SELECTION OF SURVEYING METHODS

by

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INTRODUCTION

Purpose

This investigation provides a means for judging the adequacy of the commonly available plane-surveying techniques and instruments in meeting pre-determined accuracy goals. These methods and instruments include: 1) plane table and alidade; 2) elevations by differential leveling, vertical angles, and altimetry; and 3) locations by traverse with distances by tacheometry, taping, and electronic devices, and directions by vernier transits, optical transits,and theodolites.

Because most systematic errors can be corrected or compensated for, the several sources of accidental error limit the attainable accuracy of each method or instrument. By determining, for each of the methods, the magnitude of these errors as a function of different conditions and instrument design, it should be possible to estimate the error in any future measurement. The Theory of Errors, which predicts the resultant of a series of independent accidental errors, could then be used to estimate the error for any measured point in a survey.

This is a study and tabulation of the errors in the methods; the determination of required precision rests with the user. Therefore, though the illustrations are drawn from geology, the principles and data have general application. The methods and their appropriate field procedures are well documented in existing texts and will not be described here. Simiarly, though photogrammetry is the optimal method for mapping any substantial area when the measured points can be identified on photographs, there already exists a substantial body of literature on the errors of these methods and selection has been treated by others (Aguilar 1967, 1969).

One or more of the methods considered herein would be used when: (1) available data lack the required accuracy, or (2) sketching with the aid of pace and compass, hand level, measuring wheel, range finder, and other reconnaissance methods are inadequate for the task at hand.

Meyer (1949, 1954) found errors of $\pm$0.6 degrees for angles measured with a compass similar in design to the geologist's Brunton. Brinker and Taylor (1961) reported errors of 1:20 for distance by pacing, and 1:50 for auto odometers.

Applications

Although relatively imprecise values for location, direction, and elevation are sufficient for many geologic studies, some investigations do require more precise values.

The construction of contour maps with small contour intervals (less than existing USGS topographic maps of the same area) that show topography, structure, or other parameters which include a term for point elevation would require elevations of the points correct to some fraction of the chosen contour interval. The exact size of the fraction is open for debate: the USGS Topographic Instructions call for vertical map control points to be correct within one tenth of the contour interval, the National Map Accuracy Standards (Appendix 2) provide that 90 percent of the elevations of well defined points, as interpolated from contours, be correct within one-half of the contour interval. Although there are no such standards for structural and other interpretive maps, contours at intervals smaller than the errors in measurement are obviously meaningless.

For the study of ground-water flow in unconfined aquifers, differences in water level elevations of a few tenths of a foot may be significant; therefore, a corresponding accuracy in well head elevations is necessary. Most such studies are on river flood plains where contour lines are widely spaced and for technical reasons are located less precisely than on neighboring hills, hence interpolated values would almost always be inadequate.

Attempts are sometimes made to detect and/or measure small surface displacements which may be attributed to tectonic activity, fault creep, elastic rebound, or other dynamic geologic process. In the exploitation of some earth resources there may be an accompanying subsidence or upheaval of the surface whose magnitude or areal extent may be of geologic significance. In these cases, very precise measurements are often required which necessitate the use of first order geodetic instruments and methods over the affected areas.

Other Investigations

Appendix 1 is a short subject index to the selected references. Values for errors reported in the literature are given in Appendices 4 and 5.

Several authors (Aquilar 1973, Veress 1973, Vreeland 1969, Wolf 1969) have discussed the distribution and propogation of surveying errors from different theoretical viewpoints. Unfortunately these authors did not include values for substitution into their error equations nor did they describe how these needed values might be obtained.

A second body of literature consists of papers on the errors of a particular instrument or a small group of related instruments. Some authors did quote values for the errors but the statistical measure of the errors was not consistent from paper to paper. Also, many reports were written by persons representing, or in the employ of the manufacturer of the particular instrument tested and thus had a vested interest in the results. Objective studies of groups of instruments include: (1) errors in precise leveling -

Lee and Karren (1964), Karren (1964), Geisler and Papo (1967); (2) errors in tacheometric measurements - Turpin (1954), Mussetter (1956a, 1956b), Colcord (1971); (3) errors in taping - Colcord and Chick (1968), Wood (1969), Golley and Sneddon (1974); (4) errors in altimetry - Greundler and others (1970, 1972); and (5) errors in solar azimuths - Berry (1958), Vanderaa (1964).

A third source of information is from texts and handbooks on surveying such as Brinker and Taylor (1961), Kissam (1966), Bomford (1971), Ewing and Mitchell (1970), and Clark (1973). These authors detailed the methods of correction and reduction of errors, but generally did not quote specific errors for particular instruments.

Methods of Investigation

Accidental errors in surveying result from the operator's inability to perform perfectly some task in orienting the instrument or in determining the indicated values. Common tasks include centering level bubbles, pointing at (bisecting) targets with crosshairs, matching marks, and reading instrument scales and rod intercepts.

The size of the resulting error in measurement depends upon both the operator's ability and the magnification or resolution of the instrument. Furthermore, if each task is performed independently, then each should make an independent contribution to the resultant error. Thus the resultant should be equal to a summation by the Theory of Errors of individual task errors.

Table 1 summarizes available data on measurement errors and includes error equations for those methods for which error information is partially or completely lacking. Sources and values for the available information are in Appendices 4 and 5. Several tests for measuring the error of some types

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able 1. Status of data on errors in surveying methods.

METHOD	TASKS	AVAILABLE DATA	NEEDED DATA	ERROR EQUATION
easurement of levations		Errors in altimetry		
differential eveling - recise levels	Center bubble Read rod	Test for centering error Reading error for precise invar rod		$([c \times sl]^2 + rr^2)^{\frac{1}{2}}$
differential eveling - rdinary levels, lidades with triding levels, ransit telescopes	Center bubble Read rod	Approximate equation for centering error	Rod reading error	$([c \times sl]^2 + rr^2)^{\frac{1}{2}}$
levations- ifferential eveling using ertical arc level	Center arc level bubble Set telescope with vertical arc Read rod	Approximate equation for centering error Setting error = Vertical angle Reading error	Rod reading error Setting error (mark matching error using Beaman Arc)	$([c \times sl]^2 + [m \times sl]^2 + rr^2)^{\frac{1}{2}}$
trigonometric eveling	Center arc level Read vertical angle Read rod Determine distance	Approximate equation for centering error Angle reading error Distance error for some methods	Rod reading error Stadia distance error	$([c \times sl]^2 + [var \times sl]^2 + rr^2 + [sl \times \tan va]^2)^{\frac{1}{2}}$
Beaman method	Center arc level Match Beaman and index marks Read upper, middle and lower rod intercepts	Approximate equation for centering error	Rod reading error Mark matching error	$([c \times sl]^2 + rr^2 + [m \times sl]^2 + [Beaman \times \sqrt{rr^2 + rr^2}]^2)^{\frac{1}{2}}$

Table 1. (Continued)

METHOD	TASKS	AVAILABLE DATA	NEEDED DATA	ERROR EQUATION
-self-reducing tacheometers (Wild RDS, RK-1)	Center arc level or plate level Point lower hair at rod target Read middle hair intercept	Approximate equation for centering error Pointing error = ½ reading error for rods Factor = 10,20,50, or 100	Rod reading error	$([c \times sl]^2 + p^2 + [\text{factor} \times \sqrt{p^2 + rr^2}]^2)^{\frac{1}{2}}$
Measurement of Directors	Pointing instrument at target Setting initial and reading final angle or reading 2 angular values	Reading errors or approximations for theodolites, optical and vernier transits Setting error = reading error	Effect of target design on pointing error Plane table alignment and plotting errors	$(s^2 + ar + p^2\ ar^2)^{\frac{1}{2}}$ or $(p^2 + ar^2 + p^2 + ar^2)^{\frac{1}{2}}$
Measurement of Distances		Errors in EDM Errors in taping Errors in using subtense bar Errors in using optical wedge telemeter		
-stadia method, transit and alidade	Read upper and lower rod intercepts Determine factor for inclined shots	Vertical angle reading error for determining factor from table	Rod reading error Factor reading error with Beaman arc	$\text{factor} \times (rr^2 + rr^2)^{\frac{1}{2}}$
-stadia method, self-reducing tacheometers	Point lower hair at rod target Read upper hair intercept	Factor constant at 100 Pointing error = ½ reading error for rods	Rod reading error	$100 \times (p^2 + rr^2)^{\frac{1}{2}}$

s = circle setting error, ar = angle reading error, p = pointing error, rr = rod reading error, c = bubble centering error in radians, sl = sight length, var = vertical angle reading error in radians, va = vertical angle, m = mark matching error in radians, tan = tangent of angle, x = multiplication, EDM = Electronic Distance Measurement of precise instruments were described in the literature and are referenced in Appendices 3, 4, and 5. Experiments to determine missing values and/or to evaluate existing tests for applicability to less precise instruments and methods are described in Appendix 3. Available data, new experimental values, and error equations were summarized in graphs described and used in the following sections.

DETERMINATION OF APPROPRIATE PROCEDURES

Accuracy Goals and Instrument Precision

The first step in the survey process is to determine the required degree of accuracy. Only the user can make this determination because only he knows the objectives. Because systematic errors, being correctable, are specifically excluded from consideration, and because replicate measurements are seldom made in surveying, the accuracy of the point location is a direct function of the precision of the measurement. As a general rule the survey measurements must be precise enough so as not to detract from the usefulness of the other observations and determinations made at the measured point. Survey measurements more precise than the observations and determinations probably mean a waste of time, money, and effort unless a more detailed study is planned for a later date. If maps are the goal then the National Map Accuracy Standards (Appendix 2) might be considered as a guide to the accuracy of planimetric detail. Contour maps have already been discussed by way of example.

Tentative Plan

Site examination.--Assuming all available data on the project area have been assembled, the next task is to consider physical conditions of the site. If the area is large, then a control survey of some type is necessary to maintain constant scale throughout the study area. Relief, surface texture, and vegetative cover may render some instruments and methods impossible; for example taping in badlands, or differential leveling in dense scrub or forest.

Control.--If values tied to a geodetic datum are needed, or a large area is involved, the next step is to seek out available control. Routes of control surveys by Federal agencies are shown on a series of 1:250,000 Geodetic Control Diagrams available from the USGS map distribution centers. Each diagram covers the same area as a sheet of the 1:250,000 U.S. series of topographic maps, and each bears a legend outlining the procedure for obtaining the necessary data sheets. These data sheets contain the appropriate geodetic coordinates and descriptions of the physical locations of each monument. National Ocean Survey (formerly US Coast and Geodetic Survey) control is also shown on pairs of state maps available from that agency.

If this existing control is suitably located in a large study area, it may reduce or eliminate the need for a separate control survey. Errors between points of an existing survey can be evaluated by use of agency standards (National Ocean Survey, 1974).

Tentative traverse plan.--The next step is to formulate a tentative survey route which may be outlined on available maps or on a sketch. Distances from control or arbitrary reference to the measured point should be kept to a minimum. The route should follow lines of convenient access (roads, railroads, rights-of-way) wherever possible, and lines of any significant length should be closed on some known point. It should then be possible to estimate the average and maximum distances separating measured points from control. The above information will be used in the next sections to determine which methods are adequate.

Horizontal Measurements

Statistical parameters.--A measurement error, which is the familiar univariate statistic, differs from a location error in two dimensions (horizontal plane) which is a bivariate statistic. This difference is important because the standard deviation, the measure of error, represents different percentages of the normal population; some 68 percent for the univariate population, and 39 percent for the bivariate population. This difference is reflected in the multipliers used to arrive at other confidence limits.

The bivariate value is derived from two univariate measurements. The general case at any particular confidence level describes the measured point as lying within ellipse of certain size and shape. Rigorous solution for this type of error distribution and the summation of several such errors requires matrix algebra and terms for co-variance and is too complex for general application.

If the errors in the univariate measurements are orthogonal (at 90 degrees), then the axes of the ellipse coincide with the directions of the univariate errors. Further, if the univariate errors are equal then the ellipse becomes a circle. For this particular case, the bivariate standard deviation equals the univariate standard deviation. One can then be 68 percent sure that the true value lies within one standard deviation of each of the univariate measurements, or be 39 percent sure the true value lies within a radius of one standard deviation of the measured location. Other common confidence limits can be calculated from this value by use of Table 2.

Examination of the general method of traversing shows that the error in a measured point is due to an error in distance and an error in direction and that these errors are always orthogonal. If the precision of the traverse components are approximately equal, then the location error ellipse approaches

Table 2. Probability level conversion factors (Greenwalt and Shultz, 1962)

From	To 50% PE	68.27% SD	90% MAS	99.73% NC
	Linear Error Conversion Factors (univariate)			
50%	1.0000	1.4826	2.4387	4.4475
68.27%	0.6745	1.0000	1.6449	3.0000
90%	0.4101	0.6080	1.0000	1.8239
99.73%	0.2248	0.3333	0.5483	1.0000
	Circular Error Conversion Factors (bivariate)			
From	To 39.35% CSD	50% CPE	90% CMAS	99.78% CNC
39.35%	1.0000	1.1774	2.1460	3.5000
50%	0.8493	1.0000	1.8227	2.9726
90%	0.4660	0.5486	1.0000	1.6309
99.78%	0.2857	0.3364	0.6131	1.0000
	Spherical Error Conversion Factors (trivariate)			
From	To 19.9% SSD	50% SPE	90% SMAS	99.89% SNC
19.9%	1.0000	1.538	2.500	4.000
50%	0.650	1.000	1.625	2.600
90%	0.400	0.615	1.000	1.600
99.98%	0.250	0.385	0.625	1.000

PE - Probable error, SD - Standard Deviation, MAS = Map Accuracy Standard, NC = Near Certainty the circular ideal.

If the error in measuring each location has a near-circular distribution, then the error in any series of locations can be determined by using the Theory of Errors and the radius of a circle substituted for the ellipse. As a conservative estimate of the radius, one could use the value of the larger of the two errors, or one could use a circular substitution equation such as that developed by Greenwalt and Shultz (1962):

$$\sigma_{circular} = (0.5222\ \sigma_{min.} + 0.4778\ \sigma_{max.})$$

where σ is the standard deviation. They cautioned against the application of this equation where $\sigma_{min}/\sigma_{max}$ is smaller than 0.6.

Error charts.--Charts in this and the following discussion on vertical measurements contain plots for generalized types of instruments. Also included are actual test values for a few specific instruments that are of particular interest and which will be used in later examples. Actual test values for a number of other instruments are in Appendices 4 and 5. The user may produce similar plots for the specific instruments he wishes to consider from information in Appendices 3, 4, and 5.

Figure 1, based on the experimental derivation of rod-reading error (Appendix 3), shows the error in stadia distance measurements as a function of sight length, rod-graduation size, and telescope magnification.

Figure 2 shows the errors involved in most of the commonly available distance measurement methods or devices. Plane table errors (Table 10, Appendix 3) could not be shown because plotting error, the major source of error, causes a resultant error that is a function of the scale of the map (the smallest measured error was ± 1 min., ± 60 sec. on the figure).

These or similar charts enable the user to select balanced traverse components and to estimate the error in any measurement. The optical methods of distance measurement are severely limited in range, whereas direction

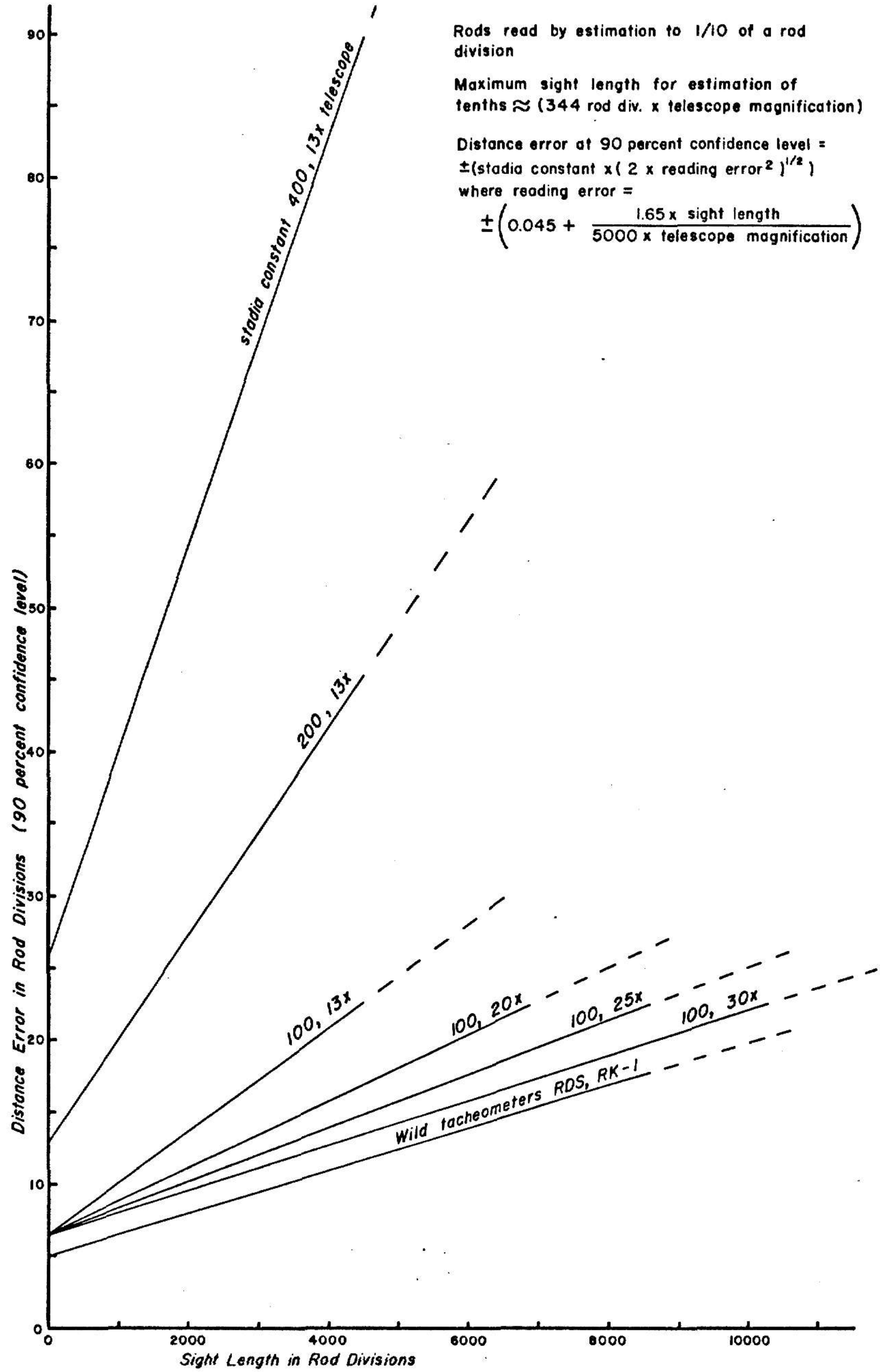

Figure 1 Errors in Stadia Distance Measurements

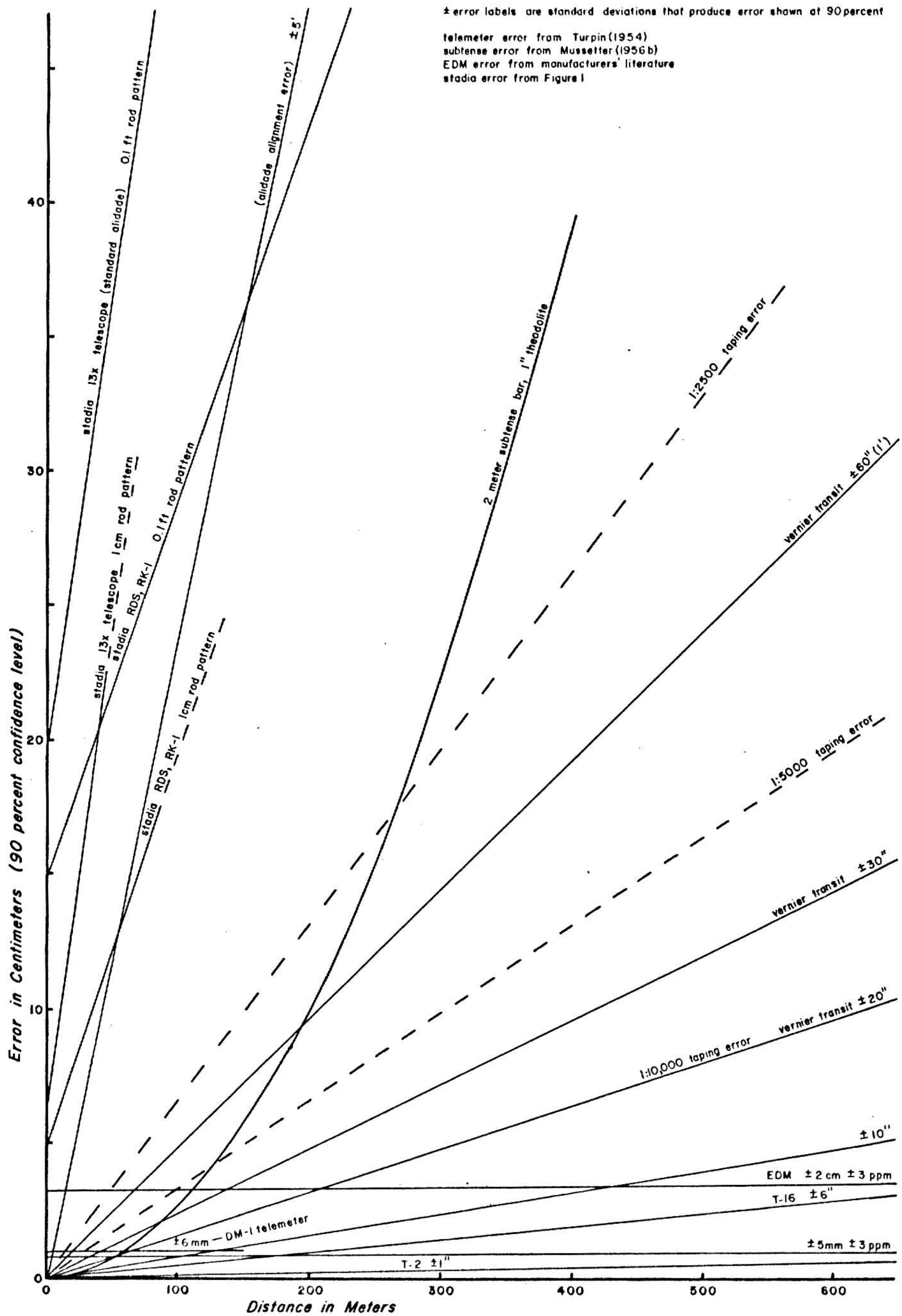

Figure 2 Errors in Horizontal Measurements

measurements are limited only by the need for intervisibility and the need for a suitable target.

Estimates of the individual errors of the tentative traverse can be derived from the graph and combined to give a bivariate error radius for each single location. This value must then be multiplied by the appropriate constant to convert it to the desired probability level. The individual error radii can then be combined using the Theory of Errors to give the location error of any point on the traverse. Practically, the spacing between traverse stations will probably depend upon the methods of measuring elevation, most of which have a short range. Therefore, this calculation must be postponed until the method of elevation measurement has been selected if both are to be done in one operation.

Vertical Measurements

Statistical parameters.--For true geometric location in three dimensional space there is a corresponding trivariate statistic and error ellipsoid. However, because elevations almost always are used, and often determined independently of exact horizontal location, the univariate statistic is appropriate for most uses. For spherical substitutions and confidence limits see Greenwalt and Shultz (1962).

Error charts.--Figure 3 shows the expected error in one sight for the standard exploration model alidade. Figure 4 shows the expected elevation error for the Wild RDS and RK-1 tacheometers. Both figures are based on the equations in Table 1 and experimental values in Appendix 3.

Errors for most optical methods of elevation measurement are shown in Figure 5. Trigonometric leveling depends upon the accuracy of both the vertical angle measurement and the distance measurement. Values can be derived using the equation in Table 1 and data in Appendix 4, but there are too many

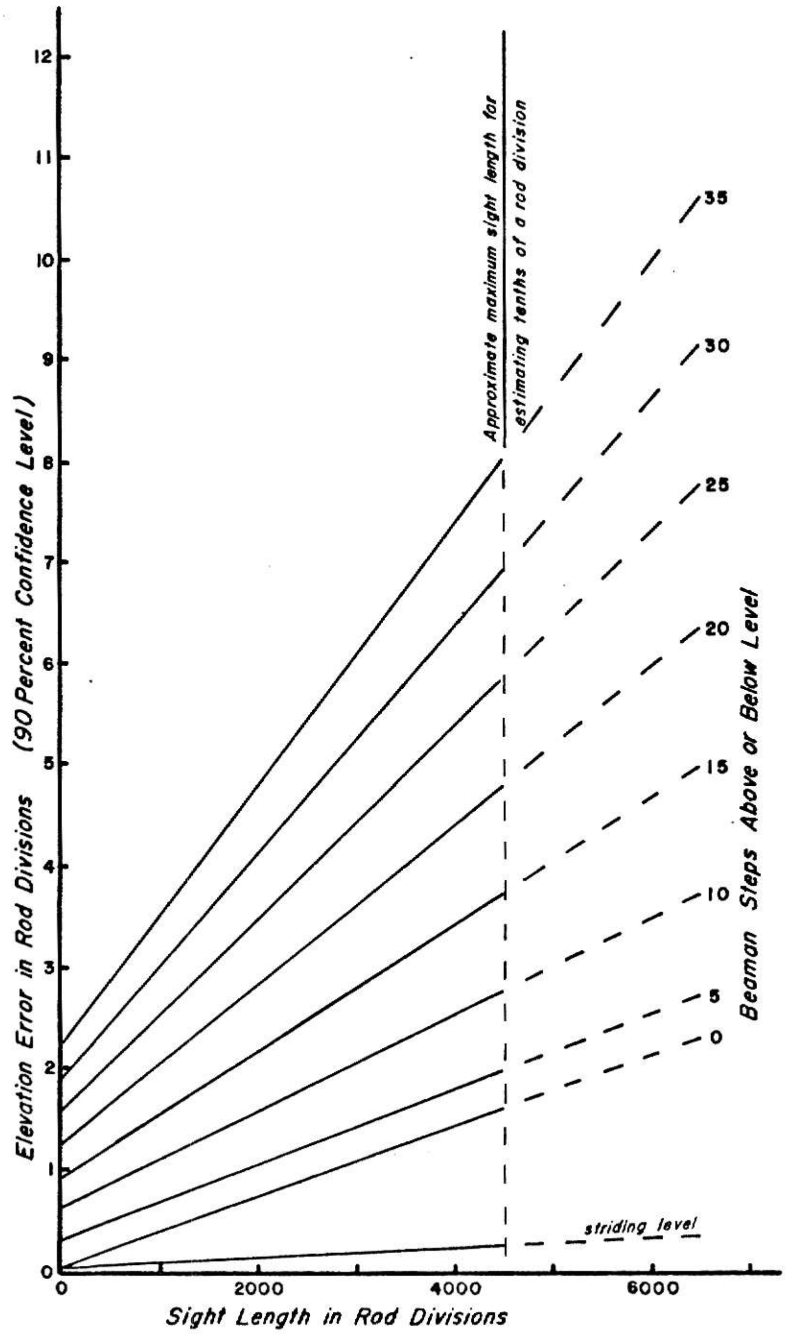

Figure 3 Errors in elevations, Standard alidade (13x telescope) and Beaman Method

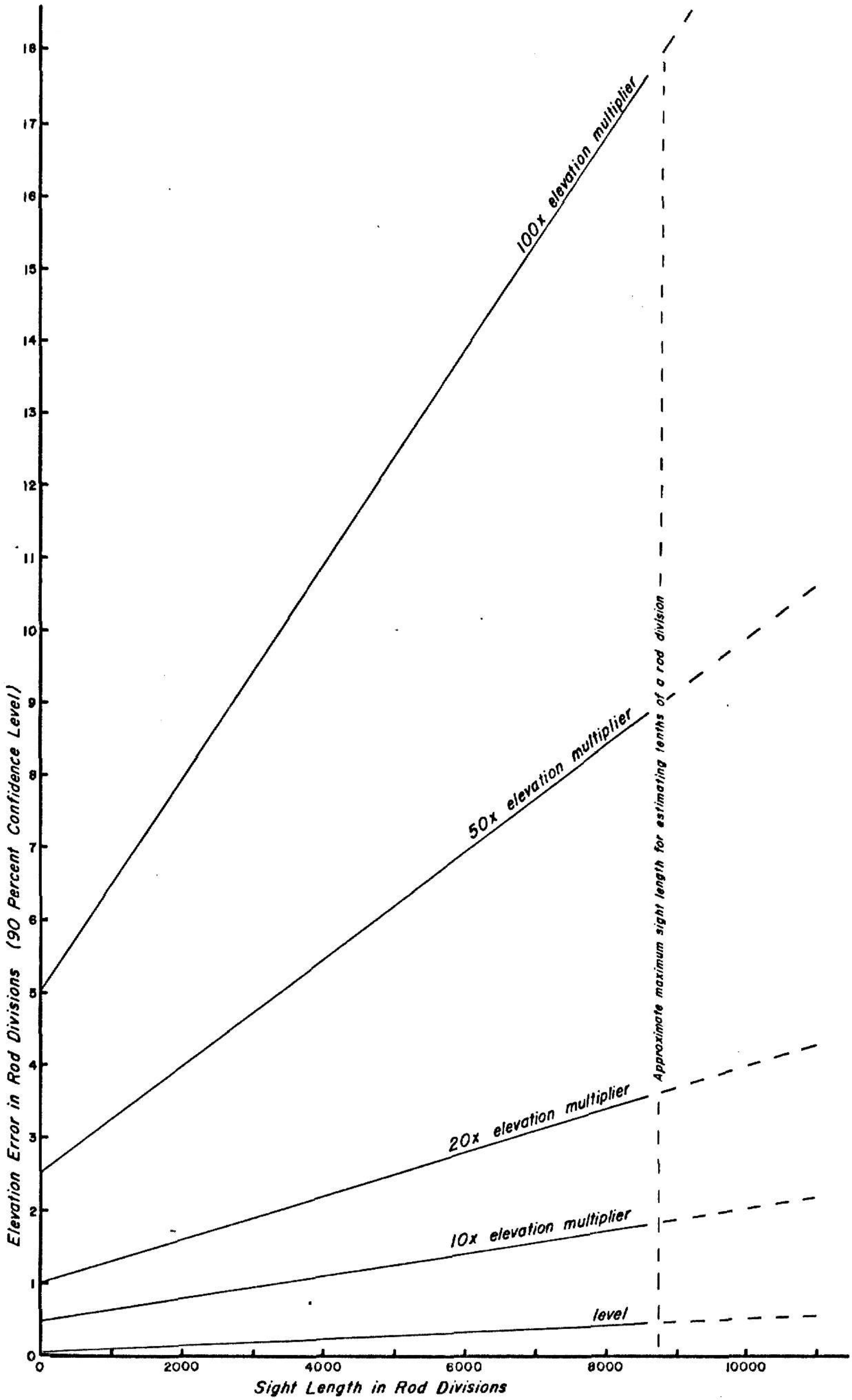

Figure 4 Errors in elevations, Wild self-reducing tacheometers RDS and RK-1

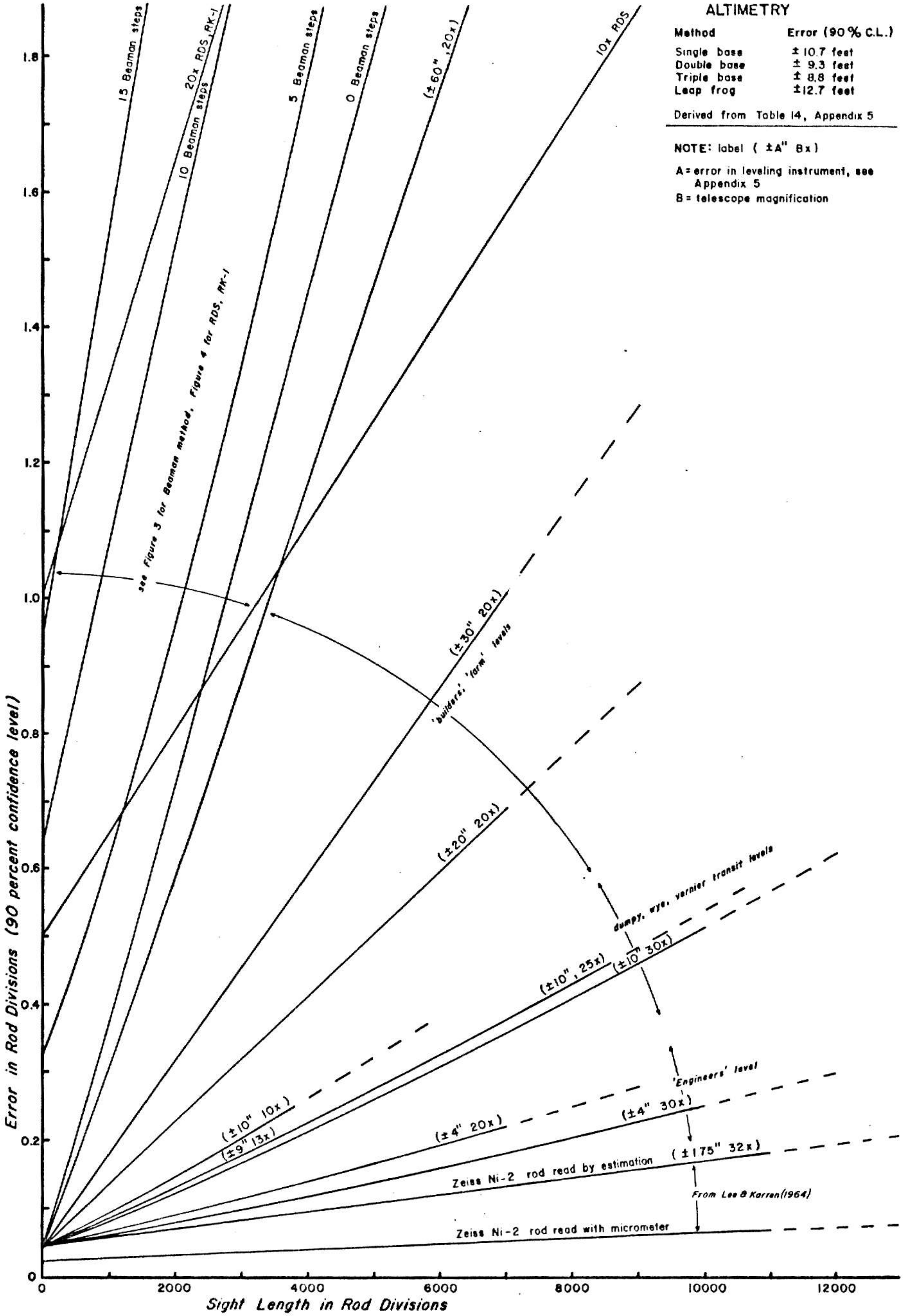

Figure 5 Errors in Vertical Measurements

variables to plot meaningful values on the general chart. Trigonometric measurements based upon a vernier transit and stadia distances fall in the same range as Beaman values.

With these or similar charts the error in each measurement can be estimated and using the Theory of Errors an estimate for the error in any measured point can be derived. For purposes of planning and selection, one selects a convenient average sight-length and divides this into the distance from control to give the number of sight-lengths necessary to arrive at the distant points. Because sight-lengths are equal, the error in each are all equal and the equation from the Theory of Errors simiplifies to:

$$\text{resultant error} = \text{error in one measurement} \times (\text{number of measurements})^{\frac{1}{2}}.$$

From a theoretical viewpoint, one will find that for any error line on the chart and any given traverse distance there exists an optimal sight length (i.e. one that will give a minimum resultant error). This optimal value increases with traverse length. Practically, shorter sight lengths mean more measurements and hence more time and field expense. Therefore, the sight lengths should be kept in the maximum third of the instrument range if scintillation and systematic errors do not interfere.

These charts and principles apply to accidental errors only, predicted results cannot be achieved unless systematic errors in the measurements are corrected or eliminated.

SELECTION FROM AMONG APPROPRIATE PROCEDURES

By using the methods and data in the preceeding sections the required precision can be used to divide the methods into adequate and inadequate groups. The next step is to select a method from the group of adequate methods.