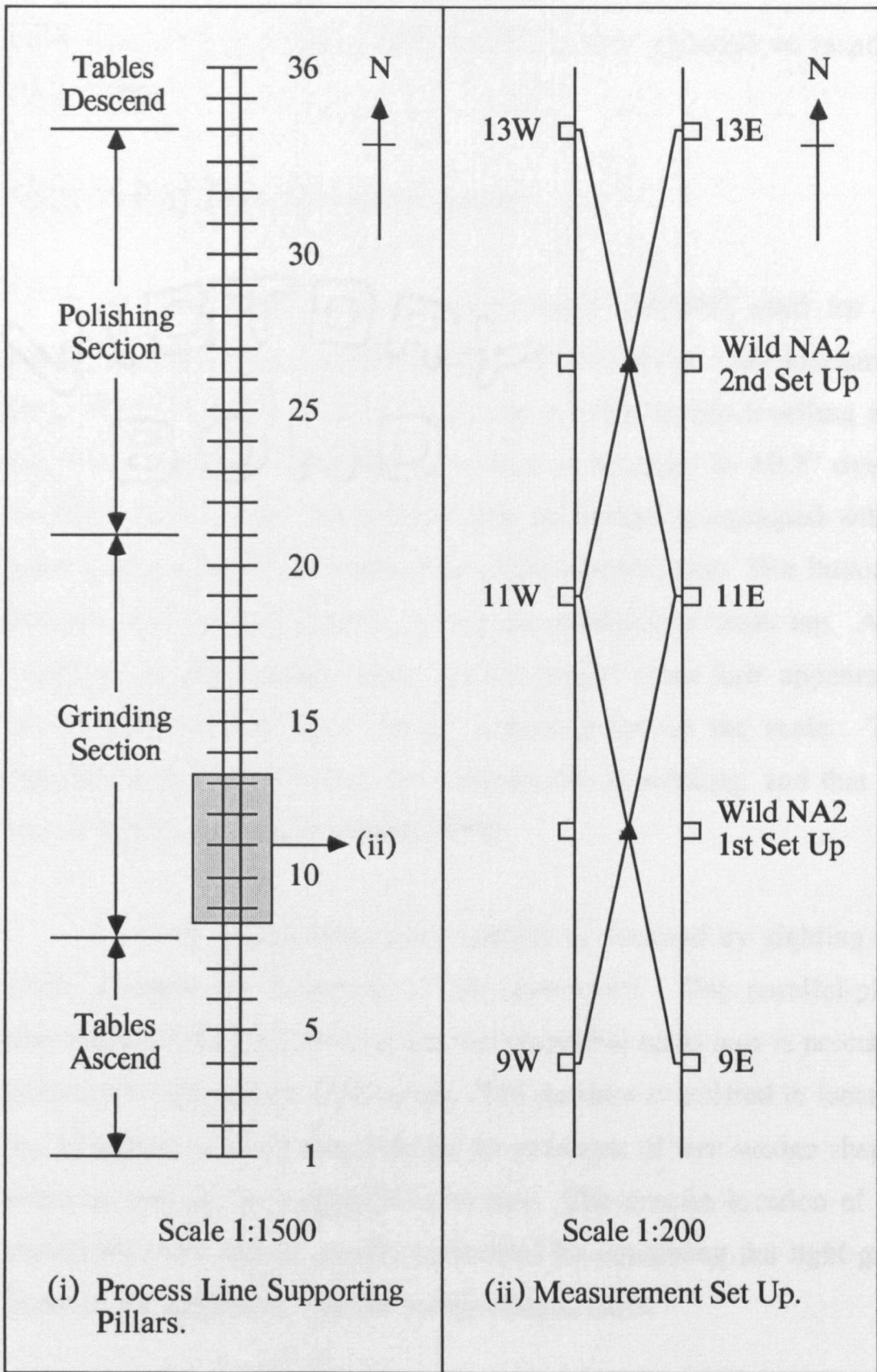
Figure 7.3 Pilkington Precise Levelling Survey - Measurement Set Up.

11W, 11E, 13E and 13W. This process is then repeated up to pillar station 33W.

7.3.3.2 Wild NA2 Automatic Level

The Wild NA2 automatic level (396001) used for the surveys is fitted with a GMP3 parallel-plate (22880). The instrument has a claimed accuracy of ±0.3 mm over a 1 km double levelling run, and has a pendulum compensator which is accurate to ±0.3" over a working range of 30'. In addition, the instrument is equipped with a press button to check the functioning of the compensator. This button is pressed prior to each reading, giving the pendulum a slight tap. As a result of the pendulums swing, the horizontal cross hair appears to "move off and float back" to its original point on the scale. The observer is then certain that the compensator is working, and that the line of sight is horizontal [Denby 1979].

An approximate scale reading is obtained by sighting the scale, through the telescope of the instrument. The parallel-plate micrometer is then adjusted so that the horizontal cross hair is precisely located on a graduation of the scale. The operator is assisted in locating the cross hair on the graduation by the presence of two wedge-shaped hairs, as well as the horizontal cross hair. The precise location of the horizontal cross hair is usually performed by equalising the light gaps between the graduation and the wedge-shaped hairs.

Once the cross hair is precisely located on a graduation, the reading is obtained in centimetres. The centimetre value is read direct

from the scale and the precise value is obtained from the micrometer, which enables a direct reading to 0.01 cm, and estimation to 0.001 cm.

7.3.3.3 Pilkington Scale Target Description

The Pilkington Scale Target was designed and manufactured for the specific application. The target consists of three components:

.

(i)   Permanent wall plug.

(ii) Wall bracket.

(iii) Wild 300 mm invar scale.

The permanent wall plug (Figure 7.4(i)) is made of stainless steel and is grouted into position in the pillar. To protect the plug between surveys a cap is fitted and the screw threads are kept well lubricated. The wall bracket (Figure 7.4(ii)) is made of stainless steel and is screwed into the permanent wall plug by an 8 mm diameter bolt. Before sighting to a pillar, the wall bracket is screwed into the permanent wall plug situated in that pillar, and levelled by means of the locking nut and permanent spot bubble. The Wild 300 mm invar scale is mounted in a 5 mm slot on a 16 mm radius brass hemisphere, and fixed into position by a grub screw. Prior to sighting, the scale is mounted on the wall bracket, and is free to move by means of the cup and ball arrangement (Figure 7.5). Before measurements take place the scale is levelled using the spot bubble.

Four scales and four brackets were manufactured. These have been numbered and are used in a fixed combination ie scale 1 is

Grout

Wall

Drilled Hole

Wall Plug

(i) Permanent Wall Plug.

Location Spigot

Locking Nut

Bracket Spot Bubble
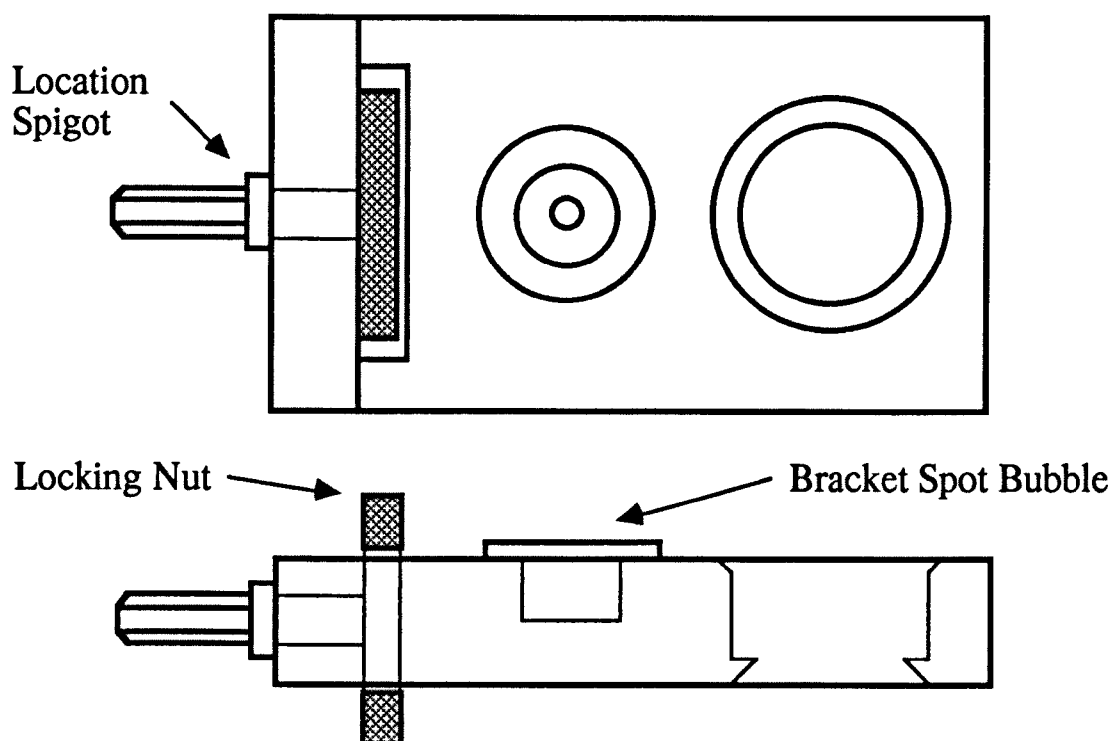
(ii) Wall Bracket.

Figure 7.4 Pilkington Precise Levelling Survey - Permanent Wall Plug and Wall Bracket.
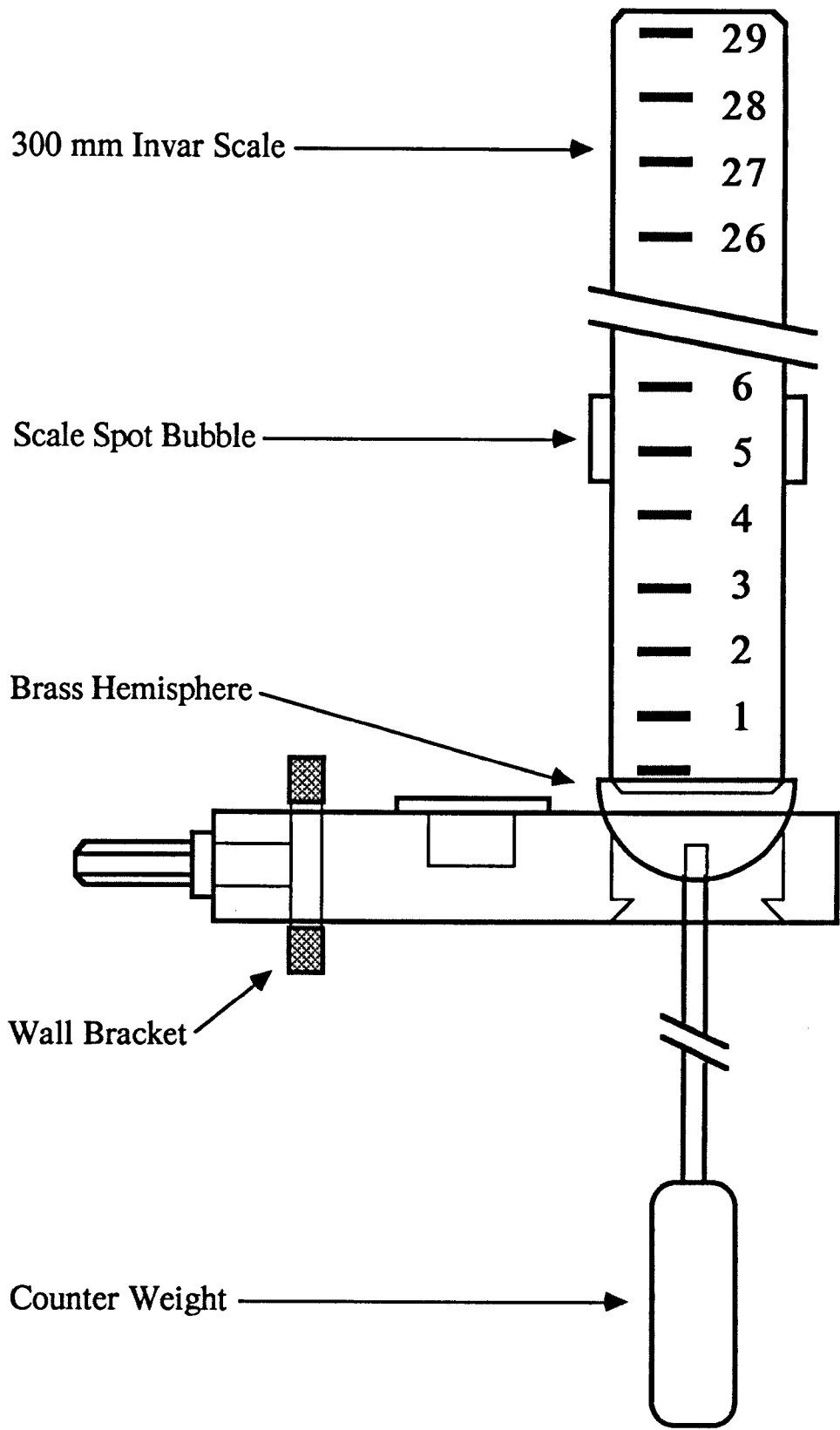
Figure 7.5 Pilkington Precise Levelling Survey - Wild 300 mm Invar
Scale Mounted on the Wall Bracket.

used with bracket 1. In addition, the same combination of scale and bracket are used in the same pillar station for each survey. However, in case it is ever necessary to use a different combination of scale and bracket in a certain pillar station, all possible scale and bracket combinations have been calibrated against the fixed combinations.

7.3.3.4 Results and Conclusions

Up to the present time ten surveys have been performed. Surveys 1 - 5 were performed before underground mining operations commenced and surveys 6 - 10 have been performed as underground mining operations progress. Each survey was performed in approximately two and a half hours and the results of the surveys are given in Tables 7.1 to 7.6 inclusive, where all values are given in centimetres.

Table 7.1 contains the pillar station (PS) levels for surveys 1 - 5, the means of which are taken as the accepted PS levels. Table 7.2 contains the PS levels for surveys 6 - 10, and table 7.3 contains the deviations of these PS levels from the accepted values.

The standard deviations of the accepted PS levels are all below ±0.02 cm, and the standard deviations of the mean PS levels for surveys 6 - 10 are all below ±0.02 cm, with the exception of two values. Comparing the mean PS levels for surveys 6 - 10 with the accepted PS levels, all agree to within ±0.02 cm. In addition, from Table 7.3, it can be seen that the deviations of the PS levels, established in surveys 6 - 10, from the accepted values are all within ±0.03 cm, with the exception of three values in survey number 7, and one value in survey number 10.

Station	Survey Number/Date					Mean	SD	Station
	1 14.12.88	2 11.01.89	3 01.02.89	4 01.03.89	5 29.03.89	(1-5)	(1-5)	
9W	10.00	10.00	10.00	10.00	10.00	10.00	0.00	9W
9E	9.79	9.78	9.79	9.78	9.79	9.79	0.01	9E
11W	9.85	9.84	9.86	9.86	9.86	9.85	0.01	11W
11E	9.89	9.87	9.87	9.88	9.88	9.88	0.01	11E
13W	9.50	9.49	9.50	9.50	9.51	9.50	0.01	13W
13E	9.49	9.47	9.48	9.48	9.51	9.49	0.02	13E
15W	9.56	9.54	9.56	9.55	9.56	9.55	0.01	15W
15E	9.86	9.86	9.86	9.88	9.90	9.87	0.02	15E
17W	9.99	9.97	9.99	9.97	10.00	9.98	0.01	17W
17E	9.81	9.80	9.81	9.80	9.82	9.81	0.01	17E
19W	9.64	9.63	9.64	9.62	9.65	9.64	0.01	19W
19E	9.06	9.05	9.05	9.04	9.07	9.05	0.01	19E
21W	9.91	9.89	9.91	9.89	9.93	9.91	0.02	21W
21E	9.78	9.78	9.79	9.78	9.83	9.79	0.02	21E
23W	4.73	4.72	4.75	4.73	4.75	4.74	0.01	23W
23E	19.77	19.75	19.77	19.75	19.78	19.76	0.01	23E
25W	4.66	4.62	4.66	4.64	4.66	4.65	0.02	25W
25E	4.28	4.26	4.30	4.29	4.29	4.28	0.02	25E
27W	9.62	9.61	9.64	9.60	9.62	9.62	0.01	27W
27E	9.79	9.80	9.82	9.79	9.80	9.80	0.01	27E
29W	9.90	9.90	9.91	9.89	9.92	9.90	0.01	29W
29E	9.72	9.72	9.73	9.71	9.74	9.72	0.01	29E
31W	9.84	9.85	9.85	9.83	9.87	9.85	0.01	31W
31E	9.95	9.95	9.96	9.94	9.98	9.96	0.02	31E
33W	9.90	9.90	9.91	9.89	9.93	9.91	0.02	33W
33E	10.01	10.01	10.02	9.99	10.04	10.01	0.02	33E

Table 7.1 Pilkington Precise Levelling Survey - PS Levels for Surveys 1 - 5.

264

Station	SD (6-10)	Mean (6-10)	Survey Number/Date					Station
			10 29.03.90	9 25.01.90	8 30.11.89	7 17.08.89	6 01.06.89	
9W	0.00	10.00	10.00	10.00	10.00	10.00	10.00	9W
9E	0.01	9.79	9.79	9.78	9.78	9.79	9.80	9E
11W	0.01	9.84	9.85	9.83	9.86	9.84	9.84	11W
11E	0.01	9.88	9.89	9.88	9.88	9.87	9.87	11E
13W	0.01	9.51	9.51	9.51	9.52	9.49	9.52	13W
13E	0.01	9.48	9.48	9.48	9.49	9.47	9.50	13E
15W	0.01	9.56	9.56	9.54	9.58	9.55	9.56	15W
15E	0.01	9.89	9.90	9.89	9.89	9.88	9.87	15E
17W	0.01	9.99	10.00	9.99	9.99	9.98	9.98	17W
17E	0.01	9.80	9.79	9.80	9.81	9.79	9.80	17E
19W	0.02	9.63	9.65	9.64	9.65	9.61	9.61	19W
19E	0.02	9.06	9.08	9.05	9.08	9.04	9.05	19E
21W	0.02	9.91	9.93	9.91	9.93	9.89	9.91	21W
21E	0.02	9.79	9.79	9.78	9.80	9.77	9.81	21E
23W	0.01	4.74	4.76	4.74	4.73	4.73	4.73	23W
23E	0.02	19.75	19.77	19.73	19.76	19.74	19.75	23E
25W	0.02	4.64	4.66	4.65	4.65	4.63	4.62	25W
25E	0.02	4.27	4.29	4.25	4.26	4.26	4.29	25E
27W	0.01	9.61	9.63	9.61	9.61	9.59	9.61	27W
27E	0.02	9.79	9.81	9.78	9.80	9.75	9.79	27E
29W	0.02	9.91	9.93	9.90	9.91	9.89	9.90	29W
29E	0.02	9.73	9.74	9.73	9.74	9.70	9.72	29E
31W	0.02	9.85	9.88	9.84	9.85	9.82	9.84	31W
31E	0.03	9.96	10.00	9.95	9.97	9.93	9.96	31E
33W	0.02	9.90	9.94	9.90	9.90	9.87	9.90	33W
33E	0.04	9.99	10.04	9.98	9.98	9.94	10.02	33E

Table 7.2 Pilkington Precise Levelling Survey - PS Levels for Surveys 6 - 10.

265

Station	SD (1-5)	Mean (1-5)	Survey Number/Date					Station
			10 29.03.90	9 25.01.90	8 30.11.89	7 17.08.89	6 01.06.89	
9W	0.00	10.00	0.00	0.00	0.00	0.00	0.00	9W
9E	0.01	9.79	0.00	-0.01	-0.01	0.00	0.01	9E
11W	0.01	9.85	0.00	-0.02	0.01	-0.01	-0.01	11W
11E	0.01	9.88	0.01	0.00	0.00	-0.01	-0.01	11E
13W	0.01	9.50	0.01	0.01	0.02	-0.01	0.02	13W
13E	0.02	9.49	-0.01	-0.01	0.00	-0.02	0.01	13E
15W	0.01	9.55	0.01	-0.01	0.03	0.00	0.01	15W
15E	0.02	9.87	0.03	0.02	0.02	0.01	0.00	15E
17W	0.01	9.98	0.02	0.01	0.01	0.00	0.00	17W
17E	0.01	9.81	-0.02	-0.01	0.00	-0.02	-0.01	17E
19W	0.01	9.64	0.01	0.00	0.01	-0.03	-0.03	19W
19E	0.01	9.05	0.03	0.00	0.03	-0.01	0.00	19E
21W	0.02	9.91	0.02	0.00	0.02	-0.02	0.00	21W
21E	0.02	9.79	0.00	-0.01	0.01	-0.02	0.02	21E
23W	0.01	4.74	0.02	0.00	-0.01	-0.01	-0.01	23W
23E	0.01	19.76	0.01	-0.03	0.00	-0.02	-0.01	23E
25W	0.02	4.65	0.01	0.00	0.00	-0.02	-0.03	25W
25E	0.02	4.28	0.01	-0.03	-0.02	-0.02	0.01	25E
27W	0.01	9.62	0.01	-0.01	-0.01	-0.03	-0.01	27W
27E	0.01	9.80	0.01	-0.02	0.00	-0.05	-0.01	27E
29W	0.01	9.90	0.03	0.00	0.01	-0.01	0.00	29W
29E	0.01	9.72	0.02	0.01	0.02	-0.02	0.00	29E
31W	0.01	9.85	0.03	-0.01	0.00	-0.03	-0.01	31W
31E	0.02	9.96	0.04	-0.01	0.01	-0.03	0.00	31E
33W	0.02	9.91	0.03	-0.01	-0.01	-0.04	-0.01	33W
33E	0.02	10.01	0.03	-0.03	-0.03	-0.07	0.01	33E

Table 7.3 Pilkington Precise Levelling Survey - Deviations of the PS Levels from the Accepted Values.

266

Pair	SD (1-5)	Mean (1-5)	Survey Number/Date					Pair
			5 29.03.89	4 01.03.89	3 01.02.89	2 11.01.89	1 14.12.88	
9	0.01	-0.21	-0.21	-0.22	-0.21	-0.22	-0.21	9
11	0.01	0.02	0.02	0.02	0.01	0.03	0.04	11
13	0.01	-0.01	0.00	-0.02	-0.02	-0.02	-0.01	13
15	0.02	0.32	0.34	0.33	0.30	0.32	0.30	15
17	0.01	-0.18	-0.18	-0.17	-0.18	-0.17	-0.18	17
19	0.00	-0.58	-0.58	-0.58	-0.59	-0.58	-0.58	19
21	0.01	-0.11	-0.10	-0.11	-0.12	-0.11	-0.13	21
23	0.01	15.03	15.03	15.02	15.02	15.03	15.04	23
25	0.01	-0.36	-0.37	-0.35	-0.36	-0.36	-0.38	25
27	0.01	0.18	0.18	0.19	0.18	0.19	0.17	27
29	0.00	-0.18	-0.18	-0.18	-0.18	-0.18	-0.18	29
31	0.00	0.11	0.11	0.11	0.11	0.10	0.11	31
33	0.00	0.11	0.11	0.10	0.11	0.11	0.11	33

Table 7.4 Pilkington Precise Levelling Survey - PSP Level Differences for Surveys 1 - 5.

Pair	SD (6-10)	Mean (6-10)	10 29.03.90	9 25.01.90	8 30.11.89	7 17.08.89	6 01.06.89	Pair
9	0.01	-0.21	-0.21	-0.22	-0.22	-0.21	-0.20	9
11	0.01	0.03	0.04	0.05	0.02	0.03	0.03	11
13	0.01	-0.03	-0.03	-0.03	-0.03	-0.02	-0.02	13
15	0.02	0.33	0.34	0.35	0.31	0.33	0.31	15
17	0.01	-0.19	-0.21	-0.19	-0.18	-0.19	-0.18	17
19	0.01	-0.57	-0.57	-0.59	-0.57	-0.57	-0.56	19
21	0.02	-0.12	-0.14	-0.13	-0.13	-0.12	-0.10	21
23	0.01	15.01	15.01	14.99	15.03	15.01	15.02	23
25	0.03	-0.37	-0.37	-0.40	-0.39	-0.37	-0.33	25
27	0.01	0.18	0.18	0.17	0.19	0.16	0.18	27
29	0.01	-0.18	-0.19	-0.17	-0.17	-0.19	-0.18	29
31	0.01	0.12	0.12	0.11	0.12	0.11	0.12	31
33	0.02	0.09	0.10	0.08	0.08	0.07	0.12	33

Survey Number / Date

Table 7.5 Pilkington Precise Levelling Survey - PSP Level Differences for Surveys 6 - 10.

Pair	SD (1-5)	Mean (1-5)	Survey Number/Date					Pair
			10 29.03.90	9 25.01.90	8 30.11.89	7 17.08.89	6 01.06.89	
9	0.01	-0.21	0.00	-0.01	-0.01	0.00	0.01	9
11	0.01	0.02	0.02	0.03	0.00	0.01	0.01	11
13	0.01	-0.01	-0.02	-0.02	-0.02	-0.01	-0.01	13
15	0.02	0.32	0.02	0.03	-0.01	0.01	-0.01	15
17	0.01	-0.18	-0.03	-0.01	0.00	-0.01	0.00	17
19	0.00	-0.58	0.01	-0.01	0.01	0.01	0.02	19
21	0.01	-0.11	-0.03	-0.02	-0.02	-0.01	0.01	21
23	0.01	15.03	-0.02	-0.04	0.00	-0.02	-0.01	23
25	0.01	-0.36	-0.01	-0.04	-0.03	-0.01	0.03	25
27	0.01	0.18	0.00	-0.01	0.01	-0.02	0.00	27
29	0.00	-0.18	-0.01	0.01	0.01	-0.01	0.00	29
31	0.00	0.11	0.01	0.00	0.01	0.00	0.01	31
33	0.00	0.11	-0.01	-0.03	-0.03	-0.04	0.01	33

Table 7.6 Pilkington Precise Levelling Survey - Deviations of the PSP Level Differences from the Accepted Values.

As far as the glass grinding and polishing process is concerned, the most important data is the relative level difference between two opposite pillars, ie pillar station pair (PSP) level difference. Table 7.4 contains the PSP level differences for surveys 1 - 5, the means of which are taken as the accepted PSP level differences. Table 7.5 contains the PSP level differences for surveys 6 - 10, and table 7.6 contains the deviations of these PSP level differences from the accepted values.

The standard deviations of the accepted PSP level differences are all below ±0.02 cm, and the standard deviations of the mean PSP level differences for surveys 6 - 10 are all below ±0.02 cm, with the exception of one value. Comparing the mean PSP level differences for surveys 6 - 10 with the accepted PSP level differences, all agree to within ±0.02 cm. In addition, from Table 7.6, it can be seen that the deviations of the PSP level differences, established in surveys 6 - 10, from the accepted values are all within ±0.03 cm, with the exception of one value in survey number 7, and two values in survey number 9.

The variations in PS levels and PSP level differences may be due to instrumental errors, observational errors or climatic changes throughout the year. However, it is important to note that their magnitude is within the claimed accuracy of the Wild NA2 automatic level. From the precise levelling surveys performed up to the present time, it can be concluded that the refined precise levelling technique is capable of determining the relative levels of the pillar stations to an accuracy of better than ±0.03 cm (±300 µm).

270

7.3.4 UN ETIS Measurement Survey PLK1

The first series of measurement surveys (PLK1) were performed from January 1989 to March 1989. This series of measurement surveys was designed to investigate the applicability of UN ETIS to monitoring the relative deformation of a single pillar pair (number 9). Three measurement surveys were performed.

7.3.4.1 Survey Description

The first measurement survey (PLK101) was performed on 30 January 1989, the second (PLK102) on 8 March 1989 and the third (PLK103) on 16 March 1989.

Each of the measurement surveys were performed in separate local coordinate systems, assuming the x-axis to run in the approximate direction of the process. The measurement set up is shown in Figure 7.6, where the following points can be defined:

T1	=	theodolite control point 1.
T2	=	theodolite control point 2.
P09s	=	target point located in a wall plug used for precise levelling (approximately 1.35 m above the floor).
F09s	=	target point located in a wall plug near the floor (approximately 0.75 m above the floor).

where

s	=	side of tunnel, ie *W* (west) or *E* (east).

Tables Descend

36

N

Polishing
Section

30

25

Grinding
Section

20

15

Tables
Ascend

10 → (ii)

5

1

Scale 1:1500

(i) Process Line Supporting
Pillars.

X

T2   Kern E2
            (352396)

P09W   P09E
F09W   F09E

T1   Kern E2
            (352393)

Scale 1:200

(ii) Measurement Set Up.
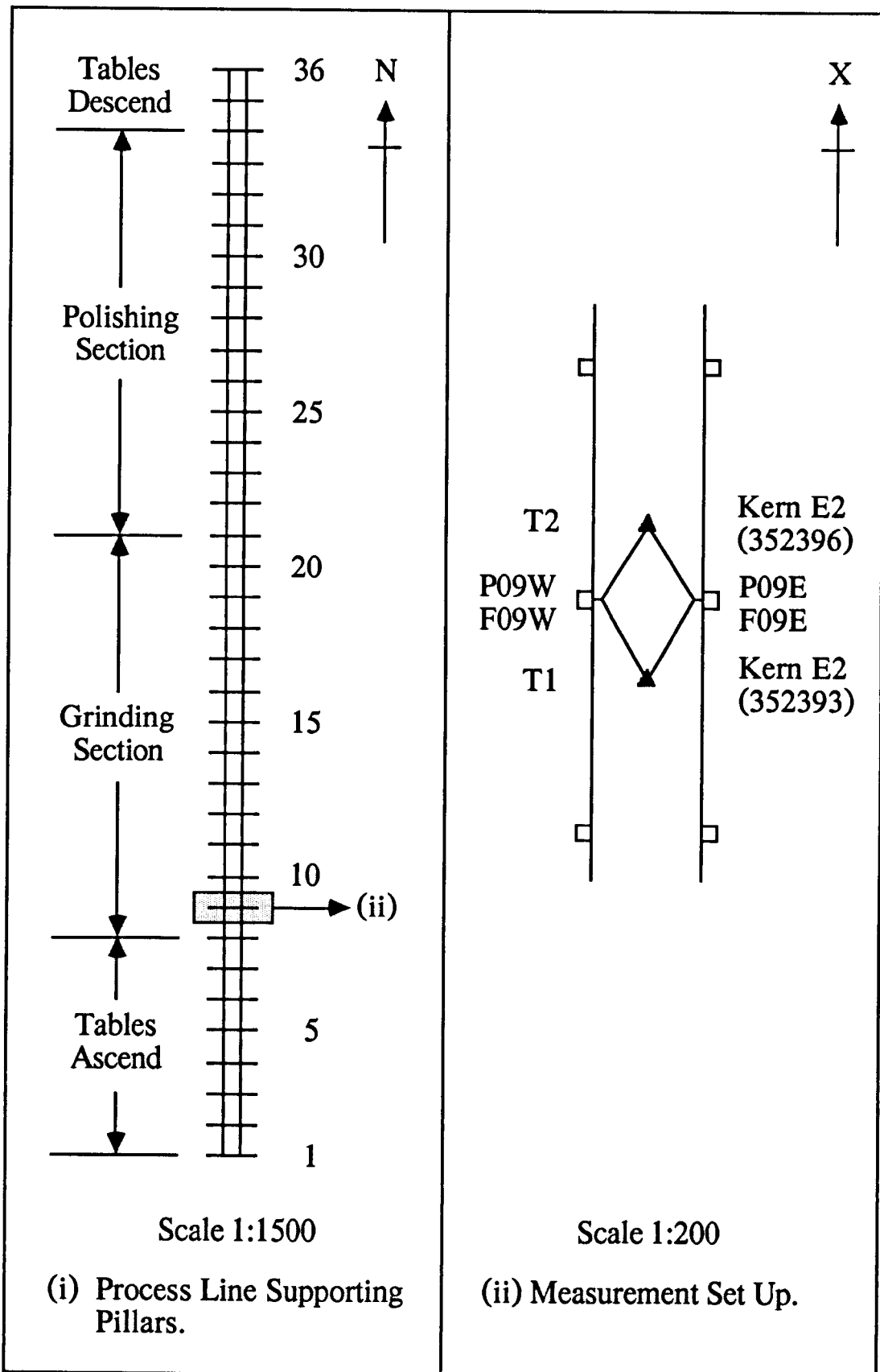
Figure 7.6 UN ETIS Measurement Survey PLK1 - Measurement
Set Up.

In all three surveys the theodolites were mounted on wooden tripods, and the local coordinate system was set up using a Kern carbon fibre scale bar (358457). Each measurement survey was performed as follows:

(i) Locate the two theodolites approximately 5 m apart.

(ii) Set up a local coordinate system by the use of a calibrated scale bar and determine the local coordinates of the theodolites.

(T1:LO, T2:LO)

(iii) Measure the pillar points in the local coordinate system.

(P09W:LO, F09W:LO, P09E:LO, F09E:LO)

Plate 7.2 shows measurements being performed to pillar point P09W.

7.3.4.2 Target Description

The UN ETIS target, developed for use in measurement survey PLK1, consists of a 1 mm spherical target mounted on the end of a 300 mm tubular extension rod (Figure 7.7).

The target has been developed to facilitate accurate sighting from both electronic theodolites. It locates in the wall plugs used for precise levelling, and the tubular extension rod permits sighting without interference from obstacles, such as pipes, located near the pillars.

Plate 7.2

UN ETIS Measurement Survey PLK1.
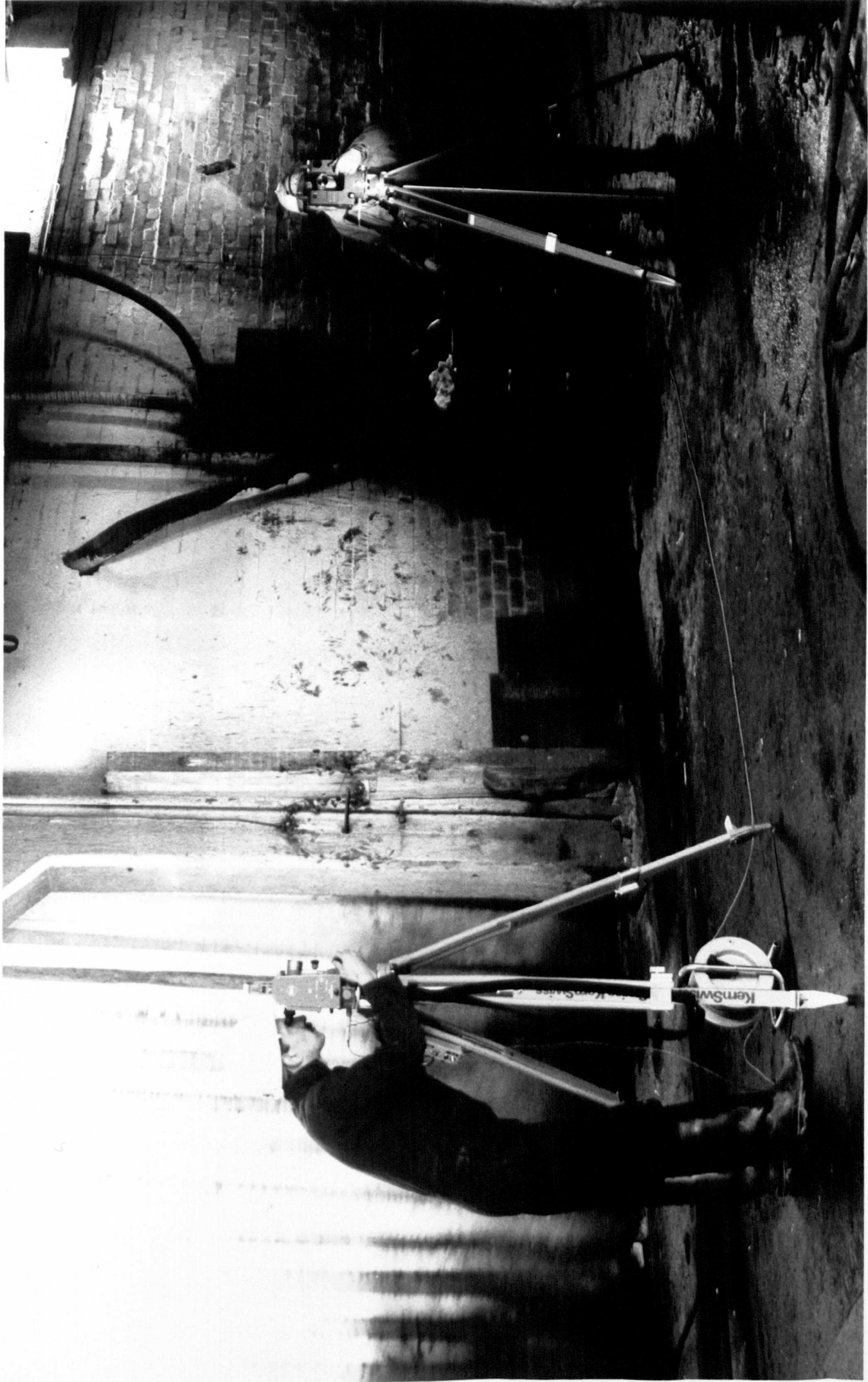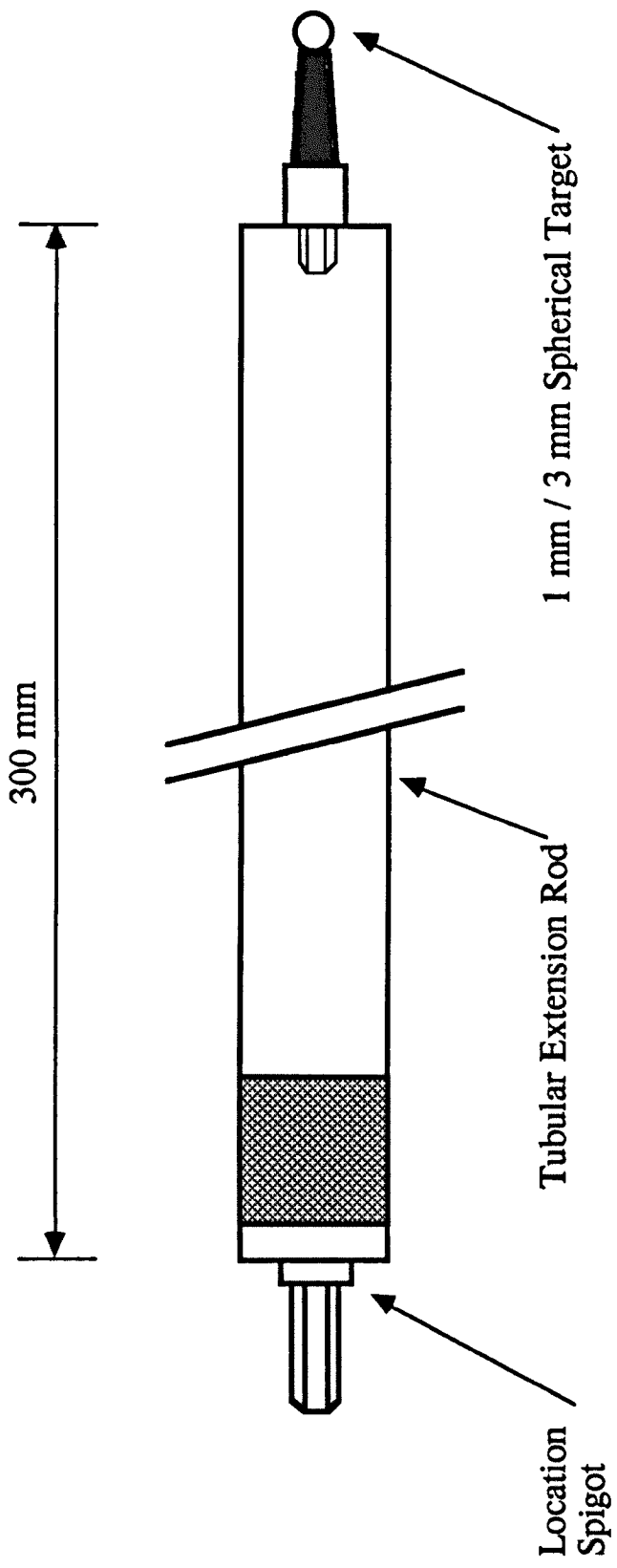
Figure 7.7 UN ETIS Measurement Survey PLK1 - Target.

7.3.4.3 Results and Conclusions

The first series of measurement surveys (PLK1) was designed to obtain very high accuracy measurements on a single pillar pair (number 9). Each of the three measurement surveys was performed in approximately two hours.

A guide to the general accuracy of the three measurement surveys can be obtained from the mean analysis data, given in Figures 7.8 to 7.10 inclusive.

The geometry distribution of a typical measurement survey, PLK103, is shown graphically in Figure 7.12. The scale bar points, shown as point numbers 01 - 04, have intersection angles in the range 55 - 100 degrees, and the pillar points, shown as point numbers 05 - 08, have intersection angles in the range 70 - 95 degrees. The sight lengths can be seen to be all approximately equal, and in the range 2.5 - 4.0 m. The geometry distribution illustrates the geometrically "good" measurement conditions, which are underlined by the low mean probable error values of 5.8 μm for the three measurement surveys.

The error distribution for the points in measurement survey PLK103 is shown graphically in Figure 7.13. The high accuracy of measurement is illustrated by the magnitude of the ellipsoid of error / probable error ratios. With the exception of point number 08 *(F09E:LO)*, all of the ratios have a value less than three.

Monitoring of the relative deformation of pillar pair number 9 was performed by measuring the relative three-dimensional

276

```
***** DATA FILE PLK101_R MEAN ANALYSIS DATA *****

Selected Point Name   :-   ????:LO
Total No. of Points   :-   8

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
     H A          V A          H A          V A          SURVEY MEAN
     3.5          1.3          1.8          2.3             2.2

Mean Mis-Intersections (mm)
        Dx            Dy              Dz            SURVEY MEAN
      0.0001        0.0082          0.0182            0.0088

Mean Ellipsoid Errors (mm)
        Ex            Ey              Ez            SURVEY MEAN
      0.0067        0.0097          0.0064            0.0076
Mean Probable Errors (mm)
        Px            Py              Pz            SURVEY MEAN
      0.0055        0.0032          0.0087            0.0058
```

Figure 7.8 UN ETIS Measurement Survey PLK101 - Mean Analysis Data

```
***** DATA FILE PLK102_R MEAN ANALYSIS DATA *****

Selected Point Name   :-   ????:LO
Total No. of Points   :-   8

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
     H A          V A          H A          V A          SURVEY MEAN
     3.9          2.0          2.1          2.1             2.5

Mean Mis-Intersections (mm)
        Dx            Dy              Dz            SURVEY MEAN
      0.0001        0.0080          0.0194            0.0092

Mean Ellipsoid Errors (mm)
        Ex            Ey              Ez            SURVEY MEAN
      0.0071        0.0088          0.0071            0.0077
Mean Probable Errors (mm)
        Px            Py              Pz            SURVEY MEAN
      0.0055        0.0032          0.0087            0.0058
```

Figure 7.9 UN ETIS Measurement Survey PLK102 - Mean Analysis Data

```
***** DATA FILE PLK103_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  ????:LO
Total No. of Points   :-   8

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A         V A           H A         V A          SURVEY MEAN
    4.0         1.6           1.8         1.3            2.2

Mean Mis-Intersections (mm)
       Dx            Dy            Dz           SURVEY MEAN
     0.0001        0.0091        0.0210           0.0101

Mean Ellipsoid Errors (mm)
       Ex            Ey            Ez           SURVEY MEAN
     0.0053        0.0074        0.0075           0.0067
Mean Probable Errors (mm)
       Px            Py            Pz           SURVEY MEAN
     0.0055        0.0032        0.0087           0.0058
```

Figure 7.10 UN ETIS Measurement Survey PLK103 - Mean Analysis

Data

```
***** DATA FILE PLK1 FOUR POINT DATA *****
```

Pt Name 1 / 2	Survey	3D Distance (m)	dZ (m)
P09W:LO	01	3.615023	0.000877
P09E:LO	02	3.615052	0.000747
	03	3.615114	0.000834
P09W:LO	01	3.665403	0.607624
F09E:LO	02	3.665401	0.607556
	03	3.665488	0.607608
P09W:LO	01	0.593743	0.593709
F09W:LO	02	0.593728	0.593695
	03	0.593842	0.593807
P09E:LO	01	0.606996	0.606747
F09E:LO	02	0.607059	0.606809
	03	0.607019	0.606774
P09E:LO	01	3.662639	0.592832
F09W:LO	02	3.662657	0.592948
	03	3.662710	0.592973
F09E:LO	01	3.614066	-0.013915
F09W:LO	02	3.614046	-0.013861
	03	3.614112	-0.013801

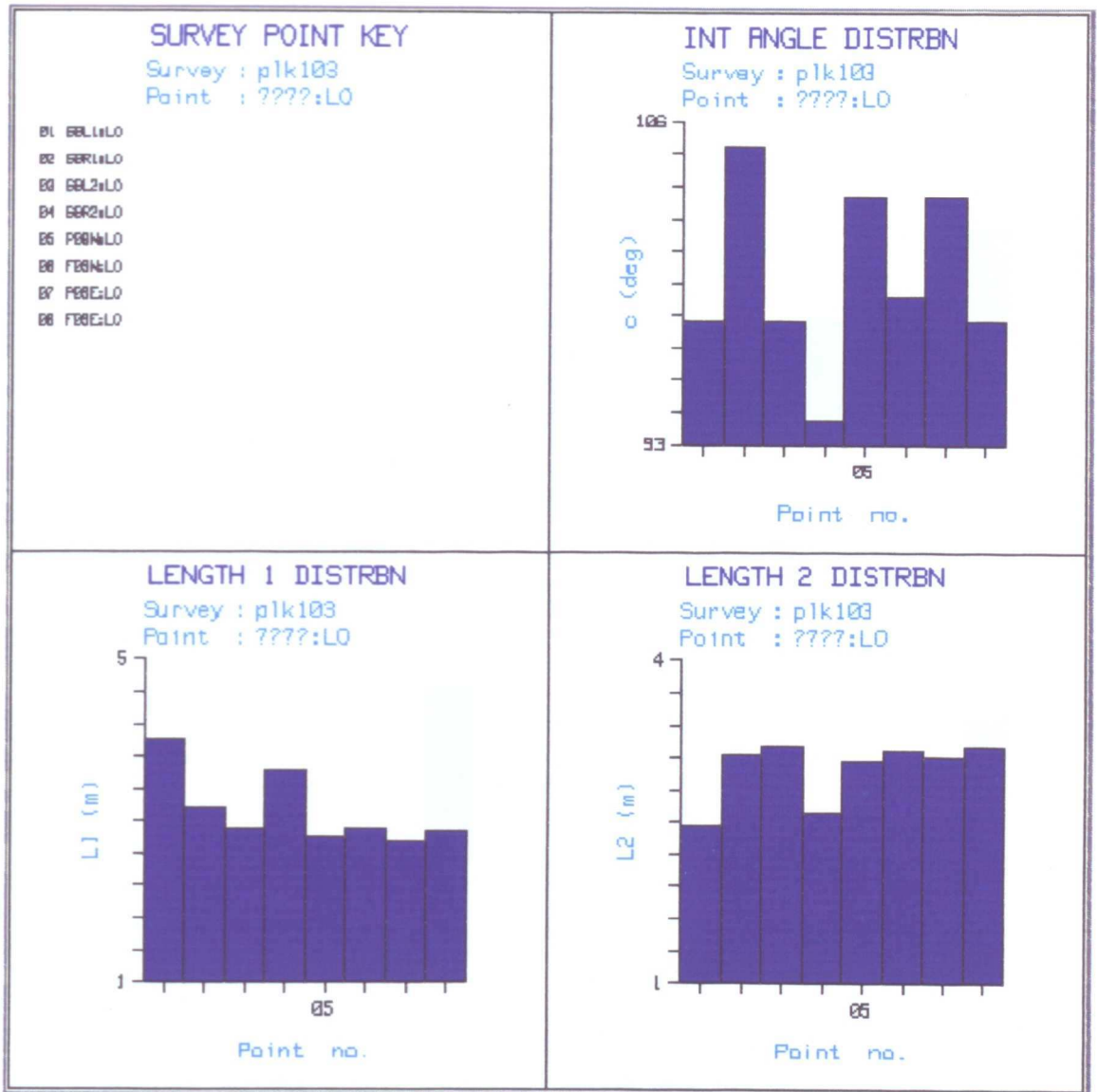Figure 7.11 UN ETIS Measurement Survey PLK1 - Four Point Data

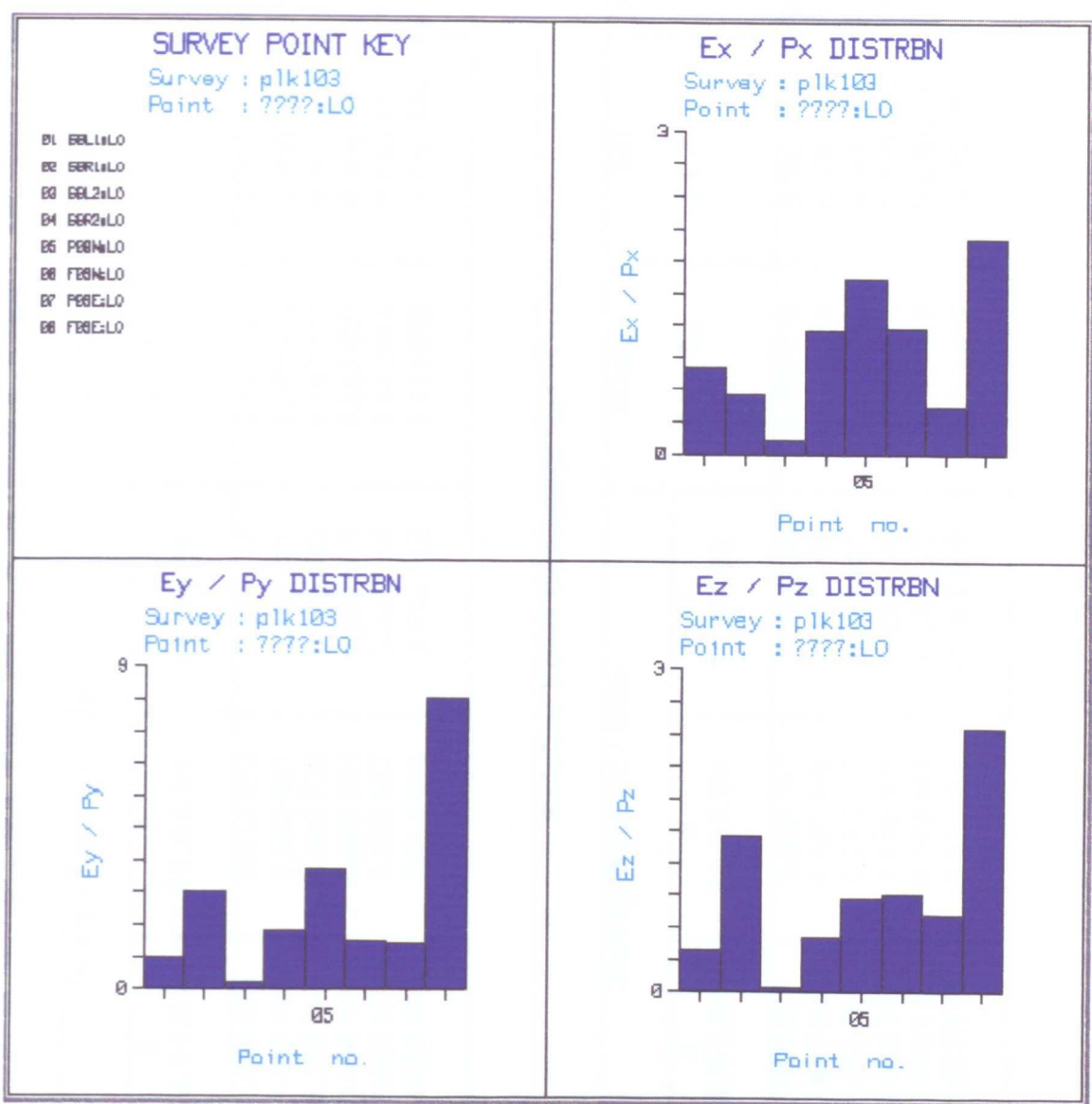Figure 7.12 UN ETIS Measurement Survey PLK103 - Geometry Distribution.

Figure 7.13 UN ETIS Measurement Survey PLK103 - Error
Distribution.

Relative Three-Dimensional Distances (m)

Survey Line	Survey Number / Date			Mean	SD	Survey Line
	1 30.01.89	2 08.03.89	3 16.03.89	(1 - 3)	(1 - 3)	
P09W - P09E	3.615 023	3.615 052	3.615 114	3.615 063	0.000 046	P09W - P09E
P09W - F09E	3.665 403	3.665 401	3.665 488	3.665 431	0.000 050	P09W - F09E
P09W - F09W	0.593 743	0.593 728	0.593 842	0.593 771	0.000 062	P09W - F09W
P09E - F09E	0.606 996	0.607 059	0.607 019	0.607 025	0.000 032	P09E - F09E
P09E - F09W	3.662 639	3.662 657	3.662 710	3.662 669	0.000 037	P09E - F09W
F09E - F09W	3.614 066	3.614 046	3.614 112	3.614 075	0.000 034	F09E - F09W

Relative Level Difference (m)

Survey Line	Survey Number / Date			Mean	SD	Survey Line
	1 30.01.89	2 08.03.89	3 16.03.89	(1 - 3)	(1 - 3)	
P09W - P09E	0.000 877	0.000 747	0.000 834	0.000 819	0.000 066	P09W - P09E
P09W - F09E	0.607 624	0.607 556	0.607 608	0.607 596	0.000 036	P09W - F09E
P09W - F09W	0.593 709	0.593 695	0.593 807	0.593 737	0.000 061	P09W - F09W
P09E - F09E	0.606 747	0.606 809	0.606 774	0.606 777	0.000 031	P09E - F09E
P09E - F09W	0.592 832	0.592 948	0.592 973	0.592 918	0.000 075	P09E - F09W
F09E - F09W	-0.013 915	-0.013 861	-0.013 801	-0.013 859	0.000 057	F09E - F09W

Table 7.7 UN ETIS Measurement Survey PLK1 - Relative Deformation Monitoring Results.

distances, and relative level differences between four points located in the two pillars. The measured data, output as four point data, is given in Figure 7.11, and the relative deformation monitoring results for the three measurement surveys are summarised in Table 7.7.

From Table 7.7, for the six combinations of points on pillar pair number 9, the mean standard deviation of the relative three-dimensional distances is ±45.2 μm, and the mean standard deviation of the relative level differences is ±54.3 μm. Thus, it can be concluded that, for a single pillar pair, UN ETIS can determine relative level differences, and relative three-dimensional distances to an accuracy of ±50 μm.

However, it is important to note that if measurement survey PLK1 were repeated for every other pair of pillars along the whole process line, that this would require thirteen measurement set ups and a total of twenty-six hours to perform, compared to the two and a half hours required for the refined precise levelling technique.

7.3.5 UN ETIS Measurement Survey PLK2

The second series of measurement surveys (PLK2) has been performed from January 1989 up to the present time. This series of measurement surveys was designed to investigate the applicability of UN ETIS to monitoring the relative deformation of a small part of the process line, which includes three pillar pairs (numbers 9, 11 and 13). A set-up survey and three measurement surveys have been performed.

7.3.5.1 Survey Description

The set-up survey (PLK200) was a combination of the three measurement surveys PLK101, PLK102 and PLK103. This was used to define the object coordinate system for the three subsequent measurement surveys. The first measurement survey (PLK201) was performed on 15 June 1989, the second (PLK202) on 12 October 1989 and the third (PLK203) on 8 February 1990.

Each of the measurement surveys was performed in an object coordinate system, assuming the x-axis to run in the approximate direction of the process, and the four points on pillar pair number 9 to be fixed points in the object coordinate system. The measurement survey set up is shown in Figure 7.14, where the following points can be defined:

T10 = theodolite control point 1.

T12 = theodolite control point 2.

F09s = target point located in pillar number 9, in a wall plug near the floor (approximately 0.75 m above the floor).

Pns = target point located in a wall plug used for precise levelling (approximately 1.35 m above the floor).

where

n = number of pillar pair, ie *09, 11* or *13.*
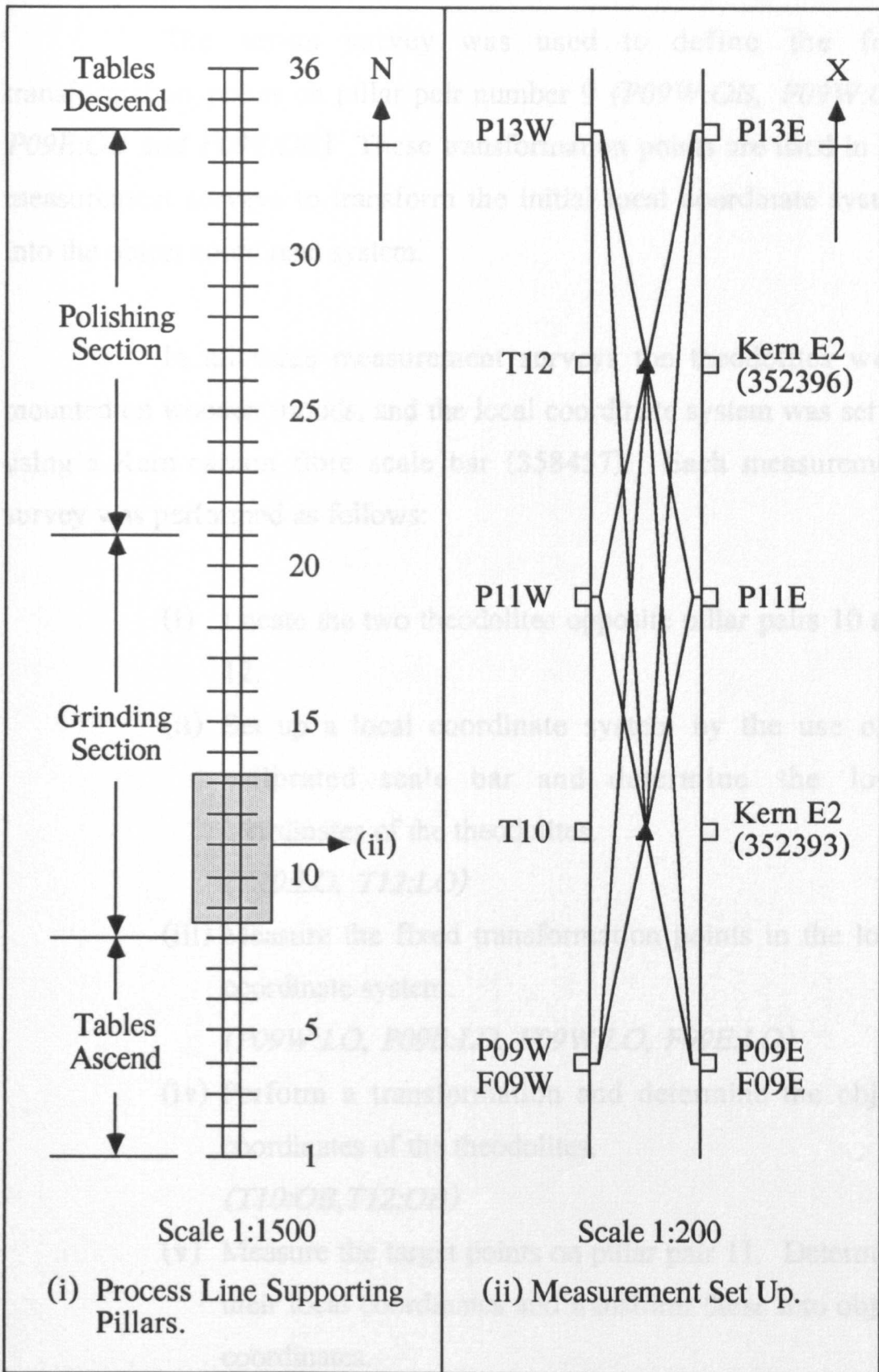
s = side of tunnel, ie *W* (west) or *E* (east).

Figure 7.14 UN ETIS Measurement Survey PLK2 - Measurement Set Up.

284

The set-up survey was used to define the four transformation points on pillar pair number 9 *(P09W:OB, F09W:OB, P09E:OB and F09E:OB)*. These transformation points are used in the measurement surveys to transform the initial local coordinate system into the object coordinate system.

In all three measurement surveys the theodolites were mounted on wooden tripods, and the local coordinate system was set up using a Kern carbon fibre scale bar (358457). Each measurement survey was performed as follows:

(i) Locate the two theodolites opposite pillar pairs 10 and 12.

(ii) Set up a local coordinate system by the use of a calibrated scale bar and determine the local coordinates of the theodolites.
(T10:LO, T12:LO)

(iii) Measure the fixed transformation points in the local coordinate system.
(P09W:LO, P09E:LO, F09W:LO, F09E:LO)

(iv) Perform a transformation and determine the object coordinates of the theodolites.
(T10:OB,T12:OB)

(v) Measure the target points on pillar pair 11. Determine their local coordinates and transform these into object coordinates.
(P11W:LO, P11W:OB, P11E:LO, P11E:OB)

Plate 7.3

UN ETIS Measurement Survey PLK2.

(vi) Measure the target points on pillar pair 13. Determine their local coordinates and transform these into object coordinates.

(P13W:LO, P13W:OB, P13E:LO, P13E:OB)

Plate 7.3 shows measurements being performed to pillar point P11W.

7.3.5.2 Target Description

The UN ETIS target used in measurement survey PLK2 is the same as that used in measurement survey PLK1 (Figure 7.7), except that it incorporates a 3 mm diameter spherical target. The increase in diameter of the spherical target is required due to the increased sight lengths encountered in measurement survey PLK2.

7.3.5.3 Results and Conclusions

The second series of measurement surveys (PLK2) was designed to test the accuracy of UN ETIS in measurements to three pillar pairs (numbers 9, 11 and 13). Each of the three measurement surveys was performed in approximately three and a half hours.

A guide to the general accuracy of the three measurement surveys can be obtained from the mean analysis data, given in Figures 7.15 to 7.18 inclusive.

The geometry distribution of a typical measurement survey, PLK203, is shown graphically in Figure 7.20. The scale bar

287

points, shown as point numbers 01 - 04, and the pillar points in the centre of the measurement space, shown as point numbers 09 - 10, have intersection angles in the range 145 - 160 degrees and approximately equal sight lengths in the range 6 - 9 m. The pillar points at the extremes of the measurement space, shown as point numbers 05 - 08 and 11 - 12, have intersection angles in the range 7 - 10 degrees and unequal sight lengths in the ranges 6 - 8 m or 21 - 22 m. The geometry distribution illustrates the geometrically "poor" measurement conditions, which are underlined by the high mean probable error values of approximately 66 μm for the three measurement surveys.

The error distribution for the points in measurement survey PLK203 is shown graphically in Figure 7.21. The relatively low accuracy of measurement is illustrated by the magnitude of the ellipsoid of error / probable error ratios. The worst case being point number 03 *(SBL2:LO)*, which has a Ey / Py value of sixty-five.

Monitoring of the relative deformation of pillar pair numbers 9, 11 and 13 was performed by measuring the relative three-dimensional distances, and relative level differences between four points in consecutive pillar pairs. The measured data, output as four point data, is given in Figure 7.19. The relative deformation monitoring results for the three measurement surveys are summarised in Table 7.8 (pillar pair numbers 9 and 11) and Table 7.9 (pillar pair numbers 11 and 13).

From Table 7.8, for the five combinations of points on pillar pair numbers 9 and 11, the mean standard deviation of the relative three-dimensional distances is ±717.2 μm, and the mean

```
***** DATA FILE PLK200_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  *
Total No. of Points  :-  20

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A         V A          H A         V A        SURVEY MEAN
    4.0         1.7          1.9         2.0            2.4

Mean Mis-Intersections (mm)
        Dx            Dy            Dz           SURVEY MEAN
      0.0001        0.0080        0.0203           0.0095

Mean Ellipsoid Errors (mm)
        Ex            Ey            Ez           SURVEY MEAN
      0.0067        0.0094        0.0073           0.0078
Mean Probable Errors (mm)
        Px            Py            Pz           SURVEY MEAN
      0.0053        0.0031        0.0085           0.0057
```

Figure 7.15 UN ETIS Measurement Survey PLK200 - Mean Analysis Data

```
***** DATA FILE PLK201_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  ????:LO
Total No. of Points  :-  12

Mean Angular Deviations (secs)
      THEODOLITE 1              THEODOLITE 2
    H A         V A          H A         V A        SURVEY MEAN
    0.9         1.1          2.0         1.7            1.4

Mean Mis-Intersections (mm)
        Dx            Dy            Dz           SURVEY MEAN
      0.0003        0.0173        0.0653           0.0276

Mean Ellipsoid Errors (mm)
        Ex            Ey            Ez           SURVEY MEAN
      0.1967        0.1115        0.0242           0.1108
Mean Probable Errors (mm)
        Px            Py            Pz           SURVEY MEAN
      0.1298        0.0268        0.0407           0.0658
```

Figure 7.16 UN ETIS Measurement Survey PLK201 - Mean Analysis Data

289

```
***** DATA FILE PLK202_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  ????:LO
Total No. of Points  :-  12

Mean Angular Deviations (secs)
       THEODOLITE 1              THEODOLITE 2
     H A        V A           H A        V A        SURVEY MEAN
     1.7        1.0           1.2        3.4            1.8


Mean Mis-Intersections (mm)
       Dx           Dy           Dz            SURVEY MEAN
     0.0001       0.0427       0.1377            0.0602


Mean Ellipsoid Errors (mm)
       Ex           Ey           Ez            SURVEY MEAN
     0.5347       0.3432       0.0669            0.3149
Mean Probable Errors (mm)
       Px           Py           Pz            SURVEY MEAN
     0.1307       0.0272       0.0411            0.0663
```

Figure 7.17 UN ETIS Measurement Survey PLK202 - Mean Analysis Data

```
***** DATA FILE PLK203_R MEAN ANALYSIS DATA *****

Selected Point Name  :-  ????:LO
Total No. of Points  :-  12

Mean Angular Deviations (secs)
       THEODOLITE 1              THEODOLITE 2
     H A        V A           H A        V A        SURVEY MEAN
     1.6        1.0           0.9        3.5            1.8

Mean Mis-Intersections (mm)
       Dx           Dy           Dz            SURVEY MEAN
     0.0001       0.0421       0.1419            0.0614


Mean Ellipsoid Errors (mm)
       Ex           Ey           Ez            SURVEY MEAN
     0.6564       0.3522       0.0662            0.3583
Mean Probable Errors (mm)
       Px           Py           Pz            SURVEY MEAN
     0.1307       0.0272       0.0411            0.0663
```

Figure 7.18 UN ETIS Measurement Survey PLK203 - Mean Analysis Data

Pt Name 1 / 2	Survey	3D Distance (m)	dZ (m)
P09W:OB P09E:OB	01 02 03	3.615078 3.615078 3.615078	0.000820 0.000820 0.000820
P09W:OB P11E:OB	01 02 03	15.302527 15.303349 15.301774	-0.002799 -0.002941 -0.003021
P09W:OB P11W:OB	01 02 03	14.630875 14.629804 14.628483	-0.001022 -0.001103 -0.001127
P09E:OB P11E:OB	01 02 03	14.886184 14.887801 14.886748	-0.003618 -0.003761 -0.003841
P09E:OB P11W:OB	01 02 03	15.086284 15.086058 15.085236	-0.001842 -0.001923 -0.001947
P11E:OB P11W:OB	01 02 03	3.619219 3.619617 3.619230	0.001777 0.001838 0.001894

***** DATA FILE PLK2 FOUR POINT DATA *****

Pt Name 1 / 2	Survey	3D Distance (m)	dZ (m)
P11W:OB P11E:OB	01 02 03	3.619219 3.619617 3.619230	-0.001777 -0.001838 -0.001894
P11W:OB P13E:OB	01 02 03	15.833656 15.831054 15.831965	0.004639 0.004577 0.004183
P11W:OB P13W:OB	01 02 03	15.194263 15.203679 15.203771	0.008770 0.008921 0.008818
P11E:OB P13E:OB	01 02 03	15.180211 15.175837 15.177050	0.006415 0.006415 0.006077
P11E:OB P13W:OB	01 02 03	15.388363 15.395910 15.396156	0.010547 0.010759 0.010712
P13E:OB P13W:OB	01 02 03	3.619862 3.619056 3.619001	0.004132 0.004343 0.004635

Figure 7.19 UN ETIS Measurement Survey PLK2 - Four Point Data
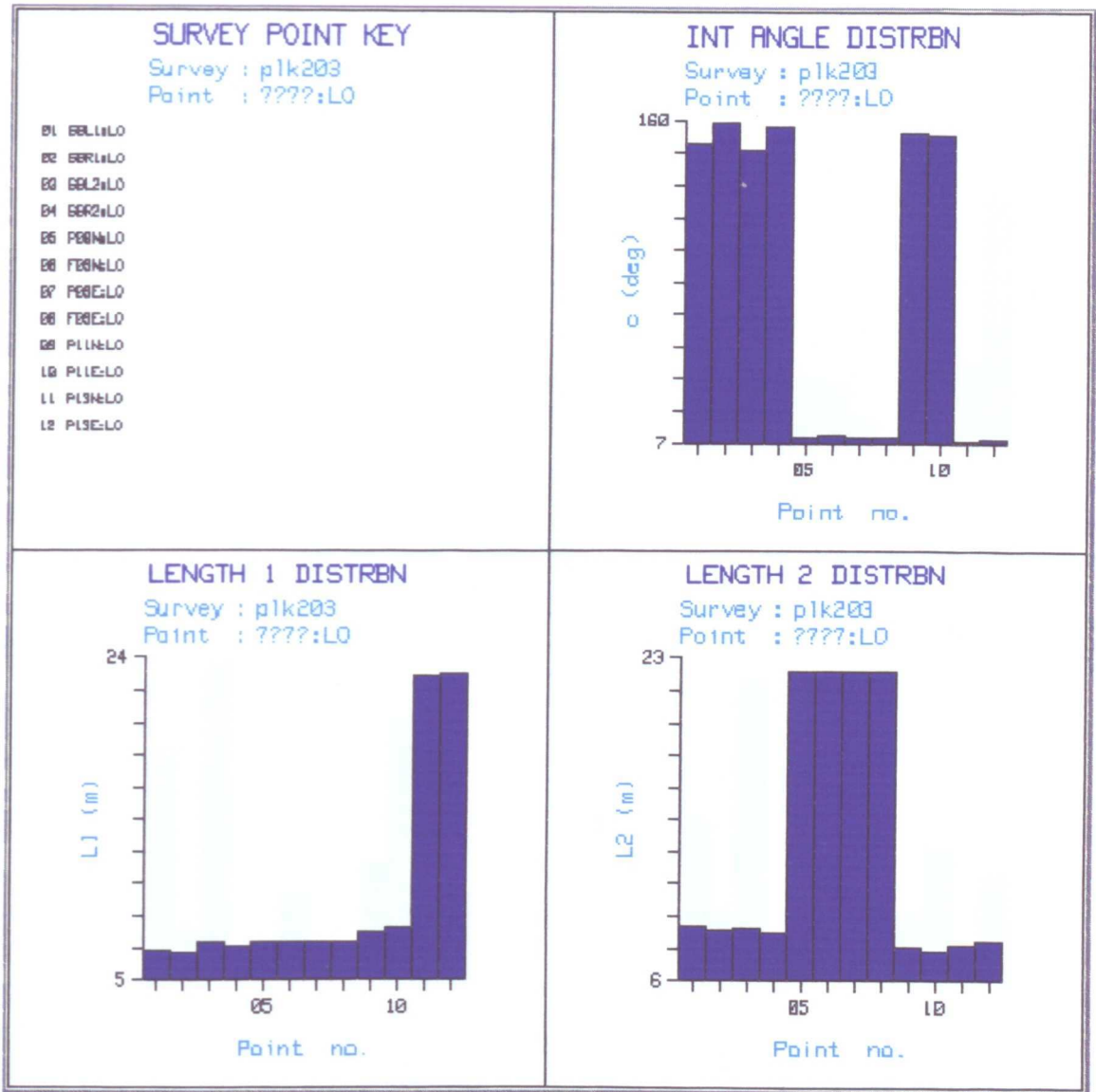
291

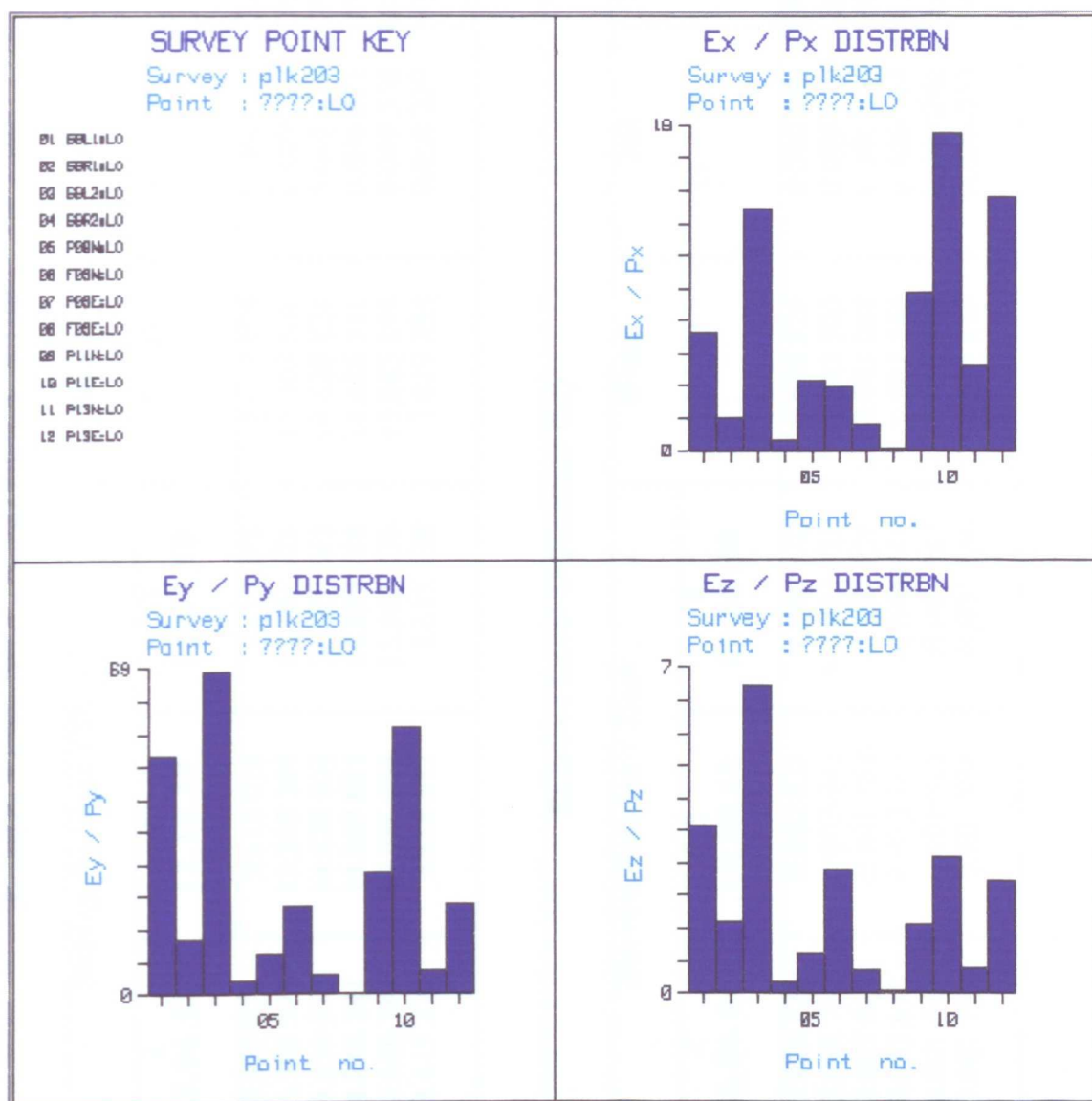Figure 7.20 UN ETIS Measurement Survey PLK203 - Geometry Distribution.

Figure 7.21 UN ETIS Measurement Survey PLK203 - Error
Distribution.

Relative Three-Dimensional Distances (m)

Survey Line		Survey Number / Date			Mean	SD	Survey Line	
		1 15.06.89	2 12.10.89	3 08.02.90	(1 - 3)	(1 - 3)		
P09W - P09E		3.615 078	3.615 078	3.615 078	3.615 078	0.000 000	P09W - P09E	
P09W - P11E		15.302 527	15.303 349	15.301 774	15.302 550	0.000 788	P09W - P11E	
P09W - P11W		14.630 875	14.629 804	14.628 483	14.629 721	0.001 198	P09W - P11W	
P09E - P11E		14.886 184	14.887 801	14.886 748	14.886 911	0.000 821	P09E - P11E	
P09E - P11W		15.086 284	15.086 058	15.085 236	15.085 859	0.000 552	P09E - P11W	
P11E - P11W		3.619 219	3.619 617	3.619 230	3.619 355	0.000 227	P11E - P11W	

Relative Level Difference (m)

Survey Line		Survey Number / Date			Mean	SD	Survey Line	
		1 15.06.89	2 12.10.89	3 08.02.90	(1 - 3)	(1 - 3)		
P09W - P09E		0.000 820	0.000 820	0.000 820	0.000 820	0.000 000	P09W - P09E	
P09W - P11E		-0.002 799	-0.002 941	-0.003 021	-0.002 920	0.000 112	P09W - P11E	
P09W - P11W		-0.001 022	-0.001 103	-0.001 127	-0.001 084	0.000 055	P09W - P11W	
P09E - P11E		-0.003 618	-0.003 761	-0.003 841	-0.003 740	0.000 113	P09E - P11E	
P09E - P11W		-0.001 842	-0.001 923	-0.001 947	-0.001 904	0.000 055	P09E - P11W	
P11E - P11W		0.001 777	0.001 838	0.001 894	0.001 836	0.000 059	P11E - P11W	

Table 7.8 UN ETIS Measurement Survey PLK2 - Relative Deformation
Monitoring Results for Pillars 9 and 11.

294

Relative Three-Dimensional Distances (m)

Survey Line	Survey Number / Date			Mean (1 - 3)	SD (1 - 3)	Survey Line
	1 15.06.89	2 12.10.89	3 08.02.90			
P11W - P11E	3.619 219	3.619 617	3.619 230	3.619 355	0.000 227	P11W - P11E
P11W - P13E	15.833 656	15.831 054	15.831 965	15.832 225	0.001 320	P11W - P13E
P11W - P13W	15.194 263	15.203 679	15.203 771	15.200 571	0.005 463	P11W - P13W
P11E - P13E	15.180 211	15.175 837	15.177 050	15.177 699	0.002 258	P11E - P13E
P11E - P13W	15.388 363	15.395 910	15.396 156	15.393 476	0.004 430	P11E - P13W
P13E - P13W	3.619 862	3.619 056	3.619 001	3.619 306	0.000 482	P13E - P13W

Relative Level Difference (m)

Survey Line	Survey Number / Date			Mean (1 - 3)	SD (1 - 3)	Survey Line
	1 15.06.89	2 12.10.89	3 08.02.90			
P11W - P11E	-0.001 777	-0.001 838	-0.001 894	-0.001 836	0.000 059	P11W - P11E
P11W - P13E	0.004 639	0.004 577	0.004 183	0.004 466	0.000 247	P11W - P13E
P11W - P13W	0.008 770	0.008 921	0.008 818	0.008 836	0.000 077	P11W - P13W
P11E - P13E	0.006 415	0.006 415	0.006 077	0.006 302	0.000 195	P11E - P13E
P11E - P13W	0.010 547	0.010 759	0.010 712	0.010 673	0.000 111	P11E - P13W
P13E - P13W	0.004 132	0.004 343	0.004 635	0.004 370	0.000 253	P13E - P13W

Table 7.9 UN ETIS Measurement Survey PLK2 - Relative Deformation Monitoring Results for Pillars 11 and 13.

295

standard deviation of the relative level differences is ±78.8 µm. From Table 7.9, for the six combinations of points on pillar pair numbers 11 and 13, the mean standard deviation of the relative three-dimensional distances is ±2363.3 µm, and the mean standard deviation of the relative level differences is ±157.0 µm. The low accuracies of the relative three-dimensional distances are due to the "poor" geometry of measurement, and the "poor" definition of the target, ie a 3 mm diameter spherical target is required for the longer of the unequal sight lengths, but this forms too large a target for the shorter of the lengths. The "poor" geometry has less effect on the relative level differences, which are of a similar order of accuracy to precise levelling.

However, if one only considers the points on opposing pillars (P11E - P11W and P13E - P13W), as would be measured in measurement survey PLK1, the standard deviations of the two relative three-dimensional distances are ±227 µm and ±482 µm respectively, and the standard deviations of the two relative level differences are ±59 µm and ±253 µm respectively. Thus, it can be concluded that, even with "poor" geometry, UN ETIS can determine relative level differences to an accuracy of ±150 µm, and relative three-dimensional distances to an accuracy of ±300 µm.

Again, however, it is important to note that if measurement survey PLK2 were to be repeated for consecutive sets of three pillar pairs, along the whole process line, this would require six measurement set ups, and a total of twenty-one hours to perform, compared to the two and a half hours required for the refined precise levelling technique.

7.4 Conclusions

Conventional monitoring of deformation due to mining involves the measurement of absolute ground movement. Measurements are usually performed over long distances, up to 1 km, and accuracies of the order of millimetres are required. Several instruments and techniques capable of performing long-range absolute deformation monitoring have been outlined in this Chapter.

The increased possibility of mining affecting important structures or sensitive industrial processes, and the need to monitor the deformation of underground openings, necessitates short-range relative deformation monitoring to a high order of accuracy.

With reference to a sensitive industrial process, the glass grinding and polishing process at Pilkington Glass Ltd in Doncaster, this Chapter has proposed two techniques for short-range relative deformation monitoring; refined precise levelling and UN ETIS. From the results obtained, the following can be concluded:

(i) The refined precise levelling technique is capable of determining relative levels to an accuracy of better than ± 0.03 cm (± 300 μm).

(ii) With geometrically "good" measurement conditions, measurement survey PLK1, UN ETIS is capable of determining relative level differences, and relative three-dimensional distances to an accuracy of ± 50 μm.

(iii) With geometrically "poor" measurement conditions, measurement survey PLK2, UN ETIS is capable of determining relative

level differences to an accuracy of ±150 μm, and relative three-dimensional distances to an accuracy of ±300 μm.

The results suggest that UN ETIS is the more accurate of the two techniques. It also has a further advantage; it is capable of performing three-dimensional relative deformation monitoring, as opposed to only monitoring relative levels. However, it is important to note that, refined precise levelling can be performed in a fraction of the time required by UN ETIS.

CHAPTER EIGHT

FACTORS AFFECTING ACCURACIES ATTAINABLE WITH ELECTRONIC THEODOLITE INTERSECTION SYSTEMS

Factors Affecting Accuracies Attainable with Electronic Theodolite Intersection Systems

8.1 Introduction

As a general guide, manufacturers usually quote the accuracy of an ETIS based on the claimed precision for a first order electronic theodolite. Taking this precision to be one second of arc the accuracy can be determined to be 1 part in 200 000, ie 50 μm over a distance of 10 m.

In actual fact, the ultimate accuracy attainable with an ETIS is a function of several interrelated factors, which may be divided into six sections:

      (i) System configuration factors.

      (ii) Set-up factors.

      (iii) Measurement factors.

      (iv) Target factors.

      (v) Operator factors.

      (vi) Environmental factors.

This Chapter details the factors as outlined above. It considers their interrelation and the steps which can be taken to reduce, or eliminate, their influence on the ultimate accuracy attainable with an ETIS.

8.2 System Configuration Factors

System configuration factors are generally fixed on purchasing, or developing, a system. There are three components of an ETIS which must be considered; electronic theodolites, computer hardware and computer software.

There is no doubt that the use of high precision electronic theodolites, able to resolve to a fraction of a second of arc, is essential if high accuracy measurements are to be obtained. The internal precision of such theodolites will, however, only be translated into high accuracy measurements if, firstly, the effect of any instrumental errors is eliminated and, secondly, if the pointing and centring of the theodolite reticule on to the target is precise. Details of the checking, determination and adjustment of instrumental errors are given in Chapter Three, section 3.2.6. In addition, it is always advisable to ensure the complete elimination of index, and collimation, errors by systematic measurement, ie complementary face left and face right observations.

The computer hardware and software are considered as fixed components of an ETIS. They do not directly affect the accuracy of coordinate determination, however, they can affect the accuracy of the analysis of measurement data. In order that the accuracy of coordinate determination be carried on through the analysis stage, the measurement data should be stored and output to enable easy interpretation by the operator. The operator, in turn, should be familiar with the intricacies of the software, so that he is capable of utilising the software to its' best ability.

8.3 Set-Up Factors

Set-up factors are variable for a specific measurement survey. There are three factors which must be considered; accuracy of setting up a coordinate system, accuracy of transformation and stability of location. These set-up factors are an important consideration in the pre-planning of a measurement survey as the accuracy of any coordinate determination depends directly on the accuracy to which the coordinates of the control points have been determined.

8.3.1 Accuracy of Setting Up a Coordinate System

The accuracy of setting up a coordinate system is affected by the method of setting up employed, and the accuracy of coordinate determination during setting up, which is obviously affected by the factors discussed in section 8.4.

The two main considerations affecting the accuracy of setting up a coordinate system are; the use of a calibrated scale bar, either in the direct setting up of a coordinate system or as part of the bundle adjustment, and the collineation procedure.

The calibrated scale bar should be located within the measurement space, and geometrically positioned so as to ensure high accuracy in determining the coordinates of the scale bar targets. The scale bar targets, themselves, raise an interesting point, worthy of note; it is generally accepted that three-dimensional targets permit more accurate sighting than plane targets, yet there isn't a calibrated scale bar available which incorporates three-dimensional targets.

As far as collineation is concerned, high accuracy can only be obtained by taking great care during the collineation procedure. Furthermore, it is interesting to note that the bundle adjustment is generally accepted as the most accurate method of setting up a coordinate system, mainly because it removes the need to perform collineation.

8.3.2 Accuracy of Transformation

The accuracy of transformation depends on the geometrical relationship between the control points and transformation points, and the accuracy of coordinate determination during transformation, which is obviously affected by the factors discussed in section 8.4.

The location of the transformation points, if a free choice is available, is critical to the accuracy of transformation. Ideally, the transformation points should be located within the measurement space, and positioned as far apart as possible. However, a compromise often has to be made whereby the points are positioned a reasonable distance apart, whilst maintaining a geometrical position which ensures high accuracy coordinate determination.

8.3.3 Stability of Location

It is fundamentally important to any measurement application, to ensure the stability of the location of the electronic theodolites throughout measurement. The liquid compensators within the instruments will damp out the majority of vibration-type movement, but there can be no corrective action to compensate for an unstable

instrument. Although instability of instruments will not prevent measurement, it will mean that coordinate determination, based on fixed control points, will render meaningless results.

Chapter Three, section 3.4.1, details three types of theodolite stand which may be used as part of an ETIS, namely:

(i) Conventional survey tripod.

(ii) Industrial measurement stand.

(iii) Permanent monument.

From the point of view of stability, the permanent monument is the most stable and the conventional survey tripod the least stable. However, in addition to stability, the portability of the stand has to be taken into account. It is, therefore, the author's opinion that the industrial measurement stand is the most applicable for use in an ETIS as it basically forms a portable, permanent monument.

8.4 Measurement Factors

Measurement factors are variable for every point whose coordinates are determined. There are two factors which must be considered; geometry of measurement and redundancy of measurement.

8.4.1 Geometry of Measurement

The geometry of measurement is governed by the geometrical relationship between the control points and the target point.

The accuracy of coordinate determination is affected by three interrelated geometrical factors, namely:

(i) Baseline length.

(ii) Sight distance from each control point to the target point.

(iii) Intersection angle.

The magnitude of the ellipsoid of error is dependent on these geometrical factors, and the magnitude of any sighting error. It is, therefore, important to establish an optimum geometrical arrangement, prior to measurement, which suits all of the proposed target points.

The baseline length defines the measurement space, and should always be greater than the distance between the extreme proposed target points. Once setting up of a coordinate system is complete, the baseline length remains fixed. However, it is the geometrical relationship between the baseline length and the sight distances which influences the magnitude of the intersection angle subtended at the target point.

Ideally, the intersection angle should be as close to 90 degrees as possible, however, for most measurement applications a wide variation in intersection angles is unavoidable. The permissible intersection angle range suggested by Kern is 60 - 120 degrees [Grist 1986b], and that suggested by Wild is 78 - 142 degrees [Kennie & Petrie 1989].

It is clear that if target points are outside of the measurement space, defined by the baseline, then the intersection angle will very quickly decrease in magnitude and the effect of sighting errors on the magnitude of the ellipsoid of error will correspondingly increase.

8.4.2 Redundancy of Measurement

From Chapter Two, section 2.4.4, it can be seen that the magnitude of the ellipsoid of error is inversely proportional to the square root of the redundacy of measurement $(n - n_0)$, ie

$$\text{Ellipsoid of error} \quad \alpha \quad \frac{1}{\sqrt{n - n_0}}$$

where

n_0 = parameters required for solution.

= 3, ie u_1 and u_2 and $(v_1$ or $v_2)$.

n = total number of observations.

= $2 . N_T . N_O$

where

N_T = number of theodolites used to sight a target point.

N_O = number of observations / target point.

It can be seen, therefore, that increasing either the number of theodolites used to sight a target point, or the number of observations

306

/ target point, increases the redundancy of measurement, which in turn reduces the magnitude of the ellipsoid of error.

In addition, if a multiple theodolite set-up is used then, in measuring large and irregularly shaped objects, a number of theodolites can be set up around an object and the need to re-locate theodolites during a measurement survey is eliminated. However, the increased accuracy and great time savings which can be gained from the use of multiple theodolites have to be balanced against the cost of further theodolites and increased personnel requirements.

8.5 Target Factors

Target factors are variable for every point whose coordinates are determined. There are three factors which must be considered, namely, target selection, location and illumination.

As stated in Chapter Three, section 3.4.2, ideally the target should present an image, the sighting of which gives rise to no discrepancy in the centring of the theodolite reticule. However, in practice this is never achievable and a compromise has to be reached where the target presents a sufficient image at varying distances and from varying directions. The three main types of target detailed in Chapter Three are:

(i)   Plane targets.
(ii) Three-dimensional targets.
(iii) Laser light spot target.

In some measurement applications, it is impractical to position targets on the object itself, because it is inaccessible or in a hazardous environment. In these cases, target selection is governed by the object to be measured, and a laser light spot target has to be used. However, in most measurement applications, targets have to be mounted on an object, and in these cases, target selection is made by the operators.

In the author's experience, there is no general rule governing which target suits a particular application. Target selection should be made in order to optimise the accuracy of coordinate determination, by matching the type, and size, of target to the requirements of the individual target point.

With both plane and three-dimensional targets, the correct illumination of the target to suit all observers is a further critical consideration. This can be achieved by the use of self-illuminable three-dimensional targets or a number of external light sources.

Planning the type, and size, of targets to use for a particular application and where to locate them, if a free choice is available, is crucial to the accuracy of the measurement survey. Furthermore, it should be noted that, this planning may be wasted if target illumination is poor.

8.6 Operator Factors

Operator factors can be considered as variable for every point whose coordinates are determined. There are three factors which

must be considered; circle / ellipse imagination, experience and fatigue. These factors are subjective and are difficult, if not impossible, to quantify.

The pointing and centring of the theodolite reticule on to the target is fundamental to achieving the optimum precision from an ETIS. It is this correcting centring of the theodolite reticule which is affected by the operator factors named above.

The use of small, precisely defined targets by ETISs has created a new problem - circle / ellipse imagination. With plane targets, very often a target which is supposed to present a circular image will present an elliptical image if not viewed from a single direction. On the other hand, poor illumination, or shadowing, of a three-dimensional spherical target can mean the observer is sighting to a partially defined circle. The latter case requires circle imagination and the former case requires ellipse imagination. In both cases, the operator should be aware of this problem, so as the reticule is positioned on the actual target centre and not on the centre of the partially defined image.

The second and third factors, which cannot be overlooked, are experience and fatigue. As with all surveying instrumentation and techniques, the more experienced operator will invariably obtain the highest accuracy measurements. Furthermore, the continuous operation of an electronic theodolite, for long periods of time, requires extreme concentration. If an operator does not allow for fatigue then measurement accuracy will suffer as a consequence.

8.7 Environmental Factors

Environmental factors, including such things as vibration, wind, temperature fluctuations and varying light conditions, are some of the most influential factors affecting the accuracy of an ETIS.

Some of these factors can be controlled, for example, vibration can be controlled by not operating machinery whilst measurements are taking place. However, it is the uncontrolled temperature fluctuations which can cause varying atmospheric refraction conditions to restrict high accuracy measurements.

The effects of vertical refraction over the short sight distances encountered by an ETIS are small, although corrections should still be applied. However, more importantly, there are occasions where appreciable lateral refraction can occur, such as lines of sight which run parallel to structures or machinery, due to horizontal temperature gradients [Russell 1989]. It is, therefore, very important that the presence of such lines of sight is eliminated in the pre-planning of a measurement survey.

8.8 Conclusions

The ultimate accuracy attainable with an ETIS is a function of several interrelated factors, which have been outlined in this Chapter. In a specific measurement survey, the influence of most of these factors can be reduced, or eliminated, by careful pre-planning or systematic measurement techniques.

With each individual measurement survey, careful pre-planning should consider all of the factors detailed in this Chapter, in order to ensure the following:

(i)   A suitable system configuration for the measurement task.   Taking into account the accuracy required of the electronic theodolites, the analysis required by the software and the output required by the hardware.

(ii) High accuracy coordinate system set up and transformation, if required.

(iii) The most stable theodolite set-up, to guarantee the fixed position of the theodolites throughout a measurement survey.

(iv) The baseline is located, taking into account the geometry of measurement, to enable high accuracy coordinate determination to all proposed target points.

(v) The number of theodolites, if a free choice is available, and taking into account the additional cost, maximises the accuracy of coordinate determination.

(vi) Ideal target definition for each proposed target point, by the correct target selection, location and illumination.

(vii) The operators are experienced and capable of performing observations to a high accuracy over a certain period of time.

(viii) The influence of environmental factors is reduced to a minimum.

In the determination of the coordinates of a single target point, systematic measurement techniques can be performed to enhance

the accuracy of coordinate determination, to a certain extent. These techniques include:

(i) Eliminating instrumental errors, by checking and adjusting the electronic theodolites prior to a measurement survey.

(ii) Performing face left and face right observations, to ensure the complete elimination of index and collimation errors.

(iii) Performing multiple observations, to increase the redundacy of measurement and, hence, increase the accuracy of measurement.

Consequently, by paying special attention to the factors outlined in this Chapter, the accuracy of measurement can be improved. Or, conversely, if these factors are ignored, then the accuracy of measurement can be detrimentally affected. It is essential, therefore, that ETIS operators are fully aware of these factors and it is evident that time spent in pre-planning, as in all other survey tasks, will be time well spent.

CHAPTER NINE

CONCLUSIONS

.

Conclusions

This thesis covers all aspects of ETISs:

- it details their principles, to enable a basic understanding of their operation and, as the theoretical basis for the field software developed as part of UN ETIS.

- it details their main components, and the considerations which should be taken into account in purchasing, or developing, a system.

- it details their peripheral components, and the considerations which should be taken into account in their selection for an individual measurement survey.

- it outlines the developments in commercially available systems, and reviews the current, manual and automated, systems and their applications.

- it details the development of UN ETIS, by the author.

- it details the field evaluation of UN ETIS in industrial measurement, for the calibration of production robots.

- it details the field evaluation of UN ETIS for short-range relative monitoring of deformation due to mining.

- it discusses the factors affecting accuracies attainable with ETISs.

All ETISs are based on the same theoretical principles and the same main components; two, or more, electronic theodolites and an on-line computer, with peripheral hardware and suitable software.

An ETIS can be purchased, as a commercially available system, or developed in-house. In both cases, the considerations which should be taken into account in the selection of the main components are; the type of system (manual or automated), the system power source (mains or battery), the suitability of the software and the type and number of theodolites. The first two choices, the type of system and system power source, should be made depending on the nature of all of the proposed measurement tasks. The latter two choices depend on whether an ETIS is being purchased or developed.

If purchasing an ETIS, there are several commercially available systems. These are based around the manufacturers own electronic theodolites and software, which is generally capable of running on a variety of computer hardware and of controlling up to eight electronic theodolites. The specific system purchased depends on the latter two choices, namely, the suitability of the software and the type and number of theodolites.

If developing an ETIS, any electronic theodolites and computer hardware can be used, and in-house software developed to suit specific user requirements.

The development of UN ETIS, by the author, began in October 1987. In order that UN ETIS be flexible for use in a variety of measurement tasks, it has been developed as a system split into two separate halves; field components and office components. The emphasis during development was to create a truely portable, battery powered ETIS capable of performing coordinate determination in the field,

whilst maintaining the ability to perform unlimited analysis of the measured data in the office.

Once the in-house development of the system was complete, this was followed by two independent field evaluations.

The first field evaluation of UN ETIS was in industrial measurement, for the calibration of production robots. The production robot surveys illustrated that a full appreciation of the requirements of calibration, and the limitations of an ETIS is essential, if the high accuracies required are to be achieved, and if these high accuracies are to be useful in calibration. It is now considered that a practical solution to this measurement task, using UN ETIS, has been developed.

The second field evaluation of UN ETIS was for monitoring deformation due to mining. The increased possibility of mining affecting important structures or sensitive industrial processes, and the need to monitor the deformation of underground openings, necessitates short-range relative deformation monitoring to a high order of accuracy. With reference to a sensitive industrial process, the glass grinding and polishing process at Pilkington Glass Ltd in Doncaster, this thesis has proposed two techniques for short-range relative deformation monitoring; refined precise levelling and UN ETIS.

From the field evaluations of UN ETIS, and theoretical tests performed with the software, it is evident that the ultimate accuracy attainable with an ETIS is a function of several interrelated factors. In a specific measurement survey, the influence of most of these factors can be reduced, or eliminated, by careful pre-planning or

systematic measurement techniques. It is essential, therefore, that ETIS operators are fully aware of these factors if the high, theoretical, accuracies are to be obtained in practice.

The major applications of ETISs, to date, have been in the aerospace and automotive industries, telecommunications and robotics. In addition to these applications, the field evaluations of UN ETIS, performed by the author, have underlined the applicability of such systems to performing high accuracy, three-dimensional, short-range relative deformation monitoring.

Although, up to the present time, very few commercially available systems have been sold in the UK, the author believes that the future will see a more widespread use of ETISs due to two reasons:

(i)   The advent of automated systems.

(ii) A further reduction in the cost of electronic theodolites and computers, enabling widespread in-house development of systems for specific applications.

However, one of the obvious conclusions of the thesis is that ETISs have definite limitations, mainly in range, geometry and time. These limitations can restrict the use of such systems, which have to compete directly with instruments and techniques such as close-range photogrammetry, refined precise levelling and sub-millimetre electronic distance measurers, eg Kern Mekometer ME5000.

On completion of the author's research, UN ETIS can be considered as a complete system, although the inherent flexibility of in-

house development will enable refinement and further development to be performed in the future, if required. Future research, therefore, will involve further field evaluations of the system. One such evaluation being, a comparison of UN ETIS and the Kern Mekometer ME5000, recently purchased by the Department, in short-range relative deformation monitoring.

Finally, whilst it is recognised that the use of ETISs in the mining industry is limited, there are certain applications where it is the most suitable technique. It is hoped, therefore, that as a consequence of this research, such systems will be considered alongside other, more conventional, instrumentation and techniques, for the monitoring of deformation due to mining.

ACKNOWLEDGEMENTS

Acknowledgements

APPENDICES

APPENDIX A

LINEARIZATION USING TAYLOR'S THEOREM

Linearization using Taylor's Theorem

The Taylor series expansion for a non-linear function, $y = f(x)$, is

$$y = f(x_0) + f'(x_0)\,\Delta x + \frac{1}{2!}\,f''(x_0)\,\Delta x^2 + \dots \text{ etc}$$

where

x_0 = an approximate value for the variable x.

$f(x_0)$ = the zero order term, evaluated by letting $x = x_0$.

$f'(x_0)$ = the first order term, evaluated by letting $x = x_0$.

$f''(x_0)$ = the second order term, evaluated by letting $x = x_0$.

Δx = $x - x_0$

When the original condition equations are non-linear, they are usually linearized using the zero and first order terms of the Taylor series:

$$y = f(x_0) + f'(x_0)\,\Delta x$$

Extending the concept further, if there are n independent variables and m functions containing all or some of these variables, the above equation may be written as

$$y = y_0 + J\Delta$$

where

y = vector (m x 1) which represents the m functions.

$$= \begin{bmatrix} y_1 \\ y_m \end{bmatrix} \cdot$$

y_0 = vector (m x 1) which represents the m functions with approximate values assigned to the variables.

$$= \begin{bmatrix} y_{01} \\ y_{0m} \end{bmatrix}$$

J = matrix (m x n) of partial derivatives, known as the Jacobian matrix; the elements are evaluated by substituting approximate values for the variables.

$$= \begin{bmatrix} \partial y_1/\partial x_1 & \partial y_1/\partial x_n \\ \partial y_m/\partial x_1 & \partial y_m/\partial x_n \end{bmatrix}$$

Δ = vector (n x 1) of variables.

$$= \begin{bmatrix} \Delta x_1 \\ \Delta x_n \end{bmatrix}$$

APPENDIX B

MATRICES FOR THE LEAST SQUARES ADJUSTMENT OF COORDINATES

Matrices for the Least Squares Adjustment of Coordinates

To enable a least squares solution to be applied to the raw coordinates derived by three-dimensional spatial intersection, equations 2.2 and 2.3 have to be linearized by Taylor's Theorem as detailed in Appendix A.

Considering the coordinates as being calculated from two variables u and v, equations 2.2 and 2.3 may be expressed as the non-linear functions

$$f_1 = t \cdot \cos v \cdot \sin u - x = 0$$
$$f_2 = t \cdot \cos v \cdot \cos u - y = 0$$
$$f_3 = t \cdot \sin v - z = 0$$

Equation B.1

The Taylor expansion defines

$$y = y_0 + J \Delta$$

Equation B.2

which may be written as

$$Y = J \Delta$$

Equation B.3

where

$$Y = y - y_0$$
$$= \text{vector of discrepancies in coordinates.}$$

$$= \begin{bmatrix} x - x_0 \\ y - y_0 \\ z - z_0 \end{bmatrix}$$

$$= \begin{bmatrix} \Delta x \\ \Delta y \\ \Delta z \end{bmatrix}$$

J = matrix of partial derivatives of equation B.1.

$$= \begin{bmatrix} \partial f_1/\partial v & \partial f_1/\partial u \\ \partial f_2/\partial v & \partial f_2/\partial u \\ \partial f_3/\partial v & \partial f_3/\partial u \end{bmatrix}$$

$$= \begin{bmatrix} -t \sin v \sin u & t \cos v \cos u \\ -t \sin v \cos u & -t \cos v \sin u \\ t \cos v & 0 \end{bmatrix}$$

Δ = vector of variables.

$$= \begin{bmatrix} \Delta v \\ \Delta u \end{bmatrix}$$

Considering equation B.3 the matrices J and Δ are simplified to

$$J = \begin{bmatrix} -\sin v \sin u & \cos u \\ -\sin v \cos u & -\sin u \\ \cos v & 0 \end{bmatrix}$$

$$\Delta = \begin{bmatrix} t \cdot \Delta v \\ t \cdot \cos v \cdot \Delta u \end{bmatrix}$$

However, the requirement of the least squares adjustment of three-dimensional coordinates obtained by spatial intersection is to determine corrections to the raw coordinates Δx_P, Δy_P, Δz_P. Therefore, the three matrices Y, J and Δ have to be re-defined. This is performed by multiplying equation B.3 by J^T, ie

$$J^T Y = J^T J \Delta$$
$$J^T Y = \Delta$$

The matrices for the least squares adjustment d, A and c are defined for the equation

$$d = A c$$

where

$$A = J^T$$

= matrix of the coefficients of the unknown coordinate parameters.

$$= \begin{bmatrix} -\sin v_1 \cdot \sin u_1 & -\sin v_1 \cdot \cos u_1 & \cos v_1 \\ \cos u_1 & -\sin u_1 & 0 \\ -\sin v_2 \cdot \sin u_2 & -\sin v_2 \cdot \cos u_2 & \cos v_2 \\ \cos u_2 & -\sin u_2 & 0 \end{bmatrix}$$

$$d = \Delta$$

= vector of the differences between the observed and computed observations.

$$= \begin{bmatrix} t_1 \cdot \Delta v_1 \\ t_1 \cdot \cos v_1 \cdot \Delta u_1 \\ t_2 \cdot \Delta v_2 \\ t_2 \cdot \cos v_2 \cdot \Delta u_2 \end{bmatrix}$$

$c = Y$

= vector of corrections to the approximate coordinate parameters.

$$= \begin{bmatrix} \Delta x_P \\ \Delta y_P \\ \Delta z_P \end{bmatrix}$$

APPENDIX C

PROBABLE COORDINATE ERROR DETERMINATION

Probable Coordinate Error Determination

Probable coordinate errors are calculated as the theoretical minimum errors obtainable with a certain geometric set up and are based on instrumental accuracy alone. Instrumental accuracy is given as

δu = theoretical angle error of a horizontal angle.

δv = theoretical angle error of a vertical angle.

Considering a two control point set up there are sixteen possible combinations of theoretical angle error, namely

P	δu_1	δv_1	δu_2	δv_2	P	δu_1	δv_1	δu_2	δv_2
1	+	+	+	+	9	-	-	-	-
2	+	+	-	+	10	-	-	+	-
3	+	-	+	+	11	-	+	-	-
4	+	-	-	+	12	-	+	+	-
5	+	+	+	-	13	-	-	-	+
6	+	+	-	-	14	-	-	+	+
7	+	-	+	-	15	-	+	-	+
8	+	-	-	-	16	-	+	+	+

For each coordinate axis, if the sixteen angle error combinations are considered as equally probable then the probable coordinate error is obtained as the mean of the coordinate errors, irrespective of the sign of the coordinate error, ie considering the x-axis

$$P_x = (\Sigma \, [\, |\delta x_{Pi}| \,]) / 16$$

where

i = angle error point combinations 1 .. 16.

However, it can be shown that for all cases, if the signs of all four angle errors are reversed then the signs of the three coordinate errors are reversed, eg

$$\delta x_{P1} = -\delta x_{P9}$$

Thus computation can be reduced to consider the first eight error points only, ie

$$Px = (\Sigma\ [\ |\delta x_{Pi}|\ ])\ /\ 8$$

where

i = angle error point combinations 1 .. 8.

Further, it can also be shown that for all cases

$$(\Sigma\ [\ |\delta x_{Pi}|\ ]) = (\Sigma\ [\ |\delta x_{Pj}|\ ])$$

where

i = angle error point combinations 1, 2, 3 and 4.
j = angle error point combinations 5, 6, 7 and 8.

therefore, for ease of computation the probable errors can be calculated considering the four angle error point combinations 1, 2, 3 and 4, namely

P	δu_1	δv_1	δu_2	δv_2
1	+	+	+	+
2	+	+	-	+
3	+	-	+	+
4	+	-	-	+

The probable coordinate errors are calculated by equation 2.19 as follows

$$Px = \frac{|2 . \delta x_{P1}| + |2 . \delta x_{P2}| + |2 . \delta x_{P3}| + |2 . \delta x_{P4}|}{8}$$

$$Py = \frac{|2 . \delta y_{P1}| + |2 . \delta y_{P2}| + |2 . \delta y_{P3}| + |2 . \delta y_{P4}|}{8}$$

$$Pz = \frac{|2 . \delta z_{P1}| + |2 . \delta z_{P2}| + |2 . \delta z_{P3}| + |2 . \delta z_{P4}|}{8}$$

Equation 2.19

APPENDIX D

PROBABLE ERRORS IN CALCULATED SLOPE
DISTANCES

Probable Errors in Calculated Slope Distances

The probable errors in the calculated slope distances t_1 and t_2 are determined from equations 2.12 and 2.13

$$t_1 = \frac{(p - q \cdot \cos c)}{\sin^2 c} \qquad \text{Equation 2.12}$$

$$t_2 = \frac{(p \cdot \cos c - q)}{\sin^2 c} \qquad \text{Equation 2.13}$$

For both distances the following errors are present

$$\delta p = (x_2 - x_1) \cdot \delta l_1 + (y_2 - y_1) \cdot \delta m_1 + (z_2 - z_1) \cdot \delta n_1$$

$$\delta q = (x_2 - x_1) \cdot \delta l_2 + (y_2 - y_1) \cdot \delta m_2 + (z_2 - z_1) \cdot \delta n_2$$

$$\delta c = \cos^{-1}(\cos (c + \delta c)) - \cos^{-1}(\cos c)$$

where

$$\cos c = l_1 \cdot l_2 + m_1 \cdot m_2 + n_1 \cdot n_2$$

$$\cos (c + \delta c) = (l_1 + \delta l_1) \cdot (l_2 + \delta l_2) +$$
$$(m_1 + \delta m_1) \cdot (m_2 + \delta m_2) +$$
$$(n_1 + \delta n_1) \cdot (n_2 + \delta n_2)$$

Considering equation 2.12 for distance t_1

$$\delta t_1 = \frac{\partial t_1}{\partial p} \cdot \delta p + \frac{\partial t_1}{\partial q} \cdot \delta q + \frac{\partial t_1}{\partial c} \cdot \delta c$$

$$\frac{\partial t_1}{\partial p} = \frac{1}{\sin^2 c}$$

$$\frac{\partial t_1}{\partial q} = \frac{-\cos c}{\sin^2 c}$$

$$\frac{\partial t_1}{\partial c} = \frac{\partial (ab)}{\partial c} + \frac{\partial e}{\partial c}$$

where

$a = -q \cdot \cos c$

$b = 1 / \sin^2 c$

$\quad = \csc^2 c$

$e = p / \sin^2 c$

$\quad = p \cdot \csc^2 c$

$$\frac{\partial (ab)}{\partial c} = a \cdot \frac{\partial b}{\partial c} + b \cdot \frac{\partial a}{\partial c}$$

$$= -q \cdot \cos c \cdot (-2 \cdot \csc^2 c \cdot \cot c)$$

$$+ \csc^2 c \cdot (q \cdot \sin c)$$

$$= q \cdot \csc^2 c \cdot (2 \cdot \cos c \cdot \cot c + \sin c)$$

$$= q \cdot \csc^2 c \cdot ((1 + \cos^2 c) / \sin c)$$

$$\frac{\partial e}{\partial c} = p \cdot (-2 \cdot \csc^2 c \cdot \cot c)$$

$$\frac{\partial t_1}{\partial c} = q \cdot \csc^2 c \cdot ((1 + \cos^2 c) / \sin c)$$

$$+ (-2 \cdot p \cdot \csc^2 c \cdot \cot c)$$

$$= \frac{((q + q \cdot \cos^2 c - 2 \cdot p \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

Therefore

336

$$\delta t_1 = \frac{\delta p}{\sin^2 c} - \frac{\cos c \cdot \delta q}{\sin^2 c} +$$
$$\frac{((q + q \cdot \cos^2 c - 2 \cdot p \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

<div align="right">Equation 2.23</div>

Considering equation 2.13 for distance t_2

$$\delta t_2 = \frac{\partial t_2}{\partial p} \cdot \delta p + \frac{\partial t_2}{\partial q} \cdot \delta q + \frac{\partial t_2}{\partial c} \cdot \delta c$$

$$\frac{\partial t_2}{\partial p} = \frac{\cos c}{\sin^2 c}$$

$$\frac{\partial t_2}{\partial q} = \frac{-1}{\sin^2 c}$$

$$\frac{\partial t_2}{\partial c} = \frac{\partial (ab)}{\partial c} + \frac{\partial e}{\partial c}$$

where

$a = p \cdot \cos c$

$b = 1 / \sin^2 c$

$\quad = \mathrm{cosec}^2 c$

$e = -q / \sin^2 c$

$\quad = -q \cdot \mathrm{cosec}^2 c$

$$\frac{\partial (ab)}{\partial c} = a \cdot \frac{\partial b}{\partial c} + b \cdot \frac{\partial a}{\partial c}$$
$$= p \cdot \cos c \cdot (-2 \cdot \mathrm{cosec}^2 c \cdot \cot c)$$
$$+ \mathrm{cosec}^2 c \cdot (-p \cdot \sin c)$$
$$= -p \cdot \mathrm{cosec}^2 c \cdot (2 \cdot \cos c \cdot \cot c + \sin c)$$

$$= -p \cdot \text{cosec}^2 c \cdot ((1 + \cos^2 c) / \sin c)$$

$$\frac{\partial e}{\partial c} = -q \cdot (-2 \cdot \text{cosec}^2 c \cdot \cot c)$$

$$\frac{\partial t_2}{\partial c} = -p \cdot \text{cosec}^2 c \cdot ((1 + \cos^2 c) / \sin c)$$

$$+ (2 \cdot q \cdot \text{cosec}^2 c \cdot \cot c)$$

$$= -\frac{((p + p \cdot \cos^2 c - 2 \cdot q \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

Therefore

$$\delta t_2 = \frac{\partial p}{\partial t_2} \cdot \delta p + \frac{\partial q}{\partial t_2} \cdot \delta q + \frac{\partial c}{\partial t_2} \cdot \delta c$$

$$= \frac{\cos c \cdot \delta p}{\sin^2 c} - \frac{\delta q}{\sin^2 c} - \frac{((p + p \cdot \cos^2 c - 2 \cdot q \cdot \cos c) / \sin c) \cdot \delta c}{\sin^2 c}$$

APPENDIX E

THREE-DIMENSIONAL ROTATION MATRIX

Three-dimensional Rotation Matrix

The three rotation angle ω, ϕ and κ form a rotation matrix M as follows

$$M = \begin{bmatrix} M_{11} & M_{12} & M_{13} \\ M_{21} & M_{22} & M_{23} \\ M_{31} & M_{32} & M_{33} \end{bmatrix}$$

$M_{11} = \cos\phi . \cos\kappa$

$M_{12} = \sin\omega . \sin\phi . \cos\kappa + \cos\omega . \sin\kappa$

$M_{13} = -\cos\omega . \sin\phi . \cos\kappa + \sin\omega . \sin\kappa$

$M_{21} = -\cos\phi . \sin\kappa$

$M_{22} = -\sin\omega . \sin\phi . \sin\kappa + \cos\omega . \cos\kappa$

$M_{23} = \cos\omega . \sin\phi . \sin\kappa + \sin\omega . \cos\kappa$

$M_{31} = \sin\phi$

$M_{32} = -\sin\omega . \cos\phi$

$M_{33} = \cos\omega . \cos\phi$

APPENDIX F

COMPONENTS OF THE B MATRIX IN THE
TRANSFORMATION SOLUTION

Components of the B Matrix in the Transformation Solution

The B matrix is defined on linearization by Taylor's Theorem as detailed in Appendix A. The B matrix is formed of the partial derivatives of the condition equations (equation 2.25).

$$B = \begin{bmatrix} \partial f_1/\partial\lambda & \partial f_1/\partial\omega & \partial f_1/\partial\phi & \partial f_1/\partial\kappa & \partial f_1/\partial X_T & \partial f_1/\partial Y_T & \partial f_1/\partial Z_T \\ \partial f_2/\partial\lambda & \partial f_2/\partial\omega & \partial f_2/\partial\phi & \partial f_2/\partial\kappa & \partial f_2/\partial X_T & \partial f_2/\partial Y_T & \partial f_2/\partial Z_T \\ \partial f_3/\partial\lambda & \partial f_3/\partial\omega & \partial f_3/\partial\phi & \partial f_3/\partial\kappa & \partial f_3/\partial X_T & \partial f_3/\partial Y_T & \partial f_3/\partial Z_T \end{bmatrix}$$

Taking M to be the transformation matrix compiled from the three rotations ω, ϕ and κ, as detailed in Appendix E, the partial derivatives of the B matrix are evaluated as follows [Moffitt & Mikhail 1980]

$$\partial/\partial\lambda \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = -M \begin{bmatrix} x \\ y \\ z \end{bmatrix}$$

$$\partial/\partial\omega \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = -\lambda M \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 1 \\ 0 & -1 & 0 \end{bmatrix} \begin{bmatrix} x \\ y \\ z \end{bmatrix}$$

$$\partial/\partial\phi \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = -\lambda \begin{bmatrix} 0 & 0 & -\cos\kappa \\ 0 & 0 & \sin\kappa \\ \cos\kappa & -\sin\kappa & 0 \end{bmatrix} M \begin{bmatrix} x \\ y \\ z \end{bmatrix}$$

$$\partial/\partial\kappa \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = -\lambda \begin{bmatrix} 0 & 1 & 0 \\ -1 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} M \begin{bmatrix} x \\ y \\ z \end{bmatrix}$$

$$\partial/\partial X_T \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = \begin{bmatrix} -1 \\ 0 \\ 0 \end{bmatrix}$$

$$\partial/\partial Y_T \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = \begin{bmatrix} 0 \\ -1 \\ 0 \end{bmatrix}$$

$$\partial/\partial Z_T \begin{bmatrix} f_1 \\ f_2 \\ f_3 \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ -1 \end{bmatrix}$$

APPENDIX G

UN ETIS MEASUREMENT SURVEY RBT301 - ALL AXES
MOVEMENT OF DEFORMATION POINTS B - F

BEST COPY

AVAILABLE
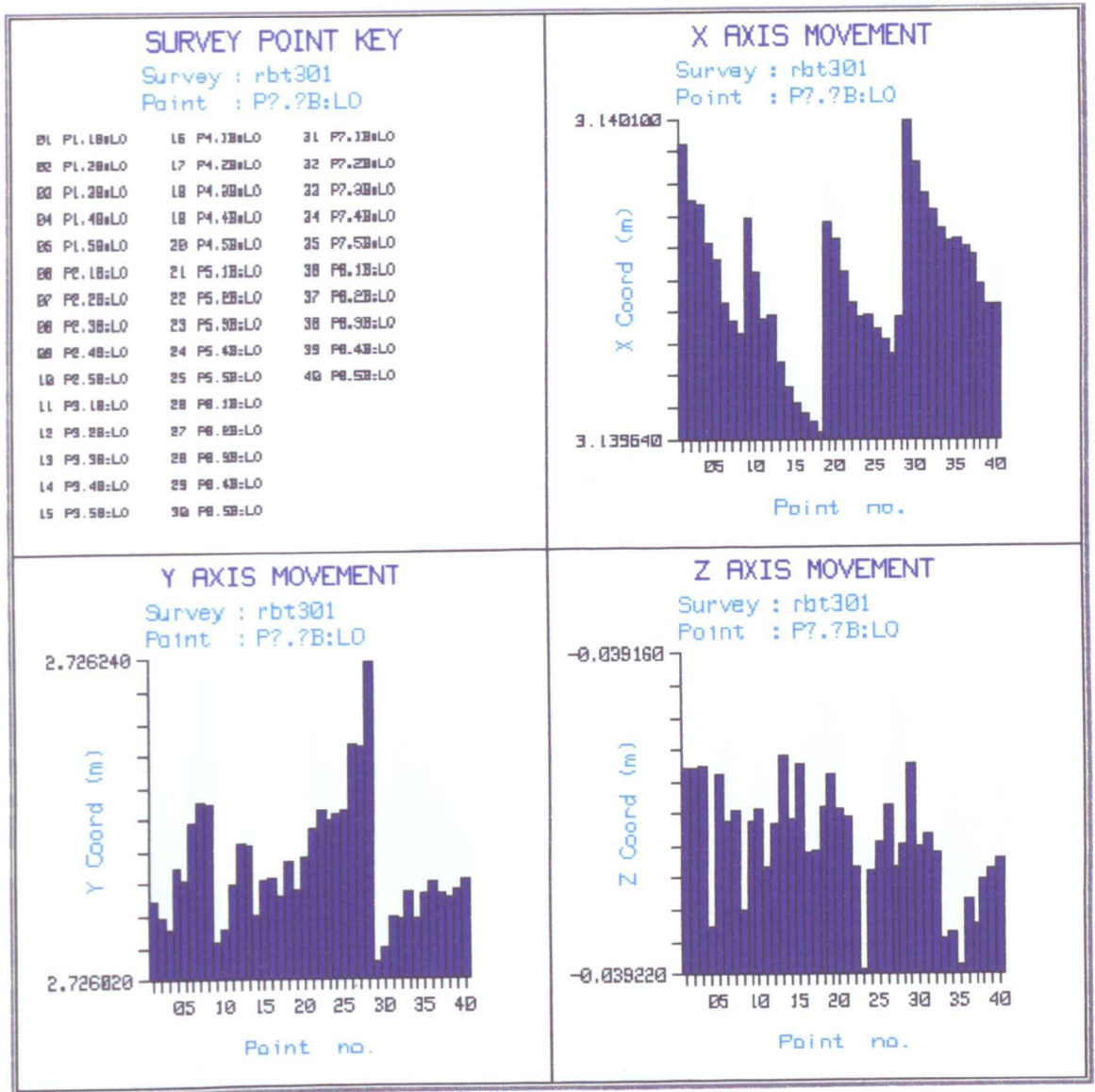
Some text bound close to the spine.

Figure G.1 UN ETIS Measurement Survey RBT301 - All Axes
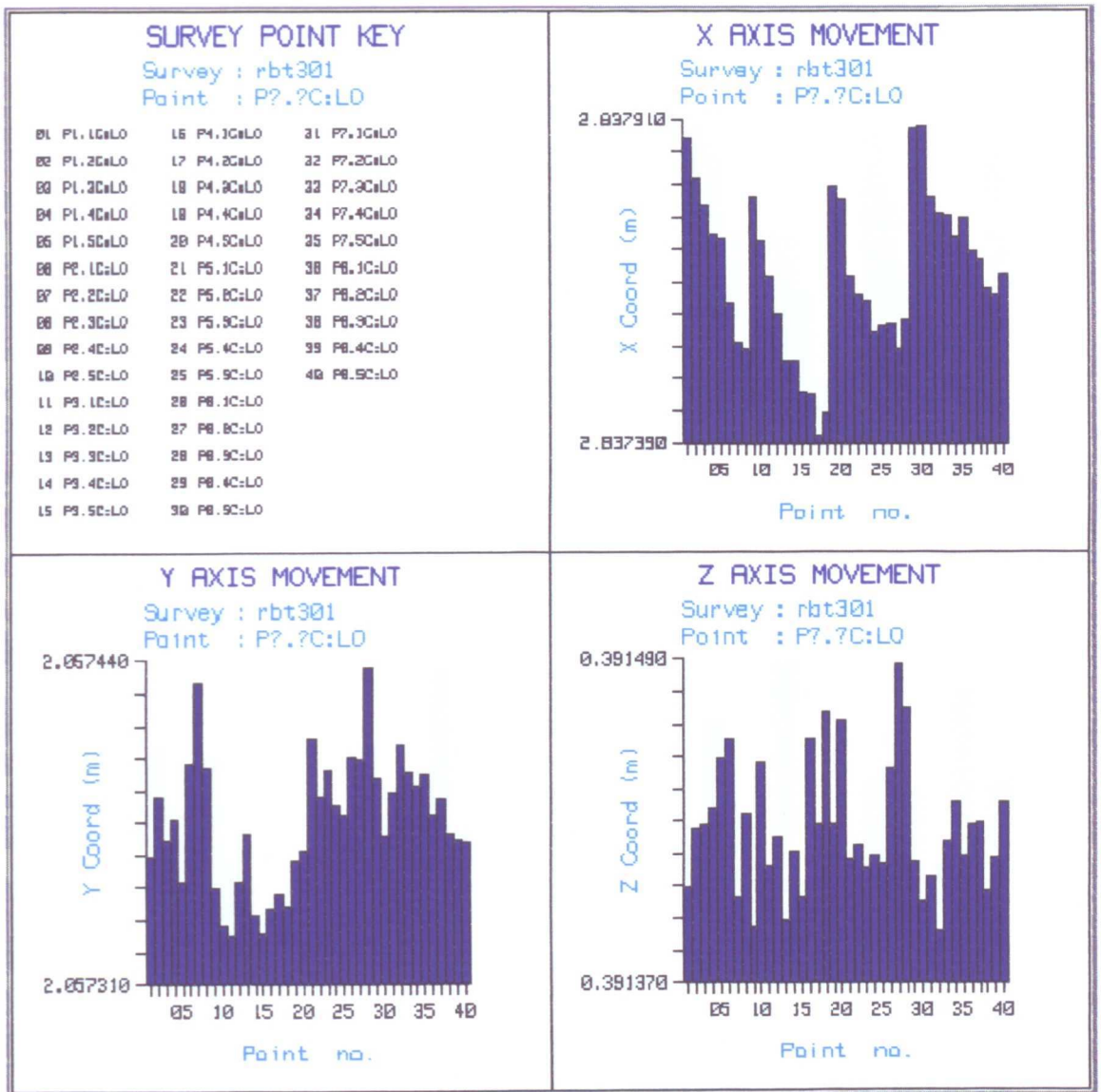Movement of Deformation Point B.

Figure G.2 UN ETIS Measurement Survey RBT301 - All Axes
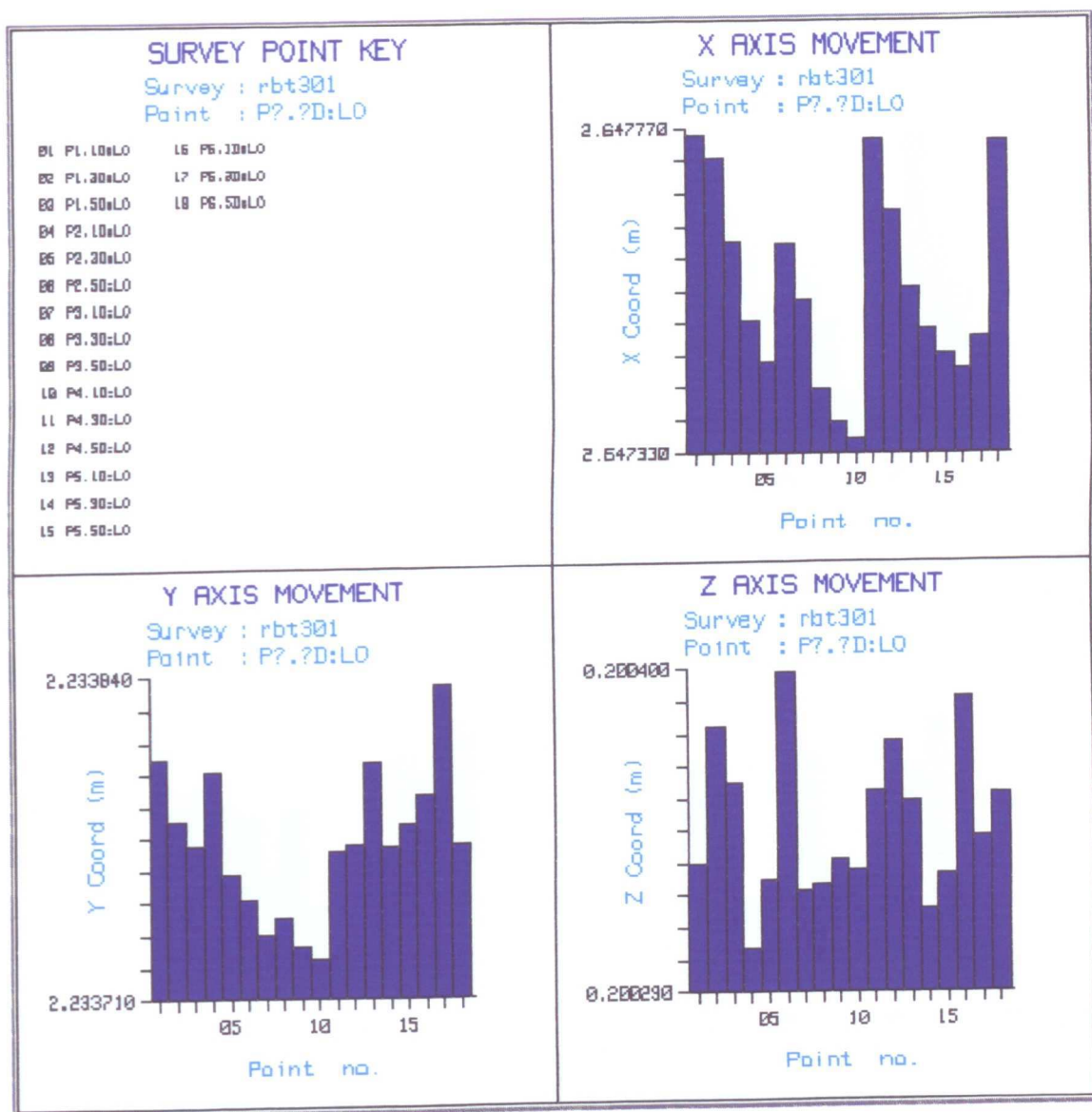Movement of Deformation Point C.

Figure G.3 UN ETIS Measurement Survey RBT301 - All Axes
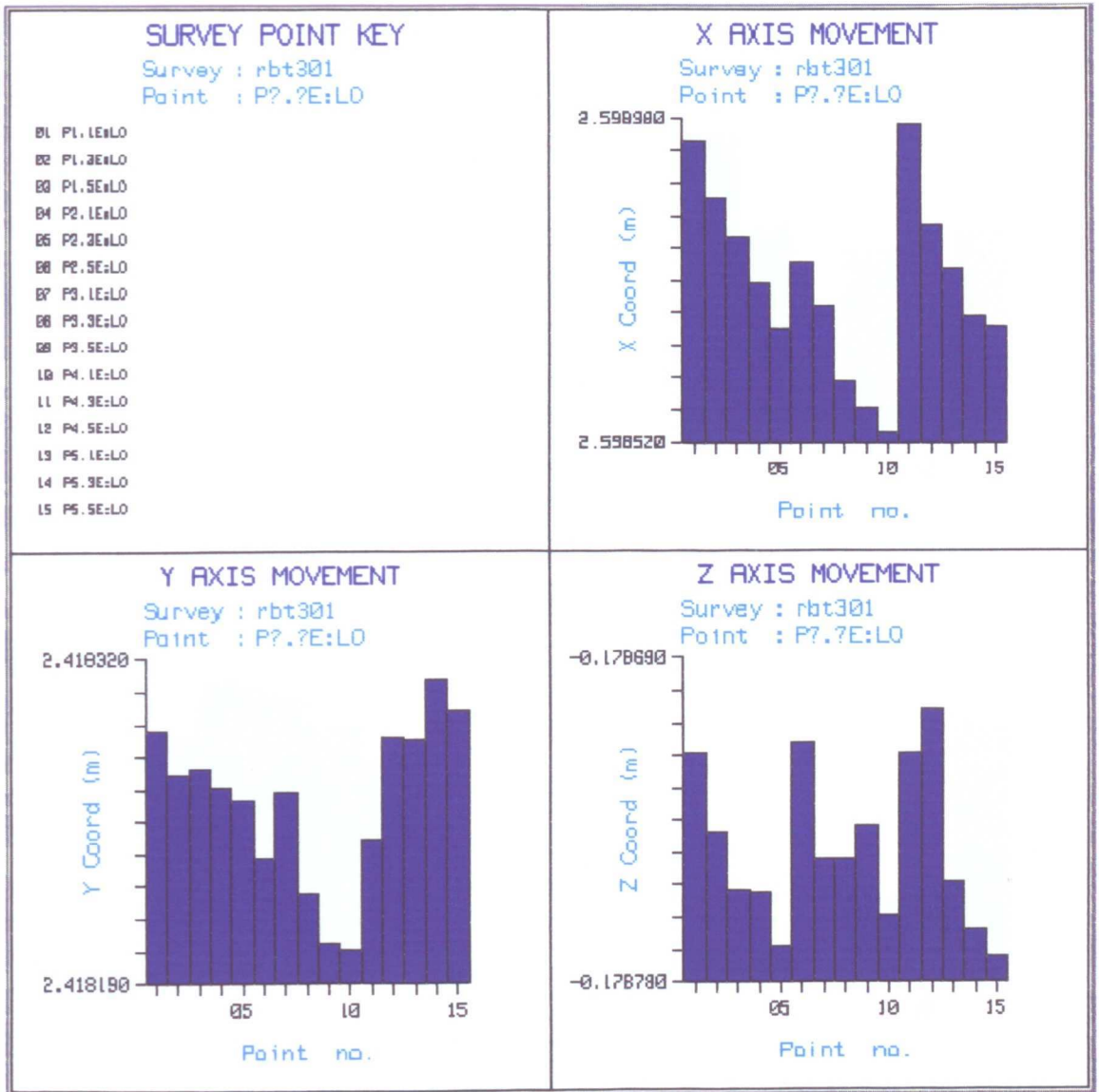Movement of Deformation Point D.

Figure G.4 UN ETIS Measurement Survey RBT301 - All Axes
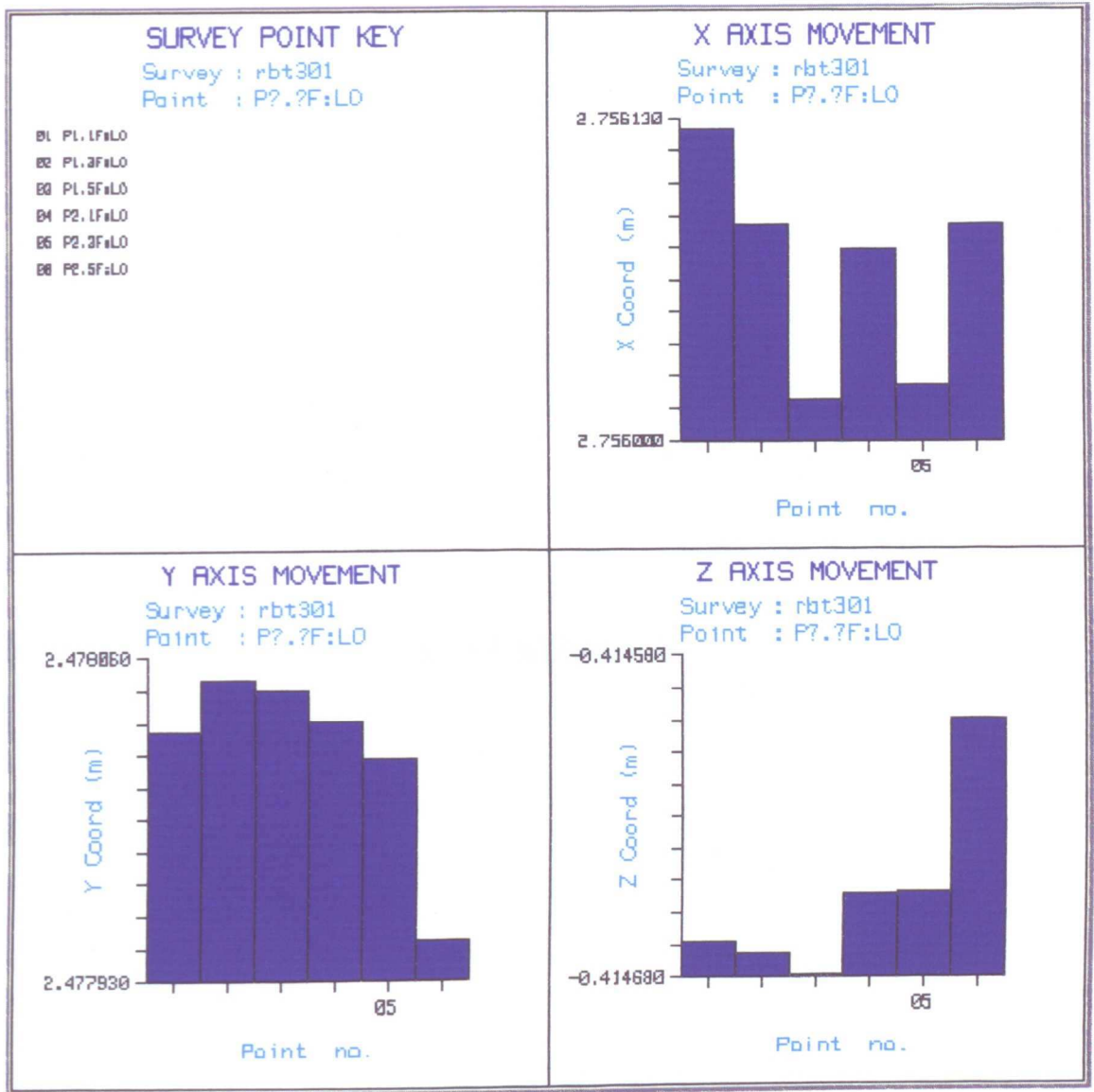Movement of Deformation Point E.

Figure G.5 UN ETIS Measurement Survey RBT301 - All Axes
Movement of Deformation Point F.

REFERENCES

References

Allan, A L, 1987.

"The Principles of Theodolite Intersection Systems".

Proceedings of the 2nd FIG / ISPRS Industrial and Engineering Survey Conference, London, 2 - 4 September 1987.

Bingley, R M, 1987.

"An Evaluation and Comparison of Three Electronic Total Stations".

BEng thesis, University of Nottingham.

Cooper, M A R, 1974.

"Fundamentals of Survey Measurement and Analysis".

Granada, London.

Cooper, S M, 1988.

"Computer Aided Mine Surveying and Planning".

PhD thesis, University of Nottingham.

Cross, P A, 1988.

"Surveying into the 1990's: New Standards and Accuracies ?".

Proceedings of the Institute of Civil Engineering Surveyors Standards and Accuracies Conference, Reading 1988.

Denby, B, 1979.

"Levelling with the Wild NA2 Level".

BSc thesis, University of Nottingham.

Diconix, 1987.

"Diconix Inkjet Printer Model 150 Operator Manual".

Diconix Inc, Dayton, Ohio, USA, 1987.

Ffoulkes-Jones, G H, 1987.

"GPS: The Current Position on Positioning".

The Institution of Civil Engineers Medal and Premium (Universities) Competition, 12 May, 1987.

Foley, A, 1988.

Personal Communication.

Department of Production Engineering, University of Nottingham.

Foley, A, 1991.

To be published.

PhD thesis, University of Nottingham.

Fort, M J, 1987.

"The Optimal Way".

Civil Engineering Surveyor, Vol XII, Issue 8, October 1989.

Freeman, S D, 1987.

"New Perspectives in 3-D Theodolite-Based Industrial Measuring Systems".

Quality Today, May 1987.

Gottwald, R & Berner, W, 1987.

"The New Kern System for Positioning and Automated Coordinate Evaluation - Advanced Technology for Automated 3-D Coordinate Determination".

Technical papers, ACSM-ASPRS Annual Convention, Baltimore, Maryland, USA.

Grist, M W, 1986a.

"Kern ECDS 1 Course Notes".

Optical & Mechanical Development.

Grist, M W, 1986b.

"Real-Time Spatial Coordinate Measuring Systems".

Land and Minerals Surveying, Vol 4, No 9, September 1986.

Hodges, D J & Scoble, M J, 1974.

"A Survey Network for Test Purposes at Llangollen".

Mining Department Magazine, Vol No XXVI.

Hodges, D J, Johnson, S M & Gordon, T A, 1990.

"Correlation of Mine Surveys with the National Grid".

Mining Department Magazine, Vol No XLII.

Hodges, D J & Cooper, S M, 1988.

"Automated Three-Dimensional Subsidence Monitoring".

The Mining Engineer, No 316, Vol 147, January 1988.

Husky, 1984.

"Husky Hunter Operations Manual".

Husky Computers Ltd, September 1984.

Husky, 1987.

"Husky Oracle Disk Drive Reference Manual".

Husky Computers Ltd, August 1987.

Katowski, O & Salzmann, W, 1983.

"The Angle Measurement System in the Wild Theomat T2000".

Wild Heerbrugg Ltd, Switzerland, October 1983.

Katowski, O, 1985.

"RMS2000 - A System for Non-Contact Measurements of Large Objects".

Wild Heerbrugg Ltd, Switzerland, May 1985.

Katowski, O, 1987.

"Automation of Electronic Angle-Measuring Instruments".

Proceedings of the 2nd FIG / ISPRS Industrial and Engineering Survey Conference, London, 2 - 4 September 1987.

Katowski, O & Huep, W, 1988.

"Theodolite Measuring Systems for Industrial and Geodetic Applications".

Wild Heerbrugg Ltd, Switzerland, 1988.

Katowski, O, 1989.

"Instruments and Systems for the Surveillance of Man-made Structures and of Areas Subject to Geophysical Movement".

Wild Heerbrugg Ltd, Switzerland, 1989.

Kennie, T J M & Petrie, G, 1989.

"Engineering Surveying Technology".

Blackie, USA: Halsted Press.

Kern, 1984.

"Kern E2 Electronic One-Second Theodolite Instruction Manual".

Kern & Co Ltd, Aarau, Switzerland, December 1984.

Ketteman, M R, 1985.

"The Applications of Automated Mine Survey Systems to Mine Surveying Practice".

PhD thesis, University of Nottingham.

Lardelli, A, 1984.

"ECDS 1 - An Electronic Coordinate Determination System for Industrial Applications".

Kern & Co Ltd, Aarau, Switzerland, September 1984.

Lardelli, A, 1988.

"ECDS2 A Mobile 3D-Measuring System".

Geodetical Info Magazine, Vol 2, No 2, February 1988.

Leitch, J G, Magill, N & McIlwrath, J C, 1988.
"Comparison of Kern DCS and Wild Mini RMS Systems in a Shipbuilding Environment".
Publication of the Teaching Company Scheme between Harland & Wolff and Queen's University of Belfast, February / March 1988.

Leitz, 1986.
"Leitz PMM - 3-Coordinate CNC Precision Measuring Machines".
Ernst Leitz, Wetzlar, West Germany, August 1986.

Methley, B D F, 1986.
"Computational Models in Surveying and Photogrammetry".
Blackie, Glasgow & London.

Mikhail, E M, 1976.
"Observations and Least Squares".
Dun-Donnelley, New York, USA.

Moffitt, F H & Mikhail, E M, 1980.
"Photogrammetry".
Harper & Row, New York, USA.

Maurer, W, 1985.
"Kern E2: Accuracy Analysis of the Graduated Circle".
Kern Bulletin 37, Kern & Co Ltd, Aarau, Switzerland, 1985.

Munch, K H, 1984.
"The Kern E2 Electronic Precision Theodolite".
Proceedings of the 17th FIG Congress (Sofia), Commission 5.

Parker, G A & Mayer, R, 1987.

"The Use of Non-Contacting Measurement Techniques in Robot Performance Measurement".

Proceedings of the 2nd FIG / ISPRS Industrial and Engineering Survey Conference, London, 2 - 4 September 1987.

Price, W F, 1989.

"Coordinate Measuring Systems".

Civil Engineering Surveyor, Vol XIV, Issue 4 (Supplement), May 1989.

Roberts, T P & Moffitt, N M, 1987.

"Kern System for Positioning and Automated Coordinate Evaluation - A Real-Time System for Industrial Measurement".

Technical papers, ACSM-ASPRS Annual Convention, Baltimore, Maryland, USA.

Russell, G A, 1989.

"Investigations into Lateral Refraction".

MSc thesis, University of Nottingham.

Schofield, W, 1980.

"Errors in Engineering Traverses and the Effects of their Adjustment".

MPhil thesis, University of Nottingham.

Sommerville, D M Y, 1934.

"Analytical Geometry of Three-Dimensions".

University Press, Cambridge, UK.

Thompson, P, 1987.

"DCS-200 - Dimensional Control System".

Civil Engineering Surveyor, Vol XII, Issue 6, July / August 1987.

Wild, 1989a.

"TMS Theodolite Measurement System".

Wild Heerbrugg Ltd, Switzerland, September 1989.

Wild, 1989b.

"ATMS Automated Theodolite Measurement System".

Wild Heerbrugg Ltd, Switzerland, September 1989.

Woodward, C A W, 1987.

"Practical Experiences with the Wild-Leitz RMS2000 System".

Proceedings of the 2nd FIG / ISPRS Industrial and Engineering Survey
Conference, London, 2 - 4 September 1987.

Zhou, G, 1986.

"Accurate Determination of Ellipse Centers in Digital Imagery".

Technical papers, ACSM-ASPRS Spring Convention, Washington DC,
USA.

Zhou, G & Roberts, T P, 1987.

"Camera Calibration for Kern E2-SE Automatic Theodolite".

Technical papers, ACSM-ASPRS Annual Convention, Baltimore,
Maryland, USA.

BIBLIOGRAPHY

Bibliography

Anon, 1989.

"Theodolites Play Vital Role in Kew's Palm House Restoration".

Article in the Civil Engineering Surveyor, Vol XIV, Issue 2, May 1989.

Aeschlimann, H, 1987.

"Instrumentation for Deformation Measurement".

Geodetical Info Magazine, Vol 1, No 8, December 1987.

Hawksley, C, 1983.

"Pascal Programming: A Beginner's Guide to Computers and Programming".

Cambridge University Press, Cambridge.

Hirvonen, R A, 1971.

"Adjustment by Least Squares in Geodesy and Photogrammetry".

Frederick Ungar Publishing Co, New York, USA.

Hodges, D J, Johnson, S M & Bingley, R M, 1988.

"Present and Future Trends in Mine Surveying".

Mining Magazine, Vol 159, No 1, July 1988.

Hodges, D J, Johnson, S M & Bingley, R M, 1989.

"Application of Computers in Minerals Surveying".

IAEG Annual Review 1989.

Kern, 1987.

"The Mobile 3D Measuring System ECDS2: Taking Measure!".

Kern & Co Ltd, Aarau, Switzerland, February 1987.

Lardelli, A, 1985.

"ECDS1: A Mobile 3D-Measuring System - The New Standard for Metrology and Quality Control".

Kern & Co Ltd, Aarau, Switzerland, June 1985.

McIlwrath, J, 1988.

"Processing of Observed Data".

Civil Engineering Surveyor, Vol XIII, Issue 10, January 1988.

Milne, P H, 1984.

"BASIC Programs for Land Surveying".

E & F N Spon Ltd, London.

Morgan, R & McGilton, H, 1987.

"Introducing UNIX System V".

McGraw-Hill Book Company, New York, USA.

Optimal, 1985.

"Industrial Measurement - Spatial Coordinate System DCS-200".

Optimal Software Ltd, February 1985.

Rohde, M W, 1988.

"Automated Data Acquisition for Engineering and Deformation Surveys Using Electronic Theodolites".

Proceedings of the 5th International (FIG) Symposium on Deformation Measurement, Fredericton, Canada, 6 - 9 June 1988.

Schneider, M G, 1982.

"An Introduction to Programming and Problem Solving with Pascal".

John Wiley & Sons, New York, USA.

Shepherd, F A, 1981.

"Advanced Engineering Surveying - Problems and Solutions".

Edward Arnold, London.

Shepherd, F A, 1983.

"Engineering Surveying - Problems and Solutions (2nd Edition)".

Edward Arnold, London.

Sobell, M G, 1985.

"A Practical Guide to UNIX System V".

The Benjamin/Cummings Publishing Company Inc, California, USA.

Uren, J, 1989.

"A Land Surveyor in a Structural World".

Civil Engineering Surveyor, Vol XIV, Issue 8, October 1989.

Wild, 1988.

"Wild Heerbrugg Presents TMS and ATMS Measuring Systems".

Geodetical Info Magazine, Vol 2, No 10, October 1988.