COMPUTER USES IN GEODESY

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The advent of computers revolutionarized most of sciences, especially technical ones. In geodesy and surveying, mostly rather complicated calculation steps with a high number of input data lead to equally numerous output data, as a rule.

Prior to the advent of digital, program-controlled computers, algorithms solving partial problems to lead in several steps to the final result have been aimed at.

Labour consumption was decisive in stating a problem; majority of the research workers preferred approximations though failing to provide reliable data but reducing the calculation volume.

Also fundamental technical problems have arisen, which seemed simply too daring for geodesy to be approached, such as that of planning geodetic operations.

Planning is the most typical constituent of engineering activities. Tracing the development of the planning process in the field of technical sciences, it appears that the former, generally empirical way of planning has actually been replaced by digital planning based on "exact" calibrating and design procedures, permitting to unambiguously determine alternatives that are optimum both technically and economically.

Planning of geodetic problems based on exact design methods is still in its early stage, especially in this country. It was the Soviet geodetic literature that has first been concerned with this problem, in particular with the basic horizontal control-networks. To expand numerically the developed algorithms by mechanical desk-calculators is rather tedious and lengthy. At the actual stage of development of the digital computer technique, especially with the possibilities offered by the digital terrain model, development, instruction and introduction of exact design methods into the geodetic practice is a technical necessity. Practically, instruction of design in connection with the development of the national horizontal control network seems to be the simplest and of the greatest urgency, but also most of the problems of engineering geodesy require to be planned.

By the numerical design of geodetic networks, the pre-determination by design-office methods of the geometrical positioning and measuring methods of geodetic networks is meant. Design input data are maps (digital terrain models), the desired point density, characteristics of the uniform distribution, characteristics of the existing control points, desired approximate co-ordinates of some new points, the desired accuracy indices, economy parameters of measuring technology. Design outputs are approximate co-ordinates of the points to be determined, the approximation degrees, exactness characteristics of the proposed measuring technology, elements of the network to be measured, economic efficiency. At the beginning of computerization, programming of the most laborious partial problems seemed to be the most advisable. This is why the first popular geodetic programs belonged to the scope of network adjustment.

Mathematical model of the problem (solution of linear equation systems) was of general interest also in other fields of technical and natural sciences, hence the use of the available library subroutines accelerated the process.

Increase of computer availability, advent of program languages of relatively simple acquirability for technical specialists eased the way to programming other, relatively simpler geodetic problems. The subsequent development is, however, not quite unambiguous, local impact of subjective and objective conditions has led to development alternatives that by now just begin to show perspective outlines.

A fundamental objective condition likely to determine computerization trends of geodetic mass calculations is the stage of organization of geodetic works. Although technology is strictly related to organization, development of computerization trends has some self-contained effect. Standards of these two objective conditions may much forward or hinder the subjective factors of familiarity.

In the case of several competent institutions, organization stage of geodetic works is characterized by the lack of perspective in division of labour. This phenomenon can be attributed to the low staff number and multiple tasks of these institutions, and, last but not least, to peculiarities of the applied measurement technologies.

Institutions of a low grade of division of labour often engage the same person to measure, to calculate, and often to do the graphical processing. In these conditions of organization, computerization of geodetic calculations is likely to prefer the use of desk (or smaller) computers.

For any known technology, geodetic works have the common feature that measurement and processing are done separately in space, at the same time this separation definitely hinders the measurement process. Namely, some checking calculations are advisably made during the measurement process itself.

It follows from the above that geodetic computerization should be focussed on progressive technologies and organization systems, existent only in their germs, so that the developed computation systems should be *convenient* also for establishments applying technologies at *lower* organization levels. As a matter of fact, a progressively developed mathematical model supplied with the appropriate software and hardware, if widely, economically and comfortably accessible, can be the driving motor of the technological and process-organizational progress of the entire branch.

This has been the postulate for the Institute of Geodesy of which to start out in its relevant research work.

First, the flow-chart of the process has to be designed, under the condition that the blocks are self-contained, at the same time they can be fully or partially connected, provided the conditions prevail.

The practically and theoretically most difficult problem consists in designing the block directly connected to the instrument in question. The first step of a practical approach for the instrument designers was to attempt data recording automation. No doubt, direct recording may eliminate important sources of error from the measurement process, at the same time encourages the subjective demands for a computer processing. Nevertheless, data recording is insufficient in itself to solve checking the measurement results in field conditions so as to provide instrument feedback.

The theoretical solution of the problem is most hindered by the present standard of the technological planning of geodetic operations. Namely, the check-feedback is practically effective only if the checked measurement results belong to relatively short time intervals. Therefore geodetic operations should be planned in technical-technological-organizatory steps so that their direct computer processing permits to eliminate eventual mistakes without a major impact on the work schedule.

The present technical standard offers two possibilities of practical implementation: either a microwave connection between the field measurement unit and the datex network with appropriate local centres, or the use of small, portable universal computer units in the field operation process, a possibility of lower technical niveau.

Thus, the complex process of computerization consists of the following blocks:

a) network design,
b) checking measurement results with feedback possibility,
c) numerical processing of measurement results by means of a program system controlled by a unified organization program,
d) plotting numerically processed measurement results by means of an automatic co-ordinatograph (plotter).

This study is concerned with the first block, i.e. planning of geodetic measurements. One of the examples will illustrate the set-out planning, the other describes the design process of the local horizontal reference network.

The condition of a set-out work that is technically convenient and economical, is its planning, that is, due selection of the set-out procedure to be applied, the instruments and the measurement methods.

The idea of the set-out planning arose about three decades ago, but there was no need and possibility of implementation sooner than by now. The need has been created by the enormously increased demand — both in quantity and in quality — for setting out "industrialized" constructions (e.g. large-slab systems). The possibility is given by the spread of computers, else the volume of computation work could hardly be managed or not at all.

A method of planning a set-out has been developed at the Institute of Geodesy, the mathematical model of which will be presented first, followed by the modalities of computerization.

As the first step in planning, it is advisable to define the *accuracy requirements* of setting-out composed of structural and placing accuracy requirements. Respect of *structural accuracy* requirements safeguards adequate positioning of building parts, structural elements relative to each other. Respect of the requirement of *positioning accuracy* is the condition of the exact relative position of constructions.

Structural accuracy requirements are specified according to tolerances specified for the given construction. Setting out tolerance T_s is calculated from the construction tolerance T_b' by means of the formula

$$T_s = k \cdot T_b \tag{1}$$

where k is a proportionality factor, in general cases $k = 0.4$.

According to Hungarian specifications, construction tolerance T_b depends on the building type and dimensions, as calculated by different kinds of functions. For instance:

$$T_e = q \cdot 23.5 \frac{L + 1790}{L + 42\,000} \tag{2}$$

where L is a structural dimension in mm; and

q a proportionality coefficient as specified in the Building and Installation Codes of Practice.

The subsequent steps of design replace the setting out tolerance T_s by the permissible structural setting out mean error M_{str}:

$$M_{str} = p \cdot T_s \tag{3}$$

where p is a proportionality factor, in the general case $p = 0.25$.

In Hungary, *positioning accuracy requirements* are specified in the Guide for Engineering Geodesy as the permissible deviation from setting out reference marks, as a function of the construction type and of the distance between the points to be set out and the reference marks. For instance, let $E = 5$ mm, thus, for $t = 100$, $E = 55$ mm.

The permissible deviation yields the permissible mean error M_{pl} of set-out positioning by means of the formula

$$M_{pl} = 2p \cdot E \tag{4}$$

where p is the proportionality factor as before.

In setting out constructions, it is possible to determine the permissible structural or set-out positioning tolerance of a point to be set out referred to all other points to be set out and/or to all control points made use of. This would, however, be inconvenient, it is therefore sufficient to calculate relative permissible setting out deviations of characteristic and critical points.

Selection of the points involved in the calculation depends on the construction type and on the situation of the control points made use of. In general, it is advisable to involve in the analysis the relatively farthest and nearest points of the construction, as well as all control points made use of, at least once.

In design, *the setting out method to be applied* should be selected. There are several view-points in selecting the setting-out method (orthogonal, polar, intersection). The most important are: features and environment of the construction to be set out, shape and density of the geodetic reference network, number and position of the points to be set out, instrumentation, personal skill, all being non-numeric factors. These view-points are weighted by the designing engineer before deciding over the selection of the most convenient setting-out method.

In knowledge of the setting-out method, the setting-out dimensions can be calculated.

The next step in design is the *dimensioning*, i.e. determination of permissible maximum mean errors in measurement operations in setting out (longitudinal setting, angular setting, bearing setting, marking) and of the means to keep them.

Permissible maximum mean errors in measurement operations result from the confrontation of the permissible maximum mean setting-out error M and of the *a priori* mean setting-out error m.

Construction safety and smoothness require to have

$$M \geqq m\ . \tag{5}$$

From economic aspects it is inadvisable to set out more accurately than needed, hence another requirement:

$$m \geqq s \cdot M \tag{6}$$

s being a proportionality factor, advisably 0.70 to 0.95, depending on the construction type. Contracting the two inequalities:

$$s \cdot M \leqq m \leqq M\,. \tag{6a}$$

Permissible mean error M may be either a positioning mean error M_{pl}, or a structural permissible mean error M_{str}. Accordingly, the *a priori* mean error m of the points to be set out may be referred either to some control point m_{pl} made use of, or to another point m_s to be set out.

Mean error of the points to be set out referred to the control points is a function of the setting-out method (orthogonal, polar, intersection etc.), of the setting-out dimension (abscissae, ordinatae, angles) and of the mean error involved in the measurement (measuring of angles, point marking). At the Institute of Geodesy, relationships have been established for computing *a priori* mean errors of setting-out methods (orthogonal, polar, determination of points in the measuring line). These relationships are likely to deliver mean errors parallel to, and normal to the straight lines connecting the control points made use of (Fig. 1).

In knowledge of both errors, mean errors of any other sense may be computed.

For instance, mean error m_{11} of points in a measuring line normal to the reference direction AB is:

$$m_{11} = \left[m_a^2 + b^2 \frac{m_i^2}{2} + m_e^2 \left(0.5 + \frac{b^2}{t^2} - \frac{b}{t} \right) \right]^{\frac{1}{2}} \tag{7}$$

where b is the setting-out dimension,
t is the distance between the used control points,
m_a the mean error of permanent point-marking,
m_i the mean error of alignment,
m_e mean error of centering the theodolite and temporary marking of control points.

Several methods have been developed for calculating the relative mean errors m_{str} of points to be set out. In independent measurement, this is most simply achieved according to the law of error propagation. For not independent measurements, it is advisable to involve the mean error of a fictive point relative to the control points. In computing for instance the mutually relative mean direction error in AB points P and R in Fig. 2, setting out dimensions of fictive point F are differences of abscissae and ordinatae.

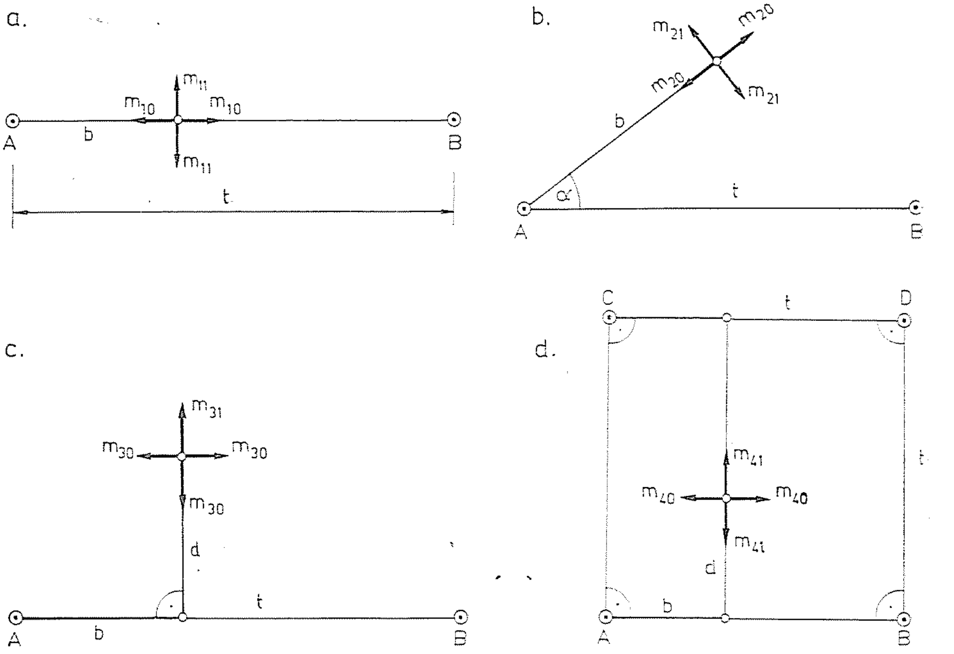

Fig. 1

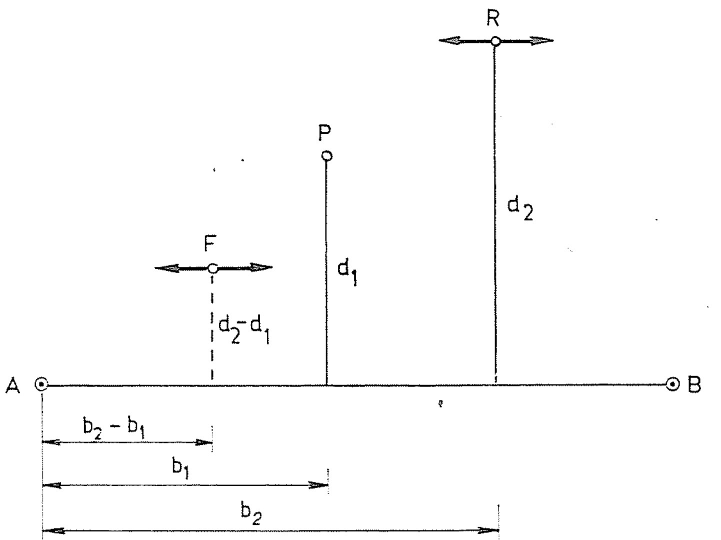

Fig. 2

This relative mean error $m_{str}(PR)$ in direction AB can be computed from the relationship:

$$m^2_{str}(PR) = m^2_{e(F)} + c^2 \tag{8}$$

where $m_{e(F)}$ is the mean error of the fictive point F in direction AB relative to control points;

c is a correction term, mean direction error in AB of a point with setting put dimensions $b = 0$ and $d = 0$ relative to control points AB.

Let us see now the computerization possibilities. According to the planning process developed at the Institute of Geodesy, selection of the setting-out method and of the points involved into the design are actually the responsibility of the designing engineer. Subsequent design steps are computerized. Actually, tests are being made to computerize the selection of points involved in the design.

Design of setting out constructions is composed of parts all of them containing dimensioning of a set-out point related to a control point or to another set-out point. Let us examine a separate part in case of two points, P and R, to be set out. The computation process is seen in the flow chart below:

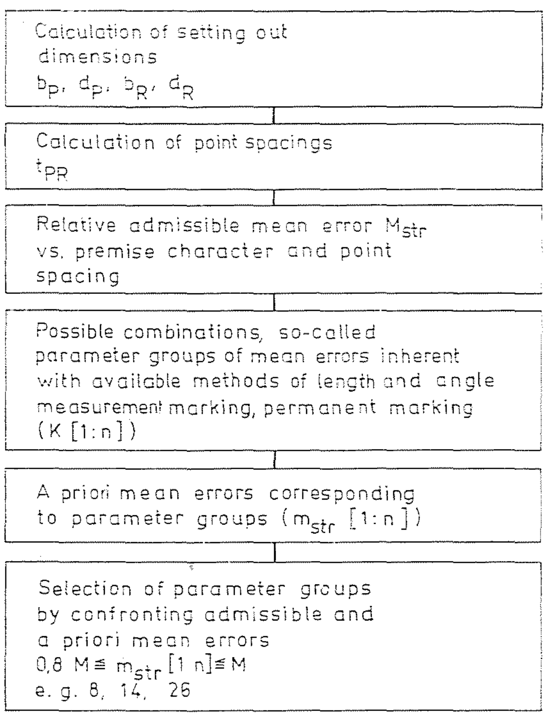

Fig. 3

The computation in the flow chart applied on each occurring point pairs, for each of them, we get different parameter groups consisting of mean errors of measurement operations needed to keep the permissible mean error result. The last step in design is to select among parameter groups obtained for the different point pairs the common ones, and to specify the corresponding combinations of longitudinal measurement, angular measurement, marking and permanent marking. Assume for example that parameter groups 10 and 15 are the final results. Over the given distance, two parameter groups correspond them as follows:

	10	15
Longitudinal setting out	8 mm	5 mm
Bearing setting out	4″	10″
Marking	1 mm	2 mm
Permanent marking	1 mm	2 mm

In conformity with metrologic-technologic studies, to respect mean errors in measurement operations, length measurements should be done by means of a steel tape and utmost care (10) or by an invar tape (15), angular measurement with one-second theodolite precisely centered by means of an optical plummet (10), or an engineering theodolite precisely centered by a plumb line (15), together with extremely careful precision in marking, and permanent point-marking.

The dimensioning may conclude to the impossibility of setting out at the desired accuracy for the given point density, by the selected setting out method, and with the available instruments. In this case either another setting out procedure has to be selected, or an instrument with a lower mean error has to be purchased, or the control points have to be densified by means of the available instruments. Selection of the best possibility among the three ones depends on the actual conditions. In general, however, a practically proven rule is that in case of many points to be set out, densification of control points is the most advisable solution.

Technical development causes geodesy to face ever newer and complexer problems. Most of them cannot be solved by virtue of the character of national networks, therefore independent networks are developed to meet demands.

Computer uses in designing independent geodetic networks, more exactly, in one phase, i.e. in dimensioning will be considered further in this study.

The network to be developed has a specific purpose; in fact, specifications for the arrangement of the points to be defined, and accuracy requirements have to be met. The planning has to develop working plans for a network meeting this double aim in the most economical manner. One of the working plans is the measuring scheme. By network design, development of the optimum measuring scheme is understood.

Layout of the network points is about defined by terrain features and special requirements. Planning precedes dimensioning and takes place in course of design in bureau, field survey and setting out, therefore in dimensioning, the spatial position of the points is generally considered as given, and the most economical distribution of measurements is aimed at. In dimensioning, data in the setting-out records and in the setting-out sketch (measurable directions and co-ordinates) are relied upon. It consists in determining the arrangement of directions and distances to be measured, and the desired accuracy.

Dimensioning means a high demand in computing work, so up to recently, concrete computations of this type were exceptional, although convenient methods have theoretically been developed earlier. Computerized dimensioning becomes efficient and economical. At this Institute, a program has been developed for a computer ODRA-1204 of the Faculty of Civil Engineering, written in ALGOL 1204 computer representation of ALGOL 60 language. Its overall flow chart is shown in Fig. 4. (Basic expressions of the computation are contained in publications on adjustment calculus.)

The program involves dimensioning by progressive approximation (gradual improvement of a basic alternative). First, the starting of basic alternative of the measuring scheme is elaborated on the basis of data in the setting out sketch and report, and checked by means of the program. Fig. 5 represents the simplified input flow chart. The data sheet has to contain the identity number and co-ordinates of the end points of directions i and of distances t designated for measurement for each stand point. Tests have been made to determine the accuracy of how the co-ordinates of the new points should be known to obtain realistic information from indices delivered by the program. In short, if co-ordinates of the new points are known with a mean error of 5 to 10% of the average side length, then the obtained information is affected by not more than 5 to 10% of mean error, convenient for dimensioning. This accuracy is possible by taking measures on a map or on a setting-out sketch.

At the end of the data row, expected network mean errors of direction and distance measuring results have to be included, depending on the accuracy of determinant measurements, as well as on the regular errors affecting magnitudes entering into the adjustment. Accuracy of the determinant measurements depends partly on the instrument type, on the repetitions of the measurement, on the quality of instrument centering and point marking, and partly on subjective factors, and on physical conditions in the survey environment and at the surveying instant. Surveying reliability can be estimated by repetition to give mean error of measurements that can also be assessed from empirical data. Deviation between mean error calculated from repeated measurements

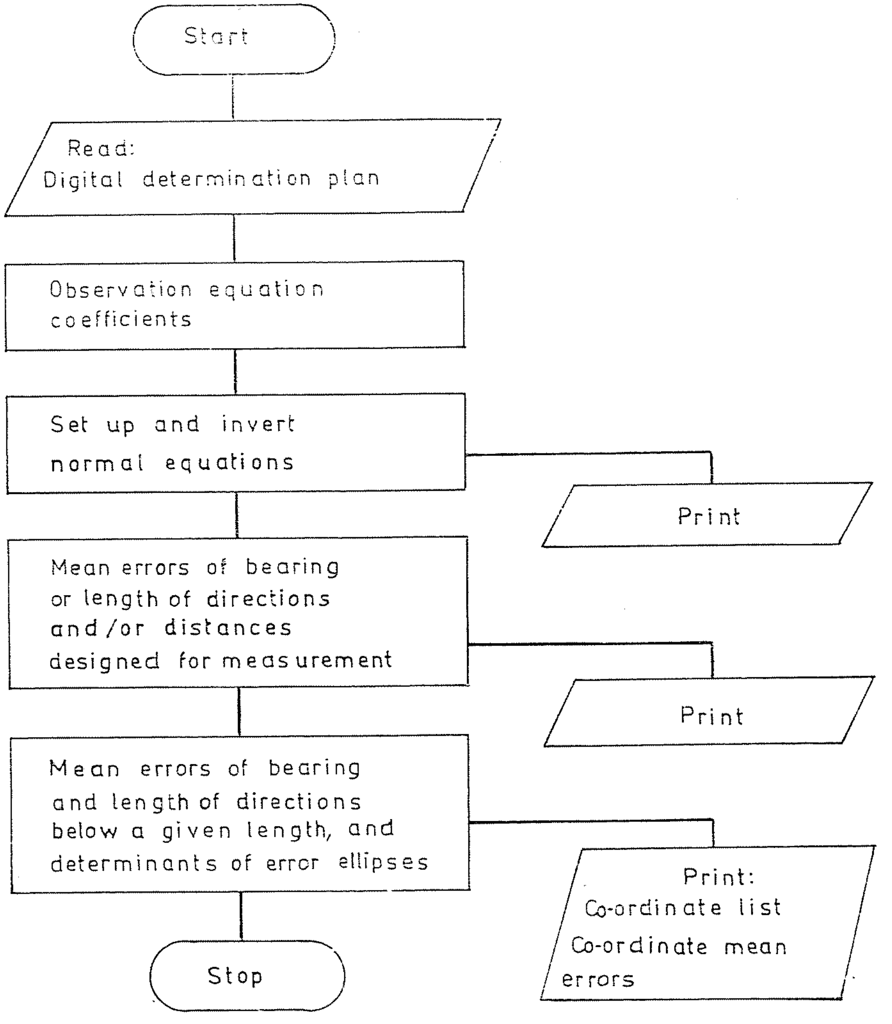

Fig. 4. Overall flow chart of the program

and network mean error of measurement results is often found to be significant, just because of regular errors intervening in adjustment. Deviations in some concrete cases are compiled in Table 1. To achieve due reliability in network dimensioning, a safety factor has to be applied on mean errors computed from repetitions, to obtain expected network mean error. Safety factor in Table 1 seems to vary about 2.0 but may be as high as 3.0. (A data sheet report is shown in Fig. 6.)

Provided that reliability indices delivered by the program meet accuracy specifications, the alternative s acceptable, else it has to be further modified

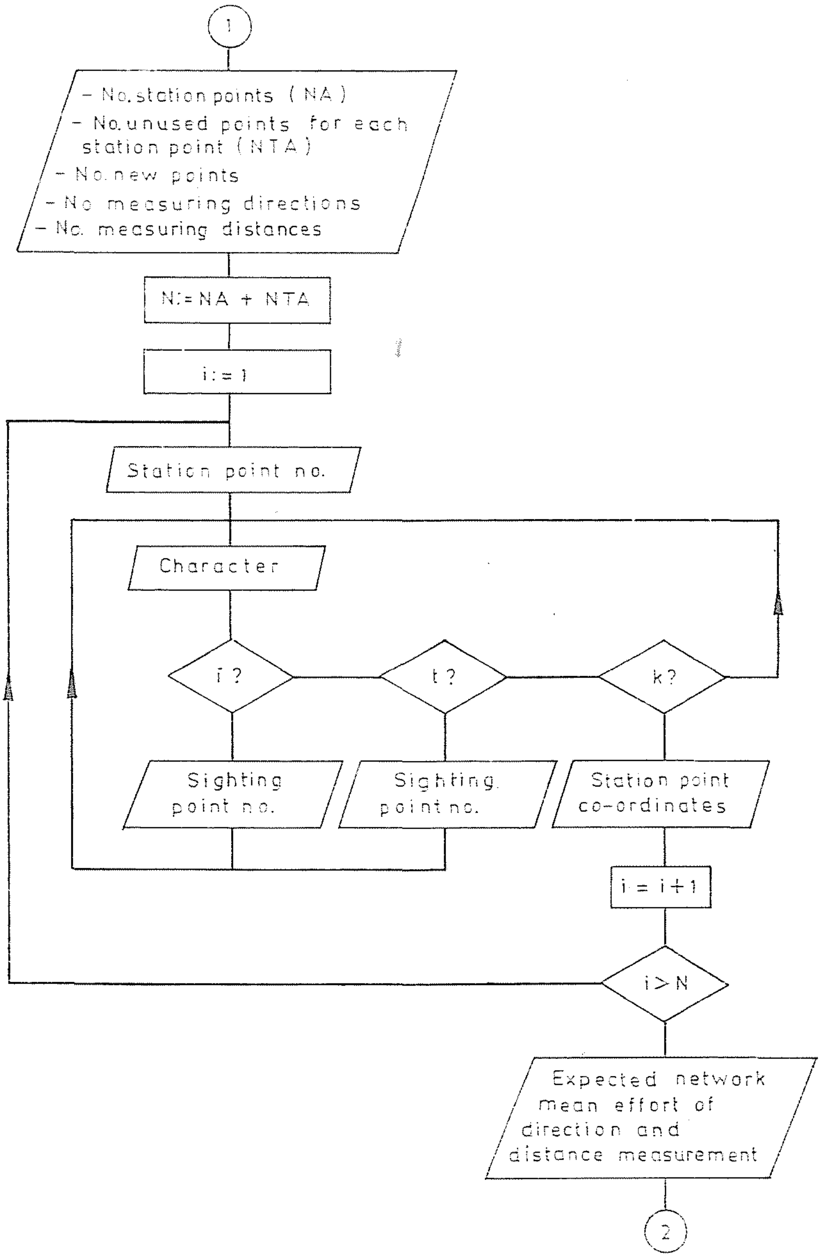

Fig. 5. Simplified input flow chart

Table 1

Net denomination and year of determination	Mean error calculated from repeated measurements, m_M	Net mean error, m	m_M/m_H
Kecskemét (1935)	$\pm 1.11''$	$\pm 0.72''$	0.6
Primary triangulation in Hungary (1927—28)	$\pm 0.18''$	$\pm 0.36''$	2.0
Tatabánya (1947)	$\pm 0.30''$	$\pm 0.88''$	3.0
Budapest, Primary net	$\pm 0.18''$	$\pm 0.31''$	1.7
Base net	$\pm 0.13''$	$\pm 0.28''$	2.2
Primary chain in Hungary (1949—54),			
Western territory	$\pm 0.20''$	$\pm 0.42''$	2.1
Eastern territory	$\pm 0.16''$	$\pm 0.40''$	2.4
Balatonkenese (1965)	$\pm 0.50''$	$\pm 1.12''$	2.2
Roggenstein (1970)	$\pm 3.00^{cc}$	$\pm 4.59^{cc}$	1.5
Csongrád (1973)	$\pm 1.6''$	$\pm 1.8''$	1.1
Pörböly (1973)	$\pm 2.3''$	$\pm 1.4''$	0.6

```
7  0  5  50   10
111111  I  111002
        I  111113
        I  111004
        I  111115
        I  111116
        I  111117
        K  4780   4350
111002  I  111113
        I  111004
        I  111111
        I  111117
        K  5000   5000
111113  I  111004
        I  111115
        I  111111
        I  111002
        K  5330   4310
111004  I  111115
        I  111111
        I  111002
        I  111113
        K  5620   3560
111115  I  111116
        I  111111
        I  111113
        I  111004
        K  5090   3700
111116  I  111117
        I  111111
        I  111115
        K  4380   3950
111117  I  111002
        I  111111
        I  111116
        K  4360   4810
1.5   0.015
```

Fig. 6

Mean errors of bearings of directions and/or of distance designated for measurement		
Station	Sighting point	Mean error
111111	I 111ØØ2	Ø1.2 MP
111111	I 111113	Ø1.3 MP
111111	I 111ØØ4	ØØ.8 MP
111111	I 111115	Ø1.3 MP
111111	I 111116	Ø1.5 MP
111111	I 111117	Ø1.5 MP
111ØØ2	I 111113	ØØ.8 MP
111ØØ2	I 111ØØ4	ØØ.Ø MP
111ØØ2	I 111111	Ø1.2 MP
111ØØ2	I 111117	Ø1.5 MP
111113	I 111ØØ4	ØØ.8 MP
111113	I 111115	Ø1.4 MP
111113	I 111111	Ø1.3 MP
111113	I 111ØØ2	ØØ.8 MP
111ØØ4	I 111115	Ø1.4 MP
111ØØ4	I 111111	ØØ.8 MP
111ØØ4	I 111ØØ2	ØØ.Ø MP
111ØØ4	I 111113	ØØ.8 MP
111115	I 111116	Ø1.6 MP
111115	I 111111	Ø1.3 MP
111115	I 111113	Ø1.4 MP
111115	I 111ØØ4	Ø1.4 MP
111116	I 111117	Ø1.7 MP
111116	I 111111	Ø1.5 MP
111116	I 111115	Ø1.6 MP
111117	I 111ØØ2	Ø1.5 MP
111117	I 111111	Ø1.5 MP
111117	I 111116	Ø1.7 MP

Fig. 7. (continued overleaf)

on the basis of program outputs and setting-out record data. An output list is seen in Fig. 7.

The program running time is 1 minute, and 2 to 3 min for a network containing 6 or 7, and 12 to 15 new points, respectively.

Bearing and length mean errors of network directions less than 3 km

Origin	End point		Mean error	Distance (m)	Relative mean error (m/km)
111111	I	111ØØ2	Ø1.2 MP		
111111	I	111ØØ2	Ø.ØØ5 M	686	Ø.ØØ7
111111	I	111113	Ø1.3 MP		
111111	I	111113	Ø.ØØ4 M	551	Ø.ØØ7
111111	I	111ØØ4	ØØ.8 MP		
111111	I	111ØØ4	Ø.ØØ5 M	1153	Ø.ØØ4
111111	I	111115	Ø1.3 MP		
111111	I	111115	Ø.ØØ5 M	72	Ø.ØØ6
111111	I	111116	Ø1.5 MP		
111111	I	111116	Ø.ØØ5 M	566	Ø.ØØ9
111111	I	111117	Ø1.5 MP		
111111	I	111117	Ø.ØØ6 M	623	Ø.ØØ9

Error ellipse data for point 111111

OMEGA	168 48 Ø4.4
A	+Ø.ØØ5 M
B	+Ø.ØØ4 M

Co-ordinate list Point no.	Y	X	Mean errors	
			MUY	MUX
111ØØ2	+ØØ5 ØØØ.ØØØ M	+ØØ5 ØØØ.ØØØ M	Ø.ØØØ M	Ø.ØØØ M
111ØØ4	+ØØ5 62Ø.ØØØ M	+ØØ3 56Ø.ØØØ M	Ø.ØØØ M	Ø.ØØØ M
111111	+ØØ4 78Ø.ØØØ M	+ØØ4 35Ø.ØØØ M	Ø.ØØ3 M	Ø.ØØ5 M
111113	+ØØ5 33Ø.ØØØ M	+ØØ4 31Ø.ØØØ M	Ø.ØØ4 M	Ø.ØØ5 M
111115	+ØØ5 Ø9Ø.ØØØ M	+ØØ3 7ØØ.ØØØ M	Ø.ØØ5 M	Ø.ØØ5 M
111116	+ØØ4 38Ø.ØØØ M	+ØØ3 95Ø.ØØØ M	Ø.ØØ6 M	Ø.ØØ8 M
111117	+ØØ4 36Ø.ØØØ M	+ØØ4 81Ø.ØØØ M	Ø.ØØ6 M	Ø.ØØ5 M

Fig. 7. Output list. Mean error of a direction measurement: +Ø1.5″. Mean error of a distance measurement: +Ø.Ø15 m

Let us see an example of how to apply the program. The experimental network at Balatonkenese of this Institute had been determined in 1965, according to the measurement line shown in Fig. 8. Between points connected by continuous lines, reciprocal direction determinations had been made in four sets, using a theodolite type Wild T2. Station adjustment resulted in an adjusted bearing mean error of $\pm 0.5''$ as an average. Co-ordinates of points No. 111002 and 111004 have been derived from points of the national network, and considered subsequently as given. In network adjustment the mean error of the measurements (adjusted bearings) amounted to $\pm 1.1''$. In possession of precision DM-instruments, a determination method of better economy was sought for. Since the presented network was up to requirements,

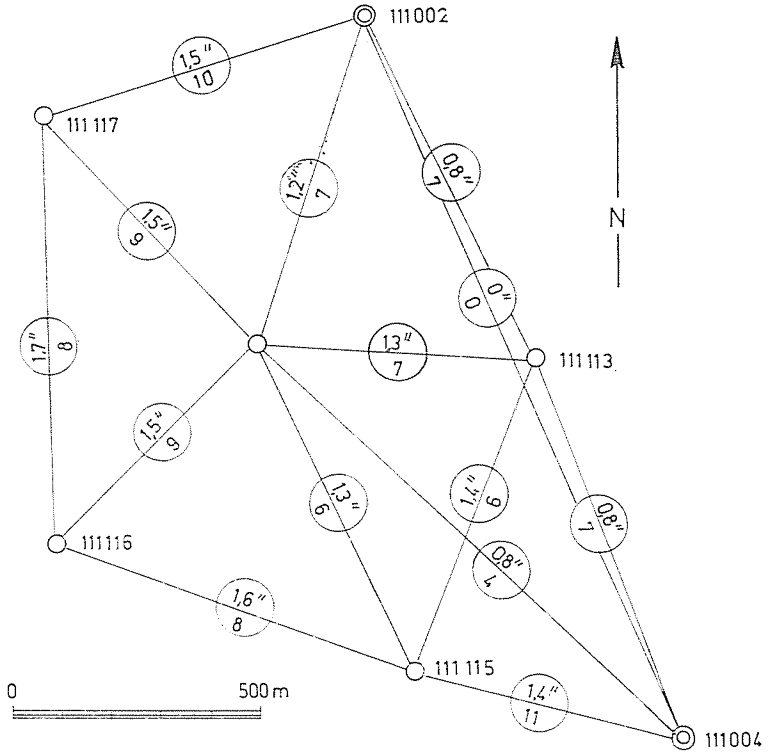

Fig. 8. CTI alternative
Mean error (″) of the bearings of directions designated for measuring and the relative mean error of their length (mm/km)

it was set as a goal to provide a network of similar accuracy. To permit comparisons, in course of the investigations, uniform network mean errors of directions and of distances were chosen to $\pm 1.5''$ ad ± 15 mm, respectively. (Taking a safety factor of 3.0, this is equivalent to required accuracies of directions and distances of $\pm 0.5''$ and ± 5 mm, respectively.) Reduction of the number of stand points means to improve economy, hence as basic alternative, determination of new points from only two points (Nos 111002 and 111004) was chosen, by determining directions for every point of the network, and distance measurements for the new points (alternative SDM—1).

Numerical values of reliability indices of the purely directional alternative (CTI) were taken as 100 per cent in computing average percentage mean errors of directions and distances, and of error ellipse dimensions, as illustrated in Fig. 9. This accuracy seems to fall short behind that of the CTI type, hence in the subsequent alternatives, in addition to the former, direction measurements tending from point No. 111111 to perimeter points have been designated, (SDM—2), or distance measurements have been prescribed for the same end points (SDM—3) finally, directions tending from the point No. 111111 to perimeter points have been designated for direction and distance measurements (SDM—4). It is obvious from Fig. 6 that varieties SDM—2 and SDM—4 provide about the same accuracy as CTI. This is especially true for the error ellipse dimensions. (Dimensions of the error ellipses of the alternative SDM—4 are seen in Table 2 to exceed those of the CTI alternative by a mere 6%.) In case of a network developed for point-motion investigations, the error ellipse dimension is the most important index. In this case the accuracies of SDM—2 and SDM—4 are practically equivalent to that of CTI, while the former two are much less labour consuming, primarily because of fewer instrument stations. Another conclusion to be drawn is that a set of direction determinations executed from the centre greatly increases the accuracy, as against distance measuring sets.

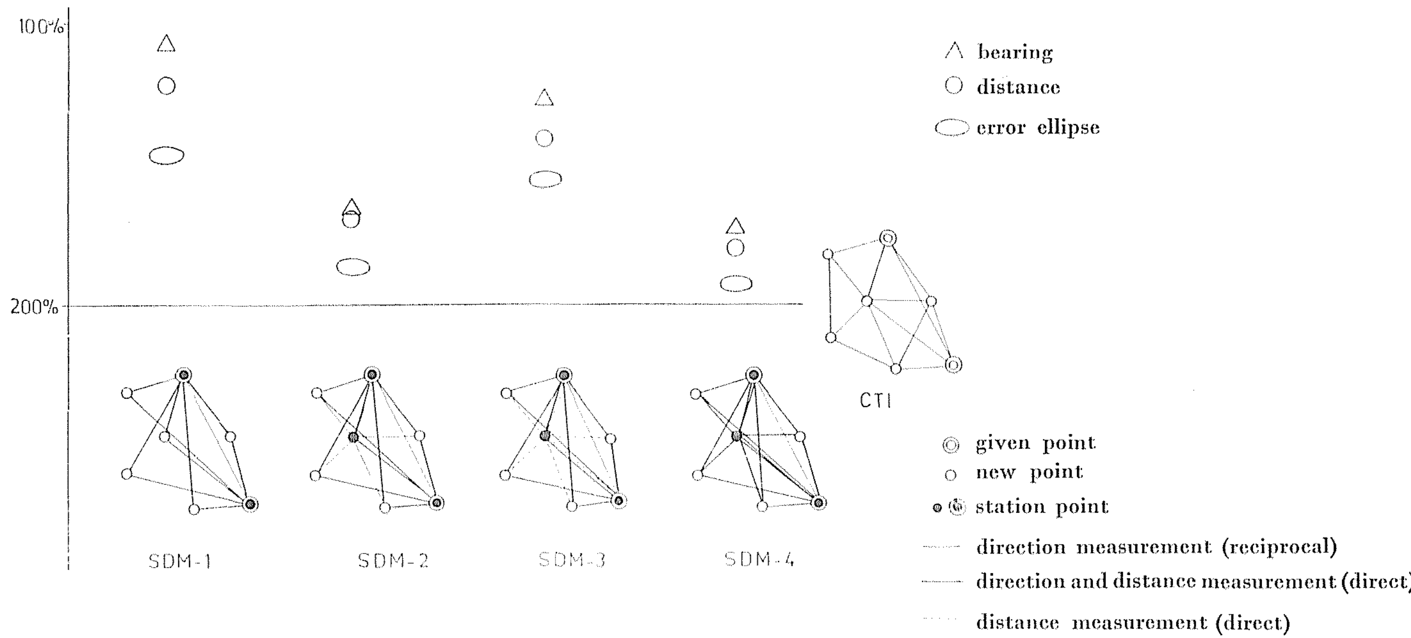

Fig. 9. Averages of accuracy indices of SDM alternatives, percentage of indices of the CTI alternative being taken as 100%.

Table 2

No. of points	Bearing of error ellipse Major radius of error ellipse [mm] Minor radius of error ellipse [mm]				
	CTI	SDM−1	SDM−2	SDM−3	SDM−4
1	169°	014°	002°	025°	003°
	5	8	5	6	5
	4	6	4	6	3
3	154°	157°	139°	158°	138°
	5	9	6	9	6
	4	8	5	7	4
5	127°	104°	114°	102°	112°
	6	9	7	9	6
	3	5	5	5	6
6	172°	063°	046°	094°	044°
	8	9	9	9	8
	6	9	7	8	6
7	055°	075°	085°	072°	084°
	7	10	8	9	7
	5	7	5	7	5

It is obvious from the example that the program lends itself — in addition to dimensioning — also to the investigation of appropriate determination methods.

Summary

Possibilities offered by the modern technique to the computerization of geodetic calculations are presented and illustrated on two examples of planning geodetic problems.

The first example illustrates algorithms and flow charts for planning geodetic settings out.

The second example is that of the dimensioning of horizontal reference networks. Flow chart of the program is presented.

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