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Development of EDMI Calibration Baseline

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ABSTRACT

Electronic distance measuring instruments (EDMI) are used by surveyors in routine length measurements. The constant and scale factors of the instrument tend to change due to usage, transportation, and aging of crystals. Calibration baselines are established to enable surveyors to check the instruments and determine any changes in the values of constant and scale factors. The National Geodetic Survey (NGS) has developed guidelines for establishing these baselines.

In 1981 an EDMI baseline at ISU was established according to NGS guidelines. In October 1982, the NGS measured the distances between monuments.

Computer programs for reducing observed distances were developed. Mathematical model and computer programs for determining constant and scale factors were developed. A method was developed to detect any movements of the monuments. Periodic measurements of the baseline were made. No significant movement of the monuments was detected.

DEVELOPMENT OF EDMI CALIBRATION BASELINE

1. INTRODUCTION

Electronic distance measuring instruments (EDMI) are used by surveyors in routine measurements of lines varying between 100 feet to two miles or even more. Modern EDMI are of the solid state type and therefore, their electronic components are stable. However, due to usage, transportation, and the aging of crystals, the constant and scale factors tend to change.

EDMI calibration baselines are established to enable surveyors to check the instruments and determine any changes in the values of the constant and scale factors of these instruments. This information provides the documented history for legal evidence, insurance, and the like. The National Geodetic Survey (NGS) has developed guidelines for establishing these baselines.

In 1981, after examining five sites, the Civil Engineering Department at Iowa State University (ISU) established the EDMI baseline according to NGS guidelines. This baseline contains five monuments located on a line along a ditch at 0, 461, 620, 770, and 1370 meters. The Iowa Department of Transportation, the Society of Land Surveyors, and the Story County Engineer cooperated in this project. In October 1982, the NGS team measured the distances between the monuments using Invar tape, HP 3808 EDMI, and MA 100 EDMI. These measurements were adjusted and the final distances were published by the NGS. The elevation differences between the monuments were also measured by the NGS team and the ISU team. The distances have a standard error of 0.2 to 0.7 mm and the elevation differences have a standard error of about ±0.01 ft.

An observation procedure for calibrating EDMI was established. A computer program was developed for reducing the distances to horizontal, detecting blunders, and computing the precision of observation.

A mathematical model and a computer program were developed to give the constant and scale factors and their standard errors of the EDMI. The program is capable of constraining the observations, the known lengths, and the known constant and scale values according to their standard errors. Using this facility, a method was developed to detect any movement of the monuments.

Periodic measurements of the baseline were made in May 1981, July and November of 1982, and March, July, and October of 1983 using HP 3800 EDMI and Leitz Red EDMI. The computer programs were used to calibrate the EDMI periodically and to detect any movement of the monuments. No significant movement of the monuments was detected. This report details the research carried out in this project.

2. THE PRINCIPLES OF EDMI

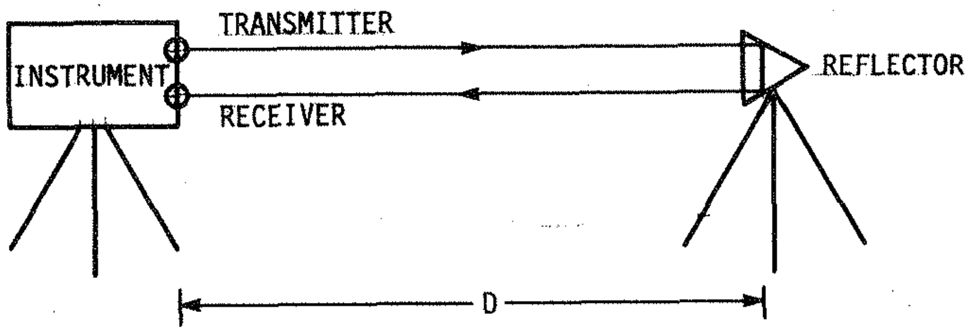

Fig. 1. Distance measurement by EDMI.

In an EDMI an electromagnetic signal is transmitted from the instrument and reflected back by a prism. The distance D between the reflector and the instrument is given

$$D = \frac{Ct}{2}$$

where C is the velocity of electromagnetic wave and t is the time taken by the wave to travel to the reflector and back. Since $C \simeq 3 \times 10^8$ m/s, the time t will be very small and difficult to measure accurately.

Alternatively the distance $D = n\lambda + L$, where n is the total number of wave lengths, λ is the wavelength, and L is the portion of the distance less than λ.

Now $C = f\lambda$ where f is the frequency of oscillation. The equation of a traveling wave front is given by

$$y = A \sin \omega \left(t_o + \frac{X}{C}\right) = A \sin \omega(t_o + t)$$

where

ω = $2\pi f$

y = the displacement of the particles perpendicular to the direction of propagation

x = the distance traveled by the wave

t_o = the initial time

A = amplitude of oscillation

$w(t_o + t)$ = the phase of the oscillation

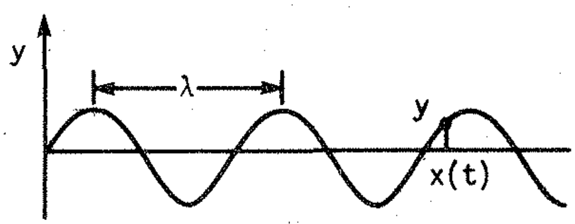

Fig. 2. Progressive sinusoidal wave.

$$y = A \sin w\left(t_o + \frac{n\lambda + L}{f\lambda}\right)$$

$$= A \sin\left(wt_o + 2\pi n + 2\pi \frac{L}{\lambda}\right)$$

$$= A \sin\left(wt_o + 2\pi \frac{L}{\lambda}\right)$$

The phase difference between the transmitted and received signal is given by

$$wt_o + 2\pi \frac{L}{\lambda} - (wt_o) = 2\pi \frac{L}{\lambda}$$

The phase difference (P.D.) can be measured

$$\therefore L = \frac{\lambda}{2\pi} (P.D.)$$

Now, if the wave is propagated at two frequencies, then under certain conditions

$$2D = n_1\lambda_1 + \ell_1 = n_2\lambda_2 + \ell_2 \; ; \qquad \ell_1 < \lambda_1$$

$$\ell_2 < \lambda_2$$

For distances

$$D \leq \frac{\lambda_1\lambda_2}{\lambda_1 - \lambda_2} \; ; \qquad n_1 = n_2 = n(\text{say})$$

then $2D = n\lambda_1 + \ell_1 = n\lambda_2 + \ell_2$

$$\therefore n = \frac{\ell_2 - \ell_1}{\lambda_1 - \lambda_2}$$

$$\therefore D = \left(\frac{L_2 - L_1}{\lambda_1 - \lambda_2} \right) \lambda_1 + L_1 \; ; \qquad L_1 \leq \lambda_{1/2}$$

$$= \left(\frac{L_2 - L_1}{\lambda_1 - \lambda_2} \right) \lambda_2 + L_2 \; ; \qquad L_2 \leq \lambda_{2/2}$$

Thus by measuring the P.D., it is possible to determine the distance D. In practice two methods are used by the EDMI to measure distance:

1) By choosing three frequencies such as $\lambda_1 = 10$ m, $\lambda_2 = 9.0909$, and $\lambda_3 = 9.95025$

$$\frac{\lambda_1 \lambda_2}{\lambda_1 - \lambda_2} \leq 100 \text{ m}$$

$$\frac{\lambda_1 \lambda_3}{\lambda_1 - \lambda_3} \leq 2000 \text{ m}$$

Thus by measuring L_1, L_2, L_3 to an accuracy of 1 cm, distances of up to 2000 m can be determined without ambiguity.

2) By choosing three frequencies such as $\lambda_1 = 20$ m, $\lambda_2 = 200$ m, $\lambda_3 = 2000$ m, so that by measuring $L_1 < 10$ m, $L_2 < 100$ m, and $L_3 < 1000$ m to three significant figures, distance up to 1000 m can be determined without ambiguity to the nearest centimeter.

The different frequencies of the signals are created by modulating the carrier wave either by amplitude or frequency modulation. An amplitude modulation is given by

$$y = (A + A_m \sin w_m t) \sin w(t_o + t)$$

and the frequency modulation is given by

$$y = A \sin (w + A_m \sin w_m t) (t_o + t)$$

These modulations are achieved by passing the carrier wave, such as the laser beam or an infrared beam, through a kerr cell which is controlled by an alternating voltage at the required modulation.

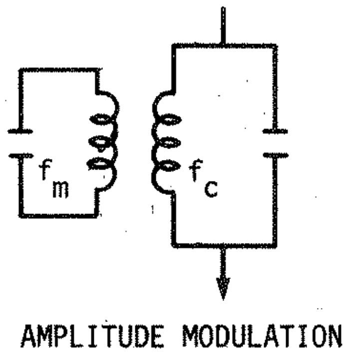

Fig. 3. Amplitude modulation.

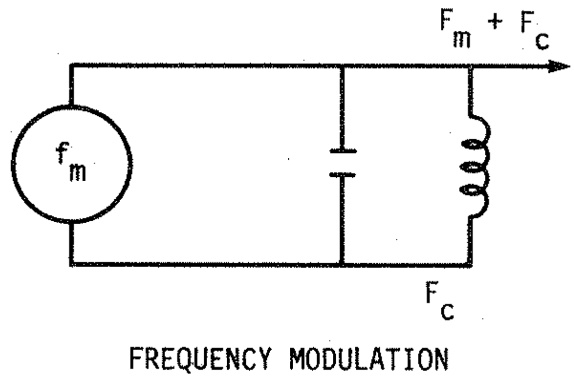

Fig. 4. Frequency modulation.

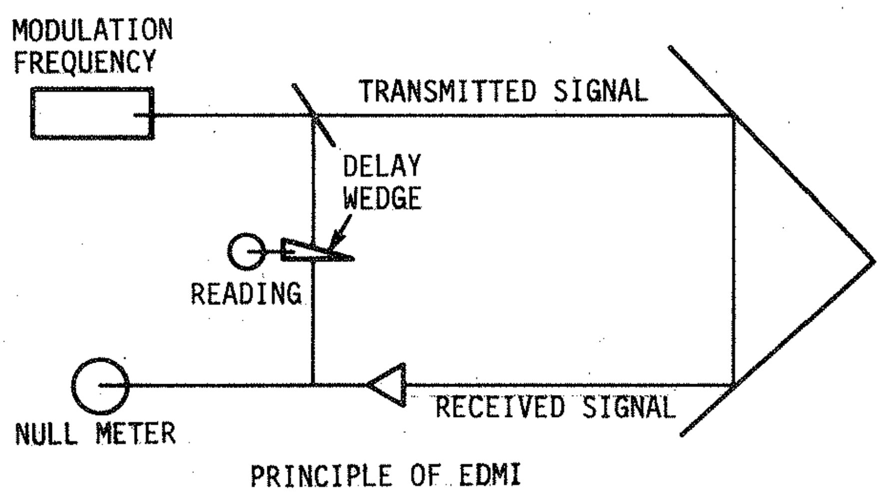

Fig. 5. Principle of EDMI.

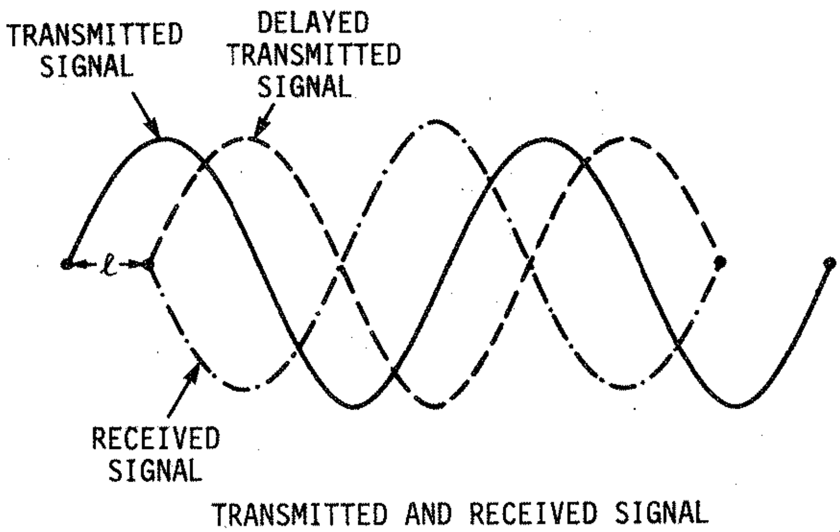

Fig. 6. Transmitted and received signal.

The phase difference between the transmitted and the received signal is measured by passing the portion of the transmitted and the received signal through a volt meter and then delaying the transmitted signal so as to give a null reading. The delay is then proportional to L. The L is determined by delaying a portion of the transmitted signal using a device such as a delay wedge (see Fig. 5).

3. THE ERRORS IN AN EDMI

The distance D measured by an EDMI is given by

$$2D = n\lambda + \ell$$

where

n = total number of full waves

λ = wave length of modulation frequency

ℓ = linear phase difference between transmitted and reflected signals.

The distance measured is subject to systematic errors. One, which is independent of the length, is due to the distance traveled within the EDMI system, swing errors, and the like. The other, which is dependent on the length, is due to variations of the atmospheric conditions, frequency drift, and so on.

The errors independent of the length, which are of significant values, are the constant error, the cyclic error, and the swing error. The constant error consists of two parts (see Fig. 7), which are 1) C_o' due to uncertainty of the electronic origin of measurement within the EDMI and 2) C_o'' due to uncertainty of the reflected position of the beam within the prism. Thus, the effective constant error

$$C_o = C_o' + C_o''$$

The cyclic error is due to the determination of L. The L is determined by delaying a portion of the transmitted signal using a device such as a delay wedge. When the transmitted signal is delayed and is mixed with the received signal, a zero reading will show on the null meter. Thus, the reading R, corresponding to the movement by the delay wedge, will depend on L. If we assume that R is proportional to L, then we will have an error. This error is typically small and cyclic with a period of $\lambda/2$ (see Fig. 8).

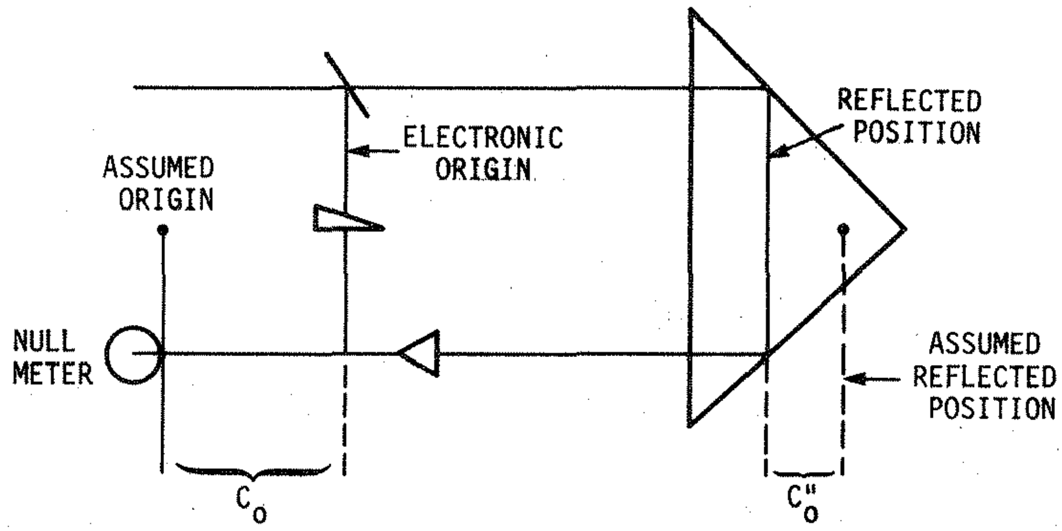

Fig. 7. Constant error.

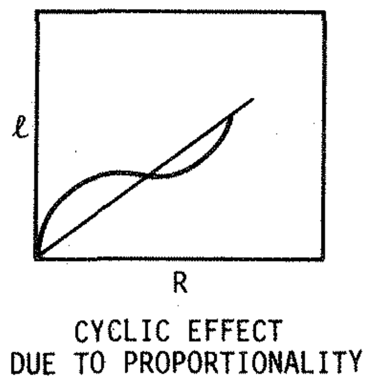

Fig. 8. Cyclic effect due to proportionality.

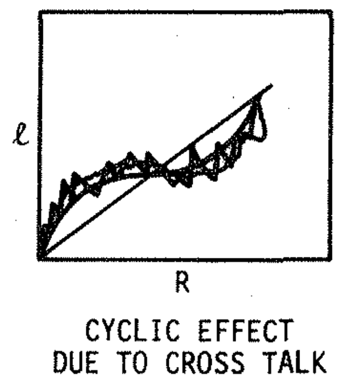

Fig. 9. Cyclic effect due to cross talk.

In practice there is also an error due to "electronic cross talk" between transmitted and received signals (see Fig. 9). The total error due to proportionality and electronic cross talk is known as cyclic error. The cyclic error can be represented by the Fourier series

$$Y = \sum_{i=1}^{n} \left(B_i \sin\left(i \frac{2\pi L}{\lambda/2}\right) + C_i \cos\left(i \frac{2\pi L}{\lambda/2}\right)\right)$$

where B_i and C_i are Fourier coefficients of the i^{th} sinus oscillation.

The swing error is due to the fact that the received signal is not a direct signal, but one that is reflected via a reflecting surface.

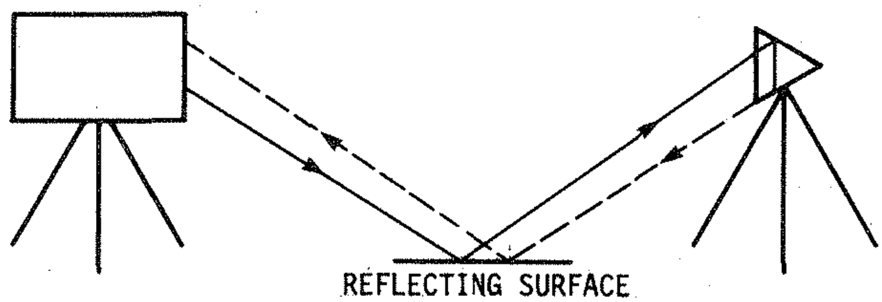

Fig. 10. Swing error.

This error is practically nonexistent in light wave instruments but does exist in microwave instruments. The effective way of eliminating this is to use a number of carrier frequencies.

The errors dependent on the length of any significance are the refraction error and the scale error. The refraction error is due to the velocity of the electromagnetic wave varying with the refractive index of the medium according to the equation:

$$C_o n_o = C_t n_t$$

where

C_o = velocity in vacuum

n_o = refractive index of vacuum, = 1

C_t = velocity in a medium

n_t = refractive index of the medium.

The refractive idex of the medium n_t depends on temperature T, pressure P, vapor pressure e, and the wave length of the carrier wave λ. The n_t for the light wave is given by

$$n_t = 1 + \left(\frac{n_g - 1}{1 + \alpha t}\right)\left(\frac{p}{760}\right) - \frac{5.5\ell}{1 + \alpha t} \times 10^{-8}$$

where

$$n_g = 1 + \left(2876.04 + \frac{48.864}{\lambda^2} + \frac{0.680}{\lambda^4}\right) \times 10^{-7}$$

t = dry bulb temperature in °C

α = 0.003661

λ = in micrometers (μm)

The n_t for microwave is given by

$$n_t = 1 + \frac{103.46\ P}{273.2 + t} + \frac{490.24\ e}{(273.2 + t)^2} \times 10^{-6}$$

where

$e = e' + de$

$e' = 4.58 \times 10^a$

$a = (7.5\ t')/(237.3 + t')$

$de = -0.000660\ (1 + 0.00115\ t')\ P\ (t - t')$

t' = wet bulb temperature in °C

In practice, the effect of e for light wave is negligible, especially for distances less than 2 km. Also in practice the frequency is compensated internally to accommodate the change in velocity. Since

$$C = f\lambda$$

$$dC \propto df$$

In modern short range instruments, the frequencies are set initially for average operational conditions and small changes to this frequency are made prior to the measurement. This effect can be seen from the following equations

$$D = C_t\ T$$

$$= \frac{C_o}{n_t}\ T$$

$$= \frac{C_o}{n}\ \frac{n}{n_t}\ T = \left(\frac{C_o T}{n}\right) \cdot \frac{n}{n_t}$$

$$= D'\ \frac{n}{n_t}$$

where n is the refractive index at which the instrument is initially set and D' is corresponding distance. Now $n = 1 + \alpha$

$$n_t = 1 + \alpha'$$

where α and α' are of the order of 0.0003. Then

$$D = D' \frac{1 + \alpha}{1 + \alpha'}$$

$$= D' (1 + (\alpha - \alpha'))$$

$$= D' + D'(\alpha - \alpha')$$

The correction factor $(\alpha - \alpha')$, which depends on the differences in temperature, pressure, and the like, is small. This correction factor can be computed or obtained from tables and charts. Most modern short range EDMI have facilities to enter this correction factor prior to measurement.

The scale error is due to the change in frequency of the modulation. In order to "lock" the frequency within very narrow limits, a quartz crystal is inserted in the circuit. The resonant vibration frequency of a crystal

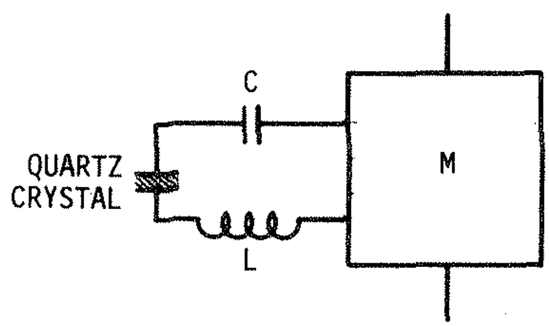

Fig. 11. Frequency drift.

is a function of its size and shape. Because crystal dimensions do change slightly with temperature and age, the frequency tends to drift. In practice, the instrument is operated in such a way that the crystal is in a temperature controlled environment. However, the dimensions of the quartz crystal change with "age" and affect the frequency, resulting in a scale error.

4. THE METHODS OF CALIBRATION

Modern EDMI are of the solid state type and therefore their electronic components are stable. However, due to usage, transportation, and the aging of crystals, the constant and scale factor tend to change. Also the constant changes for different combinations of prism and EDMI.

The EDMI must be calibrated periodically for the following reasons:

1) to check the accuracy of EDMI results
2) to determine the constant and scale factor of the EDMI under operational conditions
3) to provide documented instrument history for legal and insurance purposes
4) to maintain a uniform unit of measurement both locally and nationally
5) to maintain the standards of accuracy of surveying (e.g., 4×10^{-6} for third order triangulation, 1/20,000 for property surveys, etc.).

The calibration of an EDMI can be done under laboratory conditions as well as under field conditions. The values supplied by the manufacturers are generally those obtained under laboratory conditions and will not be discussed in this report. The field methods are the subject of this report. The advantages and disadvantages of different field methods are given below.

Baseline Method (see Fig. 12)

This method consists of measuring the distance between two established monuments by the EDMI and determining the constant, knowing the calibrated length between the monuments to an accuracy of 1 part in a million or better ($\pm 1/10^6$).

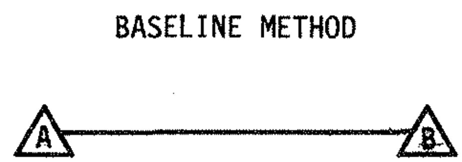

Fig. 12. Baseline method.

Advantages

1) Easy to lay out.
2) Easy to compute the constant.

Disadvantages

1) Results are misleading as the constant cannot be separated from the scale factor.
2) The EDMI is tested over one distance only.

Section Method (see Fig. 13)

In this method three or more monuments are set on a line and the distance between them determined to an accuracy of $\pm 1/10^6$ or better.

The constant and scale factor of an EDMI are determined by measuring all combinations of distances.

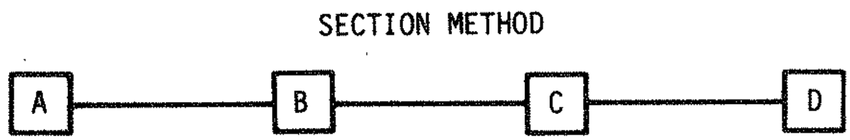

Fig. 13. Section method.

<u>Advantages</u>

1) Fairly easy to lay out.
2) Measurements can be done quickly.

<u>Disadvantages</u>

1) The calibration is done over a limited distance.
2) The monuments must be in line within limits.

<u>Intersection Method</u> (see Fig. 14)

In this method a number of monuments are set up at known points, spread out in all directions at different distances from a central point. The EDMI is set on this central point and distances are measured to all other points. From these measurements, the scale factor and the constant of the EDMI are determined by least squares.

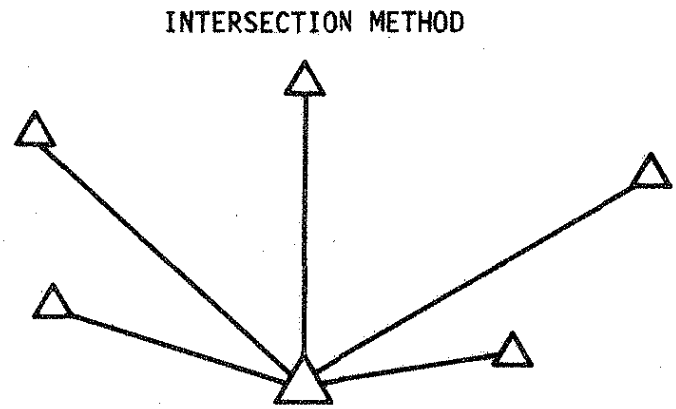

Fig. 14. Intersection method.

Advantages

1) The calibration can be done over unlimited distances.
2) A very good determination of C and S is possible.

Disadvantages

1) Measurement of distances may be time consuming.
2) The accuracy of C and S depends on the accuracy of the station coordinates.

NGS Calibration Baseline Specifications

The objective of EDMI calibration is to determine the constant, the scale factor, and the cyclic error. In most modern short range EDMI, the maximum cyclic errors are less than 5 mm and the frequencies are selected such that $\lambda_1 = 10$ m, $\lambda_2 = 200$ m, $\lambda_3 = 2000$ m, and so on so that $L_1 < 10$ m, $L_2 < 100$ m, $L_3 < 1000$ m, and the like. Since the cyclic error is proportional to L and if the distances for calibration baseline are chosen to be multiple of 10 meters, then the cyclic error will be almost negligible for the distances measured. Thus, the NGS chose the section method of calibration and selected the distances between the monuments to be multiples of 10 meters. The recommended design for the NGS baseline is shown in Fig. 15. Thus, the NGS baseline is suitable for determining scale factors and constant of a modern short range EDMI.

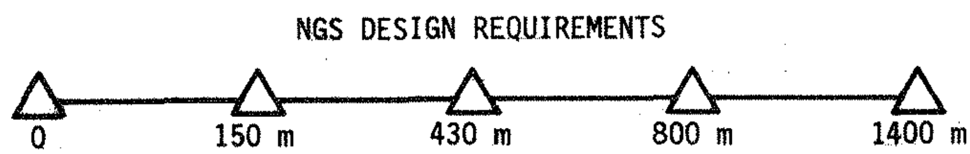

Fig. 15. NGS design requirements.

The requirements for establishing an NGS baseline are:

1) The site selected should have even terrain (see Fig. 16).

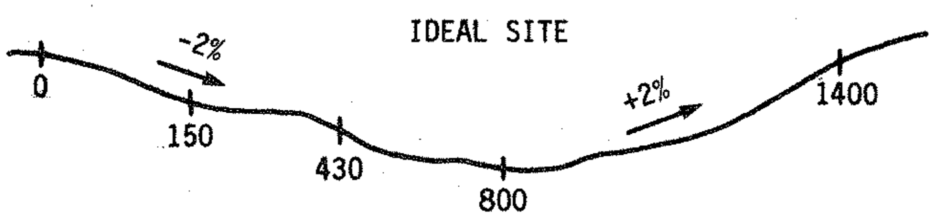

Fig. 16. Ideal baseline site.

2) The site should be easily accessible to the public.

3) No natural or man-made obstacles such as high voltage lines, fences, or the like, should be present on the site.

4) The monuments should be on line with an average tolerance of ±20" and a maximum of 5°.

5) The precise distances between monuments should be determined by using two high precision short range EDMI. In addition, 150 m distance should be taped by Invar tape. The distances are to be determined to an accuracy less than ±1 mm.

6) Since the 150 m distance will be taped, these two particular monuments should be established so that the distance between them is 150 ± centimeters. Also the design can be altered so that the terrain between these two monuments is as even as possible. This distance can be used to calibrate field tapes. The calibration tapes to be used have only 0 m and 50 m marks without graduation at the end tapes.

5. THE MATHEMATICAL MODEL FOR CALIBRATION

According to the specifications of the NGS baseline, only the constant (C) and scale factor (S) of an EDMI can be determined. The cyclic error is assumed to be negligible since the distances between the monuments are in multiples of 10 m and the modulation wavelengths are in multiples of 20 m.

Most modern short range EDMI have the facility to set the known constant and scale factor in the EDMI prior to measurement. The displayed distance is automatically corrected for these errors.

The high precision instruments used by NGS to establish the baseline distances have an accuracy of ±1 mm whereas most EDMI have an accuracy of ±3 mm.

The mathematical model for calibration must determine the scale factor and the constant of the EDMI using the NGS baseline. This model must take into account the fact that the measurements by EDMI are comparable to those of NGS measurements and that a priori knowledge of C and S has a certain precision.

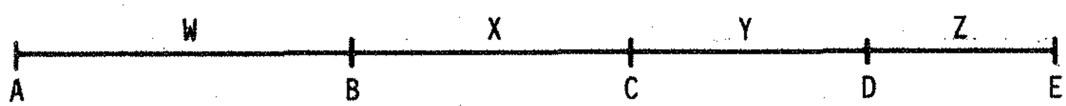

Fig. 17. Baseline distances.

Suppose A, B, C, D, and E are five monuments on line and the true distances between them are W, X, Y, Z; then the simplest method to determine the constant of an EDMI is to measure (or observe) all the distances between the monuments by the EDMI, then

$$C = \Sigma\text{observed} - \Sigma\text{known}$$

This method will give only the constant factor and not the scale factor. Alternatively,

$$C = \text{observed AB} + \text{observed BC} - \text{observed AC}$$

which is independent of known lengths

$$\text{and } S = \frac{\text{(observed) AB} - C - \text{(known) AB}}{\text{(known) AB}}$$

This method does not use all the observed distances and neither does it account for the precision of the observed distances and the known distances.

The method selected for determining the scale and constant is a method of least squares with a facility to constrain a priori parameters according to their precision. Suppose ℓ_i is an observed distance with a standard error of $\sigma_{\ell i}$, and W_o, X_o, Y_o, Z_o are the known distances of AB, BC, CD, and DE with standard error of σ_W, σ_X, σ_Y, σ_Z; C and S are the constant and scale factors with standard errors of σ_C and σ_S; ΔW, ΔX, ΔY, ΔZ, ΔC, ΔS are the errors in W, X, Y, Z, C, and S, respectively; then

$$\ell i + v_{\ell i} = a_1(W + \Delta W) + a_2(X + \Delta X) + a_3(Y + \Delta Y) + a_4(Z + \Delta Z) + C + \Delta C + \ell_i(S + \Delta S)$$

is an observation with weight

$$p_{\ell i} = \frac{\sigma_o^2}{\sigma_{\ell i}^2}$$

where a_1, a_2, a_3, a_4 are coefficients; $v_{\ell i}$ is the residual; σ_o^2 is the variance of unit weight, and

$$W + V_W = W + \Delta W \text{ with weight } P = \frac{\sigma_o^2}{\sigma_w^2}$$

$$X + V_X = X + \Delta X \text{ with weight } P_X = \frac{\sigma_o^2}{\sigma_x^2}$$

$$Y + V_Y = Y + \Delta Y \text{ with weight } P_Y = \frac{\sigma_o^2}{\sigma_Y^2}$$

$$Z + V_Z = Z + \Delta Z \text{ with weight } P_Z = \frac{\sigma_o^2}{\sigma_Z^2}$$

$$C + V_C = C + \Delta C \text{ with weight } P_C = \frac{\sigma_o^2}{\sigma_C^2}$$

$$S + V_S = S + \Delta S \text{ with weight } P_S = \frac{\sigma_o^2}{\sigma_S^2}$$

are the constant equations of the parameters. If the parameters are unknown, then these standard errors can be assumed to be ∞, which is equivalent to assuming that their weight is zero, which makes a self-calibration. The total observation equation can be written as:

$$V_{\ell i} + \ell_i - (a_1, a_2, a_3, a_4, 1, \ell_i)(W, X, Y, Z, C, S)^T$$

$$= (a_1, a_2, a_3, a_4, 1, \ell_i)(\Delta W, \Delta X, \Delta Y, \Delta Z, \Delta C, \Delta S)^T$$

$$V_W + 0 = \Delta W$$

$$V_X + 0 = \Delta X$$

$$V_Y + 0 = \Delta Y$$

$$V_Z + 0 = \Delta Z$$

$$V_C + 0 = \Delta C$$

$$V_S + 0 = \Delta S$$

Therefore, the observation can be written in matrix form as $V + L = AX$ where $X = (\Delta W, \Delta X, \Delta Y, \Delta Z, \Delta C, \Delta S)^T$

$$A = \begin{pmatrix} a_{i1} & a_{i2} & a_{i3} & a_{i4} & 1 & \ell_i \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{pmatrix}; \quad i = 1, n$$

$$L = \begin{pmatrix} \ell_i - (a_{i1}W + a_{i2}X + a_{i3}Y + a_{i4}Z + C + \ell_i S) \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}; \quad i = 1, n$$

where n is the number of observations by the EDMI.

The weight matrix P of the observations are given by

$$P = \begin{pmatrix} K/\sigma_1^2 & & & & & & & & & \\ & \ddots & & & & & & & \underline{0} & \\ & & K/\sigma_i^2 & & & & & & & \\ & & & \ddots & & & & & & \\ & & & & K/\sigma_n^2 & & & & & \\ & & & & & K/\sigma_W^2 & & & & \\ & & & & & & K/\sigma_X^2 & & & \\ & & & & & & & K/\sigma_Y^2 & & \\ & \underline{0} & & & & & & & K/\sigma_Z^2 & \\ & & & & & & & & & K/\sigma_C^2 & \\ & & & & & & & & & & K/\sigma_S^2 \end{pmatrix}$$

where K is a proportionality constant.

Thus, by the usual least-squares principle, we have

$$(A^T\ P\ A)X = A^T\ P\ L$$

$$X = (A^T\ P\ A)^{-1}\ A^T\ P\ L$$

The variance-covariance matrix Σ_X of the correction for parameters is given by

$$\Sigma_X = \sigma_o^2 (A^T\ P\ A)^{-1}$$

where

$$\sigma_o^2 = \frac{V^T P V}{N - 6}$$

is the variance of unit weight in which N = n + 6 and V = AX - L.

The variance of observation is given by

$$\sigma_i^2 = \frac{\sigma_o^2}{p_i}$$

The adjusted values of $\bar{C}$ and $\bar{S}$ are then given by

$$\bar{C} = C + \Delta C$$

$$\bar{S} = S + \Delta S$$

$$\sigma_{\bar{C}}^2 = \sigma_C^2 + \sigma_{\Delta C}^2 = \sigma_C^2 + \Sigma_{55} = \Sigma_{55} \text{ if } \Sigma_{55} >> \sigma_C$$

$$\sigma_{\bar{S}}^2 = \sigma_S^2 + \sigma_{\Delta S}^2 = \sigma_S^2 + \Sigma_{66} = \Sigma_{66} \text{ if } \Sigma_{66} >> \sigma_S$$

The values $\Delta C/\sigma_{\Delta C}$, $\Delta S/\sigma_{\Delta S}$ satisfy a t-distribution with n-2 degrees of freedom (Rainsford [10]).

Then if $\Delta C/\sigma_{\Delta C} > t_{\alpha,n-2}$ and $\Delta C/\sigma_{\Delta S} > t_{\alpha,n-2}$, it can be concluded at α confidence level that the scale and constant of the EDMI have changed; otherwise the constant and scale have not changed significantly.

6. THE MATHEMATICAL MODEL FOR MONUMENT MOVEMENT DETECTION

In practice the known distances will be determined at a time different from the observed distances. However, in the intervening period, the monuments may have moved due to natural or artificial causes. If the movements are large (compared with the accuracy of the observation), then they can be easily detected. However, if they are small, then a statistical analysis is required to detect the movement.

Suppose σ_{10}^2 is the variance of unit weight of the least-squares method in determining the C and S of an EDMI at an epoch T_1 and if σ_{20}^2 is the variance of unit weight of the least-squares method at the epoch T_2, then the value

$$F = \frac{\sigma_{10}^2}{\sigma_{20}^2} \quad \text{satisfies an F-distribution}$$

if

$$F > F_{\alpha, n_1, n_2} \qquad (n_1 \text{ and } n_2 \text{ are the respective degrees of freedom})$$

then σ_{10} is significantly different from σ_{20} at 90 - α confidence level. If so, assuming no blunders, the only possibility is that one or more monuments have moved in the direction of the line.

Now from the least-squares method we have

$$\sigma_{\Delta W}^2 = \frac{\sigma_{20}^2}{P_W}$$

$$\sigma_{\Delta X}^{2} = \frac{\sigma_{20}^{2}}{P_X}$$

$$\sigma_{\Delta Y}^{2} = \frac{\sigma_{20}^{2}}{P_Y}$$

$$\sigma_{\Delta Z}^{2} = \frac{\sigma_{20}^{2}}{P_Z}$$

Again, the values $\Delta W/\sigma_{\Delta W}$, $\Delta X/\sigma_{\Delta X}$, $\Delta Y/\sigma_{\Delta Y}$, $\Delta Z/\sigma_{\Delta Z}$ satisfy the t-distribution. If one or more of these values is $> t_{\alpha,n}$, then the monuments involved have moved. Also, in normal computation precepts, the weights for W, X, Y, Z will be high, and therefore ΔW, ΔX, ΔY, ΔZ will be small. However, the weights for the observations are small; therefore the residual V_i will be large. Again

$$t_i = \frac{V_i}{\sigma_o/\sqrt{P_i}}$$

satisfies the t-distribution. Then if $t_i > t_{\alpha,n}$, the monuments involved have probably moved. By analyzing the t's, the weights of W, X, Y, Z corresponding to the largest t_i can be made zero and a readjustment done. This procedure can be continued until

$$\frac{\sigma_{02}^{2}}{\sigma_{01}^{2}} < F_{\alpha,n_1,n_2}$$

For suspected small movements the weights of NGS values and the observations could be made the same in the readjustment and the results could be analyzed to detect the movement in the monument.

7. THE COMPUTER PROGRAM FOR CALIBRATION AND DETECTION OF MONUMENT MOVEMENT

The computer program using the mathematical model described earlier was developed in BASIC language to

(a) determine the constant and the scale factor simultaneously

(b) constrain the calibrated distances and measured distances

(c) constrain the known constant and scale factor

(d) detect any movement of the monuments

(e) maintain the history of the instrument and baseline.

See Fig. 19 for the program flowchart. Appendix III gives the listing of the program and Appendix I and II give the sample data input. Appendix IV gives sample output.

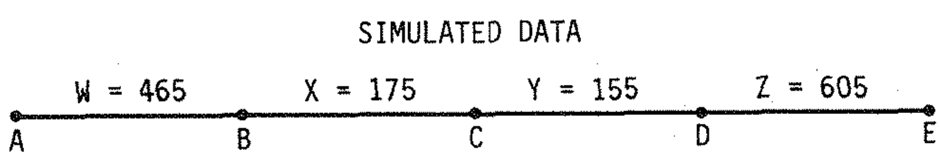

Fig. 18. Simulated baseline data.

Computations were done using simulated and real data. Tables 1, 2, 3, and 4 give the results from simulated data. Table 5 gives the results of the self-calibration of Red EDMI using the calibration baseline at ISU.

Simulated Data

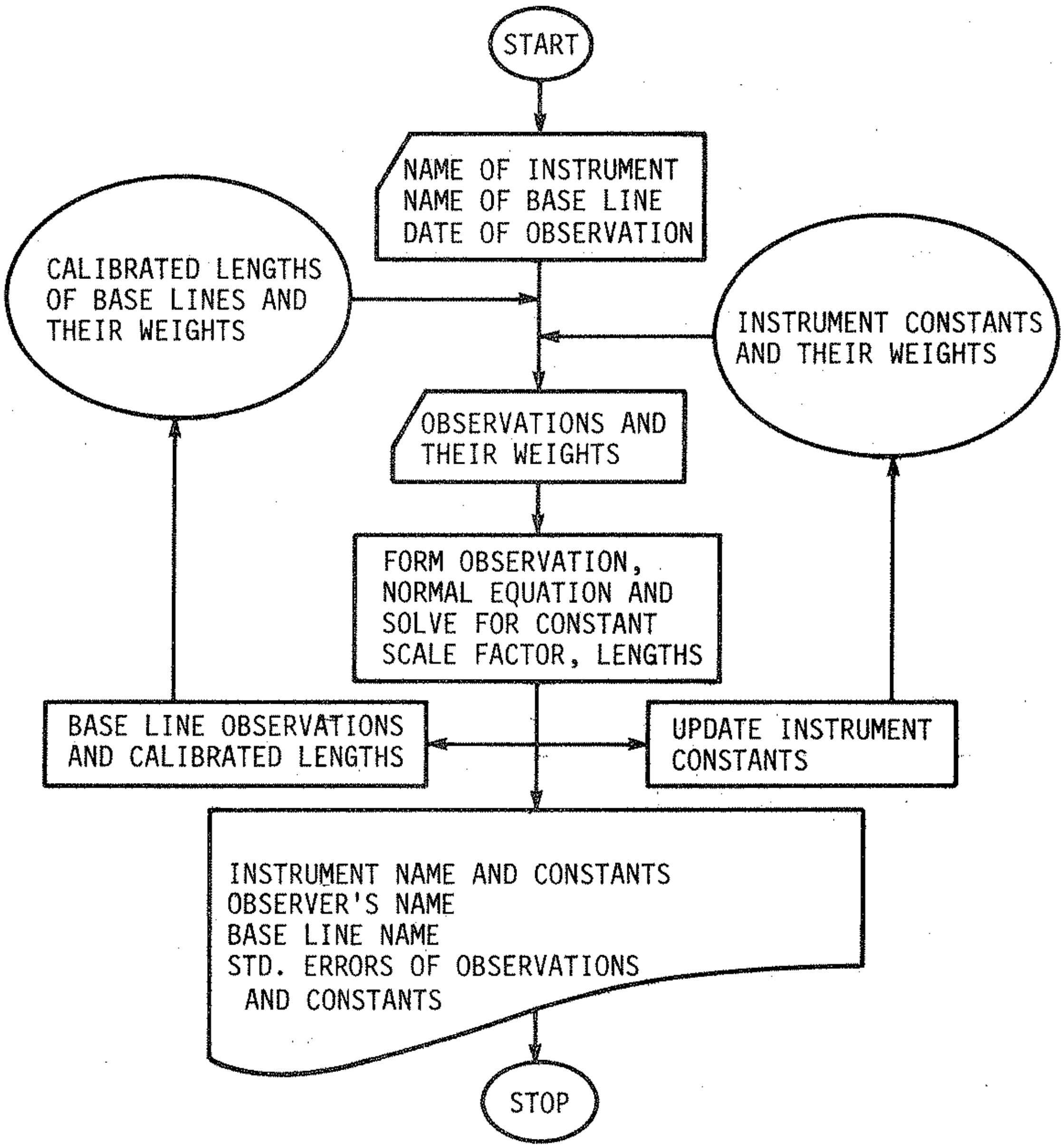

Fig. 19. Flow chart for calibration program.

Simulated data were created for different cases for the baseline shown in Fig. 18.

Case I

In this case simulted data were created from an EDMI with

Scale factor = 0.0

Constant = 0.02

Standard error of observation = 0.0

Standard error of the calibrated lengths = 0.0

Table 1. Baseline simulated data (Case I).

Data
465.02
175.02
155.02
605.02
330.02
935.02
1400.02
795.02
640.02
760.02

Results by Usual Computation Procedure

$$C = \Sigma \text{ observed} - \Sigma \text{ known}$$
$$= 0.02$$

or

$$C = \text{observed } AB + \text{observed } BC - \text{observed } AC$$
$$= 0.02$$

Results by Using the Computer Program

$$C = 0.019997 \pm 0.00001$$

$$S = 0.00000002 \pm 1.4 \times 10^{-8}$$

$$\text{Variance of unit weight} = 8.9 \times 10^{-6}$$

Case II

In this case, the simulated data were created for an EDMI with:

Scale factor	= 0.0001
Constant	= 0.02
Standard error of observation	= 0.0
Standard error of calibrated lengths	= 0.0

Table 2. Baseline simulated data (Case II).

Data
465.066
175.038
155.035
605.081
330.053
935.113
1400.160
795.099
640.084
760.096

Results of Computation by Usual Procedure

$C = \Sigma$ observed $- \Sigma$ known

= 0.08, which is incorrect

or

C = observed AB + observed AC - observed AC

= 0.02

$$\text{Scale factors} = \frac{\text{AB observed} - \text{C} - \text{AB (known)}}{\text{AB (known)}}$$

= 0.0001

Results by Computer Program

$C = 0.0198 \pm 0\ 00013$

$S = 0.0001 \pm 1.8 \times 10^{-7}$

Variance of unit weight = 0.0002

Case III

In this case simulated data were created for an EDMI with

$S = 0.0001$

$C = 0.02$

and movement of 0.01 m to monument B.

Standard error of observation = 0.0

Standard error of calibrated lengths = 0.0

Table 3. Baseline simulated data (Case III) and residuals.

	Data	Residuals After Adjustment	
W	465.00	-0.0051	
X	175.00	+0.0067	largest residual
Y	155.00	+0.0014	
Z	605.00	+0.0016	
	Data		
	465.077	0.0041	
	175.027	-0.0028	
	155.035	-0.0014	
	605.081	+0.0002	
	330.043	-0.0020	
	935.104	-0.0029	
	1400.16	0.00006	
	795.099	0.0009	
	640.084	0.00007	
	760.096	0.0010	

Results of Computation by Usual Procedure

C = observed AB + observed BC - observed AC

= 0.02

$$S = \frac{\text{observed AB} - C - \text{AB}}{\text{AB}}$$

$= 0.000123$ which has an inaccuracy actor of $23/10^6$.

Results from the computer program (under normal adjustment)

$C = 0.01889 \pm 0.02$

$S = 0.000104 \pm 0.00001$

Variance of unit weight = 0.003

$$F = \frac{0.003}{0.0002} = 15 > F_{0.01,10,10} = 4.85$$

indicates an unsatisfactory adjustment.

Computing t for the largest residual, we have

$$t = \frac{0.0067}{0.003} = 2.2 > t_{0.05,10} = 1.8$$

indicating probable movement of B.

Results of Computation by Computer Program

After analysis using F and T tests, the weights of W and X are made zero and a recomputation is done giving

$C = 0.0198 \pm 0.0001$

$S = 0.000100 \pm 2 \times 10^{-7}$

$\sigma_o^2 = 0.00011$

$$F = \frac{0.0001}{0.0002} = 0.5 < F_{\alpha,10,8} = 5.06$$

indicating satisfactory adjustment.

Case IV

In this case simulated data were created for an EDMI with

$$S = 0.00001$$

$$C = 0.02$$

Table 4. Baseline simulated data (Case IV).

Observed Data with Standard Error of 0.002	Calibrated Data with Standard Error of 0.0005
155.035	414.9997
175.036	174.9991
465.065	155.001
605.082	604.9996
330.056	
935.114	
1400.16	
760.097	
640.087	
795.101	

Results of Computation by Computer Program

Weight of observation = 0.25, weight of calibrated length = 1.

$$C = 0.0197 \pm 0.001$$

$$S = 0.000101 \pm 1.5 \times 10^{-6}$$

Standard error of unit weight = 0.0008

Results of Real Data Using ISU Baseline

Observer: Joel Dresel

Instrument: Red EDMI

Date: 5/14/81

Calibrated lengths were not available at that time. The set of observation readings are shown in Fig. 12. Self-calibration results using the computer program are

$$C = 0.0015 \pm 0.001$$

$$S = -0.5 \times 10^{-5} \pm 0.2 \times 10^{-5}$$

Standard error of unit weight = 0.0019

8. ERRORS IN EDMI OBSERVATIONS

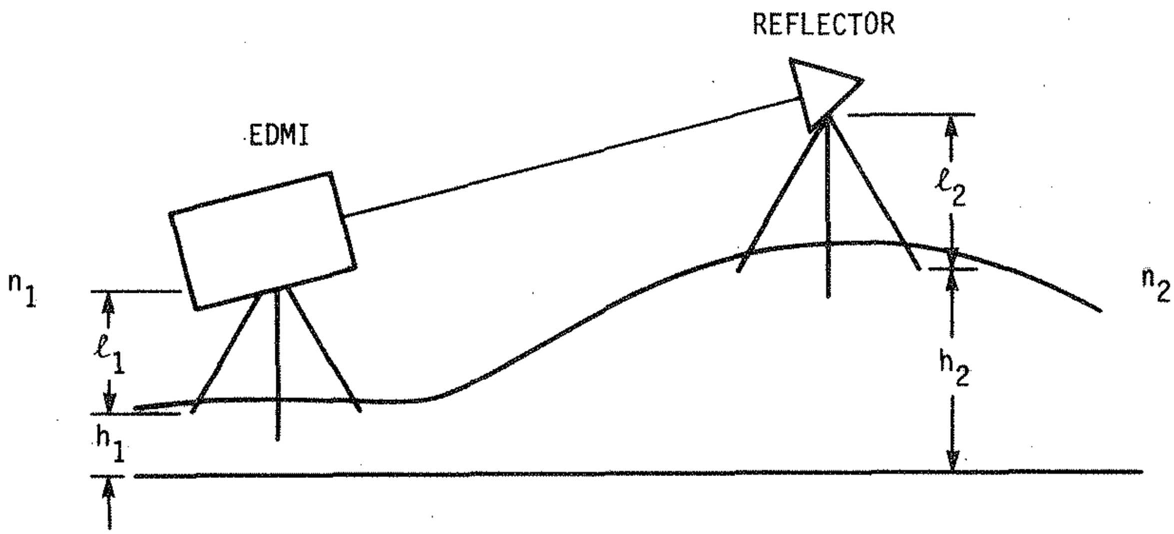

Fig. 20. Elevation of instrument and reflector.

In distance observations using an EDMI, there are not only internal errors, such as constant and scale errors, but also external errors. Among the external errors, the most significant are those due to:

1) centering EDMI or reflector precisely over the point
2) measurement of height of EDMI or reflector over the point
3) measurement of temperature and pressure.

Centering Error

Most modern EDMI equipment uses a tribrach with optical plummet for centering. The optical plummet has the advantage that it is unaffected by wind unlike the plumb bob. However, the line of sight in the optical plummet, representing the vertical, might be out of adjustment. The optical plummet has to be checked frequently for maladjustment, or the observation procedure adopted should eliminate these errors.

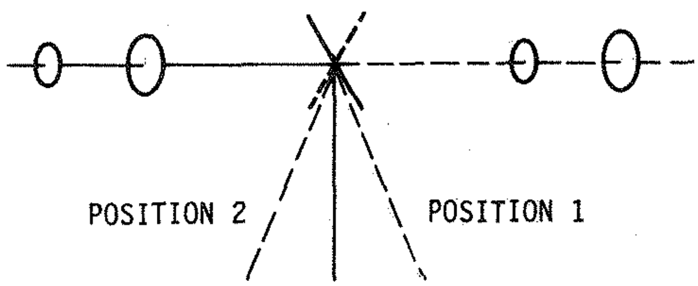

Fig. 21. Centering error.

Figure 21 illustrates the error in the line of observation. This error can be detected and adjusted by three methods, the plumb bob, rotation, or angle method.

Plumb Bob Method

In the plumb bob method, the plumb bob is used to center the tribrach over a point which is located inside a laboratory or building free of any wind effects. Then the plumb bob is removed and the optical plummet is checked over this point and any errors are adjusted by moving the cross hairs. This method is simple but it is difficult to achieve an accuracy of ±1 mm when centering a tribrach with a plumb bob.

Rotation Method

Some tribrachs have the facility that the eyepiece can be rotated about the center to sight the points. Thus, if the tribrach is first centered with the eyepiece at position 1, then if the eyepiece is rotated

180° for position 2, the line of sight will be different if the instrument is not in adjustment (see Fig. 21). The instrument can then be easily adjusted. This method is simple and accurate. However, most tribrach do not have the facility to rotate the line of sight.

If tribrachs do not have the facility to rotate the line of sight, then the tripod to which the tribrach is mounted can be set on a rotatable platform and any centering error could be determined as earlier.

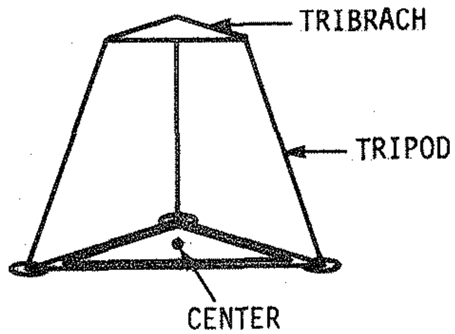

Fig. 22. Rotation method.

However, since a rotatable platform might not be available, the tripod could be mounted on a stand. The center of the stand can then be defined and the tripod, stand, and the like, could be rotated about this point (see Fig. 22).

Angle Method

In this method three targets are set about 25 feet from a point 0 such that $A \hat{0} B = B \hat{0} C \cong 90°$. A precise theodolite is mounted on the tribrach which is to be checked for centering. The angles $A \hat{0} B$ and $B \hat{0} C$ are measured accurately after centering over the point 0. Now

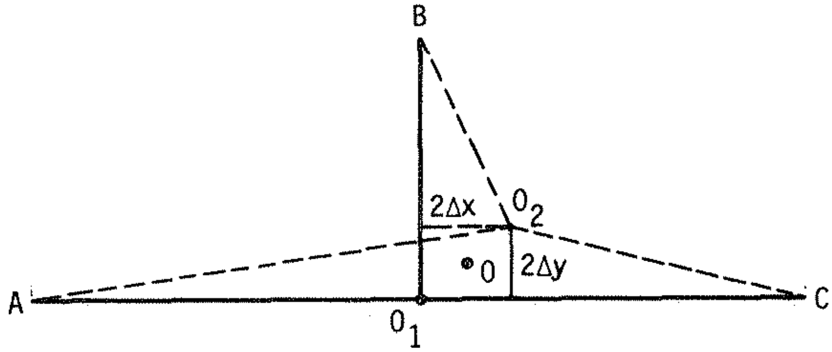

Fig. 23. Angular method.

the tribrach is rotated through 180°, recentered, and the angles are measured. If the two sets of angles are not the same, then the tribrach is out of adjustment.

Supposing the tribrach is out of adjustment, then the 1st set of angles measured is $A\hat{1}B$ and $B\hat{0}_1C$. The second sets are $A\hat{0}_2B$ and $B\hat{0}_2C$. From Fig. 23, since $A0 \gg \Delta X, \Delta Y$

$$\alpha = A\hat{0}_2B - A\hat{0}_1B = \left(\frac{2\Delta Y}{A0}\right) - \left(\frac{2\Delta X}{B0}\right)$$

$$\beta = B\hat{0}_2C - B\hat{0}_1C = \left(\frac{2\Delta X}{B0}\right) + \left(\frac{2\Delta Y}{C0}\right)$$

if

$$A0 = B0 = C0 = S$$

$$\frac{S}{2}\alpha = \Delta Y - \Delta X$$

$$\frac{S}{2}\,\beta = \Delta Y + \Delta X$$

$$\Delta Y = \frac{S}{4}\,(\alpha + B)\ ; \qquad \Delta X = \frac{S}{4}\,(\beta - \alpha)$$

where ΔX, ΔY are the errors in the centering along A C and O B, respectively.

All of the above methods were tested, and it was found that the angle method is the most accurate and the rotation method the least. The plumb bob method was the simplest and the angular method the most difficult.

After adjusting for centering error, any residual or change in centering error could be eliminated by measuring the distances in the forward and backward directions. This can be illustrated as follows

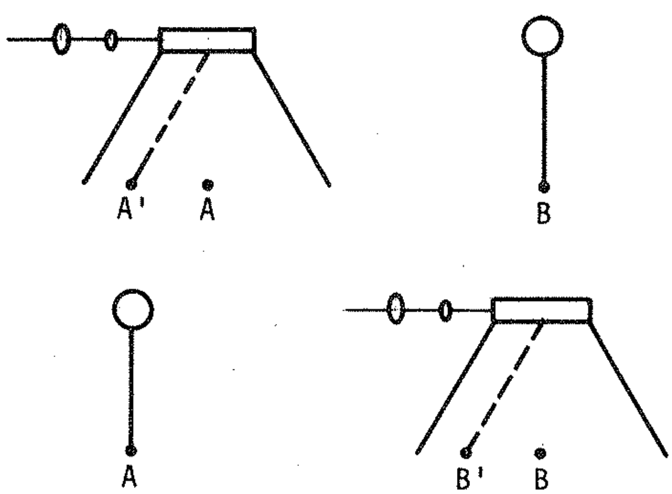

Fig. 24. Compensation for centering error.

Let AB be the distance measured. The EDMI is first set at A and the reflector at B. Due to centering error A'A, the distance measured by the EDMI is A'B. Now the EDMI is set at B and the reflector is set at A. The centering error B'B is equal to A'A and is in the same direction as the line of sight provided that optical plummets are in the same relative positions and the height of the tribrach above the points are the same in both cases.

$$\therefore AB = A'B - A'A$$

in the forward measurement and

$$BA = B'A + B'B$$

in the backward measurement

$$\therefore AB = \frac{A'B + B'A}{2}$$

which is independent of the centering error.

<u>Error Due to Height Measurement</u>

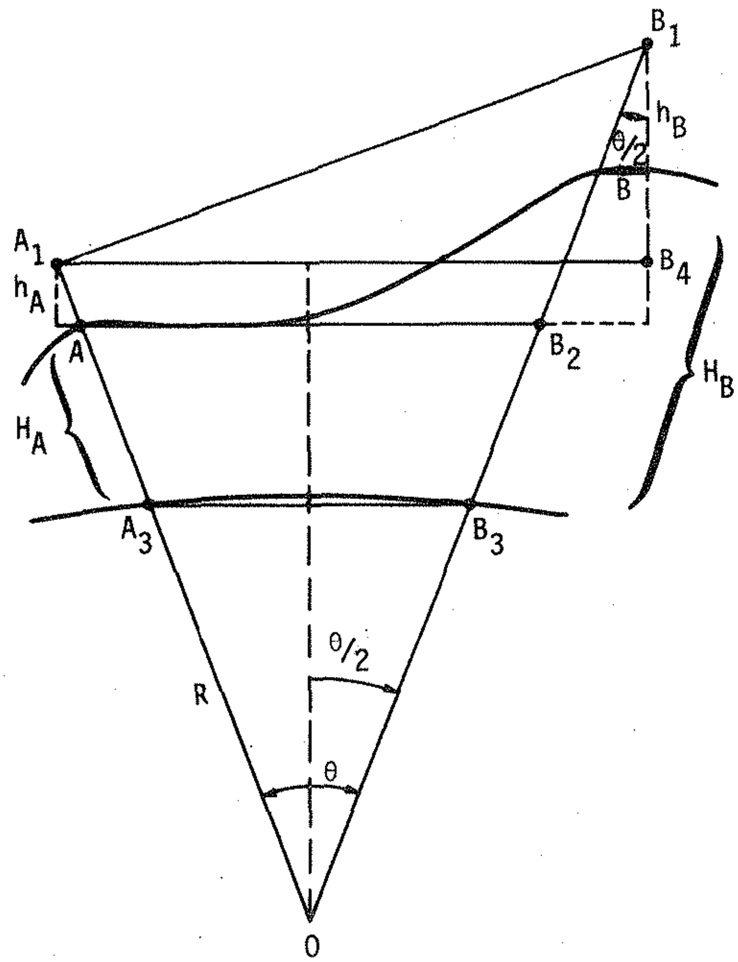

Fig. 25. Reduction to horizontal.

Let H_A be the height of station A, and H_B above mean sea level (MSL), respectively. Let L_A, L_B be the height of instrument at A and reflector at B, respectively. The slope distance A_1 B_1 measured by EDMI has to be reduced to the MSL giving A_3 B_3 or reduced to the horizontal giving A B_2.

The distance A_3B_3 is given by

$$A_3B^2{}_3 = \frac{A_1B^2{}_1 - (B_3B_1 - A_3A_1)^2}{\left(1 + \frac{A_3A_1}{R}\right)\left(1 + \frac{B_3B_1}{R}\right)}$$

where R = 0 A_3 = 0 B_3, the radius of curvature of the reference ellipsoid. The horizontal distance AB_2 is given by

$$AB_2 = \left[A_1B^2{}_1 - (H_A + h_A) - (H_B + h_B)^2\right]^{1/2} - \left[(H_A + h_A) - (H_B + h_B)\right] \sin \theta/2$$

where

$$\theta = AB/R$$

$$\theta/2 = 4.935''/1000 \text{ ft } (4.935'' \text{ per } 1000 \text{ ft.})$$

$$AB_2 = \left[A_1B^2{}_1 - \left\{(H_A + h_A) - (H_B + h_B)\right\}2\right]^{1/2} - \left[(H_A + h_A) - (H_B + h_B)\right] \cdot \sin \frac{A_1B_1 \times 4.935''}{1000}$$

Since the distances involved in the EDMI calibration are less than one mile, AB_2 is normally used instead of A_3B_3. For these distances

$$\left[(H_A + h_A) - (H_B + h_B)\right] \sin \frac{(A_1B_1) \times 4.935}{1000}$$

is negligible.

$$\therefore AB^2_2 = A_1B^2_1 - \left\{ (H_A + h_A) - (H_B + h_B) \right\}^2$$

$$= A_1B^2_1 - H^2$$

The difference in elevation between A and B is H. The error $\delta(AB_2)$ in AB_2 due to the error in $\delta(H)$ in H is given by

$$\delta(AB_2) = \frac{H}{AB_2}\,\delta(H)$$

$$\therefore \text{ if } H = 1 \text{ ft, } AB_2 = 500 \text{ ft}$$

Then an error of 0.1 ft in H will give an error of 1/5000 = 0.02/100 = 0.0002 ft in AB_2. Therefore, since the distances measured by EDMI have an accuracy of 0.01 ft, an error of ±0.1 ft in the height measurement would not significantly affect the calibration of the EDMI.

Error Due to Measurement of Temperature and Pressure

The refractive index for a light wave is given by

$$n_t \cong 1 + \frac{\left(n_g - 1\right)}{1 + \alpha t}\,\frac{P}{760}$$

$$n_g \cong 1.0003$$

$$\alpha \cong 0.003$$

The error δt in temperature gives an error δn in the refractive index which is given by

$$\delta_n \cong \frac{0.0003}{(1 + \alpha t)^2} \alpha \, \delta(t)$$

$$\cong 0.0003 \times 0.003 \ \delta t$$

$$\cong 9 \times 10^{-7} \ \delta t$$

The corresponding error in the distance is

$$\delta S = S(\delta n) \cong S \times 9 \times 10^{-7} \ \delta t$$

Thus, if $\delta t = \pm 1°$ C and S = 5000 ft, then $\delta S = 5 \times 10^3 \times 9 \times 10^{-7} = 45 \times 10^{-4} = 0.0045$. Similarly the error in distance due to error in δP in pressure is given by $\delta S = S(0.0003)(\delta P/760)$. Again, if s = 5000, δP = 1 mm, then

$$dS = 5 \times 10^3 \times 0.0003 \times \frac{1}{760}$$

$$\cong 15 \times 10^3 \times 10^{-4} \times \frac{10^{-2}}{7.6} \cong 2 \times 10^{-3}$$

$$\cong 0.002$$

Thus, it could be concluded that since the distances are measured to ±0.01 ft., the error of ±1° C and ±1 mm in temperature and pressure does not significantly affect the measured distances.

9. COMPUTER PROGRAM FOR MEASUREMENT REDUCTION

Fig. 26. Baseline stations.

In the EDMI calibration using the NGS baseline, the measurements have to be taken to eliminate both blunders and systematic errors. Systematic errors due to optical plummet can be eliminated, as seen earlier, by observing the distances in both directions. These observations in both directions could be used to detect blunders. Suppose H_{F_i}, H_{B_i} are the horizontal forward and backward distances. Then we have

$$d_i = H_{F_i} - H_{B_i}$$

where d_i is the difference between the two measurements

$$\therefore \sigma_{d_i}^{2} = \frac{\sum_{i=1}^{10} d_i^{2}}{10}$$

where σ_{d_i} is the standard error of the difference in measurements. The value $t = d_i/\sigma_{d_i}$ satisfies a t-distribution. Therefore, if $t > t_{\alpha,q}$ then the ith measurement may be subject to blunders at $(100 - \alpha)$ to confidence level.

In practice, both the forward and backward distances are measured with equal precision. Thus, if σ_H is the standard error of the horizontal distance, then

$$\sigma_d^2 = 2\ \sigma_H^2$$

$$\therefore\ \sigma H = \frac{\sigma_d}{\sqrt{2}}$$

σ_H could then be used in the calibration program to weight the observations.

The horizontal distance HD is given by

$$HD = \left[SLD^2 - \left[(EF + HI) - (EI + HR)\right]^2 \right]^{1/2} - \left[(EF + HI) - (EI + HR)\right] \sin \left(SLD \times \frac{4.935}{1000^2} \times \frac{1}{36 \times 57} \right)$$

where

SLD = slope distance between two stations

EF = elevation of the instrument station (from)

ET = elevation of the target station (to)

HI = height of instrument

HR = height of reflector

In Section 8 it was shown that the correction for refraction error is small and that in modern EDMI these corrections are entered in the EDMI while taking the measurements. The slope distances given by the EDMI are almost free of refraction error and therefore, no correction is required in the reduction.

A computer program in FORTRAN language is written to compute and print the horizontal distances, the differences between forward and backward distances, and the standard error of measurements. Figure 27 shows the flow chart of the program, Appendix VII gives the listing of the programs, and Appendix V and VI give the sample data. Appendix VIII gives sample output.

10. RECONNAISSANCE AND ESTABLISHMENT OF ISU BASELINE

Before selecting a suitable site in the area, topo sheets and aerial photographs must be studied. It is also important to discuss the suitability of the site with knowledgeable local people such as county surveyors, engineers, farm managers, and the like. The site also must be visited and a preliminary taping done.

Sites Selected

The EDMI calibration baseline at ISU was selected after carefully reviewing five sites. Figure 28 shows the sites that were considered. In the final selection the following factors were considered.

Site 1--Airport Site

Discussions with the airport manager revealed that the future expansion plans for the airport may interfere with the baseline. Also, the heavy traffic may endanger the survey crew.

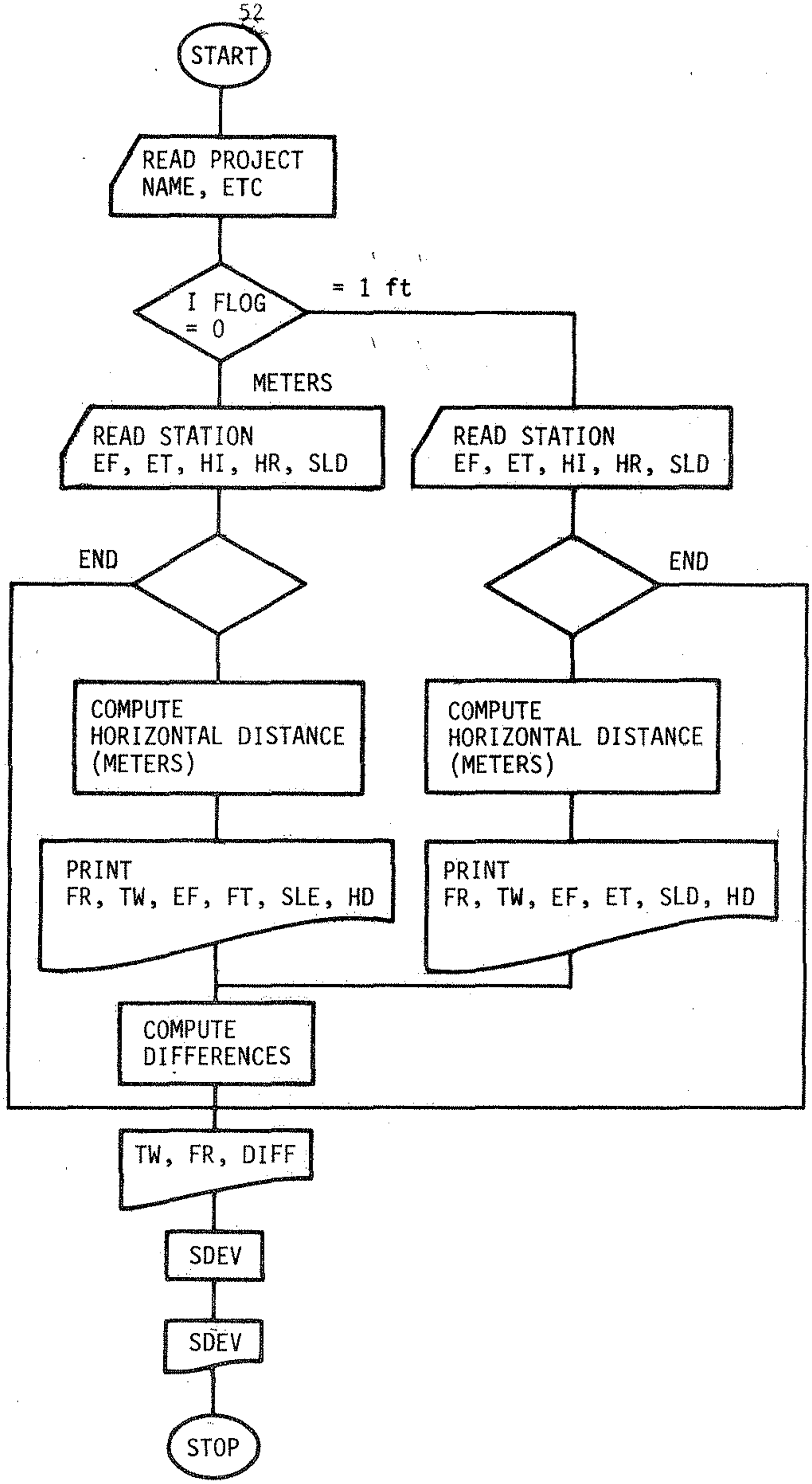

Fig. 27. Flow chart of reduction to horizontal.

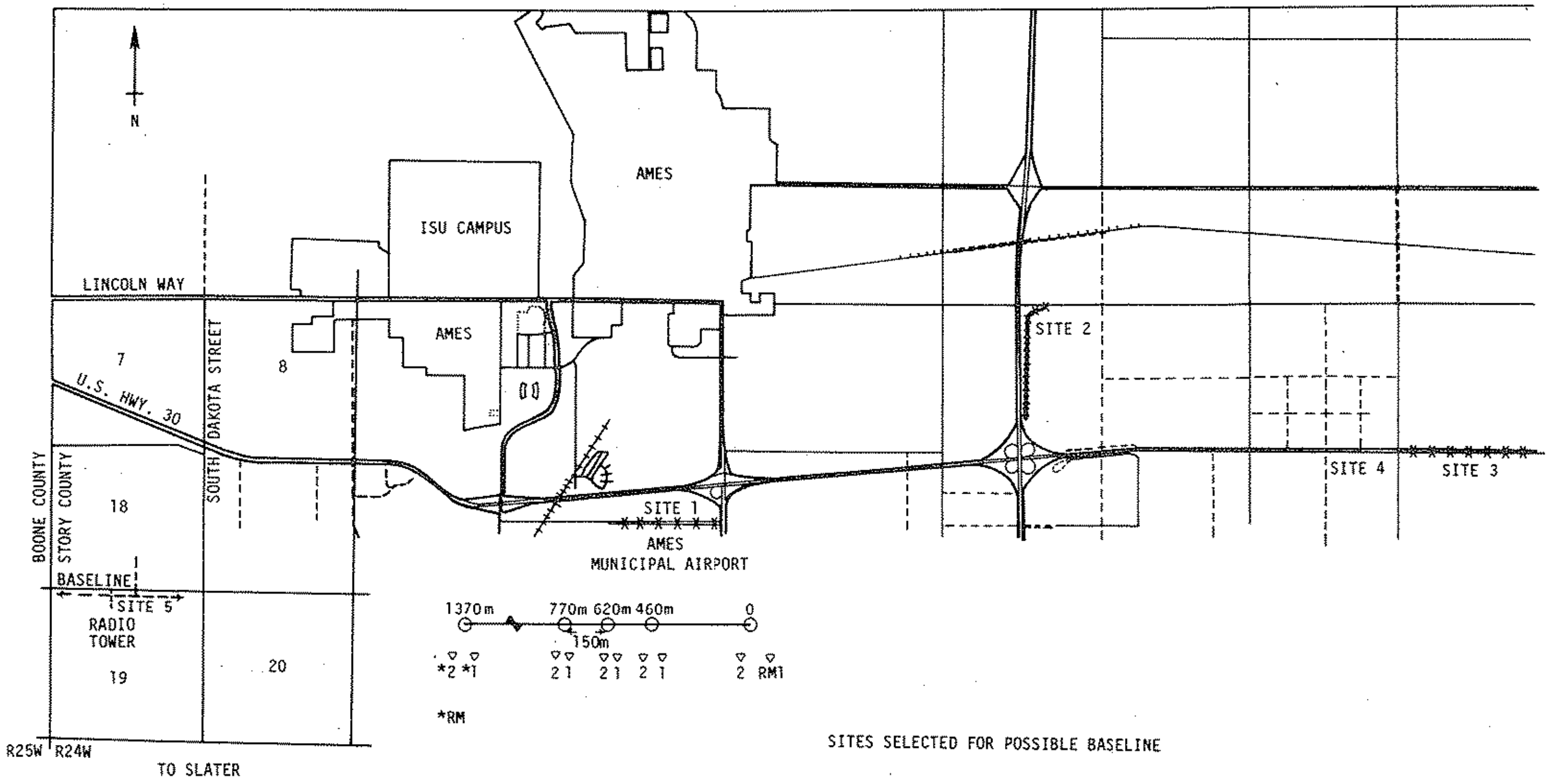

Fig. 28. Sites selected for possible baseline.

Site 2--I-35 Site

Though this site seemed ideal at first, a visit to the site showed the presence of obstructions such as high voltage lines. The terrain was also uneven.

Site 3--I-30 Site

The site was ideal. However, the Department of Transportation personnel objected because the only approach to the middle monuments was from the highway.

Site 4--County Dirt Road Site

Even though this site was ideal, it had to be abandoned because the road is going to be closed. Also landowners of the adjoining tracts objected to the establishment of these monuments.

Site 5--ISU Baseline Site

This site is on a right-of-way ditch along a county road adjoining the ISU farm. The site was good except for the presence of a TV tower at a distance of 500 feet from the baseline. This site was finally selected, even though it may not be suitable for calibrating microwave instruments.

Location

The ISU baseline is two miles south of Lincoln Way in Ames, Iowa. The property is owned by Iowa State University and the most obvious landmark is the WOI radio tower.

To get to the baseline from the north, one takes Lincoln Way to the South Dakota turnoff in West Ames and goes south two miles. The baseline is located at the north edge of section 19, T83N, R24W, in Story County. The WOI Tower is located in the center of this section.

Traveling from the south, one takes South Dakota north out of Slater four miles (see Fig. 29).

Measurement Procedure Adopted to Locate Monuments

The following procedures were used to locate the monuments:

1) Preliminary taping was done and approximate positions were marked on the ground.
2) The theodolite was set at every point and other points were sighted. The last monument was not visible from the 150 m mark.
3) In order to set the marks at visible locations, a profile leveling was performed (see Fig. 30).
4) After studying the profile, it was decided to build two mounds, about 2 to 3 feet high at both ends, and position monuments as shown in Fig. 30. This ensured sufficient clearing between the electromagnetic wave and the ground.

Establishing Monuments

The monuments were established in the following order:

1) The final positions were staked after checking with the EDMI for distance and with the theodolite for alignment. The positions were flagged for subsequent drilling.
2) Holes were drilled for the monuments and witness monuments.

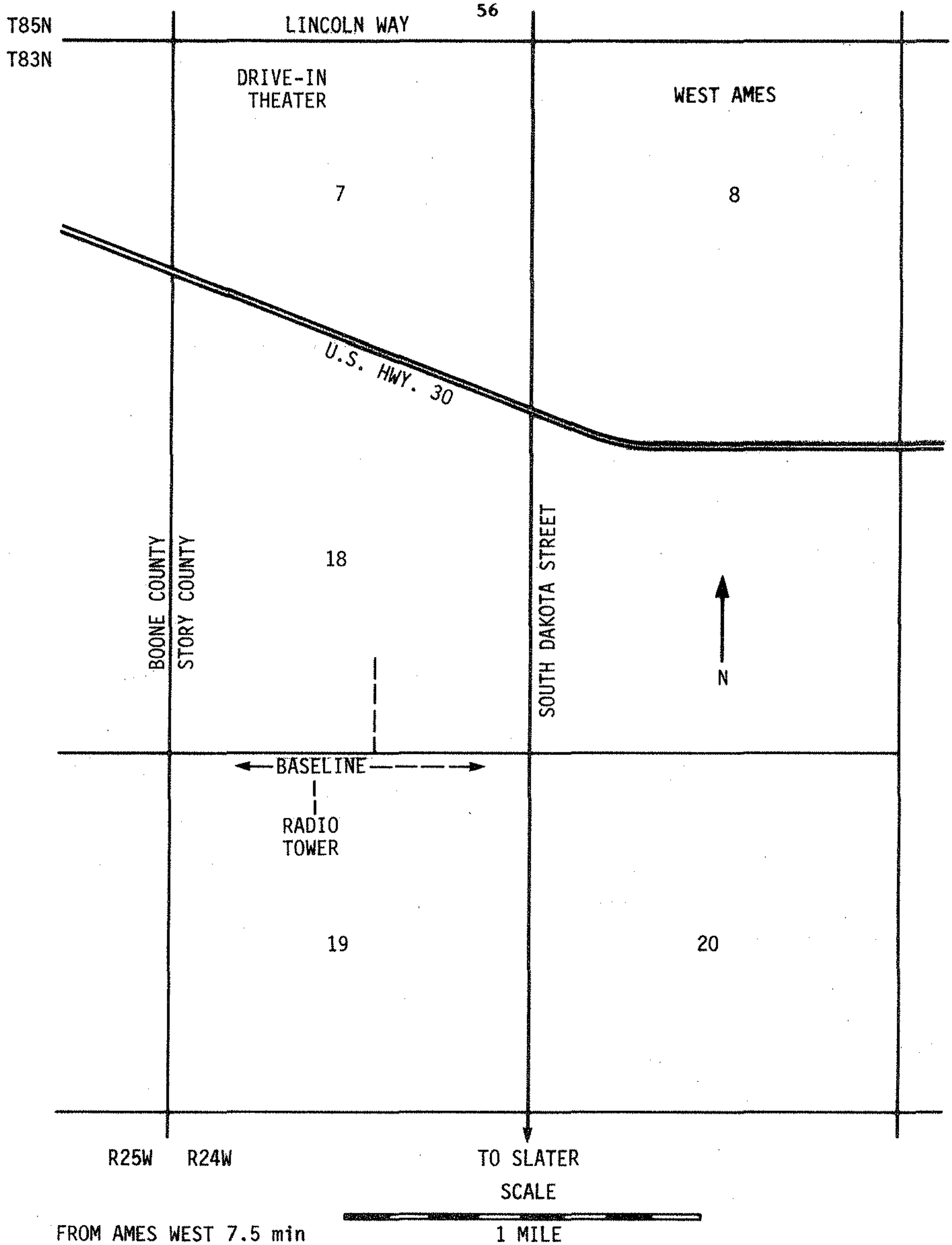

Fig. 29. Baseline location.

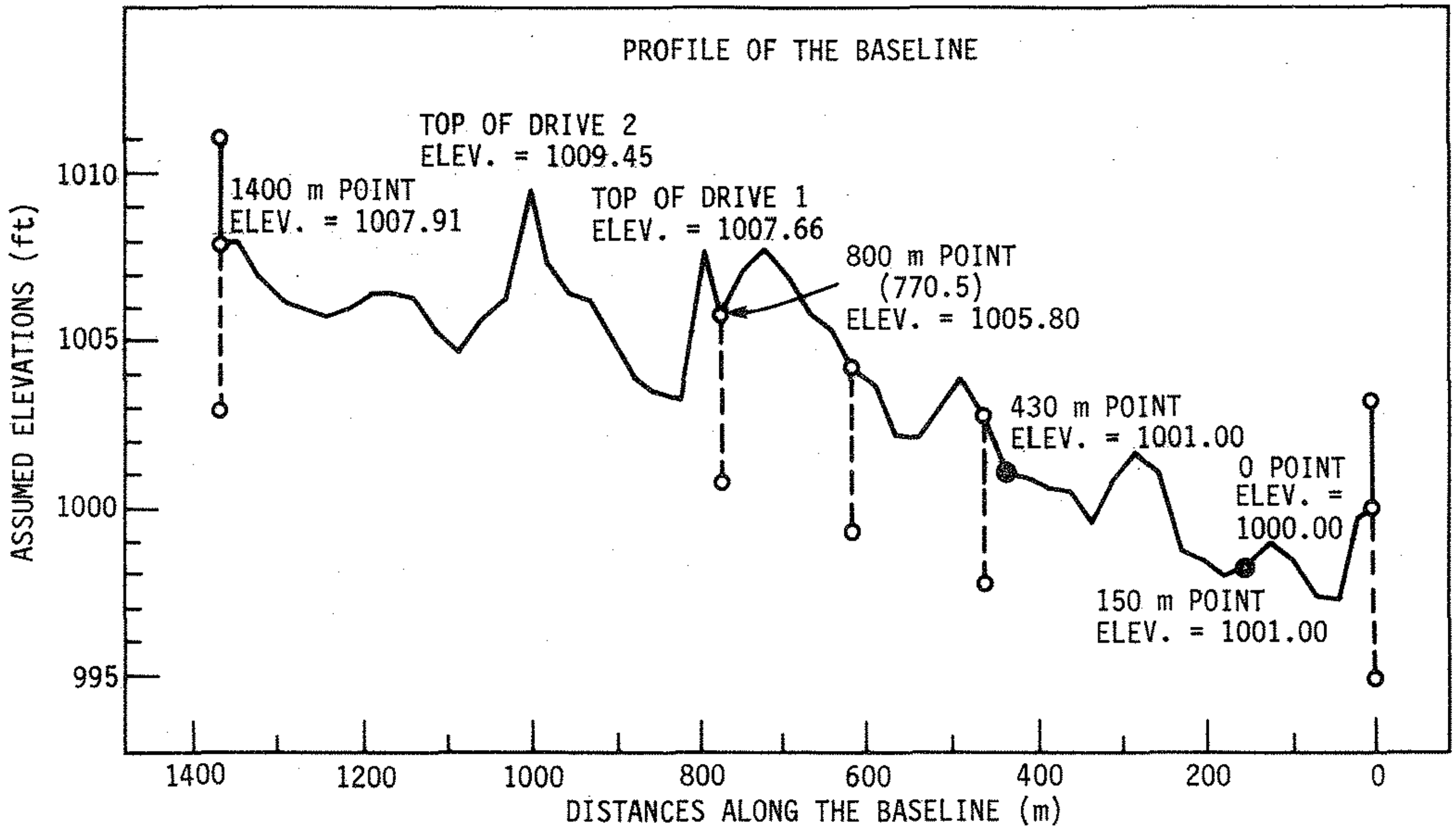

Fig. 30. Profile of the baseline.

3) Before setting the monuments in place, the holes were checked for alignment by placing the theodolite at the 770 m mark.

4) The positioning of the underground monuments was done with the aid of two stakes, placed exactly 6 ft on either side of the drilled hole. A steel tape was plumbed directly under the center of the tape. The underground monument was set in 6 in. of concrete and positioned with a bent wire affixed to the end of a range pole.

5) The positioning of the surface monuments and reference monuments was done three days after the underground monuments were set.

Two inches of sand was placed over the bottom for protection and the 5-ft-deep hole was then filled with ready-mix concrete. Positioning of the surface monuments was done in the same manner as the underground monument except the theodolite was used to ensure the placing of the monuments on line (see Figs. 31, 32, and 33). The theodolite was set up over one of the two stakes sighted on a range pole at the center of the range, and then the cap was set on line with the cross hairs of the instrument. The proper distance was reset at the center of a tape stretched between two stakes.

The initial and the 1370 m stations were elevated with the aid of concrete forms (see Fig. 33) set over the drilled holes and filled to a height of 3 ft above the normal ground surface. After the concrete hardened, these forms were removed and a mound built around the monument.

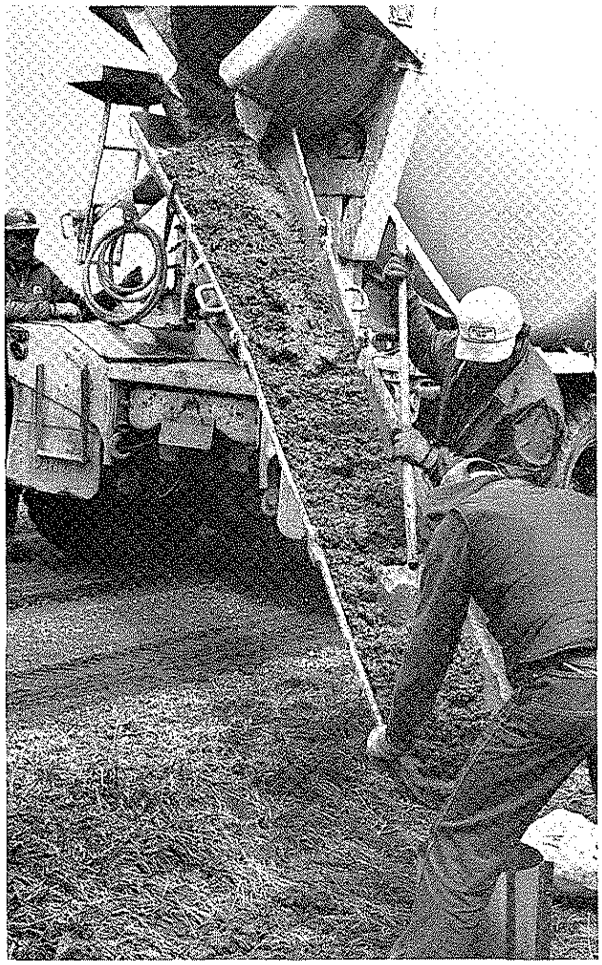

Fig. 31. Final adjustment.

Fig. 32. Positioning the monument.

Fig. 33. Filling the form.

In the final layout of the ISU baseline, the spacing is approximately 461 m (1513 ft), 620 m (2035 ft), 770 m (2528 ft), and 1369 m (4492 ft) from the initial point that lies on the east end of the line (Fig. 34).

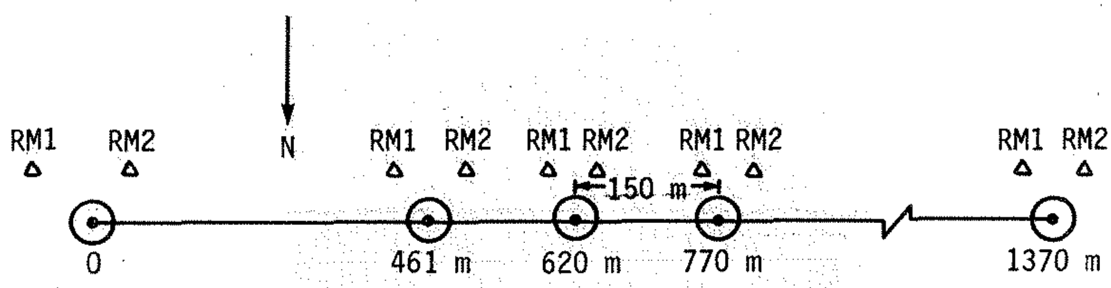

Fig. 34. ISU baseline.

To insure relocation of destroyed monuments, two precautions were taken: reference marks were set at each station for approximate relocation, and an underground monument was set directly under the surface monument for precise relocation. Table 5 gives the distances to the reference points and Fig. 35 shows the monument construction.

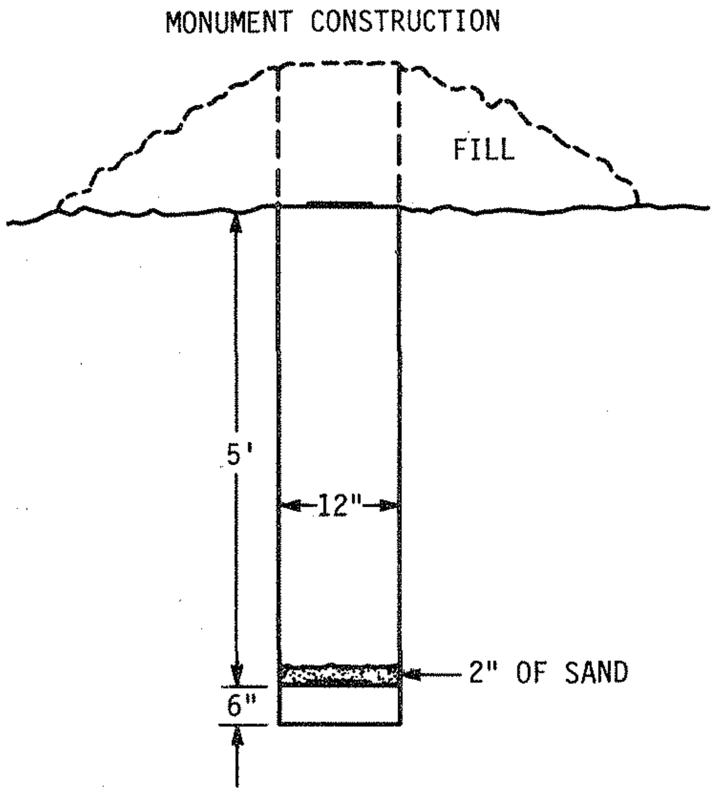

Fig. 35. Monument construction.

Table 5. Reference mark positioning.

Station	RM	Distance
0	1	20.9'
	2	16.5'
460	1	21.3'
	2	18.7'
620	1	16.4'
	2	16.5'
770	1	19.2'
	2	12.0'
1370	1	17.6'
	2	18.1'

Setting the Monuments and Establishing the Baseline

Three phases of monument location were undertaken: hole drilling, underground monument location, and surface monument location.

The hole drilling was performed after all stations were staked and marked with ribbon for easy identification. Holes were drilled by the Iowa DOT with a 1-ft-diameter earth drill, 5-ft deep. Reference mark holes were drilled 6 in. in diameter, 4-ft deep.

Surveying students at ISU, David Varner and John Dierksen, completed the reconnaisance in February 1981. The monuments were established in spring 1981. John Dierksen et al., the staff of ISU Physical Plant, and the staff of Iowa DOT were involved in establishing the monuments. In May 1981, Joel Dresel and Robert Lyon measured the distances and elevation differences between the monuments. In October 1982, an NGS team of two plus four ISU students (Scott Kool, etc.) measured the distances and elevation differences between the monuments. The 150 m distance between monuments B and C was measured with two Invar tapes by the NGS team. They also measured all distances between monuments using both HP 3808 and MA 100 EDMI. A first order observation procedure was used in these measurements. These measurements were reduced and least-squares adjustment was done. The final results were then published by NGS (see Appendix IX).

Also in October 1982, Scott Kool and Robert Lyon did a third order leveling between monuments E and an existing bench mark "IHC" on Highway 30 (see Fig. 36). Table 6 gives the elevation of E and Table 7

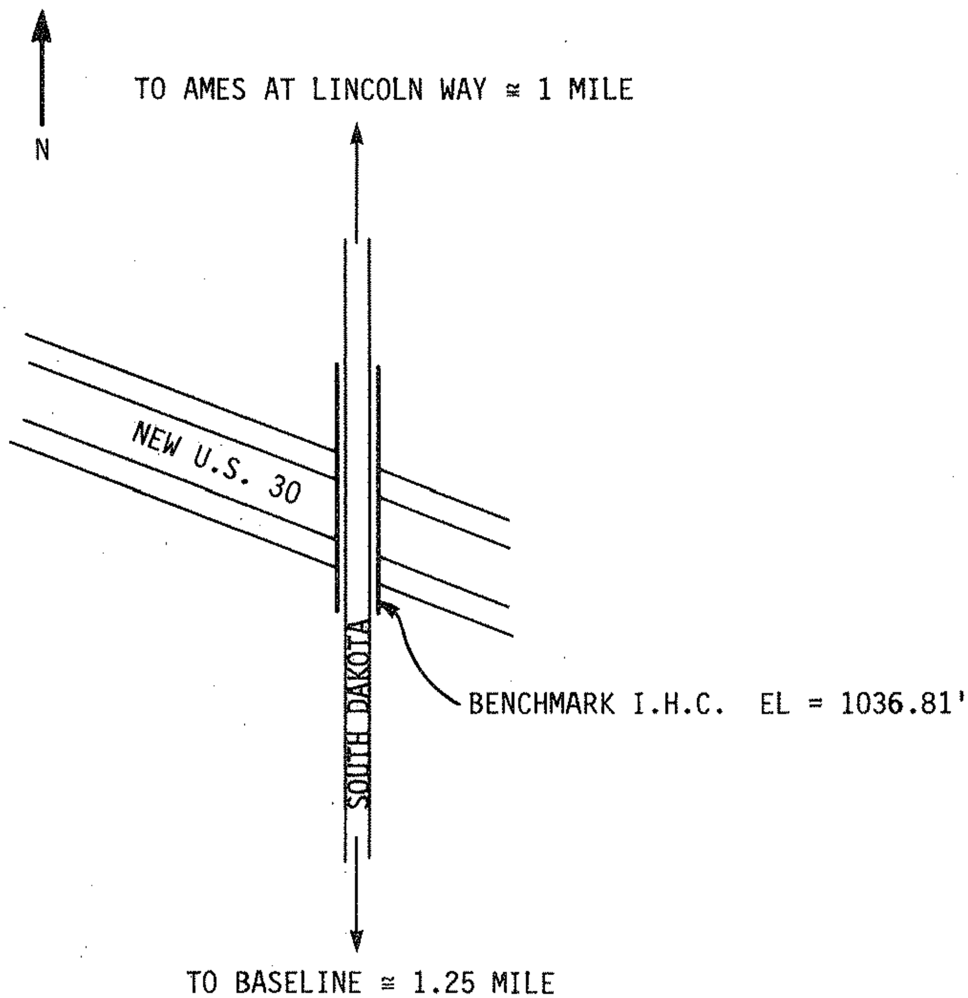

Fig. 36. Benchmark IHC.

compares the elevation differences between monuments as obtained by Scott Kool, the NGS team, and Joel Dresel.

Table 6. Mean sea level elevation of monument E(O).

Forward leveling difference between IHC & E	= 13.958 ft.
Backward leveling difference between IHC & E	= 13.927
Mean difference in elevation between IHC & E	= 13.9425
Elevation of IHC	= 1036.81
Elevation of E	= 1050.75

Table 7. Comparison of leveling between monuments.

	NGS 1982	Scott 1982	Joel 1981
B - A	3.398 ft.	3.406 ft.	3.43 ft.
C - B	0.672 ft.	0.663 ft.	0.65 ft.
D - C	1.030 ft.	1.042 ft.	1.04 ft.
E - D	3.116 ft.	3.147 ft.	3.16 ft.

11. OBSERVATION PROCEDURE

The objective of the measurement procedure should be to obtain all possible combinations of distances between the monuments under normal operations procedures. These measurements can then be used to

a) determine the horizontal distances

b) estimate the precision of observations

c) estimate the constant and the scale factors.

The following procedure was used to obtain all possible combinations of distances:

1) The tripod with a tribrach was set over each station (see Fig. 37). The centering is performed with the line of sight of the optical plummet pointing the same direction.

Fig. 37. Tripod set-up.

2) The EDMI was positioned at one station at a time, and readings were taken to all other stations by moving the prism. Only two tribrach should be used, one with the EDMI and the other with the reflector.

3) Height of instrument, height of reflector, temperature, and pressure readings were taken for each shot.

4) The known prism constant and the computed atmospheric correction factor were set in the instrument for each reading.

5) The set of readings are recorded as in Fig. 38.

12. PERIODIC MEASUREMENT AND CALIBRATION OF EDMI

In order to monitor any possible movement of the monuments and to document the instrument history, the baseline was measured by using both Leitz Red EDM and HP 3800 instruments in July and November of 1982, and March, July, and October of 1983. The observations in July

L. Cornelis
J. Popelka - Refl.

July 23, 1983
Cloudy - Still (Calm) 60

TEMP. FAHR.	BAR. PRESS.	SCALE SETTING
77°	29.4	12

INSTR A	OBS.	SLOPE DIST. (feet)	H.I./H.R. 5.2
	E	4492.247	5.1
	D	2979.417	4.8
	C	2457.000	5.1
	B	1964.368	5.4

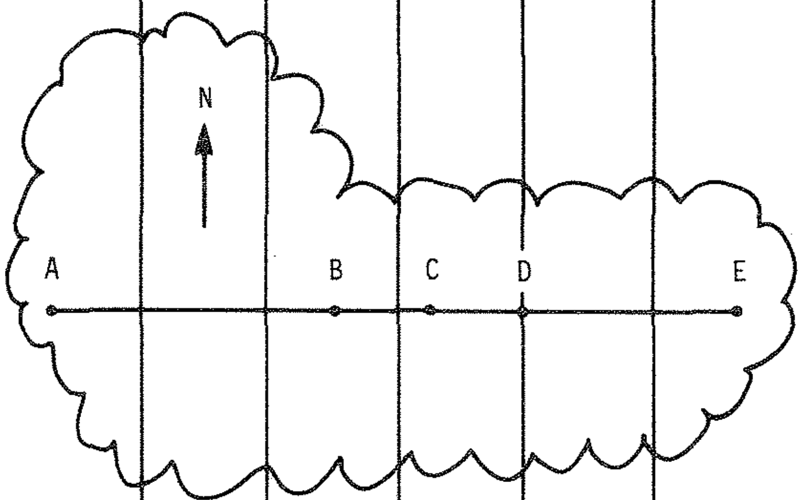

Fig. 38. Sample field notes.

1983 were made using HP 3800 belonging to the Iowa DOT while all other observations were made using the HP 3800 and Leitz Red EDM belonging to ISU. In May 1981 observations were made using only the Red EDM. All observations used the same triple prism. John Jennison, James Otto, Kostas Kiriakopoulos, Scott Kool, Joel Dresek, and Leon Cornelis were involved in these measurements. All measurements were reduced using the reduction to horizontal program (RDHZ). If the differences between the backward and forward measurements of any distance was greater than the $(t_{1,9}) \cdot$ (standard error of the differences), then that particular observation was checked for blunders, and the like, and if necessary, reobserved. A new reduction was then completed using new observations. Table 8 gives the mean of the forward and backward measurements.

These measurements were also used to calibrate the EDMIs periodically using the calibration program. Table 9 summarizes the calibration results. It also gives the standard error of the adjustment, the calibrated lengths and their standard errors, the calibrated instrument constants and their standard errors. For comparison, Table 10 also includes the NGS calibrated lengths of the baseline.

13. ANALYSIS OF THE RESULTS

According to the manufacturers, both the Red EDM and HP 3800 have an accuracy of ±3 mm. Thus after elimination of any of the blunders

$$\chi^2 = \left(\frac{\text{Std error of the difference}}{(\sqrt{2})(3)}\right)^2 > \chi^2_{0.01,9} = 21$$

Table 8. Periodic baseline measurements.

	Summer 1981	Summer 1982 (M)	Fall 1982 (M)	March 1983 (M)	July 1983 (M)	October 1983 (M)
			Hewlett Packard Observations			
AB		598.7495	598.7485	598.7468	598.7402	598.7421
AC		748.9020	748.9030	748.9005	748.8963	748.9004
AD		908.1350	908.1350	908.1307	908.1248	908.1274
AE		1369.2500	1369.2520	1369.2449	1369.2332	1369.2434
BC		150.1500	150.1580	150.1607	150.1507	150.1602
BD		309.3820	309.3980	309.3910	309.3838	309.3909
BE		770.4975	770.5062	770.5088	770.4949	770.4982
CD		159.2265	159.2320	159.2308	159.2260	159.2301
CE		620.3415	620.3460	620.3460	620.3349	620.3426
DE		461.1060	461.1060	461.1111	461.1127	461.1058
			Leitz Red EDM Observations			
AB	598.746	598.752	598.745	598.7428	598.7502	498.7447
AC	748.904	748.906	748.902	748.9060	748.8989	748.9010
AD	908.136	908.134	908.130	908.1367	908.1433	908.1328
AE	1369.253	1369.252	1369.234	1369.2545	1369.2385	1369.2516
BC	150.162	150.164	150.158	150.1595	150.1577	150.1607
BD	309.393	309.392	309.390	309.3946	309.3936	309.3921
BE	770.507	770.504	770.499	770.5030	770.5000	770.5025
CD	159.234	159.232	159.235	159.2347	159.2329	159.2368
CE	620.350	620.349	620.343	620.3467	620.3463	620.3476
ED	461.116	461.118	461.111	461.1169	461.1233	461.1154

Table 9. Monitoring baseline and EDMI.

	Summer July 1982 m (Value) ± mm (Standard error)	Fall November 1982 m (Value) ± mm (Standard error)	Spring March 1983 m (Value) ± mm (Standard error)	Summer July 1983* m (Value) ± mm (Standard error)	Fall October 1983 m (Value) ± mm (Standard error)
			HP		
W	461.112 ± 0.9	461.113 ± 1.0	461.114 ± 0.8	461.114 ± 0.8	461.113 ± 0.9
X	159.232 ± 0.8	159.233 ± 1.0	159.232 ± 0.8	159.231 ± 0.8	159.232 ± 0.8
Y	150.157 ± 0.9	150.158 ± 1.0	150.158 ± 0.8	150.157 ± 0.8	150.158 ± 0.8
Z	598.746 ± 0.9	598.744 ± 1.1	598.744 ± 0.9	598.744 ± 0.9	598.744 ± 0.8
C	-0.00640 m ± 1.5 mm	-0.002 m ± 2.3 mm	0.001 m ± 1.4 mm	-0.002 m ± 1.4 mm	-0.0002 m ± 1.0 mm
S	$6 \times 10^{-6} \pm 2 \times 10^{-6}$	$2 \times 10^{-6} \pm 3 \times 10^{-6}$	$-2.5 \times 10^{-6} \pm 2 \times 10^{-6}$	$-5.95 \times 10^{-6} \pm 2.2 \times 10^{-6}$	$-4.7 \times 10^{-6} \pm 2 \times 10^{-6}$
σ_{0i}	0.0032	0.0035	0.0031	0.0029	0.003
			Red		
W	461.113 ± 1.0	461.113 ± 0.8	461.113 ± 1.2	461.113 ± 1.6	461.113 ± 1.4
X	159.231 ± 1.0	159.232 ± 0.8	159.232 ± 1.2	159.232 ± 1.5	159.232 ± 1.4
Y	150.157 ± 1.0	150.157 ± 0.8	150.158 ± 1.2	150.157 ± 1.5	150.157 ± 1.4
Z	598.745 ± 1.0	598.745 ± 0.8	598.744 ± 1.2	598.745 ± 1.6	598.744 ± 1.5
C	0.0041 m ± 1.8 mm	0.005 m ± 1.7 mm	0.001 m ± 2.1 mm	0.005 m ± 3 mm	0.002 m ± 2.5 mm
S	$0.41 \times 10^{-6} \pm 2.9 \times 10^{-6}$	$-13.15 \times 10^{-6} \pm 2.6 \times 10^{-6}$	$2.1 \times 10^{-6} \pm 3.3 \times 10^{-6}$	$-4.3 \times 10^{-6} \pm 4.9 \times 10^{-6}$	$1.5 \times 10^{-6} \pm 4 \times 10^{-6}$
σ_{0i}	0.0039	0.0026	0.004	0.0056	0.0053

Table 10. NGS (1982) vs. ISU (1981) baseline distances.

	NGS		Red 81	
	m	mm	m	mm
W	461.1134 ± 0.4 mm		461.1136 ± 0.5 mm	
X	159.2323 ± 0.4 mm		159.2325 ± 0.5 mm	
Y	150.1576 ± 0.2 mm		150.1578 ± 0.5 mm	
Z	598.7442 ± 0.4 mm		598.7439 ± 0.5 mm	
C			1.7 ± .8 mm	
S			$1.8 \times 10^{-6} \pm 1.4 \times 10^{-6}$	
σ			0.0018	

then the malfunction of the instrument, such as centering error, should be suspected. Since σ_1^2/σ_0^2 satisfies a chi-square distribution $\chi^2_{\alpha,n}$, all observations satisfied this test. Based on periodic measurements, it was found that if the difference between the forward and backward measurement is greater than 1.5 cm, then the observation should be repeated. The most serious problem was the centering error or the failure to enter the atmospheric correction. In all cases repetition of the observation eliminated the problem.

In analyzing $\Delta C/\sigma_{\Delta C} > t_{0.01,14} = 2.62$, it was found (see Table 9) that the values of C for HP in July 1982 were significant at 99%, whereas the values of Red EDM were significant for November 1982. In analyzing $\Delta s/\sigma_{\Delta S} > t_{0.01,14} = 2.62$, it was found that the value of S was significant for observations using HP in July 1982 and July 1983. S was significant for Red EDM for November 1982.

In analyzing $\Delta W/\sigma_{\Delta W}$, $\Delta X/\sigma_{\Delta X}$, $\Delta Y/\sigma_{\Delta Y}$, $\Delta Z/\sigma_{\Delta Z}$, $> t_{0.01,14} = 2.62$, there were no significant changes at 99% confidence level.

In analyzing $\sigma_{02}/\sigma_{01} > F_{0.01,14,14} = 3.7$, no significant changes were found for HP measurements. In the Red EDM there were also no significant changes from July 1982 to October 1983, but comparing May 1981 to July 1983, there was a significant change. But on the other hand, $\sigma_{01}^2/\sigma_0^2 = \chi^2(\alpha,N)$ since $\sigma_0 = 3$ mm for Red EDM and $\chi^2(0.01, 14) = 29$, the $\sigma_{01} = 5.6$ mm is not significant.

14. CONCLUSION AND RECOMMENDATIONS

The HP measurements indicated that there was no movement of the monuments. The large change of σ_0 in the Red EDM from 1981 to 1983 may be due to cyclic effect or malfunction of the refraction correction device. It could be concluded that observations by at least two EDMI is necessary to evaluate any monument movement.

The significant changes in C and S for both the HP and Red EDM conclusively illustrate the usefulness of EDMI calibration. These changes may be due to frequency drift of the carrier wave as well to internal movement of the electronics.

Precautions have to be taken to prevent any centering error. Comparing forward and backward measurement is an effective method of detecting any error in centering.

The EDMI calibration baseline which has been established can only be used to determine the scale and constant errors, but not the cyclic errors. The present baseline could be modified to determine the cyclic error. One method is to build a 10-m long and 5-ft-high wall at one end of the baseline with facility to move the prism every 10 to 50 cms. Another method is to set up monuments every 1 m, 2 m, 3 m, 4 m, and 5 m on either side of the five monuments. It is recommended that these methods be studied and the baseline and computer program be modified to determine the scale, constant, and cyclic error of an EDMI simultaneously.

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Thanks are due members of the National Geodetic Survey for their cooperation in establishing baseline. Thanks are also due to Mr. Vernon Marks and the members of the Iowa Highway Research Board for their generous support. Last but not least thanks are due to C. E. Ekberg, Head of the Civil Engineering Department for his encouragement, and to Wallace Sanders and the staff of the ERI for their assistance in completing the project.

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APPENDIX I

Input for Calibration Program

INPUT FOR CALIBRATION

Name of Instrument, Name of Baseline
Name of Organization, Name of Observer
Date of Observation

Calibrated value of 0-460, its weight; calibrated value of 460-620, its weight; calibrated value of 620-770, its weight; calibrated value of 770-1370, its weight; correction for scale of unknown, its weight; correction for known constant, its weight

observed distance, its weight

observed distance, its weight

----

observed distance, its weight

-1 , 0

APPENDIX II

Sample Input Data

```
100    HP,ISU
200    I.S.U ,JENNISON-OTTO
300    10/16/83
400    461.1134,10,159.2322,10,150.1576,10,598.7442,10,0,0,0,0
1000   598.7437,1
1100   748.9018,1
1200   908.1296,1
1300   1369.2408,1
1400   598.7406,1
1500   150.1595,1
1600   309.3897,1
1700   770.5001,1
1800   748.8990,1
1900   150.1609,1
2000   159.2326,1
2100   620.3444,1
2200   908.1251,1
2300   309.3921,1
2400   159.2276,1
2500   461.1085,1
2600   1369.246,1
2700   770.4963,1
2800   620.3408,1
2900   461.1032,1
3000   -1,0
```

APPENDIX III

Listing of Calibration Program

```
10 rem
20 rem This is a EDMI calibration program giving the constant and scale
30 REM BY THE SECTION METHOD.THIS USES REDUCED DISTANCE AND REL.WEIGHTS
40 REM OBSERVED DISTANCES NEED WEIGHTS.THE SECTION DISTANCE HAS TO BE
50 REM ESTIMATED AND CAN BE WEIGHTED IF KNOWN.THE CONSTANT AND SCALE CAN
60 REM ALSO BE WEIGHTED,IF KNOWN.
70 dim d(10),a(26,6),b(6,26),p(26,26),c(6,26)
80 dim e(6,6),l(26,1),f(6,1),g(6,6),h(6,1),q(26,1)
90 dim v(26,1),u(1,26),r(1,26),t(1,1)
100 INPUT"NAME OF INSTRUMENT AND BASE LINE",A$,B$
110 INPUT "NAME OF ORGANIZATION AND OBSERVER",C$,E$
120 INPUT"DATE OF OBSERVATION",D$
130 PRINT TAB(30);"INSTRUMENT:",A$
140 PRINT " "
150 PRINT TAB(30);"------------------"
160 PRINT " "
170 PRINT TAB,(30);"BASELINE:",B$
180 PRINT " "
190 PRINT TAB(30);"------------------"
200 PRINT " "
210 PRINT TAB(10);"OBSERVER:";TAB(1);E$
220 PRINT TAB(45);"ORGANIZATION:";TAB(1);C$
230 PRINT TAB(10);"----------------------------------------------------------------------------------------------------"
240 PRINT " "
250 PRINT TAB(15);"DATE OF OBSERVATION:",D$
260 PRINT " "
270 PRINT TAB(15);"--------------------------------------"
280 PRINT " "
290 T=0
300 mat read d
310 mat a=zer
320 mat l=zer
330 mat p=zer
340 input w,w1,x,w2,y,w3,z,w4,s0,w5,c0,w6
350 PRINT " "
360 PRINT " "
370 PRINT TAB(30);"OBSERVATIONS BY EDMI"
380 PRINT " "
390 PRINT TAB(30);"----------------------------------"
400 PRINT " "
410 print" "
420 print tab(15);"TRUE VALUES";tab(45);"WEIGHT"
430 print" "
440 print tab(15);"____________";tab(45);"_______"
450 print" "
460 print tab(18);"W=",
470 print using"####.####",w,
480 print tab(47);w1
490 print" "
500 print tab(18);"X=",
510 print using"####.####",x,
520 print tab(47);w2
530 print" "
540 print tab(18);"Y=",
550 print using"####.####",Y,
560 print tab(47);w3
570 print" "
580 print tab(18);"Z=",
590 print using"####.####",z,
600 print tab(47);w4
610 print" "
620 print tab(15);"OBSERVED VALUES";tab(45);"WEIGHT"
630 print" "
640 print tab(15);"________________";tab(45);"______"
650 print" "
660 input s,w0
```

```
670 if s<0 then 1270
680 print" "
690 print tab(20);
700 print using"####.####",s,
710 print tab(47);w0
720 s=s-s*s0-c0
730 i=i+1
740 a(i,5)=s
750 a(i,6)=1
760 p(i,i)=w0
770 if s<d(1) then 920
780 if s<d(2) then 950
790 if s<d(3) then 980
800 if s<d(4) then 1020
810 if s<d(5) then 1050
820 if s<d(6) then 1080
830 if s<d(7) then 1130
840 if s<d(8) then 1170
850 if s<d(9) then 1220
860 a(i,1)=1
870 a(i,2)=1
880 a(i,3)=1
890 a(i,4)=1
900 l(i,1)=s-w-x-y-z
910 go to 650
920 a(i,3)=1
930 l(i,1)=s-y
940 go to 650
950 a(i,2)=1
960 l(i,1)=s-x
970 go to 650
980 a(i,3)=1
990 a(i,2)=1
1000 l(i,1)=s-x-y
1010 go to 650
1020 a(i,1)=1
1030 l(i,1)=s-w
1040 go to 650
1050 a(i,4)=1
1060 l(i,1)=s-z
1070 go to 650
1080 a(i,1)=1
1090 a(i,1)=1
1100 a(i,2)=1
1110 l(i,1)=s-w-x
1120 go to 650
1130 a(i,3)=1
1140 a(i,4)=1
1150 l(i,1)=s-y-z
1160 go to 650
1170 a(i,1)=1
1180 a(i,2)=1
1190 a(i,3)=1
1200 l(i,1)=s-w-x-y
1210 go to 650
1220 a(i,2)=1
1230 a(i,3)=1
1240 a(i,4)=1
1250 l(i,1)=s-x-y-z
1260 go to 650
1270 i=i+1
1280 p(i,i)=w1
1290 a(i,1)=1
1300 i=i+1
1310 p(i,i)=w2
```

```
1320 a(i,2)=1
1330 i=i+1
1340 a(i,3)=1
1350 p(i,i)=w3
1360 i=i+1
1370 a(i,4)=1
1380 p(i,i)=w4
1390 i=i+1
1400 a(i,5)=1
1410 p(i,i)=w5
1420 i=i+1
1430 a(i,6)=1
1440 p(i,i)=w6
1450 mat b=trn(a)
1460 mat c=b*p
1470 mat e=c*a
1480 mat f=c*l
1490 mat g=inv(e)
1500 mat h=g*f
1510 mat q=a*h
1520 mat v=l-q
1530 mat u=trn(v)
1540 mat r=u*p
1550 mat t=r*v
1560 s1=t(1,1)/(i-6)
1570 s2=sqr(s1)
1580 PRINT " "
1590 PRINT " "
1600 PRINT TAB(30);"CALIBRATED LENGTHS"
1610 PRINT " "
1620 PRINT TAB(29);"--------------------"
1630 PRINT " "
1640 FOR I= 1 TO 6
1650 FOR K= 1 TO 6
1660 G(I,K)=S1*G(I,K)
1670 NEXT K
1680 NEXT I
1690 DATA 155,175,325,465,605,625,755,775,925,1405
1700 FOR I=1 TO 6
1710 G(I,I)=SQR(G(I,I))
1720 NEXT I
1730 W=W+H(1,1)
1740 X=X+H(2,1)
1750 Y=Y+H(3,1)
1760 Z=Z+H(4,1)
1770 SO=SO+H(5,1)
1780 CO=CO+H(6,1)
1790 print "INSTRUMENT CONSTANTS:  C=",
1800 PRINT USING"##.##########",CO
1810 PRINT TAB(23);"S=",
1820 PRINT USING"##.##########",SO
1830 PRINT "STD.ERROR OF CONSTANTS:SIG C=";TAB(1);
1840 PRINT USING"##.##########",G(6,6)
1850 PRINT TAB(28);"SIG S=";TAB(1);
1860 PRINT USING"##.##########",G(5,5)
1870 PRINT "BASE LINE CONSTANTS:";TAB(1);
1880 PRINT USING"#####.#####",W
1890 PRINT TAB(20);
1900 PRINT USING"#####.#####",X
1910 PRINT TAB(20);
1920 PRINT USING"#####.#####",Y
1930 PRINT TAB(20);
1940 PRINT USING"#####.#####",Z
1950 print "STD.ERROR OFF CONSTANTS:   ";tab(1);
1960 PRINT USING"##.##########",G(1,1)
```

```
1970 PRINT TAB(25);
1980 PRINT USING"##.##########",G(2,2)
1990 PRINT TAB(25);
2000 PRINT USING"##.##########",G(3,3)
2010 PRINT TAB(25);
2020 PRINT USING"##.##########",G(4,4)
2030 FOR I=1 TO 50
2040 PRINT " "
2050 NEXT I
2060 PRINT "VARIANCE COVARIANCE MATRIX"
2070 FOR I=1 TO 6
2090 G(I,I)=G(I,I)*G(I,I)
2100 PRINT USING"##.##########", G(I,1),G(I,2),G(I,3),G(I,4),G(I,5),G(I,6)
2110 NEXT I
2130 print "APRIORI RESIDUALS"
2140 MAT PRINT L
2150 print "RESIDUALS AFTER ADJUSTMENT"
2160 MAT PRINT V
2170 print "STD.ERROR OF UNIT WEIGHT:";
2180 print using"##.##########",S2
2190 END
```

APPENDIX IV

Sample Output from Calibration Forms

INSTRUMENT: RED 1A

BASELINE: ISU BASELINE

OBSERVER:JENNISON-OTTO

ORGANIZATION:ISU

DATE OF OBSERVATION: 10/16/1983

OBSERVATIONS BY EDMI

TRUE VALUES		WEIGHT
N=	461.1134	1.0
X=	159.2323	1.0
Y=	150.1576	1.0
Z=	598.7442	1.0

OBSERVED VALUES	WEIGHT
598.7491	1
748.9059	1
908.1387	1
1369.2493	1
598.7404	1
150.1686	1
309.3964	1
770.5063	1
748.8961	1

150.1529	1
159.2407	1
620.3544	1
908.1269	1
309.3877	1
159.2329	1
461.1105	1
1369.2538	1
770.4987	1
620.3408	1
461.1124	1

CALIBRATED LENGTHS

```
INSTRUMENT CONSTANTS:  C=     0.0024536406
                       S=    -0.0000015312
STD.ERROR OF CONSTANTS:SIG C= 0.0024743900
                          SIG S= 0.0000040136
BASE LINE CONSTANTS:  461.11365
                      159.22255
                      150.15736
                      598.74400
STD.ERROR OFF CONSTANTS:   0.0014697303
                          0.0014102159
                          0.0014177003
                          0.0015449026
```

VARIANCE COVARIANCE MATRIX

0.0000021601
-0.0000000396
0.0000002359
0.0000004595
-0.0000000019
-0.0000000040
-0.0000000396
0.0000019887
-0.0000000523
0.0000002647
-0.0000000010
-0.0000006584
0.0000002359
-0.0000000523
0.0000020099
0.0000000354
-0.0000000011
-0.0000005910
0.0000004595
0.0000002647
0.0000000354
0.0000023867
-0.0000000027
0.0000003264
-0.0000000019
-0.0000000010
-0.0000000011
-0.0000000027

0.0000000000
-0.0000000067
-0.0000000040
-0.0000006584
-0.0000005910
0.0000003264
-0.0000000067
0.0000061226

APRIORI RESIDUALS

.0049
.0041
.0046
.0010
-.0038
.011
.0065
.003
-.0057
-.0047
.0084
.0007
-.0072
-.0022
.0006
.0051
.0063
-.0046
-.0047
-.001
0
0
0
0
0
0

```
RESIDUALS AFTER ADJUSTMENT
 .356168E-02
 .32325E-02
 .372894E-02
 .138433E-02
-.513033E-02
 .901719E-02
 .451362E-02
 .146902E-02
-.856752E-02
-.668284E-02
 .594281E-02
 .669821E-02
-.807108E-02
-.418639E-02
-.10572E-02
 .310177E-02
 .500434E-02
-.006131
-.690101E-02
-.299824E-02
-.250662E-03
-.247379E-03
 .24089E-03
 .198514E-03
 .153121E-05
-.245364E-02
STD.ERROR OF UNIT WEIGHT: 0.0053102096
```

APPENDIX V

INPUT OF REDUCTION TO HORIZONTAL PROGRAM

Instrument

Project

Organization

Observer

1 (for feet) or 2 (for meters)

Name of station from, name of station to

Elevation of station from, elevation of station to, height of instrument; height of reflector, slope distance (ft/m)

Repeat for all stations

END (type)

APPENDIX VI

Sample Input for Reduction to Horizontal Program

```
'RED'
'DOT'
'ISU'
'JOEL'
'MAY 81'
1
A,B
1059.01,1055.60,4.85,5.10,1964.39
A,C
1059.01,1054.94,4.85,4.80,2457.02
A,D
1059.01,1053.90,4.85,5.15,2979.45
A,E
1059.01,1050.75,4.85,4.90,4492.31
B,C
1055.60,1054.94,5.10,5.10,492.66
B,E
1055.60,1050.75,5.10,5.10,2527.91
B,D
1055.60,1053.90,5.10,5.10,1015.07
B,A
1055.01,1059.01,5.10,5.10,1964.39
C,A
1054.94,1059.01,4.80,4.80,2457.05
C,B
1054.94,1055.01,4.80,4.80,492.65
C,D
1054.94,1053.90,4.80,4.80,522.42
C,E
1054.94,1050.75,4.80,4.80,2035.27
D,E
1053.90,1050.75,5.15,5.15,1512.85
D,C
1053.90,1054.94,5.15,5.15,522.42
D,B
1053.90,1055.60,5.15,5.15,1015.07
D,A
1053.90,1059.01,5.15,5.15,2979.45
E,D


1050.75,1053.90,4.90,4.90,1512.84
E,C
1050.75,1054.94,4.90,4.90,2035.27
E,B
1050.75,1055.60,4.90,4.90,2527.91
E,A
1050.75,1059.01,4.90,4.90,4492.31
END,END
1050.75,1059.01,4.90,4.90,4492.31
```

APPENDIX VII

Listing of Reduction to Horizontal Program

```
      REAL EF,ET,SLD,HD(1000),HI,HR,DE,SDEV
       CHARACTER*4 FR(1000),TW(1000)
      CHARACTER*20 A
      OPEN(UNIT=100,TYPE='NEW',NAME='RDHZ.OUT')
      PRINT*,'INSTRUMENT:  '
      READ*,A
      WRITE(100,*),'INSTRUMENT:   ',A
      WRITE(100,*),' '
      WRITE(100,*),' '
      PRINT*,'PROJECT:        '
      READ*,A
      WRITE(100,*),'PROJECT:         ',A
      WRITE(100,*),' '
      WRITE(100,*),' '
      PRINT*,'ORGANIZATION:'
      READ*,A
      WRITE(100,*),'ORGANIZATION:',A
      WRITE(100,*),' '
      WRITE(100,*),' '
      PRINT*,'OBSERVER:       '
      READ*,A
      WRITE(100,*),'OBSERVER:        ',A
      WRITE(100,*),' '
      WRITE(100,*),' '
      PRINT*,'DATE:           '
      READ*,A
      WRITE(100,*),'DATE:            ',A
      WRITE(100,*),' '
      WRITE(100,*),' '
      PRINT*,'ENTER 1 IF DISTANCE IN FEET OR 2 IF DISTANCE IN
     * METERS'
      READ*,IFLAG
      IF(IFLAG.EQ.1)THEN
      GO TO 10
      ELSE
      GO TO 20
      END IF
   10 WRITE(100,50),'FROM','TO','ELEVATION FROM','ELEVATION
     * TO','SLOPE  DIST','HORIZ.DIST'
      WRITE(100,75),'(ft)','(ft)','(ft)','(m)'
   75 FORMAT(' ',28X,A4,13X,A4,11X,A4,10X,A3)
   50       FORMAT('0',5X,A4,4X,A2,8X,A14,3X,A13,3X,A11,4X,A10)
       PRINT*,'FR TO'
        READ(5,150) FR(1),TW(1)
       I=1
  150   FORMAT(A4)
       PRINT*,'EF,ET,HT,HR,SLD'
        READ*,EF,ET,HI,HR,SLD
            DO WHILE(FR(I).NE.'END')
          DE=ABS((EF+HI)-(ET+HR))
      HD(I)=(SQRT(SLD**2-DE**2)
     * -DE*SIN(SLD*4.935/(3600000.0*57.29577951)))*0.3048
          WRITE(100,100),FR(I),TW(I),EF,ET,SLD,HD(I)
       I=I+1
       PRINT*,'FR,TO'
      READ(5,150) FR(I),TW(I)
       PRINT*,'EF,ET,HI,HR,SLD'
      READ*,EF,ET,HI,HR,SLD
           END DO
         GO TO 30
   20 WRITE(100,50),'FROM','TO','ELEVATION FROM','ELEVATION
     * TO','SLOPE  DIST','HORIZ.DIST'
      WRITE(100,75),'(FT)','(FT)','(M) ','(M)'
      PRINT*,'FROM TO'
        READ(5,150) FR(1),TW(1)
       I=1
```

```
          PRINT*,'EF,ET,HI,HR,SLD'
            READ*,EF,ET,HI,HR,SLD
            DO WHILE(FR(I).NE.'END')
        DE=ABS((EF+HI)-(ET+HR))
              HD(I)=(SQRT((SLD/0.3048)**2-DE**2)
     *  -DE*SIN((SLD/0.3048)*4.935/(3600000.0*57.29578)))*0.3048
              WRITE(100,100),FR(I),TW(1),EF,ET,SLD,HD(I)
              I=I+1
              PRINT*,'FROM TO'
              READ(5,150) FR(I),TW(I)
                 PRINT*,'EF,ET,HI,HR,SLD'
               READ*,EF,ET,HI,HR,SLD
            END DO
 100   FORMAT('0',6X,A4,3X,A4,3X,F13.4,5X,F13.4,4X,F11.4,3X,F11.4)
 30       I=I-1
        WRITE(100,*),' '
        WRITE(100,*),' '
        WRITE(100,*),' '
        WRITE(100,*),'DIFFERENCES IN HOR.DIST MEASURED BACK AND FORTH'
        WRITE(100,*),'***********************************************'
        WRITE(100,*),' '
        SDEV=0
        DO 102 J=1,I
        IF(J.NE.1)THEN
        DO 302 K=J-1,1,-1
        IF(FR(J).EQ.TW(K).AND.FR(K).EQ.TW(J))THEN
        SDEV=SDEV+(HD(J)-HD(K))**2
        WRITE(100,101),TW(J),FR(J),ABS(HD(J)-HD(K))
 101    FORMAT('0','DIFFERENCE OF MEASUREMENTS FROM',1X,A2,
     * 'AND',1X,A2,'IS: ',F10.5)
          END IF
 302    CONTINUE
          END IF
 102    CONTINUE
        SDEV=SQRT(SDEV/(I))
        WRITE(100,*),'*****************************************
     * *********************************************'
        WRITE(100,*),' '
        WRITE(100,*),' '
        WRITE(100,103),SDEV
 103    FORMAT('0','STD.ERROR OF DIFF. IN OBSERVATION=+/-',F8.5)
                  STOP
                      END
```

APPENDIX VIII

Sample Output from Reduction to Horizontal Program

INSTRUMENT: RED

PROJECT: BOT

ORGANIZATION:ISU

OBSERVER: JOEL

DATE: MAY 81

FROM	TO	ELEVATION FROM (ft)	ELEVATION TO (ft)	SLOPE DIST (ft)	HORIZ.DIST (m)
A	B	1059.0100	1055.6000	1964.3900	598.7452
A	C	1059.0100	1054.9399	2457.0200	748.8986
A	D	1059.0100	1053.9000	2979.4500	908.1349
A	E	1059.0100	1050.7500	4492.3101	1369.2535
B	C	1055.6000	1054.9399	492.6600	150.1626
B	E	1055.6000	1050.7500	2527.9099	770.5054
B	D	1055.6000	1053.9000	1015.0700	309.3929
B	A	1055.0100	1059.0100	1964.3900	598.7448
C	A	1054.9399	1059.0100	2457.0500	748.9078
C	B	1054.9399	1055.0100	492.6500	150.1597
C	D	1054.9399	1053.9000	522.4200	159.2333
C	E	1054.9399	1050.7500	2035.2700	620.3489
D	E	1053.9000	1050.7500	1512.8500	461.1156
D	C	1053.9000	1054.9399	522.4200	159.2333
D	B	1053.9000	1055.6000	1015.0700	309.3929
D	A	1053.9000	1059.0100	2979.4500	908.1349
E	D	1050.7500	1053.9000	1512.8400	461.1126
E	C	1050.7500	1054.9399	2035.2700	620.3489
E	B	1050.7500	1055.6000	2527.9099	770.5054
E	A	1050.7500	1059.0100	4492.3101	1369.2535

DIFFERENCES IN HOR.DIST MEASURED BACK AND FORTH
****

DIFFERENCE OF MEASUREMENTS FROM A AND B IS: 0.00037

DIFFERENCE OF MEASUREMENTS FROM A AND C IS: 0.00916

DIFFERENCE OF MEASUREMENTS FROM B AND C IS: 0.00291

DIFFERENCE OF MEASUREMENTS FROM C AND D IS: 0.00000

DIFFERENCE OF MEASUREMENTS FROM B AND D IS: 0.00000

DIFFERENCE OF MEASUREMENTS FROM A AND D IS: 0.00000

DIFFERENCE OF MEASUREMENTS FROM D AND E IS: 0.00305

DIFFERENCE OF MEASUREMENTS FROM C AND E IS: 0.00000

DIFFERENCE OF MEASUREMENTS FROM B AND E IS: 0.00000

DIFFERENCE OF MEASUREMENTS FROM A AND E IS: 0.00000
****

STD.ERROR OF DIFF. IN OBSERVATION=+/- 0.00226

APPENDIX IX

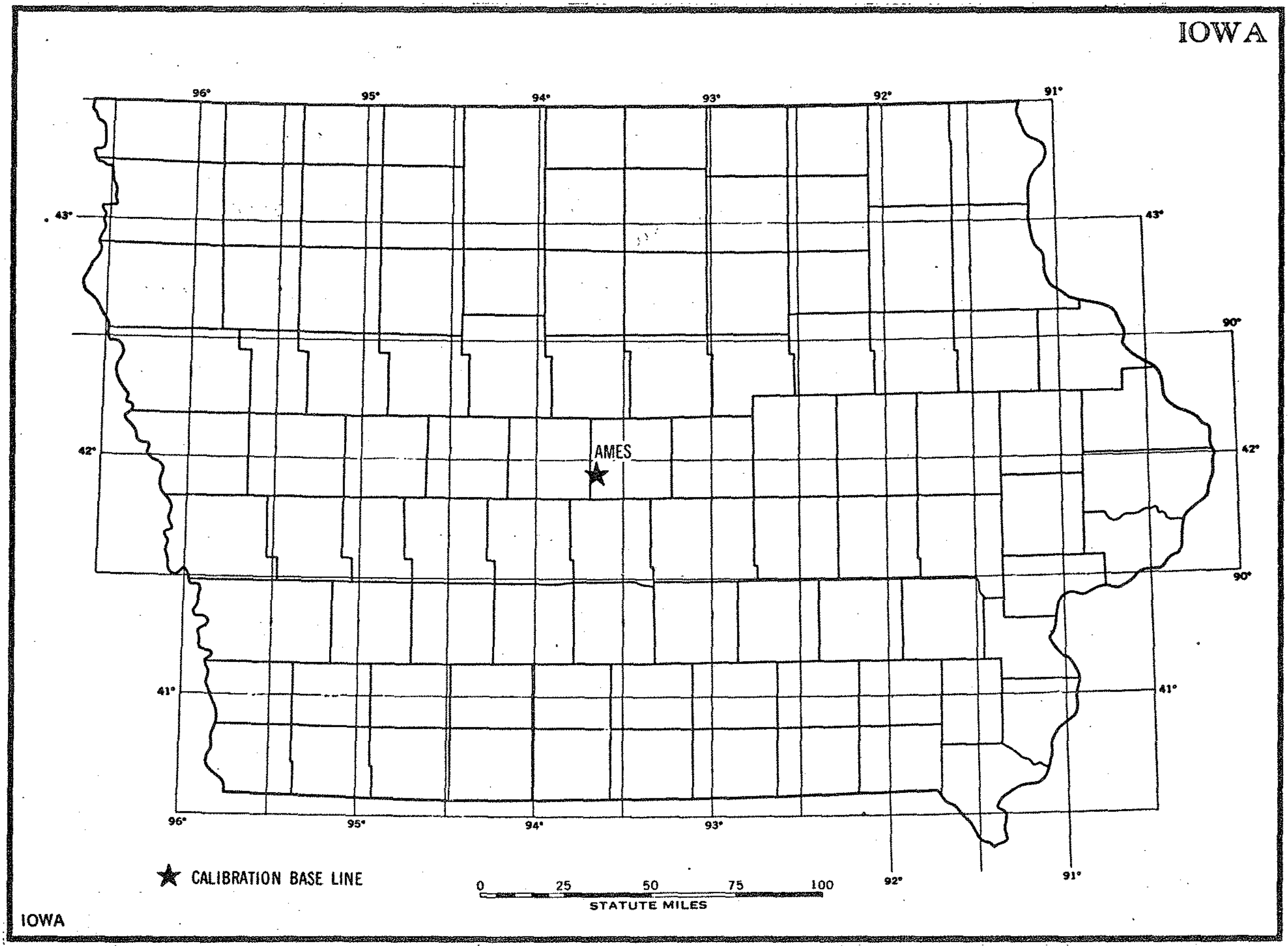
ISU Baseline Information Published by NGS

UNITED STATES DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION (NOAA)
WASHINGTON D.C.

CALIBRATION BASE LINES

STATE OF IOWA

COMPILED AND PUBLISHED BY THE NATIONAL GEODETIC SURVEY (NGS)
A COMPONENT OF THE NATIONAL OCEAN SURVEY (NOS)
ROCKVILLE MD 20852 - FEB 1983


IOWA
96°
95°
94°
93°
92°
91°
90°
43°
42°
41°
AMES
CALIBRATION BASE LINE
0
25
50
75
100
STATUTE MILES
IOWA

DEPARTMENT OF COMMERCE - NOAA
- NATIONAL GEODETIC SURVEY
KVILLE MD 20852 - FEBRUARY 2, 1983

CALIBRATION BASE LINE REPORT

CONTENTS

US DEPARTMENT OF COMMERCE - NOAA
NOS - NATIONAL GEODETIC SURVEY
ROCKVILLE MD 20852 - FEBRUARY 2, 1983

CALIBRATION BASE LINE DATA
BASE LINE DESIGNATION: AMES
PROJECT ACCESSION NUMBER: G17034

QUAD: N410934
IOWA
STORY COUNTY

LIST OF ADJUSTED DISTANCES (DECEMBER 14,1982)

FROM STATION	ELEV.(M)	TO STATION	ELEV.(M)	ADJ. DIST.(M) HORIZONTAL	ADJ. DIST.(M) MARK - MARK	STD. ERROR(MM)
0	312.421	461	313.370	461.1134	461.1144	0.4
0	312.421	620	313.684	620.3457	620.3470	0.5
0	312.421	770	313.889	770.5033	770.5047	0.5
0	312.421	1370	314.894	1369.2477	1369.2500	0.7
461	313.370	620	313.684	159.2323	159.2326	0.4
461	313.370	770	313.889	309.3900	309.3904	0.4
461	313.370	1370	314.894	908.1344	908.1357	0.6
620	313.684	770	313.889	150.1576	150.1578	0.2
620	313.684	1370	314.894	748.9021	748.9030	0.4
770	313.889	1370	314.894	598.7444	598.7453	0.4

DESCRIPTION OF AMES BASE LINE
YEAR MEASURED: REP
CHIEF OF PARTY: 1982

THE BASE LINE IS LOCATED ABOUT 6.4 KM (4 MI) SOUTHWEST OF AMES, PARALLEL TO AND ALONG THE SOUTH SIDE OF A GRAVEL ROAD WHICH INTERSECTS WITH STORY COUNTY ROAD R 38 TO THE EAST OF BASE LINE.

TO REACH THE BASE LINE FROM THE JUNCTION OF U.S. HIGHWAY 69 AND LINCOLN WAY (OLD U.S. HIGHWAY 30) IN AMES, GO WEST ON LINCOLN WAY FOR 4.8 KM (3.0 MI), TO STORY COUNTY R 38 (SOUTH DAKOTA AVE). TURN LEFT ON STORY COUNTY R 38 (PAVED SURFACE) AND GO SOUTH FOR 3.2 KM (2.0 MI) TO GRAVEL CROSSROAD. TURN RIGHT AND GO WEST FOR 0.2 KM (0.15 MI) TO THE 0 METER POINT ON LEFT (ABOUT THE SAME ELEVATION AS THE ROAD).

THE 0 METER POINT IS A STANDARD NGS DISK SET INTO THE TOP OF AN IRREGULAR MASS OF CONCRETE 33 CM (13 IN) IN DIAMETER PROJECTING 8 CM (3 IN) ABOVE THE GROUND LOCATED 6.4 M (21 FT) S FROM THE CENTER OF A GRAVEL ROAD, 3.6 M (12 FT) N FROM A WIRE FENCE, AND 2.4 M (8 FT) WNW FROM A ROAD SIGN. THE 461 METER POINT IS A STANDARD NGS DISK SET INTO THE TOP OF A ROUND CONCRETE MONUMENT 55 CM (22 IN) IN DIAMETER FLUSH WITH GROUND LOCATED 5.8 M (19 FT) S FROM THE CENTER OF A GRAVEL ROAD, 4.3 M (14 FT) N FROM A WIRE FENCE, 5.2 M (17 FT) NW FROM A POWER LINE POLE, AND 0.6 M (2 FT) LOWER THAN THE GRAVEL ROAD. THE 620 METER POINT IS A STANDARD NGS DISK SET INTO THE TOP OF A ROUND CONCRETE MONUMENT 44 CM (17 IN) IN DIAMETER PROJECTING 5 CM (2 IN) ABOVE THE GROUND LOCATED 6.1 M (20 FT) S FROM THE CENTER OF A GRAVEL ROAD, 4.0 M (13 FT) N FROM A WIRE FENCE, 32.3 M (106 FT) W FROM A POWER LINE POLE, AND 0.6 M (2 FT) LOWER THAN GRAVEL ROAD. THE 770 METER POINT IS A STANDARD NGS DISK SET IN THE TOP OF A ROUND CONCRETE MONUMENT 33 CM (13 IN) IN DIAMETER FLUSH WITH GROUND LOCATED 6.4 M (21 FT) S FROM THE CENTER OF A GRAVEL ROAD, 4.3 M (14 FT) N FROM A WIRE FENCE, 8.2 M (27 FT) E FROM THE CENTER OF A TRACK ROAD LEADING SOUTH TO A RADIO TOWER AND BUILDING, AND 0.9 M (3.0 FT) LOWER THAN GRAVEL ROAD. THE 1370 METER POINT IS A STANDARD NGS DISK SET INTO THE TOP OF A ROUND CONCRETE MONUMENT 30 CM (12 IN) IN DIAMETER PROJECTING 10 CM (4 IN) ABOVE THE GROUND LOCATED 5.8 M (19 FT) S FROM CENTER OF A GRAVEL ROAD, 4.0 M (13 FT) N FROM A WIRE FENCE, 69.8 M (229 FT) E FROM CENTER OF GRAVEL ROAD (NORTH-SOUTH) INTERSECTING WITH EAST-WEST GRAVEL ROAD, AND 0.3 M (1.0 FT) LOWER THAN THE GRAVEL ROAD. NONE OF THE DISKS ARE STAMPED.

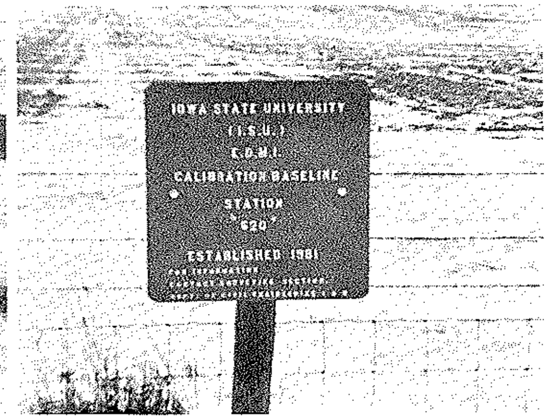
THE BASE LINE IS A EAST-WEST BASE LINE WITH THE 0 METER POINT ON THE EAST END. IT IS MADE UP OF 0, 461, 620, 770, AND 1370 METER POINTS. ALL OF THE MARKS ARE SET ON A LINE SOUTH OF AND PARALLEL TO THE EAST-WEST GRAVEL ROAD.

THE BASE LINE WAS ESTABLISHED IN CONJUNCTION WITH THE IOWA STATE UNIVERSITY AT AMES, IOWA. FOR FURTHER INFORMATION, CONTACT DEPARTMENT OF CIVIL ENGINEERING, IOWA STATE UNIVERSITY, AMES, IOWA 50011. TELEPHONE (515) 294-3532 OR 6324.

APPENDIX X

Signs on ISU Baseline


IOWA STATE UNIVERSITY
(I.S.U.)
E.D.M.I.
CALIBRATION BASELINE
STATION
1370
ESTABLISHED 1981


IOWA STATE UNIVERSITY
(I.S.U.)
E.D.M.I.
CALIBRATION BASELINE
STATION
620
ESTABLISHED 1981


IOWA STATE UNIVERSITY
(I.S.U.)
E.D.M.I.
CALIBRATION BASELINE
STATION
0
ESTABLISHED 1981